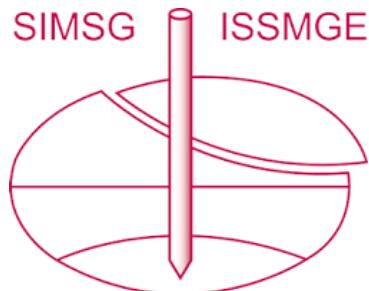


INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Fondations sur pieux

Piled Foundations

Sujets de discussion : Détermination de la force portante d'une fondation à partir des indications des pénétromètres.
Influence du groupement des pieux sur la force portante et le tassement.

Subjects for discussion : Determination of the bearing capacity of a foundation from penetrometer tests. Pile groups; bearing capacity and settlement.

Président / Chairman :

E. SCHULTZE, *Allemagne*.

Vice-Président / Vice-Chairman :

H. COURBOT, *France*.

Rapporteur Général / General Reporter :

L. ZEEVAERT, *Mexique*.

Membres du Groupe de discussion / Members of the Panel : E. GEUZE, *U.S.A.*; J. KERISEL, *France*; T. MOGAMI, *Japon*; N. NAJDANOVIC, *Yougoslavie*; R.B. PECK, *U.S.A.*; M. VARGAS, *Brésil*.

Discussion orale / Oral Discussion

V. Berezantsev, *U.R.S.S.*
L. Bjerrum, *Norvège*
A. J. L. Bolognesi, *Argentine*
A. Casagrande, *U.S.A.*
T. K. Chaplin, *Grande-Bretagne*
A. J. Da Costa Nunes, *Brésil*
E. de Beer, *Belgique*
C. Djanoeff, *Grande-Bretagne*
O. Eide, *Norvège*
E. C. W. Geuze, *U.S.A.*
W. G. Holtz, *U.S.A.*
J. Kérisel, *France*
G. G. Meyerhof, *Canada*
T. Mogami, *Japon*
N. Najdanovic, *Yougoslavie*

R. B. Peck, *U.S.A.*
H. Petermann, *Allemagne*
R. Pietkowski, *Pologne*
H. Simons, *Allemagne*
G. F. Sowers, *U.S.A.*
C. Szechy, *Hongrie*
C. Van der Veen, *Hollande*
M. Vargas, *Brésil*

Contributions écrites / Written Contributions

H. K. Begemann, *Hollande*
M. Buisson, *France*
W. G. K. Fleming, *Grande-Bretagne*
R. Haefeli, *Suisse*
T. H. Hanna, *Grande-Bretagne*
E. P. Khalizev, *U.R.S.S.*
B. Ladanyi, *Yougoslavie*
A. Mayer, *France*
G. Meardi, *Italie*
H. K. G. Muhs, *Allemagne*
A. Pellegrino, *Italie*
T. E. Phalen Jr., *U.S.A.*
H. M. Raedschelders, *Belgique*
S. C. Schiff, *France*
F. A. Sharman, *Grande-Bretagne*
H. U. Smoltczyk, *Allemagne*
G. F. Sowers, *U.S.A.*
U. W. Stoll, *U.S.A.*
Y. Tcheng, *France*
P. R. Tikunov, *U.R.S.S.*

M. KERISEL (*France*)

Messieurs, en tant que représentant de la nation invitante, j'ai le devoir et le plaisir de vous présenter les personnes qui sont assises à cette tribune :

Au centre, le Prof. Schultze (Allemagne) qui va présider ces débats; à sa gauche le Prof. Zeevaert (Mexique) qui est le rapporteur général de la Section 3B; ensuite le Prof. Mogami



L. ZEEVAERT

Rapporteur Général, Division 3B / General Reporter, Division 3B

(Japon); à l'extrême gauche M. Najdanovic (Yougoslavie); à ma gauche, le Prof. Peck (U.S.A.) et le Prof. Geuze (Hollande).

Viendront plus tard le Prof. Vargas du Brésil ainsi que le Vice-Président M. Courbot.

Le Président :

We begin the discussion of Section 3B, and to facilitate the discussion we have several points to discuss one after another. We begin with the first discussion you will find on the papers which are being distributed at the moment — that is, the determination of the bearing capacity of a foundation from a penetrometer test. First we speak about a static penetrometer, especially about the question : can the ultimate bearing capacity under the basis of a deep foundation be deduced from the cone resistance?

I now give the floor to the General Reporter, Prof. Zeevaert, to summarise the report he has given on the whole question.

Le Rapporteur Général :

The General Reporter wishes to express his appreciation and thanks to the Organising Committee for selecting him on this occasion to be the General Reporter on Section 3B, and to everybody in this room for their kind attention. In order to save as much time as possible, the General Reporter asks to be excused from reading his report as presented in the second volume of the Proceedings. However, he wishes to stress some of the most important points concerning pile foundations and, because of disagreement, necessary discussion according to conclusions obtained from 28 papers presented to this Conference, from which eight papers fall into investigation on model piles, 12 on natural size piles, 5 on theoretical investigations and 3 on special pile and pile foundations. Most tests and investigations are usually performed thinking always of the behaviour of a single pile. However, in practice we are really interested in groups of piles. It is important to remember, however, that a very large group of piles — or what the Reporter has called a pile field — shows completely different behaviour from a single pile. The stress distribution around the pile and into the soil mass is different in these two extreme cases. A group of piles of certain size will be, of course, under intermediate conditions.

Therefore it is important that investigations performed in one pile should be classified as such before extending conclusions to groups of piles in different subsoil conditions.

It appears that several authors are now in agreement on the so-called efficiency coefficient as determined in the laboratory in small model piles. However, can we apply this coefficient in practice, and under what conditions ? What is the significance of the settlement ratio in pile groups as determined in the laboratory ? Is it the same concept in the field ? I could foresee many variables, depending on stratigraphy at the site in question and compressibility of subsoil materials. Under which limitations and subsoil conditions apply these concepts in the prototype — that is to say, the settlement ratio and the efficiency coefficient ?

From the study of the papers presented to this Conference, and from previous literature, disagreement may be found in several aspects on the behaviour of piles and on the determination of the carrying load capacity. However, disagreement is sometimes fruitful, as investigators will refine or screen their findings and more light is usually thrown on to the subject.

The determination of the working load of a pile by means of the cone penetrometer and mostly in very loose or dense sediments appears to need revision and discussion. Furthermore, the Reporter could foresee subsoil materials where

compressibility is necessary to be taken into consideration and therefore the penetrometer results in some instances will not necessarily give the bearing capacity of a point bearing pile. Moreover, if this pile forms part of a group of piles, how could the penetrometers be applied to such conditions, and what are their limitations ?

