Deep Foundations

Fondations profondes

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Chairman: P. C. RUTLEDGE (U.S.A.)

Ladies and gentlemen, I declare this session of the Sixth International Conference open. Today we have an unusual opportunity to hear from a true scientist. Our speaker this morning is a palaeo-botanist and is Chairman of the Department of Botany of McMaster University in Hamilton, Ontario.

It seems a bit unusual that we should be having a botanist speak to us, but Dr. Radforth was persuaded to become interested in the engineering aspects of muskeg by our General Chairman, Robert Legget, in 1945. He has since devoted most of his time to the study of this subject and has carried on, I believe, both botanical and engineering research under the sponsorship of the National Research Council of Canada and the Defence Research Board. Our speaker actually is known all over the North American continent as Mr. Muskeg. So, we are going to hear from Mr. Muskeg about muskeg. Dr. N. W. Radforth.

(*Dr. Radforth's lecture appears on pp. 149–54.*)

CHAIRMAN RUTLEDGE

I am sure that all the members of the International Society of Soil Mechanics and Foundation Engineering are deeply grateful to Dr. Radforth not only for a graphic and interesting description and discussion on muskeg, but for a beautiful presentation of the application of the true scientific method to engineering problems. Thank you very much Dr. Radforth.

Since it is Monday morning, after a pleasant weekend, we will run a little bit late in our programme and take a 10-minute break.

(*There followed a brief intermission.*)

CHAIRMAN RUTLEDGE

Members of the International Society of Soil Mechanics and Foundation Engineering, it is my privilege to open this Sixth Technical Session of the Sixth International Conference on the subject of deep foundations and, more specifically, of pile foundations. This subject is an illustration of what seems to me to be an unfortunate divergence into two branches of our society: the highly restricted theoretical and laboratory investigations of soil mechanics on the one hand, and the practical problems of foundation engineering on the other hand. The design and installation of pile foundations is an intensely practical problem. Our distinguished General Reporter has tabulated 215 papers on the subject of piles and pile foundations that have been presented at 10 international conferences over the past thirty years. Over this same thirty years I have had personal contact with the design and/or installation of over 1000 projects involving pile foundations. With considerable regret I am forced to the conclusion that a relatively small number of the 215 papers have been usable or used by the practising foundation engineer. This is unfortunate both for the engineer and for the people who are doing the work and producing the papers. I think that the answer lies in a more careful consideration in research of the subsurface conditions where piles would form a practical foundation solution, of the ways in which piles are actually installed and used in foundations, and of the performance of the real structures that are supported on pile foundations. There are a great many problems in the design and construction of actual pile foundations. Almost every pile foundation project involves the determination and resolution of things the development of which are uncertain, uncertain at least until they are resolved on the job and the job proceeds. My plea is only that our work, our research, and our papers in this subject will concentrate more strongly on the observations and measurements of actual pile foundations in the natural soil conditions under which piles are used in practice. It is now my pleasure to introduce our General Reporter for this session, Dr. Kézdi, who is Professor of Soil Mechanics at the University of Budapest in Hungary.

General Reporter: Á. KÉZDI (Hungary)

The pile foundation has a very long history in building construction. Prehistoric lake dwellings, Roman temples, and the cathedrals of the Middle Ages all made use of this foundation element and it always proved itself to be the most reliable and most economical deep foundation. The technological age brought new and rapid development: even since the establishment of soil mechanics we can observe new and bright achievements in this field.

The reasons for the increasing importance of pile foundations are obvious. First, the weight and size of modern structures continue to grow, and the bearing capacity of the upper layers with average strength is not sufficient to carry these great loads. Secondly, it is more and more difficult to find a site where the subgrade is suitable for this type of construction. And thirdly, much of the rapid development of the last decades has taken place in areas where the subgrade consists of a thick deposit with low bearing capacity. Today there is a shortage of soils with high bearing capacity!

The ever increasing importance of deep foundations is reflected in the literature and in the conventions of soil mechanics. I prepared a table for my General Report listing ten conferences, held in the past thirty years, where the problems of piles and piled constructions have been dealt with to a considerable extent. It is evident from that table that the interest in this field is increasing steadily. This is shown, for example, by the increased proportion of the papers on piles at the international conferences (Fig. 1.); the Montreal Conference has the highest percentage. As well, there were four conferences which have dealt exclusively with the problems of piles.

However, this increasing number of conferences has its dangers, too. The difficulties, outlined by President Casagrande in the Opening Session, of keeping pace with the rapid growth of information are felt more and more, and if we examine the frequency of conferences, we can see (Fig. 2) that the interval between conferences is decreasing rapidly. If this trend continues we shall soon be discussing our problems at a permanent and eternal conference! I know that World War II is responsible for the first part of the curve, but even so the trend is there. This reminds me of the old story about the veteran soldier of World War I who told his countrymen about a battle in which he had participated

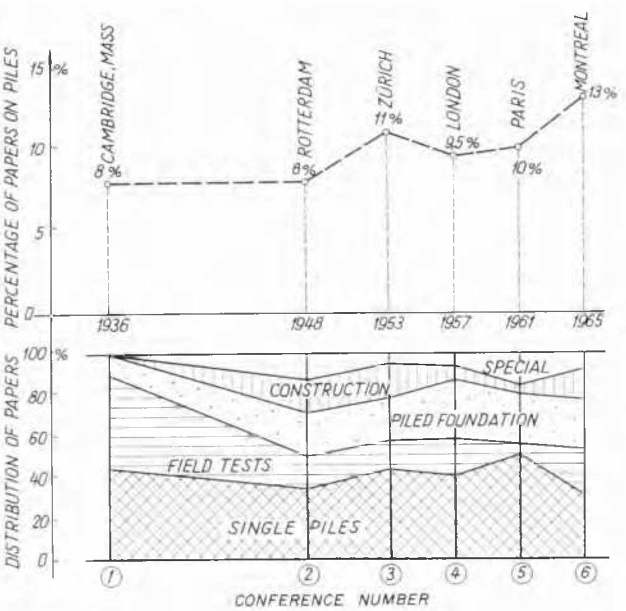

FIG. 1. Papers presented on piles at the international conferences.

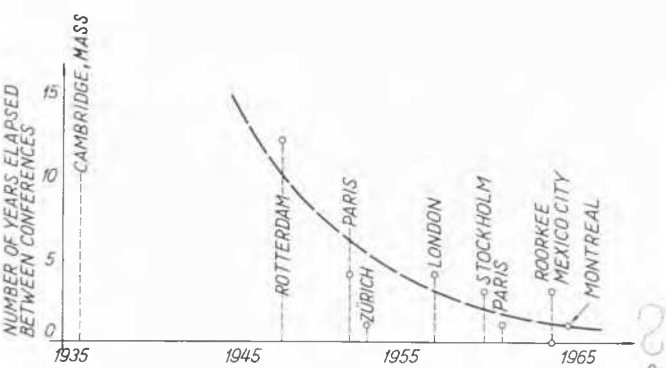

FIG. 2. Frequency of conferences.

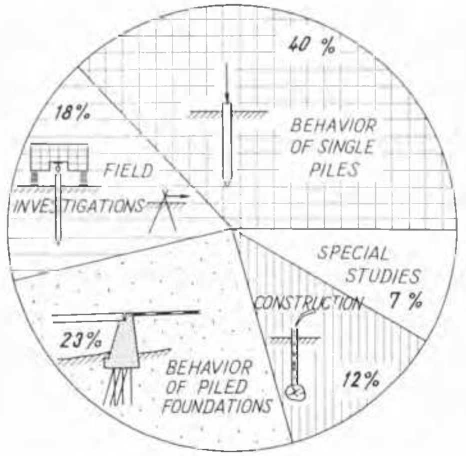

FIG. 3. Distribution of papers on piles among the different themes.

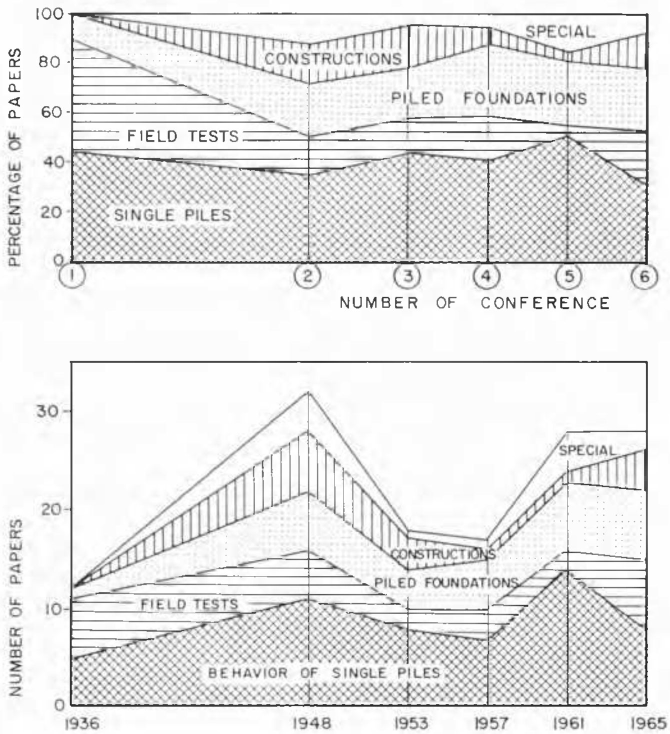

FIG. 4. Variation in distribution of papers on piles at the international conferences.

where they loaded the rifles and shot, loaded and shot again, and finally did not load at all, but just shot! Sometimes I have the feeling that we research and report, research and report, etc.

It is also interesting to note the rate of increase of the papers presented at all of these conferences. Today the rate is three times higher than in the early 'fifties. Unfortunately, the quality of the papers has not improved in the same measure as the quantity: there is much repetition and many papers which would get a rather low rating on the system proposed by Professor Casagrande. The trouble is that in most cases we have to read the paper before we can rate it. In the statistical evaluation, I could not give a rating to the papers, because I think that a rating might be unfair if given many years after the paper is written.

The distribution of the papers on piles among the different themes is presented in Fig. 3. For the classification of the themes I made use of the system proposed in my General Report. The majority of the papers belong to the first group: 40 per cent of the individual contributions fall into this category. It is interesting to examine how the number and the percentage of the papers in the five groups varied in time, at the different conferences (Fig. 4). In my General Report, I expressed my regret that the percentage of the papers dealing with field tests and measurements has decreased up to the Fifth Conference. I also regret the low percentage in the second group, because this group could be the treasury of well-documented case histories and evaluated experiences with piles.

Let us now examine the problems listed in the first category.

BEHAVIOUR OF SINGLE PILES

We only know of a few attempts to solve the basic problem of describing the behaviour of a single pile under the action of a load acting along its axis. It is customary, in investigations of spread foundations, to design a footing in two subsequent steps. We first investigate the limit equilibrium of the system; then, after the application of a suitable factor of safety, we determine the design load and also the deformations, the settlements. Unfortunately, this method is rather artificial and theoretical in the case of piles.

The reason for this is that we very seldom have a case where general failure occurs: the loading diagram does not have a vertical tangent and the end tangent is a slanting line. A given factor of safety would furnish different design loads for the same pile, depending on the value of the load on the test pile. The separate calculation of the surface friction and the point resistance as limit values, and, subsequently, without taking the settlements into consideration, the addition of these parts taking the sum as the ultimate bearing capacity is definitely a bad procedure. It neglects the process of mobilization of skin friction and point resistance. These are definitely interdependent and depend, in turn, on the settlement and compression of the pile. The initial resistance to deformation is predominantly from surface friction, but for larger settlements of the piles, point resistance becomes the dominant factor in the total resistance as ultimate load is approached. This behaviour underlines the importance of the requirement that the whole process of loading be investigated. We must also keep in mind that this process is not independent of the manner of installing the pile or of construction methods.

At present, the solutions available for this problem are rather rough approximations and they have been worked out only for some special subsoil conditions, sometimes for cases that are not encountered in practice. There is an urgent need to establish some well-defined practical cases for the application of the piles and to investigate them with regard to the loading diagram. I agree, of course, with those colleagues who emphasize the practical and theoretical difficulties encountered in studies of this sort; however, it is better to develop in this way than to make virtue out of necessity and to establish highly sophisticated theories without taking actual behaviour into account. The aforementioned typical practical cases would include a pile penetrating into a soft layer and standing on a lower layer of high bearing capacity, bored and driven piles and also some modern construction techniques, piles in more or less homogeneous material, and the variations in conditions of the pile head (fixed or free). The stresses acting on the mantle surface of piles are, particularly in clays, time dependent, a factor which has also to be accounted for.

The solutions would describe, then, the behaviour of the pile in a manner that can be used in practice. This means a method which, for a given soil profile and certain physical characteristics, enables us to predict both the probable loading diagram of the pile and the distribution of the pile load along the pile as a function of the total load.

In the field of investigations aimed at the determination of the behaviour and bearing capacity of single piles, the papers presented to this Conference point out the right way for research, at least with regard to the methods used. No mention was made of the pseudo-scientific and pseudo-theoretical pile driving formulae: I gladly agree with that, since the shortcomings and weaknesses of these have been pointed out some 25 years ago by Cummings. However, from year to year, "new" and "improved," "reliable" formulae have appeared in the literature and in practice. Fortunately, this is not the case now which supports the opinion of your General Reporter, that there is no possibility of predicting the bearing capacity and settlement of a pile under static loads from pile driving formulae!

PILE GROUPS

I proposed, as a second item of discussion, the bearing capacity and behaviour of pile groups. Investigations in this field began, like the work on the loading diagrams, quite recently. In this field, we again have a method, which, like the pile-driving formula, was well established with civil engineers and which may be regarded almost as a "superstition": I am speaking of the efficiency formulae of the past that solved the problem in a very simple, mechanical way, without taking the essential elements and factors into account. It is true that the settlement of one pile depends only on the soil surrounding the pile, but the settlement of the complete pile foundation depends on the number and spacing of the piles, on the method of construction and, above all, on the compression of the layers underneath the pile tips. The "efficiency formulae" take care of only the number and the spacing of the piles.

Pile groups are also investigated by a statical method, similar to that mentioned in my report. The role of these theories and methods in soil mechanics is like that of the theories of elastically supported, load bearing structures, using a coefficient of subgrade. The simplifying assumptions make a clear-cut mathematical solution possible, a much more elaborate one than that could be warranted by the assumptions. The fact that no paper was contributed to this conference which used these methods shows that I am not alone in this opinion.

In general, establishing the value of the bearing capacity of pile groups is a very expensive research item. Therefore I think we should use the one-to-one model tests, the measurements on existing structures. Settlement observations, and eventually built-in pressure cells furnishing continuous records on the movements, forces, and stresses, could help tremendously to set up the framework which is necessary for a theoretical solution. The cost of these measurements is a very slight fraction of the cost of the foundation and many large construction firms or even design bureaus could give a great deal of help to the profession by making these measurements. Settlement observation is particularly important in clay soils, as has been shown by the papers on this subject presented to this conference. At the present state of our knowledge, this may be the most important prerequisite for future development. With regard to testing, an extensive testing programme could be carried out on the basis of international co-operation as a real team effort.

Returning to the papers presented to this conference, it is probably safe to say that piles driven in clay are of limited value in reducing the settlement of the structure. Pile groups composed of piles driven in loose sand may have a greater bearing capacity than the sum of those of the individual piles, provided that the spacing of the piles is smaller than a given upper limit. In dense sands, nobody would drive piles! The cases which are the most important in practice, those where we have a lower layer with high bearing capacity and an upper layer with low bearing value, have not been investigated yet.

The experimental approach would be highly desirable for cases involving other than vertical loads and piles. In this field, the behaviour of the single pile, acted upon by lateral loads and moments, is much less clear than that of the vertical pile under vertical load. Here again, the measurement of the horizontal and vertical displacements and also the measurement of the stresses and forces in actual construction, set forth in several papers presented to this conference, will furnish the basis for a new theoretical treatment. The application of the coefficient of subgrade reaction to the problems of piles loaded with horizontal forces is not justified, and any theory making use of this concept represents only a rough approximation.

This brings me back to what I said about superstitions with regard to pile bearing capacity. The pile driving formulae, the efficiency formulae, and the term "coefficient of subgrade reactions" might have served the needs of former structures and given satisfactory results; however, their application today to modern construction with large dimensions, greater forces, new construction methods, and higher requirements is an unwarranted extrapolation. Therefore, we have to find methods which will give a sound basis for the design engineer of today. I feel strongly that many of the papers presented here give evidence that we are already on our way to this goal and also that the discussions today will add something to our knowledge.

(*Dr. Kézdi's General Report appears on pp. 256–64.*)

CHAIRMAN RUTLEDGE

Thank you Dr. Kézdi for your interesting and informative report.

I would like to introduce the members of the panel who are going to give us discussions today. On the right, Dr. Zeevaert of the University of Mexico, who is also our Vice-President for North America. Next to Dr. Zeevaert, Dean Meyerhof, whom you all know as a very active member of the committee that organized this conference. Next to Dr. Meyerhof, Dr. van der Veen, who is Director of the Department of Water Supply for the City of Amsterdam, and on the left, Mr. Parez, who is a contractor engaged in pile work in France. The panel discussions this morning are going to be restricted to topics (a), (b), and (d) in the printed programme. We will begin with discussion of the first topic—stresses and deformations in and around piles showing vertical loading and different soil profiles—and I will call on Dr. van der Veen to initiate this discussion.

Panelist: C. VAN DER VEEN (Netherlands)

I have been asked to say a few words on stresses and deformations in and around piles during vertical loading in different soil profiles. The problem of stresses and strains in and around piles, even if we limit ourselves to a vertical loading, constitutes such a vast field that I can touch upon only a few points. Because of the limited time available I shall not go into too much detail but will limit my remarks to some general comments.

First of all the basic question itself: what are the stresses and strains around a pile when loaded. The purpose of a pile is to transfer loads imposed by the structure to suitable soil layers. This we always have to keep in mind. Most of the time the stresses around the shaft of the pile and the point are considered separately. This is an approximation, however, and if we are to treat the problem from the point of view of more basic principles we should follow a more general procedure. Actually we have to deal with a vertical load on a half-space that consists of two materials, one of which is in the shape of a pile, on top of which a load is acting. The two materials have different elastic and plastic properties.

First of all then, to answer question given, we should know these properties. As to soil, we have not so far succeeded in describing its properties in simple rheological laws, where relations between stresses, strains, and time are given. We know that there is not a linear stress-strain relationship, that some of the deformations develop with time, and also that some of the deformations remain when the stresses have ceased to work. This alone makes a theoretical approach very difficult. But there is more.

The homogeneity of the soil changes when a pile is introduced because some of the soil has to be displaced horizontally and also vertically in order to allow the pile to penetrate. Such changes are different in a loose sand and in a dense sand. In clayey soils, pore pressure is increased through pile driving and it depends on the permeability of the clay how much time it takes before the excess pore pressure has dissipated. The pile which has been introduced is by no means always without stress. In some cases it has been observed that due to the deformations of the pile during driving, frictional forces between the pile and the soil will occur which remain when pile driving has been finished. The total sum of these vertical forces equals the weight of the pile, but the distribution of this friction along the pile is not uniform. It also makes a difference how the pile has been inserted in the soil, e.g., driving, boring, or screwing.

Then we have to confess that we do not know, at least not completely, the stresses and strains in the soil even before the pile has been placed. In particular we do not know the relationship between the vertical and horizontal effective stresses, a relationship which is important with respect to the bearing capacity of the pile. This point is stressed in the paper by *Kérisel, et al. (4/9)* who point out that the horizontal compressibility is of great importance.

If we have a soil which is not consolidated, stresses and strains are not fixed quantities but will vary with time and thus influence the bearing capacity of the pile, causing negative skin friction, for example. Negative skin friction can reach very high values; this has been observed in the Netherlands and the paper by *Johannessen and Bjerrum (4/8)* gives a similar result for a pile founded through a soft layer in rock. The shape of the pile has to be taken into consideration. It may be cylindrical or tapered, or have an enlarged tip or a cuff, as described in my own paper *(4/23)* to this Conference.

Begemann's paper *(4/1)* stresses the importance of the loading programme: the load can be applied quickly or slowly, or be repeatedly taken away and applied. All this has an influence on the stresses and strains around the pile.

I have still to mention, and you are aware of it, that the properties of the material of which the pile is made of are equally important. We know that reinforced concrete does not show a linear stress-strain relationship and has a plastic behaviour.

Summing up, Mr. Chairman, the stresses and strains in the very simple case of a vertically loaded pile in soil are influenced by many factors: the properties of the materials involved, the homogeneity of the soil, the stress history of the soil, the change being caused in the soil stresses and strains by inserting the pile in the soil, the occurrence of pore pressures. It is no wonder that no one has ever tried, or succeeded so far, in solving this problem in a general way. We know however that valuable results of tests are available.

Mr. Chairman, the question is this: what must be done to approach a better solution?

First of all, we must increase our knowledge of the basic laws of soil mechanics. I think a lot of research has to be done on that subject and it is essential for this purpose that money and people be available. We know, and it is demonstrated by the contributions to this Conference, that a large amount of very intelligent and careful research has been done and is being done in this field.

However I feel that we should do much more. There is a danger that in our branch of technology, meaning the building profession which is the oldest technical profession, we are lagging behind when compared with many other similar branches such as electronics, chemistry, and aeronautics. Much more basic research is done in these fields which means that progress in them is much more spectacular than in our own branch. It is my opinion that our research should be organized more comprehensively, that there should be more co-operation and more long-term planning so that new research can be grounded in the results obtained from earlier investigations. As it is now, very often research is merely a repetition of investigations already performed. I have a strong feeling, Mr. Chairman, that unless we change our habits on this point we will come to the conclusion in perhaps a short time that soil mechanics has not made much progress in the past few years.

Even under ideal circumstances, however, it will take a very long time before we will have solved the basic problem of stresses and strains in soil. Therefore, in the meantime, I strongly emphasize that to fulfil the immediate needs of the construction engineer we must continue to perform tests for simplified cases, such as pure friction piles or pure end bearing piles. The General Reporter, in his report, made an important suggestion, in proposing to establish recommendations for loading tests on piles.

In my opinion the most interesting case for the practising engineer is that in which the ultimate bearing capacity is reached, because this is a simple case to investigate and it also allows for a fair estimate of the safety of the foundation or the probability of failure. Summarizing, we should concentrate on solving that part of the problem first and combine theoretical considerations, while they are being developed, with simple field tests that give quick and reliable data about the soil conditions and properties, in so far as they relate to the bearing capacity of a pile.

I am very much aware that I had to condense my remarks up to a point where they perhaps are not sufficiently clear, but I hope this will serve as a contribution to the discussion. Thank you very much for your kind attention.

CHAIRMAN RUTLEDGE

Thank you Dr. van der Veen. Dean Meyerhof, would you care to comment briefly on Dr. van der Veen's remarks?

Panelist: G. G. MEYERHOF (Canada)

Dr. van der Veen has indicated some of the problems associated with determining the influence of piles on the stresses in the soil. Some years ago, we made some field investigations in uniform sandy soils with penetrometers near single piles which had parallel shafts and in some cases had parallel shafts with an expanded base or displacement casings. These indicated the changes which occurred near the pile, extending to a distance approximately three or four times the pile diameter from the centre in a horizontal direction, and approximately four to five times below the base. We could in this way also distinguish approximately the zones of intense shear deformations in the piles, which are roughly one-half of the limits indicated above, but an analysis as Dr. van der Veen mentioned previously is extremely complicated since the elastic and plastic theory has to be combined with the real deformation properties of the soil.

CHAIRMAN RUTLEDGE

Thank you, Dean Meyerhof.

Piles are rarely used as single piles in foundations; they are almost invariably in groups. As Dr. Kézdi has pointed out, there seems to be a growing number of papers concerning pile groups, and properly so. Our second topic is a discussion of the performance of pile groups. I will not bother to read it; you have it in your programme. I am going to ask Dr. Zeevaert to open the discussion on this subject of the loading of pile groups.

Panelist: L. ZEEVAERT (Mexico)

Upon the suggestion of the General Reporter that I talk on pile-soil interaction in pile foundations, I take pleasure in presenting to you some conclusions drawn from my professional experience. The practising foundation designer is interested mainly in obtaining the first significant figure of the problem that will enable him to predict with safety the behaviour of a pile foundation. This discussion will only apply for a large group of piles and is intended to visualize, in a simplified form, the mechanics of pile-soil interaction—the factors involved and, therefore, the magnitude of the problem encountered.

The bearing capacity of a large pile foundation is mainly related to the settlement or differential settlement that such a foundation may undergo upon load application. Since it is impossible to test a large group of piles, one is compelled to study the behaviour of a prototype and correlate this with the working hypotheses used to design a pile foundation. To illustrate the principles involved in obtaining the order of magnitude in the behaviour of large pile foundations, I will take the case of a friction pile foundation in a compressible material like clay, silt or silty clay.

It has been my experience that for the good performance of friction pile foundations, the foundation structure should be rigid or semi-rigid and the selected rigidity compatible with the superstructure. When this is the case, the pile and soil mass are forced to work as a unit. Assume, Fig. 5, a pile foundation consisting of a large number of long piles capped with a rigid foundation structure. The piles are driven into a firm soil deposit in such a way that they will experience a uniform and very small penetration under safe working loads.

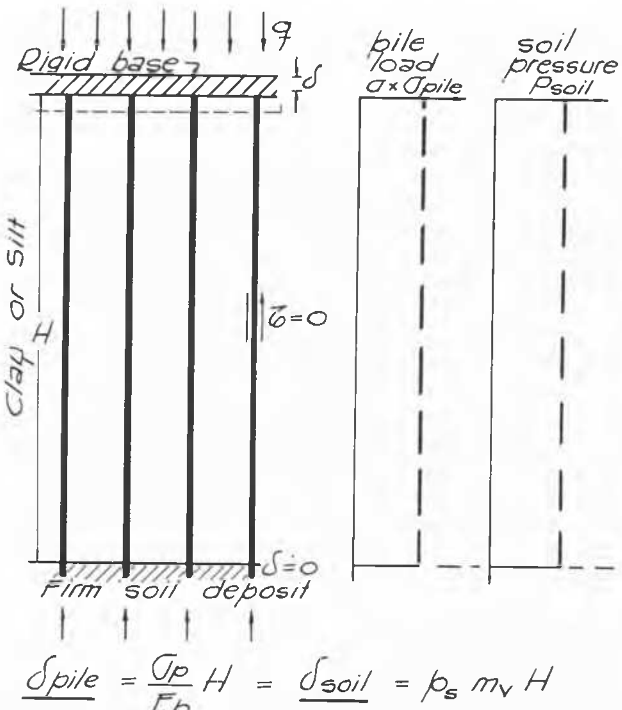

FIG. 5. Point bearing piles.

Under these conditions, it is important to investigate the deformational behaviour of piles and soil so that when the load is applied, the piles and soil between the piles will experience the same vertical displacement; that is to say:

$$\delta_{\text{pile}} = \delta_{\text{soil}}. \tag{1}$$

This condition requires zero shearing stresses between pile and soil. The share of vertical stresses taken by the soil and that taken by the pile will depend on the compressibility of the soil, the modulus of elasticity of the pile, the number of piles, and the cross-section of the piles used. Hence

$$\begin{aligned} p_{\text{soil}} &= q[1/(1+\alpha\beta)] \\ \sigma_{\text{pile}} &= q[\alpha/(1+\alpha\beta)], \end{aligned} \tag{2}$$

in which

$$\alpha = m_v E_p$$

$$\beta = \frac{a = \text{pile cross-section}}{\alpha_t = \text{soil tributary area of pile.}} \tag{3}$$

Knowing the mechanical properties of the material and the geometry of the problem, one may calculate the pile loads and thus the order of magnitude of the vertical displacement. In this case, it will be small and will be governed by the material and length of the piles, that is to say:

$$\delta_{\text{pile}} = (\alpha_p/E_p)H. \tag{4}$$

The case mentioned above is, of course, the simplest case of behaviour of a large group of piles with their points bearing in a firm subsoil deposit.

