FOUNDATIONS OF BUILDINGS IN CLAY
FONDATIONS DE STRUCTURES SUR ARGILE

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Chairman N. A. TSYTOVICH (U.S.S.R.)

Before opening the Second Main Session, I wish with your permission, to make the following remarks.

Engineers have been faced with the problem of rational foundation construction in clayey soils for many years. This problem presents a number of difficulties that are only being gradually overcome by applying the latest data of soil mechanics.

The method of designing foundations according to limiting deformations (settlements) is to be considered the most advanced procedure at the present time. This method calls for the development of sufficiently precise techniques of predicting both the magnitude of the settlements and their rate in time.

On the basis of research conducted in recent years, it is becoming evident that the theory of filtration consolidation of clay, in its true form, is applicable for the prediction of foundation settlement only for a restricted number of eases of fully saturated weak clayey soils that do not contain gases and have practically no structural strength.

In all other cases (for instance precompacted clays), it is necessary in predicting settlement to take into account, not only primary filtration consolidation, but the secondary effects as well. Natural compaction should be estimated, not according to preconsolidation pressures, which cannot be accurately established, but by the magnitude of the primary pore pressure existing in the clayey soil at the beginning of construction and measured in the field.

Not only their natural-historical state is of importance, however, in estimating the compaction of clayey soils, but also the skeleton creep of the soil and the quantitative relationships between the external pressure and the structural compressive strength, between the acting head gradient and the initial gradient and also between the initial pore pressure and the maximal pore pressure developed upon loading the soil.

A knowledge of these relationships makes it possible to quantitatively predict, not only the settlement in weak saturated clays in time, but also the settlement in precompacted and overcompacted clays in time.

The investigation of a single pile and of groups of piles foundations is another instance in which soil creep is of essential importance. The only basis for estimating pile bearing capacity and settlement is a prediction of the variation in the effective stresses in time in the clayey soils surrounding the piles. The structural strength and relaxation of the total stresses should also be taken into consideration.

A correct solution of the problems put before our session will be of vital significance for foundation building practice. I would like those taking part in the discussion to make concrete proposals on this matter.

Now I wish to invite Dr. de Mello, the General Reporter of our session, to give a brief summary of his State-of-the-Art Report on Foundations of Buildings in Clay.

General Reporter V. F. B. DE MELLO (Brazil)

Prof. De Mello's State-of-the-Art-Report appears on pp. 49 of the State-of-the-Art Volume.

Chairman N. A. TSYTOVICH

Thank you very much Dr. De Mello for your interesting summary of the State-of-the-Art paper concerning foundations on clay. Ladies and gentlemen, please let us hear all questions to the General Reporter in the written form. I wish to invite the delegates to take part in the panel discussion. The members of our panel are: Dr. Golder from Canada, Dr. Kézdi from Hungary, Dr. Mohan from India, Mr. Pérez Guerra from Venezuela and Dr. Rosenblueth from Méxioo. I wish to call upon Dr. Kézdi.

Panelist A. KEZDI (Hungary)

In order to determine the bearing capacity of clays, we usually make our first estimate on the basis of the failure theories. For this purpose, we may use more or less sophisticated formulae starting from Terzaghi's theory through that of Balla, Meyerhof, De Beer and others. There is one common thing in these theories: they assume sliding surfaces and total mobilization of the shear strength on those. Even in laboratory model tests, we can hardly experience this type of failure and, in reality, it almost never occurs. A constant and untolerable rate of settlement can well be regarded as another type of failure, where no failure surfaces occur, but the ultimate bearing capacity was reached at. The conditions of this type of failure have not yet been fixed, and I think that several others can be added to this. A further step in determining the ultimate bearing capacity can only be made if we study, classify and mathematically describe the types of failures in clay, as it was brilliantly made for sands by Dr. Vesić.

Using the above-mentioned theories which assume sliding surfaces, we always have some difficulty with the value of the safety factor to be used. For this purpose, I suggested the diagram which was first introduced for the analysis of slope stability. In this case, we determine, for a given footing, and for a given load and curve, pairs of values of (c, ϕ), for which the safety factor is equal to unity. The calculations are easy: we select different values, determine the bearing capacity factors and we have a single equation for c. Now, we plot, on the same diagram, the (c, ϕ) points obtained by appropriate tests; we have a clear picture on the scattering of the strength parameters and also on the limiting values of the safety factor (ν). This can be obtained by the usual way, by drawing the line $\overline{OP}$ and calculating $\nu = \overline{OP}/\overline{OA}$ (Fig. 1). If we wish to analyse the deviation of the values obtained by different theories, we may draw several lines corresponding to the appropriate formulae.

A minor point in the problem of the bearing capacity of shallow footings is the case of eccentric and inclined loadings. We have again several formulae, for instance, the general formula of Brinch Hansen; however, the experimental proofs are - at least in clay - still missing. Here I would like to point out, in connection with the eccentric loads, that the safety factor, for a given value of ϕ, is the same for the centric and eccentric loading, if the eccentricity is due to the partial removal of the load (Fig. 2). This is the case, for example, of a silo, where only the one half of the cells is filled.

Passing to the problem of consolidation, I would like to show you the results of a theory, which does not make use of the poor pressure concept put forward by Terzaghi. Investigations on the structure of clays made clear that many traits of the behaviour of cohesive soils could be explained by the arrangement of the flat or needle-like particles. Compression and shearing stresses cause an increase in the local order of the particles - these changes, however, require time. Figure 3 shows the usual assumption on the variations of the local order. Making some fundamental assumptions regarding the degree of the local order ($x = X - c\nu$) with x as an equilibrium value and ν the volume change (void ratio), it is possible to derive a differential equation; its solution is given on Fig. 4. The total settlement (s) consists of two parts, the first part is the initial settlement and the second the consolidation settlement. There are three constants: K_1, K_2 and t_1, and it can be seen that the components of the settlements

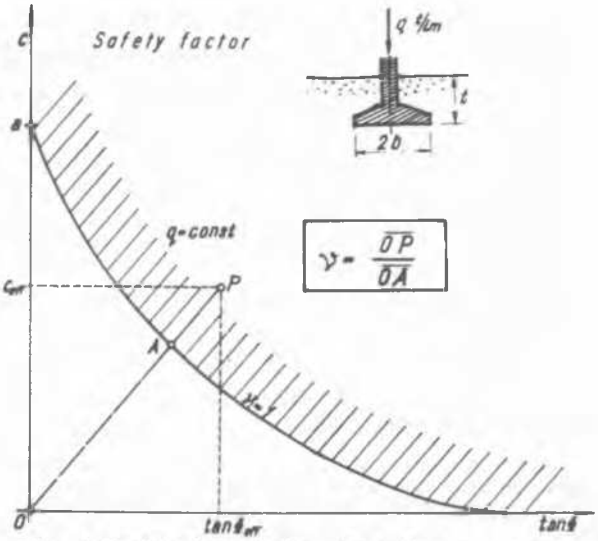

Fig. 1 Scattering of the strength parameters c and ϕ

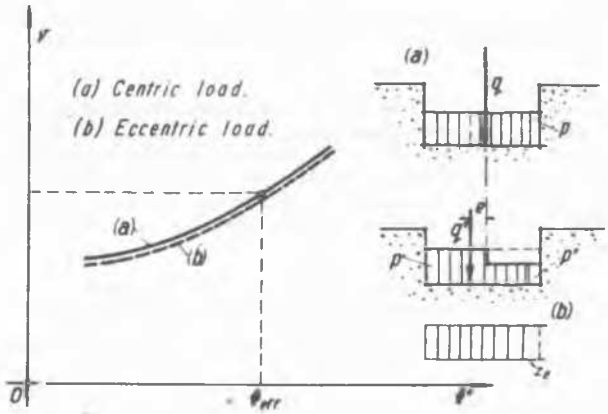

Fig. 2 Safety factor for centric and eccentric loading.

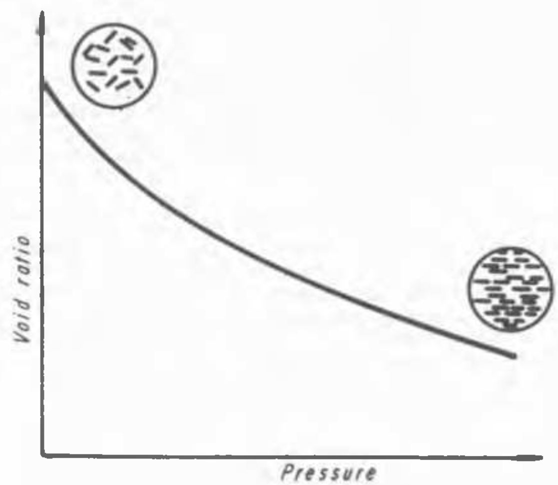

Fig. 3 Arrangement of clay particles upon loading.

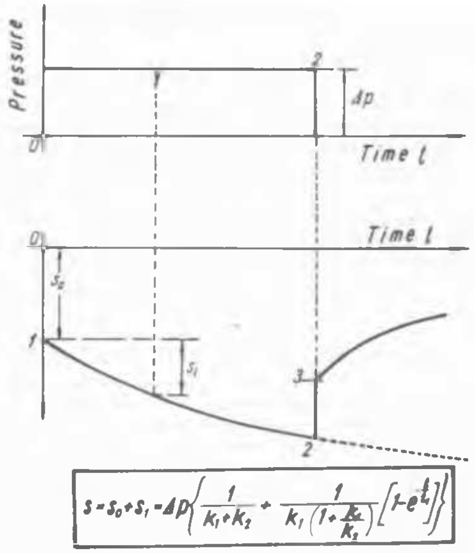

Fig. 4 Differential equation for total settlement.

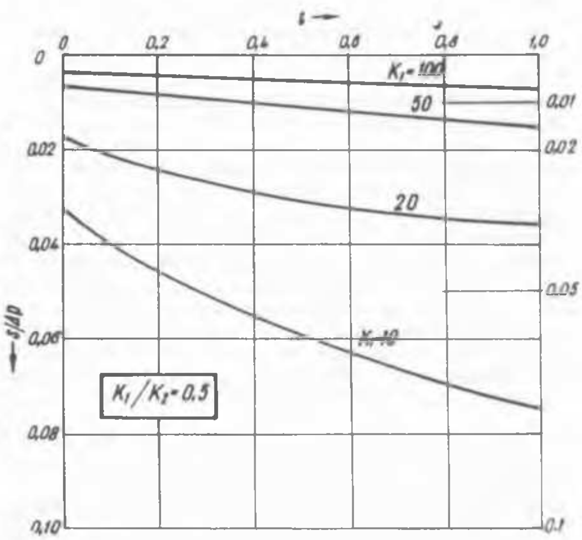

Fig. 5 Consolidation curves for different values of the ratio K_1/K_2.

are not independent from each other (K_1 and K_2 are to be found in both terms). Figure 5 shows some consolidation curves for different values of K_1/K_2.

It may be mentioned that this method of calculation has been applied to the problem of static soil compaction; for this purpose, a generalization of the loading scheme was needed: the passing of a roller increases first the pressures at a cross section and after it, the pressures decrease. The sudden change in the value of the constants clearly indicated the critical phase composition, i.e. moisture contents, above which the static compaction is impossible.

Chairman N. A. TSYTOVICH

Thank you Dr. Kédi for your interesting lecture specially on problems of the bearing capacity of plates. Now I invite Mr. Péres Guerra, the president of Ingeniería de Suelos in Caracas, Venezuela.

Panelist G. PEREZ GUERRA (Venezuela)

The discussion by this pannelist of the several subjects requested by the General Reporter, will be made from the standpoint of the common foundation engineer, who is always looking for practical and simple ways to discharge his everyday duties.

SUBSOIL INVESTIGATIONS

While there is a variety of equipment and tools for subsoil exploration, the wash boring seems to be still the most current procedure. Hollow-shaft mechanical augers, with or without drilling mud, have been developed. They eliminate the need for a casing and permit sampling through the shaft, but they have limitations in regard to depth of exploration and obstacles encountered. On this account, they work beautifully some times, but in other cases they may not work at all. Wash boring looks primitive and old, but it is flexible to cope with varlying conditions and is of moderate cost.

How the hole is advanced, however, is of secondary importance, provided that the method permits a reasonable control of the depth of exploration and that it gives the operator some indications on changes of strata.

Clays are routinely sampled for identification purposes with the 2" split-spoon, and here again we have an old tool which proves satisfactory in the majority of cases.

Classification alone usually gives a good idea about the expected behavior of the clay as a foundation, but the final design decision would require a numerical value of the strength. Three common tests are currently employed to measure it: the field vane, the unconfined compression (UC) and the triaxial compression (UU or CU). The field vane is fast and economical; the other two tests require undisturbed sampling.

SAMPLERS

For soft to medium-stiff clays the open end, thin tube, or the stationary piston samplers are employed. Stiff to hard clays will require the double-tube (Denison type) sampler. The Sweedish foil sampler is available in some countries for sampling soft to stiff clays.

Most of the samplers mentioned recently in the literature employ whether the principle of the Denison double tube or use foils to reduce friction. Plastic and fabric foils have been proposed but have raised objections on account of stretching. (Kallstenius, 1961). One new type of sampler is a patented device which uses special mud to reduce friction and a plug to close the lower end till the sampling depth is reached, and which is then released and floats back to the surface carried by the mud. (Begeman, 1961).

The taking of really undisturbed samples (the "perfect" sample) is a goal that will presumably never be reached; even conceding the absence of mechanical damage to the sample, there is still the disturbance by the stress release caused by the extraction of the sample. The current approach to counter this disturbance is by laboratory methods employing refined techniques and by more realistic stress analysis. These circumstances have fostered the trend to use in-situ tests whenever possible.

Sligthly disturbed tube samples generally give acceptable results for practical purposes. The reduction of strength due to the disturbance has been considered beneficial to safety.(215)(*). This effect,however,can in extremes invalidate the results, even when using 6" samples.(Hall,1964). The least disturbance with samplers is obtained with the foil sampler, as attested by observations and results. In Table I of the State of the Art Report it is noticed that the best comparisons are in cases where the foil sampler was used.

FIELD VANE TEST

The vane test has been the subject of much discussion and of innumerable comparisons with other current tests. The analytical evaluation of this test has been considered complex(Gibbs et al.,1960) and its practical results have been attributed to a fortuitous cancellation of errors.(Ladd,1967).

The conflicting opinions about the vane test make its appraisal rather baffling for the practicing engineer. Theoretically, the mechanism of rupture would seem to be not very well known, and the measured strength not applicable to the system of stresses imposed by usual stability problems; the latter objection, however,would seem to be of less importance for shallow footings. In practice, on the other hand, the test is widely used in routine investigations generally with good results, and is considered by many to represent the true strength of clay.

It is recognized that,apart from theoretical considerations, the results of the vane can be affected by many factors, among them layered soils(316),roots(Leussink and Wenz, 1967), anisotropy(Di Biagio and Aas,1967), fissures(324) and pebbles(87). On the other hand,is specially succesful in sensitive clays, difficult to sample(Brinch-Hansen, 1950), or clays so soft as to make handling impractical, as was the case in testing soft peats in the delta of the Orinoco River

The vane is usually employed in soft to stiff clays with strength up to 30 tons per sq,m. but there is reported one extremelly strong 3 x 6 cm. vane which can shear soils of up to 60 tons/sq.m.(32).

The vane has been correlated mainly to the UC and the UU triaxial tests. With so many variable factors affecting the correlation terms, it is not surprising the wide variation of results reflected by Fig. 6 of the State of the Art Report. Side by side with rather good coincidence(Andresen and Bjerrum,1958)(Flaate, 1965)(279), there are not so good comparisons(Eden 1965) and some which are completely off(32)(Leussink and Wenz,1967). The statistical treatment of the data gives a better idea of relative values and in general improves the results of the comparison.(169)(Flaate,1965).In one interesting case an apparent difference between vane and UC was found to be due to end friction effect in the UC,which after being corrected proved to be equal to 0.93 vane.(116).

The number of cases where the comparison has been satisfactory seem to be much more frequent than otherwise, with practical differences not larger than 20%, and usually less.

The comparison of the vane test with other field or laboratory tests(e.g. the Sweedish cone), and among UC and UU tests themselves has shown variations of comparable magnitude. With good quality samples the results are usually satisfactory from the practical point of view. In one case the F.of S. of a slide was 1.08 with vane strength and 0.98 with the cone.(Broms and Bennermark,1967).In another UC=0.96 cone.(Flaate,1965).

In spite of all the differences and variations, the tests considered can furnish good working design values (Lo and Stermac, 1964)(Ertel, 1967), in accordance with the general consensus of experience, provided that sound basic judgement is used to take into account the circumstancial factors that may affect the results. Sampling, classification and some other correlative strength test area required for each clay deposit to establish the necessary confidence (87).

In general it is to be expected that the results of the vane test come out higher than the UC and somewhat lower than the UU(Gibbs et al.,1960). As a consequence some designers reduce sistematically the

(*) Figures in brackets are reference numbers of the Bibliography of the State of the Art Report.

results of the vane in a fixed percentage, to supposedly obtain strengths on the UC level.(Broms,1966). If it is considered that usually the vane strength employed is an average of several tests, and taking into account the natural variability of soils, the relative variations of the tests themselves and the uncertainties of the failure load, this sistematic reduction would not seem justified.The final answer should be, as mentioned by the General Reporter,in the establishing of statistical confidence levels for each given locality.

Stiff and hard clays are almost always fissured and this defect weakens the clay and produce extremely erratic variations in all types of strength tests.(Skempton and Petley,1967). It is known that the size of the sample is determinant on the measured strength, larger samples showing less -- strength.(Bishop and Little,1967). In fissured London Clay the true strength can be as low as 50% of the results of conventional tests in small samples. This fact alone has much to do with erratic results in the comparison of the common tests(324).

When dealing with fissured clays, which is very frequent,the lower values of vane,UC and UU tests should govern the design, in accordance with general experience.

Correlation of UC with the small laboratory vane called "Torvane" is most satisfactory for routine work. The same applies to the readings of the pocket penetrometer.

LOAD TESTS

With few exceptions(Di Biagio and Aas,1967) the term of comparison has been the strength determined in triaxial tests on block samples taken by hand, or in field load tests One interesting variety of the latter is the piston load test performed at a constant rate of penetration, the so-called CPR piston test(Butler,1964). These tests, specially with pistons of 8" or more in diameter, have shown strengths appreciably lower than UU tests(Hooper and Butler,1966) and with much less dispersion. In fissured clays it has been a common finding that piston or plate tests give strengths which plot near the lower limit of the envelope of conventional UU tests(331). In cases where the UC has little dispersion, it has agreed closely with plate load tests(Ertel, 1967).

PENETRATION TESTS

Penetration tests are widely used as a way to determine the bearing capacity of soils, primarily sands. The two most common types are the static(Dutch) cone penetrometer and the split spoon dynamic penetration test. Some use is also made of several other penetrometers, among them the dynamic 2-1/2" cone, after Peck(1953).

STATIC PENETROMETERS

In regard to the static penetrometer, there have been reported several experiences and evaluations of its possibilities in clays. The point resistance of the cone is considered as a model load test of a deep circular footing. The bearing capacity factor N_c figured back from the test has shown a tendency to diminish with increase of -- strength: from 30 for soft clays to 20 for a clay of c=10 tons/sq.m.(Wesley,1967), to 10 for a clay of average c=30 tons/sq.m. (de Beer,1967). N_c = 20 has been recommended as an average(258).In London clay an average value of 18 has been reported (311), with tendency to decrease for the higher penetration resistances. In another test in London an average value of 15.6 was found(324). In soft clays the overburden load may have some influence on the correlation, mainly for medium to large depths. Some variations have been found to accompany differences in techniques and rates of penetration.

The decrease of N_c for increased resistance could be caused by the typical reduction in cross section above the cone point, which would interfere with the general shear type of failure present in stiff clays. This reduction would no affect the readings in soft clay, in which a local shear failure would obtain(311).

Penetration resistances tend to decrease with increase in cross section of the cone (311), but no significant difference was found with various types of penetration points, from the classical Dutch point to a flat circular one(324).

As the evaluation of N_c from the cone penetrometer test depends on the determination of the true strength of the clay,N_c values show variations from this source as well as from dispersion of the test itself. Not much success has been had in relating the cone penetrometer to the vane and UC tests. In one case(32) the N_c was 15.5 when related to the vane and 30 when related to the CU triaxial. This situation seems to be a reflection of the differences experienced in the other types of tests, as commented above.

In addition to the point resistance, the static penetrometer permits the measurement of the adhesion resistance on a sliding sleeve, data which is usually shown as a curve of accumulated adhesion. In the opinion of the writer this measurement would seem to underestimate the true adhesion value of the clay. For this reason it would seem better to use for design an empirical reduction of the shear strength as measured by the point resistance. The adhesion readings of the cone have been applied to purposes other than measurement of strength(13). Generally speaking, both phases of the penetration test(point resistance and sleeve adhesion) are to be considered as field strength tests, which require for a rational interpretation, boring and sampling for identification of the soils involved.

In summarizing, the interpretation of penetration tests in clay looks difficult and confuse at the present time and more investigation is needed to evaluate the several factors that may influence it. An N_c value of 10 would be expected from theoretical considerations (de Beer,1967), while the 18 to 20 values recommended by others could be practical conservative values. As with other field tests, it could well be that correlation always remain to be only possible on a local basis.

There are also a few references to the use of the static penetrometer for the determination of compressibility parameters(258) (Bachellier and Parez,1965). The latter, using the same penetrometer employed for the IRABA tests in France, found an empirical correlation factor which for the clays tested varied within relatively narrow limits. More extensive investigations would undoubtedly be required for the method to come into practical use.

As it has been more or less established that shape of the cone is not a major factor of variation, as against area of cross section and rate of penetration, it would seem recommendable to relate future experiences to the Dutch penetrometer and technique. This penetrometer has a tradition in Europe and is the most frequently found the world over.

DYNAMIC PENETROMETERS

The performing of the spoon penetration test while doing wash borings and disturbed sampling has been considered as a free bonus of the proccess. Relative density of sand has been related to the SPT, but fortunately this subject is out of order in this session.

Attempts have equally been made to relate the blows of the SPT to the shear strength of clays. Reported in the literature and summarized in Fig. 27 of the State of the Art Report, are correlations of N/q_u (in kgs./sq.cm.) ranging from 5 to 50, with the most frequent values from 5 to 8. Some of the correlations are backed by extensive testing and apply to a particular place (225)(182). As noted by the General Reporter, the scatter of the correlation probably is due more to the variations of the clay than to the test itself. Activity of the clay seems to influence the results (182)(510),as well as fissures and pre-consolidation.

In testing two extensive deposits of preconsolidated clays in Venezuela, it was noticed that in spite of scatter, the ranges of blow counts corresponded to similar ranges of strength, and that the blow counts were rather consistent throughout each deposit.

In general, it seems that unless a systematic local correlation is developed, the results of the SPT in clays are to be taken as only indicative of strength, as stated by many authors(89)(Means,1960)(309).

A few investigators have sougth a relation between SPT and compressibility(260)(Malcev 1960)(112), but the results do not seem as yet to be useful for practical work.Probably the situation in this field is similar to the one related to strength.

PRESSUREMETER

In the line of field tests both for compressibility and for strength, it is believed that the most promising is the pressuremeter test developed by Menard. There are increasing publications of experience with this apparatus, reporting results rather satisfactory from a practical point of view.

As an in-situ, stress-strain test, the pressuremeter yield data related to the whole phenomenon of deformation under load, which permit the establishing of working values for pre-consolidation,bearing capacity and compressibility.

In regard to strength, comparisons have been made with vane and UU tests(Rochette and Hurtubise,1964), and with plate and UU tests(Greenland,1964).The pressuremeter agreed with the vane; the UC and UU tests had a wide variation, with an average below the vane strength but exibiting a similar trend in depth. The pressuremeter showed a reasonable agreement with the load tests, but with much more variation and with a tendency to indicate higher strength. The type of soils tested may have had much to do with both the scatter and the higher strength, as they were fissured clay, fissured weak siltstone and varved clay.

For pre-consolidation loads, the pressuremeter seems to furnish good working values in accordance with tests conducted in soil of CL, CH and MH type(Mori and Tajima,1964). This is an aspect of great interest, as a majority of clays to be loaded with building foundations are pre-consolidated to some extent. At present, short of actual oedometer or triaxial tests, the designer , looking for a preliminary value of pre-consolidation, has to resort to correlations with the liquidity index.

The evaluation of bearing capacity and settlements with the pressuremeter, has been presented by its inventor(Menard et al.,62) making use of a long list of experimental parameters;a series of full-sized load tests gave results which seem to substantiate the theory(Menard,1963).

The measurement of soil properties and prediction of behavior on basis of pressuremeter tests are probably as relative and as subject to variations as most other procedures of soil mechanics; yet, they are necessarily far more accurate than many correlations currently in use. The ability to run several tests in a site in a relatively short time and to furnish data on several levels of the profile, may make of the pressuremeter a useful tool of soil engineering. The flow pressure and the limit pressure defined by the pressuremeter represent physical limits of behavior, and the test is based on recognized rational theories.

This is not to imply that the pressuremeter is to be considered a panacea-there may be situations where its use may prove impractical or its interpretation unclonlusive- not that it would furnish the sole basis for establishing foundation criteria

Everything considered, it is a bit surprising that the pressuremeter, at more than ten years of its invention, has not come into more general use. Efforts should be made toward increasing its availability and simplifying its interpretation.

(Ø,c) SOILS

The vane and UC tests are strictly applicable only to Ø = 0 cases. The interpretation with (Ø,c) soils is very personal with the designer, as it will require a certain weighing of cohesive and frictional characteristics. A measure of friction can be had with the vane test by using vanes of different numbers of blades(Farrent,1960), but this is not a current procedure.

Static penetrometers, plate load tests and pressuremeter tests, can give a measure of bearing capacity, but the complete analysis of the components of strength will require triaxial compression tests.

ON CONSISTENCY LIMITS

Taking up an ad-libitum subject, the writer would like to bring up a point that may be of some interest.

It would be an inanity to mention the extensive use made of the consistency limits in soil mechanics. The determination of these limits in routine work means hundred of tests made by laboratory technicians. Frequently they do nothing else for weeks, it being certainly a depressing perspective to face the plate-clapping and finger-rolling routine day in and day out. It is known that personal errors may affect the results, and while a laboratory can always make periodic checks of the reproducibility of results, it is difficult to take into account the deterioration of technique as the day advances and the hands get tired and the minds dulled.

Several years ago there appeared in Russia and in India some proposals for the use of a cone as a substitute for the liquid limit machine; some experiences made in the United States(Sowers et al.,1959) showed considerable promise, but lately no mention has been made of the subject in the literature, giving the impression that no more work is being done.

The use of the cone do not necessarily has to be restricted to the liquid limit, but could perhaps be extended to cover the plastic limit too.

It would seem that soil mechanics need new methods for these determinations, which would shorten the time required and which would eliminate as much as possible the personal factor.

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Chairman N. A. TSYTOVICH

Thank you very much for your interesting report Mr. Pérez.

Now Dr. Mohan, please deliver your contribution.

Panelist D. MOHAN (India)

Even though piles have been in use for so many years now, the knowledge of their behaviour and correct assessment of their load carrying capacity is far from perfect. The reason is that the problem is fairly complicated for a single pile itself and is much more difficult for a group of piles. Some of the important parameters that influence the behaviour of a single pile are the elastic properties of pile material; the elastic, elasto-plastic and plastic properties of the soil; shape and size of piles; method of installation; the type of soil and its physical state including its stress history and time factor involved in driving and testing the piles. It is, therefore, obvious that one cannot get a simple and perfect expression to predict the load bearing capacity of pile which takes into account all the above mentioned parameters. The best the one can get is a solution which, with proper modification, based on judgement and experience of the designer, will provide a safe structure. The factor of safety in such a case is thus a combination of the factor of ignorance, factor of experience and factor for the risk involved.

In the light of above arguments, it is clear that any method old or new, will have its own limitations. Even though the old methods which are mostly based on rule of thumb are gradually being replaced by highly sophisticated and computerised methods, the level of confidence on the latter have not altered significantly. The field engineer or a designer has his own doubts and reservations about these methods. What could possibly have been wrong with our researches in this direction? One of the reasons that I can see is that while we have paid much attention to perfect the computation technique, we have either overlooked or not paid enough attention to the fundamental physical parameters involved in it. Take for example, the method suggested by Poulos and Davis (1968) for prediction of behaviour of a single vertical pile subjected to axial loads. The problem has no doubt been attacked in a right direction analytically. But apart from the fact that the basic assumptions involved in this approach will affect the results, the reliability of the results will be governed by the degree of correctness to which the physical parameters E, E' and μ are assessed. There is no single reliable method to determine these parameters and a lot more work is necessary in this direction.

ANALYTICAL METHODS

The main advantages of Poulos and Davis method is that (1) it gives distribution of stress along the pile-length; (2) the method can be used to analyse pile of any shape e. g. single and multiple under-reamed piles etc., (3) the group effect can be taken into account on a more rational basis, (4) the computations are quicker compared to the method of Seed and Reese (1957). Unfortunately, there are no **field or model test data available to** verify the reliability of this method. This is mentioned by the authors themselves ".....an assessment of the accuracy of such predictions must await comparision with the results of carefully controlled model and field tests".

The method of Seed and Reese (1957) is well supported by field and model tests. It is mainly useful in friction piles with a low point resistance. The reliability of the method depends on the correct assumption of the utilisation of friction forces along the pile shaft. Much work is yet to be done to standardise a method for a correct assessment of these frictional forces. Unlike Polous and Davis method, it is not possible to extend this method to determine the group effect.

PILE DRIVING FORMULAE

Numerous dynamic formulae have been proposed in the past. Their basic principle is that the energy imparted to the pile by the driving hammer is equated to the resistance offered by the ground through which the pile is driven. To this belong the Engineering News Record, Dutch, Ritter, Hiley and several other formulae. Even though the usefulness and reliability of these formulae have been questioned several times, they are still widely used and most of the piling firms estimate their pile capacities with the help of these formulae. What can be the reason of this paradoxical situation? One of the reasons is that they are very handy. The designer does not get lost in the mathematical computations. If you ask the designer why he adopts them when they are not reliable, he will probably reply - "after all what is reliable in Soil Mechanics"? However, this gives a clue to the fact that most of our sophisticated analytical methods, if presented in the form of ready made tables, charts, and nomograms will look more attractive and will be acceptable to the field engineers and designers.

WAVE EQUATION

Another analytical approach in this direction is the prediction of ultimate bearing capacity of pile by wave equation. Even though this method was proposed in 1938 (Glanville et al - 1938), it has come to prominence only recently through the efforts of Smith (1962). The study carried out by Forehand et al (1964) has established that the successful application of this method requires a knowledge of static and dynamic properties of soil, dimensions of piles and properties of the material from which it is made, and information of physical properties of pile driver and associated equipment used. Here again, we are faced with the same problem - how much reliable our knowledge is about the soil properties?

LOAD TESTS

In spite of many sophisticated computerised techniques, the load tests still continue to be the most reliable method of prediction of ultimate load bearing capacity of piles. The static cone penetration test hás also been used to a limited extent. The obvious advantage of these methods over any analytical method is that they do not involve any assumption regarding the soil properties and the complicated soil-pile interaction is taken into account automatically. The load tests too pose their own difficulties - (a) how to define and determine the ultimate load from the load settlement curve; and (b) what can be the best possible method of carrying out the test so that it is most economical, less time-consuming and simulates the actual loading conditions. It is encouraging to note that there is a growing consciousness on the part of researchers to standardise the load test. However, amongst the widely varying practices, there is still need to find one method which fulfils the above mentioned criteria and is accepted universally. This is more essential now in view of the fact that exchange of literature on an international level is helping a lot in advancement of various sciences. An international gathering of this type can play an important role in standardising such practices.