The authors presenting results on friction pile tests reported reduction coefficients from 0.2 to 1.8. There is also disagreement on the values of the parameters to be used for point bearing capacity in piles. The Reporter believes these items require revision and discussion from the members of the board and members of this Conference well acquainted with these problems, and in general more evidence of well-conducted field tests is necessary.

Here it should also be mentioned that the pulling test has been used in several countries to obtain from the total load the point bearing load of a pile. In model tests this may be true. However, under field conditions the mass forces play a very important part, since when pulling a pile the negative friction phenomenon takes place. This may be recognised by the fact that the state of effective stresses in the mass is reduced, as in the case of positive friction, an increase of effective stresses takes place.

Concerning tests on piles I wish to recommend the classification in the technique of field pile testing in order to be able to reach in the future conclusions on the type or types of tests more convenient under certain well-defined subsoil conditions, and in conjunction with the work the piles are supposed to perform.

However, it is extremely important to define what should be considered as the definition of ultimate load in piles and the consideration of a critical load at which large and continuous settlements start to take place. Many times equivalent piles are compared at the so-called ultimate load. The Reporter strongly believes that in practice piles are not equally good if the ultimate loads are the same, but piles may be equally good if settlements are equivalent for a certain critical load.

Finally, if time permits, the Reporter wishes to invite, with the approval of the President of the Board, to present evidence and discussion on a problem the General Reporter has experienced. Because of heave of the soil during pile driving, negative friction sets in immediately after this process has been completed. Therefore piles assumed to work under positive friction and point bearing actually work under point bearing and negative friction conditions.

Le Président :

Thank you very much for your interesting summary of the reports on this question. I now call upon Mr Kérisel.

M. KERISEL

Messieurs, je vais donc ouvrir le débat sur la question *a*, que je puis résumer ainsi : Y a-t-il ou non homothétie dans les fondations profondes ?

Jusqu'à présent on suppose, implicitement ou explicitement, que le coefficient sans dimension que l'on appelle N_q et qui figure dans la célèbre équation de Terzaghi — coefficient N_q qui est le facteur multiplicateur de la pression q donnée par le poids des terres au même niveau que la fondation — est fonction du seul paramètre φ angle de frottement des terres. Par conséquent, sur un graphique où l'on porterait en abscisses les résistances de pointe du pénétomètre ou, plus généralement les pressions limites sous la fondation profonde, et en ordonnées les pressions q , tous les points que l'on obtiendrait dans des expériences faites sur un sable uniforme et semi indéfini, seraient situés sur une seule

et même ligne droite qui passerait par l'origine, et ceci, quel que soit le diamètre et quelle que soit la profondeur.

A la station expérimentale de l'IRABA, que quelques-uns d'entre vous ont visitée, nous avons désiré vérifier avec le plus de soin possible, ce point et le résultat de nos premières recherches est qu'il n'y a pas homothétie, tout au moins dans le sable dense que nous avons utilisé.

Vous pouvez voir, sur le premier cliché (Fig. 6, page 76, vol. II) que les courbes que nous avons obtenues, dont chacune se réfère à un diamètre donné, sont entièrement différentes et vous remarquerez que, pour un faible diamètre — diamètre de 45 mm qui est celui du pénétromètre usuel du commerce — la résistance de pointe du pénétromètre est pratiquement constante après qu'une profondeur de 1 mètre approximativement a été atteinte, tandis que pour un diamètre plus grand, la pression limite sous la fondation augmente d'une façon plus continue, cette pression pour une profondeur donnée étant plus petite que celle du pénétromètre de 45 mm, et la différence diminuant avec la profondeur et augmentant avec le diamètre.

Cette évolution est confirmée par des essais *in situ* que nous avons faits sur le lac Maracaïbo avec des piles de gros diamètres allant jusqu'à 1 350 mm, toujours, je le répète, dans des sables denses, et qui se réfèrent à des sables ayant approximativement une résistance de 300 kg/cm² au pénétromètre sous la pointe. Le Dr Simons vous montrera tout à l'heure ces essais de charge qui sont très exceptionnels puisqu'ils ont atteint plus de 2 000 tonnes sur une pile de 1 350 mm.

L'ensemble de ces expériences, en même temps que d'autres expériences qui ont été relatées par divers auteurs dans des sables denses, vous donnent le panorama de la figure 7, page 78, volume II.

Vous pouvez voir que l'un de ces essais de charge, le plus important, celui qui a dépassé 2 000 tonnes, se réfère à un paramètre q , de 30 tonnes par m², ou encore, puisque γ' , la densité déjaugée était approximativement 1, cela veut dire que la pile avait une fiche de 30 mètres dans le sable dense.

La question qui se pose est de savoir si ces résultats sont ou non en opposition avec les résultats donnés par les écoles hollandaise, belge et yougoslave, et je m'excuse d'en oublier, et dont les expériences ont fait l'objet d'une synthèse de M. Menzenbach.

A cette question, je répondrai non, parce que je pense que plus le sable est dense et aussi plus le pieu est court, et plus grand est le manque d'homothétie, et ceci m'est apparu assez évident lorsque j'ai étudié les essais de charge dans des milieux moyennement serrés et en particulier lorsque j'ai regardé en détail les 80 essais de chargement auxquels se réfère la très belle étude de M. Menzenbach.

Dans cette étude, il a porté en abscisse le rapport de la résistance limite sous la pointe du pénétromètre à celle sous la fondation et en ordonnée la section droite de la fondation. Si on rejoint tous les points qu'il a portés, en essayant de tracer les courbes d'égale pression sur le cône, on peut voir que plus grande est cette pression et plus éloigné de 1 est le facteur d'homothétie. Egalement on peut voir que plus grande est la section du pieu, plus éloigné on est également de ce facteur unité d'homothétie.

Finalement, ceci nous conduit, pour les sables denses au cliché suivant : (Fig. 9, page 78, volume II).

Ceci est mon interprétation de ce qui se passe dans les sables denses. Vous voyez, sur cette figure, d'abord la courbe 45 mm qui est celle du pénétromètre commercial et à côté nous avons une première zone hachurée qui est limitée par des courbes 300 mm et 600 mm de diamètre, qui sont les dimensions extrêmes assez habituelles des pieux, et enfin, vous avez une deuxième zone hachurée comprise entre 1 000 mm, c'est-à-dire 1 mètre de diamètre, et 1 500 mm, 1,50 m de diamètre, et correspondant à ce que nous appelons en français les piles colonnes.