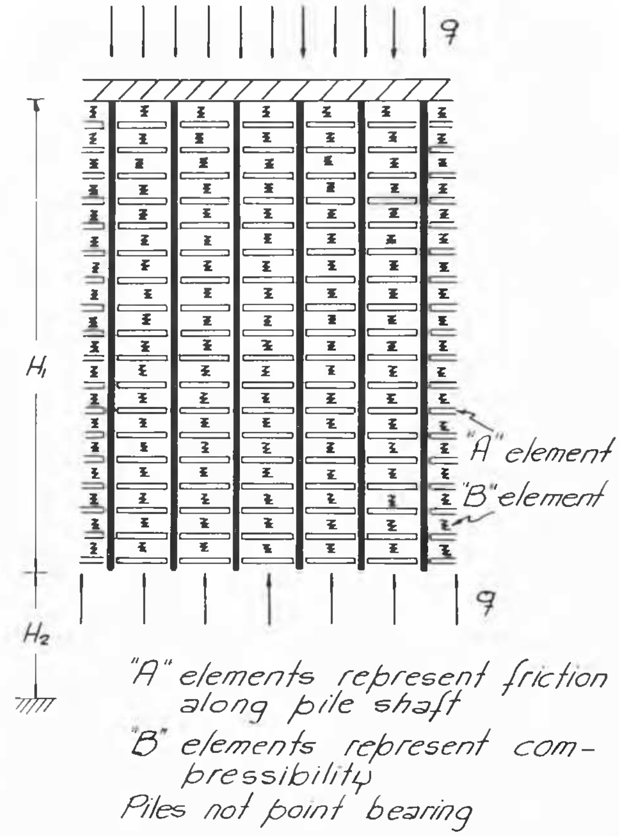

FIG. 6. Simplified model of soil mass and friction piles.

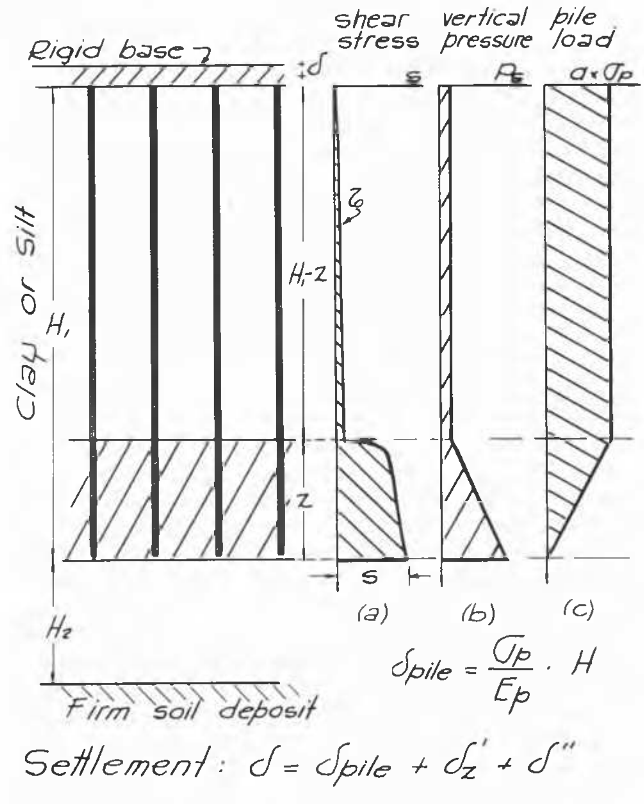

FIG. 7. Friction pile.

To better visualize the mechanics of a large friction pile foundation, let us imagine a simplified model of the soil mass and piles, Fig. 6. Here we are replacing the soil by rigid elements "A" that can develop friction along the pile shafts and elements "B" that represent the compressibility of the soil. Under these circumstances, we may visualize that when the piles are not point bearing, as is the case in compressible materials, the total unit load q will be transmitted to the soil at the bottom of the piles, and the first plates of our model at the bottom are forced to take the load upon mobilization of all friction resistance between plates and pile. The result of this experiment is shown in Fig. 7, where it may be seen that the soil will consolidate within the piles to a certain distance Z until static equilibrium is reached. For this to take place, a relative displacement is necessary between pile and soil; therefore, the ultimate friction is mobilized in the zone shown in Fig. 7. The remainder of the soil mass and piles to a distance $H_1 - Z$ will be in a similar condition to that explained before in Fig. 5. Therefore, the shearing stresses between pile and soil may be considered negligible for practical purposes, Fig. 7a. Hence, the settlement of the foundation is a function of the following values:

(a) δ_{pile} = small elastic deformation of the piles under the share of load they are carrying in distance $H_1 - Z$;

(b) δ'_z = consolidation of the soil mass within the piles to a distance Z where the positive friction was mobilized to obtain equilibrium; and

(c) δ'' = consolidation of the compressible material under the pile tips in the thickness of the compressible layer H_2. Hence

$$\delta = \delta_p + \delta'_z + \delta''. \tag{5}$$

From the above discussion, it will be recognized that the load carrying capacity of a friction pile foundation depends largely on the allowable settlement of the foundation. However, when a large group of piles is driven into a saturated compressible soil deposit, expansion of the soil mass takes place because of volume soil displacement produced when the piles are driven. High hydrostatic pressures are induced in the soil pores, and since the total pressure in the soil mass remains constant, a large reduction in the effective soil pressures takes place. Therefore, the soil mass expands; upon dissipation of the excess pore pressures the soil tends to resume its initial position. However, this phenomenon cannot take place freely any more, since the piles will interfere; consequently, negative skin friction develops. There will be a stress transfer from soil to piles, and the piles will have to take higher loads. The zone of positive friction mobilization, Fig. 8, in the lower part of the piles will be increased correspondingly. The friction along the pile shaft will be now similar to that shown in Fig. 8a, and the vertical pile load as in Fig. 8b. The settlement of the foundation in this case will be

$$\delta = \delta_p + \delta''_z + \delta''. \tag{6}$$

(a) δ_p = small elastic deformation of pile in zone $H_1 - Z$ of negative friction because of total load of pile added by negative friction;

(b) δ''_z = consolidation of soil in zone Z because of positive friction mobilization;

(c) δ'' = consolidation of layers under the pile tips in thickness of compressible layer H_2, as before.

The figures shown are only indicative of the stress configuration along the pile. The configuration, however, depends on the mechanical properties of the materials encountered in the different layers pierced by the piles.

In Mexico City, large friction pile foundations have been designed using this philosophy. Settlement observations of the behaviour of these foundations show good agreement with calculations, thus demonstrating that the simplified assumptions mentioned here provide a valuable tool to estimate the order of magnitude of the problem.

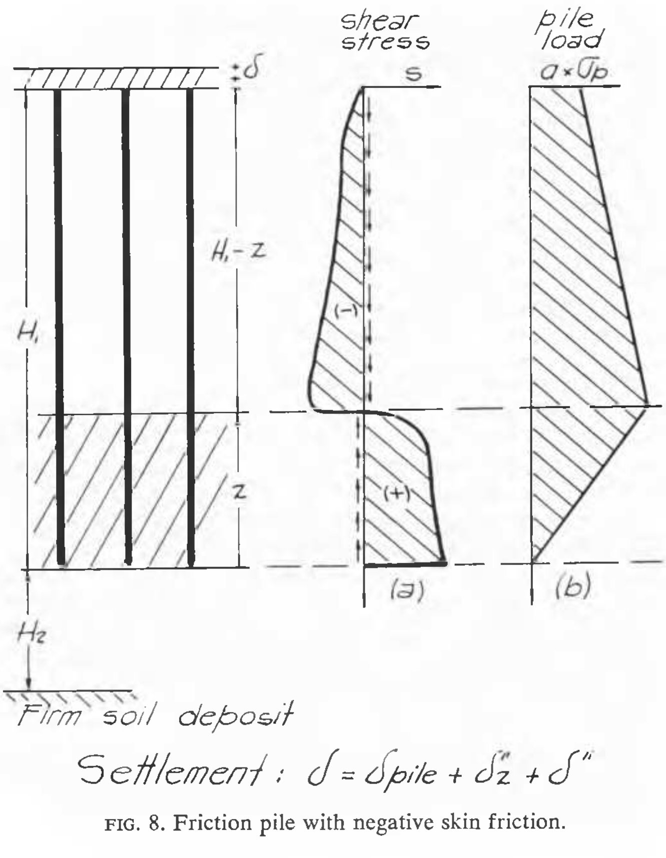

FIG. 8. Friction pile with negative skin friction.

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CHAIRMAN RUTLEDGE

Thank you Dr. Zeevaert. Dr. van der Veen, do you have any comments on Dr. Zeevaert's presentation?

PANELIST VAN DER VEEN

I stressed in my first contribution to this panel discussion that it is of great importance for the constructing engineer that simplified cases of pile foundations be investigated. I think Professor Zeevaert has made an excellent contribution in this field, even more so as the buildings erected by him in

Mexico City, of which the foundations were constructed according to his design, are a proof what good results can be obtained in this way. Of course we should realize that simplifications, necessary as they are, may not be valid in all cases so that we always have to be sure that such simplified assumptions as are made are justified in the particular case under consideration. This also holds for varying soil conditions. For instance, in sandy soils it is important to consider whether the density of the sand is higher or lower than the critical density. But I think this is a point on which Professor Meyerhof is the expert, so I will limit my remarks to what I have now said.

CHAIRMAN RUTLEDGE

Continuing on this same subject of pile groups, I am going to ask Dean Meyerhof to present a discussion.

PANELIST MEYERHOF

The earlier panel discussion on the behaviour of different piles under vertical loading in various soils has shown that fundamental knowledge about single driven piles is still rather limited. It may, therefore, seem premature to discuss the interaction between pile groups and soil. However, this problem is of such great practical importance that the principal factors influencing the bearing capacity of driven pile groups and the effect of pile caps should be discussed, as suggested by the General Reporter.

BEARING CAPACITY OF FREE-STANDING DRIVEN PILE GROUPS

The total ultimate bearing capacity of driven pile groups with caps clear of the ground, or free-standing pile groups, is frequently considered to be the smaller amount of either the sum of the bearing capacities of the individual piles or the bearing capacity of an equivalent pier foundation. Full-scale loading tests on such groups driven into soft clay by Masters (1943), as well as similar model tests by Whitaker (1957) and Sowers *et al.* (1961) showed, however, that for the usual pile spacing of about 2½ to 3½ pile diameters the group capacity in uniform clay is governed by individual pile failure at an average pile load of only about two-thirds of the maximum single-pile value reached at a spacing of about 7 or 8 pile diameters. This difference can be explained by the overlapping of the individual zones of shearing deformation in the soil near the piles, which leads to a reduced skin friction of interior piles of the group while corner piles fail practically at the maximum single-pile load. Thus, for a given pile spacing the bearing capacity per pile in a group in uniform clay decreases as the size of the group (number and length of piles) increases. A qualitative analytical model of such group action of closely spaced friction piles driven into clay has been given by Taylor (1948) but a quantitative solution of this complex problem of the deformation pattern of loaded soil reinforced by piles has not yet been obtained. For an exceptionally close pile spacing of less than about 2½ pile diameters, the bearing capacity is governed by that of an equivalent pier and can readily be estimated from the base resistance plus the perimeter shearing force minus the enclosed soil weight.

On the other hand, field loading tests on free-standing groups of piles driven into loose sand by Cambefort (1953), as well as similar model tests by the General Reporter (Kézdi, 1957), Stuart *et al.* (1960), and others reported to this Conference, showed that for the usual pile spacings the group capacity in originally loose sand is governed by individual pile failure at an average pile load of about twice the maximum single-pile value reached at a spacing of about 6 or 7 pile diameters. This group capacity factor decreases rapidly with increasing relative density of the sand. These results are supported by theoretical estimates of the compaction and deformation of loose sand by pile driving (Meyerhof, 1959), which leads to an increased density and angle of internal friction of the sand. This increase is greater around interior piles than near corner piles of the group and can be estimated approximately from the principal stress changes in the sand by pile driving. For a given pile spacing the bearing capacity per pile in a group in loose sand increases therefore with the size of the group. For an exceptionally close pile spacing the bearing capacity is governed by equivalent pier failure and the group capacity can be estimated accordingly.

However, corresponding model tests by Stuart *et al.* (1960) and *Kishida and Meyerhof (4/10)* on free-standing pile groups in dense sand have shown that the group capacity is governed by individual pile failure at an average pile load which may be only about two-thirds of the maximum single-pile value. This reduction of the group capacity in dense sand can largely be explained by the decrease of the original density and angle of internal friction of the soil by pile driving on account of the dilatancy of the sand in the overlapping zones of shearing deformation in the soil near the piles. Consequently, for a given pile spacing the bearing capacity per pile in a group in dense sand decreases as the size of the group increases, and for an intermediate (critical) density of the sand no significant change of the group capacity would be expected. The General Reporter expressed the feeling that the dilatancy of the sand is limited to a rather narrow zone around the pile and cannot influence the bearing capacity in practice due to the decrease of the critical void ratio by greater vertical pressures. However, an analysis of the full-scale observations and loading tests on single piles driven into dense sand by Kérisel (1961 and 1964) indicates that the loosened zone in the sand along the piles has a width of about 5 times the pile diameter. Moreover, the corresponding deduced angle of internal friction of the sand at these piles was found to be about 3 degrees smaller than the original friction angle of the dense sand, while for similar loading tests on single piles in loose sand the deduced angle of internal friction at the piles was about 3 degrees greater than the original friction angle of the sand. It may be concluded, therefore, that for individual pile failure the bearing capacity of free-standing model pile groups in sands of various relative densities agrees in a qualitative way with the observations on single full-scale piles in similar sands and supports the approximate theoretical estimates.

BEARING CAPACITY OF PILED FOUNDATIONS

The total ultimate bearing capacity of driven pile groups with caps or foundations resting on load-bearing soil, or piled foundations, can often be considered equivalent to that of a pier foundation with its base at the depth of the pile points, and the total bearing capacity can then be estimated accordingly. At present only loading tests on model piled foundations are available for comparison with the theory. The test results of such groups in soft clay by Whitaker (1960) showed that the bearing capacity is governed by that of an equivalent pier at least up to a pile spacing of 3 or 4 times the pile diameter, which in practice would cover most pile layouts.

On the other hand, the test results on model piled foundations in sand reported to this Conference by *Kishida and Meyerhof (4/10)* showed that the corresponding failure mechanism in sand varies considerably with the original relative density and the pile spacing. For piled foundations in loose sand and a close pile spacing of less than about 2½ pile diameters the bearing capacity is again governed by that of an equivalent pier. Yet for a greater pile spacing the bearing capacity is given by the sum of that of the individual piles plus the surface bearing capacity of the cap, as would be expected theoretically. In the latter case the surcharge from the cap raises the bearing capacity of the individual piles above that of a similar free-standing pile group. For piled foundations in dense sand, however, equivalent pier failure does not occur at any pile spacing due to the effect of the dilatancy of the sand mentioned before. Here the total bearing capacity is thus equal to that of the cap and the sum of the individual piles including the surcharge effect of the cap.

It can thus be concluded from both theory and model test results that pile caps when resting on soil can have a marked influence on the ultimate bearing capacity of pile groups at customary pile spacings in both clay and sand. Before applying these results to the design of piled foundations in practice, it is to be hoped that full-scale loading tests will be made on such foundations for comparison with the results of the model tests and theoretical estimates of their bearing capacity. Field observations are also particularly important for an assessment of the settlement of pile groups under various conditions of loading and layout in different soils, since model tests cannot give any quantitative information on the foundation movements to be expected in practice.

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CHAIRMAN RUTLEDGE

Thank you Dean Meyerhof. Monsieur Parez, do you wish to make a comment on this subject?

Panelist: L. A. PAREZ (France)

Les essais sur groupes de pieux en vraie grandeur nécessitent la mise en œuvre de charges considérables, c'est pourquoi ils ont été généralement réalisés sur modèles réduits, et trop souvent sur des modèles vraiment trop petits et à des profondeurs faibles. Je pense que, dans ces essais, le facteur prépondérant est la liaison des têtes de pieux, c'est-à-dire la rigidité de la semelle et de l'encastrement des têtes de pieux dans la semelle.

GROUPES DE PIEUX DANS L'ARGILE

Il est assez étonnant que la zone de déformation qui se développe au voisinage d'un pieu foncé dans l'argile *lors de son chargement* puisse avoir une influence qui s'étende latéralement jusqu'à 7 à 8 fois le diamètre du pieu, et surtout dans l'argile molle. Une telle influence se fait sentir lors du battage car il y a un refoulement important du sol, mais le refoulement est beaucoup plus faible lors d'un essai de chargement.

Pour les entraxes de pieux habituels et économiques de 2,5 à 3,5 diamètres, la déduction globale d'un tiers de la capacité portante ultime semble nettement trop forte, car si effectivement les pieux périphériques et surtout les pieux d'angle voient leur capacité portante peu réduite, les pieux intérieurs devraient alors, pratiquement, être comptés pour zéro.

Par prudence, et dans l'état actuel de nos connaissances, il est souhaitable de recommander de ne pas utiliser de groupes de pieux dans l'argile comprenant des pieux placés à l'intérieur du groupe. Mais, en réalité, pour les groupes de pieux dans l'argile, le véritable problème n'est pas, en général, un problème de rupture, c'est un problème de tassement: celui-ci varie avec le nombre de pieux du groupe et peut donc donner d'un groupe à un autre des tassements différentiels importants, souvent incompatibles avec la structure.

GROUPES DE PIEUX DANS UN SABLE DENSE

Ces pieux portent surtout par la pointe. La capacité portante limite du groupe est inférieure à la somme de celles des pieux uniques; elle décroît lorsque les entraxes diminuent. Ceci est une extension des résultats trouvés par Kérisel (1961). En effet, lorsque les pieux sont assez rapprochés, l'ensemble peut être assimilé à une colonne de grand diamètre et Kérisel a montré que la pression limite de pointe décroît lorsque le diamètre croît dans les sables très denses. La zone de dilatation du sable a été trouvée, par Kérisel, comprise entre 0,9 et 2,5 diamètres du pieu.

Cette diminution de la capacité portante limite du pieu dans un groupe en sable dense a relativement peu d'importance si on remarque que, d'une façon générale, elle est limitée par la résistance propre du matériau constitutif du pieu lui-même.

GROUPES DE PIEUX DANS UN SABLE LÂCHE

On comprend aisément que le fonçage des pieux augmente à l'intérieur du groupe la compacité naturelle du sable lâche, ce qui augmente la capacité portante moyenne et surtout celle des pieux intérieurs. Les expériences de Kézdi (1957) sont très significatives, lorsqu'il passe des pieux en ligne aux pieux en carré, toutes autres choses restant égales par ailleurs.

Peut-on considérer comme parfaitement permanente l'augmentation de compacité d'un sable sous la nappe à l'intérieur d'un groupe de pieux surtout s'il y a écoulement d'eau, en particulier lorsqu'il y a influence de la marée?

Lorsque la résistance des pieux est donnée pour une part appréciable par la pointe, le groupe de pieux peut être avantageusement remplacé par un ou deux pieux de grand diamètre, il en résulte une économie non seulement sur le coût de l'ensemble des pieux mais aussi sur les semelles qui les coiffent.

Enfin, la semelle sur groupe de pieux a bien une influence sur la capacité portante de ceux-ci, pour autant qu'elle repose sur un sol qui puisse transmettre les contraintes sans fluage superficiel, mais cette influence doit décroître avec le temps, car, en général, les sols qui nécessitent des fondations par pieux se consolident, tassent et, au bout d'un certain temps, la semelle n'est plus en contact avec le sol. On peut donc penser qu'il serait en général imprudent de tenir compte de l'effet de la semelle dans un projet.

CHAIRMAN RUTLEDGE

Thank you M. Parez. The third subject we are going to discuss, as proposed by our General Reporter, is the most important features of pile loading tests to be included in the recommendations on their use and evaluation. I am again going to ask M. Parez to initiate discussion on this subject. M. Parez.

PANELIST PAREZ

Le président Rutledge m'a ensuite demandé de traiter le point "d" proposé par le Rapporteur général Kézdi qui demandait de définir les caractéristiques les plus importantes des essais de chargement de pieux à inclure dans des recommandations afin que, d'un pays à un autre, les essais dont nous parlons puissent être comparables. Je classerai ces caractéristiques en quatre catégories: 1) la charge, 2) les appareils de mesure, 3) la cadence des lectures, 4) l'utilisation des résultats. Je suppose qu'il s'agit d'uniformiser les essais habituels, dits de routine, et non pas les essais spéciaux réalisés pour une recherche particulière (essais de chargement répétés, par exemple).

CHARGE

La charge peut être réalisée:

—soit par un lest (sable, pièces de béton, rails, etc.),

—soit par d'autres pieux utilisés en traction,

—soit par des ancrages profonds au moyen de câbles scellés nettement plus bas que la pointe des pieux (plus de 5 m par exemple). C'est ce dernier mode qui bouleverse le moins le sol au voisinage du pieu à essayer, mais c'est aussi le plus coûteux.

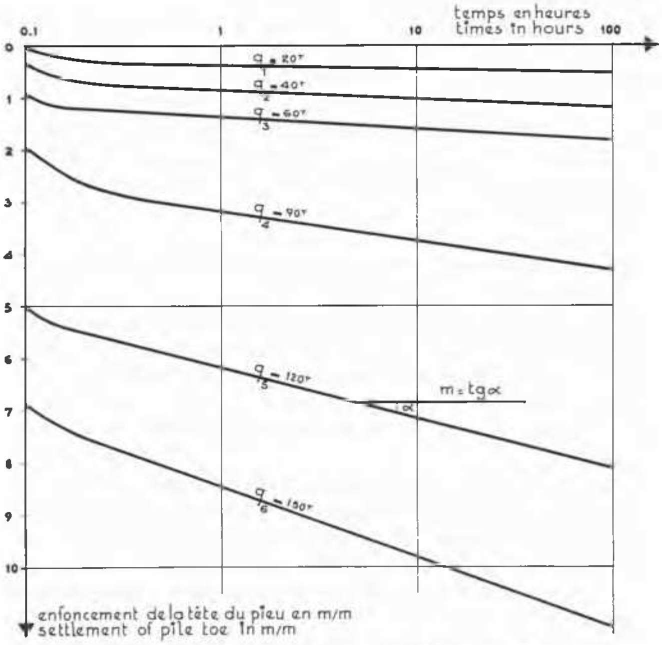

FIG. 9. Enfoncement de la tête du pieu *versus* temps.

Dans tous les cas, un vérin étalonné doit être placé entre la tête du pieu et la charge.

APPAREILS DE MESURE

Les points de fixation des appareils de mesure du déplacement de la tête du pieu doivent être scellés dans une zone où la répartition des contraintes dans la section du pieu puisse être considérée comme uniforme.

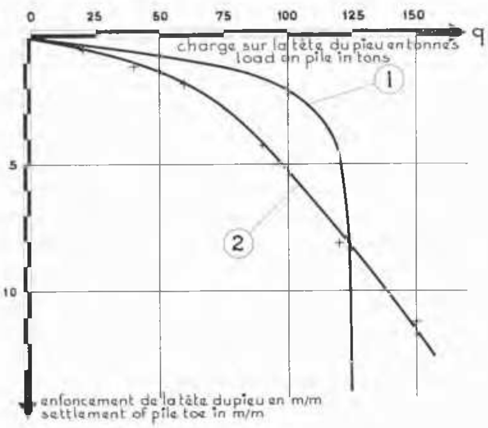

FIG. 10. Enfoncement *versus* charge sur la tête du pieu.

Ces appareils (comparateurs, flexigraphes), au nombre de 3 au moins, ont une précision comprise entre 1/10 et 1/100 de millimètre. Ils ne peuvent pas être posés sur le sol voisin qui n'est pas fixe au cours de l'essai; on doit reporter leur appui le plus loin possible de la charge et vérifier que ces appuis ne bougent pas, sinon il faut faire une correction des mesures avec la même précision. Le plus simple, mais aussi le plus coûteux, est de disposer d'un pieu de chaque côté du pieu à essayer.

CADENCE DES LECTURES

Deux cas sont à considérer selon que le pieu à essayer sera ou non utilisé dans la construction.

Si le pieu est utilisé sous l'ouvrage définitif, l'essai ne peut guère être poussé à plus de 1,5 fois la charge réelle du pieu. Sinon on peut pousser l'essai jusqu'à 2 et même parfois 2,5 fois la charge portante prévue. La montée en charge se fera par 2 ou 3 paliers jusqu'à la charge portante puis on déchargera par paliers et on remettra en charge par de nouveaux paliers. On doit rechercher une stabilisation de l'enfoncement pour chaque palier. Afin de ne pas passer trop de temps aux lectures je propose de procéder de la façon suivante:

Sous une charge q maintenue constante on lit les déplacements de la tête du pieu et on les porte sur un graphique en fonction du logarithme du temps (fig. 9). Lorsque la courbe est devenue sensiblement linéaire on la prolonge par une droite jusqu'à 100 heures. J'appelle m la pente de cette droite. C'est l'ensemble des résultats trouvés pour 100 heures qui sera utilisé dans le tracé de la courbe efforts-déformations (fig. 10).

RÉSULTATS

L'allure de cette courbe peut prendre la forme 1, dans ce cas, il est facile de parler de rupture, ou bien la forme 2

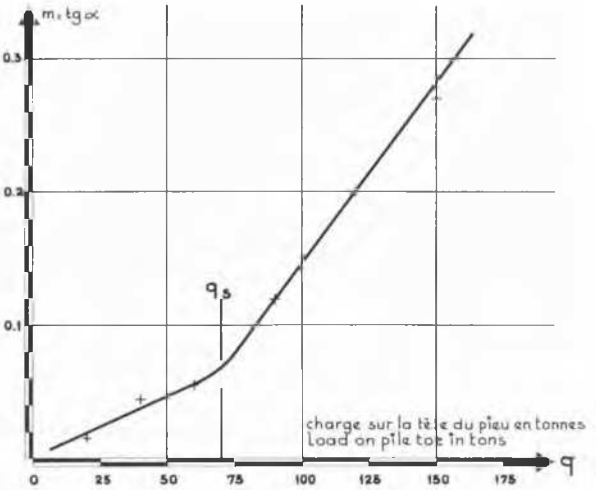

FIG. 11. Charge sur la tête du pieu *versus* tan α.

(pas d'asymptote verticale). Comme l'a signalé le Rapporteur général tout-à-l'heure, la forme 1 est rare.

Si, en France, on admet qu'il y a rupture pour un déplacement de 20 cm, qu'en est-il dans d'autres pays? A Mexico, par exemple, que prendre dans ce cas comme charge portante en service du pieu? Une fraction d'une charge de rupture qu'on ne connaît pas?