LATERAL LOAD RESISTANCE

A survey of literature for analysis of piles subjected to lateral loads shows that the state of affairs is equally gloomy in this case also. Even though there are many analytical and empirical methods available in the literature, each presents its own difficulties. One of the rational procedures is to treat the pile as a beam on elastic foundation and using Winkler's hypothesis and concept of modulus of subgrade reaction, analyse the problem. Even though an excellent paper has been presented by Terzaghi (1955) on modulus of subgrade reaction, it still leaves many questions unanswered. For example, is it correct to assume that the modulus of subgrade reaction is independent of load? In other words, how far are we justified in assigning only one single value for the modulus of subgrade reaction of a soil at a particular depth? So far its variation with depth is concerned, it has now been established that it varies linearly with depth for sands and normally consolidation clays and is constant with depth for stiff and over consolidation clays. If a more precise variation of modulus of subgrade reaction with loads is also established, one can use the method of Reese and Matlock (1960) or some similar method with suitable modifications. The lateral load test will however continue to play an important role in years to come and there is an urgent need for its standardisation on lines similar to the vertical load test.

PILE GROUP EFFECT

Next we come to the question of analysis of pile groups. It has been known since long that the load carrying capacity of a group of piles for a particular settlement is not just the number of piles times the load carrying capacity of one single pile, yet there is no reliable method to determine quantitatively the actual load carried out by a pile group. Unlike single piles, the load tests of pile groups in the field are not generally possible and only very few tests have been reported in the literature. Most of the studies have been confined to model tests only and based on these a few authors (Whitaker 1957, Safferi-Tate 1961 and Sowers et al 1961) have found out efficiency factors with various configuration and pile spacing. The results of these investigations differ from each other so much that nothing conclusive is available.

MODEL TESTS

One of the main difficulties with model tests in Soil Mechanics is the control on physical properties of the soil. Many research workers are now loosing faith in the reliability of model tests and going in either for very large model tests or full scale field tests. In one of his lectures delivered by G. Leonards at Roorkee (India) very recently, he stated that due to difficulty in simulating the physical properties of soil, it is impossible to get any repetitive and reliable results in model tests on piles. There are, no doubt, many difficulties in controlling the physical properties of soil but I am not unduly pessimistic on this issue. Much more research is obviously necessary to standardise the procedure for obtaining repetitive soil properties like the density of soil, moisture content etc.

Poulos (1968) has put forward an analytical approach to find the influence of one pile over the other in a group. They have used Mindlin's solution for the effect of vertical point load acting inside the semi-infinite space on some other point in the same space. The method appears to be rational but again we are faced with the difficulties of ascertaining the physical parameters, E, E' and μ of the soil and the pile. The author rightly points out that "In applying the theoretical solution to the field problems, it must be borne in mind that the theory at present takes no account of various aspects which may influence the behaviour of a group such as the order of driving of the piles, layering of the soil profile. Also the theory does not take into account any raft action between the pile cap and the soil". The author suggests that because of the difficulty in determining the values of the soil pile parameters, it may be more satisfactory to carry out a field loading test on a single pile and subsequently to apply the theoretical values of settlement ratios to the results of this test in order to predict the settlements of a group. In view of the fact that the soil in the field may not be homogeneous, the authors caution - "It must, however, be borne in mind that such non-homogeneity may modify the interaction between piles as compared with the case of homogeneous mass and thus limit the validity of the theoretical curves in this paper. Further study is necessary in this direction". Will all these precautions and limitations at present not scarce a design engineer and compel him to adopt the old conservative empirical methods like Converse - Labaree, Seiler - Keeney or Feld formulae?

COMBINED GROUP OF VERTICAL AND BATTER PILES

One gets completely lost when he has to design a pile group consisting of vertical as well as batter piles subjected to vertical and horizontal forces along with moments. Even though there are many analytical methods (Hrennikoff 1949) as well as graphical (centre of rotation method) none of them takes into account the efficiency factor which is so strongly advocated in case of pile-groups subjected to vertical loads only. There has been no systematic study either in the field or in the laboratory to find the efficiency factor for pile groups subjected to lateral loads. There is therefore an urgent need for the development of a general analytical method to determine the influence factor for a pile with general configuration (consisting of vertical and batter piles) and subjected to general loading (vertical and horizontal forces with moments). This is needed both for two and three dimensional problems. This study should subsequently be checked by systematic field and model tests.

SUMMARY AND CONCLUDING REMARKS

I would finally like to summarise the points on which we might focus our attention.

(1) An intensive research is necessary to study the physical parameters involved in the pile foundation problem.

(2) Not to scarce a designer with complicated mathematical computation, a systematic study should be carried out to prepare tables, charts or nomograms based either on the analytical approach or field and model tests.

(3) The vertical as well as lateral load tests on single piles being unavoidable at this stage, a standardisation of the technique of carrying out these tests as well as of interpretation of the results need an exhaustive study.

(4) To make the model test results more useful and reliable, a standardisation of the techniques for control of physical properties of the soil is very essential.

(5) A systematic study is urgently needed to find the efficiency factor for pile groups which consists of vertical and batter piles and are subjected to most general type of loading.

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Chairman N. A. TSYTOVICH

Thank you very much Dr. Mohan for your interesting talk on pile behavior. Now I call on Dr. Golder.

Panelist H. Q. GOLDER (Canada)

In general terms settlement IS the problem.

For 'average' buildings and 'average' soils we know enough now to be sure of avoiding rupture in shear of the clay supporting the building.

If we look for examples of shear failure of building foundations in clay, what do we find?

Skempton's paper on 'Kippen' - published in 1942 (Skempton, 1942)
- this was an unusually soft clay shear strength below the crust was 340 lb/sq.ft.

The Transcona Grain Silo - occurred in 1913 (Peck & Bryant, Geotechnique Vol. III (1953) White, L. S. ditto).
This was an unusually heavy building.

There may be others.

But today we should not get 'rupture'; certainly not for 'average' conditions.

Settlement IS the problem.

How much?

How quickly?

Settlement of what? a building, or one footing in a building?

This immediately introduces the concept of 'deformation' - not at all the same thing as settlement. If we know the stresses in the soil due to the building we can make a reasonable stab at calculating the settlement.

We start with Boussinesq's (1885) solution for an ideal elastic material - and a point load. We integrate this to get a loaded area (or a line load) - in fact we use Newmark's (1947) very useful graphical solution. We can now say what the settlement will be at any point of a uniformly loaded rectangle. But for the loading to remain uniform the building must be completely flexible - i.e. the loading is a collection of vertical rods. Such buildings do not come often, although oil tanks approximate to this type of loading.

OR we can say this building is so stiff that it will not deform, and using the work of E. N. Fox (1948) we can say the settlement will be so much. Between these limits the foundation engineer can say <u>nothing</u>! Between these limits the problem is a structural one and a very complex one. Today it is probably possible - for the first time - to analyse a structural frame and by successive approximations to arrive at reasonable values for the final settlement of the individual footings.

But <u>buildings</u> do not consist of structural frames. They consist of frames stiffened by panels. I learned that by observing bomb-damaged buildings in London during the Second World War. And I think it probable (I do not know) that the biggest uncertainty in calculating the settlement of a partially flexible structure is not the soils data but the knowledge of the stiffness of the 'framed and blocked' structure.

I think that we soils engineers are doing not too badly!

Now what about rate of settlement? Here we are on much more shaky ground. I have very little confidence in our predictions of rate of settlement. We very seldom get the classical case of a thin layer of clay between two layers of sand with uniform pressure distribution through the clay (the conditions of most of our laboratory tests). Instead we have to decide, a) where are the drainage layers?, b) is the foundation itself a drain?, c) can we cope mathematically with three-dimensional consolidation in this problem?, d) can we cope with anisotropy in the soil?, and e) anyway, can we cope with the difference between a sample tested in the laboratory and the soil mass? In most cases the answer is NO!

But does this matter? I doubt that it does. If I can tell a structural engineer that the final total settlements of the footings of a given building are not going to exceed certain values, what does he do? He designs the building so that it can accept these values. He does not argue that the creep in the concrete over a period of time will compensate for differential movements and keep the stresses within tolerable limits. This may in fact occur and may account for the fact that many buildings stand up. But I have yet to meet the structural engineer who will include this as part of his design, OR the Building Code which will let him!

So that I have yet to be convinced of the necessity of predicting the rate of settlement for a building on clay. Please note I am not talking about fills, embankments, roads or dams!

I realise that, in exceptional or unusual circumstances, the differential settlement between two footings may be greater at a certain time during consolidation than when the final values have been reached. But I would have little confidence in a building which depended on an exact prediction of the time rates of settlement for its safety, and none in its designer.

Allowable Settlements

Professor de Mello has also asked me to talk about 'allowable settlements'. I am going to be involved in this problem at the next Pan-American conference in 1971. I don't know the answers but I have thought of a few questions.

Who does the allowing? The Building Code, the Architect, the Structural Engineer, the Foundation Engineer, the Client, the Owner, or the User?

Must a building look safe? e.g. Tower of Pisa - do they really want it straightened? or just stopped?

Is settlement which can be seen by the layman always unallowable?

I remember a long single storey office building built in England on peat during the War. It was over a quarter of a mile long and there was a corridor full length right down the middle. The settlement in the centre of the corridor was at least a foot more than the ends. It could be clearly seen and looked very odd. But the building served its purpose until the end of the war when it was no longer needed.

Is settlement which can be seen by the trained eye allowable?

At Westminster Cathedral in London (not Westminster Abbey) the lintel over the main door has settled about 3" at the south end compared to the north end. This has distorted the arch over the doorway. The cause is simple and obvious. The very high and heavy campanile is situated just to the south of the doorway, and the London clay below the structure has consolidated under its weight. Not many people notice it. It was first pointed out in print by Guthlac Wilson (1946). Only the trained eye would see it. It does not affect the function of the structure. Is it allowable? If the architect had been told that it would happen would he have allowed it? How would the engineer have prevented it?

If settlement cracks a vitreous panel is it allowable?

If the panel is changed to something less rigid, is the same settlement allowable? If the usage of the building is unimpaired then can settlements be considered allowable? How much settlement can the structure safely stand?

I hope to get some answers in the discussion.

Floating Foundations

Floating foundations (or partially floating foundations) are used for two different reasons. (Golder, 1965). The first is to reduce settlements, the second to reduce shear stresses in the soil.

In the first case the soil may have considerable shear strength and an excavation can be made to the required depth and the foundation constructed therein. Settlement will be limited to the reversal of bottom heave which occurs during excavation, plus the settlement caused by whatever pressure is applied to the soil in excess of the overburden pressure if the foundation is not fully floating.

In the second case the shear strength of the soil is very low, the lower limit being zero in which case we have a foundation analogous to a ship floating in water. With very low shear strengths the problems are almost entirely construction problems. It is very simple to design a fully floating foundation. It is also very simple to design one which cannot be built. Once the foundation exists in the ground the settlement should be zero. For a given shear strength there is a theoretical limit to the depth of an excavation and this should be considered in design. However this is not necessarily the limiting depth of the floating foundation if suitable construction techniques are used.

The greatest objection to floating foundations is that they are generally expensive.

DeMello's Guide Lines for Discussion

a) levels of confidence of design decisions
b) local experience applied in other places
c) lines of research and collection of information.

a) Levels of confidence of design decisions

As regards prediction of total amount of settlement on clays - quite good - but for rate of settlement very dubious except in the simplest cases.

For design decisions affecting floating foundations - high - but construction decisions are another matter.

As to allowable settlements we can probably produce a design which avoids trouble; but we may be wasting money in doing so.

b) Local experience applied in other places

We must be very careful about this. The best way to deal with a difficult problem in an area which is new to you is to find the best local man and ask him to work with you. You don't have to believe all he says, indeed you should argue with him and make him prove his points by reference to actual case histories.

It may be that there is no local experience. Then we will automatically be relying on our experience of 'apparently' similar soil. But the ordinary index properties may not be enough to describe the soil. I remember many years ago seeing the clay at the site of the Sasumua Dam in Kenya. Two things struck me as unusual, its light weight and its soapy feel. When we did simple laboratory tests the weight per cubic foot, the water content and the liquid limit seemed all wrong, although I cannot now remember the figures. I advised the contractor, for whom I was working at the time, to make special provision for drying the clay before placing. Naturally he did not get the job. Later, as you all know, Terzaghi was called in and discovered that the unusual properties of the clay were due to an unusual mineral, Halloysite, in its composition. He was able to draw on experience of similar clay in Indonesia to aid in solving the problem.
The stiff fissured London clay can be used as another example. When working with fissured clays, the fissure pattern is most important. It affects greatly the strength values obtained from samples of a given size. Index properties will not help on this problem, but of course visual field observations in a cut or a tunnel or a large hole will.

The last thing I want to see is a "Code of Practice for the Application of Local Practice and Experience to other Clays around the World". This would be most dangerous.

c) Lines of research and collection of information

Collection of information, viz. case histories, is most valuable. Short of a major breakthrough by the structural engineers so that they can analyse the whole building and not just the frame, I see no alternative way of dealing with 'allowable settlements'.

Following this breakthrough, if it occurs, and if then structural engineers are willing to vary their working stresses with time, then further work on rate of settlement would be justified. Failing that I would regard work on rate of settlement as academic - but not to be neglected on that count. It would be good to be ready when the structurals arrive.

I see carefully documented case histories, plus field tests and observations as the best lines to follow. This of course costs money. It is beyond the resources of most universities. But by collaboration much can be done. I cite the recent work in Britain on large diameter bored piles with and without belled bases, and several recent cases in Canada of field testing programs costing well over $100,000 each, to decide points we don't know. These tests, paid for by enlightened government or semi-government bodies will pay off in design savings and will be published in due course. This is the way I think we should go.

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Chairman N. A. TSYTOVICH

Thank you Dr. Golder for your very interesting information specially on problems of settlement prediction of structures erected on clay. Now finally I wish to call upon Dr. Rosenblueth from Mexico.

Panelist E. ROSENBLUETH (Mexico)

In his masterful paper of the state of the art de Mello (1969) remarked that the era of statistical analyses and confidence limits must inescapably come to soil mechanics. Soil mechanics is indeed entering this stage. Yet it would be desirable that this phase be superseded. Ultimately, decisions must be taken on the basis of decision theory, bayesian statistics, and the soil-structure system must no longer be viewed by soil mechanicists as almost only soil and by structural engineers as almost no soil, with timid glimpses of each other's domain.

There are some papers, very few, that provide bases for hope that this more desirable state of affairs may not be far away.

The present writing aims at suggesting a framework to broaden the system with which deal foundation engineers and their colleagues from above and from below the foundation, and to introduce a systematic application of decision theory in problems involving buildings on clay. The formulations proposed often pose problems which are still not tractable; suggestions are made to help overcome these difficulties.

<u>The nature of things and the purpose of engineering decisions</u>[a]

Soil mechanicists deal with wide dispersions of soil properties. To them, the dictum "every quantity which occupies the mind is random" seems a truism. However, its meaning is more profound than might be construed from the spread of the mechanical properties of a material identified as belonging to a single stratum. The mind would still deal with random quantities if all the soil underlying a foundation could be tested, either in situ or in the laboratory, and if the loads to be imposed could be weighed beforehand. Randomness is inevitable due to instrumental and sensorial inaccuracies and to errors in the theoretical frameworks into which the results of measurements are inserted. It is also inevitable because the engineer designs not for past but for future events.

The role of classical statistics is a description of some properties of random variables. But description is not decision, and an engineer's principal function is to take decisions. Probabilities and bayesian statistics, not classical statistics permit the engineer to move from his purely descriptive chores to the exercise of his profession.

The solution of real engineering problems consists in discerning which among the several alternative courses is the preferable, the optimum, course of action.[a1] To resolve such problems there must exist a set of design objectives, preferences, or a scale of values, and there must be alternative courses of action, preference for one of which is not established a priori (Ackoff, Gupta and Menas, 1965).

A systematic approach to the analysis of a subject's decision problems requires introduction of a scalar which bears a one-to-one relation with preference and varies monotonically therewith. This scalar quantity is called <u>utility</u>. Problems then consist in finding that alternative which maximizes utility.

Often it is advantageous to introduce the concept, borrowed from mathematical programming, of an <u>objective function</u>, which is to be maximized or minimized. This is a scalar that varies monotonically with utility within the context of a given problem. For example, for given fixed quantities of concrete and steel, one may wish to use as objective function in the design of a foundation girder the stiffness of that girder and seek to maximize it. Alternatively one could minimize the total cost of the girder given a set of restrictions, and that cost would be the new objective function.

Classical statistics provides some information

[a] These paragraphs are partly based on Rosenblueth (1969b).

[a1] When the subject decides for himself, "preference" and "desire" are synonymous. When an engineer decides for an individual, for a group or society, he must replace "desire" with "benefit". In what follows, the term "preference" will be used with both meanings.

on one step in the solution of engineering problems, but this is of limited value. Consider, for example, the matter of confidence intervals. When a statistician says "the mean of a random variable lies between x_1 and x_2 ($x_1 < x_2$) at a confidence level of 95 percent" he is not stating that the probability is 0.95 that the mean lies in that interval. Rather he is saying that "if x had some usually unstated probability distribution (often a gaussian distribution) and if the expectation of x were either smaller than x_1 or larger than x_2, then the probability would be no greater than 0.05 that the number of observed values of x lying between x_1 and x_2 would have been equal to or greater than the number observed in this interval." The relation between this statement and decision taking in engineering is remote.

Calculation of confidence intervals really serves only to satisfy a convention. This entente is so widespread that such statements give a feel for the statistics of a random variable, which in itself is a useful thing. As well, the existence of such a convention justifies the publication of results in this guise for the same reason that the publication of some essentially useless scientific papers is justified -- attaining recognition by the scientific community.

In practical applications, classical statistics provide no rational procedure for differentiating between the level of confidence required for soil properties affecting crucial decisions (say, shear strength under a nuclear reactor) and those which are related with irrelevant matters (say, the compressibility of dense sand under a 6-ft fence). A further limitation is the following: before an engineer examines a set of test results he has an idea of the properties of the material from merely looking at it (so much so that he immedietely detects gross errors in the reported test results) but classical statistics offers no means for incorporating this prior knowledge.

In bayesian decision theory prior knowledge is formally melded with statistical information in a manner useful for decision taking, through application of Bayes' theorem. Before incorporation of statistical data there are a number of exhaustive, mutually exclusive possible hypotheses, H_i, each having prior probability $P(H_i)$ of being true. Statistical information (obtained in an experiment) constitutes an event, say A, which is known to have taken place but which had a prior probability $P(A\ H_i)$ of occurring if the ith hypotheses is true. The (posterior) probability that the ith hypothesis be true given that A occurred, $P(H_i|A)$, is furnished by

$$P(H_i|A) = \frac{P(A|H_i)\ P(H_i)}{P(A)} \qquad (1)$$

where $P(A) = \sum_i P(A|H_i)\ P(H_i)$ is the probability of A independently of which hypothesis is true.

As an example suppose that a soils engineer knows that at a given site he will meet either soft clay or an outcrop of a stiffer formation and he assigns these possibilities the respective probabilities 0.01 and 0.99. Now he learns that there is a building standing at the site and that it has had foundation problems. He reasons that this may have come about with a probability of 0.9 if the building had been standing on the soft material and of 0.2 otherwise. He computes the posterior probability of there being soft clay as

$$\frac{0.9 \times 0.01}{0.9 \times 0.01 + 0.2 \times 0.99} = 0.043$$

This calculation would aid him in deciding on the type of soil exploration to adopt.

As a second example, suppose that the engineer knows of eight very different theories for predicting the bearing capacity of a circular deep foundation (see de Mello, 1969). If he is not versed in the theory of plasticity his only basis for judgment will be the confidence that the various authors inspire in him and the dates at which the theories were published. Accordingly he would assign different probabilities to the various theories. By looking at test results and the behavior of existing foundations he would be able to compute the probability distribution of the bearing capacity of the foundation whose design he confronts.

Equation 1 has been generalized to deal with continuous random variables (Raiffa and Schlaifer, 1961). In that form it can be used for the computation of the posterior probability distributions of parameters which define soil properties.

Much of the opposition to the use of Bayes' theorem in this context stems from the difficulties inherent in specifying satisfactory prior distributions. These should incorporate all the prior information available consistent with time restrictions and, since both this information and the restrictions vary from one individual to another, the results are based on subjective considerations. There are basically two alternatives in this state of affairs. One consists in ignoring prior information utterly and basing decisions on implicit, unconfessed prior distributions, apparently taking statistical data at face value. This approach is the essence of conventional statistics. The other consists in applying engineering judgment so as to incorporate prior information intuitively even if cryptically. It seems preferable to deal with that prior knowledge openly, with all cards visible on one's table.

There is a vast class of problems for which the task of establishing the prior distribution can be much simplified. When one can be sure thet a variable has a certain kind of distribution (normal, Poisson, etc) because the conceptual model adopted so dictates, it is often possible to use prior distributions for the parameters of the variable's distribution, whose functional form will not change as statistical information is incorporated. Such distributions are known as <u>conjugate</u> or <u>natural conjugate</u> distributions (Raiffa and Schlaifer, 1961). For example, if the occurrence of a series of events is idealized as belonging to a Poisson process, the natural conjugate distribution for their mean rate of occurrence is the gamma-1 distribution. Incorporation of experimental data yields a gamma-1 distribution. For the expectation of a normal distribution of known variance the conjugate distributions are normal.

Often the conjugate distributions are defined by two parameters whose interpretation is intuitively appealing; one parameter corresponds to the prior estimate of the expected values of the variable and the other measures the uncertainty in this estimate. Calculation of the posterior distribution is usually simple under these conditions.

The true difficulties in the application of Bayes' theorem lies in the need for complicated analytical derivations, arising from the optimization process, in practically all cases of interest and for practically all cost functions. Much would be gained by setting up computer programs capable of solving several common types of problems for arbitrary probability distributions.

At any rate, in principle one can take both theoretical considerations and observed phenomena into account to arrive at the probability distributions of the variables pertinent in design. This is a necessary step for the calculation of (expected) utilities and the choice of optimal design alternative.

In civil engineering design it is usually convenient to take as objective function the quantity

$$E\,X = B - C - D \quad (2)$$

where E denotes expectation, B is the benefits derived from the existence of the work in question, -- which for the sake of brevity will be called "the building," -- C is its initial cost and D is the losses due to damage or failure. All these quantities are expected present values.

If values are actualized (discounted or translated into present values), using a constant rate of continuous interest, say γ, and if all modes of failure imply, among other things, that the building ceases to produce benefits, one may write

$$B = \int_0^{\infty} bR\, e^{-\gamma t}\, dt \quad (3)$$

and a similar expression for C. Here b is the expected net benefit per unit time (after subtracting the cost of maintenance) and R(t) is the <u>reliability function</u>, that is, the probability that the building does not fail up to time t.

For D one may write

$$D = \sum_i \int_0^{\infty} D_i f_i\, e^{-\gamma t}\, dt \quad (4)$$

where D_i is the expected loss in case the building should undergo the <u>i</u>th mode of damage or failure and f_i is the corresponding <u>failure density function</u>. In other words, f_i is the time derivative of the failure distribution function, F_i, where $F_i(t)$ is the probability that the structure has failed or been damaged by time t in the <u>i</u>th mode. When only one failure mode is considered, say i = 1, R is equal to $1 - F_1$. Otherwise, $R = 1 - \sum_i F_i$ if the modes are independent or, more generally, R = 1 - F, where F is the probability that at least one mode of failure takes place.

If failure, or damage, does not impair the building's functions, or if the building is immediately and systematically repaired or rebuilt, R must be replaced with one in eq 3 and the second member in eq 4 becomes an infinite series corresponding to successive failures and repairs. (More generally, in eq 3, R must be replaced with one minus the probability that the building ceases to function.) The first term in the series is given by eq 4. If $\int_0^{\infty} f_i\, e^{-\gamma t}\, dt \ll 1$ there is little error in dropping all higher terms.

In many cases b depends little on the design parameters, and either $\int_0^{\infty} R\, e^{-\gamma t}\, dt \doteq 1/\gamma$ or R should be replaced with 1 in eq 3. The problem reduces then, approximately, to minimizing C + D, and this sum may be taken as the new objective function.

In some problems it is licit to assume that failure either occurs upon dedication of the building or not at all. The time variable may then be omitted and eqs 3 and 4 written

$$B = B_0 R \quad (5)$$

$$D = \sum_i D_i F_i \quad (6)$$

where B_0 is the benefit derived from the existence of the building if it does not fail.

In order to compute the reliability and failure distribution functions one must find, as functions of time, the probability distributions of resistance (or stiffness) parameters and load parameters. Problems concerning the design of building foundations on clay present special and very interesting aspects in connection with the calculation of these probability distributions as well as matters concerning modes of failure and their consequences.

The purpose of foundation engineering

In exceptional situations, such as that of a foundation on point bearing concrete piles, relatively small deformations may cause fracture in the foundation and thereby a variety of failures in the building (see de Mello, 1969). Usually, though, large deformations are not objectionable from the viewpoint of what damage they may cause in the foundation itself but in the superstructure and in nonstructural elements.

The usual approach to the question of settlements has been to list a set of "permissible" total settlements, slopes and angular distortions due to differential settlements (the subject reviewed by de Mello, 1969). There are several reasons why this way of focusing the problem is objectionable. For example, there is no indication in any of the lists of "allowable" values published of how serious the consequences of exceeding those values might be, of how much should be spent on the foundation in order that the values not be exceeded, nor of the spread in what will actually happen given that one has computed some differential settlements using conventional methods of analysis.

Evidently one should treat probabilistic quantities as such, at least when the spread or uncertainty in their values is as great as in foundation engineering. Also, one should specify loss functions, which vary with total settlement, with tilt and with angular distortion, rather than setting absolute limiting values, at least when the relation between computed settlements and the consequences thereof are as uncertain as in foundation engineering.

The paper by Reséndiz and Herrera (1969) deals with one problem concerning building foundations in clay essentially as proposed in the foregoing paragraph, even if the approach used by the authors is simpler than that to be proposed in this paper.

Definition of the system to consider in analysis is far from a trivial question. The system should include at least the building and the soil on which it rests but in principle it could comprise the entire economy of the nation, and more. Too wide a definition becomes unmanageable, however. It seems advisable to include in the system, in detailed fashion, the building and the soil under it; in somewhat vaguer form the surrounding and nearby structures and utilities, and only vaguely the much wider items.

For example, social benefits derived from the job opportunities created by the construction of the building in an area suffering from unemployment could be recognized by adjusting the cost of construction, entering eq 2 with a reduced value of C.

Such considerations will be omitted in the rest of this paper.

A rational approach to the matter of settlements

The preceding discussion suggests an approach to to the matter of settlements of buildings on clay. In general terms it consists of the following steps.

1. Define the possible alternatives in design.

2. For each alternative compute the expected benefits to be derived from the existence of the building, the expected initial cost and a set of expected loss functions of total settlement tilt and angular distortions.

3. For each alternative compute the probability distribution of these total and differential settlements as functions of time.

4. By combining the results of steps 2 and 3, find the present values of the expected losses due to settlement.

5. Compute the objective function E X for each alternative, as given by eq 2 and select the alternative that maximizes E X.

These steps will now be examined in detail. For the sake of generality it will be assumed that the restrictions imposed by building codes are not applicable.

Design alternatives

A formal process of optimization for the totality of conceivable alternatives would be a formidable task even with the aid of large-capacity computers. Part of this is obviated by making use of the engineer's judgment and experience, which drastically narrow the range of cases worth examining. Further simplification may be obtained by, for example, choosing the optimal load factor or working stress in the steel, with some but probably not excessive departure from the optimum solution, rather than dealing with the amount of reinforcement at each section, top and bottom, of every foundation girder.

The questions of the number of alternatives to consider and of the degree of refinement justified in analysis will be taken up later.

Benefits derived from the existence of the building

The benefits to consider depend on the preferences of the subject for whom the engineer wishes to optimize. Among such subjects there is, of course, the owner, for whom the benefits may consist of the rent produced by the building, its advertising value, its role in production (as in the case of a building which houses a plant for the generation of electricity or for water treatment) or the shelter it provides for the owner, his family and his household. Inasmuch as engineering is a profession, there is always society, for whom the benefits extend far beyond the lifetime of the owner; also there are the architect, the builder and all other persons who participate in the conception and construction of the building and whose reputation hinges on the building's permanence. The direct benefits to the engineer, as derived from the existence of the building, are implicit in the foregoing concepts, for he is paid by the owner (directly or through the architect) and he owes allegiance to society and loyalty to his colleagues and collaborators. The question of how much time, money and effort he should spend on exploration, tests, analysis and design will be considered later.

It is usually easy to estimate the rent that a building will produce. The other kinds of benefits mentioned can be estimated in the same manner as utility in general (Fishburn, 1967; Rosenblueth, 1969b). A particularly useful guide in this task consists in first answering the question "How much would the subject be willing to pay for such a benefit?," where the subject may be the owner, society or those who participate in the conception and erection of the building.

While estimation of direct initial costs is usually a simple matter, it may sometimes be proper to consider the benefits derived from creating jobs for the construction workers, as pointed above.

Loss function

Consider first the matter of settlements. Ordinarily the complete relationship between expected loss and some type of settlement cannot be obtained, due to a lack of reliable data. However, the expected loss usually increases at an increasing rate with settlement. Accordingly, it is often adequate to assume that it is proportional to the square of the settlement* over a wide range of settlements, even if the idealization breaks down for extremely large settlements.

Different loss functions must be postulated for total settlement, tilt and angular distortion, the latter perhaps differing over each bay of the building. The following example may help elucidate the task of estimating less functions.**

The example concerns an apartment building, 60 m tall, having reinforced concrete frames and whose hollow-block partitions are tied to the frame. It measures 20 x 20 m in plan. It rests on a very compressible but impervious clay, so that settlements can be expected to take place at a very slow rate, say 90 percent of the total reached in a period of a few months (for angular distortions and tilt) to a few years (for average settlement). There are no nearby structures. The building is surrounded by sidewalks and underground municipal utilities. For the sake of brevity in presentation, only settlements due to gravity loads will be considered.

The average peripheric settlement is directly related with damage to the sidewalks and municipal utilities. From past experience it is estimated that 40 cm of average settlement of the periphery would cause damage to the sidewalks and utilities whose repair would cost about 4 k.† The worth of the inconvenience to pedestrians is estimated on the basis of how much they would probably be willing to pay to avoid the nuisance caused by repair work. This gives about 1 k. Then there is the expected loss to the city while the utilities are being repaired. Following a practice that is common in the application of decision theory to the design of electrical systems and to their maintenance, it will be assumed that failure to give a service to which people are already accustomed implies a loss of ten times the rate charged for that service. On this basis the loss is estimated at 2 k. There is also a probable decrease in rentability of the entire building, which is taken as 1 k, and the harm done to the reputation of those associated with the building, which may amount to 6 k, assuming that during one year they would suffer a small decrease in the expected number of their clients.

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* The suggestion is due to C A Cornell. It served as basis for the loss function assumed by Reséndiz and Herrera (1969).

** The example is abstracted from actual cases in the files of DIRAC, Consulting Engineers, Mexico City. When judging the estimates to follow, one should keep in mind that construction costs vary from one country to another and even within a single country and that in some cities the inhabitants are quite used to seeing cracked plaster and huge settlements.

† k = kilodollars.

Similar considerations of the probable need to make changes in the ground floor, combined with the foregoing figures, suggest a total expected loss of 10 k for the owner, 4 k to the city and 8 k to the architect and the builders. The engineer would give a greater weight to the owner's losses than to those of others since the owner must pay to decrease all the expected losses. Thus he may arrive at a weighted total expected loss of 10 + 0.5(4 + 8) = 16 k.

For average peripheric settlement it is reasonable to assume a loss function varying as the square of the settlement. It could be argued that some of the quantities considered would not change with settlements increasing beyond 80 cm, say, but the range of greatest interest lies in smaller settlements, and other losses such as in rentability would increase rapidly with settlement. Hence, the simple hypothesis will be adopted, giving

$$D_1 = 16(\rho/40)^2 = (0.1\rho)^2$$

where D_1 is the expected loss, in k, due to an average peripheric settlement of ρ, in centimeters.

To estimate D_2, the expected loss as a function of $\bar{\theta}$, the average tilt, one must consider damage to sidewalks and municipal utilities once again. Further losses stem from a decrease in rentability due to sloping floors and unsightly inclination of the facade, difficulties in the operation of elevators, a decrease in the capacity to resist lateral loads and, for very large tilts, the possibility of overturning.

The first group of losses interacts with those due to average peripheric settlement, so that, rigorously, $D_1 + D_2$ is not the sum of a function of ρ and a function of $\bar{\theta}$. However, $\bar{\theta}$ must be very large and ρ very small for the interaction term to be significant. Hence it may be assumed that D_1 depends only on ρ while D_2 is a function of $\bar{\theta}$ alone.