Vous voyez, par conséquent, que si nous prenons une pression q faible, il n'est pas question d'homothétie, c'est-à-dire qu'il n'est pas question d'une égalité entre la pression donnée par la pointe du pénétromètre et la pression limite sous la pointe du pieu ordinaire ou sous la base des piles colonnes : il y a là, au contraire, un facteur de similitude qui peut atteindre sept ou huit unités. Mais vous voyez que plus on descend, c'est-à-dire au fur et à mesure qu'augmente la charge des terres, ce facteur de similitude diminue, tout en restant largement supérieur à 1. Au contraire si nous prenons le cas des milieux moyennement serrés (Fig. 11, page 79, volume II) qui est le cas des milieux que l'on rencontre très souvent en Belgique et en Hollande, des milieux qui donnent 100 kg en majeure partie, voire 150 kg/cm² sous la pointe du pénétromètre, vous voyez que dès qu'on atteint q égal à 10 tonnes, 15 tonnes au m², ce qui veut dire par conséquent 10 ou 15 mètres dans un sable qui est soumis à la pression d'Archimède, c'est-à-dire noyé dans l'eau, il y a un facteur de similitude qui est peu éloigné de 1 si bien que pour ces milieux dès qu'on dépasse les profondeurs ci-dessus le pénétromètre donne des indications valables qui peuvent être prises avec un facteur de réduction très faible, en dehors bien entendu, du facteur de sécurité qu'il convient d'appliquer.

Ce qui est important également, c'est de noter que si toutes ces courbes s'inscrivent entre la courbe 45 mm et la courbe relative aux gros diamètres, il semble bien, vous le voyez, que le rayon de courbure augmente au fur et à mesure que le diamètre augmente et il semble bien également — et ceci est tout à fait logique — que pour les diamètres très grands la courbe se réduise à une droite passant par l'origine, et, dans les expériences que j'ai faites à Maracaïbo avec de très fortes valeurs de q , il semble bien que la pente de cette ligne droite par rapport à la verticale ne soit pas très éloigné du

coefficient de Prandtl $\text{tg}^2 \left(\frac{M}{4} + \frac{\varphi}{2} \right) e^{M \text{tg} \varphi}$ dans les fondations

superficielles, car il est bien évident qu'une fondation de diamètre infini, même pour une profondeur importante n'est pas très différente d'une fondation de surface.

Il y a une autre question qui peut se poser à l'ingénieur à propos de ces facteurs de similitude. Il m'a semblé, en analysant les expériences relatées par Menzenbach, que pour des milieux sableux très peu serrés, le facteur de similitude pourrait être inférieur à 1, c'est-à-dire, que la pression du cône est plus petite que la pression limite sous la base de la fondation. Je connais d'autres expériences qui tendent à le faire croire, mais il est évidemment très difficile de conclure, avec précision car, pour des milieux non serrés d'une part, les erreurs que l'on peut faire dans les mesures sont du même ordre que ce que l'on veut mesurer, et d'autre part dans les milieux non serrés il y a souvent une portion argileuse et par conséquent quelque chance de développement de pression interstitielle, ce qui introduit par conséquent un facteur de complication.

Maintenant, nous pourrions nous demander s'il y a une explication à ce manque de similitude et quelle peut-être être. Il est évidemment prématûré de conclure, d'abord parce qu'un seul milieu ne peut pas donner la clé du problème; d'autre part parce que, pour un milieu il faut plusieurs essais, et chaque essai — tout au moins dans la cuve de l'IRABA — dure plusieurs mois, trois mois pour remplir la cuve, avoir un milieu à peu près homogène, et trois mois pour faire les mesures de densité. Mais une chose me paraît certaine, c'est qu'il n'y a pas, autour du pieu, au moment de l'équilibre limite, un volume plastique continu adjacent à la surface latérale du pieu, il y a au contraire un chenal plastique ou même plusieurs chenaux, je pense, et à l'intérieur de ce chenal en milieu dense se développe le phénomène de « dilatancy » avec augmentation de la densité sur les bords de ce chenal. Par conséquent il y a augmentation de volume à l'intérieur,

diminution de volume sur les bords; la seule chose que l'on puisse dire avec certitude, c'est qu'il faut que le total de ces augmentations de volume et de ces diminutions de volume soit égal au volume du pieu qui est introduit. Mais ce qui est intéressant dans de tels équilibres, c'est la possibilité d'avoir un équilibre plus favorable que celui de Boussinesq et celui qui a été généralisé par Midlin, qui sous-entend que le milieu puisse supporter des tractions. Or, un milieu sableux ne le peut pas et par ailleurs on n'a jamais noté de fissure de traction dans un milieu sableux après l'enfoncement d'un pieu.

Par conséquent, je conclurai en disant que la théorie à faire pour les fondations profondes est certainement une théorie élasto-plastique et qu'elle se réfère beaucoup plus à des phénomènes de silos dans lequel le milieu exerce des pressions considérables sur les bords, ou encore si vous le voulez, à des phénomènes qu'on appelle dans le livre de Nadai sur la plasticité, des phénomènes « extrusion through dyes », c'est-à-dire des phénomènes d'extrusion à travers des filières.

Mais, je le redis encore, il est prématuré de vouloir conclure, d'abord parce que nous avons trop peu d'expériences et aussi parce que, d'une façon générale, il faut se défier des généralisations.

Le Président :

M. Kérisel, je vous remercie beaucoup pour votre contribution si complète. Nous tous qui avons visité la station d'IRABA, sommes vivement impressionnés par l'échelle et la précision des essais que vous y effectuez.

Maintenant je demande au Prof. Geuze de prendre la parole.

M. E. GEUZE (Etats-Unis)

I would, Mr Chairman, very much like the opportunity to make some comments on the very interesting statements of Prof. Kérisel. I regret that a little accident has prevented me to visit the IRABA station.

It is obvious in the first place that the comparison between the limit of penetration resistance of a small and a large diameter pile point presents a difficult problem.

With increasing diameter of the pile the soil body involved in the penetration process increases in dimensions. As a result of the change in properties with depth, the resistance to penetration depends on the displacement of the pile point. Test results therefore never show a well defined resistance limit. The only method by which any comparison can be made is to take the ratio between the displacement and the diameter as a dimensionless parameter. These displacement ratios should be compared with the average stress on the pile point. As shown by myself (1953), the resistance at equal displacement ratios then becomes equivalent, provided that the rates of displacements are equivalent.

As Prof. Kérisel has not shown the time-displacement curves obtained at constant loads, it is not possible to analyze his test results on this basis.

As a point of interest I would like to mention that the stress-displacement ratio curve becomes a unique characteristic for a pile point of a given shape when applied to various diameters, provided that the displacements are limited to the time-independent parts of the deformation of the soil around the pile point (Geuze, 1960, Fig. 1).

The relationship as shown is of interest because of three different aspects.