Je propose une methode déduite du rapport de Cambefort et Chadeisson au 5[ième] Congrès à Paris: Soient m les pentes de chacun des segments de droite tracés dans le graphique enfoncements en fonction du logarithme du temps (fig. 9). On porte, en coordonnées normales, les valeurs de cette pente m en fonction de la charge q (fig. 11). On obtient une courbe qui a une brisure.

Je propose d'adopter comme charge de service la charge q_s correspondant à la brisure, ou tout au moins une part importante de cette charge.

CHAIRMAN RUTLEDGE

Thank you M. Parez. Dr. Zeevaert, do you have any comments on M. Parez's discussion?

PANELIST ZEEVAERT

I agree with Mr. Parez, that one should codify a rational procedure for the testing of piles in order to be able to compare the tests made at different places, as has been already suggested in previous general reports.

The problem is too vast to be commented in such a short time; however, I will add one requirement to the list: that for each pile test the stratigraphical conditions of the site should be assessed and at least a report giving a good visual classification of the materials encountered, when it is not possible to supplement the information with their mechanical properties. It is also very important to know the state of hydraulic conditions at the location of the pile tests.

When the basic information is secured, one may be able to learn on the proper application of the soil properties and of the interpretation of the tests. The information thus obtained will enable one to predict the behaviour of pile groups more accurately. In case of friction piles in clays and clayey silts, the time element is very important as has already been noted. For example, in these cases the pile load should be investigated until continuous and uniform penetration is obtained under sustained load. Furthermore, one should be able to determine separately the point bearing and side friction behaviour in full-scale tests.

We certainly know, and must keep in mind, that a single pile test does not necessarily represent the group action, either from the viewpoint of load settlement or ultimate load behaviour.

CHAIRMAN RUTLEDGE

We are running late, gentlemen. Can we confine our break to about 10 minutes and return for discussion from members of the conference.

(There followed a brief intermission.)

A. VAN WAMBEKE (Belgique)

M. le Rapporteur général a exprimé son scepticisme quant à la valeur des formules de tassement proposées par M. Ménard dans sa communication. Je le comprends d'autant mieux que j'ai également partagé son opinion mais, après réflexion et à la lumière des résultats obtenus à l'aide de cette méthode, mon scepticisme initial a fait place à la confiance. Je vais vous dire pourquoi.

Une méthode de calcul de tassement ne peut être tenue pour valable que si, d'une part, elle satisfait au principe d'homogénéité et que si, d'autre part, elle donne des résultats suffisamment concordants avec les observations de tassements réels. Le Rapporteur général de la troisième section, le professeur de Beer, a déjà fort justement attiré l'attention du congrès sur la nécessité de respecter le *principe d'homogénéité.*

Aux deux facteurs qu'il a évoqués en parlant de la nécessité d'harmoniser la *justesse* des formules et la *précision* de la détermination des paramètres mécaniques intervenant dans ces formules, j'ajouterais un troisième: le choix de la *densité de recherche* qui est à faire en tenant compte, d'une part, de l'hétérogénéité du terrain et, d'autre part, de la précision souhaitée dans l'estimation des tassements.

Tous ces facteurs d'appréciation sont précisément à la base du jugement favorable que je porte sur la méthode de calcul des tassements proposée par M. Ménard.

En effet, ces formules ont un degré d'exactitude suffisant pour le degré de précision avec lequel les caractéristiques mécaniques sont mesurées. De plus, elles sont *saines* tout en restant *simples.* Elles sont saines car elles font intervenir, ce qui est tenu pour essentiel maintenant, des modules de déformation tenant compte de la différence de nature des champs de contrainte induits dans le sol par la sollicitation. Elles demeurent simples car elles ne nécessitent, le coefficient de Poisson mis à part, que la connaissance de deux caractéristiques du sol: un module déviatorique (mesurable soit *in situ* à l'aide du pressiomètre, soit en laboratoire à l'aide du triaxial) et un module sphérique (mesurable à l'œdomètre ou pouvant être estimé, pour un type de sol donné, à l'aide du module déviatorique). Le coefficient rhéologique α qui intervient dans les formules est égal au rapport des deux modules précédents.

Comme la détermination des caractéristiques nécessaires peut se faire, soit en laboratoire, soit *in situ,* on peut adapter la densité de recherche et le mode d'examen à l'hétérogénéité du terrain, les méthodes *in situ* convenant, de préférence aux méthodes de laboratoire, pour les cas de terrains dont l'homogénéité n'est pas reconnue ou qui ne permettent pas le prélèvement d'échantillons représentatifs non remaniés.

Enfin, sur le plan du *contrôle* par observation et comparaison des tassements réels, les résultats obtenus jusqu'à maintenant sont encourageants et montrent que les tassements trouvés par cette méthode ne sont pas plus éloignés de la réalité, bien au contraire, que ceux donnés par des méthodes plus habituelles.

N. M. DOROSHKEVICH and A. A. BARTOLOMEY (U.S.S.R.) (Presented by N. A. Tsytovich)

This contribution relates to the method of calculating pile foundation settlements using solutions of the theory of elasticity, based on studies conducted at the Chair of Soil Mechanics, Basements and Foundations of the M.I.C.E. under N. A. Tsytovitch.

This study views these settlements as a result of basement soil compaction due to the load effects conveyed by the piles. The nature of the load conveyance invites a breakdown of pile foundations into the four groups: (1) single piles, (2) single-row pile foundations, (3) multi-row long strip (continuous) pile foundations, (4) groups of piles.

Some methods were developed in the U.S.S.R. to calculate the settlements of single piles (Syvtzova, 1963; Ogranovitch, 1963). Pile loading into the soil was viewed as a sum of two loads: the load distributed along the pile shaft and that under the tip of the pile.

The experiments on pile foundations carried out at the M.I.C.E. made it possible to determine the nature of conveying the load on to the soil in the case of joint pile operation (Doroshkevitch, 1960; Bartolomey, 1964) and to secure the analytical solution to determine the pile foundation settlements. These solutions are based on Mindlin and Melan's problem and charted in Fig. 12. Fig. 13 showing the comparison between the theoretical sheets of vertical stresses and the experimental ones, bears further witness to the necessity of accounting for the depth of applying the loads.

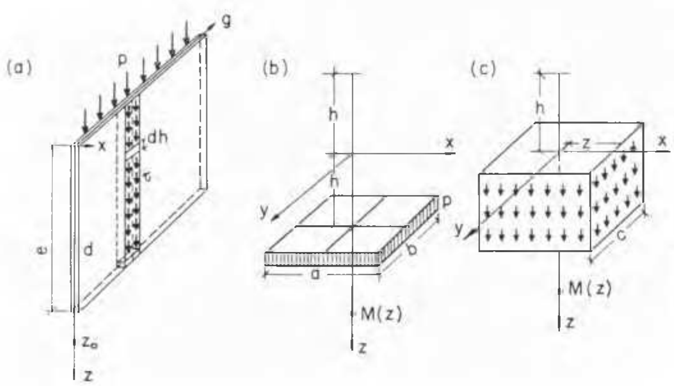

FIG. 12. Calculation pattern schemes: (a) single-row pile foundations; (b) and (c) groups of piles.

To determine single-row pile foundation settlements use was made of A. A. Bartolomey's solution obtained by integrating the equation of vertical sheets in the plane problem (Melan, 1932; Gorbunov-Possadov, *et al.*, 1954) for the lateral surface loadings, as well as those acting in the plane of lower pile tips. The equation results in the following form:

$$W = \frac{P}{\pi E_1 L}\left(\sum_{n=0}^{2} a_n W_n + bW_3 + 2ln\frac{Z_0}{l}\right), \qquad (1)$$

where W is the settlement in cm, E_1 is the modulus of deformation, l, L are the length of and the interval between the piles, P is the load into the pile in kg, Z_0 is the depth of the layer under the compression in cm. According to Tsytovitch the boundary of the layer under the compression must be assumed at the depth where stresses do not exceed the structural strength of the soil.

A_n, b are dimensionless ratios depending on the nature of load conveyance; these ratios are determined by the method of minimal squares on the condition that the movements of soils along the pile stem are permanent. n is the ratio accounting for the form of the distribution of skin friction forces (an oblong, with $n = 0$, a triangle, with $n = 1$; a curve of the second order with $n = 2$); W_n, W_3 are the movements caused respectively by skin friction forces and those in the plane of lower pile tips. Let us denote the values of sums

$$\sum_{n=0}^{2} A_n W_n + bW_3$$

through $C_{(n)}$ and, for the sake of convenience, confine formula (1) to

$$W = P/\pi E_1 L[2l_n(Z_0/l) + C_{(n)}]. \qquad (2)$$

The value of $C_{(n)}$ is derived from Table I depending on the Poisson ratio μ of the width of single-row pile foundations d/l, the nature of the distribution of skin friction forces n, and the soil resistance under the pile tip.

It can be recommended that the settlements for multi-row long-strip pile foundations and groups of piles be determined on the strength of elementary summarization with regard to the stresses in the direction of the axis of the pile and located in the footing thereof. The determination of the stresses in the pile footing must be based on the skin load of the

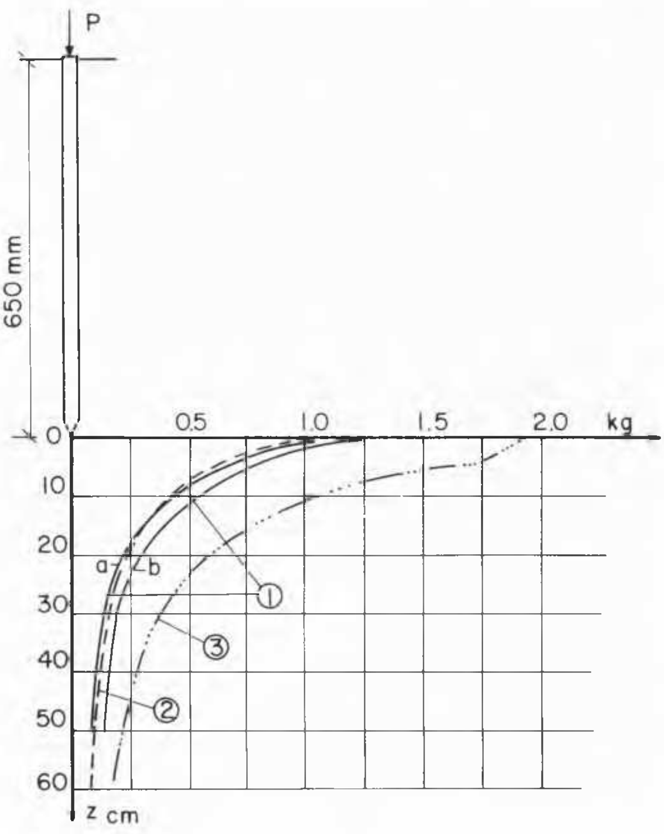

FIG. 13. Stress distribution under piles of single-row pile foundation: (1) experimental: (*a*) at the interval between the piles 6*d*, (*b*)—3*d*; (2) theoretical—for a strip according to Mindlin; (3) theoretical—for a strip according to Boussinesq.

TABLE I. RATIOS OF STRESS CHANGE WITH DEPTH FOR VARIOUS PILE SIZES

μ	n	$d/e = 0.025$ $C_{(n)}$	$d/e = 0.035$ $C_{(n)}$	$d/e = 0.050$ $C_{(n)}$
	0	1.391	1.361	1.325
0.30	1	1.403	1.390	1.347
	2	1.405	1.382	1.352
	0	1.466	1.437	1.401
0.35	1	1.480	1.455	1.419
	2	1.477	1.457	1.426
	0	1.545	1.517	1.481
0.40	1	1.634	1.639	1.645
	2	1.560	1.536	1.503

foundation and the loading in the plane of lower pile tips (Fig. 12b, 12c), account being taken of the depth of their application.

To obtain solutions instrumental in determining the vertical compressive stresses use was made of the Mindlin equation. The following expression was obtained as a result of the equation's binary integration and conversion to dimensionless co-ordinates, in order to define the stresses due to the evenly spread vertical load in action in the plane of lower pile tips.

$$\sigma_z = [p0.1mn/4\pi(1-\mu)][\tfrac{1}{2}(y_0 - y_{10}) + y_1 + y_2 + y_3 + y_4 + y_5 + y_6 + y_7 + y_8 + y_9) \quad (3)$$

$$y_i = -(G/A_1B_1) + (G/C_1D_1) - (F/A_1^2B_1^3) - (R/C_1^2D_1) + (S/C_1^2D_1^3) - (T/C_1^3D_1) + (U/C_1^3D_1^3) - (V/C_1^3D_1^5),$$

$$G = 0.5(1-2\mu)(K-1)m_1$$

$$F = 1.5(K-1)^3 m_1$$

$$E = 0.125(K-1)^3 m^3$$

$$R = 1.5[(3-4\mu)K(K+1)^2 - (K+1)(5K-1)]m_1$$

$$S = 0.125[(3-4\mu)K(K+1)^2 - (K+1)(5K-1)]m^3,$$

$$T = 15K(K+1)^3 m;$$

$$U = 2.5K(K+1)^3 m^3,$$

$$V = 0.1875K(K+1)^3 m^5,$$

$$B_1 = \sqrt{[(0.5\Delta_1 mn)^2 + (0.5m)^2 + (K-1)^2]},$$

$$A_1 = (0.5\Delta_1 mn)^2 + (K-1)^2,$$

$$C_1 = (0.5\Delta_1 mn)^2 + (K-1)^2,$$

$$D_1 = \sqrt{[(0.5\Delta_1 mn)^2 + (0.5m)^2 + (K+1)^2]}$$

A similar expression was obtained for the stresses of the evenly distributed vertical skin loads of pile foundation where m, n, K are dimensionless parameters of the value: $n = a/b$; $m = b/h$; $K = z/h$, and a, b are length and width of the foundation, h is the depth of load effects equal with the length of the pile, Z is the depth of the point at issue.

These expressions assume a far simpler form when converted to

$$\sigma_Z = \alpha_1 P_1 \quad (4)$$

$$\sigma_Z = \alpha_2 P_2 \quad (5)$$

where P_1, P_2 = effective load in the plane of lower pile tips and skin load of the pile foundation in kg/sq.cm., their determination depending on the properties of soils; and α_1, α_2 are ratios of stress changes with depth for single-time loading. The values α_1 and α_2 were calculated with the use of an electronic computer with a view to various sizes of pile foundations and charted in Table I (Doroshkevitch, 1961).

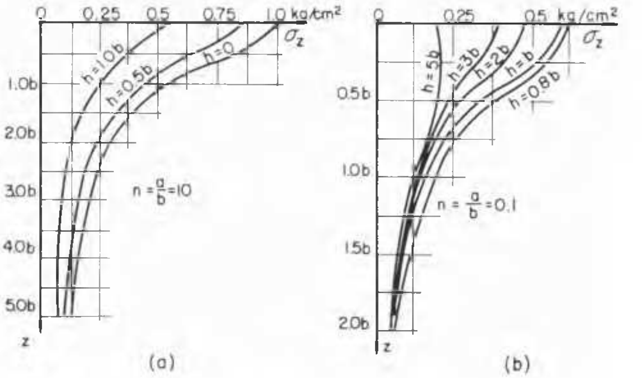

FIG. 14. Compressive stress diagrams under pile foundations of various sizes (a) from vertical loading in action in the plane of lower pile tips; (b) from vertical loading in action along the edges of pile foundations.

TABLE II. COMPARISON OF CALCULATED AND OBSERVED SETTLEMENTS

Type of pile foundation	Settlement in mm calculated	Settlement in mm actual
1. Single-row pile foundation of 5-storey residential apartment	14.0	15.2
2. Single-row pile foundation of 5-storey residential apartment	10.7	12.0
3. Two-row pile foundation of 5-storey residential apartment	12.3	11.9
4. Two-row pile foundation of 5-storey residential apartment	12.0	12.7
5. Group of piles	9.82	9.02
6. Pile field for a tower of 14-storey residential apartment	80.0	78.0

Figs. 14a and b represent the compressive stress sheets for various sizes of foundations in terms of horizontal (a) and vertical planes (b). Table II indicates the outcomes of the comparison between the calculated values of settlements and the actual values, resulting from visual observations.

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B. LADANYI (Canada)

Lo and Stermac (4/13) should be complimented for their interesting paper presenting a simple and reliable method for estimating the maximum pore pressure induced by driving piles into a normally consolidated clay. I wish to make only some general comments concerning the choice by the authors of the pore pressure ratio ($\Delta u_s/p$) as a basic experimental parameter for estimating the pore pressure variation in clay under combined stresses.

The pore pressure ratio ($\Delta u_s/p$) has been defined by the authors in paper *4/13*, as well as by the first author in his previous papers (Lo, 1961; Bjerrum and Lo, 1963), as the pore pressure resulting from shear, Δu_s, divided by the effective consolidation pressure, p. As the pore pressure due to shear, Δu_s, was considered to be equal to the second term of the Skempton's original pore pressure equation (with $B = 1$),

$$\Delta u = \Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3) \tag{1}$$

the pore pressure ratio is given by

$$(\Delta u_s/p) = [A(\Delta\sigma_1 - \Delta\sigma_3)]/p. \tag{2}$$

For the pore pressure ratio defined by Eq 2, Lo (1961) found that for a normally consolidated clay its value was a unique function of the major principal strain. The ratio can, therefore, be considered as a useful experimental parameter with a promising application in the estimation of pore pressure.

It should, however, be mentioned that from a theoretical standpoint, a clearer picture could have been obtained without a loss of the above-mentioned advantages, if in the ratio the pore pressure due to shear had been defined as the second term of the modified Henkel's pore pressure equation (Wade, 1963):

$$\Delta u = \Delta\sigma_{\text{oct}} + a\Delta\tau_{\text{oct}}, \tag{3}$$

giving an alternative definition of the ratio

$$\Delta u_s/p = a\Delta\tau_{\text{oct}}/p, \tag{4}$$

where σ_{oct} and τ_{oct} are the octahedral normal and shear stresses respectively, and a is a parameter which is thought to describe, approximately, the pore pressure behaviour during shear under combined stresses. The latter definition has been used also by *Shibata and Karube (2/48)* in the paper presented to this Conference.

It is clear that, being defined in terms of stress invariants, Eq (4) has a more general application than Eq (2). On the other hand, it is evident that the second term in Eq (3) is, in fact, equal to ($-\Delta\sigma'_{\text{oct}}$), that is, to the variation of the effective mean normal stress during shear, with opposite sign. Eq (4) can, therefore, be written also as

$$\Delta u_s/p = -\Delta\sigma'_{\text{oct}}/p. \tag{5}$$

It can be seen that the pore pressure ratio according to the second definition, Eq (5), will show the same characteristics of a parameter independent of applied stresses as that according to the first definition, Eq (2), with an additional advantage of being independent of the principal stress direction and therefore, more convenient for studying the pore pressure variation in clays under combined stresses (Ladanyi, 1963, 1965).

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H. GRANHOLM (Sweden)

Our general reporter has outlined the trend of the development of piles from short ones to very large ones, with lengths of 250 ft and a diameter of as much as 10ft. In my country, Sweden, we actually have used even longer piles, up to 360 ft (109 m) but they are very slender ones with a diameter of only 9–11 in. (cross-sectional area—80–100 sq.in., 420–800 sq.cm).

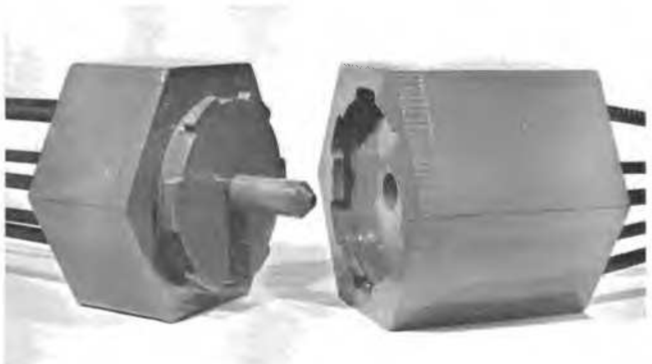

FIG. 15. Male and female steel couplings for the pile splices.

The piles are of concrete, prefabricated in lengths of about 30 ft and spliced together with a steel coupling that has the same bending stiffness as the pile itself (Fig. 15). In order to acquire sufficiently high bending stiffness, the percentage of reinforcement is rather high. The reinforcement consists of corrugated steel bars with a yield point of 85,000 lb/sq.m. (6000 kg/sq.cm.). The compressive strength of the concrete is specified to be not less than 7000 lb/sq.in. (500 kg/sq.cm.). The piles are provided with a point footing made of an extremely hard steel (Fig. 16), so that they can be driven through hard layers and even penetrate bedrock (Fig. 17). Our experience with this type of reinforced high quality pile is very favourable. The percentage of piles lost during operation does not exceed 0.35. Battered piles can be driven just as easily as vertical piles.

The heavy reinforcement is necessary primarily to render the pile stiff enough and also strong enough to remain straight and unbroken when it penetrates the soil. Another important reason is that the pile must withstand the effect of the reflected shock wave which causes considerable tensile forces during the driving operation.

The piles are being driven with a 3-ton drop hammer.

FIG. 16. The piles are provided with a special point made of a very hard steel.

FIG. 17. The steel point has penetrated into bedrock and has developed a firm bearing.

Experiments are now going on to study the advantages of a very long but slender ram. Theoretical investigations have shown that by using a long ram instead of a short one we obtain a much more favourable shape for the compressive shock wave and smaller peak stresses in the concrete. In this way it is possible to transfer much more impact energy to the pile without causing damage. I consider this an important line of development in pile driving technique.

Loading tests on spliced piles with an area of 94 sq.in. (600 sq.cm) have shown that the ultimate load can amount to 380 tons. Buckling does not occur even in a rather soft clay because of the lateral support given by the surrounding material.

Long-time strain measurements on a 200-ft-long pile have shown that the much discussed risk of overloading from negative skin friction does not exist. Our experience drawn from more than 300 large buildings erected on foundations on long piles within a space of six years also confirms that the negative skin friction can be considered negligible. This, of course, holds good only if the surface of the soil is not loaded. In all cases where the surface is loaded, in approach fills for bridges, for instance, the weight of the overburden must be taken into account.

I am aware that this piling technique differs in many respects from common practice in other countries. It has proved, however, to be applicable not only for buildings and bridges but also for the very intricate task of founding quai piers and docks on a soft soil of insufficient bearing capacity.

J. KÉRISEL (France)

Je voudrais consacrer cette intervention à quelques observations sur la notion de module de réaction en matière de pieux chargés latéralement.

Dans les communications présentées à ce Congrès (fig. 18), *Davisson and Robinson (4/4)* se réfèrent à ce module E_s en envisageant 2 hypothèses, la première selon laquelle il est constant, et la deuxième suivant laquelle au contraire, comme le supposent par ailleurs Matlock et Reese (1961) il varie proportionnellement à la profondeur x, $E_s = kx$. Par ailleurs, *Kubo (4/11)* dans sa communication pense que ce module peut s'écrire $E_s = kx/\sqrt{y}$, x la profondeur et y le déplacement horizontal, c'est-à-dire que le module deviendrait infini au point de déplacement nul situé sensiblement entre les zônes de butée et de contrebutée. Kubo admet que k peut varier avec la largeur, mais peu au-delà d'une largeur de 0,20 m.

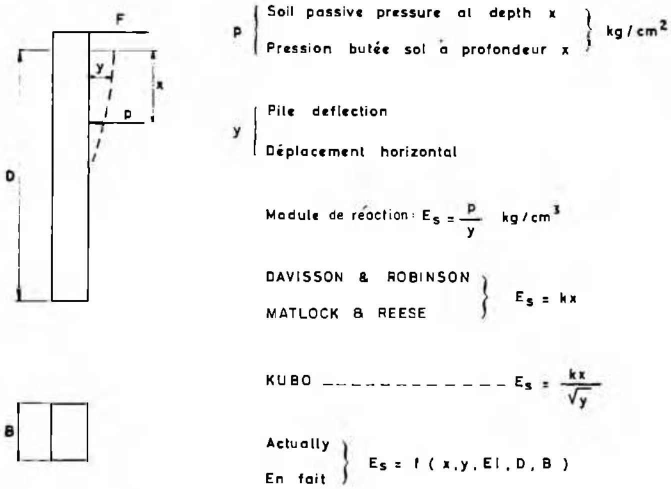

FIG. 18. Module de réaction.

Je voudrais exprimer mon avis sur ce module de réaction; au Congrès de Mexico, nous avons exposé les résultats d'essais à échelle grandeur de traction horizontale sur des pieux caissons métalliques rectangulaires, fichés dans une argile homogène avec financement conjoint de divers organismes suivant un programme de 4 ans, en faisant varier

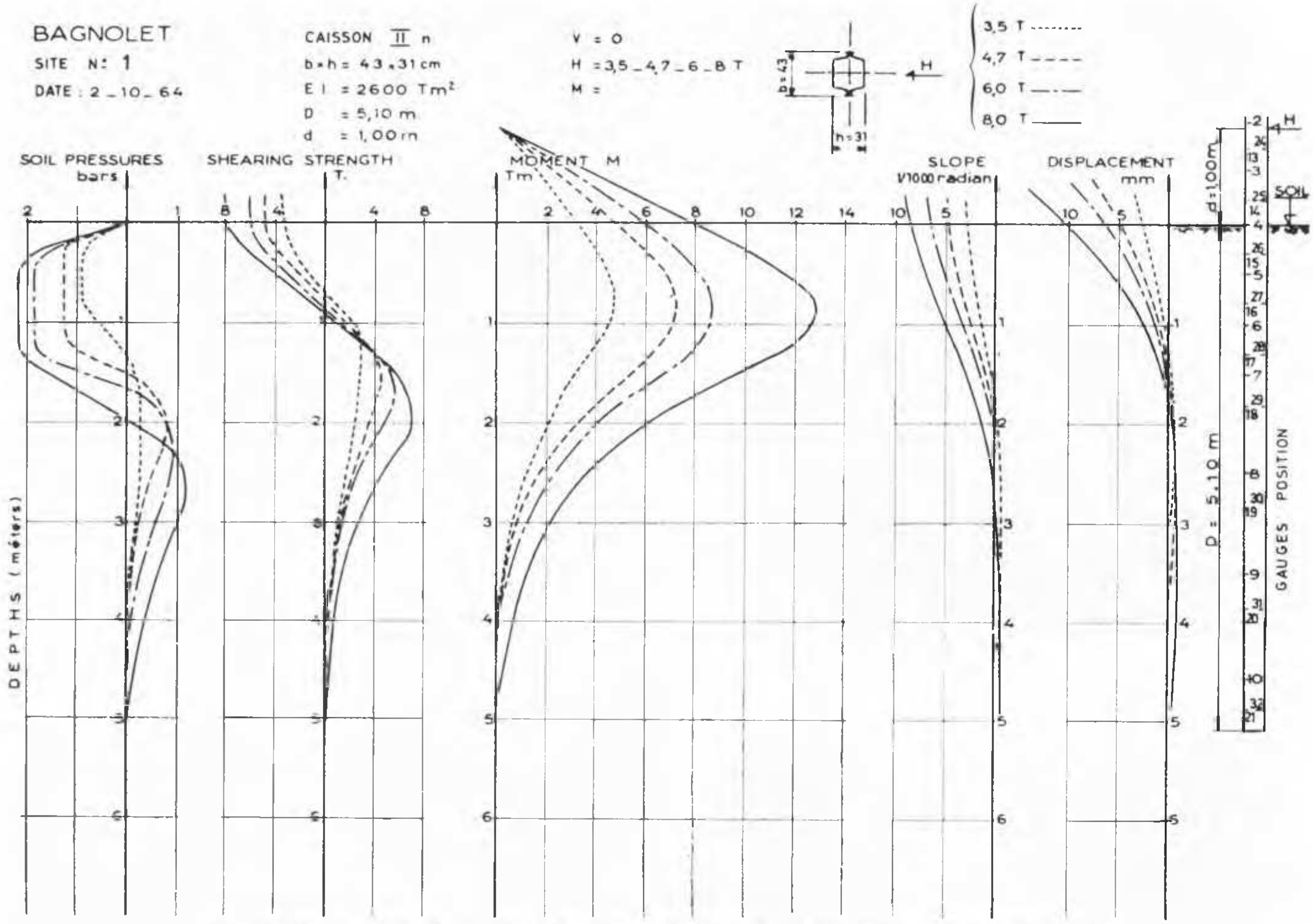

FIG. 19. Essais à échelle grandeur de traction horizontal sur des pieux caissons métalliques.

systématiquement les 3 paramètres qui nous ont paru dominer le sujet, la profondeur, la raideur et la dimension frontale. Trente-trois appareils (fig. 19) d'enregistrement simultané permettent de mesurer le moment, l'inclinaison verticale et le déplacement horizontal d'où l'on déduit l'effort tranchant et la pression du sol, ce qui permet pour chaque expérience de donner l'ensemble des 5 courbes, dont l'une est la dérivée de la précédente, déplacement, inclinaison, moment, effort tranchant et pression de terrain.