In general, losses related to sloping floors is independent of the height of the building while those related to unsightliness are certainly an increasing function of height.

Costs arising from difficulties in the operation of the elevators depend essentially on the total run of the elevators. Expected losses from a decrease in resistance to lateral loads and from the danger of overturning can be computed in a relatively straightforward fashion.

An idea of some of these losses can be had from case histories in which it is known how much the owners have been either prepared or unwilling to pay for underpinning to straighten their buildings.

Taking these data into account and proceeding as for D_1 it is estimated that

$$D_2 = 80(100\,\bar{\theta})^2$$

where D_2 is in k and $\bar{\theta}$, in radians, is the combination of the tilts in two orthogonal directions.

Angular distortions produce, chiefly, cracking of plaster, walls and partitions, structural damage and difficulty in operating doors and windows. It is worth noting that cracking of plaster often occurs with no appreciable differential settlements and that damage from angular distortions is greatest in the lower stories. There is also a reduction in the capacity to resist lateral load.

The quantity most directly related to the ensuing losses, among quantities at whose estimate it is simple to arrive, in the sum of the variances of the slope, measured along the column centerlines of the building in two orthogonal directions. This measure is adequate if the total length of walls and partitions is approximately equal to the total length of the lines through column centroids in each direction in typical stories. Otherwise a different weight should be ascribed to the variance depending on the concentration of nonstructural elements.

For the example in question it is estimated that the expected loss is given by

$$D_3 = 20\,\Sigma(100\,\theta_x - 100\,\bar{\theta}_x)^2 + 20\,\Sigma(100\,\theta_y - 100\,\bar{\theta}_y)^2$$

where D_3, the expected loss, is in k, θ_x and θ_y are the slopes, in radians, between consecutive columns in the x and y directions respectively and $\bar{\theta}_x$ and $\bar{\theta}_y$ are mean values of θ_x and θ_y.

(In a more rigorous treatment, D_2 and D_3 would have included some loss of prestige. It would also have been proper to recognize that an engineer looses prestige if his design is excessively conservative by current standards.)

The total loss for a given design is the sum of the present values of D_1, D_2 and D_3. Discounting may be performed assuming that D_1 occurs the years after completion of the building while D_2 and D_3 take place one year after completion. A more accurate treatment of discount is unwarranted since the losses in question take place in no more than a few years, except for a minor contribution due to the decrease in lateral-force capacity, as failure under lateral load may occur many decades after completion of the building.

A more refined treatment of expected losses due to settlements would incorporate the fact that part of the settlements take place during construction and cause less damage than has been assumed. However, for buildings on clay, particularly on clay of the type being considered in the example, most of the settlements take place once construction has been completed. Moreover, great refinement is not justified in this step of the calculations, since at optimum the utility is stationary relative to the design parameters.

In some cases of very large settlements or of shear failures there is need to consider "intangible" losses as well as those dealt with in this example. The most delicate one met in practice is the loss of human life. Even if he dislikes admitting it openly, the engineer always associates a dollar value with human life whenever he takes a decision affecting safety, for the probability of failure is always finite over a finite period of time. Ignoring the issue does not resolve it and involves the danger of assuming too high or too low a value for this concept.

On the other hand the question "How much is a human life worth" has no sensible answer. Instead one should ask "How much is society willing to pay to save a human life?" It has been found that, if the life is anonymous, society usually behaves as though it were willing to pay an amount equal to the present value of the average individual's expected contribution to gross national product.*

Probability distributions of settlements

It will first be assumed that soil properties are to be explored through sampling and laboratory testing. A bayesian approach to the calculation of the probability distributions of settlements involves then the following steps.

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* R L Ackoff, personal communication; see also Rice (1966). The result can be justified rationally under the assumption that there is no emotional value attached by society to the life of its individuals.

1. Establish prior distributions of the soil properties which govern settlements as functions of time (instantaneous moduli, compressibilities, preconsolidation pressures, permeabilities, boundary conditions, etc). This can be done on the basis of site inspection, topographic features, geologic information, the behavior of existing structures and knowledge acquired from nearby borings.

2. Convert laboratory test results to obtain the distributions of the actual properties of the samples.

3. Convert the properties of the samples into properties of the soil in situ.

4. Combine the results of steps 1 and 3 through use of Bayes' formula (eq 1) to obtain the posterior distributions of the relevant soil properties.

5. Establish the probability distributions of the "actions"[a] that may affect the soil, including the effects of excavation, pile driving, construction, dead and live loads, earthquake and wind, etc.

6. If only one theory is deemed applicable for the prediction of settlements, establish the probability distribution of actual settlements given their computed values, under the assumption that these values were known deterministically. If more than one theory is thought to have significant probability of being applicable, this step should be carried out for each theory and each should be assigned a probability that it hold approximately true.

7. Combine the results of steps 4-6 to obtain the probability distributions of the settlements.

Adaption of this procedure is obvious to cases in which other methods of soil exploration are used.

This sequence of steps can be incorporated into Terzaghi's <u>observational method</u>, described by Peck (1969). Results of step 7 can then be modified in the light of field observations and an additional application of Bayes' theorem. In fact, such an approach is used informally in much of the contemporary practice of soil mechanics.

Ordinarily there is experience concerning the application of theories for settlement prediction using the results of laboratory tests rather than the actual soil properties and with nominal rather than with actual loads. It is proper, then, to take this experience into consideration, through an additional application of Bayes' theorem.

In many practical problems computation of the probability distributions of angular distortions must reflect the interaction between building and soil. Hence, the building's stiffness must be characterized by probability distributions, as must the yield moments in the foundation when plastic action is mobilized to adjust to the differential settlements imposed by the soil deformations.

Often soil mechanicists live under the delusion that loads are known with almost deterministic accuracy, although even dead loads involve large uncertainties. When construction nominally complies with the construction drawings, actual dead loads commonly exceed their conventionally computed values by more than 20 percent (Freudenthal, 1962; Rosenblueth, 1970). The existence of buildings having one or two more stories than the number assumed when designing the foundation is a more frequent occurrence than many inexperienced engineers like to believe. In the present paper these major discrepancies will not be considered and attention will be centered on "ordinary" differences between nominal and actual loads.

Live loads of course tend to be shrouded in greater uncertainties than dead loads, and the uncertainties are far greater in connection with forces due to earthquakes, wind and impact.

The task of establishing and computing the complete probability distributions of the random variables mentioned previously would be formidable and would serve no useful purpose. One need only compute those functionals or parameters of the distribution functions which are relevant in the calculation of the loss functions. If the latter are approximated by quadratic functions of the settlements, only the expectation and variance of the distribution functions need be computed.

This is precisely the simplification which makes the problem studied by Reséndiz and Herrera (1969) tractable. It is true that the authors assume that the compressibilities of different soil elements are uncorrelated. This leads to gaussian distributions for the settlements, and a gaussian distribution is completely described by two parameters. Even if in an attempt for greater realism in the assumptions one were to recognize finite correlations between compressibilities it would still suffice to compute expected values and variances of the settlements.

Calculation of present values

Ordinarily present values are obtained as in answer to the question "How much should I invest now, at a constant rate of continuous compound interest, to cover exactly all the expected losses?" The matter of deciding on this rate of interest is of prime importance. Decisions to build, to repair or to opt for a more conservative design have to compete with the possibility of investing outside the system being designed. Hence, the most obvious answer is to take for actualization purposes the interest rate at which investments are normally conducted in the wider context. This would imply, however, that future rent and expenditures would be reckoned at future prices, which unnecessarily increases the numerical toil. If a constant inflationary rate is expected to operate for many years, one may ignore the phenomena by deducting this rate from the rate of interest.

Uncertainty as to the benefits expected from the existence of a building may differ appreciably from the uncertainty implicit in the risk associated with "normal" investments. This difference can also be conveniently incorporated into the analysis of utility by modifying the interest rate.

In some situations the special need for capital at certain future time intervals can be predicted. Actualization functions other than the exponential are in order for such problems.

The cost of engineering

Sometimes the engineer works under a rigid set of restrictions fixed, on the one side, by the budget and time limitations imposed by the owner and, on the other, by building code requirements. When there is freedom to choose the extent of studies in connection with a given project, he must decide in such a way as to optimize the end result.

For the sake of simplicity it will be assumed that the engineer is working on a cost-plus basis and

[a] <u>Actions</u> is used here in the sense of the French <u>solicitations</u> or the Spanish <u>solicitaciones</u> to embrace imposed forces and imposed deformations as well as any other phenomenon that may appreciably affect soil behavior.

that he is getting paid by the owner. To take the optimal decision means, then, to make a new objective function, say Y, a maximum, where Y is equal to X (eq 2) minus the cost of engineering to the owner.

The design alternatives to consider depend on the extent of the engineering studies. For this reason it is convenient to visualize the decision process as shown schematically by the tree in fig 1. The term "experiments" in the figure covers site inspection, soil soundings, sampling, laboratory testing, analysis and other engineering studies. "Results" identifies the possible outcomes of these studies. "Actions" refers to the possible adoption of design alternatives. Finally "states of nature" embraces the actual loads on the foundation, the actual soil properties and the theories that truly apply in the case in question; that is, the behavior of the system.

White circles in the figure identify situations in which the engineer takes a decision; black circles, situations in which the subsequent course of events escapes the engineer's control ("decisions taken by nature"), branches beginning at the first type of node are identified by different utilities, or values of the objective function, so that rational behavior calls for choosing the branch leading to the highest utility. Nodes of the second type lead to branches associated with probabilities of occurrence, which permit compounding the corresponding utilities to arrive at expected values.

At the terminals of the last branches one may write the utilities associated to the corresponding alternatives. By proceeding from right to left one defines the optimal decisions to be taken and the maximum utility associated with the process at this stage (this utility is known as the <u>value of process</u>).

Thus, this type of schematic presentation helps clarify the process of decision taking.

A tree can be made to have any number of stages, or groups of branches, so as to depict situations in which the information supplied by various stages of exploration, testing or analysis is used to take new decisions as to the extent of additional studies. The process of optimization at each stage must take into consideration the cost of deciding the extent of the studies, as well as the cost of the studies themselves, and must be based on the information available at that stage. The first group of decisions of this type usually does not justify a formal process of optimization but may advantageously be replaced with estimates at the intuitive level.

<u>A problem of total and differential settlement under static load</u>

As a first simplified illustrative situation consider a building having a rigid, partly compensated foundation and resting in a clay formation. The soil consists of horizontal layers and lies on an incompressible halfspace.* The building's base is symmetrical about two orthogonal axes. It exerts a known mean gross pressure on the ground, say q_g, which is centered in the base. The compression ratio, C_{ri} of the ith layer is assigned a normal probability distribution† with unknown mean $\overline{C}_{ri}$, and known <u>dispersion</u> (standard deviation) s_i (s_i^2 is known as the <u>variance</u>). It will be assumed that C_{ri} is not correlated in space. It is desired to find the optimal sampling program‡ and the optimal depth of excavation.

The natural conjugate prior distribution of C_{ri} is normal (Raiffa and Schlaifer, 1961). The parameters of the prior distribution are $E\,\overline{C}'_{ri}$ (its expectation) and n'_i (equivalent to the number of specimens in a fictitious prior sampling process). The prior variance of $\overline{C}_{ri}$ is s_i^2/n'_i. (A single prime is used to refer to the prior distribution, no prime to the sample results and double prime to the posterior distribution.)

Sampling the ith layer will produce say, n_i specimens, with mean

$$E\,\overline{C}_{ri} = \frac{1}{n_i}\sum_{j=1}^{n_i} C_{rij} \tag{7}$$

where subscript j identifies the jth specimen in the sample.

The posterior distribution will be characterized by parameters $E\,\overline{C}''_{ri} = (n'_i\,E\,\overline{C}_{ri})/n''_i$ and $n''_i = n'_i + n_i$ and the posterior variance of $\overline{C}_{ri}$ will be s_i^2/n''_i. However, before sampling, the test results are unknown, but the probability distribution of $E\,\overline{C}_{ri}$ can be deduced from the prior distribution and the size of the sample. The following relations (Raiffa and Schlaifer, 1961) concerning this distribution (which is conditional, as it depends on the prior distribution) will be useful in developing the theory in question. They correspond to fixed n_i and given $E\,\overline{C}'_{ri}$ and var $\overline{C}'_{ri}$:

$$E\,E\,\overline{C}''_{ri} = E\,\overline{C}'_{ri} \tag{8}$$

$$\text{var}\,E\,\overline{C}''_{ri} = s_i^2 n_i/n'_i n''_i \tag{9}$$

$$\text{var}\,\overline{C}''_{ri} = s_i^2/n''_i \text{ deterministically} \tag{10}$$

Now let ρ_i stand for the contribution of the ith layer to the building's mean settlement, let $\mu_i = E\,\rho_i$, $\sigma_i^2 = \text{var}\,\rho_i$ ane let m'_i and v'_i denote the prior expectation and variance of μ_i respectively. Under the assumption of linear behavior, and admitting, as done by Reséndiz and Herrera (1964), that the stress distribution in the clay is not modified by the space distribution of compressibility, μ_i will be proportional to $q\overline{C}_{ri}$. Here q is the net applied pressure, equal to the contact pressure q_g minus $D_f\gamma_1$ if D_f is the depth of excavation and γ_1 is the unit weight of the first layer, under the assumption that D_f does not exceed the thickness of this layer. Denote by α_i the constant of proportionality. In the notation of Reséndiz and Herrera, but using subscript i to identify a layer rather than a sublayer, $\alpha_i = f_i/\overline{C}_{ri}$.

Similarly, $\sigma_i^2 = F_{0i}$ is proportional to s_i^2, and var $\theta_i = \beta_i\sigma_i^2$ where θ_i is the ith layer's contribution to the angle of tilt and $\beta_i = (F_{1i} + F_{2i})/F_{0i}$.

Following Reséndiz and Herrern, the objective function will be taken as the initial cost of excavation minus the cost that would be required by total compensation and plus the present value of the expected loss due to average settlement and to tilt. It will be assumed that the latter losses are proportional, respectively to $(\Sigma\rho_i)^2$ and to var $\Sigma\theta_i$. Thus, the

* This is the example analyzed by Reséndiz and Herrera (1969) where, however, the parameters of the probability distributions of the clay compressibilities are assumed to be known.

† See Reséndiz and Herrera (1969) for a justification of the assumption that these distributions are normal.

‡ Attention will be limited to programs of fixed sample size, as this problem does not particularly lend itself to sequential sampling.

expected value of the total losses, given the soil properties, is

$$X = -C_2 q/\gamma_1 + \left[C_3 \, E\,(\Sigma\rho_i)^2 + C_4 \, \Sigma\beta_i\sigma_i^{\,2}\right] q^2 \qquad (11)$$

where $C_{2,3,4}$ are constants, C_1 is reserved for the initial cost of the building excluding excavation. From eq 11,

$$X = -C_2 q/\gamma_1 + \left[C_3(\Sigma\mu_i)^2 + \Sigma(C_3 + C_4\beta_i)\sigma_i^{\,2}\right] q^2 \qquad (12)$$

Before sampling, the expected value of X is

$$E\,X' = -C_2 q/\gamma_1 + y' q^2 \qquad (13)$$

where $y' = C_3\left[(\Sigma m_i')^2 + \Sigma v_i'\right] + \Sigma(C_3 + C_4\beta_i)\sigma_i^{\,2}$ (14)

and $v_i' = \alpha_i^{\,2} s^2/n_i'$

The optimum depth of excavation is that which minimizes $E\,X'$. Denote by q_o' and X_o' the corresponding values of q and $E\,X'$. Differentiating eq 13, equation to zero and solving, one finds

$$q_o' = \frac{C_2/2\gamma_1}{y'} \text{ but } \leq q_g \qquad (15)$$

$$X_o' = -\frac{(C_2/2\gamma_1)^2}{y'} \text{ but } \geq X_g \qquad (16)$$

where X_g is the value of $E\,X'$ associated with $q = q_g$.

Equation 15 defines the best decision possible if the soil is not to be sampled, while eq 16 measures the economic consequences of that decision.

After soil testing, the situation is described by replacing primes with double primes in eqs 14-16. In order to decide whether to sample and, if so, how many specimens to test, use will be made of eqs 8-10. Suppose first that the cost of soil exploration including the testing of n_i samples from the ith layer is $a + \Sigma b_i n_i$ where a and b_i are constants. Then eq 13 gives rise to

$$E\,X'' = -C_2 q/\gamma_1 + y'' q^2 + a + \Sigma b_i n_i \qquad (17)$$

Proceeding as with eqs 14-16,

$$X_o'' = -\frac{(C_2/2\gamma_1)^2}{y''} + a + \Sigma b_i n_i \text{ but } \geq X_g + a + \Sigma b_i n_i \qquad (18)$$

The gain derived from sampling is

$$G = X_o' - X_o''$$
$$= (C_2/2\gamma_1)^2 \,(E\,1/y'' - 1/y') - a - \Sigma b_i n_i \qquad (19)$$

with the restriction that q not exceed q_g.

It is worth sampling if and only if G is positive when $\Sigma b_i n_i$ is replaced with its minimum possible value, $\min_i b_i = b_j$, say, and $E\,1/y''$ with its corresponding magnitude (for which, $n_i = 0$ for $i \neq j$ and $n_j = 1$). The optimum sampling plan is either no sampling (if $G \leq 0$ under these conditions) or that set of n_i which maximizes G in eq 19.

Notice that $E\,y'' = y'$. (This can be proved straightaway by using eqs 8-10.) It follows as a corollary that if sampling does not entail the possibility of changing q, $E\,X''$ will only differ from $E\,X'$ by the cost of sampling. This conclusion is consistent with the assumption that sampling does not change the soil's properties appreciably.

Equation 19 may be evaluated through numerical integration or using Monte Carlo analysis. Then the optimum set of n_i may be found by trial and error. An approximate evaluation of $E\,1/y''$, which ignores the restriction at X_g, is achieved through a change of variable, which produces an integral that can be evaluated in terms of the complex error function w (see Handbook of mathematical functions ...., 1964), which in turn is tabulated and for which an applicable asymptotic expression is available.

An even less accurate but simpler solution, valid when the optimum q is not very close to q_g, results from expanding $1/y''$ in terms of $(y'' - y')/y'$ and calculating its expectation term by term in the ensuing series. (The procedure is not formally correct because the series does not converge uniformly, but it can be shown to provide the correct answer when all the moments of the distribution of y'' exist. This is the case when the prior distribution of the expected compressibility is normal.) Preserving only the first term of the series is satisfactory when all the n_i are small or moderate. Making use of the relation $E\,y'' = y'$ it is found that

$$E\,1/y'' - 1/y' \doteq \text{var}\, y''/y'^3 \qquad (20)$$

It follows from eqs 10 and 14 that

$$\text{var}\, y'' = C_3^{\,2} \,\text{var}\,(\Sigma m_i'')^2$$
$$= 2C_3^{\,2}(\Sigma\bar{v}_i'')\left[2(\Sigma m_i')^2 + \Sigma\bar{v}_i''\right] \qquad (21)$$

where $\bar{v}_i''$ is var E m_i'' (var $\bar{C}_{ri}''$/var $\bar{C}_{ri}$) var $\mu_i' = (n_i/n_i'')v_i'$, according to eq 9.

In practice the coefficients of variation of the compression ratio often vary over such a small range in each layer that they may be taken as deterministic quantities. (For example, data reported by Reséndiz and Herrera, 1969, indicate a coefficient of variation of the order of 0.5 for the coefficients of volume change, and hence for the compression ratios, of Mexico City clay as well as of Chicago clay.) These coefficients of variation, $c_i = s_i/C_{ri}$, rather than the variances, s_i^2, are then to be taken as known.

Under these conditions the conjugate distribution is also normal. Equation 12 becomes

$$X = -C_2 q\gamma_1 + \left[C_3(\Sigma\mu_i)^2 + \Sigma A_i\mu_i^{\,2}\right] q^2 \qquad (22)$$

where $A_i = (C_3 + C_4\beta_i)k_i^2$ and $k_i = \sqrt{F_{0i}}/f_i = \sigma_i/\mu_i$ is the coefficient of variation of ρ_i. Therefore, eq 14 becomes

$$y' = C_3(\Sigma m_i')^2 + \Sigma(C_3 + A_i)v_i' + \Sigma A_i m_i'^2 \qquad (23)$$

where $v_i' = c_i^2 m_i'^2/n_i'$. Finally eq 21 becomes

$$\text{var}\, y'' = \text{var}\left[C_3(\Sigma m_i'')^2 + \Sigma A_i m_i''^2\right]$$

Although it is not difficult to calculate an explicit expression for var y'' the result is cumbersome. It will suffice to note that

$$A_i^2 \,\Sigma\bar{v}_i'' + V \leq \text{var}\, y'' \leq (1 + A_i/C_3)^2 V \qquad (24)$$

where V is var y'' when the σ_i are known, that is as given by eq 20.

There are also occasions when both $\bar{C}_{ri}$ and s_i are unknown and there is no known relation between them. The joint conjugate distribution of $\bar{C}_{ri}$ and $1/s_i^2$ is normal-gamma; the marginal distribution of $\bar{C}_{ri}$ is student, and that of $1/s_i^2$ is gamma-2 (Raiffa and Schleifer, 1961). The problem can be given a treatment similar to those presented in the foregoing paragraphs but evaluation of var y'' requires numerical integration. Hence, it is not worth introducing the approximation in eq 20. Numerical integration to evaluate $E\,1/y''$ may be too time consuming, and a Monte Carlo approach seems indicated.

In most practical problems there is enough prior information to estimate the expectations and coefficients of variation of the compression ratios, so that the prior statistics m_i' and σ_i can be had with ease. In order to establish n_i' on subjective bases it may be advisable to ask oneself the question "If I tested one sample of this material and obtained a result m, how much would I change my estimate of the expected value of the settlement?" Then use may be made of the fact that

$$m_i'' = \frac{n_i'm_i' + n_i m_i}{n_i''}$$

so that, when $n_i = 1$,

$$n_i' = \frac{m_i - m_i''}{m_i'' - m_i'}$$

Alternatively, one may use tables of the cumulative normal distribution and directly choose v_i' in answer to the question, "How probable is it that μ_i will exceed m_i' plus so many times $\sqrt{v_i'}$?" For example, the probability is 2.3 percent that $\mu_i > m_i' + 2\sqrt{v_i'}$.

Numerical example

A building 50 m tall with rectangular base measuring 10 x 20 m weighs 3400 + 70 D_f (in tons; D_f in meters). It is to be founded in a clay layer 18 m thick that weighs 1.8 ton/m^3 and has an expected mean compression ratio of 0.20. A second layer of clay 10 m thick weighing 1.65 ton/m^3 and with expected mean compression ratio of 0.15 rests on incompressible material (fig 2). The initial cost of the building is $C_1 + C_2D_f$, where C_1 = 500 k and $C_2/C_1 = 0.03$ m^{-1}. Further, $C_3/C_1 = 1.14$ m^{-1} and $C_4/C_1 = 1500$. (These are the data assumed in the numerical example by Reséndiz and Herrera, 1969, save for C_1 which did not have to be defined there.)

For the prior distribution of the mean compressions, produced by a unit set pressure applied at the foundation, Reséndiz and Herrera (1969) find $m_1' = 2.663 \times 10^{-2}$ m^3/ton and $m_2' = 0.155 \times 10^{-2}$ m^3/ton. The same reference gives quantities which it denotes by v_i', and which are equal to the area of specimens subjected to consolidation tests times the squared coefficients of variation of C_{ri}. Assuming the specimens in question to be circular in cross section, with a diameter of 1.5 in. = 3.76 cm, the squares of these coefficients of variation are

$$c_1^2 = \frac{9 \times 10^{-2}}{(\pi/4)\ 3.76^2 \times 10^{-4}}$$

$$= 81 \quad , c_1 = 9.00$$

Similarly, $c_2^2 = 18 \quad , c_2 = 4.24$

Hence, $s_1 = 0.20 \times 9.00 = 1.80$ and $s_2 = 0.15 \times 4.24 = 0.636$. Also, according to Reséndiz and Herrera, $\sigma_1 = \sqrt{F_{01}} = 2.34 \times 10^{-4}$ m^3/ton, $\sigma_2 = \sqrt{F_{02}} = 0.076 \times 10^{-4}$ m^3/ton, $\beta_1 = 0.170$ and $\beta_2 = 0.130$. It will be assumed that $n_1' = n_2' = 0.2$.

The problem will first be solved assuming that s_1 and s_2 are known. Hence so are σ_1 and σ_2.

Since $\alpha_1 = 2.663 \times 10^{-2}/0.20 = 0.1332$ m^3/ton and $\alpha_2 = 0.0103$ m^3/ton, $v_1' = 0.1332^2 \times 1.80^2/0.2 = 0.287$ m^6/ton^2 and $v_2' = 0.002$ m^2/ton^2. Then substituting in eq 14, $y' = 165$; from eq 15, $q_0' = 0.025$ ton/m^2. This stands in marked contrast with $q_0 = 9.2$ ton/m^2 obtained by Reséndiz and Herrera. The difference is entirely due to the uncertainty in the compression ratios. It is such an important difference because c_1 and c_2 are unrealistically high.

A more reasonable situation ensues from taking $c_1 = c_2 = 0.5$, say. Then $\sigma_1{}^2$ and $\sigma_2{}^2$ become negligible compared with the rest of the terms in eq 14. Under the present assumption, $s_1 = 0.1$, $s_2 = 0.075$, $v_1' = 8.89 \times 10^{-4}$ m^6/ton^2 and $v_2' = 0.01 \times 10^{-4}$ m^6/ton^2. With these values eq 14 gives $y' = 0.962$ and eq 15 gives $q_0' = 4.45$ ton/m^2, which is smaller than $q_0 > 3400/10 \times 20 = 17.0$ ton/m^2. From eq 16, $X_0' = -18.2$ k.

This solves the question of the decision to take if no sampling is contemplated. Suppose now that a = 0.3 k and b_i = 0.1 k per specimen for all i. From eq 21, almost independently of n_2, var $y'' \doteq 0.921\,[1 + 0.556\ n_1/(n_1 + 0.2)\,]\ n_1/(n_1 + 0.2)$. Evidently the optimum n_2 is zero. According to eqs 19 and 20, then,

$$G = 18.0\,[1 + 0.556\ n_1/(n_1 + 0.2)\,]\,n_1/(n_1 + 0.2) - 0.3 - 0.1\ n_1$$

This is a maximum, G = 26.0 k, for n_1 = 10. Consequently, the optimum sampling program comprises ten specimens from the first layer and none from the second.

Had the engineer been more familiar with the soil he might have chosen $n_1' = n_2' = 2$. In this case, $y' = 0.504$, $q_0' = 8.25$ ton/m^2, max G = 9.8 k, corresponding to $n_1 = 15$, $n_2 = 0$. Curiously, although n_1' and n_2' are ten times larger than in the foregoing case, the optimum sampling plan involves testing a larger number of specimens. Still, the expected benefit to be derived therefrom is less than half of that in the first case.

Both situations are displayed in fig 3.

Now c_1 and c_2 will be assumed to be known, both equal to 0.5. Then $k_1 = 4.9 \times 10^{-4}$ and $k_2 = 5.8 \times 10^{-4}$. It is found that A_1 and A_2 are negligible in comparison with C_3. Consequently, both bounds in eq 24 are practically equal to V and there is no change in the optimum exploration program.

Discussion

The weakest among the simplifying assumptions of the foregoing treatment lies in that the compression ratios are taken as uncorrelated in space. As a consequence the solution underestimates the variance of settlements and the variance of the angle of tilt. At the same time it gives no information on the advisability of drilling more than one boring at the site, which would disclose possible systematic variations in compressibility. Undoubtedly it would be of great value to develop a theory that explicitly recognizes this space correlation and gathering data to permit a realistic description of the correlation.

Other shortcomings are more easily overcome. For example, the introduction of appropriate additional random variables would take care of random differences between measured and actual in situ properties of clay, of random differences between nominal and actual imposed loads and of random influences of other phenomena, such as the procedures of excavation and construction, the effects of neighboring buildings and other nearby structures and the consequences of changes in ground water and piezometric levels.

Approximate procedures have been developed for the analysis of soil-structure interaction under sustained load taking into account creep of the structure as well as soil consolidation, assuming that both phenomena obey linear differential equations (Hell, 1969). These theories assume that properties of both the soil and the structure are known deterministically. Yet on the basis of such procedures, damage caused by angular distortions due to differential settlements could be incorporated by increasing the values of $C_4\beta_1$ appropriately.

Uncertainty about the thickness of the deposits can be dealt with in much the same manner as uncertainty about their compressibilities.

The following tentative conclusions of this study seem warranted, taking into account the improvements suggested in the last few paragraphs.

1. Even strong familiarity with soil conditions, as signified by a_1 of the order of 20, does not preclude the advisability of sampling and testing, at least for moderately heavy buildings on moderately compressible clay.

2. The optimum number of specimens to be tested from any one layer is a rapidly decreasing function of the depth of the layer and increases with the layer's expected compressibility.

3. Testing specimens from a single boring at the site of a contemplated building practically does not guard against the possibility of the building's tilting as a consequence of differential soil compressibility; several borings are required for this purpose. The justification for this statement is that uncorrelated differential compressibilities have practically no effect on tilt. Most of the phenomenon must be attributed to systematic variations of compressibility in horizontal directions.

4. The greater importance of upper, rather than lower, clay formations is more pronounced in connection with differential settlement than with average settlement.

Clearly the present state of the theory of optimum sampling does not allow a rapid, trustworthy calculation of the best sampling plan. It is useful, therefore, to keep in mind that the maximum loss incurred out of excessively extensive sampling plans is always smaller, and usually much smaller, than the direct cost of the excess in exploration and testing.

Design against shear failure

The main difference between the treatment described for design against settlements and that against shear failure lies in that the loss functions for the latter contingency can not reasonably be approximated as a quadratic function of a soil property having a smooth probability distribution. Rather, there is a negligible loss from this cause when the strength lies above some limit and a rapid increase up to a very high loss for intermediate values of the strength.

For building foundations on clay under static conditions the loading process may often be idealized as the application of sustained stress. In other foundation problems the gradual application of load may produce consolidation to the extent that the probability of shear failure becomes a decreasing function of time. This is often true in storage yards (Loasslak and Wenz, 1969) and in some dams on clay (Weigh and Hawa, 1969; Stefanoff and Zlatarev, 1969).

Were it not for consolidation one would expect that the probability of shear failure would be an increasing function of time for buildings on a wide class of soils, including most clays (see Singh and Mitchell, 1969; Vyalov and Mesohyan, 1969). Yet, the increase in strength with consolidation makes this phenomenon unimportant, at least in many practical cases (Bishop and Lovenbury, 1969; Vyalov and Meschyan describe a general procedure for taking into account changes in loads and in soil properties with time when studying the possibility of creep failure).

The increase of shearing deformations with time should not rigorously be discarded. Aside from the time-dependent deformations directly attributable to shear there is the influence of shear on consolidation (Hanrahan and Mitchell, 1969). Yet, because of the crudeness of other simplifying assumptions it seems proper to disregard the time dependence of shearing deformations, at least as a first approximation.

In view of these considerations it is normally justified to assume that shear failure either takes place on the day the building is dedicated or not at all.

If the failure surface in clay were independent of the space distribution of soil strengths and if these strengths were uncorrelated, the probability distribution of bearing capacity could be approximated as gaussian. However, among the kinematically possible failure surfaces, that giving the smallest value governs the bearing capacity. Hence, the extreme type 2 distribution could be expected to apply as a good approximation (Gumbel, 1958). Space correlation will probably lead to distributions intermediate between extreme type 2 and normal.

Under the circumstances it is premature to attempt the formulation of a usable optimization model.

Losses due to accidental loads

With loads due to earthquake or wind, uncertainty about the maximum disturbance that will take place in any moderate period of time often exceeds by orders of magnitude the uncertainty in soil properties. It is permissible, then, to take all variables other than the disturbance as deterministic and to replace them with their modes.

These same disturbences may often be idealized as Poisson processes. The natural conjugate for the prior distribution of the rate of occurrence (Raiffa and Schlaifer, 1961) is then gamma-1. This type of problem has been dealt with in structural design to resist earthquakes (Esteva, 1968; Esteva, Elorduy and Sandoval, 1969) incorporating the consequences of uncertainties in the characteristics of the disturbances and in the strength parameters.