In the first place it shows that the law of similitude applies, as long as the stresses around the pile point do not reach the yield limit. Even though the stress-strain relationships then are essentially non-linear, the displacement of the pile point shows to be proportional to its diameter. We therefore must conclude, that at equal magnitudes of the applied

stresses at the pile point and at equal magnitudes of the stresses at the outer boundary of the affected area, the strains are equal at homothetic points.

In the second place it shows that limit of validity of similitude is somewhat higher for the smaller diameters. This is possibly due to the effect of the frictional stresses transferred by the end portion of the pile. It will be shown later that there are strong indications in favour of the hypothesis that the magnitude of these stresses does not depend on the diameter of the pile. They would therefore have a stronger and more favourable effect in the case of small diameters as compared with large diameter pile points.

I will return to the third aspect later on, but for the moment I would prefer Prof. Kérisel to answer my question.

M. KÉRISEL

Je pense que je répondrai à propos de la question d puis qu'il s'agit de l'interprétation de la charge limite sur la courbe charge-tassement.

M. GEUZE

I would like to know another thing from Prof. Kérisel, and that is how the point resistance was measured : that is to say, were these results obtained for the actual point of the piles only or was some part of the shaft involved in the transference of the forces to the pile ?

M. KÉRISEL

En ce qui concerne les pieux pénétromètres de l'IRABA, c'est extrêmement simple; nous disposons d'un terrain d'une course de 1 mètre maintenant qui permet d'enfoncer continûment pendant 1 mètre la pointe du pénétromètre, et d'autre part, pendant cet enfouissement, nous opérons en simultané, c'est-à-dire que nous mesurons en même temps la force totale sur la tête du pénétromètre ou du pieu de 216 mm. et la force sous la pointe de cet appareil et ceci avec deux appareils de mesure indépendants.

Les mesures de raccourcissement du tube au moyen des appareils électriques de capacité à enregistrement automatique permettent aux deux étages extrêmes une bonne vérification.

Ailleurs *in situ* sur chantier j'avais toujours un ou plusieurs vérins plats type Freyssinet sous la base de la fondation et des vérins en tête.

M. NAJDANOVIC (Yougoslavie)

En se rapportant à la question de la détermination de la force portante de pieux de fondation à base d'essais de pénétromètres, on peut dire, d'une manière générale, que la résistance de pointe du pénétromètre diminue lorsque le diamètre du pénétromètre augmente et, d'autre part, que cette résistance augmente avec la densité du milieu et, dans une certaine mesure, avec la profondeur.

Les essais de fondations profondes dont nous a parlé M. Kérisel tout à l'heure, représentent une contribution importante à cette question et apportent plus de clarté dans la résolution du problème de la force portante de pieux de fondation dans le milieu serré, c'est-à-dire en milieu sableux.

La conclusion qu'on peut tirer de ces constatations, c'est que la force portante du pieu de fondation est moindre que celle calculée à la base des essais de pénétromètre et que cette différence entre ces deux forces portantes croît avec la différence de diamètre.

Pour obtenir la force portante du pieu de fondation il faut donc réduire la force portante calculée à base d'essai de pénétromètre. On ne peut pas donner une règle absolue

à ce sujet car il y a plusieurs facteurs dont il faut tenir compte, mais d'après notre constatation il y a une tendance à augmenter le facteur de réduction, augmenter pour ne pas dépasser une certaine valeur de force portante estimée convenable dans le cas donné. D'autre part, il y a aussi une tendance à diminuer la force portante du pieu de fondation, du moment que le facteur de réduction tombe au-dessous d'une certaine valeur qui est le plus souvent admise être égale à 2.

Dans la discussion qui va suivre j'espère que d'autres intervenants vont nous donner leurs conclusions sur ce sujet, suivant leur expérience et les essais qu'ils ont effectués.

Le Président :

That is the end of question (a). Mr Vargas will speak not only about static penetrometers but also about dynamic penetrometers and pile foundations. I ask Mr Vargas to take the floor.

M. M. VARGAS (Brésil)

In my opinion penetration tests, static or dynamic, can give only an index of the soil properties, which can be related to bearing capacity only by means of statistical analysis. I think this is quite true in the case of shallow foundations, where a statistical relation between penetration resistance and results of plate bearing tests can be established for certain types of soil occurring in certain regions. Based on such correlation, the allowable pressure can be predicted from the penetration index, provided that both penetration resistance and bearing tests are standardised.

In the case of pile foundations, however, things are not so simple, because it is impossible to consider that the allowable load on a pile depends only on soil characteristics, as is generally the case for shallow foundations. It depends as much on the dimensions of the pile as on the characteristics of the soil. So, in my opinion, the only way to establish a correlation between penetration tests and the bearing capacity of a pile, is to consider the penetration test as a sort of model test of a pile driving. I mean, to consider the driving of a penetrometer into the soil as the driving of a model pile whose dimensions are a fraction of the dimensions of the actual pile. The procedure obviously requires an interpretation of the test results.

But any interpretation of a model test supposes a theory behind it, on the basis of which the interpretation is done. In my opinion that was, in short, what Mr Kérisel did when he tried to determine the bearing capacity of piled foundations based on penetrometer resistance observations. He had to suppose that in a cohesionless soil the point bearing capacity of a pile divided by the area of its point cross-section can be expressed by the multiplication of the earth pressure by a bearing capacity coefficient N_q . He did so to show that the ratio N_q , cannot be regarded as a function of a single soil property — its angle of friction — but depends also on the dimensions of the pile.

During the last two years I have had the opportunity, in an extensive piling for a large steel-mill foundation in São Paulo, Brazil, to try to correlate N_q with penetration resistance indexes, obtained by standard penetration tests (number of blows per foot). A large amount of loading tests was made on pre-stressed concrete and wood piles varying from 12 to 25 m long. But the trial turned out a complete failure, because it becomes quite evident that N_q did change more with the dimensions of the pile than with the soil properties. For instance, the computed average value of N_q from 17 tests on a group of 7 000 wood piles for the foundation of the above-mentioned steel works was 240. These piles were 11 to 14 m long, driven through a superficial soft organic clay layer, 10 m thick, to a 2 to 3 m thick

compact sand layer with a standard penetration resistance over 25 blows per foot. The average point diameter of these piles was 18 cm. For precast concrete piles, 18 to 20 m long, with 35 × 35 cm cross section, the value of N_q was 175. Approximately the same value of N_q was obtained by means of tests on pre-stressed cylindrical concrete piles 40 cm in diameter and 20 cm long.

So it is quite clear that, in the above-mentioned case, the value of N_q could not be predicted by the dynamic penetration test, and in my opinion also it could not be predicted by means of the static cone resistance. To make that prediction possible a theory is needed that would take into consideration all the above mentioned facts. As we actually do not dispose of such a theory — or at least the theories we now dispose of seem not to agree entirely with reality — I do not think we are in a position to try to establish any valid correlation between penetration resistance and allowable loads on piles.