A ce jour, les séries de mesures avec une première largeur frontale de 43 cm sont terminées. On y a fait varier la profondeur de 2,25 m à 5,20 m et la raideur du caisson EI de 1 600 tonnes/m.ca à 14 000 tonnes/m.ca. De ces expériences ont été tirées les modules de réaction représentés sur la fig. 20, modules calculés en prenant la pente de la tangente à l'origine dans les diagrammes pression de terrain–déplacements horizontaux. Il apparait que le module de réaction pour de faibles raideurs n'est pas éloigné de varier proportionnellement à la profondeur, ce qui va dans le sens indiqué par Matlock et Reese (1961) alors que pour les caissons raides il augmente avec la profondeur lorsque l'on s'approche du point de déformation nulle, ce qui va dans le sens indiqué par *Kubo* (*4/11*).

Mais, et ceci est capital, le coefficient de proportionnalité avec la profondeur est beacoup plus grand pour les faibles raideurs, de telle sorte que, à une profondeur de 0,50 m par exemple, le module est 4 fois plus grand avec $EI = 1\ 600$ tonnes/m.ca. qu'avec $EI = 14\ 000$ tonnes/m.ca.

De plus, le module pour une même raideur varie avec la profondeur totale du caisson, mais moins largement: dans un intervalle de 10 pour-cent environ. Il est très probable que le module varie avec la largeur frontale; nous le saurons bientôt.

Enfin il va de soi que, pour un même effort en tête, le déplacement n'est pas le même sur toute la hauteur où agit la butée. Donc, la pente de la tangente à l'origine dans les diagrammes pressions–déplacements ne peut convenir dans le calcul quelle que soit la profondeur. Le module de réaction est donc une constante, si j'ose dire, qui dépend d'au moins 5 paramètres.

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J. B. MARTINS (Portugal)

In Mozambique, Portuguese East Africa, 10 bridges with a total span of over 500 m have been recently (1964) built across a swampy area on the delta of the Limpopo River, together with another bridge with a 190-m span. This latter bridge is a mixed structure with prestressed concrete beams and a suspended steel span (Figs. 21 and 22).

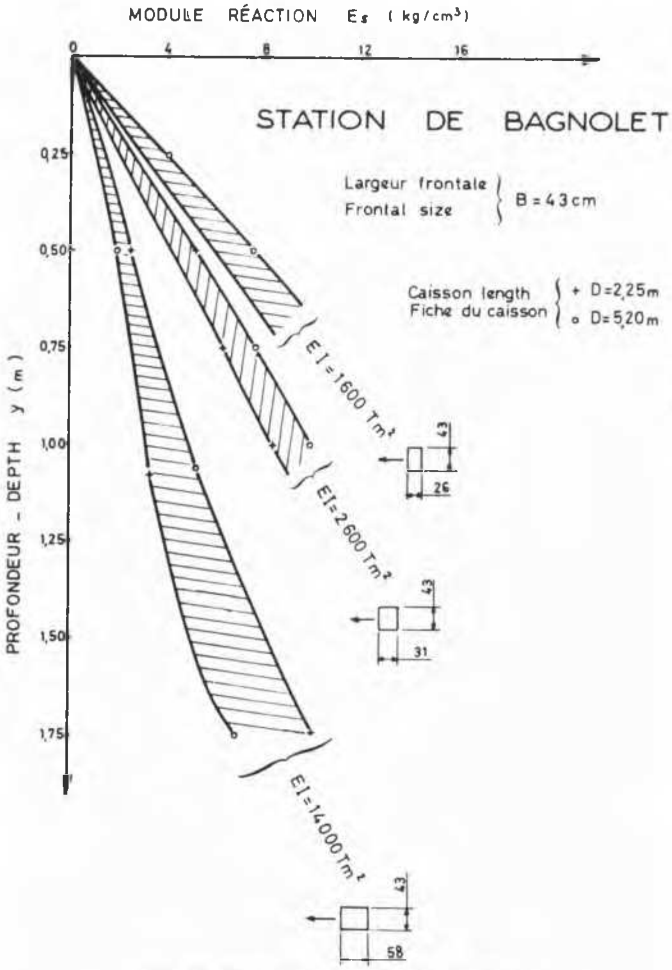

FIG. 20. Valeurs de module de réaction.

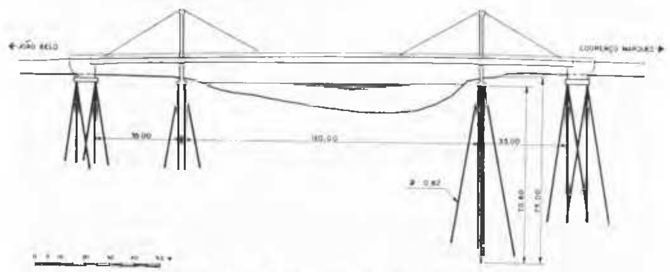

FIG. 21. Limpopo River Bridge.

Special foundation problems arose at the site since there was a deposit of soft clay over 40 m thick (Fig. 23). Reinforced piles, 0.82 m in diameter, were driven, some of them inclined 20 per cent on the vertical, with the point at depths between 60 and 75 m. These are thought to be among the deepest in the world. Tests have been caried out on some of the piles and in some cases residual settlements of more than 3 cm were obtained, Fig. 24 (in one case the residual settlement reached 9 cm), for loads of 300 tons (about 1.5 cm of the working load). Although the settlements were greater than those permitted by any of the 25 codes of practice refered to by Chellis (1951) or by other codes used in Europe, the piles were accepted since cycles of loading and unloading at the working load proved the good behaviour of the piles. The measured settlements of

FIG. 22. Limpopo River Bridge, erection of steel beams.

the pier vary between 4 and 5 mm and those of the abutment (right bank) between 18 and 24 mm. Thus the foundations seem to be in good condition.

Although more research is needed on this point, referring to the subject 4 of the agenda set up by the General Reporter it is suggested that cycles of loading at the working load should be performed before and after the application of the greatest load and the results should be taken into account in establishing the criterion of whether the piles are in good condition or not.

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J. A. J. SALAS (Spain)

One of the most surprising developments in soil mechanics during the last few years has been the almost complete oblivion into which the Mindlin equations have fallen. Certainly some objections may be made against their use in soils, a material which greatly differs from the elastic solid which has served as a starting point to determine these equations, but almost every objection stated against the use of the Mindlin equations applies equally to the Boussinesq equations, which are constantly used.

The only objection which seems to apply to the Mindlin equations but not to Boussinesq's is that the former imply the existence of tensions above the force. However, this objection disregards the simple fact that the stress field represented by the Mindlin equations must be overlapped with the field, due to the weight of the soil itself; thus, such tensions sometimes develop into compressions, or else they are reduced to such a degree that they can be resisted by the cohesion of the soil.

The major reason for oblivion into which the Mindlin equations have fallen might be due to their complexity, but this objection has been completely overruled with the advent of electronic computers.

D'Appolonia and Romualdo (1963) have already offered us one example of the above, and at this Conference, *Thurman and D'Appolonia (4/21)* offer us yet another. On our part, we have submitted some additional examples; among these (in paper *4/18*) we may mention the solution to the important point of negative friction.

Nevertheless, we are fully aware of the limitations of this theoretical approach and our paper is clearly directed to the case of a soil with a fair amount of cohesion, with

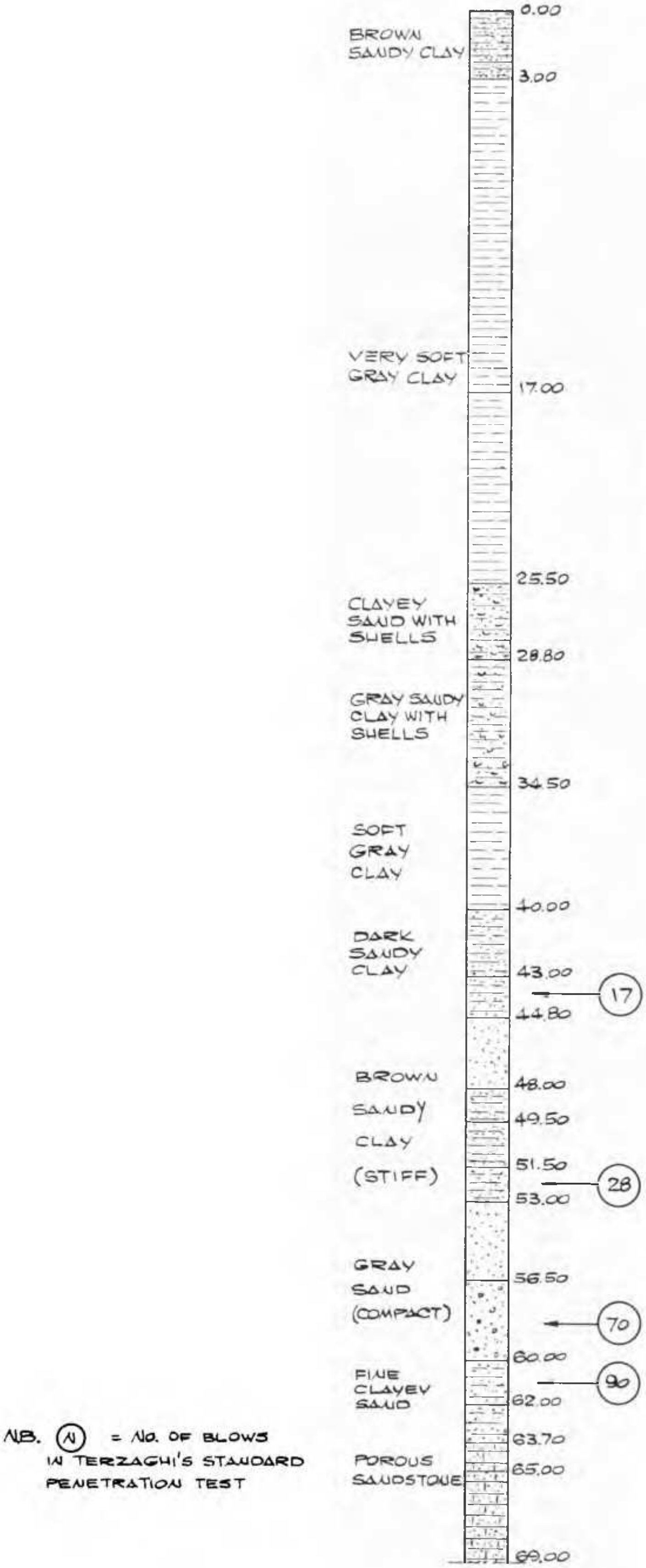

FIG. 23. Foundation profile at right bank pier, site of Pile No. 8.

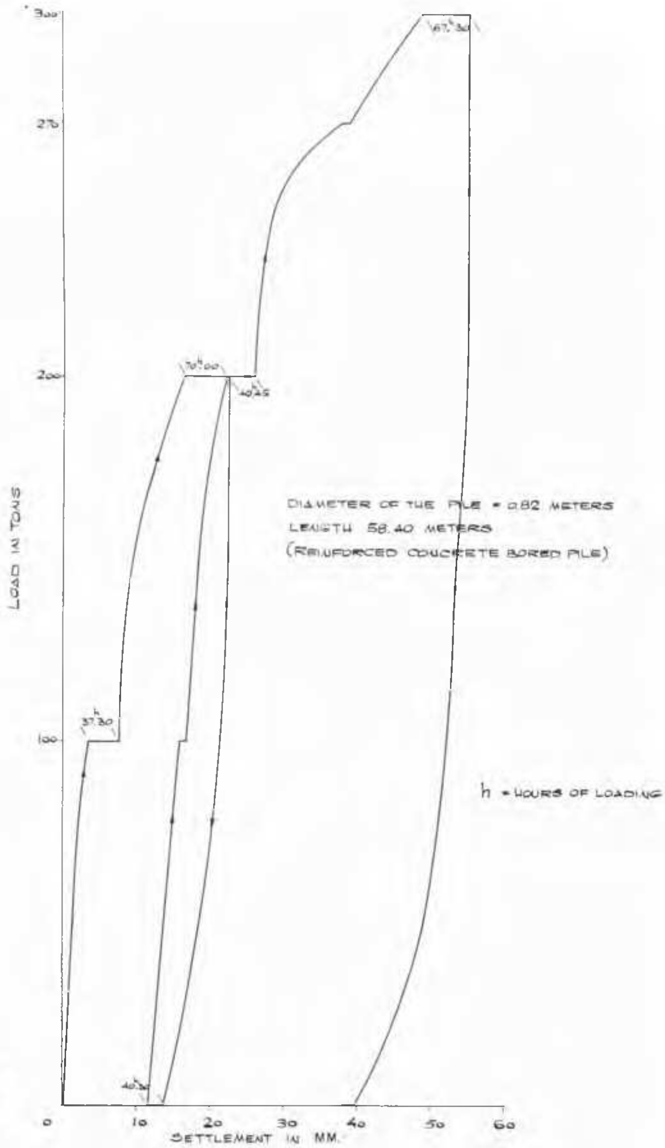

FIG. 24. Load test results, right bank pier, Pile No. 8.

noticeable adhesion between the pile and the soil, and with deformation characteristics which permit the assumption of the existence of a more or less constant Young's modulus, a description applying to a certain kind of clay. The selection of the negative friction problem, characteristic of soft clays, demonstrates the same fact. For this reason, it is not surprising that, as the General Reporter has pointed out, the values obtained do not correspond well to the data reported by *Kérisel, et al.* (4/9), as the experiments covered in that paper refer only to granular materials.

A more serious disagreement betwen theory and practice is that we assume that there is no sliding between the pile and the soil where it is included. However, our paper points out that the results of the analysis demonstrate that in practical cases, with real adhesion values, there may be sliding present in a portion of the pile.

However, sliding cases are almost as simple as the case presented in the paper and the same can be said of many other different cases, such as groups of piles, the effects of shallow foundation on piles, negative friction in spontaneous

COMPATIBILITY EQUATION IN TERMS OF DEFORMATIONS:

$$\int_{0}^{Z_p} \emptyset(C)\, K(C,Z)\, dC + \Phi \int_{Z_p}^{L} K(C,Z)\, dC = \text{constant} \qquad (0 \leq Z \leq Z_p)$$

BEING Z_p = ELASTIC ZONE AND Φ = CONSTANT VALUE OF $\emptyset$ (C) IN $Z_p \leq C \leq L$

EQUIVALENT SYSTEM OF EQUATIONS IN FINITE DIFFERENCES

$$\sum_{m=1}^{m=n-1} K(C_m, Z_q)\, A_m + A \sum_{m=n}^{m=20} K(C_m, Z_q) = \text{constant} \qquad (q = 1, 2, \cdots, n-1)$$

BEING n = NUMBER OF SLIDING PARTS OF THE PILE COUNTED FROM THE BOTTOM

AND A = MAXIMUM LATERAL STRESS.

RESULTS OBTAINED FROM THE SOLUTIONS OF THE SYSTEMS.

MINIMUM LOADS AND SETTLEMENTS PRODUCING SLIDING IN THE MENTIONED NUMBER OF PARTS.

n	μ = 0,00		μ = 0,50	
	Loads / A	Settlements x GL/A	Loads / A	Settlements x GL/A
1	11.5385	9.7727	12.1717	9.3987
2	14.2198	12.1393	15.0244	11.6738
3	15.5408	13.3584	16.3610	12.8158
4	16.3657	14.1550	17.2146	13.5659
5	16.9439	14.7415	17.7946	14.1003
6	17.3779	15.2043	18.2292	14.5213
7	17.2205	15.5916	18.5583	14.8586
8	17.9991	15.9255	18.8207	15.1434
9	18.2325	16.2234	19.0332	15.3885
10	18.4329	16.4970	19.2047	15.6010
11	18.6089	16.7567	19.3538	15.7990
12	18.7665	17.0083	19.4705	15.9677
13	18.9107	17.2615	19.5805	16.1414
14	19.0458	17.5248	19.6624	16.2842
15	19.1752	17.8084	19.7469	16.4505
16	19.3019	18.1276	19.8041	16.5789
17	19.4312	18.5133	19.8712	16.7571
18	19.5657	19.0097	19.9094	16.8812
19	19.7200	19.7726	19.9500	17.0550
20	20.0000	≥ 21.3241	20.0000	≥ 17.2908

FIG. 25. Sliding study, equations and results.

consolidation cases, not due to superficial overloading (where the unitary deformation of the soil is not constant in relation to the height), etc. In these cases, as in many others, the flexibility provided by computer treatment of the problem is so great that it is highly difficult to tabulate all the possible results due to their enormous number. However, it should be taken into account that, with the use of a computer, the time-consuming work of tabulation has been done away with. The main thing is to have ready-made programmes available; in this way, the computer resolves the equations in practically the same period of time an engineer in the past required to look for his tables.

We are going to demonstrate briefly the results obtained by feeding into the computer a condition of failure of the adhesion starting from a force between a section of the pile and soil, thereby producing sliding, a case which has been solved in our Laboratory by Mr. Lorente de Nó, Jr. The integral equation which must be solved now is the one indicated in Fig. 25, Φ being the limit value of the adhesion. In the same figure the equivalent equation in finite difference appears, A now being the maximum value of the force produced by the adhesion between the pile and the soil and n the number of sections of the pile where sliding has occurred. There results, therefore, a non-homogeneous system of $n-1$ equations with n unknown values, which are the $n-1$ efforts A_m of the elastic zone of the soil and the corresponding settlement which is produced as these n parts slide.

Once the system is solved in a function of the settlement k, the solutions are of the form $A_i = \alpha_i + \beta_i k$ and, therefore, the load producing this sliding is given by a linear function, $P_n = a_n + b_n k$. However, the values which may be given to settlements k are not arbitrary ones, as all possible P loads which produce sliding of the last n portions must be between the maximum and the minimum loads producing the plastic zone of the n-1 and $n + 1$ last portions, respectively, i.e., only the points included between the intersections of

$$P_{n+1} = a_{n+1} + b_{n+1} \cdot k \text{ and}$$
$$P_{n-1} = a_{n-1} + b_{n-1} k \text{ with } P_n = a_n + b_n k$$

will be solutions to the problem.

Out of the solution of the problem for $n = 1$, it is deduced that if the load is $P < 11.5385A$ for $\mu = 0.00$ or $P < 12.1717A$ for $\mu = 0.50$, no sliding occurs and the pile works under an elastic regime.

Once these limit loads have been reached, the last section exhausts its resistance, and slides. If the applied load continues to increase, the efforts of the elastic zone also increase, and the settlements grow linearly with the load until the load reaches a given value ($P_2 = 14.2198A$ for $\mu = 0.00$ or $P_2 = 15.0244A$ for $\mu = 0.50$) which produces the sliding of the last two portions. From this P_2 value onwards, the settlement continues to grow linearly with the applied load, but more rapidly than before, until a load P_3 is reached which produces exhaustion of the last three parts. The processes continue to be repeated in this same way until sinking load is reached ($P_{20} = 20A$ for every μ). It should be emphasized that, once the last parts have slid, the development of the plastic zone along the shaft is very rapid, small load increases being enough to produce this sinking.

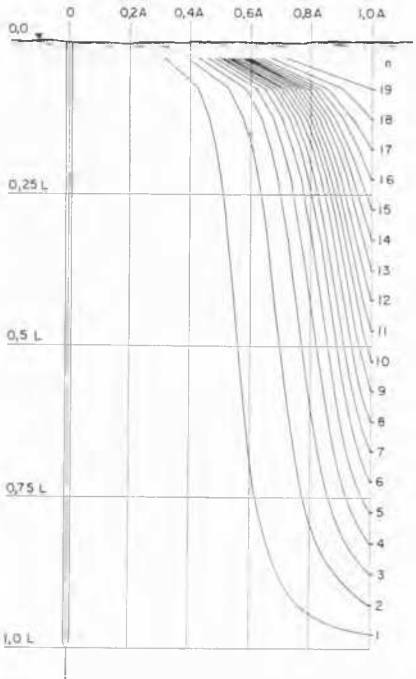

FIG. 26. Stress variations according to the plastic zone. $\mu = 0.00$.

The results thus obtained are represented in the table of Fig. 25 for $\mu = 0$ and 0.5. Fig. 26 shows the stress variation for $\mu = 0$. Fig. 27 presents the theoretical curve, deduced in this manner, of a load test on a pile until the ultimate load is reached.

In this treatment, we have assumed that there is no resistance at the point, but only side adhesion. However, it is also very easy to assume that the force that can be developed in the portion between the deeper part of the

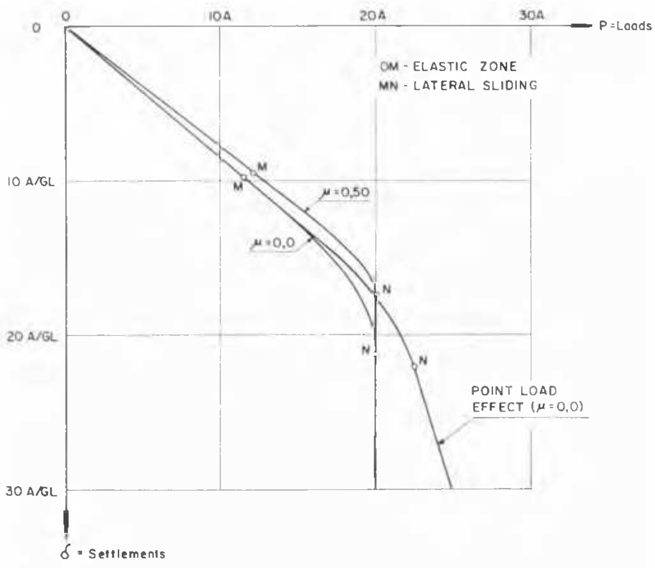

FIG. 27. Theoretical load-settlement curve deduced for a pile.

pile and the soil is not A, but A + point resistance. A calculation has been made of this case with a very large point resistance, with the results summarized in Fig. 27. It is clear that no failure takes place here, even though all the sections of the pile have slid. Failure will only occur when the resistance is surpassed by the point, which we have assumed to be quite large.

Additional cases have been studied, including that of two piles with equal loads, at a mutual distance equal to 80 per cent of their length, their settlement being then 20 per cent greater than that of an isolated pile (for $\mu = 0$). If the distance is $0.5L$, the settlement will be 29 per cent larger, or 23 per cent if $\mu = 0.5$. If the distance is $0.1L$, the settlements will be 57 per cent and 49 per cent, respectively, but it will increase up to 111 per cent if there is a third pile which, together with the other two in the plane, forms an equilateral triangle ($\mu = 0$).

All these results should undoubtedly be regarded only as approximations to real behaviour, but their validity is scarcely less than that of the ones we handle every day to calculate pressures and settlement in shallow foundations. Thus, these approximations are highly valuable, if prudently applied, of course, for the prediction of real behaviour. In any case, we believe they provide an absolutely essential basis for the correct interpretation of experimental results.

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CHAIRMAN RUTLEDGE

Thank you, Dr. Salas. We will now ask our General Reporter, Dr. Kézdi, if he will summarize our morning session. Dr. Kézdi.

GENERAL REPORTER KÉZDI

On reaching the end of the discussions on piles and piled foundations, in the course of which we have heard many important contributions, interesting remarks, and some additions to the printed papers, I would like to emphasize some features that emerge clearly from the work done.

We have seen, first, how geology has influenced the development of soil mechanics in the different parts of the world. The behaviour of piles driven into soft layers in the Netherlands, the investigation of the pile groups in Mexico City, the influence of soil conditions on the design of piled foundations all require a different approach and different treatment, and the consideration of the specific geologic conditions is essential for the soil engineer. Geologic conditions are also responsible for the difficulties encountered in the application of experiences at one building site to others in different parts of the world. It is desirable, therefore, to include a description of some features of the geology of the building site in any report on piled foundations.

In my General Report I expressed a wish for reports on stresses and deformations in and around piles during vertical loading, because, as Professor Meyerhof noted, fundamental knowledge about single driven piles is rather limited. It is, therefore, with pleasure, that I register the interesting model tests carried out by *Kishida and Meyerhof (4/10)*—the correlation of the results obtained in model tests with the behaviour of real piles is extremely interesting. The experiments carried out by Professor Tsytovich and his co-workers also give hope that discovery of a reliable method for computation of the settlement of piled foundations is near. The investigations of the neutral stresses arising around piles driven in clay carried out by Ladanyi also represent an important contribution.

My third remark concerns the bearing capacity of pile groups. We have a definite development in the understanding of the behaviour of pile groups and we have reached a much better stage. The discussion presented by Zeevaert relates to the case of friction piles in compressible soils, pointing again toward the influence of geology, and it gives a clear picture of the mechanics of this type of foundation by dividing the settlement of the pile head into three parts, with instructions on how to calculate these values. The transfer of stresses from pile to soil and *vice versa* is clearly explained. Meyerhof's contributions to the study of the interactions between pile groups and soil clarified particularly the importance and influence of pile cap on the ultimate bearing capacity. Other factors affecting the failure mechanism were also clearly displayed.

These contributions almost performed the task I mentioned in my presentation, a rather quick accomplishment I might say!

The same is true for the contributions to the fourth item I proposed for discussion. I am glad to draw attention to M. Parez' very fine discussion on testing procedures. The outline of the French practice will be a very useful basis for the establishment of some tentative rules. The proposed method for determining the bearing capacity based on test loadings seems to be a reliable tool in the design of pile foundations. In this connection we might mention the method proposed by Széchy at the Fifth Conference: the data derived from repeated loading and unloading of the piles made it possible to determine the limit state at which the development of plastic deformation begins. Comparison of the known methods is highly desirable for the future.

The panel discussion did not treat the problem of the lateral resistance of piles; it was, therefore, very useful that Kérisel intervened in this question. Based on the test results, the outlook for a reliable theory in this field is very good.

I should like to close my remarks with a story written some eighty years ago by a Hungarian novelist, K. Mikszáth.