In what follows, and merely for illustrative purposes, it will be assumed that the amount of statistical information available is sufficiently large so that the parameter in the Poisson process may be taken as deterministic. Uncertainties in resistance will be neglected. Then the probability that no events occur in interval 0,t with intensity greater than that which would cause failure (say, shear failure in clay) is $R(t) = \exp(-\lambda)$ where λ is a function of the resistence. $R(t)$ fulfills the definition given earlier for the reliability function of the system; in the present case the system comprises the building and the soil in which it is founded.

It follows from eq 4 that, if the system ceases to function when failure occurs, and considering a single mode of failure,

$$D = D_1 \int_0^{\infty} e^{-(\gamma+\lambda)t}\, dt = \frac{\lambda D_1}{\gamma + \lambda} \qquad (25)$$

where γ is the discount rate.

If the system is rebuilt systematically after failure, with no change in its design, eq 25 must be replaced with a geometric series whose sum is $\lambda D_1'/\gamma$. The difference between D_1 and D_1' is that the former includes the benefits lost because of failure while the latter includes only those lost during reconstruction or repair. Changes in design for the structure to be rebuilt tend lower the expected losses and make the series converge to a value smaller then $\lambda D_1'/\gamma$. Hence, in all cases one may write

$$D \leq \min\left(\frac{\lambda D_1}{\gamma + \lambda}, \frac{\lambda D_1''}{\gamma}\right) \qquad (26)$$

where $D_1^{*} = D_1$ but usually $D_1^{*} \doteq D_1$.

Often $\lambda << \gamma$ and so eq 26 may well be replaced with the simpler expression

$$D = \frac{\lambda D_1}{\gamma} \quad (27)$$

Failure from earthquake effects can usually be associated directly with maximum ground acceleration or velocity (Esteva, 1968). These are practically proportional to exp αM, other things being equal, where α is a constant and M is the earthquake's magnitude (a measure of its energy release). If $\lambda(M)$ stands for the rate of occurrence of earthquakes having a magnitude greater than M and originating in a given volume of the earth's crust, λ is approximately proportional to $\exp(-\beta M)$ over a wide range of values of M, where β is a constant for the volume in question and varies little over wide regions of the crust. Combining the effects of all volumes of the earth where earthquakes having an appreciable effect on the structure may originate, it follows that the design value of the maximum ground acceleration or velocity, say z, is exceeded with a rate given approximately by

$$\lambda(z) = Bz^{-\beta'} \quad (28)$$

where $\beta' = \beta\alpha$. β' varies from one part of the world to another and is larger for acceleration than for velocity. Values of the order of 2 to 3 are not uncommon.

Substituting eq 28 in eq 27 and assuming that the initial cost in the range of interest is given by

$$C = C_1 + hz^p$$

where C_1, h and p are constants, one finds

$$C + D = C_1 + hz^p + BD_1/\gamma z^{\beta'} \quad (29)$$

This may be taken as the objective function X to be minimized. Differentiating with respect to z and equation to zero one finds the optimum resistance, z_0, to be provided in design.

$$z_0 = (BD_1\beta'/\gamma hp)^{1/(p+\beta')} \quad (30)$$

and hence the corresponding recurrence period

$$T = 1/\lambda(z_0) = z_0^{\beta'}/B \quad (31)$$

Numerical example

The foundation of a tall, slender building founded in clay has already been designed for vertical loads. Preliminary calculations indicate that the foundation will require redesign -- additional piles, greater depth of excavation, a base widening -- to resist earthquakes. In this example z will be taken to stand for the base shear coefficient and will be assumed to be proportional to the maximum ground velocity. Other pertinent data are, $D_1 = 10{,}000$ k, $\gamma = 0.05\ yr^{-1}$, $B = 5 \times 10^{-5}\ yr^{-1}$, $\beta' = 2.0$, h = 1000 k, p = 2. These values of h and p are intended to include the change of the cost of the structure and foundation to resist the design value of z. The numerical values chosen are not unreasonable for a ten-story building costing about 500 k, having a rather ductile structure and founded on medium clay in a moderately seismic region.

Substitution of the data into eq 30 gives z_0 = 0.10, while eq 31 gives T = 200 yr.

The present formulation permits evaluation of the consequences of a change in each of the pertinent parameters. For example, if either the discount rate, γ, or the sensitivity, h, of the cost of the building to its capacity to withstand earthquakes is multiplied by 2, z_0 is reduced 16 percent and T is reduced 29 percent.

Discussion

It seems most desirable to set the stage for the application of decision theory to design under accidental loads. To this end it would be worth processing the data available on the capacity of model and prototype foundations tested to failure. This would provide the statistical distribution of the ratio of actual to computed bearing capacity. Thus one would have the basis for adapting a conceptual model of random behavior in order to establish the probability distribution of the bearing capacity.

The last term in eq 29 may be written $BD_1/\gamma\ \beta' z^{\beta'}$, where z now stands for the expected value of the resistance and is a random variable. Replacing the objective function with C + E D it is seen that this modification is equivalent to multiplying B by E 1/ β' in eqs 29-31. But if the probability density function of is finite in the neighborhood of zero, E 1/ β' is infinite. No design would be satisfactory under these conditions. Hence, the approximate treatment outlined above is untenable if combined with a simplistic approach to the matter of the probability distribution of the bearing capacity.

This situation arises principally from taking eq 27 instead of eq 26 and out of assuming that $\lambda(M)$ is proportional to $\exp(-\beta M)$ for all magnitudes. The latter is known not to be the case (Rosenblueth, 1969a); $\lambda(M)$ drops below the values predicted by this relation for both very large and very small magnitudes. Hence $\lambda(M)$ must be precisely defined, especially in the range of very small M. The fact remains that the probability distribution of the bearing capacity must also be carefully defined, particularly in the tail of exceptionally small values.

Concluding remarks

The present paper has touched on problems typical of building foundations in clay. Questions have been approached from the viewpoint of decision theory and regarding the combination of soil, foundation and building as a system. It has been pointed out that neighboring and nearby structures and utilities should often be considered as part of the system under study, and that some indirect recognition should also be awarded to much vaster systems which embrace the one under consideration.

An explicit solution was found for a design governed by settlement. It was possible to define the optimum sampling program. However, the solution had to be based on drastic simplifying assumptions. A more general and powerful Monte Carlo approach should be developed to permit more realistic idealizations.

The study of buildings subjected to earthquake loading indicated the need for a careful definition of the disturbances, as a stochastic process, and of the bearing capacity, as a random variable.

Despite present shortcomings of the probabilistic approach to these questions, great benefits can be envisaged from an attitute consistent with a systems approach and the application of decisions theory.

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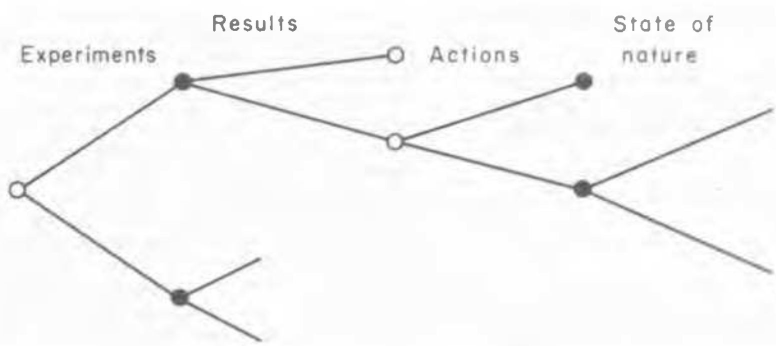

FIG 1 DECISION TREE (BASED ON RAIFFA AND SCHLAIFER, 1961)

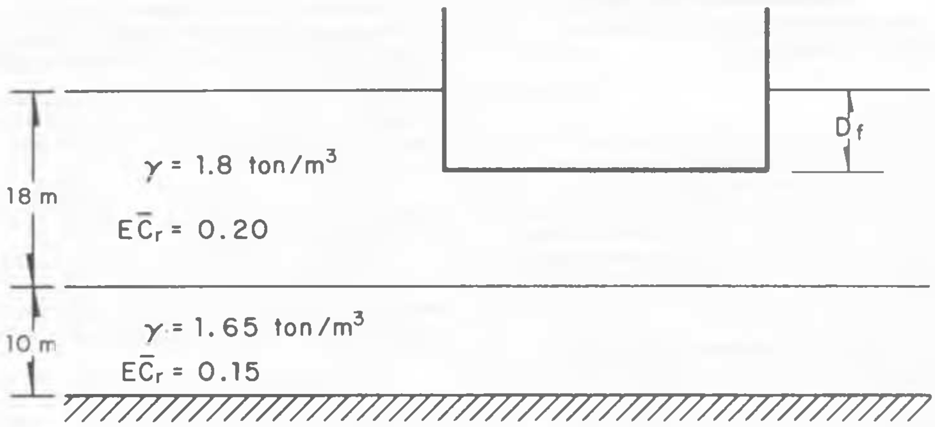

FIG 2 NUMERICAL EXAMPLE

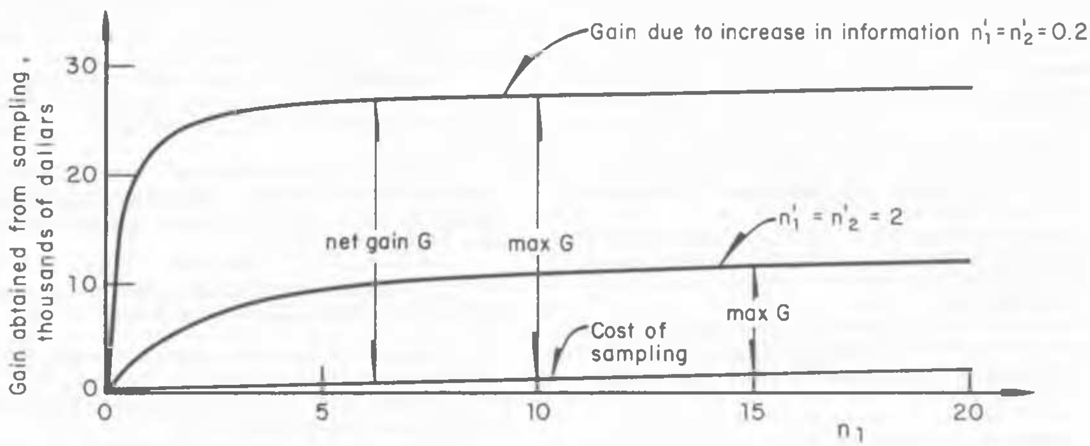

FIG 3 GAIN OBTAINED FROM SAMPLING

Chairman N. A. TSYTOVICH

Thank you very much Prof. Dr. Rosenblueth for your excellent and interesting information on the combined performance of soil and superstructures.

Ladies and gentlemen, it is time for 20 minutes intermission. It is time to receive registrations for oral participation selecting only 10 participants.

RECESS

Chairman N. A. TSYTOVICH

Since we only have one hour left for discussions or comments of questions from the floor I suggest that we restrict each commentary to not more than 5 minutes. Now I call Dr. Vesić.

A. S. VESIC (U. S. A.)

This discussion is related to topics a) and c) proposed by the General Reporter and is concerned with the ultimate point and skin resistances of piles and piers in stiff clay.

For over two decades now it has been common practice to attribute the skin resistance of all saturated clays to adhesion between clay and pile material and to compare this "adhesion" to the undrained shear strength ("cohesion") of the clay. Comparisons of this kind showed invariably that the apparent adhesion was approximately equal to the undrained shear strength of undisturbed soil as long as the latter did not exceed about 0.5 to 0.7 ton/sq.ft (soft to firm clays). There was, however, a considerable discrepancy between measured skin resistances and undrained shear strengths in the case of stiff and hard clays. This well known fact is shown in Fig. 1, which represents a summary of most of the known comparisons of this kind. Different types of piles to which test data refer are shown in this figure by different symbols explained in the legend. The source of data is marked next to each point by a letter, the explanation of which can also be found in the legend.

Many investigators of this problem have made attempts to interpret the observed trend of results, shown in the figure, by empirical formulae stating that the adhesion c_a is equal to a definite fraction of und rained shear strength c_u. Or, it was proposed generally that $c_a = \beta c_u$, where β should be a number between 0 and 1. Limiting values of β from observations on drilled piers in London clay (Skempton, 1959) are marked in Fig. 1.

In our studies of this problem (Vesić, 1967) we have come to the conclusion that there is no direct correlation between the shaft adhesion and undrained shear strength, at least for stiff to hard clays. After presenting some arguments explaining why such comparisons appear to be unreasonable, we suggested that the skin resistance f_o of deep foundations in stiff or hard clays should be compared with the frictional component of their drained shear strength and analyzed in terms of an equation $f_o = K_s\overline{q} \tan \delta$, used for piles in cohesionless soils such as sands. Since that time we have had opportunity to analyze several load test results indicating clearly this $\phi \neq 0$ behavior of piles and piers in stiff clay.

In a contribution to this Conference W. C. Sherman has reported results of load tests with instrumented piles in a stiff, tertiary clay, having an undrained shear strength of 1.6 ton/sq.ft. If we look carefully at his measured distributions of pile load along the shaft, we find clear indications of frictional character of skin resistance. With measured drained angle of shear resistance $\phi' = 22°$ and $c' = 0$, and assuming that the friction angle between the pile and the soil, δ is equal to ϕ', one finds for the coefficient of lateral soil pressure on pile skin $K_s = 2.47$. This value appears to be quite realistic for non-displacement type piles driven into a preconsolidated clay. A similar value of $K_s = 2.44$ is found by analyzing the results of tests with piles from the Bagnolet site in an Oligocene stiff clay age with undrained shear strength of about 1.0 ton/sq.ft (Kérisel, 1964). A clearly frictional behavior of this clay is again evident from increase of both point and skin resistances with depth. The recorded pile behavior indicates in this case $\phi' = 26^o$, while the drained triaxial tests on clay samples in the laboratory showed $\phi' = 22^o$.

In summary, it appears that it is not justified to compare resistance of piles and piers in stiff or hard clays with undrained shear strength of these soils. Comparisons with fractional component of drained shear strength make, generally, more sense, and give, in our experience, quite sensible results. New systematic research is, of course, needed to examine the parameters affecting skin resistance of frictional soils, particularly K_s.

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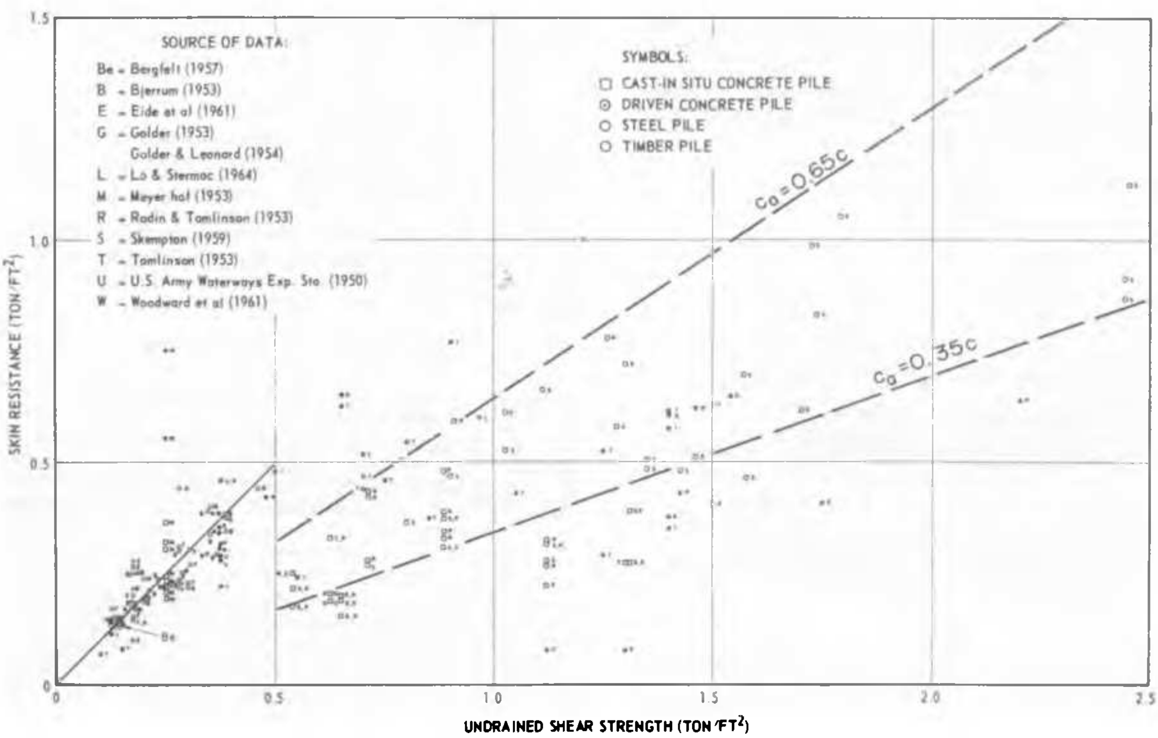

Fig. 1. Comparison of Pile Shaft Adhesion and Undrained Shear Strength of Adjacent Soil.

Experimental Facts; Proceedings, North American Conference on Deep Foundations (Congreso Sobre Cimientos Profundos), Mexico City, Vol. I, p. 5-44.

Skempton, A. W., 1959: Cast-in-situ Bored Piles in London Clay; Géotechnique IX, pp. 153-173.

Vesić, A. S. 1967: A Study of Bearing Capacity of Deep Foundations, Final Report, Project B-189, Georgia Institute of Technology, Atlanta, Georgia, pp. xvi + 264.

General Reporter V. F. B. de MELLO

I call upon our Chairman Prof. Tsytovich to present us his discussion using the same prerogative of limiting himself to 5 minutes.

N.A. TSYTOVICH, Z.G. TER-MARTIROSYAN, N.M. DOROSHKEVICH and A. JUMADILOVA (U.S.S.R.)

The experimental investigations on normal stress relaxation in clayey soils surrounding piles were conducted on a special rig (Fig. 1). Total stress gauges (6) and pore pressure gauges (7) were put inside the testing tank which was filled with the soil mass. The pile model was driven into the soil by means of a jack, and the total stresses in the soil and the pressure of the pore liquid were measured, both at the soil-pile contact surface and at various distances from the pile.

The experiment was conducted under conditions of a plane stress state and plane deformation. For this purpose, the rubber cushion (bag) (5) was filled with compressed air or water, and was used to apply the external pressure to the surface of the clay.

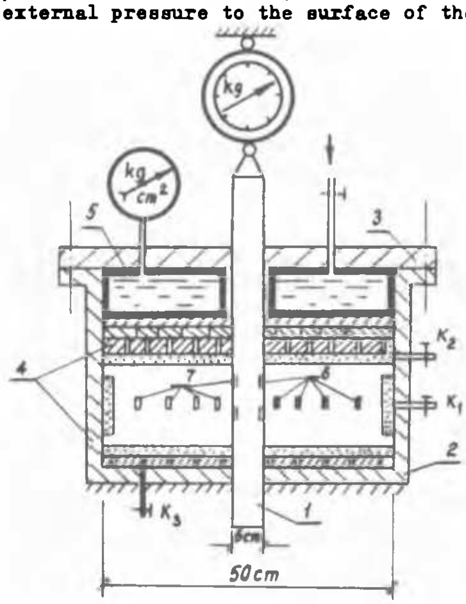

Fig. 1 - Experimental rig: 1 - pile; 2 - testing tank; 3 - thrust plate; 4 - filtering plates; 5 - rubber bag; 6 - total stress gauges; 7 - pore pressure gauges.

The curves in Fig. 2 show the variation in the total, pore and effective radial pressures with time at the soil-pile contact surface, expressed as a fraction of the initial pressure (curves 1', 2' and 3').
The effective stresses and pressures of the pore water, as is evident from the curves, have extreme variations of signs, and the total stress is relaxed immediately after driving the pile to the stabilized value. This effect, apparently, can be explained by the simultaneous progress of the filtration and relaxation processes. In the initial period of time the filtration processes have not had enough time to cover the whole volume of soil being tested and purely relaxational processes are predominant. This leads to an increase in pore pressure. Then, as the filtration processes continue to spread, a stage develops in which the pore pressure drops. This stage is predominant up to the end of the process. The nature of the variation in effective stresses compliee well with the known fact, occurring in pile driving, that the bearing capacity is lower during the initial period and gradually increases with time.

The analytical investigations given below are based on physical prerequisites that follow from the experiment. It is assumed that the clayey soil can be conceived as an elastic-creeping medium filled with a compressible liquid. The rheological equation of state of the soil is presented in the form of an hereditary creep equation (N. A. Tsytovich, Z. G. Ter-Martirosyan, et al, 1967).

$$\varepsilon(\tau_1)-\varepsilon(t)=a(t,\tau)\frac{\Theta'(t)}{1+\xi}-\int_{\tau_1}^{t}\frac{\Theta'(\tau)}{1+\xi}\frac{\partial}{\partial\tau}a(t,\tau)\,d\tau \quad \ldots(1)$$

where $\varepsilon(\tau_1)$ and $\varepsilon(t)$ are the initial and time-varying void ratios, respectively; $\Theta'(t)$ is the time-varying sum of the principal stresses and ξ is the coefficient of lateral pressure

$$a(t,\tau)=a_m+a_1\left[1-e^{-\eta(t-\tau)}\right] \qquad (2)$$

where a_m and a_1 are the coefficients of instantaneous and long-term compaction, and η is the coefficient of creep damping.

The pore fluid (water, air bubbles and dissolved air) obeys the linear law of compression with a coefficient of volume change a_w equal to (Ter-Martirosyan and Tsytovitch, 1965):

$$a_w=(1-J_w)/Pa \qquad (3)$$

where J_w is the coefficient of saturation and Pa is the atmospheric pressure.
Let us consider the stress-strain state of an unbounded hollow soil cylinder with an inside diameter $2r_0$ and outside diameter $2R_2$, into which an absolutely rigid watertight cylinder of a diameter $2R_1$ is forced. If it is assumed that there is free filtration on the external surface of the soil cylinder and conditions of rigid contact are complied with on the internal surface, i.e. continuity of displacement, we will have a complex filtration-relaxation problem. This will require the simultaneous solution (1) and the differential equation of axisymmetric soil consolidation (Tsytovich, Ter-Martirosyan, et al, 1967).

$$\frac{\partial\varepsilon}{\partial t}+a_w\bar{\varepsilon}\cdot\frac{\partial P_w}{\partial t}=\frac{1+\bar{\varepsilon}}{\gamma_w}\cdot k\left(\frac{\partial^2 P_w}{\partial r^2}+\frac{1}{r}\cdot\frac{\partial P_w}{\partial r}\right) \qquad (4)$$

taking into consideration the following filtration and relaxation boundary conditions

$$\frac{\partial Pw(R_1,t)}{\partial r} = 0; \qquad u(R_1,t) = u_m \tag{5}$$

$$Pw(R_2,t) = 0; \qquad u(R_2,t) = 0$$

where R_1 is the radius of the pile and R_2 is the radius of influence;

and the equilibrium equation

$$\Theta(t) = \Theta'(t) + 3\,Pw(t) \tag{6}$$

The following procedure was accepted for solving the foregoing system of integro-differential equations: first the axisymmetrical plane problem of filtration consolidation was solved (Tsytovich, Zaretsky, et al, 1967) under the assumption that a time-constant load acts on the soil-pile contact surface. Then, on the basis of this solution, an integral equation of the contact stresses was set up to establish the law of stress variation with time. This equation has to comply with the condition of constant initial displacements at the soil-pile contact surface (relaxation condition).

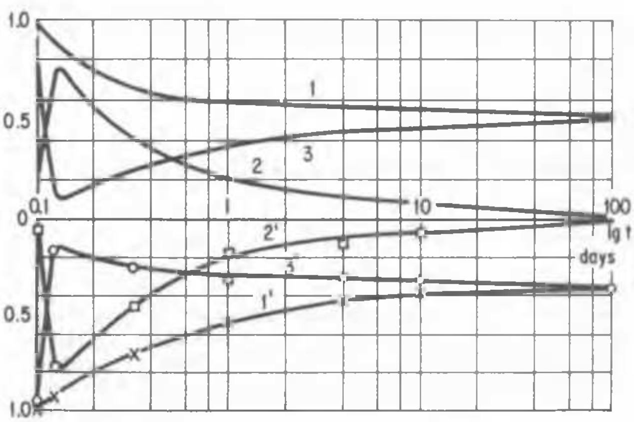

Fig. 2. Experimental curves (1', 2' and 3') and theoretical curves (1, 2 and 3) showing the variation in time of the total, effective and pore pressures around the pile.

In its final form, the expression for determining the time-varying total stress (pressure) is:

$$q(t) = q(\tau_1)\sum_{i=1}^{\infty}\left(C_{1i} + C_{2i}e^{-\alpha_i t} + C_{3i}e^{-\alpha_{2i} t} + C_{4i}e^{-\alpha_{2i} t}\right) \tag{7}$$

where $q(\tau_1)$ is the initial value of the total stress, determined by the solution from the linear theory of creep (Bezukhov, 1961) and the pore pressure is

$$Pw(t) = -\frac{A(1-\mu)\,R_1^2\cdot q(\tau_1)}{2R_1^2\left[1-2A(1+\mu)\right] + (1-2\mu)R_2^2}\sum_{i=1}^{\infty}L_i V_o(X_i, r/R)\,\times$$

$$\times\left(D_{1i}e^{-\beta_i t} - D_{2i}e^{-\beta_i t} + D_{3i}e^{-\alpha_i t} + D_{4i}e^{-\alpha_{2i} t} + D_{5i}e^{-\alpha_{3i} t}\right) \tag{8}$$

and

$$A = \frac{a_m}{(1+\xi)\,\mathcal{E}(\tau_1)\,a_w}$$

where Vo is the combination of Bessel functions:

$$V_o(X_i, r/R_2) = J_o(X_i, r/R_2)Y_o(X_i) - Y_o(X_i, r/R_2)J_o(X_i) \tag{9}$$

where Xi are the roots of characteristic equation (10).

$$J_1(X_i, R_1/R_2)Y_o(X_i) - Y_1(X_i, R_1/R_2)J_o(X_i) \tag{10}$$

where C_i, D_i, L_i, α_i and β_i are constant values depending upon the soil properties and the geometrical parameters of the problem. They are determined by complex expressions (Jumadilova, Ter-Martirosyan, 1969)* in accordance with the following coefficients: the instantaneous (a_m) and long-term (a_1) compaction, creep damping η, water permeability k, initial porosity $\mathcal{E}(\tau_1)$, average porosity $\bar{\mathcal{E}}$, radius of pile R_1 and radius of influence R_2.

The derived expressions were used to calculate an example with the following parameters:

$a_m = 0.004\ cm^2/kg$; $a_1 = 0.02\ cm^2/kg$;

$\eta = 2\times10^{-3}\frac{1}{min}$; $k = 2\times10^{-7}$ cm/min;

$a_w = 0.003\ cm^2/kg$; $\xi = 0.56$; $\mathcal{E}(\tau_1) = 1.2$;

$\bar{\mathcal{E}} = 1.0$; $R_1 = 15$ cm and $R_2 = 150$ cm.

The results of these calculations are given in Fig. 2 (curves 1, 2 and 3). This confirms the similar character of the experimental (1', 2' and 3') and theoretical (1, 2 and 3) data.

ACKNOWLEDGEMENTS

Investigations on stress relaxation in clayey soils surrounding driving piles were conducted by members of the Department of Soil Mechanics, Bases and Foundations of the Kuibyshev Civil Engineering Institute in Moscow under the supervision of Prof. N. A. Tsytovich. The experimental work was carried out by postgraduate A. Jumadilova in conjunction with Associate Professors N. M Doroshkevich and Z. G. Ter-Martirosyan; the mathematical analysis was carried out by Associate Professor Z. G. Ter-Martirosyan and postgraduate A. Jumadilova.

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Chairman N. A. TSYTOVICH

Now I Call Mr. Milović

D. MILOVIC (Canada)

I should like to refer to the paper of K. F. Egorov and I. A. Simvulidi: "Calculation of Footings on Compressible Foundation Beds", Vol. II, pp. 77-84.

The authors have shown the theoretical solution for the calculation of vertical stresses and displacements produced by a rigid circular foundation in an elastic isotropic layer of limited thickness, underlain by a rigid base. Solution is obtained by means of the Bessel functions and second power Fredholm integral equations and based on a no friction condition existing between the soil layer and the rigid base.

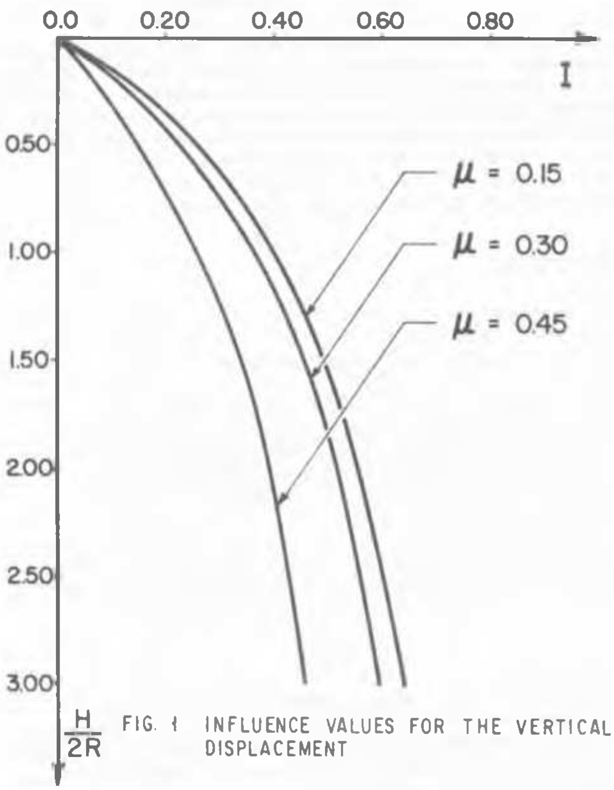

FIG. 1 INFLUENCE VALUES FOR THE VERTICAL DISPLACEMENT

In the practice the contact between the elastic layer and rigid base is very often rough and I should like to show some of the results which I have obtained for no horizontal displacements condition.

Fig.1 shows the influence coefficients for the vertical displacement of the surface due to vertically loaded rigid circular foundation resting on the elastic layer of finite thickness, underlain by a rough rigid base. The results have been obtained by the finite element method.

As it has been expected, the influence values for the vertical displacements in the case of rough rigid base are smaller than those shown by Egorov and Simvulidi for the smooth base.

Chairman N. A. TSYTOVICH

Please, Dr. Moretto read your contribution.

O. MORETTO (Argentina)

The use of bearing capacity formulas in engineering practice proves frequently a problem of judgment that deserves some attention. Whenever foundations involve soils that undergo small deformations, so that settlement under the working load will be within the limits that are known tolerable for the structures, designs are made on the basis of the failure load computed with a bearing capacity formula affected by an adequate factor of safety. By far, in number, the great majority of foundations are presently designed, either consciously or unconsciously, on this basis.

In applying a bearing capacity formula, the problem arises as to which are the suitable values of c and ∅ to be used. For clay soils, the most ready solution and generally the safest , though not always the most representative, is obtained resorting to the undrained values c_u and $\emptyset_u$. However, in unsaturated clays and some saturated soils, during the construction period and even during the time required to reach the maximun live load by occupancy, a significant amount of consolidation may take place which, unless the soil is highly dilatant, implies that the zone of the subsoil that is being compressed by the load increase will react under partially drained conditions.