M. KÉRISEL

Je voudrais simplement dire deux mots à la suite de l'intéressante intervention du Prof. Vargas.

Bien entendu, s'il s'agit d'établir une théorie exacte concernant N_q , nous pourrons attendre encore quelque temps. J'ai trouvé des N_q de l'ordre de 1 000, avec de fins pénétromètres mais pour de larges fondations, même dans du sable serré, je n'en ai jamais trouvé dépassant 50 ou 60. Je pense que la véritable approche du problème pour nous qui devons être pratiques, consiste à tracer des courbes du genre de celles que je vous ai projetées, chaque faisceau de courbes correspondant à un poinçonnement donné au pénétromètre en se bornant pour commencer à des poinçonnements de 300, 200 et 100 kg/cm². Dans chaque faisceau, on trouvera à une profondeur donnée la pression limite sous les diamètres de 30, 60, 100 et 150 cm. Je pense qu'avec tous les essais qui sont pratiqués dans le monde — et il y en a beaucoup — si on note soigneusement les conditions du sol, on peut parfaitement arriver à clarifier ce panorama que j'ai peut-être eu trop d'ambition de vous donner tout à l'heure, mais je crois que c'est là l'approche pratique du problème pour l'instant.

J'ajoute également que les conclusions que je donne sont relatives à un sable serré, par vibrations, et hier, M. Chaplin m'a fait remarquer que ce cas correspondait à un coefficient au repos plus élevé que dans les sables naturels. Nous recommencerais nos expériences avec un sable versé de très haut et avec faible intensité.

Le Président :

There are some other remarks concerning the first point ? If that is not the case we come to the second point — bearing capacity and settlement of pile groups. We have divided this into two points — efficiency coefficient and settlement ratio. Prof. Zeevaert will speak on the first point.

Le Rapporteur Général :

Concerning the problem of large groups of piles first I should say I have no experience whatsoever on model tests in small piles therefore it is difficult for me to discuss the efficiency coefficient, that is to say, the coefficient that should be used to multiply the ultimate load, of one pile, to obtain the ultimate load for a group — this is of course one thing that has been studied carefully in the laboratory and at least we know that for certain spacing, say about 3 or 4 diameters, this coefficient has a tendency to be about 1. In the field it might have some correlation — that I am not sure to substantiate as actually there are many cases when a large group of piles, or a pile field, behaves completely

differently from a small group of, say, 5 to 10 piles. This has been my experience, and of course my experience has been mostly in Mexico City where we have, as you know, very high deformations. Actually there, one can detect deviations from phenomena one cannot see in other places. From my experience I can say that in order to find out what load a pile should carry one has to introduce also considerations of stratigraphy, compressibility and the general geological possibilities of the site in question. So actually the term of reduction coefficient for ultimate bearing capacity from these tests is so far the result of investigations in the laboratory. The same could be said for the settlement ratio, as to how much will a group settle if one pile settles so-much, this problem is one of the questions that one has to work out under actual field conditions considering the stratigraphy and compressibility of the different layers encountered.

M. R. B. PECK (Etats-Unis)

It is highly important that for certain idealized conditions we have knowledge of the relative behavior of single piles and groups of piles, with regard both to bearing capacity and to settlement. This theoretical knowledge is justified primarily because it sharpens our judgment when we come to consider practical problems. As Prof. Zeevaert has pointed out, each time we approach a practical problem we should ask ourselves the question, "Is there in fact likely to be any definite relationship between the load capacity or the settlement of a single pile and that of a group of piles or that of a whole pile foundation?" The answer to this question is probably a qualified "No".

In connection with piles driven into sand, for example, the fact that the piles have to be driven introduces great complexity into the consideration of questions of both settlement and efficiency. As pile driving takes place, the density of the sand changes. The manner in which it changes and the final pattern of density surely depend on the number of piles and to some extent on the order of pile driving. This is particularly true if the length of the piles is determined as is customary on the basis of a penetration requirement. The effect of the change in density may be important. It may, indeed, override the effect of the factors considered to be the significant variables in the idealized or theoretical solutions for efficiency or for settlement ratios.

Similarly, in connection with pile groups or pile foundations in clays, the fact that the piles have to be inserted into the ground, and are usually inserted by driving, is likely to cause disturbance of the structure of the clay. This may be followed by reconsolidation so that the properties of the clay are different before and after pile driving and the final properties of the clay mass in which the piles are embedded are likely to depend on the number of piles, their dimensions, their spacing, and many other factors.

Therefore, in order fully to understand the implications of the efficiency ratio or the settlement ratio for a group of piles in clay, we need to know much more than we do now about the effects of the sensitivity of the clay to remoulding, of the compressibility of the clay, of its consolidation characteristics, and of its strength after consolidation to various degrees. These factors, at least at present, are hardly subject to a completely rational interpretation. Inasmuch as such matters as the order of pile driving are certain to differ from job to job and are not likely to be controlled by the engineer, there must always be an element of uncertainty in relating the results of a load test on a single pile to the behavior of a group of piles or to the behavior of a pile foundation. This element of uncertainty may be so important as to relegate to a minor position any other consideration of a more theoretical nature.

M. KÉRISEL

Je voudrais ici me déclarer tout à fait d'accord avec le Prof. Peck en ce qui concerne l'étude possible de l'interaction entre plusieurs pieux dans un sable.

Sur le cliché qui va vous être passé (Fig. 14, page 81, Vol. II) vous verrez la distribution des densités autour du pieu dans un milieu sableux serré.

Voilà le chenal plastique auquel je faisais allusion tout à l'heure. La densité initiale est de l'ordre de 1,75 — un peu plus de 1,75 — et vous voyez que dans le chenal marqué nous avons des 1,67, 1,66 et 1,65.

Supposons maintenant que nous ayons deux pieux grandeur nature, côté à côté, il est évident que les deux chenaux qui vont se produire vont interférer; en particulier, il est possible que la courbure du chenal ne reste plus la même et que ce chenal plastique aille chercher un appui résistant sur le pieu voisin. C'est pourquoi les essais en modèle réduit dans les milieux sableux denses paraissent un peu risqués, dans des cuves de dimensions trop réduites souvent, mais mieux vaut faire ces essais que de ne pas en faire, tandis que, au contraire, dans les milieux non serrés comme les argiles, je pense qu'ils peuvent donner déjà des indications intéressantes. Et, à ce sujet, dans les trois communications sur les groupes de pieux de M. Berezantsev, celle de Saffery et celle du Prof. Sowers, il y a l'indication déjà que l'inter-distance entre les pieux ne suffit pas pour caractériser le coefficient de rendement, ni non plus pour caractériser le facteur multiplicateur du tassement et qu'il faut aussi introduire le facteur profondeur comme dans l'étude du pieu isolé.