It is about a country blacksmith who acquired great skill in performing operations for some eye diseases. His fame even reached the University and a famous professor and doctor went to see his operations. There was a patient with glaucoma in both eyes, and the blacksmith, in the presence of the professor, removed it from one eye, with an ease and skill hitherto unknown, by using a simple penknife. The professor, startled by this, tried to explain all the dangers and hazards he invited by so doing. All these explanations made the hand of the blacksmith tremble, so that he could not perform the operation on the other eye. If we go too far in the sophistication of our theories and forget about the facts of the practice, then we shall likely share the fate of this blacksmith. The point up to which we may proceed without danger lies certainly beyond those symbolized by the tools of the blacksmith; however, a clear realization that there is such a point is extremely important.

(The remarks of the General Reporter for Session 6 presented to the Closing Session appear on pp. 594–5.)

CHAIRMAN RUTLEDGE

Thank you, Dr. Kézdi. On behalf of all members of the Conference, I would like to thank our panel members who have worked hard to prepare for this discussion and to express our thanks to those who presented discussions and our apologies to those for whom the time did not permit us to let them join the discussion. Our particular thanks go to Dr. Kézdi, the General Reporter, for this session. The session is adjourned.

WRITTEN CONTRIBUTIONS

CONTRIBUTIONS ÉCRITES

S. P. BRAHMA (India) and C. S. BRAHMA (U.S.A.)

The paper by *Mohan, et al. (4/17)* clearly points out two important aspects of pile foundation design. (1) Load tests alone (of short duration) do not properly predict the nature of settlement which may occur after a few years. (2) Proper site investigation is necessary for the scientific design of pile foundations. The authors have indicated that the settlement is primarily due to consolidation of the clay layers (Strata III and IV). However, it is not very clear how this conclusion has been reached. Nor have other possible explanations for the settlement been discussed. It is felt that at least negative skin friction should be looked into for proper scientific investigation of the problem. Unfortunately the data regarding consolidation, shear test strength, and the diameter and spacing of piles have not been supplied in the paper and it is not possible to show the exact calculations for this particular case. However, an example of the calculation of negative skin friction and its effect on settlement in a similar structure is detailed below, using assumed data which seem reasonable in respect of other given particulars of the warehouse.

The mechanism of negative skin friction on a pile is illustrated. The weight of the new fill as well as the driving of the piles causes gradual settlement of the top compressible layer (Stratum I) which is likely to be reclamation material. However, the downward movement of piles driven into the sand stratum is prevented by the resistance offered by the sand. Hence, there is a relative movement between the piles and the surrounding soil. This results in negative skin friction which has two important effects in this case. (1) The piles are subjected to increased loads as the top layer tends to drag the piles down by shear along the pile surfaces. (2) The confining pressure on the sand layer on which the enlarged bases of the Franki piles are resting is reduced. The load-carrying capacity of enlarged based Franki piles resting on the sand layer is largely dependent on the confining pressure to which the supporting sand deposit is subjected. Hence, it is clear that negative skin friction will not only increase the imposed load on piles but will also reduce the load-carrying capacity of piles.

The analysis given below will show that the negative skin friction may be a decisive factor in computation of the load-carrying capacity of piles. As stated before, the following data are assumed: shear resistance of Stratum I, S, = 300 lb/sq.ft.; diameter of the pile, d, = 24 in.; area of the enlarged base of the pile, A, = 4 sq.ft.; N_r for the sand stratum = 32.

The following data are calculated from the Table I of paper *4/17*: initial confining pressure at the base of the pile, P_1 = 1.11 tons/sq.ft.; confining pressure at the base of the pile when negative skin friction has occurred, P_2 = 0.28 tons/sq.ft.

I. Increase in the pile load due to negative skin friction (Teng) = $S\pi dL$ (where L = length of pile embedment above the bottom of compressible layer that is, in this case the thickness of Stratum I) = $(300 \times \pi \times 2 \times 34)/2240$ tons = 29 tons.

II. Decrease in load carrying capacity of pile from reduction of the confining pressure due to negative skin friction: (*a*) initial load carrying capacity due to bearing before the onset of negative skin friction (Zeevaert, 1960), Q_1, = $A\ N_r\ P_1 = 4 \times 32 \times 1.11$ = 143 tons; (*b*) load carrying capacity of the pile due to bearing when negative friction has occurred, Q_2, = $A\ N_r\ P_2 = 4 \times 32 \times 0.28$ tons = 36 tons. Reduction in load carrying capacity = $Q_1 - Q_2$ = 143 tons − 36 tons = 107 tons.

During load test, the pile was resisted by Strata I and II. However, after a short period negative skin friction not only

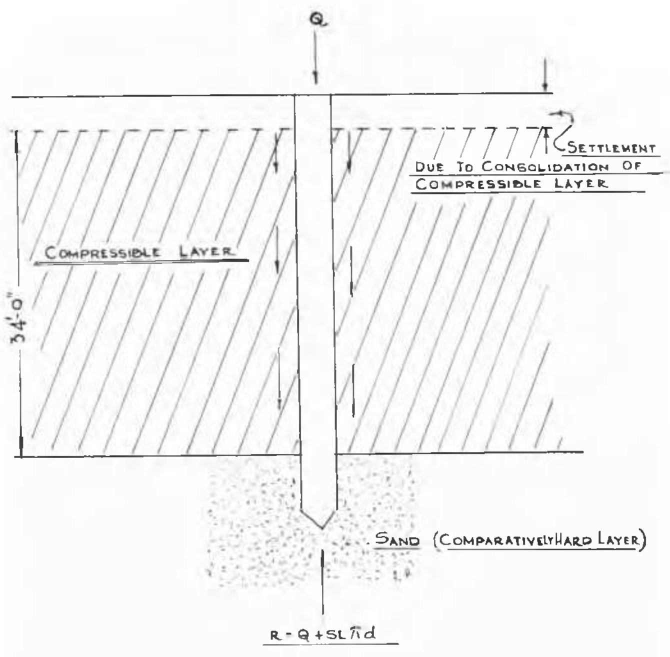

FIG. 28. The mechanism of negative skin friction on a pile.

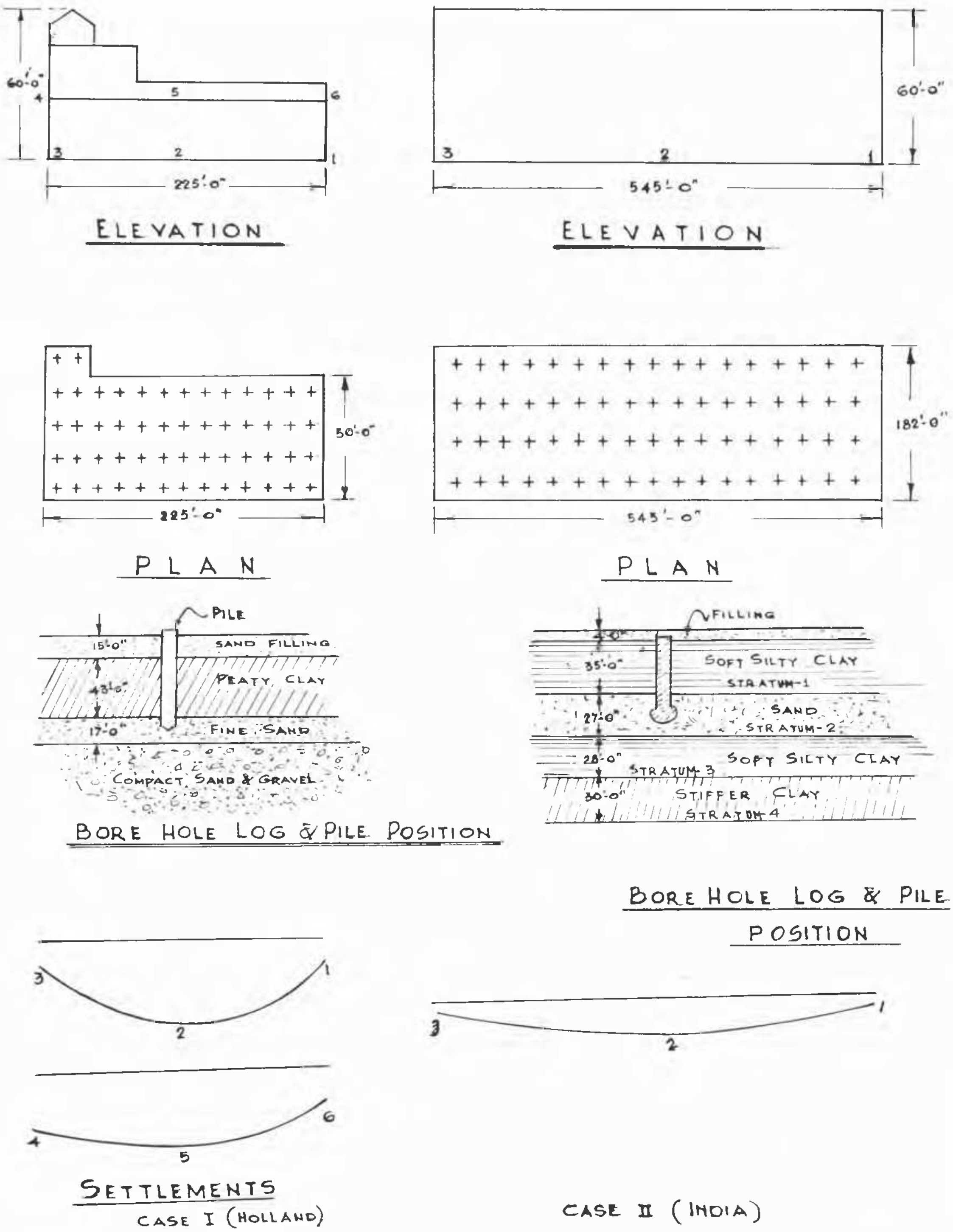

FIG. 29. Case histories in Holland and India.

entirely annulled the resistance of Stratum I but also increased the imposed load on the pile by 136 tons. This mechanism may easily cause settlement.

An example is given below where large settlement took place although there was no soft layer below the pile foundation. In Holland a factory building, 225 ft × 50 ft, was constructed on 66-ft-long piles (Széchy). It was not possible to drive deeper piles because the resistance was so great that the tops of the piles were crushed. In this case the largest settlement recorded before remedial measures were undertaken was more than 2 ft. Subsurface investigation revealed that there was a top layer, 15 ft thick, of sand filling; below this was a 43-ft-thick layer of peat mixed with some clay overlying a layer of fine sand 17 ft thick; and below that was compact sand and gravel. In this case the cause of settlement can only be explained by negative skin friction which occurred due to the great compressibility of the peaty clay.

Let us now compare this with the problem under discussion (Fig. 29). The similarities between Case I and Case II are: (1) that in both cases the piles are resting on sand layers above which there are compressible layers; and that negative skin friction may occur in these conditions and (2) in both cases the shapes of the settlement diagrams are similar. The dissimilarity between Case I and Case II is that in Case I there is no soft layer under the piles and in Case II there are soft layers under the piles.

The above findings should reasonably suggest that the mechanism of negative skin friction may more appropriately explain the settlement in Case II that is the problem of this paper and that the seat of settlement is the sand layer and not the clay layers. The settlement analysis with time for clay layers has not been given in the paper *4/17*. Nor have pressure *versus* void ratio data which indicate the state of consolidation of a stratum been supplied for Stratum I. The state of consolidation is an indicator of the possibility of the occurrence of negative skin friction. Therefore, in the absence of these data it is difficult to come to any definite conclusion.

The authors have also indicated that the average length of piles (42 ft) was small compared with the width of the building. What length of piles are the authors suggesting in this case? For the sake of argument, let us assume that the contention of the authors is correct, that is, that the consolidation of clay Strata III and IV is primarily responsible for the settlement. In that case longer pile lengths (up to 90 ft) would have caused greater settlement as the tips of the piles would be nearer the soft layers (Strata III and IV). Moreover, it may be noticed for the example cited that there was a settlement exceeding 2 ft even with 66-ft-long piles for a 50-ft-wide building.

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L. FERNANDEZ-RENAU (Spain)

The interesting paper by Dr. *Begemann (4/1)* suggests some brief remarks. In July 1965, pull-out tests on *in-situ* piles were conducted at the site of a future power station at San Adrian (Barcelona, Spain), owned by Hidroelectrica de Cataluna, S.A. At the site of the tests, exploratory soil borings and static penetrometer soundings were performed. Fig. 30 shows (*a*) the soil profile based on exploratory borings; (*b*) the Dutch cone resistance diagram; (*c*) the grain size distribution curves; (*d*, *e*) the results of pull-out tests on two 630-mm-diameter piles, 10 m long.

The soil is an alluvium of the Besos River delta. The tests were performed at a location about 200 m from the shore of the Mediterranean Sea.

Pile (d) was installed by excavating inside a steel casing, which was driven in successive stages. The soil was removed at each stage using a bailer, which never was lowered below the casing shoe. As soon as the cased boring reached the

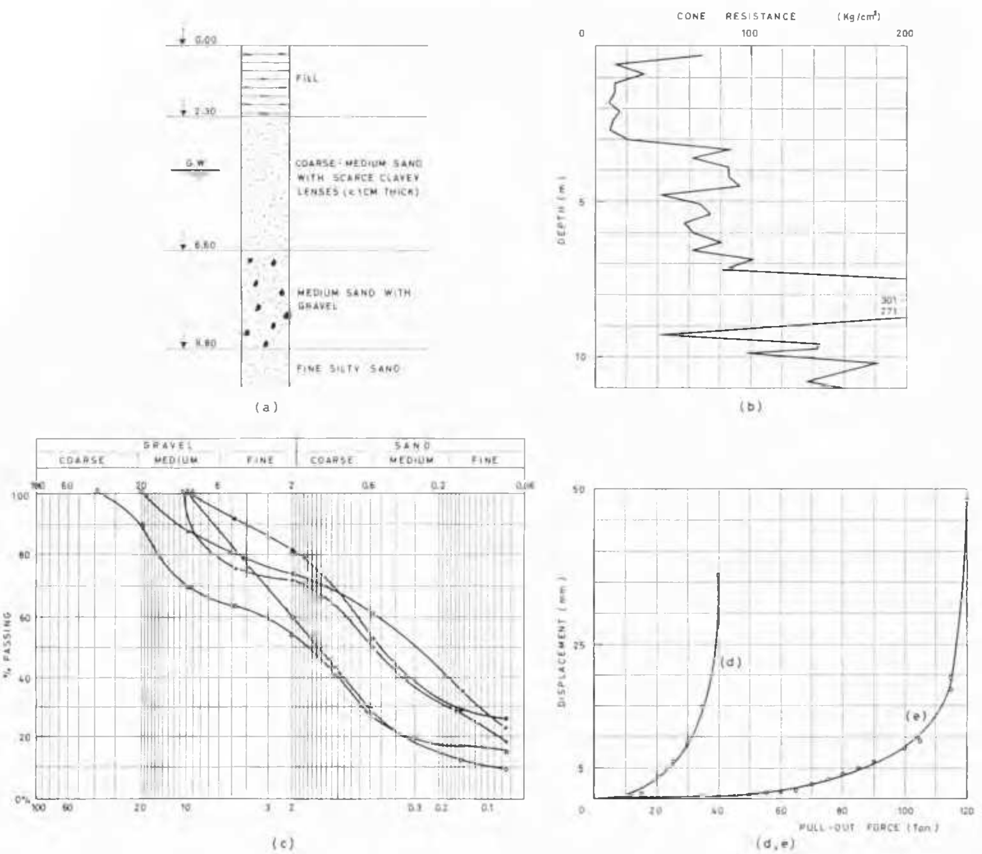

FIG. 30. Foundation conditions and results of pull-out tests: (*a*) soil profile; (*b*) Dutch cone resistance diagram; (*c*) grain size distribution curves; (*d*, *e*) pull-out test results on two 630-mm-diameter piles, 10 miles long.

final depth of 10 m, tremie concrete was placed and the casing finally extracted.

Pile (e) was installed by continuously driving a casing, externally lubricated by bentonite mud, down to a final depth of 10 m. The soil was then removed from within the casing and the water in the hole was replaced with bentonite mud introduced from the bottom. The casing was pulled out slowly to allow time for the bentonite mud to deposit a cake on the walls of the hole. Tremie concrete was then placed in the unlined hole.

Both piles were reinforced by 8 rods each 25 mm in diameter and a 6-mm-diameter spiral at 20-cm pitch. Piles (d) and (e) were 4.30 m apart axis to axis. Maximum pulling forces were 40 tons for pile (d) and 120 tons for pile (e).

The cone sounding was performed with the 10-ton hydraulic rig manufactured by Gouda Maschinenfabriek (Holland) prior to the installation of the piles. No adhesion jacket was provided.

The log (Fig. 30a) consists mainly of coarse sand down to 6.60 m and gravelly sand from 6.60 m to 10 m. To compare the San Adrian tests with the results presented by Dr. Begemann (*4/1*), the writer has computed the maximum pulling force of an imaginary "pointless prefab pile," 630 mm in diameter and 10 m in length. The computation was based on data presented in Fig. 1 of Dr. Begemann's paper. By taking a line slightly below the one labelled "sand (coarse-gravel)" and using the cone resistances (Fig. 30b), a maximum pull-out force of 70 tons is computed. It is interesting to compare the computed value of 70 tons with the actual results of 40 tons for pile (d) and 120 tons for pile (e).

Fig. 1 in Dr. Begemann's paper refers to "undisturbed, natural deposits situated below the ground water level." At San Adrian, the water table is located at a depth of 4 m; but practically no friction should be expected from the upper fill soils (10 kg/sq.cm. cone resistance). Presumably the reduction coefficient curve for a "pointless prefab pile" (Fig. 1 of Dr. Begemann's paper) has been based on pull-out tests on precast piles of smaller diameter—and different shape—than the imaginary 630-mm-diameter pile for which our calculations were made.

The installation procedure of pile (d), no matter how carefully performed, may cause some soil density reduction. This may explain why the maximum pull-out force of pile (d) is less than that of the imaginary 630-mm-diameter pile. In this connection it should be remembered that as soon as the casing is extracted, the concrete rigidity plays an important part in the ability of the concrete to fill the annular space left by the extraction of the casing. For type (d) piles it seems logical to assume a reduction coefficient curve located to the right of the one labelled "pointless prefab pile" in Fig. 1 of Dr. Begemann's paper. In other words, the reduction coefficient for pile (d) might be about 17 per cent.

The installation procedure of pile (e) does not follow the normal processes using bentonite drilling muds. Pile (e) was installed by using the equipment available at the site. The purpose was to determine the possible lubricating effect of the bentonite cake on the lateral friction of the *in-situ* piles. The process of installation of pile (e) minimizes the soil density reduction. However, it does not introduce any appreciable compaction effect, as can be expected from a displacement pile.

Since the maximum pulling force of pile (e) is 1.7 times that of the imaginary driven pile, the writer is inclined to think that driving an isolated pile in dense cohesionless soil may cause a reduction in the density of the soil close to the pile, associated with an increase in density of other soil zones. In this connection it seems pertinent to remember the "decompression channels" referred to by Kérisel (1961).

From a pull-out resistance standpoint it could be concluded that bentonite mud excavated piles behave more satisfactorily than displacement piles. However, more evidence than the tests presented here would have to be gained. The reduction coefficient curve for bentonite mud excavated piles would fall at the middle of the corresponding graph shown in Fig. 1 of Dr. Begemann's paper. (The reduction coefficient for bentonite mud excavated piles would be about 50 per cent.) This cannot be understood in the case of loose cohesionless soils, remarkably compacted by displacement piles.

The direct transposition of cone sounding results to the estimate of pulling resistance of *in-situ* excavated piles does not seem to be possible in the same way as presented by Dr. Begemann for displacement piles.

For a specific type of soil, Fig. 1 of Dr. Begemann's paper shows the same reduction coefficient regardless of soil density. This seems to indicate that both Dutch cone and displacement pile would change the density of soil to the same relative extent. The reduction coefficient would account for the scale effect, shape of friction jacket, and difference between materials (steel cone and concrete piles).

In any case, the extremely good behaviour of pile (e) cannot be understood if a lubricating effect is associated with the bentonite cake. Such a cake must have been squeezed and partially destroyed by the concrete either mechanically or chemically through the stiffening capacity of the cement calcium ions.

The average pull-out unit resistance of pile (e) is 0.6 kg/sq.cm.

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Y. KOIZUMI (Japan)

With respect to the topics proposed for discussion, I would like to describe briefly some results of the test conducted by our research group (Chairman: Hisao Nagai) on the pore water pressures induced by pile driving and on the ultimate bearing capacity of pile foundations.

The layout of the test foundations is shown in Fig. 31. A total of 10 steel piles, 30 cm in diameter, with closed ends, were driven statically by a winch device to a depth of

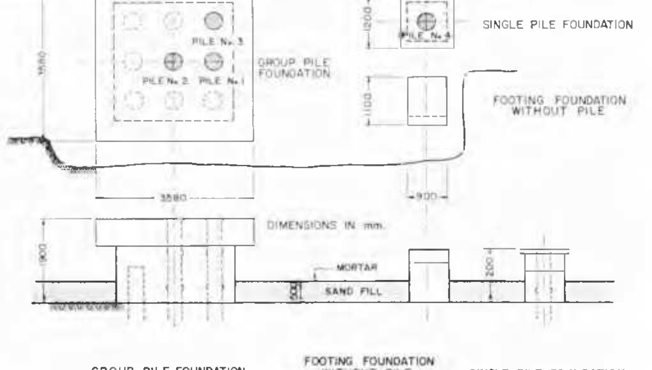

FIG. 31. Layout of experimental foundations.

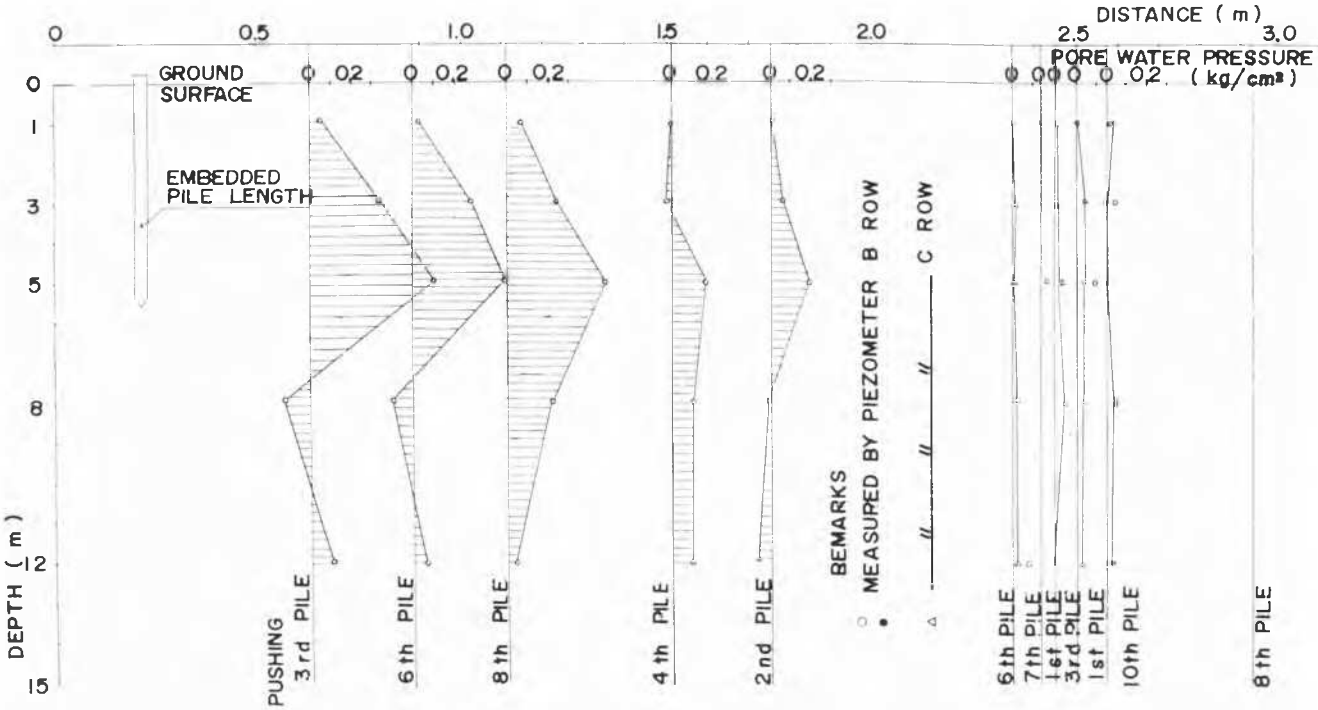

FIG. 32. Development of pore water pressure in subsoil during installation of piles.

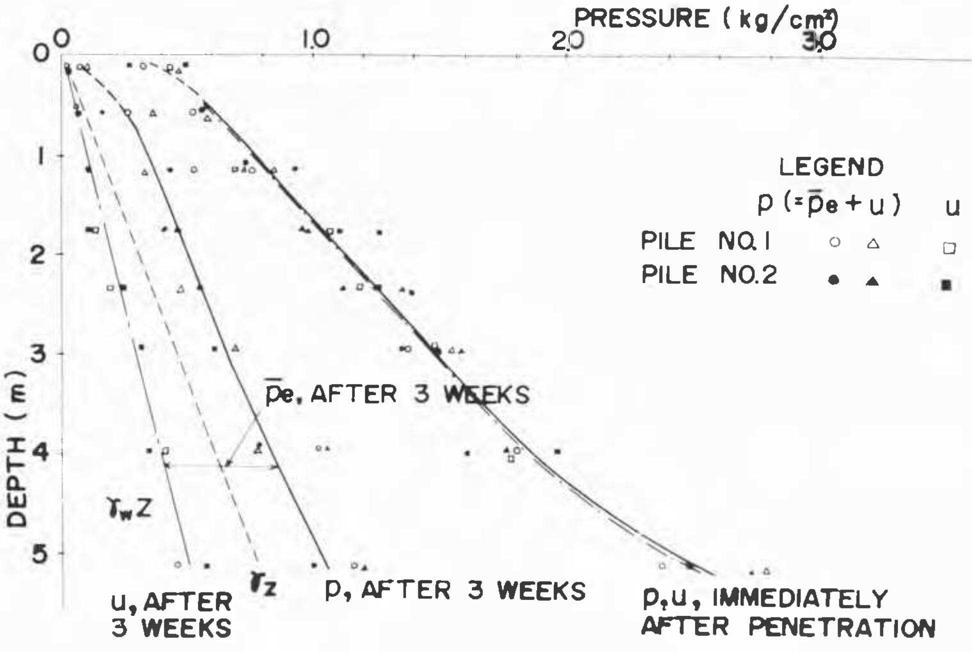

FIG. 33. Change in effective earth pressures and pore water pressures acting on pile surfaces after pile installation.

5.55 m. The pile group consists of 9 piles, spaced 90 cm centre to centre. The subsoil consists of soft, normally consolidated, silty clay which extends to a depth of about 12 m. The shearing strength of the clay varies from 0.2 kg/sq.cm. near the surface to 0.4 kg/sq.cm. at the tip of the piles and its sensitivity is very high, the natural water content being nearly equal to the liquid limit.

The earth pressures and pore water pressures acting on the surfaces of the piles were measured, during and after penetration of the piles, by pressure cells mounted on the piles. The pore water pressures were also measured in the soil at different distances and depths by piezometers.