To obtain an idea of what this partial drainage may mean for the shear strength of the soil, triaxial

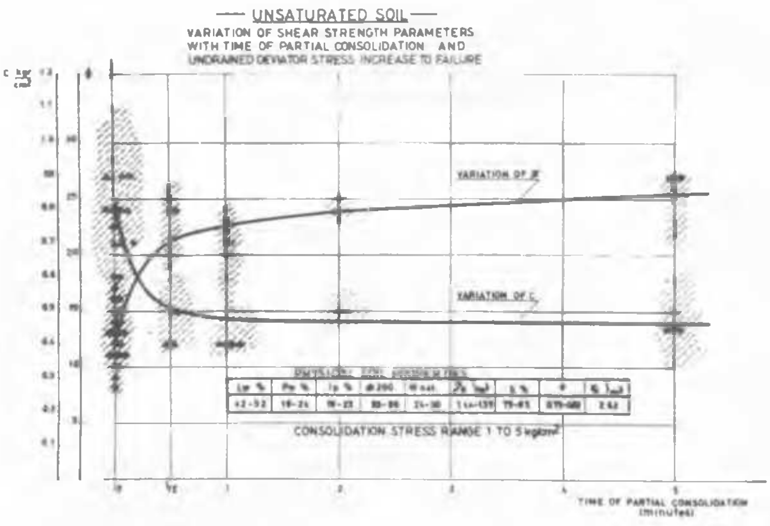

Fig. 1 Variation of the apparent cohesion and angle of internal friction with partial drainage for an unsaturated soil.

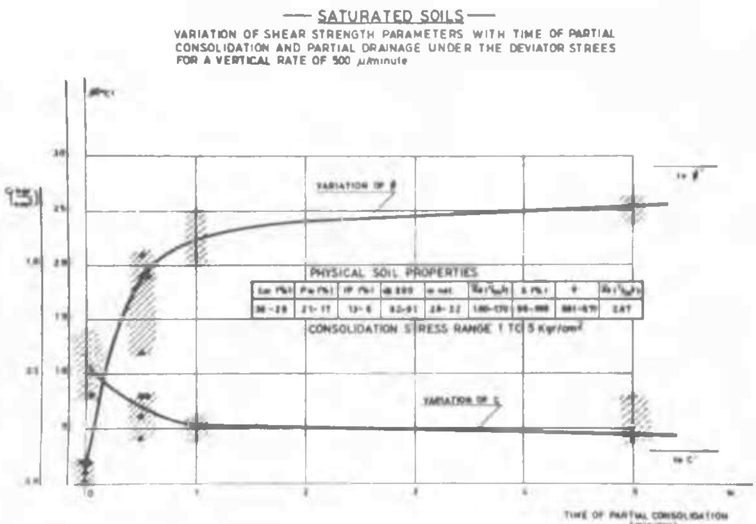

Fig. 2 Variation of the apparent cohesion and angle of internal friction with partial drainage for a saturated silty clay.

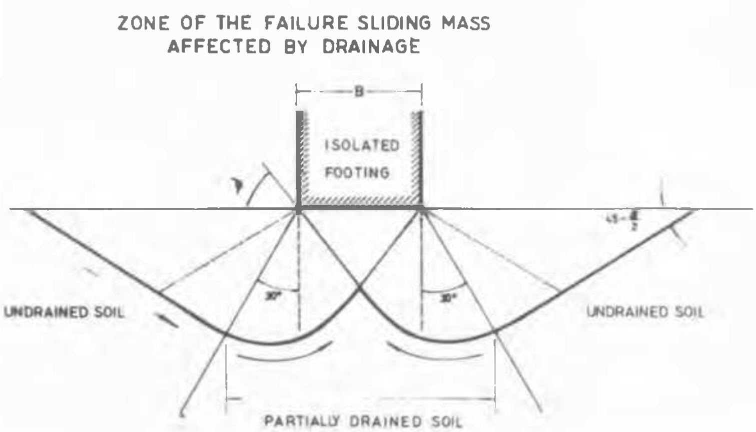

Fig. 3 Simplified assumption for determining bearing capacity of partially drained soils.

tests have been run in which samples 5 cm in diameter and 12.5 cm high were subjected to the following routine:

1) No drainage allowed (Q tests)

2) Under the all around consolidation pressure, the samples were permitted to drain unidimensionally at top and bottom for periods of 1/2, 1, 2 and 5 minutes and then they were sheared increasing the deviator stress under undrained conditions.

3)Under the all around consolidation pressure, the samples were allowed to drain unidimensionally at top and bottom for periods of 1/2, 1, 2, and 5 minutes and then they were sheared increasing the deviator stress at a rate proportional to the expected or assumed velocity of loading in the field.

For a clay soil with a degree of saturation varying between 75% and 95%, Fig. 1 shows how both the angle of internal friction and the cohesion vary when drainage is allowed only under the all around compressive stress for time periods up to 5 minutes. It may be seen that while the angle of internal friction increases very rapidly, the apparent cohesion decreases at a much slower rate. Fig. 2 refers to tests in which, after partial drainage under the all around compressive stress for time periods up to 5 minutes, the deviator stress was increased at a rate of 500 microns per minute.The soil tested is a saturated silty clay.

Partial drainage produces a substancial increase in soil resistance. Even if it is theoretically possible that the partially drained strength of dilatant clays may be smaller than the undrained one, no case has been encountered as yet where such a decrease of stress takes place within the sequence of drainage periods indicated above.
Consolidation drains only that portion of the sliding mass of soil involved in failure that is located within the pressure bulb developed below the footing. Assuming that, for isolated footings, this mass extends downward fanning from the edge of the shape of the failure surface is not changed and remains equal to that developing in a mass with uniform strength, as a first approximation, it may be assumed that the unit bearing capacity to be considered in design may be taken as equal to:

$$q_r = \alpha c_R + (1 - \alpha) q_u$$

in which q_R is calculated bearing capacity obtained using the shear parameters of the partially drained soils, q_u is the computed bearing capacity of the undrained material. The value of α has to be estimated from an analysis of the relative magnitude of the reaction that develops along the surface of sliding. For preliminary purposes in practice it has been taken as $\alpha = 0.5$.

Chairman N. A. TSYTOVICH

Thank you for your interesting remarks. Now Mr. Burland, please.

J. B. BURLAND (England)

I would like to present some observations of the consolidation settlement of model strip footings on deep beds of homogeneous normally consolidated clay and to compare these observations with theoretical predictions. The apparatus used for the tests has been described in detail by Burland and Roscoe (1969).

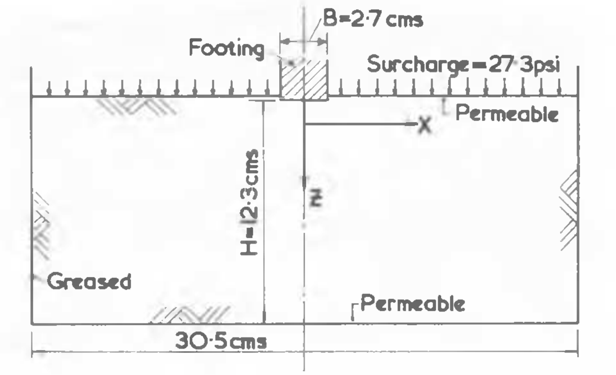

Fig. 1. Details of typical footing test.

The overall geometry of a typical test is shown in Fig. 1. The tests were carried out under plane strain conditions in a steel framed, glass-sided box with all its surfaces heavily greased. The displacement of a large number of points within the clay bed were observed by means of X-ray and lead shot markers. The clay was placed in the apparatus at twice its liquid limit and then consolidated one-dimensionally. The relevant properties of the clay (Spestone Kaolin) are: $C_c = 0.715$; $\phi' = 23°$ and $C_u / {}_v' = 0.25$ (from simple shear tests). The model footing consisted of a rigis strip resting on the upper surface of the clay bed which was acted upon by a uniform surcharge pressure applied through a rubber membrane. The top and bottom surfaces of the clay layer were free-draining. The model fotting was loaded by means of a hydraulic jack and loading was carried out in three increments; the time between each increment being sufficient to allow full primary consolidation.

In Fig. 2 are plotted the observed relationships between net average footing pressure and consolidation settlement (expressed as a proportion of the breadth B of the footing) for two tests. It should be noted that the pressure increments were small, being approximately 1/3 and 1/6 of the undrained bearing capacity for tests A and B respectively. For both tests the immediate settlements were too small to be detected.

The chain-dotted lines in Fig. 2 represent predictions of consolidation settlement made by means of the simple oedometer method using

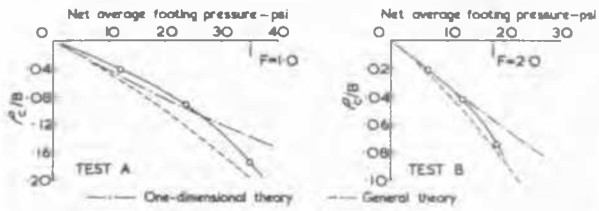

Fig. 2 Observed and predicted relationships between net footing pressure and consolidation settlement for two model footing tests.

Test details:

Test A -	Surcharge	=	27.3 psi
	B	=	2.7 cm
	H	=	12.3 cm
Test B -	Surcharge	=	28.3 psi
	B	=	5.08 cm
	H	=	14.0 cm

the value C_c quoted previously. The dotted lines represent predictions made by means of the general stress-strain theory for soft clay which was outlined in my paper to Session 1 (Burland 1969) and given in detail by Roscoe and Burland (1968). This latter theory only requires a knowledge of C_c and $\emptyset'$. It can be seen from Fig. 2 that the simple classical one-dimensional method and the more rigorous analysis give very similar predictions which are in good agreement with the observations.

Fig. 3 shows the observed vertical displacements (expressed as a proportion of the settlement of the footing) for test A beneath and outside the footing for various values of Z/B. These observations are compared with the predictions of the simple one-dimensional theory making use of the vertical elastic stress distribution. The theoretical curves (shown chain dotted) have been fitted to the observed displacements at the centre line. The theoretical settlement profiles can be seen to be in good agreement with the observations.

The open triangles in Fig. 3 represent the observed cumulative horizontal displacements at the end of the third increment of footing pressure, i.e. at a pressure corresponding to the undrained bearing capacity. Even at such high pressures the lateral displacements do not exceed 10 per cent of the setlement of the footing. At lower footing pressures the ratio between maximum horizontal displacement and settlement is even smaller so that the drained deformations are nearly one-dimensional.

In Fig. 4 (a) are plotted the vertical displacements at various depths beneath the centre of the footing for test A. For

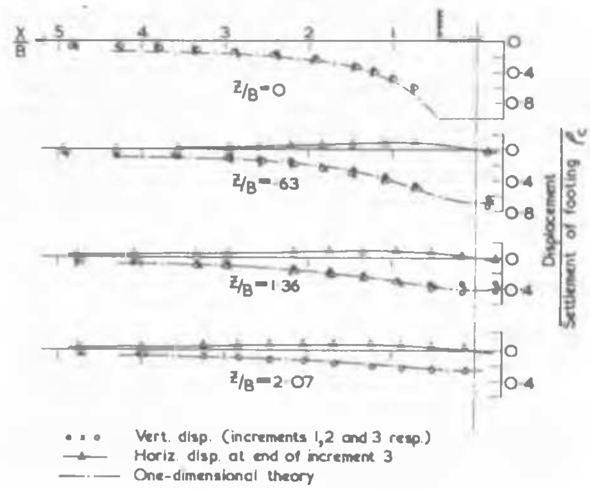

Fig. 3 Model footing test A. Observed cumulative vertical and horizontal displacement, expressed non-dimensionally, for various values of X/B and Z/B.

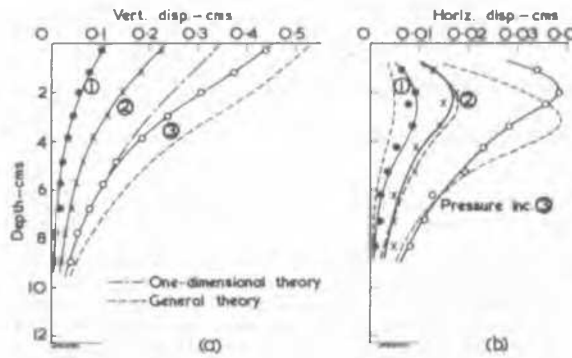

Fig. 4 Results of model footing test A showing relationships between depths and
a) cumulative vertical displacement beneath the centre, and
b) cumulative horizontal displacements beneath the edge of the footing.

clarity, comparison with the predictions of the two theories is confined to the third pressure increment. The horizontal displacements beneath the edge of the footing are plotted in Fig. 4(b) (note the large horizontal scale) where they are compared with predictions using the general stress-strain theory. (The simple oedometer method cannot, of course be used to predict horizontal displacements.) In spite of the small magnitude of the horizontal displacements the predictions agree very well with the observations.

In summary the results of the model footing tests show that the drained deformations beneath a footing on normally consolidated clay are cloely one-dimensional. The accuracy of the classical oedometer method for predicting consolidation settlements for normally consolidated clays has been confirmed even when the clay layer is very thick. The more rigorous method proposed by Burland (1969) has been shown to give good agreement not only with the measured vertical displacements but also with the horizontal displacements. The method is based on a general stress-strain theory for soft clay which only requires a knowledge of C_c and $\emptyset$ (Roscoe and Burland 1968.)

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Chairman N. A. TSYTOVICH

Now Mr. Schmertmann

J. H. SCHERTMANN (U.S.A.)

This discussion pertains to only one small portion of Dr. de Mello's broad and excellent state-of-art treatment. Mr. Pérez Guerra also noted this problem. It is the problem of determining the constant N_{cp} in the formula $R_p = N_{cp}c$, which is the equation relating static cone bearing and undrained shear strength in purely cohesive soil.

As Dr. de Mello notes on p.84 of Volume I, the correlation data are few and confusing. N_{cp} appears to vary from approximately 8 to 30.

Consider a cone penetrating a clay. A cylindrical hole remains after penetration. It would appear that the elastic-plastic theory for the radial expansion of an infinite cylinder might be of some use. In this connection, there exists a remarkable paper by Bishop, Hill and Mott (1945) in which these authors investigate the use of static cone punches to determine the strength characteristics of ductile metals. They demonstrated in a convincing way that cavity expansion theory could explain the meaning of their cone-punch penetration tests in terms of the elastic-plastic properties of the ductile metals they tested. In soil mechanics terminology, their important conclusion was that the cone penetration resistance depends not only on the undrained shear strength c, but it also depends on the ratio of modulus to strength, E/c.

The University of Florida conducted a preliminary experimental program to check this theory in clay. Pressuremeter tests, similar in principle to Ménard's, but using equipment designed by GeoProbe of Quebec, provided independent measurements of E and c. These tests were made after enlarging, by auger, a 1.4 in. diameter cone hole to about the 2.7 in. initial diameter required for our pressuremeter tests. The previous cone bearing values, R_p, could then be compared with the pressuremeter data to obtain N_{cp}. Fig.1 shows a clear trend despite scatter — N_{cp} increases significantly with the ratio E/c. The cavity expansion theory also predicts such a trend. The pressuremeter determined shear strength is noted next to each point. Note also that the N_{cp} range in this figure, namely 8 to 25, is about the same as noted by Dr. de Mello and Mr. Pérez Guerra.

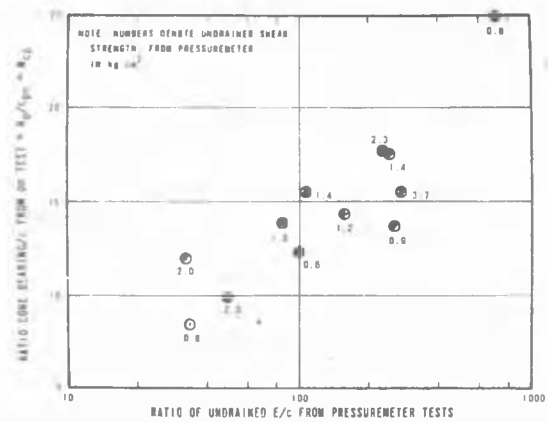

FIGURE 1. DEPENDENCE OF N_{cp} ON THE RATIO E/c IN SOME GAINESVILLE CLAYS

What an engineer needs to determine N_{cp} more accurately from only cone data is some way to sense the two unknowns, E and c. This requires two independent measurements of a clay's strength characteristics. The ordinary static cone provides only one, cone bearing. However, the recent development of the friction-cone, with its ability to measure also local adhesion along a small length of rod just above the cone point, provides the two independent measurements required — at least in principle. The ratio of local friction to end bearing, called the friction ratio, can be shown mathematically to relate to E/c for the case of insensitive clays. The practical result is that for insensitive clays the friction ratio provides an index to the value of the "constant" N_{cp}. Figure 2 presents our data on this point, plus some data extracted from Begemann (1965). We have plenty of scatter but perhaps the trend is encouraging.

Although our results are preliminary, both theoretical and experimental investigations suggest that at least some of the observed scatter in the N_{cp} values can be explained by the previous failure to consider the importance of a deformation characteristic, such as E, in this relationship.

ACKNOWLEDGEMENTS

Mr. Zaid al Awqati (1969) contributed the above

approach to this problem while a graduate student at the University of Florida. Assistant Professor Ronald E. Smith chaired his supervisory committee.

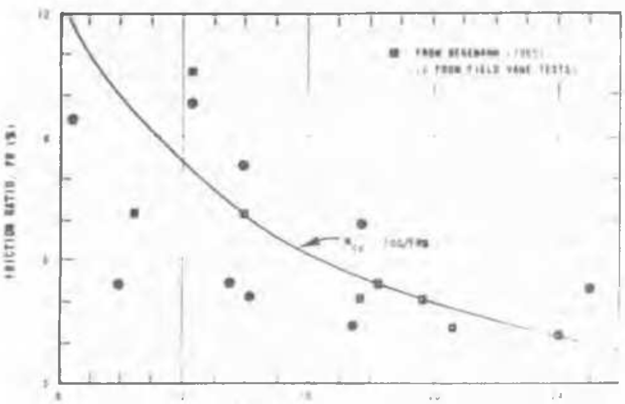

FIGURE 2. DATA FROM GAINESVILLE AND DUTCH CLAYS SHOWING THAT FRICTION RATIO MAY BE AN INDICATOR OF N_{cp} after [illegible]

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Chairman N. A. TSYTOVICH

Now Mr. Bakholdin, will read his discussion

B. V. BAKHOLDIN (U.S.S.R.)

The new method of dynamic tests of piles has been worked out at the Research Institute of Foundations and Underground Structures of the Gosstroy. This method was based on the results of field tests. These tests were made on clayey soils, which had the consistency from liquid to semi-solid.

In order to obtain the values of stresses inside a pile and the displacements of the pile during its driving, reinforced concrete test piles were equipped with strain gauges and displacement gauges. Wire strain gauges were glued on steel rods, that were attached to the reinforcement inside a pile. The displacement gauge was made as a rheostat and belonged to the potentiometer type. Its body was installed on an immobile steel frame and its mobile part was stiffly fixed on the pile.

Oscillograms of stresses and displacements, which were obtained during driving the piles 6 m long and with cross-section 30 x 30 cm into stiff clays, are shown on Fig. 1. The curve 1 shows displacement of the pile, curves 2 and 5 show the stresses in the pile near its point and its head respectively and curves 3 and 4 show the stresses in the pile near half and two-thirds of its length respectively.

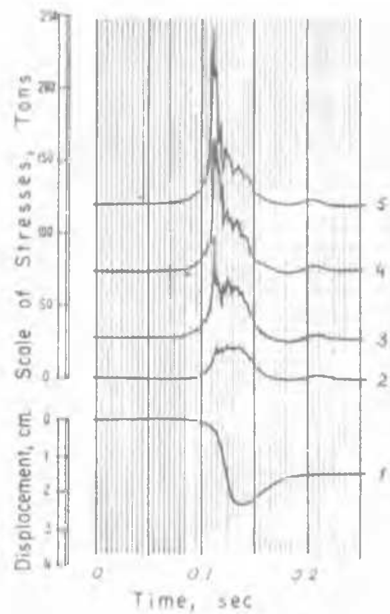


Fig 1. - Oscillograms of stresses in a pile and its displacements at the time of driving.

It can be seen from the given oscillograms, that the character of pile displacement under a blow is imperiodical, but not the oscillatory one, as it had been supposed earlier. We think that this fact is caused by high skin friction and by the influence of elastic properties of the pile itself and of the wooden pad of pile cap. So, the results of field tests show that the existing solutions based on a theory of reduction of periodic oscillations, are unsuitable for an estimation of pile driving parameters.

The oscillograms of stresses in a pile during driving have the peaks at the beginning of driving, especially in sections near the pile head. These peaks are caused by the inertia of the pile mass. Generally the recorded stresses in a pile are not the stresses caused only by soil resistance. Actual soil resistance may be found with a gauge installed near the pile lower end.

The variation of point resistance for one of the test piles during a blow, when a hammer was dropped from different heights, is shown on Fig. 2. Test pile 6 m long and with cross-section 30 x 30 cm with a strain gauge near its point passed the soft-plastic clays and at a depth of 5,15 m pile point was embedded into the layer of stiff-plastic loam. To drive the pile, bar diesel hammer having ram weight of 1,8 tons was used. Oscillograms of soil resistance (curves 1), of pile displacements (curves 2) and the curves that show relation between soil resistance and pile displacements are denoted on Fig.2 with letters a, b, c and d,respectively. These curves are given for the cases, when the hammer dropped from the height of 0,8, 1,08, 1,4 and 1,89 m correspondingly.

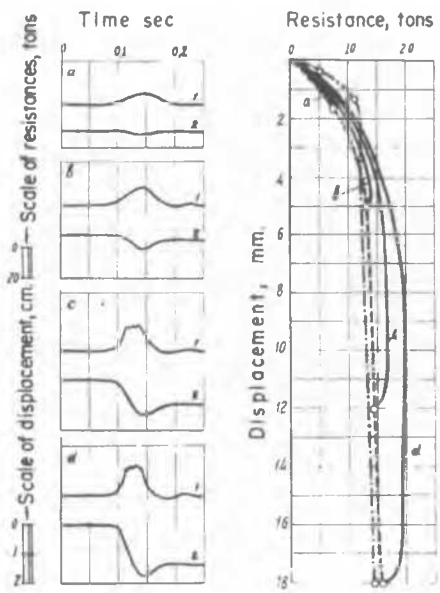


Fig. 2. - Soil resistance during pile driving.

As it is seen from the figures, the more is the height of hammer drop, the higher is a maximum value of point resistance. By the way the moment of getting up the highest point resistance does not coincide with that, when the maximum displacement takes place. It can be explained by the well known fact that when the rate of displacement is high, a viscous resistance of soil is considerable.

To make clear the behaviour of soil under load, it is necessary to estimate the values of soil resistance at the moment when the displacement of pile downwards changes into elastic upheave. At this moment the speed of pile displacement is equal to zero and the resistance of the soil from a physical point of view is close to the static one.

As it is shown on Fig. 2 with dotted line, the point resistance depends upon pile displacements, when their rate is equal to zero. It is possible to examine the mentioned diagram as that of momentary settlements of soil under imaginary static load (which does not cause a viscous resistance). A similar diagram, which was obtained from the strain gauge near the pile tip at the time of pile load test, is shown on the same figure with a touch-dotted line, Comparing these two diagrams we see that they are similar, especially when the soil resistance is ultimate. Because of good coincidence of load-displacement diagrams under static and dynamic loads, there was suggested a new method of dynamic tests of piles with the use of strain gauges.

According to this method the bearing capacity of piles can be determined from the results of measured stresses on the contact surface between hammer and pile under a blow. We assume that to the bearing capacity of pile corresponds an elastic limit of pile displacements. This value can be obtained from the oscillograms of stresses and displacements and corresponds to the contact stresses at the moment, when a speed of pile displacement is equal to zero.

The relation between contact stresses and elastic limit of pile displacement can be defined by the formula:

$$p_z = p_k \left(1 + \frac{q}{Q}\right),$$

where: p_z - elastic limit of pile displacement

p_k - contact stress at the moment when the displacement of pile downwards changes into elastic upheave

q - weight of pile

Q - weight of hammer and pile cap.

To measure the contact stresses, we use a strain gauge meter, which is installed into the cap of the hammer. At the same time the displacement of the pile is measured with the potentiometer displacement gauge. Recording of stresses and displacements is made on one photo-film by means of an oscillograph or other rapid self-recorder.

For approximate designs the bearing capacity of the pile can be defined according to the given formula using the oscillograms of stresses and displacements for the period of one blow. If it is necessary to have the exact data and to obtain the general character of load-settlement diagram, we do some dynamic tests of piles with the different heights from which hammer is dropped. The stresses into the pile head and displacements of a pile are recorded. From these oscillograms using the above-mentioned formula in the cases of different displacements the elastic limit of pile settlement can be estimated. Also the stress-displacement diagram may be made that allows us to find out the bearing capacity of a pile without static load tests.

Chairman N. A. TSYTOVICH

Thank you Mr. Bakholding. Now please Prof. Kérisel.

J. KERISEL (France)

Je voudrais présenter quelques remarques sur le calcul de la pression limite à court terme: il est admis que celle-ci est égale au produit du coefficient N_c par c_u, cohésion non drainée. La seconde figure présentée par le Prof. de Mello, montre une grande dispersion de c_u suivant la méthode: scissomètre, compression simple, UU ou CU triaxial. Je crois que cette dispersion est encore plus grande pour les argiles molles où j'ai certaines évidences de valeurs de c_u au scissomètre doubles de celles données par le triaxial.

Mais la question pour moi est de savoir si, partant d'une valeur donnée c_u dans une argile homogène, N_c varie lorsqu'on fait varier les dimensions de la fondation. La réponse est oui, pour les argiles tout comme pour le sable. N_c diminue lorsque la dimension augmente et il est donc imprudent d'utiliser pour de larges fondations les résultats de pénétromètres.

Ceci a été montré pour les fondations superficielles par le Prof. de Beer, par Tcheng et d'autres, par le Prof. Vesić, M.Tcheng et moi-même pour les fondations profondes dans le sable, et enfin par Adam et moi-même pour des fondations profondes d'argile de Bagnolet: N_c diminue de 2.5 à 1 lorsque la dimension augmente de 10 cm à 100 cm, bien entendu en se référant toujours au même type d'essais pour c_u; il s'agissait d'une argile $c_u = 1.2$ kg/cm^2. Le laboratoire des Ponts et Chaussées fait actuellement avec le même équipement des mesures dans les argiles molles où il est probable que, tout comme dans les sables non serrés, il n'y a pas d'effet d'échelle.

Ceci serait de nature à nous faire penser que pour de larges fondations construites sur des argiles molles, on sait calculer tassements et pressions limites. Le Prof. Bjerrum pense qu'on sait le faire bien qu'il admette que, par ailleurs, les tassements des fondations d'immeubles sur argiles molles sont plusieurs fois ceux calculés. Je ne partage pas son optimisme; des observations sur un barrage très large à la base (200m) sur argiles molles aussi bien que des observations sur des remblais sur sols mous, mettent en évidence des tassements plusieurs fois supérieurs à ceux de la prévision exactement comme pour les immeubles de Norvège cités par le Prof.Bjerrum.

Je ne crois pas que ceci soit à relier à un effet d'échelle comme pour les argiles consistantes. Mais ceci nous montre que, aussi bien pour les argiles consistantes que pour les argiles molles, en ce qui concerne la prévision de la capacité portante et celle des tassements, il reste encore beaucoup à faire en mécanique des sols.

Chairman N. A. TSYTOVICH

Thank you very much Prof. Kérisel for your very interesting remarks on pile foundations. Now Mr. Van Wambeke.

V.E.A. VAN WAMBEKE (Belgique)

Mon intervention m'est suggérée par les discussions de ce matin.

Je désire vous faire part de l'expérience personnelle acquise notamment au cours des travaux de recherche actuellement en cours en Belgique.

Je parlerai de deux choses : l'estimation et l'observation des tassements d'une part, la comparaison des résultats d'essai in situ d'autre part.

Pour les tassements, il y a trois points à considérer : les formules théoriques, l'investigation et le contrôle.
Il me semble, en premier lieu, que l'ordre d'importance qu'il convient de leur accorder est inverse de celui dans lequel je les ai énumérés et qui est habituellement adopté. Le contrôle et l'observation me semblent prépondérants et une bonne investigation est plus importante que la recherche d'une formule théoriquement exacte. Les formules les plus simples seront toujours les meilleures. Quant à l'investigation, elle doit être plus poussée qu'elle ne l'est habituellement : le nombre d'essais doit être plus élevé et les types de méthode d'investigation - judicieusement choisis - doivent être multiples.

Le contrôle, habituellement inexistant ou trop sommaire, conduit à des observations insuffisantes ou insuffisamment communiquées.

La double nécessité d'une investigation et d'un contrôle valables doit conduire à confier à des institutions indépendantes, si possible internationales, la mission de rassembler les renseignements tirés de l'investigation des sols et du contrôle des tassements des constructions.

On ne verra jamais assez grand dans ce domaine si l'on désire aboutir.

Pour les résultats d'essais in situ, je dirai que pour les sols de notre pays les appareils les mieux adaptés semblent être le pénétromètre statique et le pressiomètre, les pénétromètres dynamiques pouvant utilement servir d'appoint.
Le scissomètre, par contre, ne paraît pas convenir. Je crois personnellement que les deux méthodes mentionnées (pénétromètre statique et pressiomètre) peuvent et devraient se compléter harmonieusement.

Le pénétromètre est économique et de maniement facile. Le pressiomètre, plus délicat à mettre en oeuvre et plus coûteux, présente l'avantage de donner de très intéressants résultats chiffrés : les caractéristiques de rupture et de déformation sont séparées, les modules de déformation sont déterminés dans la gamme des pressions de service.

Je signalerai pour terminer et pour illustrer ce que je viens de dire sur l'intérêt de l'utilisation simultanée des deux méthodes que le coefficient K intervenant dans la formule de Buisman, qui donne le coefficient de compressibilité C en fonction de la résistance en pointe pénétrométrique r_p et de la pression verticale du terrain au niveau de l'essai p_b :

$$C = K \frac{r_p}{p_b}$$

est très vraisemblablement donné par le rapport du module pressiométrique E à la résistance en pointe r_p .

Chairman N. A. TSYTOVICH

Thank you very much. Now I want to call on Dr. Reséndiz.

D. RESENDIZ (Mexico)

I would like to refer to a question raised by Prof. De Mello and discussed a few minutes ago by Prof. Vesić, namely the evaluation of the so called coefficient of adhesion of piles driven in clay. As used by De Mello such a coefficient was defined as the ratio of the developed resistance along the pile shaft to the undrained shear strength of the soil in situ.

Prof. De Mello presented a slide showing the coefficient of adhesion, β , against the initial undrained strength of the soil (see Fig. 14 ,p. 77, of the State-of-the-art Report). In that figure, β changes from about unity to less than 0.2 as the undrained strength of the soil increases 30 times. Since the major source of strength variation in undisturbed clays is the overconsolidation ratio, I wonder if Prof. De Mello's figure could be draw equally well taking oversonsolidation ratio as the abscissa instead of undrained strength. If this is the case, the coefficient β would be larger than one for normally consolidated clays and would decrease steadily down to values much smaller than one as the overconsolidation ratio increases.

I think the fundamental reason for this behavior lies in the pattern of pore pressure development of the various soils, and therefore in this particular case, as in many other cases in soil mechanics, things are much more clear if one thinks in terms of effective stress instead of referring to total stress and undrained strength.

Normally consolidated and slightly overconsolidated clays develop positive pore pressure during pile driving. Dissipation of that pore pressure with time gives rise to an increase of effective horizontal stress in the soil around the pile and therefore to an available strength larger than the undisturbed strength of the soil in situ. Thus, the coefficient β will be above unity in this case.

On the other hand, clays with a high overconsolidation ratio develop negative pore pressure upon remoulding by pile driving, and as this negative pore pressure dissipates the effective stress decreases and the void ratio of the soil increases in the immediate vicinity of the pile shaft. Hence, after equilibrium of pore pressure is reached, the available strength of highly overconsolidated clays around driven piles is smaller than the undisturbed strength of the soil in situ. This fact accounts for the observed value of the coefficient β below unity.

Let me show some field data to support the previous interpretation. These data correspond to concrete piles driven down to 37 m at a site in Mexico City in the soil profile shown in Fig. 1. It is seen that, except for a short portion near the top, the whole pile shaft was embeded in normally consolidated clay.