Le Président :

Merci bien, M. Kérisel. Any other remarks on this point? If that is not the case, we have out of the original scheme two other points which are written on the distributed papers, and the third point for this session is the bearing capacity of piles in clay. We have divided it into two questions : the first question — the coefficient N_c multiplying the cohesion "c" in the calculation of the ultimate pressure under the basis of a deep foundation. To this point Prof. Peck will make some comments.

M. R. B. PECK

If I may, I would like to combine my remarks on this point with the second point, which deals with...?

Le Président :

Coefficient multiplying the cohesion in the average lateral friction and the reduction coefficient of this.

M. R. B. PECK

In connection with piles of ordinary size embedded in relatively soft plastic saturated clays (corresponding to a $\phi = 0$ condition) the point resistance is usually a very small fraction of the total resistance. There has been some disagreement in the Conference papers as to whether the factor N_c should be as small as 5 or perhaps as large as 9 or 10, but the reason I chose to combine this discussion with one concerning side friction is simply that the value of N_c usually does not make very much difference. It is the value of the side friction that governs the bearing capacity of a single pile.

About two years ago, at the request of the Highway Research Board in the United States, I reviewed what load test data could be found in connection with the bearing cap-

acity of single piles in clay under the conditions of essentially no change in water content. I arrived at a conclusion that I thought was quite well substantiated by the evidence; namely, that the bearing capacity of such a pile could be determined quite accurately by multiplying the embedded area by the undrained shear strength, or roughly by half the unconfined compressive strength, of the material. This seemed to be in agreement with practically all the data that were available. It seemed not to depend particularly on the sensitivity of the clay.

This was, of course, an empirical conclusion, and like all empirical conclusions was likely to have its limitations. I was a little surprised, however, to discover that the next project with which I had any connection turned out to be a rather marked exception to this rule — somewhat to my embarrassment. The project involved a few piles, rather long ones, in North Dakota, in a lacustrine deposit with a depth of more than 150 ft. The capacity of the piles was estimated originally on the basis of the rule I have just described. The unconfined compressive strength of the clay, a *CH* material, was about 1·1 kg per sq cm. The clay was very slightly laminated but had in general a rather homogeneous structure and had a low sensitivity, of the order of 2 or 3.

Since there was some uncertainty about the applicability of empirical rules, it seemed desirable to have some load tests made. Fortunately this was done. Tests were carried out on steel pipes of cylindrical shape, on steel H sections, and on timber piles. It was found that the ultimate capacity of the steel piles was of the order of 35 per cent of the predicted value and that the capacity of the timber piles was somewhat better — on the order of 70 per cent of that predicted. It was also found that when load tests were made on the same piles at increasing intervals of time after driving, up to about three weeks, the capacity of all the piles decreased instead of increased with time.

At about the same time the records became available for a load test in Massachusetts carried out by the firm of Haley and Aldrich. A cylindrical steel pile of 14-in. diameter was driven through a sand deposit into a clay. The sand was kept from contact with the pile by a casing, so that the pile was supported strictly within the clay portion of the deposit. The clays were of somewhat similar characteristics to those at the North Dakota project, with unconfined compressive strengths of about 0·6 kg per sq cm. The ultimate capacity of the piles was found to be only 43 per cent of that which would be obtained by multiplying the embedded area by the undisturbed shear strength of the material.

One of the papers presented in this Conference suggests that the empirical rule that I have discussed is not conservative enough and that one should take some sort of an average of the remoulded and undisturbed strengths as the strength of the clay. At the North Dakota site, piles carried a load that was less than the product of the embedded area and the remoulded strength of the clay; therefore even such an average would hardly be conservative.

One then asks, what can be the explanation for behavior like this? The answer, of course, is that as yet we do not know all the factors. One thing is apparent, however : the timber piles developed more strength than the steel piles. This can be partly explained on two grounds : the clay is more likely to consolidate around a timber pile because there is opportunity for drainage into the pile; and the coefficient of friction or adhesion between clay and steel may be less than that between clay and timber or less than the shear strength of the soil itself.

Bror Fellenius in tests in Stockholm, in which he had pairs of piles of identical dimensions, one wood and one covered with sheet metal, found a reduction of about 20 per cent in the capacity of those piles that had a steel cover;

therefore, part of the discrepancy that I have mentioned may be a consequence of the materials involved.

But the facts are that we do not understand all the complicated phenomena that are associated even with a single test pile in a plastic saturated clay, and I am sure that much more field information of the type I have mentioned and much more detailed study of the phenomena themselves will be necessary before we can come to a proper understanding.

I might add that if we are in this rather unfortunate position in connection with single piles we are, in my opinion, in not too favorable a position in connection with groups of piles and pile foundations. This is not to give the impression, of course, that we should abandon all hope; it merely indicates that most of our generalities that are based either upon empirical studies or upon theory have exceptions that we learn about only from time to time in the field. And as Dr Terzaghi has exemplified so often in his career, it is these odd exceptions which we cannot at first understand that are the most promising guides for future research.

M. MOGAMI (Japon)

We performed some test piling in the City of Tokyo, made on several piles 7 m long, and about 30 cm in diameter. We observed the distribution of the axial stress by attaching the strain gauges on the inside walls of the piles. The distribution of the longitudinal stress was found to be linear with the depth, and it confirms that the shearing stress between the outside wall of the pile and the soil is uniformly distributed.

The soil was clayey and very soft, and as the result of our observations of axial stress in piles we could confirm that the distribution of shearing stress on the wall is uniformly distributed.

Then the coefficient to be multiplied into *C* in the calibration of the average lateral friction is unity, for our experiments. But the number of experiments we did was quite limited and the decision will be postponed until after further studies.

But anyway, the purpose of getting such a ratio is to obtain a convenient way of getting the ultimate bearing capacity of a pile in soft ground, because it is a simpler way of getting necessary data. However, I want to mention that the time effect is very important in this case. In the experiment we performed the time effect was quite important for getting bearing capacity and other necessary data. The bearing capacity estimated by the loading test on piles was at first about 50 per cent of that estimated after five weeks, and the bearing capacity as measured thus increases with time, initially quite rapidly and later not so rapidly.

The other data, for example, the earth pressure on the wall of a pile, increased quite rapidly in the first few days but it increased slowly later. This confirms the time effect on the determination of bearing capacity. Therefore the coefficient which is necessary to multiply by *C* in the calculation of lateral friction would change from time to time, because the bearing capacity itself changes. Therefore we have to mention that the time effect is very important in this respect.