The distribution of the maximum pore water pressure induced by driving the piles is shown in Fig. 32. The pore water pressure in the soil close to the piles increased with depth and showed a slight decrease just below the pile tip. The radial distance influenced by pile driving is practically limited to about six times the pile diameter.

During and immediately after pile driving, the lateral effective stress at the pile surface was zero since the observed total pressures were approximately equal to the pore water pressures (Fig. 33). This implies that the clay surrounding the pile lost its shearing strength due to severe disturbance. As time elapsed, the pore water pressure decreased gradually over about three weeks to the hydrostatic pressure. It should be noted that the lateral effective stress acting on the pile surface at that time was greater than even the effective overburden pressure. The stress, which did not change during the following test period (8 months), might be considered the reconsolidation pressure to the surrounding remoulded clay.

After pile caps were constructed, we made load tests on the pile foundations which were preceded by vibration tests and lateral loading tests. Fig. 34 shows the load-settlement curves for the single pile and the group pile foundation. The ultimate bearing capacity for the group pile foundation is not well defined, suggesting that failure occurred progressively. The maximum soil reactions at the bottom of the pile caps, which were measured by load cells, were only a small fraction of the ultimate bearing capacity of the footing without piles.

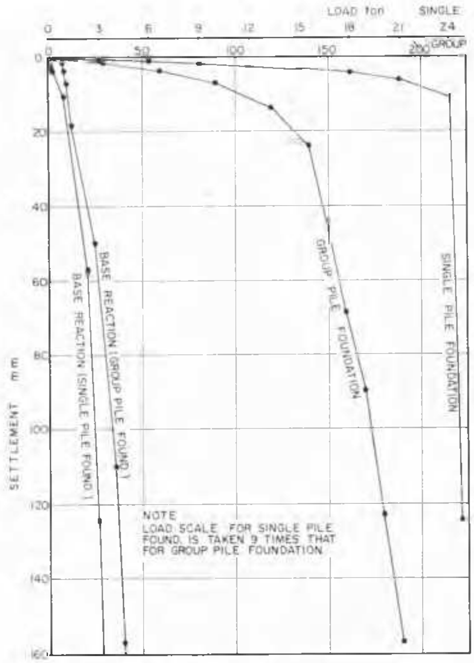

FIG. 34. Load-settlement curves.

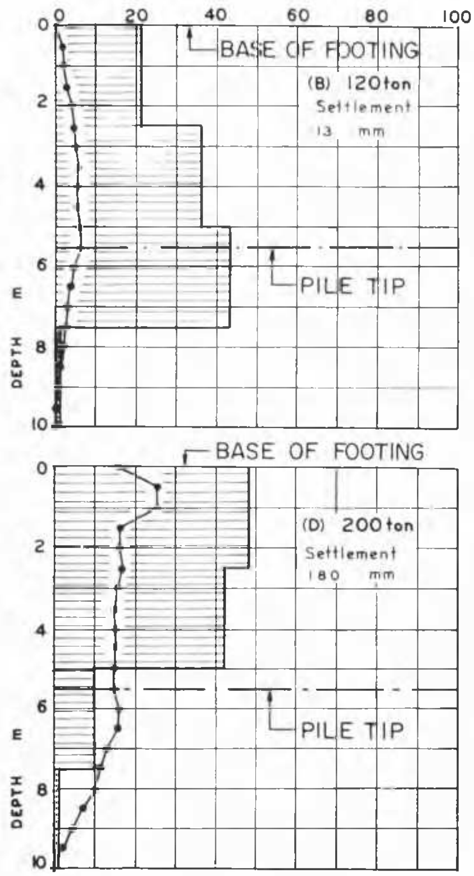

FIG. 35. Distribution of compression zone beneath footing.

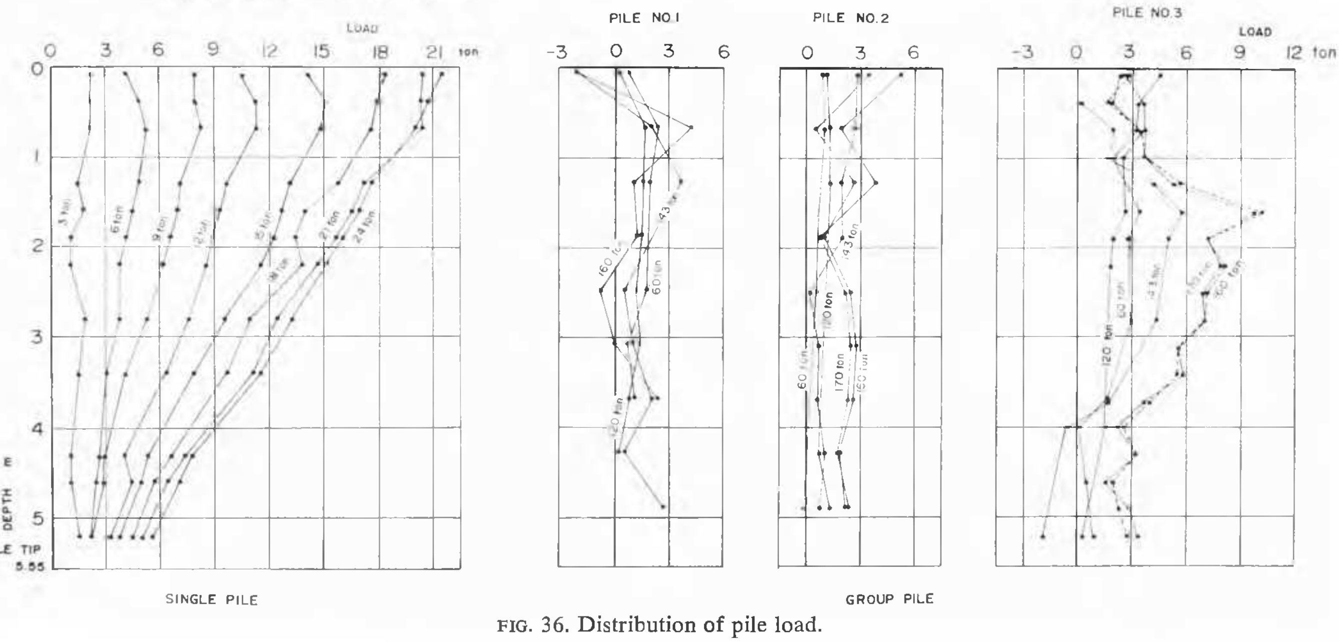

FIG. 36. Distribution of pile load.

The vertical and horizontal movements of the soil inside and outside of the pile group were also measured by the settlement points and inclinometer. The result shows that until the incipient failure started, the soil enclosed within the pile group did not move much, except near the pile tips (Fig. 35). This observation is also supported by measurements of the earth pressures acting on the pile surfaces.

The distribution of pile load measured with SR-4 gauges attached to the piles is shown in Fig. 36. The maximum shearing stresses developed on the surface of the single pile were approximately equal to the shearing strengths of the original soil. For the pile group, the corner piles carried as much as twice the load carried by the centre pile, and 1.5 times the load at the edge piles, for the working load. This inequality of load distribution among the piles tended to decrease after the incipient failure, but persisted even in the ultimate state.

T. W. Lambe and H. M. Horn (U.S.A.)

In our paper *(4/12)*, the plots of time *versus* building movement and pore water pressure terminate in December, 1963, and January, 1964, respectively. Considerable additional data have been obtained and are presented in Table III. In addition to the movement of the reference points referred to in Fig. 2 of the paper (Points 3, 4, 9, and 10), the movements of Points 1 and 12 are also presented.

To date, the north face of Building 10 has undergone a net settlement since the start of pile driving of between 0.07 and 0.10 ft. Of this total, between 75 and 90 per cent occurred in the period between December, 1963, and June, 1965. During this time, the excess pore water pressure head has dissipated to a few feet at most.

The plot of settlement as a function of time shows that the rate of settlement has decreased considerably since the end of 1963. However, during the past year, the north face of the building appears to have settled as much as 0.02 ft, even though only about 2 feet of excess pore water pressure was dissipated during this period. It is possible that a portion of this apparent settlement is the result of surveying inaccuracies.

Comparison of the net settlement at Point 1 with that at Point 3, and that at Point 10 with that at Point 12 indicates that Building 10 has rotated about the junction between it and the rest of the M.I.T. building complex. It is concluded from this that the effect of pile driving for the Materials Center on the settlement of the nearby buildings extended over a distance of approximately 100 ft from the construction site. The rotation of Building 10 due to settlement resulting from pile driving corresponds to a maximum settlement distortion (δ/L) of 1/1000. Observations made by the authors of the exterior brick facing of this building revealed no noticeable cracking even though portions of the building had undergone settlements of over 0.7 ft before the start of pile driving for the Materials Center (Horn and Lambe, 1964).

It should be noted that the maximum net settlements measured to date are approximately twice the value which was estimated using the stress path method. (The predicted gross settlement is close to the measured value.) The displacement of clay during pile driving below the pre-augured zone may have disturbed the soil beneath Building 10. An increase in compressibility from disturbance might explain why measured net settlements are more than estimated values.

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K. Y. Lo (Canada)

In his general report, Dr. Kézdi mentioned that a comparison of the results of field measurements in paper *4/13* with the values obtained by Nishida's theory would be of interest. The following brief discussion will show the limitations of the theories for estimation of pore pressures set up due to pile driving based on calculation of stresses around the pile assuming the soil to be a viscous, elastic, or plastic solid.

The equation for the induced pore water pressure within the failure zone is given by Nishida (1963) as

$$u_r = \tfrac{4}{3}C_u \log_e (R/r) + (A - \tfrac{1}{3})\, C_u\sqrt{[3 + 4 \log_e^2(R/r)} \quad (1)$$

for $r_1 \leqslant r \leqslant R$, where R and r_1 are the radius of the failure zone and pile, respectively.

From the Eq 1, it may be deduced that (*a*) the pore pressure decreases rapidly from the pile skin to the outer limit of the failure zone and (*b*) the pore pressure increases only slightly with depth according to the increase in undrained shear strength, C_u.

These predictions are obviously contradictory to the results of field observations as reported in paper *4/13* in which it was shown that (1) within the failure zone, the pore pressure is sensibly constant and (2) the maximum pore pressure increases with the initial effective overburden, not with the undrained strength which has only a small rate of increase with depth.

Moreover, a knowledge of the actual radius of the failure zone is necessary to estimate u_r in Nishida's theory. This prerequisite is not required in the author's method which is based on a consistent theory for pore pressures.

REFERENCE

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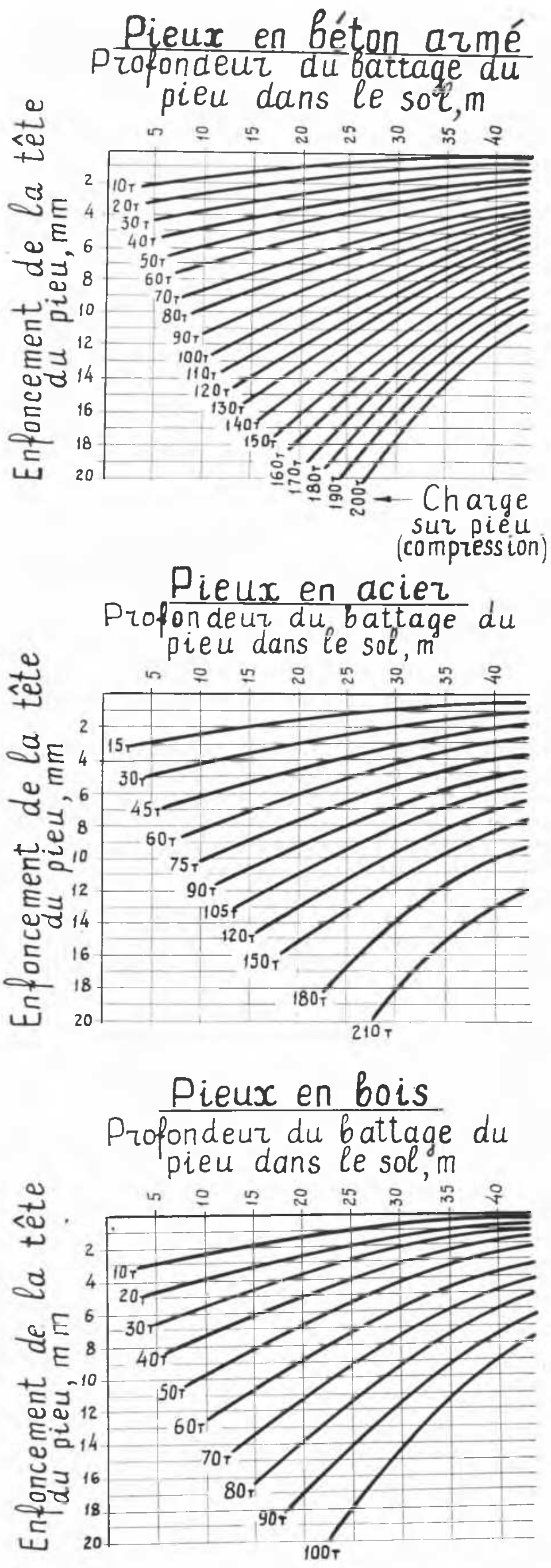
A. A. Luga (U.R.S.S.)

Dans plusieurs cas de l'établissement de projet et de l'exécution des fondations sur pieux des ouvrages on exige de connaitre des valeurs de l'enfoncement qui peuvent être obtenues sous l'action de la charge calculée en exploitation. Comme on sait, il existe plusieurs méthodes de calcul de l'enfoncement des fondations sur pieux. L'emploi de ces méthodes exige des données détaillées relatives aux propriétés physico-mécaniques des sols de fondation compressibles. Pour la construction courante il est impossible d'obtenir pratiquement de telles données de calcul à cause du manque de

TABLE III. MOVEMENTS OF BUILDING 10 AND PORE PRESSURES (SINCE DECEMBER, 1963)

Date	Net settlement (ft × 10^{-2})				Water level elevation (ft)				
	Pt. 1	Pts. 3 and 4	Pts. 9 and 10	Pt. 12	PMS-1	PMS-2	PMS-3	PMS-4	Well 13-1
Dec. 20, 1963	0.0	2.6	0.5	−0.4	20.6	10.4	24.4	11.0	5.0
May 16, 1964	2.3	6.6	3.9	1.4	16.2	12.3	16.2	12.8	10.6
Dec. 30, 1964	3.5	8.9	5.1	1.5	13.6	12.1	14.1	12.1	12.2
June 20, 1965	4.9	10.2	7.1	2.9	12.7	12.2	13.2	12.4	12.2



temps et de travail considérable nécessité. Pour les travaux de fondations sur pieux, le calcul de l'enfoncement est compliqué par les difficultés liées à la technique de l'exécution des travaux. Dans le cas des sols homogènes de fondation les valeurs de l'enfoncement des pieux battus pratiquement au même niveau de profondeur sont assez largement variables en fonction de la résistance au battage du pieu.

Les pieux doivent être battus jusqu'aux sols favorables de fondation au point de vue des constructeurs. L'auteur rapporte comme favorables tous les sols cohérents et non cohérents évalués par les coefficients de consistance supérieures à 0,5 et tous les sols rocheux; dans ce cas-là les pieux sont battus au refus correspondant à la charge calculée; les outils de battage des pieux doivent être appropriés au poids des pieux à enfoncer et à la charge de calcul. Sans maintien de ces conditions imposées et importantes pour l'exécution des fondations sur pieux, on peut arriver à des valeurs élevées de l'enfoncement des ouvrages.

En se basant sur les résultats des essais *in situ* statiques des pieux battus en béton armé, en acier et en bois, on établit la dépendance dont il faut tenir compte (fig. 37) et qui permet de prédéterminer les valeurs de l'enfoncement des fondations projetées sur pieux sollicités dans les conditions des pieux isolés sans l'influence des pieux voisins; pour cela il ne convient pa de procéder aux essais statiques des pieux et on peut ne pas avoir des données des essais à la compression des pieux et des sols de fondations.

Aux fondations sur pieux sollicités dans les conditions des pieux isolés sans l'influence des pieux voisins on rapporte des constructions qui satisfont à une des conditions suivantes.

1. Les pieux doivent être appuyés sur les sols rocheux de résistance différente. Dans le cas des sols rocheux ébouleux, la partie inférieure du pieu est battue dans ceux-ci au refus voisin de zéro;
2. La distance entre les axes des pieux suspendus (frottement) dans le plan de leur partie inférieure doit être plus de six fois l'épaisseur du fût;
3. La partie inférieure des pieux doit être battue dans les couches de gros éléments (cailloux, galets, blocs arrondis) ou dans les sols cohérents de la consistance rigide.

Ayant à leur disposition des données connues (matériau de pieu, profondeur du battage dans le sol et charge de calcul) et utilisant de tels graphiques le chercheur et le constructeur atteignent la valeur cherchée maximum de l'enfoncement des pieux. Cette valeur peut être imposée aux fondations en exploitation.

Aux limites des charges admissibles sous les pieux isolés on agit sur la zone faible de sols compactés par marteau-pilon; en conséquence de cela, des mêmes sols procurent des propriétés à la compression rapprochées (suivant des valeurs) indépendamment de leur nature. C'est pourquoi des sols sur lesquels on prend appui des pieux isolés suspendus (frottement) ne sont pas différenciés des types particuliers. Comme règle, les dimensions de la section transversale du pieu (ϕ = 25 à 60 cm) sont dépendantes de sa longueur; comme on le sait, en prolongeant la longueur des pieux pour la plupart on augmente leur section en travers.

On représente sur la fig. 38 les caractéristiques d'enfoncement pour des pieux camouflet ayant la base élargie (ϕ = 1,2 à 1,5 m). On admet d'effectuer des calculs de l'enfoncement des pieux-poteaux par la méthode de calcul direct à la compression du fût de pieu en fonction de la charge de calcul

FIG. 37. Caractéristiques d'enfoncement et de battage de divers pieux.

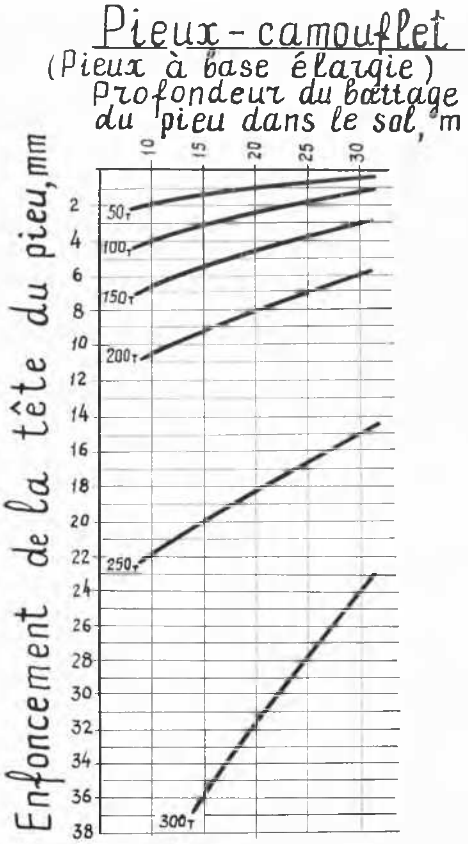


FIG. 38. Caractéristiques d'enfoncement et de battage des pieux camouflet.

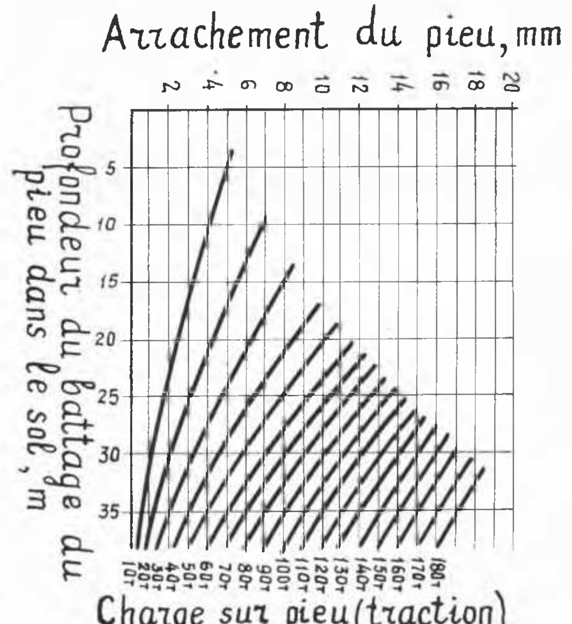


FIG. 39. Caractéristiques d'arrachement et de battage des pieux.

sans tenir compte de l'influence des forces de frottement des sols au fût du pieu.

On montre sur la fig. 39 des valeurs de l'arrachement des pieux battus à traction sous des charges différentes. Les différences d'allongements des fûts de pieux en matériaux divers sous efforts d'arrachement sont pratiquement négligeables.

La relation dont on doit tenir compte pour déterminer l'enfoncement des pieux isolés n'est obtenue qu'à partir de la charge axiale; les graphiques ne sont pas appropriés au cas de chargement direct des sols entourés par le remblaiement de certains matériaux ou à la pose des blocs environnants des pieux. Le complément à la charge appliqués latéralement provoque des valeurs élevées de l'enfoncement; l'application de celle-ci à un côté mène à la fracture des pieux et à la destruction de l'ouvrage.

Il est à noter que lorsque des essais statiques sur pieux sont effectués à bref délai, les enfoncements n'atteignent que certaines limites relatives de la valeur de l'enfoncement (dans les essais à la charge de calcul), soit 90–95 pour-cent de l'enfoncement possible total du pieu dans l'ouvrage. En autant que les courbes de calcul sont représentées comme les enveloppes des valeurs expérimentales de l'enfoncement, cet écart faible est pratiquement absorbé par les limites des enveloppes.

Pour des fondations sur pilotis élevés, des valeurs de serrage (traction) des fûts de pieux (à la distance du pied des pilotis au niveau des sols superficiels) doivent être ajoutées aux valeurs calculées de l'enfoncement (arrachement) des pieux.

Pour dresser ces graphiques, des résultats de 1260 essais statiques sur pieux et plus de 10 000 données expérimentales relatives à l'enfoncement des pieux ont été utilisés.

G. Meardi (Italy)

I have read very interesting papers and heard learned discussions on the evaluation of the bearing capacity of foundation piles on the basis of the resistance of the soil and of the method of installation of the piles. One (*Lo and Stermac, 4/13*) dealt with pore pressure set up in clayey silty soils during the driving of the piles and the increase in bearing capacity owing to the consolidation due to dissipation of increase in pore pressure. Therefore I think it interesting to refer to my latest experiences which clearly show the phenomenon and indicate, in my opinion, a criterion which, though approximate, will enable us to take advantage of the phenomenon as I have done so far with success.

In the *Proceedings* of the Roorkee Symposium (pp. 51–9) I reported that in a work (indicated there as No. 3) executed off Gela (Isle of Sicily—Italy) in silty clayey soils the set of the driving of 750-mm-diameter piles came down in 72 hours from 100 to 150 mm to 3 to 4 mm, Fig. 40. The loading tests carried out some weeks after the driving indicated bearing capacities that could be calculated with the dynamic formulae adopting the set value obtained after 72 hours (Fig. 41) and applying normal safety coefficients.

After this experience I have always believed that, as far as driven piles in silty clayey soils are concerned, we can adopt the bearing capacity evaluated on the basis of the set obtained 5 to 6 days after driving the pile with 10 blows of hammer. I have observed in later experiences that if we go on driving conical piles with further blows, the set increases, as if the pile bearing capacity should diminish when the pile is driven deeper. I think that the resistance to driving is temporarily diminished because of the increase in pore pressure due to the increased displacement of the soil, and that therefore the actual future resistance of the pile is represented much better by the first set obtained after a few days.

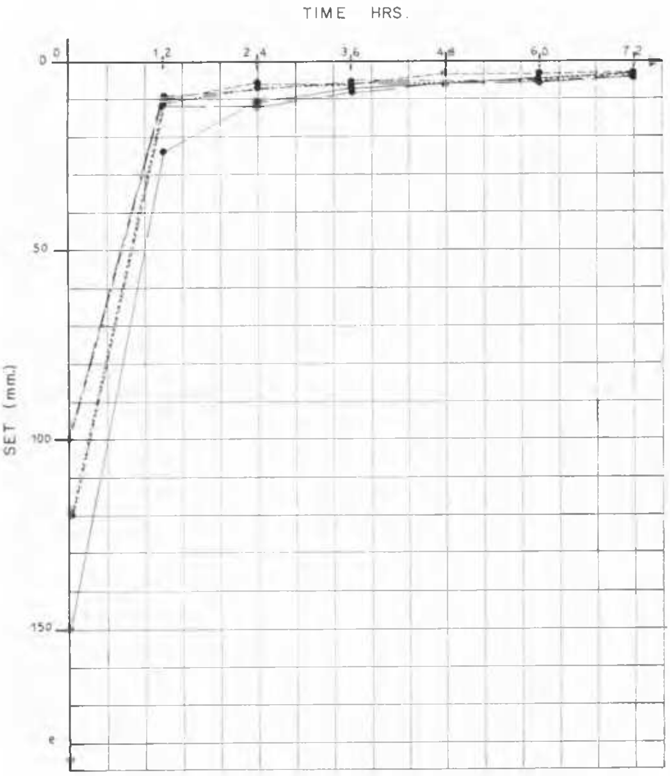


FIG. 40. Set *versus* time for 4 piles: ϕ 75 cm, driven 12 m; hammer weight = 6.75 tons; pile weight = 12.5 tons; fall = 0.8 m.

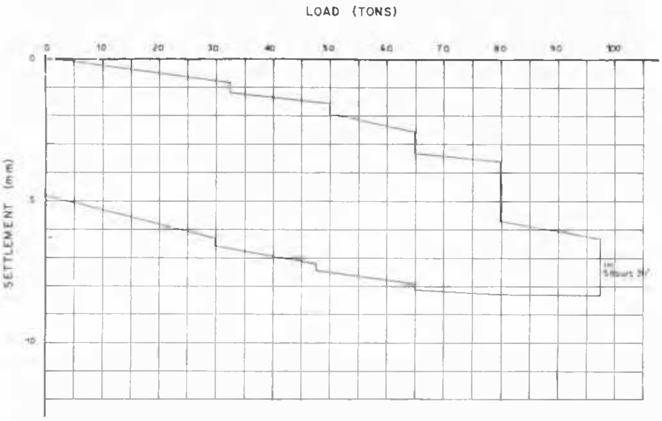


FIG. 41. Load test of a pile. L in the soil = 12 m; set at the end of the first driving = 100 mm.

Of course it is not necessary to test all the piles again. We can calculate the ratio between the set of the first driving and the set obtained after a few days on one pile out of ten and adopt the ratio so obtained with caution.

With piles cast *in situ* in a driven casing it is not always advisable to leave the casing in the soil for some days in order to estimate the set again; but it is even worse to give up the benefit of consolidation, as we nearly always do. In these cases we can calculate the pile by prudently applying the static formulae of bearing capacity based upon the strength measured in laboratory or, better, *in situ* (with the vane test) which are not influenced by the temporary phenomenon of pile driving; the value obtained will be corrected on the basis of loading tests executed on the pile after one or, better, after a few weeks.

In all the works I have executed I noticed that the resistance of the pile calculated with loading tests is a little superior to that calculated with the static formulae and far superior to that calculated on the basis of the set at the end of the casing driving exactly as we have seen for piles precast and then driven.