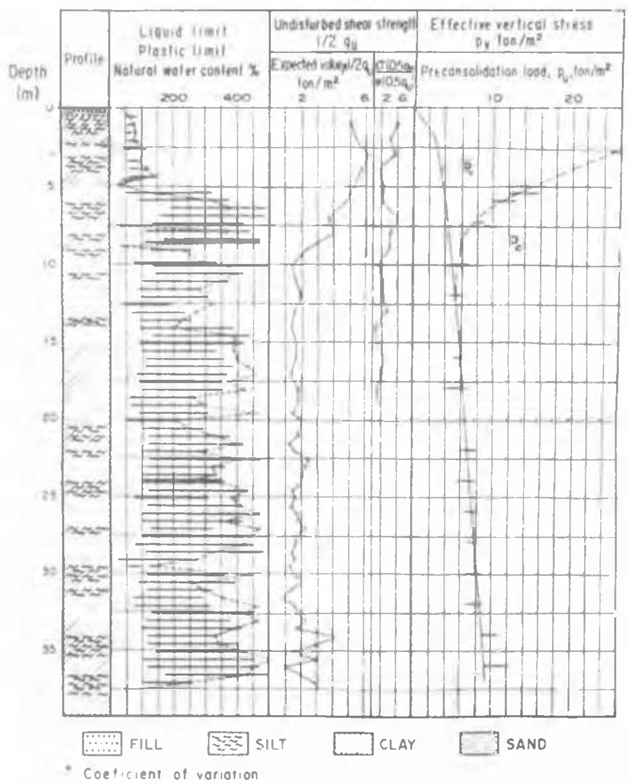


Fig. 1 Profile and Soil Characteristics

The coefficient β was evaluated in a number of piles with different rest periods between driving and testing and the results are shown in Fig. 2.

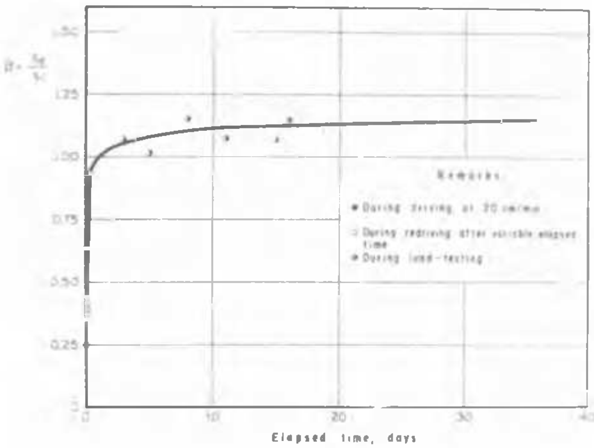


Fig. 2. Variation of β with time.

It is seen that β increases with time, as the clay around the pile shaft consolidates. This occurs rather rapidly, and after a few days the developed resistance attains an equilibrium value larger than the initial strenth of the soil in situ. As this happenes, β increases and finally stabilizes at a value close to 1.2.

However, the ratio of developed resistance to available shear strength along the shaft, computed in terms of effective stress and drained strength, is found to be unity. The computation was made assuming a K_o condition to compute the horizontal stresses since, at least in Mexico City clays, the increase in horizontal stress from pile driving reverses with time as water migration and soil relaxation take place around the pile.

The soil parameters used evaluated in the laboratory were $K_o = 0.6$ and $\phi' = 28°$.

In general, I feel that the rational way to handle this problem involves the use of effective stress and drained strength of the soil and that, if this is done, the coefficient β should be practically unity for most of the usual pile materials in every soil type.

Chairman N. A. TSYTOVICH

Thank you very much Dr. Reséndiz, Mr. Vyalov please.

S. S. VYALOV (U.S.S.R.)

This paper deals with the results of investigations that were conducted to study the mechanism of long-term deformation and failure of clayey soils. The investigations consisted in creep tests carried out on a series of identical soil specimens. At the same time, the structure of this soil was studied at various states of deformation.

The creep experiments were carried out under conditions of simple shear and consisted in twisting hollow cylindrical specimens under various constant shear loads, beginning with loads sufficiently large to cause instant failure and ending with small loads which lead only to damped deformation. In order to determine changes in the structure of the soil, the tests were interrupted at different stages of deformation, namely: transient creep (Fig. 1, curve II, section OA), steady flow (Fig.1, curve II, section AB) and progressive flow (Fig. 1, curve II, section BC).

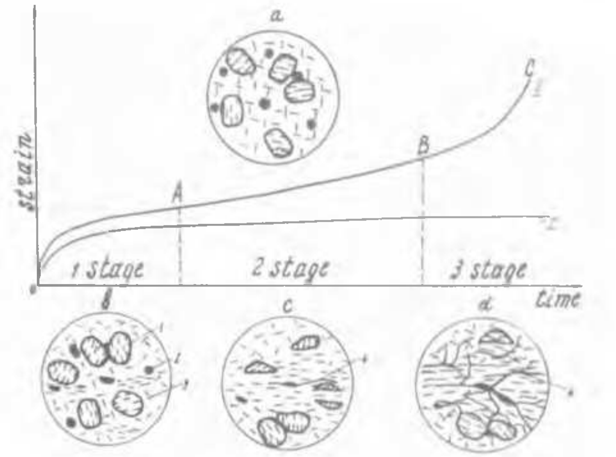


Fig. 1. - Curves of damped (I) and undamped (II) creep and schematic diagrams of the soil structure in various stages of deformation.
a) initial structure; b) in stage I; c) in stage II; d) in stage III.
Legend: 1 - aggregates; 2 - defects of the structure; 3 - cementing clay; 4 - fissures.

The investigations were conducted on artificial specimens of Jurassic clay of a semisolid consistency. This soil is characterized by high dispersion and plasticity. The content of fractions less than 0.005 mm in size is 56 per cent; the lower yield point is 48 to 50 per cent; the lower plastic limit is 26 per cent.

The soil structure was studied by the petrographic and electron microscopy methods. The microphotographs of the soil structure at the initial stage and for various stages of deformation are given in Fig. 2. The changes that occur in the soil structure in the process of creep are shown schematically in Fig. 1.

Before deformation, the structure of the Jurassic clay (Fig.2a) was characterized by the presence of microaggregates occupying about 65 to 75 per cent of the area of the microsection. Small amounts of mineral fragments (mostly quartz) were also present. The space between the above-mentioned components was filled by a randomly oriented clayey mass which seems to serve as a filler.The aggregates in the soil are rock that was not destroyed by preparation. It was formed over a long period of geological development and possesses rigid cementing bonds. Evidently, these bonds are stronger than those produced during the preparation of the specimen.

Clearly seen in Figures 1a and 2 are voids and cavities on the contacts between the aggregates. These are the weakened portions of the soil, the defects in its structure.

After the first stage of deformation - transient creep (Fig.1, curve II, section 0A)- and under the damped creep condition (Fig.1, curve I), when mainly elastic deformation develops, the number and size of the voids and cavities are reduced; at some places they are compressed and stretched out in the direction of shear. This is an indication of local displacement of the particles and aggregates, and they become more closely packed. The longer the deformation process continues, the more structural defects are obviated. Along with the healing of existing defects, new ones appear. However, under these oonditions the first phenomenon predominates and leads to a strengthening of the soil. Due to this phenomenon a number of investigators have described an increase in soil strength after prolonged deformation in conditions of damped creep (Vyalov and Pekarskaya, 1968).

If the stress is less than the limit of long-term strength, deformation at a damped rate terminates in the stabilization of the deformation. If the stress is larger than the limit of long-term strength, the deformation passes over into the second stage - steady flow at a constant rate (Fig. 1, curve II, section AB).

In the steady flow stages, along with the continuing healing of defects the aggregates break up and the clay particles are gradually oriented in the direction of shear (Figs. 1c and 2b); this promotes further deformation. The phenomenon of re-orientation of clay partioles in the creep process with a simultaneous drawing of water into the shear zone was previously established by Goldstein (1968) and Turovskaya (1964). At the same time, in the weakened portions of the soil the bonds are broken and very fine microfiseures appear. They diverge from the voids or extend from one void to another. When the stage of steady flow is prolonged, the orientation of the clay particles becomes more orderly. Extensive zones of particles oriented in the direction of shear appear. These areas are the weakest parts in the structure of the soil. Fissures form more frequently along the zones with distinct orientation of the particles. This helps to weaken the bonds and promotes subsequent deformation.

At the same time, partial rupture of the bonds and their weakening, caused by re-orientation of the particles fissure formation, is compensated for by recovery which is due to the continuing healing of defects and partial closing of fissures. At this stage, equilibrium is set up. This equilibrium gives rise to the development of the stage of deformation that proceeds at a practically constant rate.

Subsequently, as re-orientation and fissure formation oontinue to develop, and the processes of weakening and rupture of the bonds, as well as the formation of new defects, begin to predominate over the recovery processes. Equilibrium is violated and deformation passes over to the new, third stage of progressive flow (Fig. 1, section BC, curve II) in which the creep process develops at a continuously increasing rate.

The breaking-up of the micro-aggregates and re-orientation of the particles continue in this stage. Portions consisting of re-oriented particles occupy greater and greater areas (Figs. 1d and 2c). In these areas, the defects of structure develop intensively and at a continuously increasing rate, as does the growth of old fissures and the formation of new ones. The size of the fissures varies in a wide range - from ultra-fine (fraction of a millimetre) to 2 or 3 mm in width. Frequently, the fissures extend over the whole micro-section in all directions, seeming to divide the soil into separate parts.

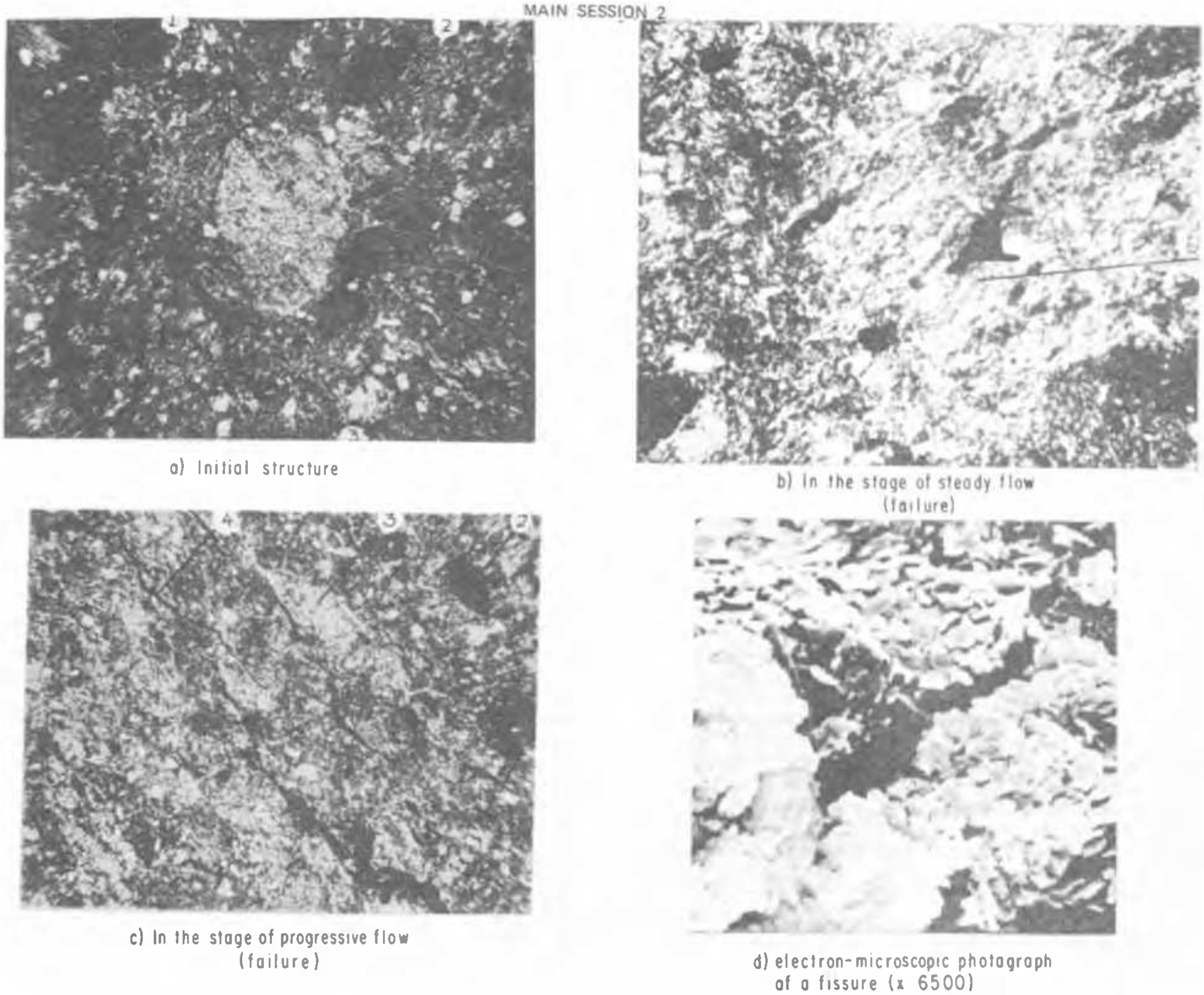
Certain microfissures widen and then merge, forming macro-fissures which lead to failure. The predominant orientation of the large fissures depends upon the magnitude of the load and on the length of the creep process. Upon rapid failure, when the particles do not have enough time to become re-oriented fissures occur mainly at the contact surfaces of the microblocks. The longer creep develops, the greater the degree of re-orientation of the particles and, consequently, the more the soil is weakened.

Re-orientation of soil particles and a reduction in the effective area of the cross-section due to fissure development lead to an increase in the actual stresses. These, in turn, give rise to the continuous increase in the rate of deformation that is characteristic of the progressive flow stage.

Failure itself is the result of fissure development and occurs, as a rule, when their density (i. e. area of the fissures per unit area of the soil cross-section) reaches a definite value for the given soil. For instance in investigating Jurassic clay of a semi-solid consistency, the critical density of the fissures was in the order of 40 per cent. Hence, it may be anticipated that a possible criterion of clay failure (of plastic, solid and semi-solid consistency) is the reaching of the critical fissure density.

Thus, during the creep process, two opposed phenomena - strengthening and weakening - occur. Strengthening is due to a reduction in the number and size of the cavities, voids and microfissures, i.e. to healing of the defects of structure, and also to more compact packing of the soil particles in shear. Weakening is due to the break-up of the aggregates, re-orientation of the soil particles in the direction of shear, and the initiation and development of microfissures, i.e. a growth of defects of the structure. If strengthening predominates over weakening, creep deformation is damped. If strengthening and weakening compensate each other, steady flow at a constant rate is developed. Finally, if weakening predominates, progressive flow is originated. This stage ends in soil failure due to the growth of defects.

The smaller the acting stress, the slower soil particles are re-oriented and fissures developed; consequently, the longer the time required to reach the limiting density of fissures and for soil failure to occur. In other words, the well-



a) Initial structure

b) In the stage of steady flow (failure)

c) In the stage of progressive flow (failure)

d) electron-microscopic photograph of a fissure (x 6500)

Fig. 2. Changes in soil structure in the creep process (microphotographs X 40)
Legend: 1- aggregates; 2 - defects of a structure; 3- cementing clay; 4 - fissures.

known phenomenon of soil strength reduction in time can be explained by internal changes of the material that take place when it is subject to long-term deformation under creep conditions, as treated in this paper.

The obtained data, which illuminate the deformation mechanism, served as the basis for deriving the equation of long-term strength dealt with in the report of Vyalov (1964) at the first Session of the Conference.

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Chairman N. A. TSYTOVICH

Thank you for your interesting speech.

In closing this session, I wish to say a few words about those who took part in its work.

First of all we must thank our General Reporter, Dr. de Mello, who did a huge amount of work in summarizing the numerous results of the research that had been conducted in the last years, including those presented in the Proceedings of this Conference, as has been reported here.

It is important to note that the results of the summarized investigations make it possible to more accurately estimate possible foundation settlements (including those of pile foundations) for structures built on clayey soils, taking into account primary and secondary consolidation of the clays and their degree of compaction.

I also wish to thank all the speakers for actively participating in the work of our session.

I propose that the editorial board of our journal "Geotechnique" organize a special section to deal with all the interesting cases of foundation settlement of structures erected on clayey soils. It is essential that the geological structure of the soil mass in the whole range of the zone of compression be described, and the values of the design moduli of deformability and the creep parameters (according to the accepted rheological theory) be given, as well as the characteristics of natural compaction, the values of the structural strength, the initial head gradient, the coefficient of the initial pore pressure and so forth, or that other comprehensive characteristics of soil deformability be given and properly substantialized. An analysis of this data will enable limiting foundation settlement values for structures to be established. It will permit us to determine the practical validity of one or another theory of settlement prediction.

I thank you for your kind attention.

General Reporter V. F. B. de MELLO (Brazil)

In the few minutes allowed for the parting remarks it will not be possible to insert any more than a very sincere vote of thanks for the many interesting contributions, and a very brief mention of some items that have brought forth the requested note of debate.

I should first take liberty to single out Dr. Rosenblueth's presentation as constituting a very important beacon for the development of the field along the lines proposed by my report. It was my task to look at the past and present: I chose to emphasize that the analysis should be statistical, the progress in the field being associated with a raising of the lower confidence level. Dr. Rosenblueth points to the future, asserting that the right approach to the design decisions should thereupon rely more and more on analysis based on decisions theory.

Regarding the problem of rough and smooth contact faces, brought out by Dr. Milovic, the interest in establishing the upper and lower bounds to the solutions is being frequently tendered mathematically: to my knowledge, however, very little experimental data exist to date to aid the design decision as to the behaviour to be associated with a given interface.

Moretto discusses the interference of a partial drainage under a footing, on the applicable strength parameters and bearing capacity equilibrium. Consolidation (drainage and/or absorption) will take places not merely under the "pressure bulb" postulated, but wherever the change of stresses has created excess pore pressure, and the dissipation of a localized pore pressure affects the surrounding soil elements (as would happen to the zones marked as undrained in Moretto's Fig. 3). It is my opinion that the complexities and uncertainties surrounding such situations will always fall back, at some point, into what was stated to be (P.52 of the State of the Art Report) a "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."

Schmertmann brings out the importance of the E/c ratio in clays, in interpreting the Ncp values of static penetrometers. We have tended to consider that E/c varies very little because of a tendency for both values to vary together. The discussion constitutes an interesting example of improved confidence levels of correlations. Moreover, I take the liberty to emphasize the point, brought out by the discussion, that often the bane of field investigations has been the attempt to solve a single equation for two or more simultaneous unknowns. The conjugate use of the static penetrometer and the pressuremeter, as mentioned by Wambeke, constitutes a suggestion along the same line.

Kerisel's discussion on the influence of dimensions on the undrained Nc value in homogeneous clays really throws me into some confusion. Surely the dimension of the footing must be taken into account in using penetrometer results. But it has been my impression that this fact results from the excessive deformations (proportional to B) required to develop the failure, so that settlement criteria take over in establishing allowable pressures. Although experimental evidence of load tests on large diameters rarely goes to deformations of about 0.1 B as seems required, it as been assumed that at such a deformation the failure condition would reproduce approximately the Nc value, independently of diameter. Unless I have interpreted the discussion incorrectly, it seems that the clue given by Kerisel's Nc values of 2.5 and 1 is that he is referring either to what I would call Nc values, or to allowable pressure Nc values. It appears important to clarify this point.

As can be seen from the several contributions to this Session, we shall always have a lot of work ahead of us in complementing or revising present knowledge.

M. APPENDINO and M. JAMIOLKOWSKY (Italy)

In relation to our paper from Session 2 "Foundation for 200 m high chimney on an overconsolidated clayey silt",[1] we are presently in a position to complete the information on the foundation behaviour reporting field observations performed from January to July 1969, when construction of the chimney was completed.

On the basis of these further observations we can make the following remarks:

1) An increase of foundation load (W) from 14220 tons to the final load of 16245 tons corresponded to an average increase of the total settlement of about 2 mm; see fig. 1.

The calculations were made with contact pressure distributions corresponding to the total weight of chimney and in one case with the reduction of buoyancy deriving from the re-establishment of the water table after the excavation disturbance (at about 8 m a.s.l.)

A Young modulus of 200,000 kg/cm^2 and a Poisson ratio of 0.16 were assumed in the concrete.

Results are given in Table 2. The results from the deflection values indicated that to obtain a correspondence between measured and calculated deflections it is necessary to allow that the pressure distribution changes progressively from a parabolic shape with downwards concavity to a parabolic shape with upwards concavity (see curves a and c, fig. 1). This behaviour may be explained with a decrease in the long run of the deformation modulus of soil at the edge of the foundation where deviatoric stresses were initially larger. Peripherical high pressures due to wind loads could give also a substantial contribution.

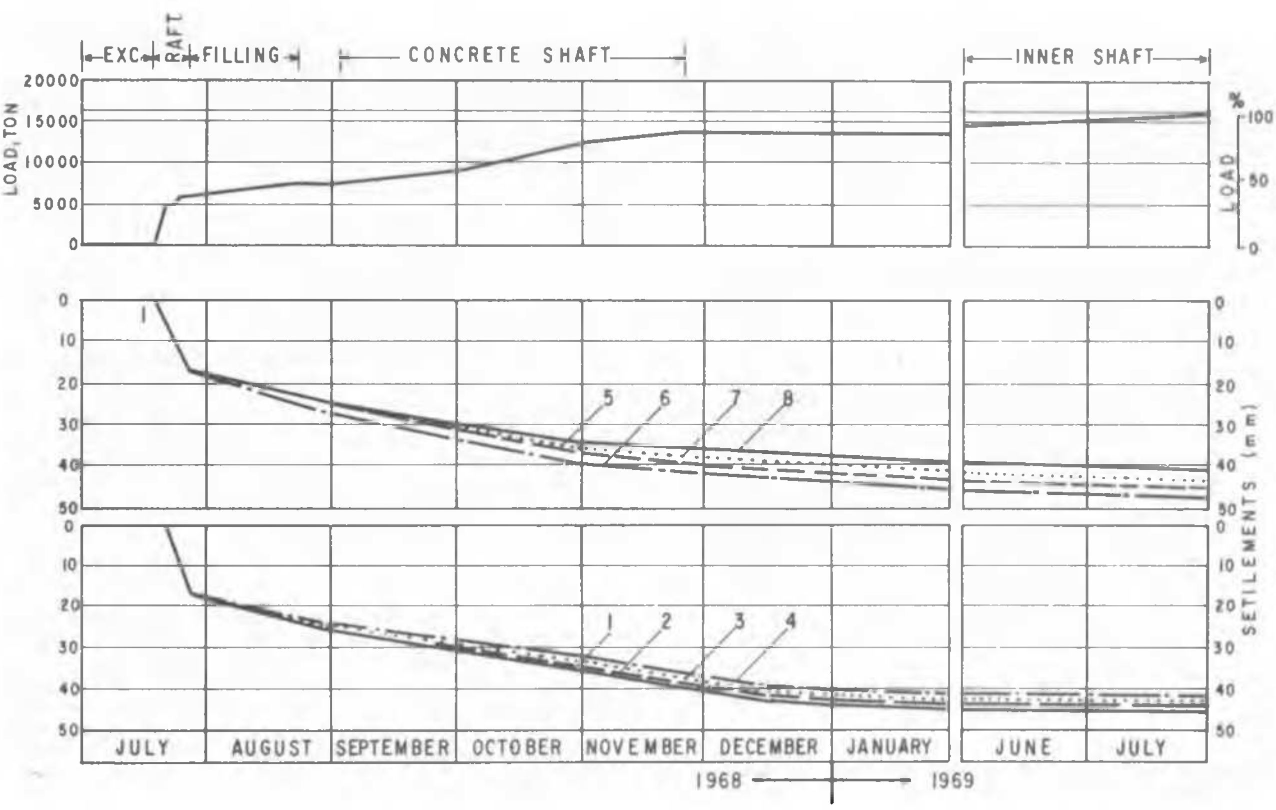


Fig. 1. Settlements and loads during chimney construction

2) The piezometers readings indicate that the pore pressure is still decreasing but there is a tendency to reach a stable level ranging approximately from 46 to 47 m above sea-level.

3) The deflections of the cantilevered edge of foundation decrease in the long run as it may be seen from the data indicated in the Table 1.

The deflections were calculated with different soil pressure distributions as shown at fig. 2.

If this hypotheses were valid, the soil layer below the foundation would have a deformation modulus variable according to fig. 3.

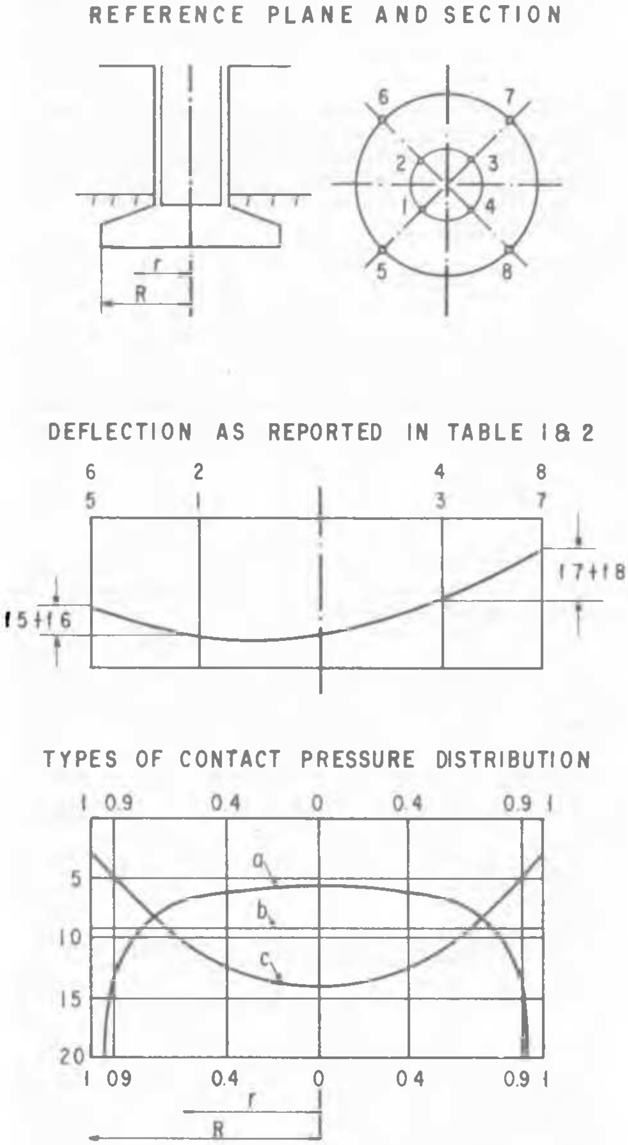


FIG. 2

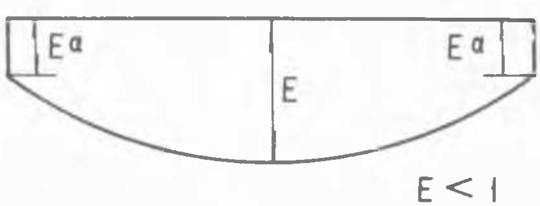


FIG. 3

To understand in a better way the behaviour of the soil we put seven earth pressure cells, of the BRS type, below the foundation of a second chimney which is now under construction at the same site.

TABLE 1

DATE	DEFLECTIONS f5	f6	f7	f8	AVERAGE DEFLECTION	% OF LOAD
DEC 1968	232	127	380	427	2.91 mm	87
MAY 1969	108	000	1.82	3.85	1.69 mm	92
JUNE 1969	020	020	1 15	3.18	1.08 mm	97
JULY	010	080	060	240	0 525 mm	100

f1, f2 = DEFLETION AT REFERENCE POINTS 1,-2-3-4-5-6-7-8 INDICATED IN FIG 2.

TABLE 2

TYPE OF PRESSURE DISTRIBUTION	LOAD (W) %	PRESSURE t/m² R=0	r/R=1	DEFLECTION (M+S)
a	87	5.88	13.51 *	2.79 mm.
b	92	9.27		1.66 mm
c	100	1.5	1 50	0.58 mm.
	100	1.3 **	1 16 **	0.54 ** mm

* CONTACT PRESSURE AT r = 0.9 R
** VALUES CALCULATED WITH BUOYANCY REDUCTION

REFERENCE

1. M. Appendino and M. Jamiolkowsky (1969), "Foundation for a 200 m high chimney on a clayey silt", Proc. Seventh Int. Conf. on S. M. and F. E., Mexico, Vol 2 pp. 9-15

Referring to Appendino and Jamiolkowsky's additional information on the behaviour of a 200 m high chimney foundation[1], it may be interesting to point out some results obtained from a calculation of the raft internal stresses, made according to the three reaction hypothesis given in Fig. 2 as types of contact pressure distribution[1]. To make calculations easier, the raft external circular ring, of variable height, has been replaced with a circular ring of constant height and of equal deformability.

Assuming the stresses in hypothesis b equal to unity, the radial maximum moment and the maximum shear in the hypotheses a and c result respectively: hypothesis a: 1.7; 1.2; hypothesis c: 0.6; 0.8. It is obvious that the commonly adapted hypothesis b may be dangerous when the soil reacts, even during a short time, as in hypothesis a.

(1) This reaction distribution diagram takes into account only the stresses due to the weight of the shafts.

Deflections reported[1] in Table 2 include, even if not exactly, the shear effect, of remarkable consequence in a structure of this kind, which is variable depending on the type of load.

The Young modulus for the 350 kg/cm^2 cubic strength concrete is probably higher than 200,000 kg/cm^2, as adopted in the deflections calculation; a higher value would give a closer correspondence with measured deflection in hypotheses a and c.

REFERÊNCE

1 - Appendino, M. and Jamiolkowsky, M., "Foundation for a 200 m high chimney on a clayey silt-Additional information", Proc. Seventh Int. Conf. on S.M. and F.E., Vol 3, Written contribution to Session 2.

V. ESCARIO (Spain)

I was glad to read the Report presented by Aitcheson and Woodburn, entitled "Soil Suction in Foundation Design", since it supports some of the basic ideas I presented to the Conference on Expansive Soils, Texas, (V. Escario, 1965) which briefly, were included in the corresponding Proceedings.

Field conditions shall be, reproduced in the laboratory as accurately as possible. Therefore, the swelling tests shall be carried out handling the two variables which take part in the phenomenon: exterior forces and suction. It is useless trying to anticipate the behaviour of an expansive soil by flooding the sample, as it is usually done, despite the corrections which may afterwards be applied by the existing formulae, specially knowing that the principle of effective stress can hardly be applied to these cases.

Consequently, four years ago I informed in Texas about the tests which we were performing, applying a suction to the source of water by means of a mercury column. Although the concept was then clearly established, the procedure had the practical difficulty of being limited to a maximum suction of -1,0 Kg/sq. cm. In view of the foregoing, I continued investigating the possibility of applying suctions higher than -1,0 Kg/sq. cm. to the water. As a result of this, two years ago I submitted a

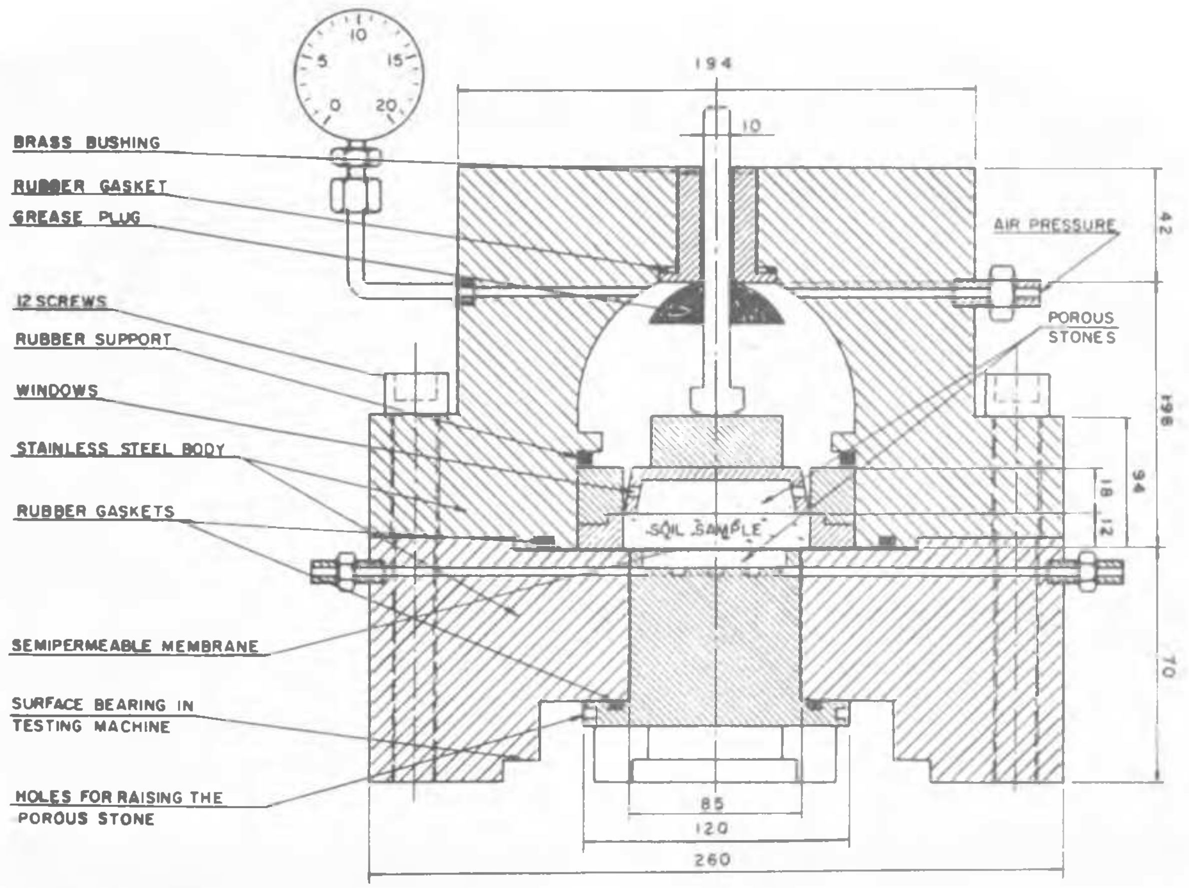


Fig. 1. Apparatus for applying suction to the feeding water

Report entitled "Measurement of the swelling characteristics of a soil fed with water under tension" (V. Escario, 1967) at the International Meeting held in Madrid, by the working group organized under the OCDE auspices, entitled "International Cooperative Research on the Prediction of Moisture Content under Road Pavements".