Le Rapporteur Général :

According to my experience on the reduction coefficient, I actually have to confess that I have never used this coefficient as obtained from tests made at other sites. I see as quite natural the variations reported from 0·2 to 1·8, and we are not going to question the results of investigators that are giving us this valuable information, because they are carefully made. These are the results. But why do we obtain this inconsistency in the results? Well, I can think of several reasons. One is that the technique of making a pile test is not equal for all

the results. If we are to get useful results we have to be consistent and compare only those results of those tests that are made equal and under similar subsoil conditions.

Then the other point is that usually where a pile is driven in the ground, the stratigraphy or geological conditions of the site are rarely carefully studied, and all we do is say "Here we have clay, let's divide the load by the average values of the unconfined compression test" or vane test, or cone, or whatever it is — "and find a reduction coefficient that in most cases is recommended to be used at another site". All this, I say, is very dangerous. Therefore I guess that concerning these coefficients, again, I do not know if we are trying to simplify soil mechanics a little bit too much.

M. E. GEUZE

I would like to proceed with my discussion on the validity of model relationships at penetration of piles.

The lateral friction results from two major phenomena at penetration, i.e. the lateral displacement causing increased radial stress and peripheral shear. In an elastic solid the increase in magnitude of the radial stress by a radial displacement is independent of the pile diameter. In normally consolidated clays this may approximately be the case if the pile is driven at a relatively high rate. The lateral friction will then be considerably reduced by the remoulding effect of the peripheral shear. During the rest period after the driving the effective radial stress will build up with time as a result of pore pressure dissipation and the clay in the immediate vicinity will regain its strength by thixotropic action.

Assuming that these effects would be independent of the pile diameter, various diameters were compared in load tests and it was found that the friction obtained after a build-up period approximately satisfied this assumption.

The main reason may well be that the remoulding effect and the thixotropic regain of strength are both independent of the pile diameter.

The third point I wanted to make preceding Prof. Kérisel's interesting statements concerns the phenomenon of the residual stress.

Pile driving is a repeated process of compression (and decompression) and rigid body displacement of an elastic member with respect to the surrounding soil medium. When a pile is driven through soft soil layers into comparatively firm substrata, the positive friction (in upward direction on the pile) is partly reversed by elastic decompression of the pile and its relative displacement to the surrounding soil, after driving is stopped. The upper part of the pile shows the largest displacements and the reversing action therefore is strongest here. In its final (unloaded) state of equilibrium the weight of the pile is carried by the friction forces over part of its depth. Below that depth the friction forces are but little affected by the decompression. Their final distribution is governed by the stress relieve at the pile tip.

When a (static) load test is carried out in the usual manner by increasing the loads on the pile head in increments, a reversed process will take place. Negative friction is reversed to the extent of the relative displacement of the compressed pile with respect to the surrounding soil; positive friction, initially decreased by elastic decompression of the pile, will increase to its maximum value before the stresses on the tip of the pile will increase their initial value.

Again acting on the assumption, that the magnitudes of the residual friction stresses on the lower portion of the piles would be independent of their diameter, it was found that the residual force at the pile tip could be used as a dimensionless parameter.

The phenomenon of the residual force is of considerable practical importance, as zero displacements at the pile tip

over a certain range of the head loads raise the bearing capacity of the pile as a whole. It is also an indication of the relatively high bearing capacity of slender piles and could be used to advantage in the penetration of firm soil layers to larger depths than with piles of larger diameter.

The author had an opportunity on various occasions to use thin section piles for practical solutions of foundation problems, where low penetration energies had to be combined with high bearing capacities.

The model relationship as described then proved to be a great help in pre-determining both the penetration energy and the bearing capacity as functions of depth.

M. MOGAMI

About the point just mentioned by Dr Geuze : I have some remarks to make on the residual stress in soil. We confirmed that some kind of residual stress appears in sand when we perform the model test on the pile penetration, and if we take up the load from the pile this stress measured on the bottom of a model box does not disappear. It confirms the existence of some kind of a residual stress in the soil when the pile is driven in. But this has not yet been confirmed for the hammer-driven pile into sand. I think that the effect is not so severe in the actual case; but at least at some depth the effect of a residual stress would be more than that expected at the shallower level.

Le Président :

Time is short, and therefore we will not discuss the last question of the general procedure of pile tests and the definition of ultimate bearing capacity, but we suggest that this question be discussed later, in the free discussion. I will only mention that there is a general need for a standardisation of the pile testing, because you see in the different papers that there are always other methods for this purpose, and perhaps we can begin an attempt to correct this situation by some proposals for a standardisation.

Also there is a need for a general definition of the ultimate bearing capacity. There are some people who fix it by a settlement in absolute figures, or as a ratio to the diameter, or in other cases it is fixed as maximum pressure; but there are many cases where we have no maximum pressure and then that is not possible. Then we are forced to take another conventional pressure, and this pressure changes also in the different cities and countries. And finally we can take loading and unloading results and separate the elastic and the permanent load settlement curve and try to find some construction to find the ultimate bearing capacity.

That is only to give some suggestions for the free discussion.

If you agree, I will now ask Mr Chaplin to begin the free discussion before the interval, because Mr Chaplin has very little time : he must leave us at eleven o'clock. If you agree we can hear the discussion of Mr Chaplin now and then take an interruption of 15 minutes.

M. T.K. CHAPLIN (Grande Bretagne)

In his General Report Prof. Zeevaert very kindly suggested that I should present an explanation of the nomenclature used in my paper (3B/6), and also should explain very briefly how the different tests were performed, of which the results are presented in Figs 2 to 6 of my paper. I am very glad to have the opportunity to respond to the General Reporter's suggestion, particularly as the compressibility of sand is such an important factor in the topics (a) and (b) which we are discussing.

Figs 2, 4 and 6 of my paper (vol. II, pp. 35 and 36) represent triaxial compression tests carried out on samples 4 in.

(10 cm) diameter and about 8 in. (20 cm) high. Throughout each test the lateral pressure was increased at such a rate that there was no lateral strain, using a lateral strain indicator of the type developed by Dr Bishop. The sand samples were set up under initial suctions (to prevent collapse) of the order of 0·3 to 1·5 lb./sq. in. (0·02 to 0·1 kg/cm²). In Figs. 2 and 4 the horizontal axis represents the effective major principal stress, plotted to a square root scale. In Fig. 6 the corrected proving ring reading is simply the observed reading on the proving ring minus the calculated correction for the changes of water pressure on the underside of the piston. The corrected reading is therefore proportional to the principal stress difference (sometimes called deviator stress), and is also plotted to a square root scale.