H. Mori (Japan)

Ménard (4/15) tries to establish, using his pressuremeter test, some rules regarding bearing capacity or settlement of foundations as a function of parameters. In paper *1/16, Meigh and Greenland* present a comparison of the results of a Ménard pressuremeter test and of the strength of soft rock obtained by laboratory testing. I would like to make some comments on the relationship between the Ménard test and laboratory testing of soil with respect to the strength and consolidation characteristics of soil.

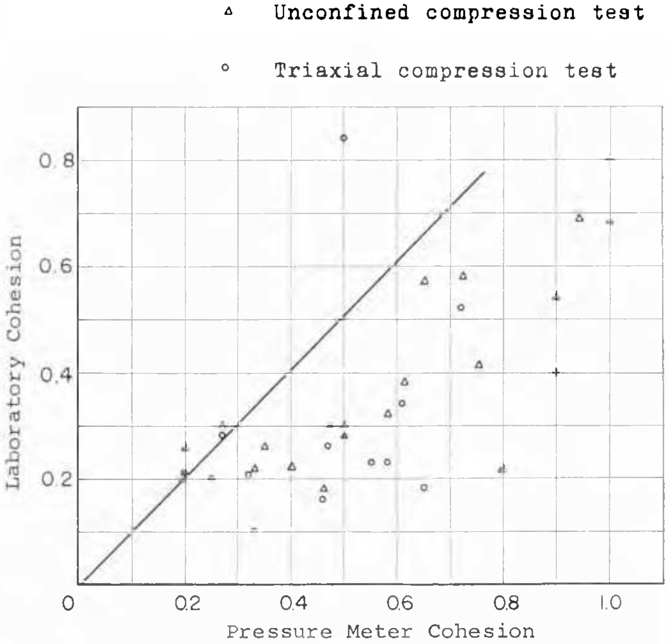


FIG. 42. Relationship between cohesion by pressuremeter test and by laboratory test.

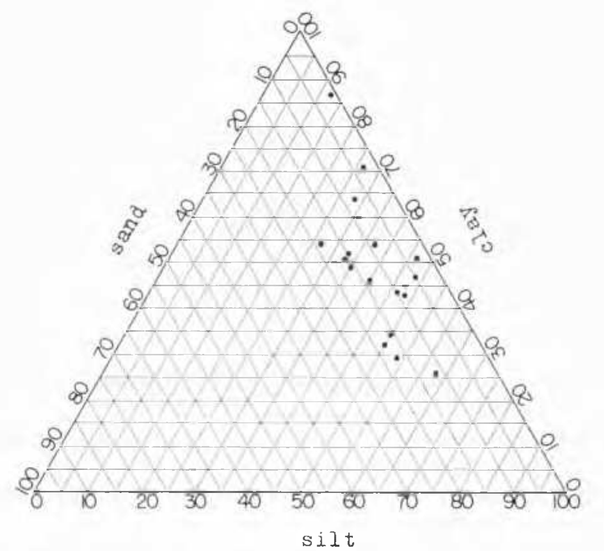


FIG. 43. Gradation of samples.

In Fig. 42, the shearing resistance of cohesive soils obtained by laboratory testing was plotted comparatively with that obtained by pressuremeter test. As shown in Fig. 43, soil samples containing less than 20 per cent sand were used for this study. The samples were taken by a 75-mm, thin-walled tube sampler with a stationary piston. The triaxial or unconfined compression test was conducted by strain control measurement at a strain rate of 1.5 to 2.0 per cent per minute for a cylindrical specimen 35 mm in diameter.

According to the results shown in Fig. 42, the value of shearing strength of a cohesive soil calculated from the limit pressure by a pressuremeter test is generally higher than the value obtained by laboratory testing. The source of this considerable difference in strength has not yet been completely analysed, but the disturbance due to sampling and trimming is supposed to be one of the reasons why the cohesion value obtained by laboratory testing is lower than that obtained by pressuremeter test.

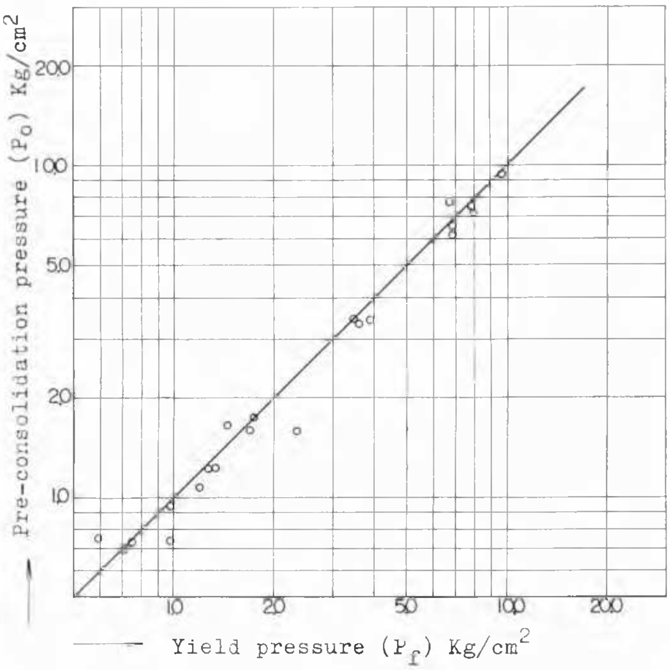


FIG. 44. Relationship between pressuremeter test and consolidation test.

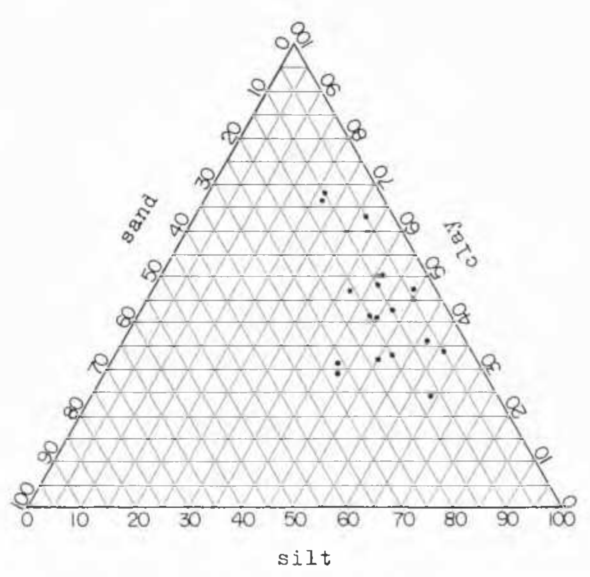


FIG. 45. Gradation of samples.

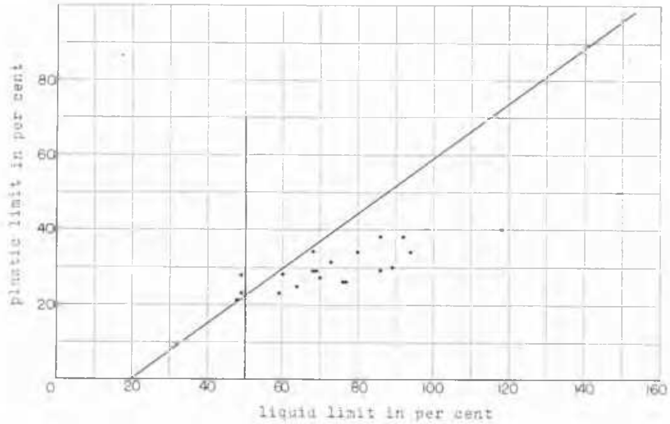


FIG. 46. Plasticity chart.

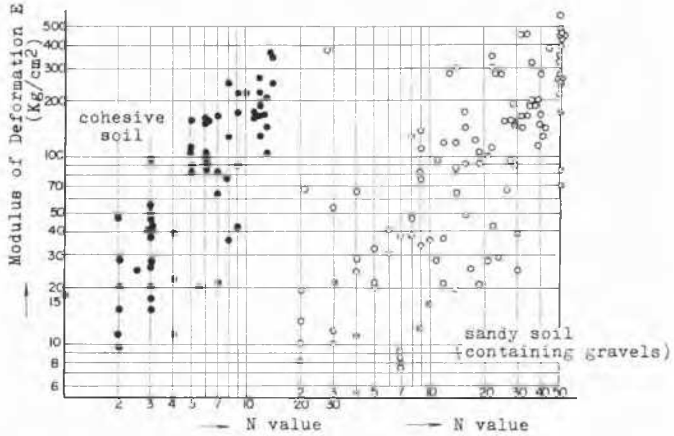


FIG. 47. Relationship between N-value and modulus of deformation, E.

After the consolidation tests for samples taken at ten different sites in Japan, a preconsolidation pressure for each sample was computed, using the method suggested by Casagrande. The yield pressure of clayey soil was measured by Ménard's pressuremeter at equivalent elevation in drill holes. The values of preconsolidation pressure thus computed are compared with values of yield pressure as shown in Fig. 44.

The grading texture and Atterberg indices of the samples are illustrated in Fig. 45 and Fig. 46, respectively. The samples having preconsolidation pressures higher than 3.0 kg/sq.cm. are considered as overconsolidated clay. The yield pressure P_f by the pressuremeter test corresponds to the effective stress which is computed by subtracting the hydrostatic pressure from the value measured by the pressuremeter. As shown in Fig. 44, the preconsolidation pressure for either normally or overconsolidated clay agrees, with a fairly high degree of accuracy, with the yield pressure found by pressuremeter.

Some samples of overconsolidated clay in Osaka indicated a preconsolidation pressure about 30 per cent lower than the yield pressure obtained by pressuremeter. However, for this particular clay, the research of some authors has shown that the clay had been prestressed in a lateral direction by a geological folding motion. It has also been experimentally proved that, for this clay, the preconsolidation pressure for a specimen subjected to horizontal pressure is considerably higher than the preconsolidation pressure in a vertical direction. Consequently, the yield value by the pressuremeter test may be a helpful guide for us to use in estimating the preconsolidation pressure of a cohesive soil, especially when the natural deposit is isotropically consolidated.

The relationship between modulus of deformation obtained by pressuremeter test and number of blows by standard penetration test is illustrated in Fig. 47. A certain relationship may be recognized from the diagram, but the plotted points are quite scattered. A closer relationship might be obtained if the comparison were made for a soil which is similar in its geological origin, grading texture, or Atterberg limits. But it seems to be difficult to derive a simplified general rule for the relation between number of blows and modulus of deformation.

H. MUHS (Germany)

In connection with deep foundations, especially for pile foundations in Berlin, it has been customary for perhaps 10 years to investigate the density of sandy deposits, which are usually the bearing stratum for the piles, by soundings. For this purpose we use a deep-sounding apparatus developed by us just after the Second World War. Working with only one tube, we measure the point resistance by the alteration of a vibrating wire in the lower end of the tube. With the apparatus we are able to investigate the subsoil at depths down to about 25 m and up to a resistance of 5000 kg.

During the 20 years since we began to use this method we have observed that in many cases the point resistance, which was rather high in the upper layers of uniform fine to medium sands (Fig. 48), dropped in the deeper layers and that very often these layers consisted of coarser and more variable sands. As it is just these sands which generally are estimated to be a good material and to be a particularly good foundation stratum for piles or caissons, we were faced with an unfortunate situation. According to the results of borings and to our general experience with such sand, a layer of especially high bearing capacity was reached, whereas according to the results of the soundings this layer, with a low point resistance and therefore low relative density, had to be classified as a not particularly suitable stratum for the concentrated loads of point-bearing piles.

This problem was met rather often. Because of its importance in practical work we performed several series of systematic soundings in three typical non-uniform materials

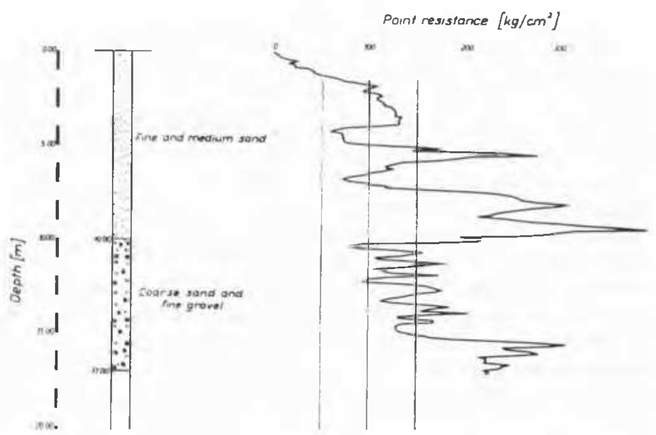


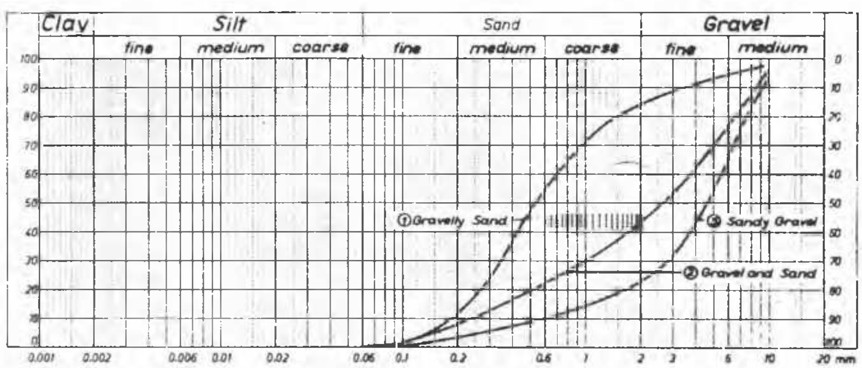
FIG. 48. Typical results of deep-sounding apparatus for uniform sand over less uniform sand.

(Fig. 49), consisting of a non-uniform sand, a sandy gravel, and a mixture of sand and gravel. As we do not estimate sounding tests in boxes or tanks with limited dimensions because of the undesirable influence of the walls, we carried out the tests in a larger test pit at our testing area (Fig. 50), building up the materials to a height of 3.3 m in thin layers, compacting them in such a way that at first three fields with a low density and three fields with a high density were produced. Later the three materials were brought in with a medium density.

During filling the density of the different test fields was checked constantly by normal methods. In doing this it proved very difficult to determine the density of the loose non-uniform materials exactly. Therefore the density was also determined by isotopic measurements and it was found that all nine test fields were built up practically uniformly with the required quite different densities.

When filling had been completed in each field two or three soundings were performed. Because of the nearly constant density in each field, the curves obtained for the point resistance were very similar, so that for each density of each material one representative curve could be drawn (Fig. 51).

From a comparison of the point resistance at a definite depth and the relative density of the different fills, it could be concluded that in all three non-uniform materials investigated the point resistance increases very strongly with the relative density. This has been known to be true of uniform



$U_1 = 3.7$ $C_1 = 0.9$
$U_2 = 16$ $C_2 = 1.1$
$U_3 = 9.2$ $C_3 = 2.4$

FIG. 49. Typical grain size curves for materials tested.

sands for a long time. But it was found (Fig. 52) that for the condition of *equal* relative density the point resistance in uniform sands is higher than in non-uniform sands, which is somewhat surprising, as at first one would be inclined to suppose that the point resistance of non-uniform materials would be higher because of their higher dry densities. But that proves to be true only in the case of extremely high relative densities ($D_r > 0.75$) which are not found in uniform sediments and also very seldom in uniform sands, compacted in the usual way, as it is nearly impossible to bring them to a higher density than about 0.7. On the other hand it is easy to compact non-uniform materials to a higher relative density than 0.7, and you can find such materials even *in situ* in this dense state. Therefore the point resistance of soundings can be higher in non-uniform materials, but the reason for this is their higher relative density. If the relative density is the same in both cases, the point resistance in the uniform sand is higher.

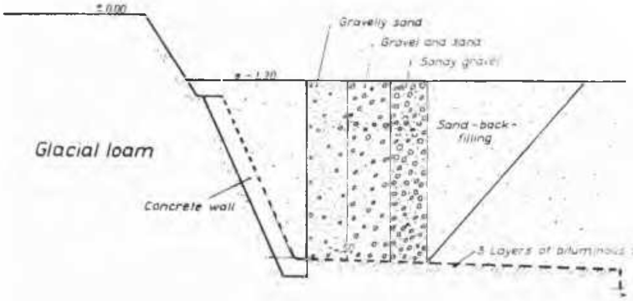


FIG. 50. Large-scale test pit.

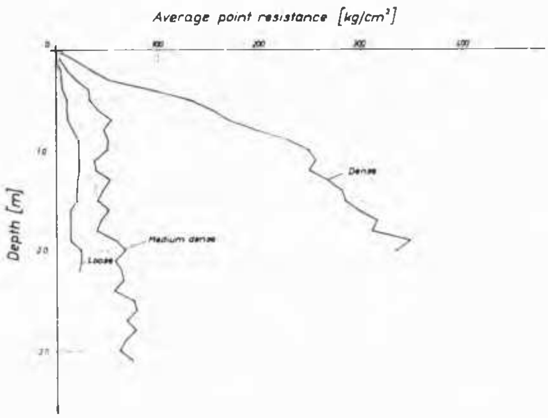


FIG. 51. Typical results of soundings on prepared materials.

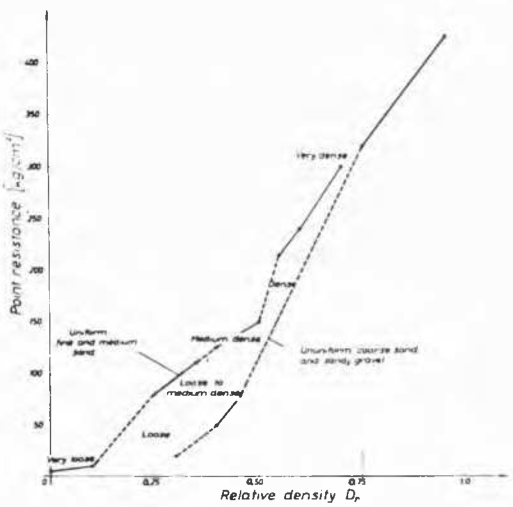


FIG. 52. Comparison of results of soundings for uniform and non-uniform materials at varied relative density.

Now we must look at the reason for this result. We think that it can be explained with the term "compactibility," which was defined by Terzaghi in 1925, in his book *Erdbaumechanik*, as the relation $D_f = (e_0 - e_d)/e_d$. The larger the difference $e_0 - e_d$ the larger D_f is, and the smaller e_d the larger again is D_f. The difference $e_0 - e_d$ is large and e_d is small in non-uniform soils. Thus D_f is large in non-uniform materials, which can be compacted easily and well. In contrast to uniform sands non-uniform materials can be brought to a high degree of relative density without too much energy. Therefore the non-uniform material possesses a high D_f and is known as a material which compacts well.

We should not be surprised that we find that such materials have a relative low point resistance when they are not in a state near the highest density. Sounding is an act of displacement and I think it is easy to understand that a rod pressed into a non-uniform material of low or medium density will find a smaller resistance than in a uniform sand, since the rod will very easily move the finer grains in the non-uniform material into the spaces between the bigger grains, a task which is much more difficult in a uniform sand. Therefore, what we found by our comparison tests is understandable: that the point resistance under a condition of equal relative density is lower in the non-uniform, that is the heavier, material. Therefore we can see again—what we know from earlier investigations—that with sounding methods you can only infer the relative density, and not find the dry density or the void ratio.

As the natural non-cohesive sediments are not to be found with the loosest void ratio e_0 but with the natural void ratio e_n, we can say that the point resistance of sounding depends on the relation

$$D'_f = (e_n - e_d)/e_d. \qquad (1)$$

The larger this relation, the smaller the point resistance or, vice versa, the smaller this relation, the higher is the point resistance. For instance, when e_n equals e_d then D'_f equals 0 and the point resistance will be very high. The plot of the representative point resistances found in the three investigated non-uniform materials and that known for uniform Berlin sand from earlier investigations (Fig. 53) shows the basic dependence of the point resistance on the ratio D'_f. I think that we can say that this ratio does determine the amount of the point resistance in non-cohesive soils.

In connection with the investigations for deep foundations these results can be of importance. For if relatively low values for the point resistance in non-uniform, non-cohesive sediments are obtained you must not judge them according to the criteria for a uniform sand which are generally used and which state that a value of 150 kg/sq.cm. means a dense layer with a relative density of more than 0.5. Using the usual gradation for the relation between the point resistance and the relative density for uniform sands, you can add an amount up to 80 kg/sq.cm. to classify the relative density of non-uniform materials. In so doing you will arrive at a more favourable judgment on the density condition. Indeed, we

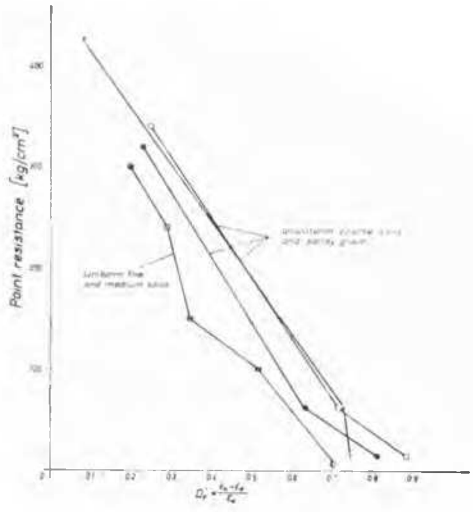


FIG. 53. Comparison of results of soundings for uniform and non-uniform materials for varied "compactibility."

know only a little about the bearing properties of non-uniform materials with a low or medium relative density in comparison with our knowledge of those of uniform sand. We will therefore begin in this year with large-scale loading tests in non-uniform materials.

Y. NISHIDA (Japan)

Begemann (4/1) comments on the pulling of piles. According to my study I think that a pile with a flat base will cause less compaction of the soil around the shaft than a pile with a sharp point. I would ask whether or not he has checked the effect of pile driving on the compaction of soil. If the soil is in the same condition before pulling there may be no difference in the maximum force between the two types of pile point.

This discussion may not be valid for *Berezantzev*'s report *(4/2)* unless Russian references are consulted, but I think that the stresses in the soil (upper AB level) are different from those in the natural state because the bottom force applies. Therefore the value of γD may not be used for the vertical pressure. He presents a very useful paper practically, but it is related to foundations on the ground surface where a uniform load is distributed around it as many people did until today. I think it is different from the deep foundation if considered strictly from the mechanics point of view.

Cambefort (4/3) says that the skin friction varies linearly. I do not agree with his comment unless the skin friction is measured practically. It may be true after the failure of a foundation, but I think that it is not so before the failure of soil, as the author's own Fig. 9 explains.

Davisson and Robinson (4/4) show their experiments on the effect of pile space on the value of the coefficient of horizontal subgrade reaction, k, which seems to be a very useful contribution in this field of study. I think that the coefficient, k, must be dependent not only on the depth but also on the width of the pile. The lateral loading test on a pile to determine the value of k may be based on Hetenyi's equation (1), in which k is already assumed to be a constant, as almost all studies in this problem do. I do not think it is reasonable.

Gibbs and Merriman (4/5) comment on the group effects. I think that the group effect has less significance when the piles are based in a firm layer except in the study of negative friction. I would like to ask to what points of group effect they paid attention.

An excellent report on negative friction is presented by *Johannessen and Bjerrum (4/8)*. I agree with their view that drag adhesion may be proportional to the effective vertical stress but only if it is comparatively near the ground surface; surely much less drag adhesion must be expected at depths near the hard stratum because of the condition of soil deformation. It must be noted that the horizontal stress is not proportional to the vertical stress if the shear stress develops, as easily seen from the condition of stress equilibrium.

Very interesting results, particularly in the case of a dense sand, are presented by *Kishida and Meyerhof (4/10)*. They assume the range of 3 to 5 times the pile diameter for the increase of internal friction angle around a pile. I would point out that the range should not be simply assumed but depends on other many factors. I have presented some studies on this problem (Nishida, 1961a, 1961b). Eq (1) may be assumed, but I think the superposition of Eq (1) is doubtful in the case of small pile spacings. I also have some doubt about the cap failure zone in Fig. 2b. According to a study of stress equilibrium I would say that the similar relation between the bearing capacity and the pile space shown in Fig. 5 can be quantitatively obtained, even if no change of the friction angle of sands is expected in the smaller pile spaces. The authors' experiments agree closely with my model tests for 5 piles. I hope to read their theory of prestressing and change of principal stress in sands related to group piles, which unfortunately is not presented here.

Professors *Lambe and Horn (4/12)* present an interesting application of the stress path method. I would like to know how to observe the lateral stresses by driving piles and the process of calculation. Unless the methods and the process are explained, I cannot comment on the comparison between their calculated and measured values.

Lo and Stermac (4/13) present useful information in their paper. They conclude that the induced pore pressures are equal in the failure zone of soil surrounding the pile, but I do not think this conclusion is reasonable from the point of view of mechanics since the applied stresses are not equal at the distances from the pile. The equal pore pressure seems more likely to be due to the water escaping between the pile and the soil or to the effects of skin friction. In the failure zone the authors find no direct summation of pore pressure due to adjacent piles, but I think this value depends on the mechanical properties of soil and on the pile spacing. Their comments on group effects may be accepted for the clay having the higher value of E/C_u (where E is Young's modulus of clay, and C_u the undrained shear strength of clay) or for the case of wide pile spacing. I would show my theoretical calculation of the pore pressure in group piles in the following way. In the failure zone: $a \leqq r \leqq R$

$$\Delta u = C_u\left[2\frac{R^2}{b^2} + \frac{4}{3}\log_e\frac{R}{r} + \left(A - \frac{1}{3}\right)\sqrt{3 + \left(3\frac{R^2}{b^2} + 2\log_e\frac{R}{r}\right)^2}\,\right], \quad (1)$$

In the outer zone: $b \geqq r \geqq R$

$$\Delta u = C_u\left[2\frac{R^2}{b^2} + \left(A - \frac{1}{3}\right)\sqrt{3\left(\frac{R}{r}\right)^2 + 9\left(\frac{R^2}{b^2}\right)^2}\,\right], \quad (2)$$

where Δu is the pore pressure induced in clay between piles; a is the pile radius; r is the horizonal distance from the pile axis; $2b$ is the pile space; A is the pore water coefficient defined by Skempton; and R is the radius of the failure zone around a pile in groups which is found by the following relationship:

$$\pi\left(\frac{1}{3}\log_e\frac{R}{a} - \frac{1}{2} + \frac{1}{2}\frac{R^2}{b^2}\right) + 3\left(1 - \frac{R^2}{b^2}\right)\frac{C_u}{E} - 1 = 0. \quad (3)$$

If the clay is in failure in the whole ground between piles, it follows that

$$\Delta u = C_u\left[2 + \frac{4}{3}\log_e\frac{b}{r} + \left(A - \frac{1}{3}\right)\sqrt{3 + \left(3 + 2\log_e\frac{b}{r}\right)^2}\right]. \quad (4)$$

It may be interesting to note that the larger the pile space and the higher the value of E/C_u, the larger the zone of failure becomes although it seems not to exceed about 10 times the pile diameter.

Locher (4/14) presents useful information on negative friction. I would like to ask about the influence of the pile space on the negative friction, since the vertical stress is not only a function of the depth but also of the distance from the pile. The state of stress around the pile depends greatly on the installation of piles, different from its natural state. On this point I would ask the author's opinion.