In the above work I proposed the use of an apparatus to determine the swelling characteristics where the suction of the feeding water is obtained by applying air pressure to the upper part of the sample, which in turn is in contact with water at atmospheric pressure through a semipermeable membrane; that is, using a system similar to the one employed in the well known pressure membrane apparatus to obtain the curves suction/moisture content of a soil.

A high pressure triaxial cell was then being used, conveniently adapted.

Figure 1 schematically shows the new apparatus we have now in use, as presented only a few days ago to the 2nd International Conference held in Texas (V. Escario, 1969).

The results obtained may be seen in the Proceedings of the Conference.

References

Escario, V. (1965) "Engineering Effects of Moisture Changes in Soils". International Research and Engineering Conference on Expansive Clay Soils. Texas A & M University. pp 23-25.

Escario, V. (1967) "Measurement of the swelling Characteristics of a soil fed with water under tension". International Cooperative Research on the Prediction of Moisture Content under Road Pavements, Working Group under the auspices of OCDE, Madrid meeting.

Escario, V. (1969) "A New Method for in situ Measurement of Pore Water Tension", International Conference on Expansive Soils, College Station, Texas.

A. A. GRIGORIAN (U.S.S.R.)

The industrial methods of construction and the tendency to increase the loads on footings provide a basis for future development on the use of pile foundations in loess collapsible soils.

Nowadays the piles, that completely or partly cross the layers of collapsible soil, are being widely put into practice, specially in loess soils of the first type. To the first type belong loess layers, for which the collapse under overburden pressure is practically impossible. The design of piles in collapsible soils is made according to data obtained from the field tests.

In the case of theoretical solutions there are a lot of difficulties and it is still impossible to estimate exactly the stress condition in the soil around a loaded pile.

In order to investigate the behaviour of driven friction piles, which does not cross loess collapsible layers, experimental investigations were carried out (Grigorian and Mamonov, 1968). Piles were driven through a collapsible soil with a natural (low) water content. Then the soil around them was wetted before the static load test and during it. The length of the test piles was 5-7 m with a cross-section of 30 x 30 cm.

The field tests with the special piles were carried out and the ultimate values of point resistance and skin friction were separately obtained. The sum of these values in all cases in saturated loess was equal to the ultimate load, which was obtained by tests performed in the whole pile. Therefore to know the behaviour of a pile in collapsible soils one may examine separately these two resistances.

The shear forces between the pile and the saturated loess are mainly due to the skin friction, because there is generally no cohesion in this soil. In

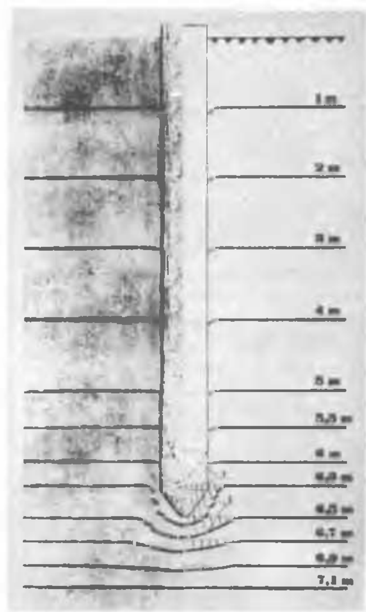


Fig. 1 - Cross-section of pile after test with indicator lines that show the deformation of the soil around the pile: 1-before static load test; 2-after static load test.

the state of failure there was a slip between the pile and the loess. The horizontal lines, formed by individual elements acting as deformation indicators, after the pile was driven into the soil, are shown in Fig. 1. The indicators near the pile, along its length, had no vertical displacements although the settlement of a pile was about 30 cm. The point resistance depends mainly on the relation between the volume of the compaction zone, formed by driving, and the volume of the deformation zone under the action of a static load.

When the load on a pile increases to almost the value of the ultimate load, the elastic strains of the soil mostly take place in the compaction zone. At the same time the residual strains are small. The complete settlement of a driven pile in loess soil upon reaching the ultimate load is less than 3-5 mm.
Under the ultimate load of a pile the stresses at the lower part of the boundary of the compacted zone reach a certain value, which is called the initial deformation pressure of saturated loess.

When the load on a pile is greater than the ultimate one, the deformations of the loess start outside the compaction zone. In this moment a sharp increase of settlement is observed.

When the natural density of the soil outside the compacted zone is high, then the bearing capacity of the pile is also high. This is due to the fact that the vertical load, causing an intensive compaction of the loess outside the compaction zone, must be also large.

In loess soil, point resistance takes the main part of the total bearing capacity of a pile. For instance, in homogeneous, loose, highly collapsible loess (the coefficient of relative collapseness δ_{col}, taken from oedometer tests, was 0,05-0,07) the ultimate load on the pile with a length of 5,6m and a crosssection of 30 x 30 cm in saturated soil was 15 tons. Point resistance in this case was equal to 8 tons.

Therefore to obtain a high efficiency when using the driven piles in collapsible soils, under conditions of possible wetting, it is necessary to put the pile points in soils with $\delta_{col} < 0,02$.

In order to increase the bearing capacity of foundations on high collapsible soils of the first type, we suggest to use a short conic pile-footing, which is placed on a punched base. This pile has an upturned cone shape (cone, truncated cone, pyramid), made of concrete or reinforced concrete. It can be precast or cast in situ.

The bed of the pile is formed by a rammer, falling from a certain height (Fig. 2). The shape and size of the rammer are equal to those of the pile. In this manner the soil is displaced under percussion action and is compacted to take the shape of the pile. By means of this method the bearing capacity of pile-footing on collapsible soil may be largely increased.

As compared with known conic piles, the suggested pile has a much bigger angle of cone and base diameter. That also causes its higher bearing capacity. After wetting the soil the pile with angle of cone 34°, length 2m and diameter of base 1,2 m had a bearing capacity of 50-80 tons, which depended upon the density of the loess outside the compaction zone.

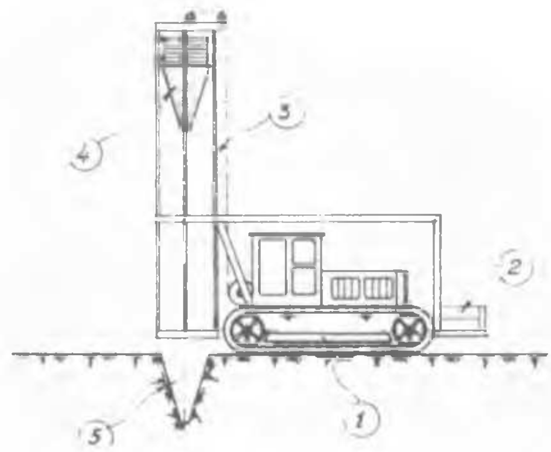


Fig. 2- Scheme for pressing out the base to make the pile-footing: 1. bulldozer; 2, weight; 3, frame; 4. rammer; 5. base for pile-footing.

The pile-footing needs in this case rather less reinforcement than usually. The suggested type of pile-footing has an advantage as compared to usual types of driven piles. This is due to the fact that the normal compressive stresses, developed on all contact surface, are mainly resisting the vertical load, applied to this pile-footing. The shear stresses along inclined have a minor influence. In the field tests that were carried out in the South Ukraine, eng. Goldfield took part.

When dropping the reinforced rammers with conic or truncated cone shape, the contact surface of the holes was found to be very dense and smooth. It gave the opportunity to cast the concrete into the hole without timber forming.

In the cases of large thickness of collapsible soil layer (10-30 m), when the load on the foundations is great, a widely used practice is to use different sorts of cast-in-place piles with bulbs near their lower end. The bulbs are made in the following way:
1. boring, 2. pressing the concrete into the soil near the bottom of bore hole by vibro method and 3. pressing out the soil aside by means of special multipetal equipment with the aid of jacks. The latter method is the most promising, because it gives an opportunity to increase the bearing capacity of bulbs by means of compacting the soil without excavating it.

REFERENCES

A. A. Grigorian and V. M. Mamonov (1968), "Estimating the bearing capacity of a driven friction pile in loess soil of first type", Proceedings of the 3rd Budapest Conference on Soil Mechanics and Foundation Engineering, pp. 549-558, Budapest.

N. JANBU (Norway)

I agree with panel member, Dr. Golder, that theoretical time rate estimates so far, and as a whole, have failed miserably in predicting the actual time-behaviour of structures and fills on clay. But I believe this is because a vast majority is still using the classical stress-theory of consolidation. If, instead, a proper strain-theory, allowing for non-linearity, is applied, one will find that 80 - 90% of previous errors go down the drain - in more than one meaning.

To illustrate the difference between a stress - theory and a strain - theory, reference is made to Fig 1. A 20 m thick normally consolidated clay layer, overlain by 1 m of sand, is carrying $10t/m^2$ uniformly distributed. The tangent modulus is $M=15 \cdot p'$; hence m=15, and $c_v=8\ m^2/\text{year}$. The ground water level coincides with the clay surface, and initial hydrostatic pore pressure is assumed. The simple vertical stress distribution with depth is also shown.

For each selected depth the vertical strain can be calculated by the formula:

$$\mathcal{E} = \frac{1}{m} \ln \frac{p_o' + \Delta p}{p_o'}$$

For instance at elevation - 10 m, where $p_o'=12t/m^2$ and $\Delta p=1t/m^2$ one gets,

$$\mathcal{E} = \frac{1}{15} \ln \frac{22}{12} = 4.1\%$$

Thus, the strains have been obtained for the entire clay layer, and the $\mathcal{E}-z$ curve has an area $\delta_c=98$ cm. This same settlement, 98 cm, and the same strain distribution can be obtained by using $\frac{C_c}{1+e_o} = 0.153$.

Now, by the classical stress theory the time rate is governed by the Δp- distribution. Here Δp is a constant equal to $10t/m^2$. Therefore, to reach 50% consolidation or 49 cm,

$$t = T\frac{H^2}{C_v} = 0.197\ \frac{20^2}{8}\ \text{yrs} \simeq 10 \text{ years}$$

From the strain - theory, however, it is the shape of the $\mathcal{E}-z$ curve that determines the rate (see Fig. 6, page 195 in Vol I of this conference).

The calculated $\mathcal{E}-z$ diagram is accurately represented by a rectangle of area 50 cm^2, and a parabola of 48 cm^2 in area. One can then obtain the resulting δ-t curve by adding the two δ-t curves vertically after each is calculated and plotted in separate δ-t diagrams. Ommitting details of calculation, the final result is represented by the fully drawn curve in Fig. 1.

When comparing the time required by the stress-theory, t_u to that needed by the strain theory, $t_{\mathcal{E}}$ it is seen that:

The fill settles many times faster by the strain-theory, especially in the early phase of the consolidation. For instance, to reach 50% consolidation it takes less than four years by the strain-theory as compared to 10 years by the stress-theory.

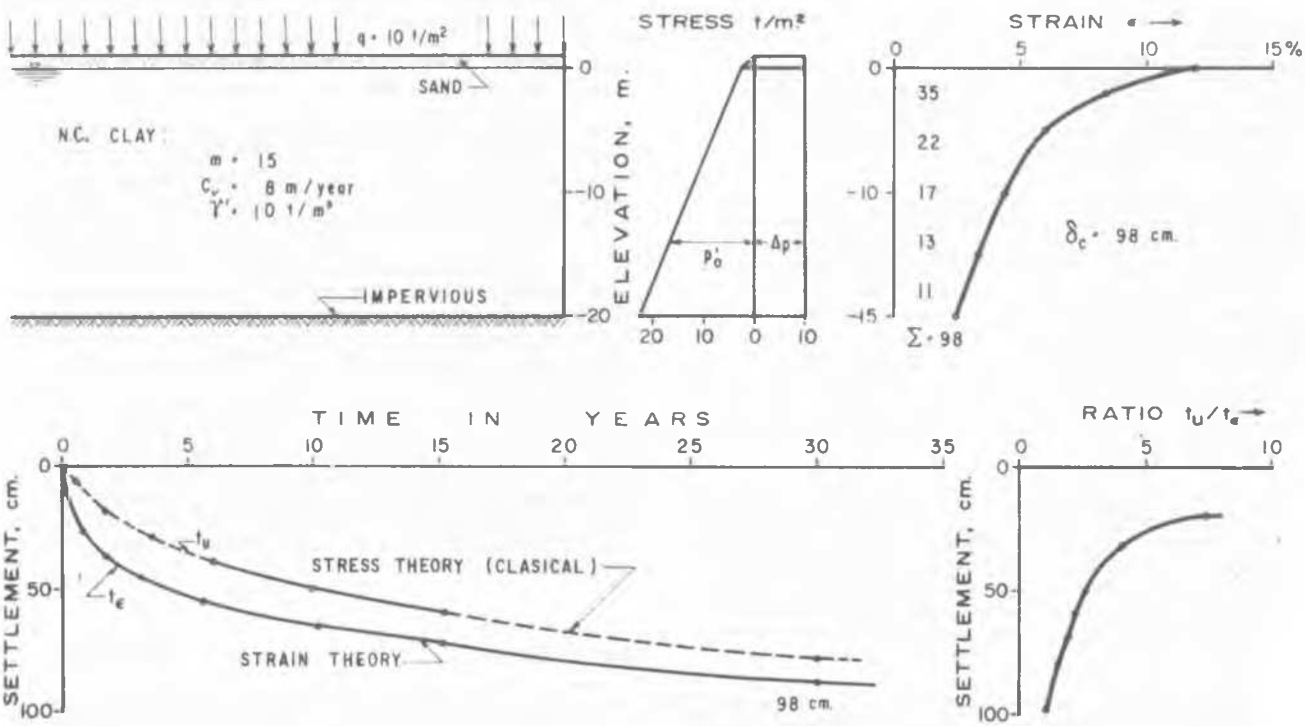


FIG. 1

Here, the comparison is made for a theoretical case to avoid influences of unknown factors in the comparison. However, studies of several available case records for fills and structures on approximately normally consolidated clays have disclosed that the strain theory predicts the time-rate reasonably well. At least, I have yet to encounter a well-documented case record where the differences between the strain-predicted and the observed behaviour is alarming, or even unsatisfactory for practical purposes.

J. MATHIAN et R. PAUBEL (France)

COMPLEMENT A LA COMMUNICATION FIGURANT AUX PAGES 173 à 181 DU VOLUME N° 2

9 - OBSERVATIONS COMPLEMENTAIRES

9-1 Tassements observés récemment sur les ouvrages de Bourg-lès-Valence

Depuis la rédaction du rapport visé ci-dessus les mesures de nivellement ont été poursuivies régulièrement.

Ces mesures montrent que le tassement observé depuis la mise en eau des ouvrages se continue. Au mois de juin 1969, soit 18 mois après l'arrêt des épuisements et 12 mois après la mise en eau des ouvrages aux niveaux définitifs d'exploitation, la vitesse de tassement est encore de l'ordre de 1 mm par mois.

Le basculement de l'usine vers l'amont a également continué à s'accentuer légèrement (voir figure n° 8 bis complétant la figure n° 8 du rapport).

Rien ne permet donc de penser qu'il y ait tendance au transfert du poids de l'eau sur les pressions interstitielles.

A cette remarque, nous rattacherons certaines mesures effectuées lors d'essais de disjonction des groupes hydro-électriques de l'usine. A l'occasion de ces essais, on a constaté qu'une surélévation momentanée du niveau en amont de l'usine, de l'ordre de 1 m d'amplitude pendant une trentaine de minutes, accentuait temporairement le basculement de l'usine vers l'amont. Cette accentuation a été mesurée en observant l'un des pendules de 30 m de hauteur visé au paragraphe 5-2 : le déplacement momentané du pied du pendule est de l'ordre de 0,15 mm vers l'amont et dure approximativement le même temps que la surélévation du niveau d'eau en amont de l'usine.

Cette dernière observation confirme le fait qu'une augmentation rapide de la masse d'eau surmontant l'argile provoque un tassement instantané du squelette solide. D'autre part, la grande régularité des courbes de tassement observées depuis la

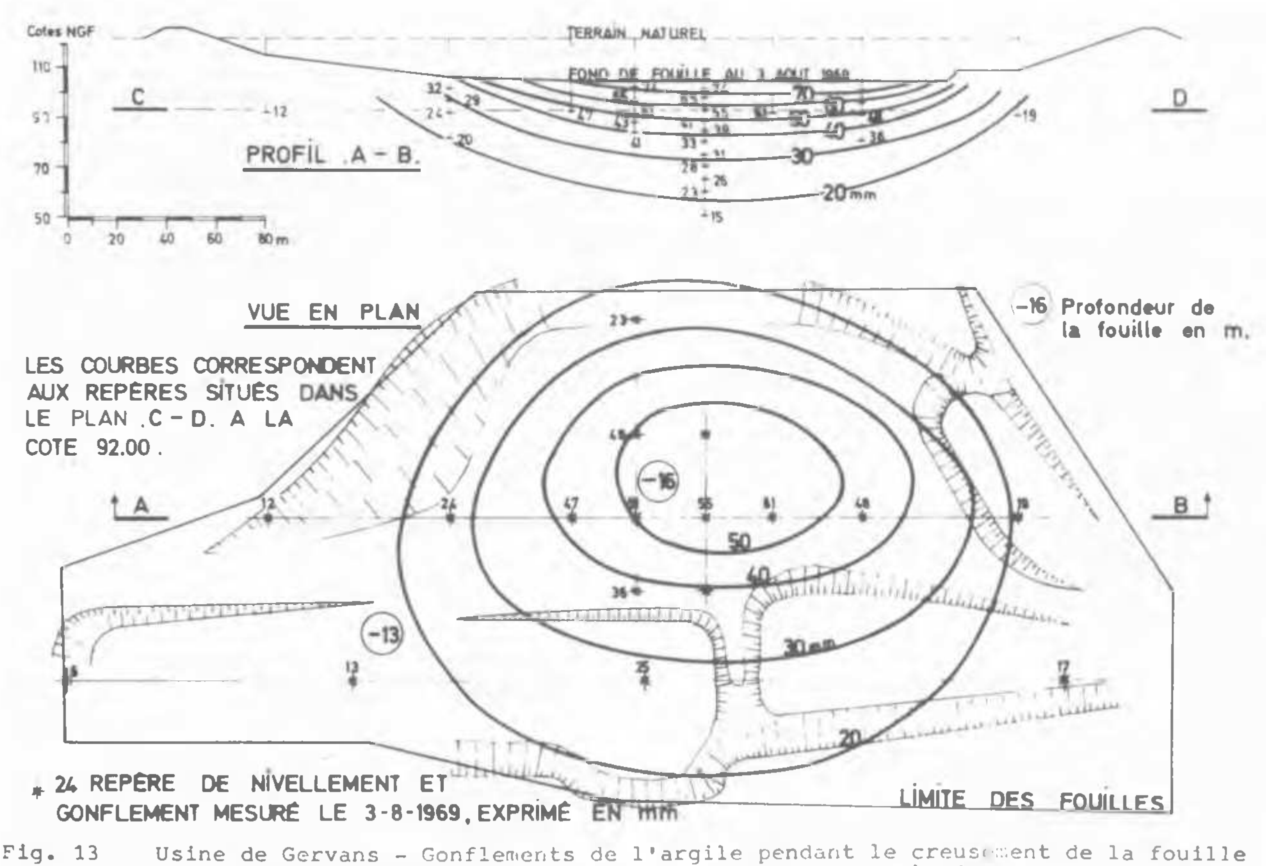


Fig. 13 Usine de Gervans - Gonflements de l'argile pendant le creusement de la fouille
Courbes d'égal gonflement le 3 août 1969 (fouilles à 16 m de profondeur)

mise en eau démontre la vraisemblance de l'hypothèse formulée précédemment : pour expliquer au mieux les mesures de gonflement et de tassement, il faut tenir compte non pas des variations des charges effectives des terres enlevées ou du béton mis en place, mais des variations du poids total de ces matériaux, en y comprenant la masse d'eau qui les baigne ou les surmonte.

L'hypothèse de l'eau agissant par son poids n'est peut-être pas tout à fait exacte, mais elle semble donner des résultats qui sont plus en accord avec la réalité que ceux obtenus dans l'hypothèse classique qui ne prend pas en compte la pression d'eau.

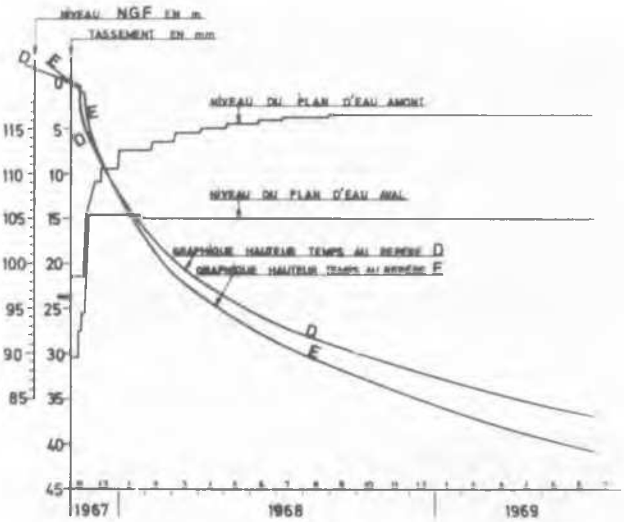


Fig. 8 bis Usine de Bourg-lès-Valence
Courbes de tassement
(mises à jour en juillet 1969)

9-2 Gonflements de la masse d'argile observés pendant les terrassements de la fouille de l'usine de Gervans

Les terrassements nécessaires à la construction d'une nouvelle usine hydro-électrique, celle de Gervans, ont commencé au printemps 1969.

Cette usine est fondée sur la même formation d'argile pliocène que l'usine de Bourg-lès-Valence. En profitant des expériences précédentes, on a cherché à analyser de manière plus détaillée les gonflements de la masse d'argile pendant toute la phase de terrassement.

Le nombre de repères mis en place dans l'argile avant le début des terrassements est d'environ 80. Ces repères ont été installés dans des forages verticaux. Pour déterminer leur altitude à un moment donné, on utilise une torpille à ailettes escamotables, ce qui a permis d'installer plusieurs repères (jusqu'à 10) dans le même forage. Les repères les plus profonds sont situés à 50 m de profondeur en dessous du toit de l'argile.

A chaque campagne de nivellement, il est alors possible de tracer dans un plan ou dans un profil donné des courbes d'égal gonflement par rapport à l'état initial.

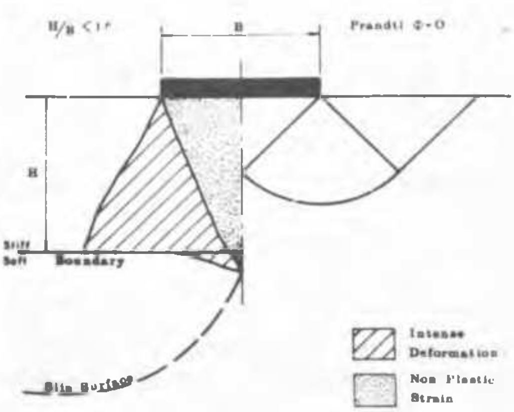
La figure n° 13 a été établie de cette manière, à une époque où la fouille avait atteint 16 m de profondeur. Au centre de la fouille le gonflement maximal des couches superficielles de l'argile dépasse 70 mm ; il est de 20 mm à 50 m de profondeur.

Les réactions sont extrêmement rapides puisque les relevés, ayant servi à établir ces courbes, ont été effectués alors que le chantier de terrassement était en pleine activité et que les travaux avaient commencé seulement trois mois auparavant.

Bien qu'elle soit relative à une autre usine que celle de Bourg-lès-Valence, cette figure a l'avantage de bien mettre en évidence l'expansion générale de la masse d'argile pendant le creusement d'une grande fouille ; dans l'espace, les surfaces d'égal gonflement ont grosso modo l'allure de calottes sphériques dont les centres se situeraient tous sur la verticale du centre de gravité de la fouille.

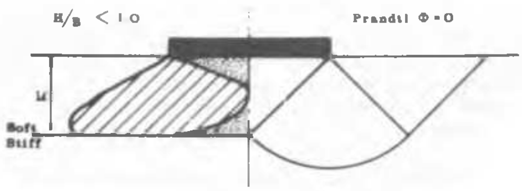
G. G. MEYERHOF (Canada)

The paper by Mandel and Selendon in the present Conference Proceedings (Vol. 2, p. 157) provides a valuable analysis of the ultimate bearing capacity of strip foundations on rigidly supported soil strata of limited thickness in relation to the footing width. For the special case of purely cohesive materials this analysis agrees with the previous theory and experiments by the writer[1] who additionally treated the cases of circular and rectangular foundations on such materials. However, it should be noted that for saturated clays in the undrained state both the failure mechanism and the bearing capacity depend on the true (Hvorslev) cohesion and internal friction of the clay and not on the apparent (undrained) cohesion and $\phi = 0$ condition. This conclusion is confirmed by the results of experimental studies of the ultimate bearing



a) Stiff clay overlying soft clay

capacity of layered clays in the present Conference Proceedings (Brown and Meyerhof, Vol. 2, p. 45). Typical failure patterns observed beneath strip footings on saturated clay strata used in this investigation are shown in Fig. 1 and differ considerably from those used in the theoretical analysis. Accordingly, the latter authors have suggested semi-empirical relationships to estimate the bearing capacity of foundations of various shapes on clay strata of limited thickness.



b) Soft clay overlying stiff clay

Fig. 1. Typical rupture figures

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T. K. NATARAJAN (India)

The paper on "Storage Yard Foundation on Soft Cohesive Soils"[1] clearly brings out the significance of horizontal deformations within a soft clay stratum. The many instances of horizontal movement of abutment piles in India testify to this. I wish to add yet another experience we had in recent months.

A certain cargo jetty in Kandla Port in India, was designed to rest on hollow RCC piles, 20" dia. ,at 12ft centres. The subsoil was 48 ft of soft clay (with c = 350 psf on the basis of vane tests) underlain by 20 ft of medium dense sand, followed by very stiff clay. The port authorities had completed the installation of the piles, penetrating through the soft clay layer and 5 ft into the sand layer, driven to a set of 3/4" under the last 10 blows using a 4500 kg single acting steam hammer with a drop of 28". At this stage and not earlier the problem was referred to us for an analysis.

Pile load tests were made. Suitable corrections were made for the removal of clay surrounding the piles due to anticipated dredging of slopes as well as for negative skin friction.

The safe load per pile was determined to be 70tons with a factor of safety of 1.5 against bearing capacity failure. The factor of safety for the slope under the jetty at 1:3, was 1.5 against rotational failure. In view of the rather low factor of safety, no filling was permitted immediately behind the jetty and the approach road 100 ft wide was also made to rest on piles.

Despite the provision of piles and despite assurances of stability of slopes against rotational failure, using shear strength values based on conventional tests, there were indications that the soft clay layer would tend to move horizontally towards the seaside, on the basis of observational data. Since the piles did not have adequate embedment into the sand and since laboratory tests indicated a low creep strength, viz., of about 0.6 of the ultimate shearing resistance, the possibility of a toe kick-out of the piles could not be altogether ruled out.

As a corrective measure, a 2½ ft thick RC diaphragm wall penetrating deep into the stiff clay, to act like a cantilever sheet pile wall in order to withstand the lateral thrust, had to be provided in front of the quay face. The wall was wholly buried below the sea bed level. The jetty was declared safe only after this was done.

This clearly demonstrates the need for consideration of bending stresses in all such piles, such as abutment piles etc., driven through soft cohesive soils. I believe this mode of failure as distinct from and in addition to other possible modes of failure, therefore deserves to be considered, in any foundation analysis, under these circumstances.

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G. SANGLERANT (France)

Le paneliste Pérez Guerra a bien voulu citer au cours de son exposé les travaux de J. Gielly, P. Lareal et G. Sanglerat concernant l'étude de la compressibilité des sols et la prévision des tassements a l'aide du pénétromètre statique.

Les recherches entreprises a ce sujet conjointement depuis plusieurs années par l'Ecole Centrale Lyonnaise et l'Institut National des Sciences Appliquées de Lyon ont fait l'objet d'une première communication à la "Conférence on in situ investigations in soils and rocks" (Réf. 1). Depuis lors, les études statistiques ont été poursuivies et actuellement elles portent sur plus de 350 couples de comparaison pénétromètre-oedomètre réalisés sur différents chantiers de France et de Belgique. Ces études ont permis de compléter et de parfaire les résultats publiés des 1965 (Réf. 2 et 3) au sujet du coefficient α figurant dans la relation

$$m_v = \frac{1}{\alpha R_p}$$

permettant de calculer le coefficient de compressibilité volumétrique m_v en fonction de la résistance à la pointe du pénétromètre statique R_p.

Les coefficients α ont été donnés pour différents types d'alluvions modernes: argile peu plastique (CL), argile très plastique (CH), sol tourbeux et sable (Réf. 1).

Il est intéressant de noter que les géotechniciens anglais, A.C. Meigh et B.O. Corbett ont trouvé lors de l'étude des tassements de réservoirs à essence en Arabie (Réf. 4) des résultats qui recoupent les valeurs de α proposées pour les argiles (CH). Il en est de même pour les résultats obtenus par A.S. Correia Mineiro dans les sables d'Angola (Réf. 5).

Les études statistiques ont, par ailleurs, permis de mettre en évidence que quelle que soit la nature des sols étudiés, le coefficient de compressibilité C_c est lié a la résistance à la pointe du pénétromètre statique, R_p.- La figure A indique le fuseau des valeurs trouvées limité par deux hyperboles équilatères.

Cette figure est capitale car il est possible d'en tirer la conclusion pratique suivante:
"Dès que la résistance à la pointe R_p du pénétromètre statique dépasse 12 bars, les bâtiments courants fondés superficiellement avec des contraintes raisonnables ne doivent pas subir de tassements excessifs".