In paper *4/18* an interesting discussion of the application of Mindlin's equation is presented by *Salas et Belzunce.* I would like to point out that the stress around a pile by using this equation in a similar manner has already been reported by Brazilian engineers (*Proceedings of the Rotterdam Conference*). I would ask the authors how they estimate the influence of Young's modulus of the pile in the stress distribution in the soil. On the stress on the pile-soil interface reference can be made to the following contributions: Nishida, 1963; Nishida, 1964.

In paper *4/19 Shashkov* presents some practical data to verify his conclusions, which are I think easily deduced from the simple principles of soil mechanics. However, I think that pile diameter is also a factor of determining bearing capacity, because it has some effects on the soil properties around the pile.

In paper *4/21 Thurman and D'Appolonia* present an interesting computer study of piles. Since their theory is not shown in detail I find it difficult to comment. I think Eq (4) should not be applied for a pile under a smaller load, because the skin friction does not develop in the whole length of the pile. I would say that some similar analytical studies were done in Japan. In Eq (5) σ_f is not easily available from Mindlin's and Westergaard's works, in order to suit the boundary conditions of pile and soil. I think that in the initial stage of loading the ratio of settlement to load is higher for a shorter pile than for a longer pile from the mechanics point of view on the semi-infinite solid. In this respect the authors' calculation in Fig. 6 shows no distinct results.

Węgrzyn (4/24) presents the extremely interesting discovery that $y_p/\tan\phi$ is a constant. I would ask whether or not it is applicable for every kind of soil and for every type of pile. Also, how does one evaluate c in the calculation. The theory depends to a great extent, on c yet c has to be found by a calculation basing on the theory of a beam on the elastic foundation using the same c.

In paper *(4/25) Whitaker and Cooke* present an excellent way to separate the load factor into two parts, the shaft resistance and the base resistance. There are very few studies on the load factor in the field of soil mechanics. I think the relationship between the pile shaft resistance and the pile base resistance should be studied because they are not independent of each other. Unless the curve corresponding to Fig. 5 is arranged by measuring the shaft resistance and the base resistance separately for each pile at each site, it is not as easy to apply their idea to practical design as they say. The shaft resistance has little significance in piles with enlarged bases if they are rigid compared with the soil.

I would like to ask Messrs. *Williams and Colman (4/26)* why they take the value of the power of 3/2 in their Eq (1), while they assume the linear relationship of settlement-adhesion for the pile shaft. To add some information to their empirical analysis I present my calculations based on the theory of elasticity. The immediate settlement (ρ) at the centre of a circular base subject to a uniform load (p_b), when the base is at depth t and has the radius R_b in the ground where Young's modulus is E_b and Poisson's ratio is ν, is

$$\rho = \frac{(1+\nu)P_b}{(1-\nu)4E_b}\left[(3-4\nu)R_b - 4(1-2\nu)^2 t + (5 - 12\nu + 8\nu^2)\sqrt{4t^2 + R_b^2} - \frac{2(5-8\nu)t^2}{\sqrt{4t^2+R_b^2}} - \frac{8t^4}{(4t^2+R_b^2)^{3/2}}\right], \quad (5)$$

and the immediate settlement of the same base at its edge is

$$\rho = \frac{(1+\nu)P_b}{(1-\nu)4\pi E_b}\left[(6-8\nu)R_b - 2(1-2\nu)^2\pi t - \frac{(5-8\nu)t^2}{\sqrt{t^2+R_b^2}}F_1\left(\frac{R_b}{\sqrt{t^2+R_b^2}}, \frac{\pi}{2}\right) + \left\{(10 - 24\nu + 16\nu^2)\sqrt{t^2+R_b^2} - \frac{t^2}{\sqrt{t^2+R_b^2}}\right\}E_2\left(\frac{R_b}{\sqrt{t^2+R_b^2}}, \frac{\pi}{2}\right)\right] \quad (6)$$

where $F_1[R_b/\sqrt{(t^2+R_b^2)}, \pi/2]$ and $E_2[R_b/\sqrt{(t^2+R_b^2)}, \pi/2]$ are the complete elliptic functions of the first kind and of the second kind respectively. These equations become identical to Boussinesq's expressions when the depth, t, is zero. I don't think that either the bearing capacities or the settlements of the pile base and of the pile shaft are independent of each other.

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G. B. O'RORKE (Hong Kong)

Johannessen and Bjerrum (4/8) predict that the settlements of foundations on piles driven through soft compressible clay layers may be caused by negative skin friction and compression of the clay by surcharge.

Such cases occurred during 1963 and 1964 in Hong Kong where the effective stress in soft clay was increased by a lowering of the water table so that consequent compression of the clay with negative skin friction caused long piles in building foundations to settle further into the underlying residual soil on which they were founded. A report on these building settlements has been made by Lumb (1965), but a brief description here of such recent cases, which confirm the predictions of Johannessen and Bjerrum on quite a large scale, may be of interest as falling within the first category of problems proposed for discussion by the General Reporter.

Within a densely populated area of about 90 acres, many buildings were observed to be damaged in such a way as to indicate settlements of the foundations. The district had been largely developed over the preceding 30 years and most of the older buildings were 3- to 5-storey terrace houses on shallow foundations, but during the last 9 years the type of construction had changed to framed buildings of up to 20 storey on long piles of various types.

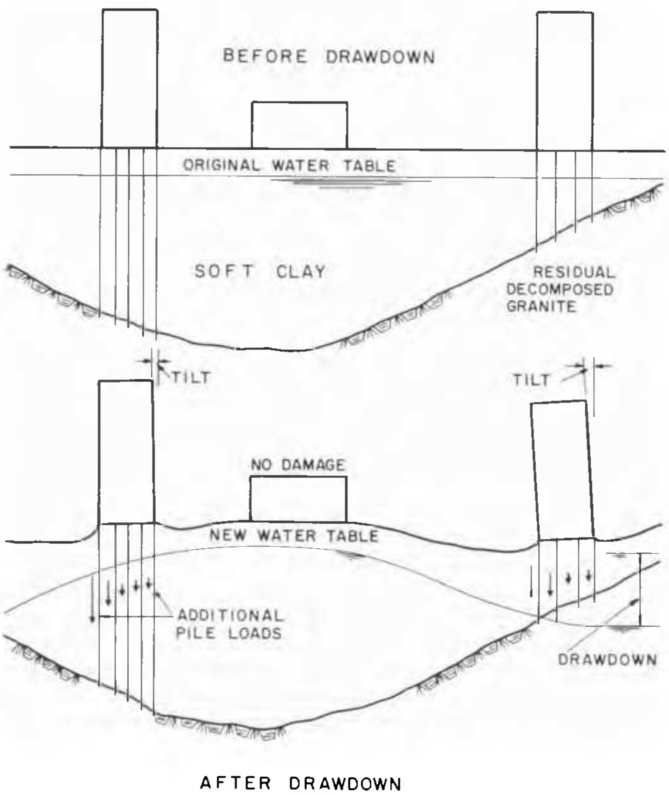

FIG. 54. Manner of settlements due to drawdown and negative skin friction on piles (diagrammatic).

Over most of the district there is a 10- to 20-ft thick layer of fill overlying marine deposits, most of which are very soft and soft clay, and finally there is residual decomposed granite overlying bedrock. The marine clay is of low strength (c_u = 200 to 600 lb/sq.ft.), high compressibility, and low permeability; it generally lies below sea level and varies in thickness from 0 to 35 ft. The residual decomposed granite is essentially dense, coarse sand with up to 50 per cent silt and clay and is of high strength, low compressibility, and fairly high permeability; the thickness of the layer varies from 50 to 120 ft in this district.

Almost all buildings in the area have wells for flushing-water supplies and many of the newer bigger buildings had been provided with deeper wells so that in 1963 and 1964 a much greater quantity of water had been removed from the ground than previously. As well, 1963 was an exceptionally dry year so that the quantity of recharge water entering the area was drastically reduced and the water table was progressively lowered. By mid-1964 the drawdown was of the order of 15 to 20 ft, corresponding to an increase in effective pressure on the soil of about ½ ton/sq.ft.

Investigations in mid-1964 showed that out of some 60 buildings showing damage due to settlement, 26 were founded on long piles passing through the clay into decomposed granite and in 7 such cases the settlement was attributed to drawdown of the water table acting in the manner already described. (The other causes to which settlement was attributed were consolidation under self-load and influence of adjacent constructions in which negative skin friction would not play an important part.)

Theoretical calculations indicated that full consolidation of the clay layer by such a drawdown would take up to 20 years depending on thickness and this was borne out by field observations which showed scarcely any reduction in pore water pressure in the clay where the layer was thick, but considerable reductions at the edges of the deposits where the thicknesses were much less. The directions of tilt of the damaged buildings were also consistent with this concept as shown on Fig. 54.

The measured angles of tilt of the buildings on long piles damaged by settlement due to drawdown were found to be roughly proportional to clay thickness, but inversely proportional to pile penetration into decomposed granite. In fact, a statistical analysis of buildings on long piles which showed damage due to settlement from any cause together with a similar number of such buildings that showed no damage in the same area indicated an extremely high statistical probability that buildings would be safe or unsafe from damage according to whether pile penetration into decomposed granite was greater or less than the thickness of the overlying clay. It was therefore recommended that this be used as a rough working rule for new building foundations in this district and others in Hong Kong with similar soils. The soil conditions are in fact quite typical of much of the built-up area, as a lot of it is on reclaimed land, and medium- to coarse-grained granite bedrock with associated overlying residual soil is widespread.

Although the numerical data in the report by Lumb (1965) do not include an assessment of adhesion between the clay and the piles, the relating of pile penetration to elimination of building damage may be considered to render this to some extent unnecessary. However, further examination of the records in the report, or on which it was based, may enable values to be deduced.

Finally it should be recorded that the damage suffered by the buildings on long piles resting in decomposed granite could be described as architectural only as affecting appearance and not with respect to safety, severe damage being restricted to older buildings on shallow foundations. It is felt that with proper consideration of soil conditions, for which more reliable methods of assessing negative skin friction would be most useful, even this type of superficial damage could be eliminated on future buildings.

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A. G. THURMAN (U.S.A.)

Professor Kézdi's statement in his General Report that Thurman and D'Appolonia's computational method *(4/21)* neglects the compression of the pile is in error. This is indicated by step 3, p. 324, of the paper. Kézdi's misunderstanding, plus other questions about the paper which were raised in private discussions at the Conference, indicate that the computational method is not adequately explained in these *Proceedings*. Because of the space limitations and because the paper was intended to show the results of typical computations made to this date, it was impossible to explain the details of the computational method. Consequently, the description of steps 1 through 14 (p. 324) only indicates the general procedure which was used.

Readers interested in the procedure are referred to a paper presented at the Conference on Deep Foundations in Mexico City (Thurman and D'Appolonia, 1964). There, the computational method is not only discussed in detail but comparisons are made between computed pile action and field measured pile action of both friction and end-bearing piles. It is my understanding that the *Proceedings* of that Conference will be available shortly.

It is important to note that the computational method does not depend upon particular failure stress equations such as Eqs 4 and 5 or upon particular tip-punching equations. That is, whenever they become known, more correct equations may be incorporated into the computational method without changing its basic concepts. The essence of the method is only dependent upon the concept of elasticity such as expressed by Eq 1. Although this concept is subject to question, the data presented in the Mexico paper indicate that predictions of pile action can be made within a valuable order of magnitude by using elasticity concepts.

At this time, the least known variable of this or any other method of pile analysis which is based upon static equilibrium is the factor K of Eq 5. K defines the relationship between vertical stress in the soil adjacent to the pile and horizontal stress at the pile-soil interface. It has no relationship to the coefficients of passive or active earth pressure. It may vary between wide limits and it will be affected by the coefficients of lateral earth pressure at rest, the type of soil, the type of pile, and the method of pile placement. Hopefully, this property will become better known as the sciences of soil mechanics and soil-structure interaction develop.

The importance of the variable K is indicated in Figs. 3, 4, 7, and 8 where computed pile action with $K = 0.5$ is compared with pile action with $K = 2.0$. These values seem to be realistic limits of K. Consequently, despite a limited concept of K, it is apparent that pile butt movement can be predicted for a particular load with valuable accuracy.

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A. S. VESIĆ (U.S.A.)

I should like to discuss the first two subjects proposed by the General Reporter, with special reference to piles in sand.

Until a few years ago surprisingly little was actually known concerning stresses and deformation around piles in sand. On the basis of observations made in our large-scale model investigations at the Georgia Institute of Technology (Vesić, 1963) it was concluded that, as soon as a pile has penetrated over about five diameters into a sand layer, the stress condition in a cylindrical zone of surrounding soil above the pile tip resembled that existing in a silo (Fig. 55). Thus, the existence of a zone of stretching or stress relief in the vertical direction around the pile shaft was postulated. In this way it became possible to explain the quasi-constancy of point and skin resistances observed both at the Chevreuse Station and in our own experiments.

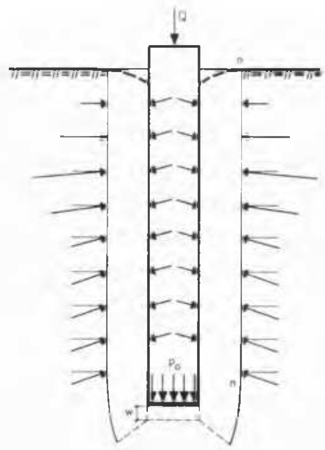

FIG. 55. Stress conditions around a deep foundation in sand.

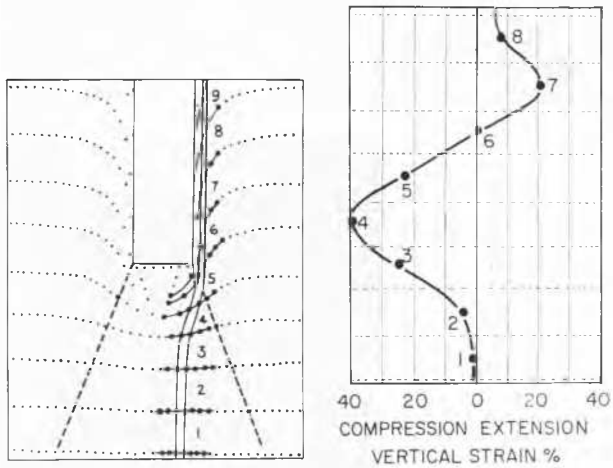

FIG. 56. *Left*, deformation pattern around a pile in sand (after Robinsky and Morrison, 1964); *right*, vertical strains for a column of soil originally in the direction of the pile axis.

Subsequently published data on measured vertical stresses (Kérisel and Adam, 1962) and measured deformation patterns in the sand mass (Robinsky and Morrison, 1964), (from which the enclosed Fig. 56 was prepared) present strong support for our arguments. There should be no doubt left that the stress conditions around piles and deep foundations in sand might be significantly different from those that existed in the ground prior to their placement. One exception, perhaps, is the case of a pile descending through very compressible strata to a bearing stratum of dense sand. Such a pile would most frequently penetrate only a few diameters into the sand stratum so that the effective vertical stress in vicinity of such a pile's tip may be close to the stress which existed at the same elevation prior to pile penetration. Additional carefully planned experiments, that should include both stress and strain measurements in a greater variety of soil conditions, are needed on this important subject.

Concerning the interaction between pile groups and soil with reference to pile caps, we have recently completed a series of large-scale experiments with groups of 4 and 9 piles in a symmetrical square array, driven at distances from two to six pile diameters into artificial deposits of dry sand (Figs. 57 and 58). The experiments were performed under my direction at the Georgia Institute of Technology, as a part of an extensive research programme financed by the U.S. Bureau of Public Roads and the State Highway Department of Georgia. All piles tested have been made of aluminum tubes, 4 in. (10 cm) in diameter and 60 in. (150 cm) long. Strain gauges were mounted on the inside of the tubes to allow separate recording of individual pile point and skin loads. Two soil situations were generally reproduced in different series of tests: a homogeneous, medium dense sand mass at a relative density of about 60 per cent and a two-layer mass, consisting of an upper stratum of very loose sand ($D_r \sim 20$ per cent) underlain by a stratum of dense sand ($D_r \sim 80$ per cent).

Test results indicate that the caps did contribute to the bearing capacity of the groups of 4 piles as much as if they were shallow footings. However, their contribution was considerably less in the case of groups of 9 piles, where, instead, the centre pile was able to support more load than the outer piles.

Interesting data were obtained on the efficiencies of piles in a group, data that point out the probable scale effect in the phenomena of pile group action in sand. It is known that several investigators working with very small, "nail-size" piles have reported efficiencies of 2 or even higher at close spacings of about two pile diameters. The highest efficiency that we have found in the case of a group of 4 piles in homogeneous, medium dense sand was about 1.5, including the effect of pile cap, and about 1.25 excluding that effect. In the case of a four-pile group with pile points penetrating about three pile diameters into dense sand over-

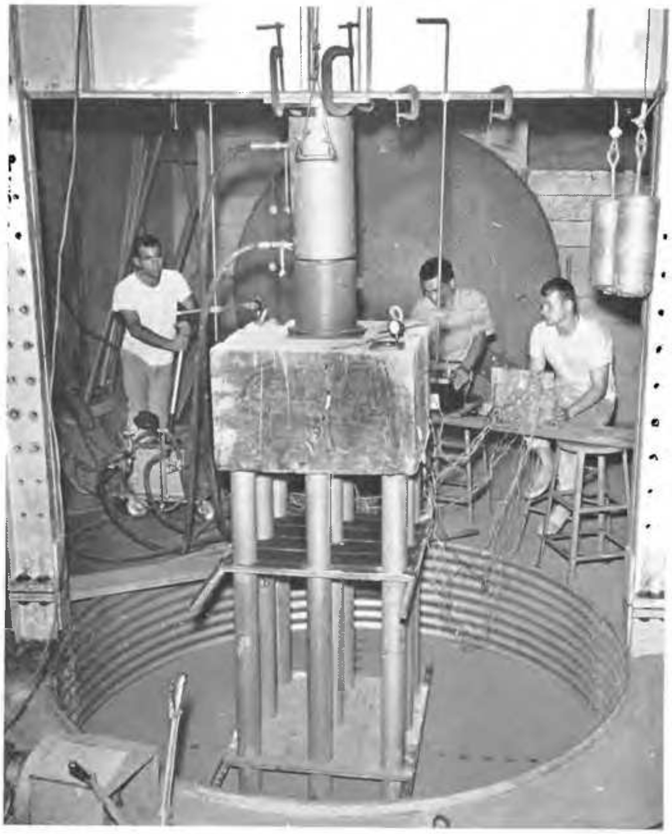

FIG. 57. A group of nine piles in the process of being forced into soil.

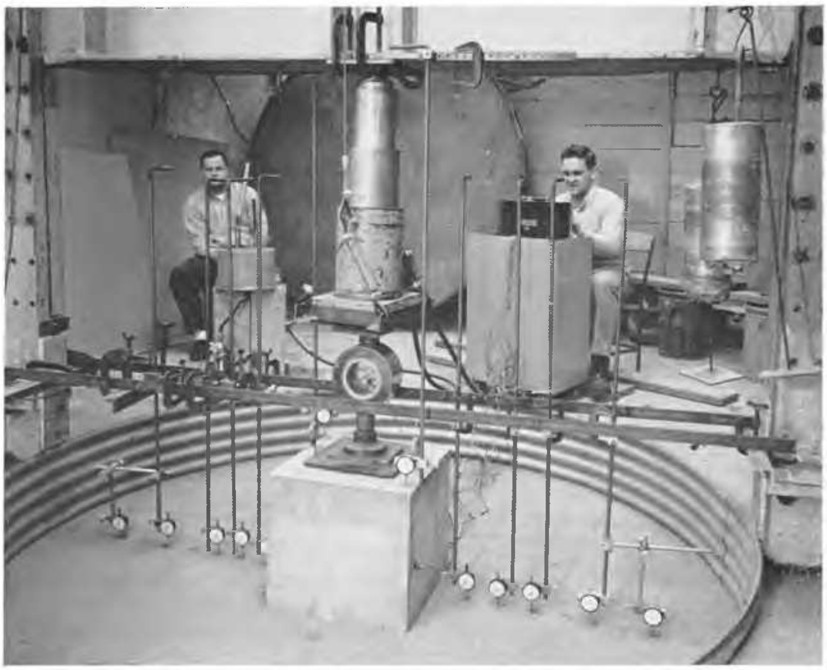

FIG. 58. Test on a group of four piles with concrete cap.

lain by a very loose sand stratum, the efficiency was found to be only about 1.1, most of the increase over 1 coming from the contribution of the pile cap and a slightly higher skin resistance. Similar or smaller efficiencies were found in the classical Press experiments.

It should be added that our observations do not support the idea that a pile group in sand can be analysed as a block foundation defined by the exterior perimeter of the group, no matter how small the pile spacing.

Obviously, investigation of the problem of the bearing capacity of pile groups is only beginning. In the past the relative length and spacing of piles, the number of piles, and the geometry of the group have been recognized as significant factors that may affect the group's efficiency. From our tests results and some other considerations we have concluded that, even in simplest cases of homogeneous soil conditions, soil compressibility must be a significant factor as well. In the case of piles in sand this implies a direct scale effect, (Vesić, 1964), in addition to other possible consequences. Thus, in the absence of reliable data on the behaviour of larger piles in a group, and in view of the trend shown by our investigations, it is not advisable to count, in design, on efficiencies higher than 1.

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B. P. WALKER (Canada)

Kishida and Meyerhof (4/10) have presented a theory to predict the ultimate bearing capacity of free-standing pile groups and piled foundations under both central and eccentric loading. The task that the authors have set themselves is indeed a formidable one as very little research has been carried out in this general field.

The theory is based on the assumption that an increase or decrease in density during pile driving is the controlling factor in the ultimate load of the group. Thus, where the effect of pile driving is to increase the sand density, the theory predicts an ultimate load on the group greater than that carried by the sum of the single piles. Research recently completed by the writer at the Building Research Station, England, in association with Imperial College, suggests that the behaviour of piled foundations under a central load in sand is too complex to be accounted for solely by changes in density during pile driving.

This research was carried out on foundations made up of 25 model piles, ¼ in. in diameter and 7¼ in. long. The piles were pushed individually into sand at an initial porosity of 42 per cent to form groups at spacings of from 2 to 8 pile diameters apart. A rigid cap was then cast directly on the surface of the sand and the foundations centrally loaded. The base and shaft load carried by each pile was measured by load cells contained within the piles.

The results of a test where the piles were spaced at four times the pile diameter will be described as they illustrate the complexity of the problem. Fig. 59 shows the load-

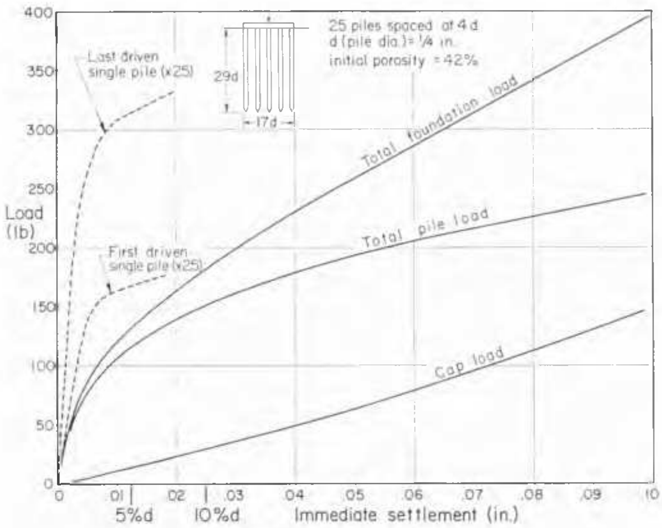

FIG. 59. Load-settlement curves.

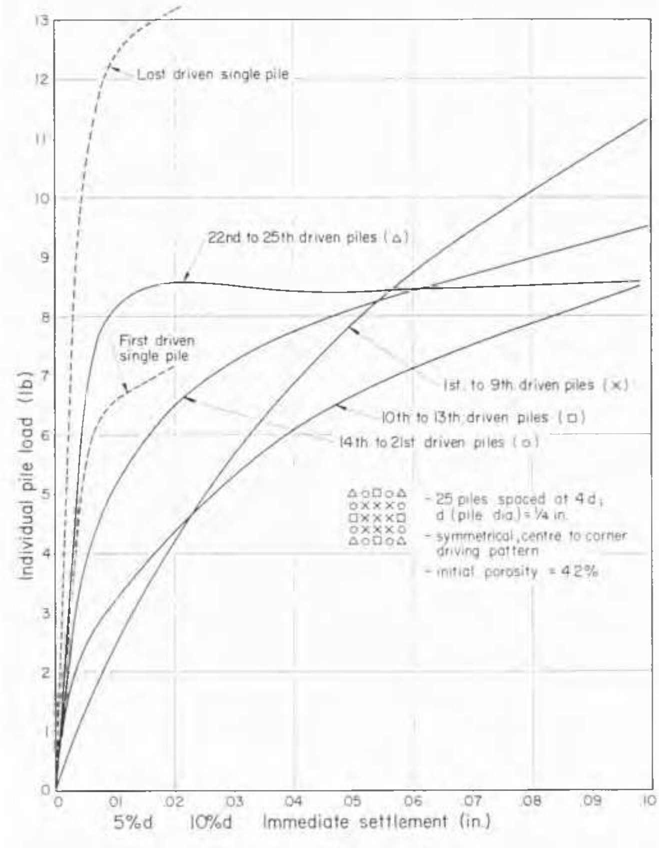

FIG. 60. Load distribution curves.

settlement behaviour of this foundation and the distribution of load to the piles and to the cap. It can be seen that at small settlements, most of the foundation load is carried by the piles while the load carried by the cap only becomes significant at large settlements.

To provide a comparison with the behaviour of single piles, the load-settlement curves of the first driven and last driven single piles have also been shown in Fig. 59. The load in both cases has been multiplied by 25 so that the curves may be compared directly with the previous ones.

It is clear that the last driven single pile carries more load at any settlement than the first driven pile or, that the ultimate bearing capacity increases as pile driving continues. Using the ultimate bearing capacity of a single pile as an indication of the change in density during pile driving, these results indicate a significant increase in density. If the authors' theory is correct, one would expect the capacity of the piles when part of a foundation to be greater than the capacity of the first driven pile multiplied by the number of piles—in this case 25. It can be seen that this is not the case. The total pile load is little greater at ultimate than the load carried by the first driven pile; in fact, they both tend toward the same failure envelope. It should also be noted that failure of the piles in the foundation takes place over a greater settlement than for the single piles.

It is useful now to examine the load distribution within the piles for the same test. This is shown in Fig. 60. The solid lines represent the behaviour of piles within the group while the dotted lines once again indicate single pile behaviour. It can be seen that there is a great difference in the load-settlement behaviour of piles within the foundation. The load carried by the last driven (corner) piles increases rapidly and the piles fail at relatively small settlements while the load carried by the first driven (central) piles increases more gradually and there is no very marked failure. Failure of the piles for this driving pattern occurs progressively and less catastrophically from the last driven to the first driven piles.

The early load-settlement behaviour of individual piles can be related to an initial small base load which each pile maintains after driving. This base load is greater beneath the last driven piles than beneath the first, due partly to the effect which the driving of subsequent piles has on piles which have already been driven. At larger settlements the greater loads carried by the central piles appear to be a result of more favourable positions within the group. Thus, in this research, the stress conditions beneath each pile were of major importance to the load-settlement behaviour of individual piles within the group. Although an increase in density occurred during pile driving, this factor was not of great importance to the load carried by individual piles and therefore to the behaviour of the whole group.

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