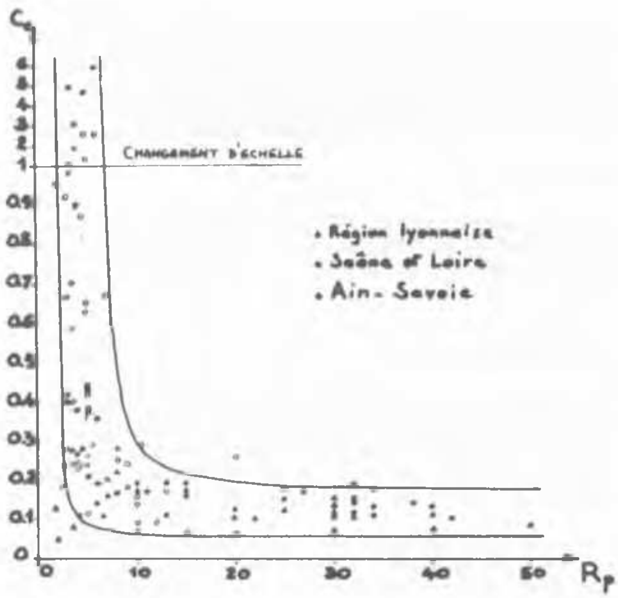

Fig A: C_c fonction de R_p

Lorsque R_p est inférieur à 12 bars, des tassements non négligeable peuvent se produire et il y a lieu, a ce moment là, de déterminer la teneur en eau w et de se reporter aux tableaux qui donnent des valeurs de C_c ou de α en fonction de R_p et w.

Grâce au pénétromètre statique on peut donc apprécier rapidement, avec une bonne approximation, les tassements absolus et différentiels éventuels puisque cet appareil permet de déceler sans risque d'erreurs ou d'omission la résistance et l'épaisseur des couches compressibles sous jacentes.

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3. L. PAREZ AND BACHELIER (1965), "Contribution à l'étude de la compressibilité des sols à l'aide du pénétromètre à cone", Montréal, Section II, p. 3.

4. A. C. MEIGH and B.O. CORBETT (1969), "A comparison of in situ measurements in a soft clay with laboratory tests and the settlement of oil tanks", Conference on in Situ Investigations in Soils and Rocks, London

5. A.J. CARREIA MINEIRO (1969), "Obras de Engenharia civil do projecto mineiro de Cassinga Companlira mineira do Lobito ANGOLA" Fomento, Lisboa, 7 (1) : 3 - 51

E. B. SOUTO SILVEIRA and J. O. JUNQUEIRA FRANCO (Brazil)

Upon a suggestion of Prof. Victor F. B. de Mello, consistent with the concept summarized in his remarkable State-of-the-Art Report, and in accordance with a practice employed long since on several earthwork projects on which he is the Soil Mechanics Consultant for Promon Engenharia, we have had some opportunities of establishing for practical use general power functions of $\mu = f(\sigma_3, \sigma_1 - \sigma_3)$.

In each case, several tests with different values of σ_3 were carried out, and the individual groups of values of $(\sigma_3, \sigma_1 - \sigma_3, u)$ were statistically analysed, in order to obtain the best fitting surface in the space

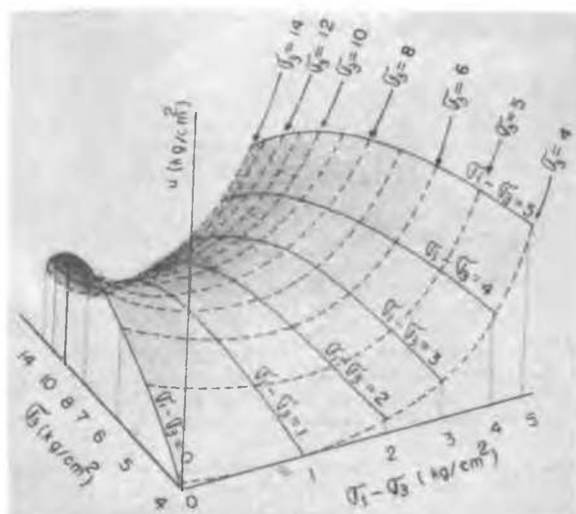


Fig.1 - Best fitting surface

defined by the three above variables. The statistical analysis was developed through a computer program, according to the following steps: given several powers of the independent variables (σ_3 and $\sigma_1 - \sigma_3$) and their combined products, the program made a research on which of them was the most significant one, established a first best fitting function on this single variable, and analysed its statistical significance; if it proved valid, the research continued in the next step, adding each time the next significant value. Thus, the final equation has the form:

$$\mu = a + b\sigma_3 + C(\sigma_1 - \sigma_3) + d(\sigma_3)(\sigma_1 - \sigma_3) + e\,\sigma_3^2 + f(\sigma_1 - \sigma_3)^2 + g(\sigma_3)\,(\sigma_1 - \sigma_3)^2 + h(\sigma_3)^2\,(\sigma_1 - \sigma_3) + i(\sigma_3)^2(\sigma_1 - \sigma_3)^2 + \ldots$$

in which the coefficients a, b, c... may be zero, or not.

The research starts with two terms (power function of the most significant variable), and, at every step, a new term is added, the most significant one. The research finishes when, from an engineering point of view, the benefit introduced by the addition of a new term is no longer significant.

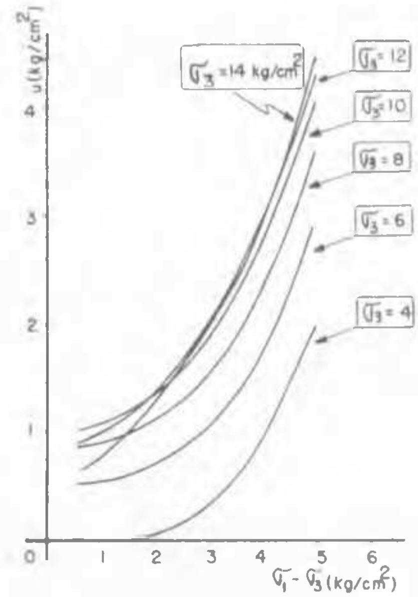
Fig. 1 shows the $\mu = f(\sigma_3, \sigma_1 - \sigma_3)$ best fitting surface determined, which is found to be curved in both coordinates σ_3 and $\sigma_1 - \sigma_3$.

It should be mentioned that, when a surface is established, the values for a hydrostatic pressure ($\sigma_1 = \sigma_3$) would be given by the trace of the general surface of $\mu = f(\sigma_3, \sigma_1 - \sigma_3)$ on the plane of $\sigma_1 - \sigma_3 = 0$, that is, a curve of $\mu = f(\sigma_3)$. However, the statistical analysis seems to indicate that a direct regression on pairs of values of the chamber pressure σ_3 on the samples, prior to the application of any deviator stress ($\sigma_1 - \sigma_3$), does not necessarily belong to the same universe as the surface function of $\mu = f(\sigma_3, \sigma_1 - \sigma_3)$, as may be visualized in Fig. 1b. This means that the pore pressure induced by a hydrostatic pressure on a soil sample does not necessarily follow the same law as that of the pore pressure induced by a deviator stress - which seems to agree with Markus Reiner's considerations about the elasticity theories, on which the pore pressure coefficients are generally based, and about the elastic dilatancy of materials that are constituted by particles.

With a view to establishing the merits of the procedure on the basis of unquestionable experimental data, the exemplifying computations were carried out using 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., Soil Mechanics Series nº 61, 1960, for the $\overline{Q}$11, $\overline{Q}$12, $\overline{Q}$13, $\overline{Q}$14, and $\overline{Q}$16 tests (table IV and Figs. 28, 29, 30, 31 and 33 loc.cit.), with about the same degree of saturation (around 76%). The values of the confining pressure vary from 4 to 14 kg/cm^2.

Figs. 2 and 3 show comparisons between the observed values during the tests with σ_3 = 6 kg/cm^2 (Fig.2) and σ_3 = 14 kg/cm^2 (Fig.3) and the values calculated according to the statistically best-fitting surface - equation of Fig.1. As may be observed, even the highest error, corresponding to Fig. 2, is rather small, even when the equation was established for a relatively large range of the confining pressure values (4 to 14 kg/cm^2).

With the observed values of μ for σ_3 = 4 and 10 kg/cm^2, and with $\sigma_1 - \sigma_3$ = 2 and 3 kg/cm^2,



Traces parallel to the μ x σ_3 plane.

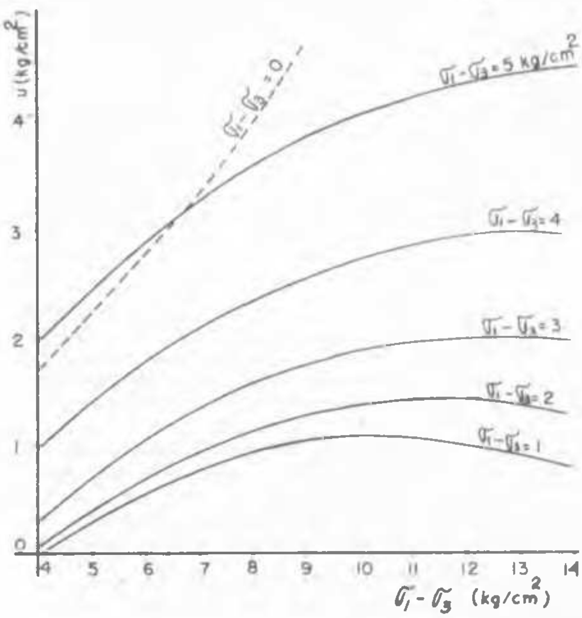


Fig. 1b- Traces parallel to the μ x $\sigma_1 - \sigma_3$ plane the traditional pore pressure coefficients A_1 and B_1 were calculated; with the same values of σ_3, but with a larger range of $\sigma_1 - \sigma_3$ (1 and 4 kg/cm^2), A_2 and B_2 were also calculated.

Fig.2 shows two straight-lines, for comparison with the observed data and with the values taken from the statistical surface of Fig.1: straight-line (1) shows the variation of u with $\sigma_1 - \sigma_3$, for $\sigma_3 = 6$ kg/cm^2 (as the other two curves), and for A_1 and B_1; and curve (2) shows the same variation, for A_2 and B_2.

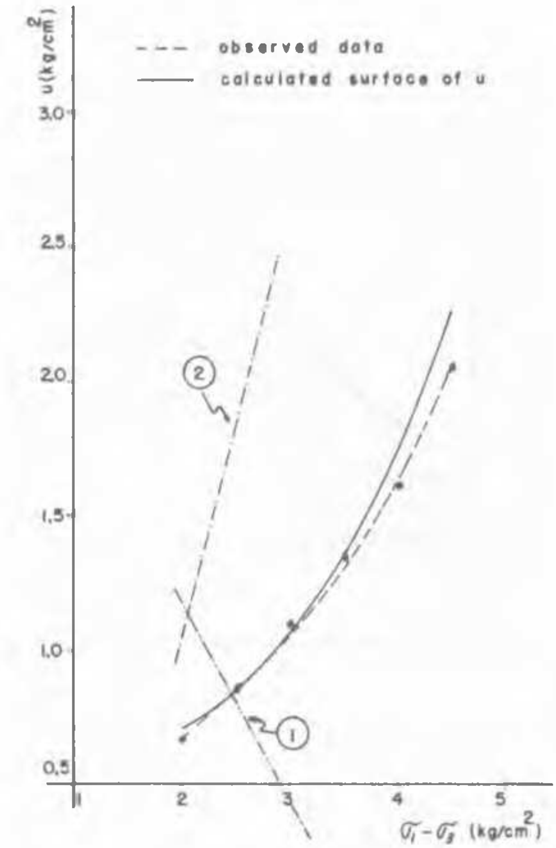


Fig.2- Comparison between observed and differently calculated values for σ_3 = 6 kg/cm^2

These straight-lines show that any computation based on a linearization of the function $u = f\ (\sigma_3,\ \sigma_1 - \sigma_3)$ only gives reliable values when the parameters for computation of the pressure coefficients A and B are very close to the values one wishes to calculate; they show therefore that the heavy responsibility of obtaining a reliable result lies on the choice of appropriate values for the determination of A and B, as mentioned in the State-of-the-Art Report under discussion.

The same procedure was carried out for σ_3 = 14 kg/cm^2, as indicated in Fig. 3.

Although this method has been under general use among us, and the conclusions above exemplified have been repeatedly confirmed

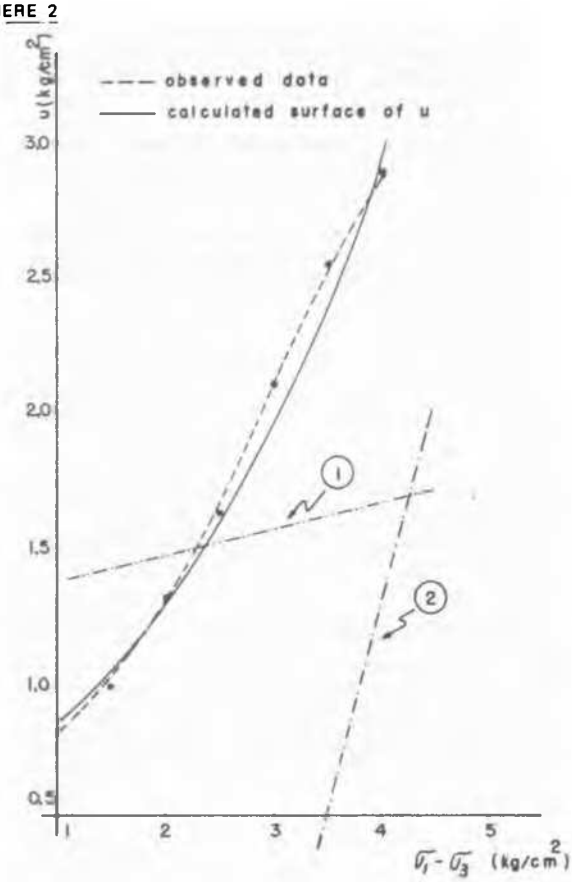


Fig.3 - Comparison between observed and differently calculated values for σ_3 = 14 kg/cm^2

in varying measures depending on soil properties and stress levels, it is felt that the concept of the procedure and its handy computational application is sufficiently documented through the single example presented, based on a very meticulous test program.

A. S. VESIC (U. S. A.)

This discussion is concerned with effects of scale and compressibility on behavior of shallow foundations. Scale effects differing from those predicted by the classical earth pressure theories have been known in bearing capacity and earth pressure phenomena for quite some time. Yet, the understanding of the variety of reasons for their existence has come only in very recent years, (De Beer, 1963, 1965; Vesić 1964, 1965; Kérisel, 1967). It was shown that the relative compressibility of soils, both with respect to gravity forces and with respect to their shear strength, increases with foundation size. The mentioned studies also indicate that, in case of shallow foundations, the average shear strength mobilized along a slip line under the foundation decreases with foundation size. (There are actually three independent reasons for this decrease of strength with foundation size

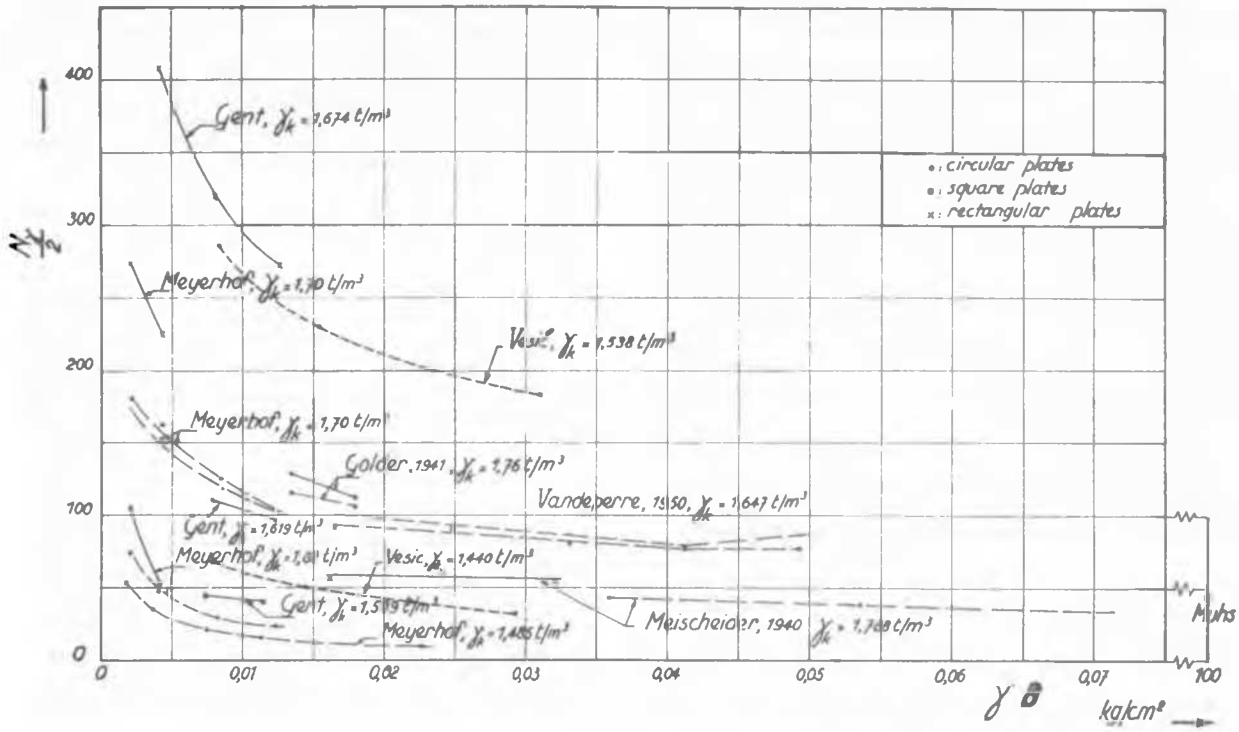


Fig. 1. Variation of bearing capacity factor N_γ with footing size (after De Beer, 1965).

a) the curvature of Mohr envelope; b) progressive rupture along the slip line; c) presence of zones or seams of weakness in all soil deposits. The relative contribution of each of the reasons varies with soil type and the range of footing size; their total effect being discernible in practically all soils.)

In view of these facts a decrease in apparent values of bearing capacity factors with size should be expected, to a certain degree, in all soils. The most conspicuous of all is probably the decrease of N_γ-values with increased size of surface footings on sand. Fig. 1, taken from De Beer, 1965, shows that this decrease has been apparent in all major experimental studies of the problem of bearing capacity of shallow footings. As the largest of these footings has been only of the size of one meter (3.3 ft) square, there is a great practical, as well as theoretical interest in possibly establishing whether the N_γ-values shown in the figure tend asymptotically to some minimum.

Let us examine the two possible answers to this question with the use of data obtained in tests with Chattahoochee sand in controlled conditions (Vesić, 1967). First the four points from Fig. 1 marked "Vesić, γ = 1.538 t/m³" are replotted in Fig. 2, which shows the ultimate resistance q_o of surface footings in ton/ft² *versus* the footing width in ft. The same figure shows in thin lines the q_o-values for large footings according to the third term of the conventional bearing capacity equation. Also shown, in solid line, are the measured penetration pressures of deep circular footings in the same sand. Should the surface plate data in Fig. 1 tend asymptotically to any of the N_γ-values shown, the q_o-values in Fig. 2 would need to exceed the deep penetration pressure. It is hard to conceive that a footing may have a greater bearing capacity at the surface than at great depth. The alternate conclusion is that there is no minimum value of N_γ for large footings and that an upper limit of bearing capacity exists for all surface footings. It can be postulated that this upper limit depends on the relative density or void ratio of the soil in question and that it does not exceed the deep penetration resistance of a footing of the same shape and size. This would suggest that large surface footings may also fail exclusively in punching shear, as apparently all deep footings do. This possibility should not come as a surprise if one considers the mentioned fact that the relative compressibility of soils under footings increases with footing size.

The data presented in this discussion indicate that the conventional theory of bearing capacity of shallow foundations, at present limited to the rigid-plastic idealization of soil behavior and general shear failure pattern of the soil mass, may not always be able to offer an adequate assessment of the scale effects. It is believed that an extension of this theory, to be based on a more realistic model of a compressible solid can provide needed answers for a greater variety of problems appearing in engineering practice.

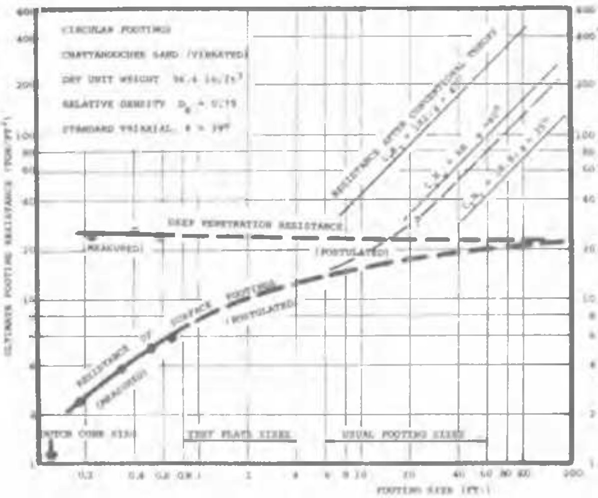


Fig. 2. Variation of ultimate resistance of footings with size.

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E. P. VIDELA and J. R. NADEO (Argentina)

SYNOPSIS

This paper presents the results of compression and pulling tests with slow application of the load carried out on a number of piles used or the construction of a Cold Rolling Mill (Propulsora Siderúrgica Works, Ensenada, Buenos Aires, R.A.). The tests were conducted with total variable loads of up to 200 Tn., and the relationships load-total settlement and load-plastic settlement were recorded for each case. A complete report of the tests has been published in the last volume of Specialty Session No. 8 Memories.

The tests were carried out on square precast concrete piles (35 x 35 cm) driven to a depth of 13.50 m below ground level. Because of the soil stratigraphy and the works characteristics it was necessary first to remove by excavation the upper layer of Type C.H. soft clay, 6.00 m deep. After drilling to a depth of 9.00 m below ground level with a 40 cm bore through the clay-silt strata, Type M.H. very firm and Type M.L. silty and hard, the piles were driven into the soil.

Loads were increased in stages and were maintained at each stage until settlement had substantially ceased before proceeding to the next stage. First a maximum compression load of 80 Tn was reached and was maintained a minimum of 12 hr. before removing it completely. The net settlement or plastic deformation produced by that maximum load was then measured.

After this, loads were increased in stages again until a maximum compression load of 200 Tn was reached. This load was maintained a minimum of 24 hr. before removing it completely and measuring the resultant plastic deformation. The results of the 6 compression loading tests carried out - in 2 of them failure occurred with a load of less than 200 Tn are shown in the Load-Settlement, Settlement-Time, and Time-Load Increments curves presented in our paper published in Specialty Session N° 8 Memories.

The five pulling tests were made mainly to verify the amount of total settlement produced by loads approximately equal to working loads. Loading was increased in stages as before until a maximum of 40 Tn to 60 Tn was reached. The loads were then removed and the plastic deformation determined. In one case a decision was made of producing the failure of the pile, which occurred at 180 Tn.
All tests results were represented in the aforementioned curves. An analysis of these results showed that there was no correlation between the total settlement values measured in different tests, therefore more attention was given to plastic deformation. In all tests - compression or pulling - that had not reached the failure stage the plastic deformation values measured were found to be around the mean value represented in Fig. 1.

This figure shows that the plastic deformations produced by pulling loads are greater than those produced by compression loads, even when these were of a smaller value. If we accept the validity of the relationship pulling load-plastic deformation, it can be concluded that the friction produced during a compression load test for a certain plastic deformation it is a function of the deformation. Therefore, and as long as failure is not reached, it results that about 60 per cent of the load is carried by the tip.

The relationship Load-Plastic Deformation for a compression and a pulling load test that had reached failure was represented in Fig. 2 with the object of analysing the results for greater plastic deformations.

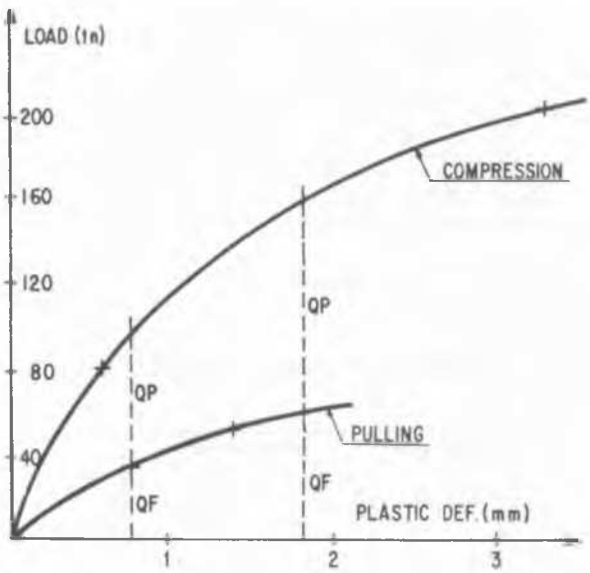

Fig. 1

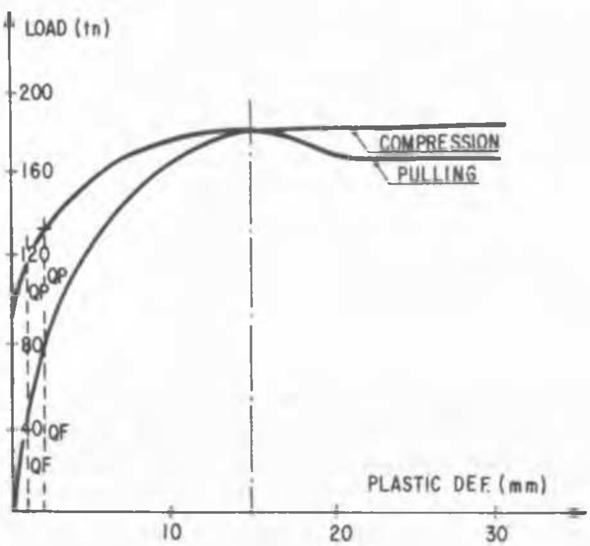

Fig. 2

The study of Fig. 2 shows that until approximately 2 mm the tip carries about 60 per cent of the load applied, the same as in Fig. 1. As plastic deformation increases the load carried by the tip is gradually transferred to friction until all the applied load is taken by friction when deformation is 15 mm, in this particular case.

Thou this is not the more general case - that in which a residual resistance remains in the tip after friction resistance is over - the above assumptions are valid as long as they are applied to similar situations.

CONCLUSIONS

1. Differing from total deformation values, the plastic deformations measured in compression and pulling load tests were quite similar for the same load.
2. The study of plastic deformations seems to be a useful tool to analyse the stress transference phenomenon from pile to soil as tip and friction resistance.
3. For small plastic deformation values the pile tip takes about 60 per cent of the applied load. As plastic deformation is increased friction cooperation it is also increased, becoming more and more important until reaching the failure stage, when practically all the load is carried by friction.
4. In similar situations bearing capacity of a pile could be calculated as the sum of the friction resistance and the residual tip resistance; if this is so the admissible load obtained by applying a total safety coefficient does not represents proportionally the admissible loads taken by tip an friction, and only an acceptable security margin is obtained that the applied loads will not produce failure. Hence, it would be convenient to use parcial security coefficients.

K. WEISS (Germany)

Muhs and Weiss[1] have reported the results of four loading tests with a 2.0 x 0.5 m footing in their paper about the influence of the load inclination on the bearing capacity of shallow footings. It was rather easy to determine the amount of decrease of the bearing capacity due to the inclination and eccentricity of the load by putting the respective failure load in proportion to the failure load of the corresponding vertical central load case. The following conclusions could be derivated therefrom:

1. The influences of eccentricity and inclination of the load are independent of each other;

2. The bearing capacity decreased approximately by the factor tan δ, when the applied load is inclined under the angle δ.

However, these results have been proved so far by the four tests only for an angle of inclination of $\delta = 20^{\circ}$ and an eccentricity e of 1/6 L. Additionally they are only true for the case when the inclined load acts in the direction of the longer side of the footing.

In the meantime sixteen further tests have been carried out in two series with $\delta = 10^{\circ}$ and $\delta = 30^{\circ}$ in a dense sand-gravel mixture, in which also the eccentricity was increased to 1/3 L (Figure 1). The different load cases have been compiled together with the corresponding failure loads in Tables 1 and 2; the coefficients in the last three columns were calculated by dividing these failure loads by those of the corresponding vertical central load case (Tables 1 and 2).

Fig. 1 - Test set up for $\delta = 30^{\circ}$ and e = L/6

Table 1

Test results for $\delta_S = 10^{\circ}$

Test Nr	Arrangement	failure load P (t)	influence of e	influence of δ	influence of e and δ
1	$\delta_s = 0$, $e = 0$ (L = 0,2 m)	77,3	-	-	-
2	$\delta_s = 10^{\circ}$, $e = 0$	64,3	-	0,83	-
3	$\delta_s = 10^{\circ}$, $e = L/6$	46,8	0,73	0,79	0,61
4	$\delta_s = 0$, $e = 0$	28,0	-	-	-
5	$\delta_s = 10^{\circ}$, $e = L/3$	36,2	0,56	0,95	0,47
6	$\delta_s = 10^{\circ}$, $e = 0$	64,2	-	0,83	-
7	$\delta_s = 0$, $e = L/6$	59,2	0,77	-	-
8	$\delta_s = 0$, $e = L/3$	38,0	0,49	-	-

The average reduction values due to the influence of the eccentricity with respect to the inclination of the load are compiled for all tests together with the corresponding variances and coefficients of variation in Table 3.

It can be seen from the values in the last column that the bearing capacity under an eccentricity

Table 2

Test results for $\delta_S = 30^{\circ}$

Test Nr	Arrangement	failure load P (t)	influence of e	influence of δ	influence of e and δ
1	$\delta_s = 0$, $e = 0$ (0,5 m; L = 2,0 m)	114,3	-	-	-
2	$\delta_s = 30^{\circ}$, $e = L/6$	33,9	0,83	0,46	0,30
3	$\delta_s = 0$, $e = L/6$	74,4	0,65	-	-
4	$\delta_s = 0$, $e = 0$	46,1	-		-
5	$\delta_s = 30^{\circ}$, $e = 0$	40,9	-	0,36	-
6	$\delta_s = 30^{\circ}$, $e = L/3$	21,7	0,53	0,41	0,19
7	$\delta_s = 0$, $e = L/3$	52,3	0,46		-
8	$\delta_s = 30^{\circ}$, $e = L/6$	33,9	0,83	0,46	0,30

Table 3

Average test values of reduction

influence of	reduction to
e = L/6	$\mathcal{E}_e = 0,73 \pm 0,03$ $s = \pm 0,07$ $v = \pm 9\%$
e = L/3	$\mathcal{E}_e = 0,51 \pm 0,02$ $s = \pm 0,04$ $v = \pm 9\%$
$\delta = 10^{\circ}$	$\mathcal{E}_{\delta} = 0,84 \pm 0,04$ $s = \pm 0,08$ $v = \pm 9\%$
$\delta = 20^{\circ}$ (see table in paper)	$\mathcal{E}_{\delta} = 0,62 \pm 0,02$ $s = \pm 0,03$ $v = \pm 5\%$
$\delta = 30^{\circ}$	$\mathcal{E}_{\delta} = 0,42 \pm 0,02$ $s = \pm 0,05$ $v = \pm 11\%$

*) variance
**) coefficient of variation

of e = 1/6 L amounts to about 73% and that one under an eccentricity of e = 1/3 L to 51% of the bearing capacity of the centrically loaded footing. When the load application is inclined in the direction of the longer side of the footing the bearing capacity decreases under an inclination of $\delta = 10^{\circ}$, 20°, 30° to approximately 84, 62 and 42% respectively of the bearing capacity of the vertically loaded footing.

For the decrease $\mathcal{E}_\delta$ of the bearing capacity due to the inolined load, it has been proposed in the paper by Muhs and Weiss the equation $\mathcal{E}_\delta = (1-\tan\delta)$ based on the tests with 20° load inclination. The oomparison of the herewith oalculated values $\mathcal{E}_\delta$ for 10, 20 and 30° ($\mathcal{E}_{\delta,10^\circ}=0.83$, $\mathcal{E}_{\delta,20^\circ}=0.64$, $\mathcal{E}_{\delta,30^\circ}=0.42$) with the deorease $\mathcal{E}_\delta$ resulting from the tests shows practioally a ooaplete agreement, so that the applioation of this equation for loads inclined in direotion to the longer side of the footing seems to be justified for all angles of inclination δ being used in praotice.

REFERENCE

1. H. Muhs and K. Weib (1969), "The influence of the load inclination on the bearing oapacity of shallow footings", Proo. Seventh Int. Conf. on S. M. and F. E., Mexico, Vol 2, pp. 187-194