Fig. 3 (p. 35) represents the results of isotropic consolidation tests carried out in the triaxial apparatus. σ denotes the effective ambient (hydrostatic) stress, and is plotted to a square root scale. None of the tests have been corrected for penetration of the membrane into the voids of the sample as the pressure increases; this penetration is a serious source of error at lower pressures, particularly in samples of low compressibility. Fig. 5 (p. 35) presents settlement measurements in dams obtained by cross-arm installations during construction; here $\bar{\sigma}$ represents the effective *nominal* overburden pressure, which is the nominal calculated overburden pressure (height of fill \times average bulk density) minus the observed pore water pressure.

There is a most important difference between the laboratory results presented in my paper and those of Prof. Schultze and his co-authors (papers 1/56, 1/57, 1/58 and 2/17). Whereas their tests were carried out in conventional cedometers, and are therefore subject in some cases to large errors due to arching and side friction, the laboratory results in my paper were all obtained by methods which entirely prevented arching and side friction. In my Ph. D. Thesis (Ref. [3] of my paper) I suggested that the results of cedometer compression tests on sand are likely to be reasonably reliable only when the sand is dense or very dense. I feel it is highly significant that the tests of Messrs Schultze and Moussa (paper 1/58, Fig. 2) show that the compression strain of their denser samples is very closely proportional to the square root of the applied pressure, which is in agreement with the result of my own paper.

Since my paper was written I have re-assessed the possible errors in the isotropic compression tests by Dr Fraser (Ref. [6] of my paper) due to membrane penetration into the voids of the sample. It now seems likely that even at low principal stress ratios between unity and $1/K_0$, the volumetric compressibility does depend markedly on the stress ratio.

Those who are interested in measuring the compressibility of sand without arching or friction losses will find a description of triaxial compression tests at zero lateral strain (K_0 compression tests) in Bishop and Henkel's book on the triaxial test, which was published by Arnold of London in 1957.

To sum up, the compressibilities of discrete solids (which include soils with a granular structure) on loading and unloading depend on the shape of the particles (roundness, sphericity and surface texture), their grading, limiting porosities and relative porosity, previous stress history and initial stress state. The intangible factor is the local development of "domains" and zones of locally higher or lower porosity during deposition and compaction. I hope there will be several papers in the next International Conference which report compression tests on granular materials, and which give the fullest possible description of the materials used and the methods of deposition and compaction.

Le Rapporteur Général :

To reach at this moment definite conclusions of what we have discussed is quite impossible, however it appears necessary to study more these problems on ultimate load of piles vs penetrometers, because, as you have seen, there is large disagreement. The same may be said for : reduction coefficients and effective ratios, etc.

The statistical studies made for certain locations should be used with discretion, for I believe that if applied at other sites the deviations may be very large.

We have to study also more on the similitude of models, in relation with the behaviour of the prototype and maybe four years from now we shall be able to find better solutions to our problems.

La séance fut levée à 10 h. 55 et reprise à 11 h. 20

Le Président :

I open now the free discussion. We have so many papers to discuss here that it is quite impossible that all papers will be discussed; many of them must be published later as a written discussion. I ask the speakers to speak very shortly, perhaps only five minutes, because if you take too much time at the beginning of the discussion the rest of the speakers will not have the opportunity to speak here.

M. C. VAN DER VEEN (Hollande)

In presenting my general report in Section II, Techniques of field measurement, I already drew attention to the excellent paper of Dr Kérisel and I should like to congratulate him on his contribution to the understanding of the bearing capacity of a pile.

I should like however to give a very brief comment.

First of all this. I'm definitely of the opinion that the tests performed by Dr Kérisel do not lead to the conclusion that the cone penetrometer cannot be used for the determination of the bearing capacity of a pile. Personally my conclusion went the other way round : I am inclined to ask what exactly is a pile ? I mean, when it is very short and it has a large cross section then it is better defined as a pier.

What I means is illustrated by the pictures shown by Dr Kérisel. Fig. 1 and 2 give for two different kinds of sand the point pressure at ultimate failure load against the natural effective stress in the field. There appears to be difference between the cone and the "piles caissons", but it is obvious that those "piles caissons" are not what we ordinarily consider to be piles. For normal piles there is less difference, less still when we take into consideration that as a rule the length of a pile is at least about 10 m. In that case the natural effective stress (pression des terrains) is 10 t/m² or so.

The same holds for Fig. 2. My conclusion is that we need not be too shocked by Dr Kérisel's results.

Further as field evidence shows, there is very reasonable agreement between the cone resistance measured by the Dutch penetrometer and the actual point bearing capacity of piles in quite a number of cases. I collected a fair number of data in my report "The bearing capacity of a pile predetermined by a cone penetration test", presented at the London Conference.

In this respect I might draw your attention to the valuable contribution of Dr Menzenbach to this Conference. May I put forward just one slight remark.

Menzenbach has taken into consideration more data than I did in my paper, presented to the London Conference; he comes in this way to more spread in the results. This however is not astonishing, as he took into consideration tests which were not executed with the high accuracy and complete

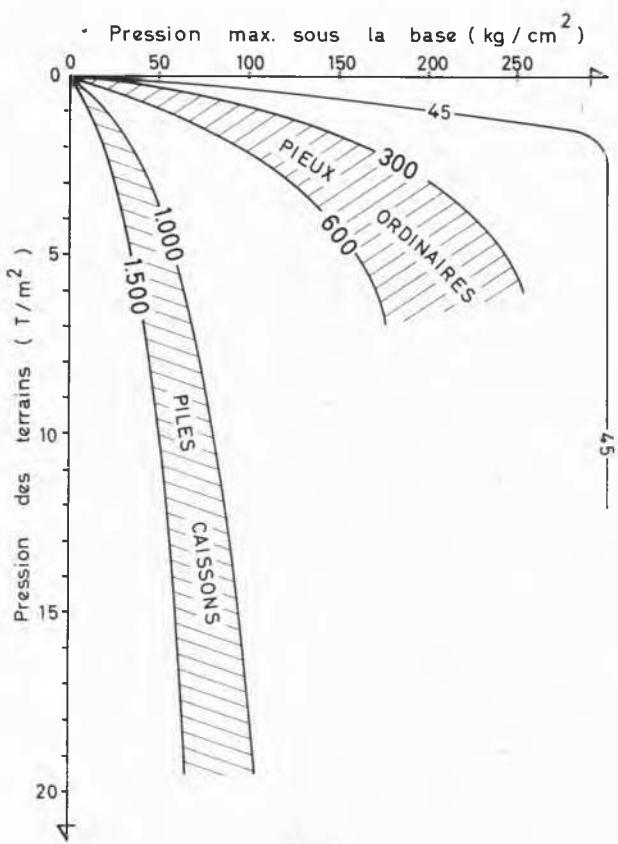


Fig. 1

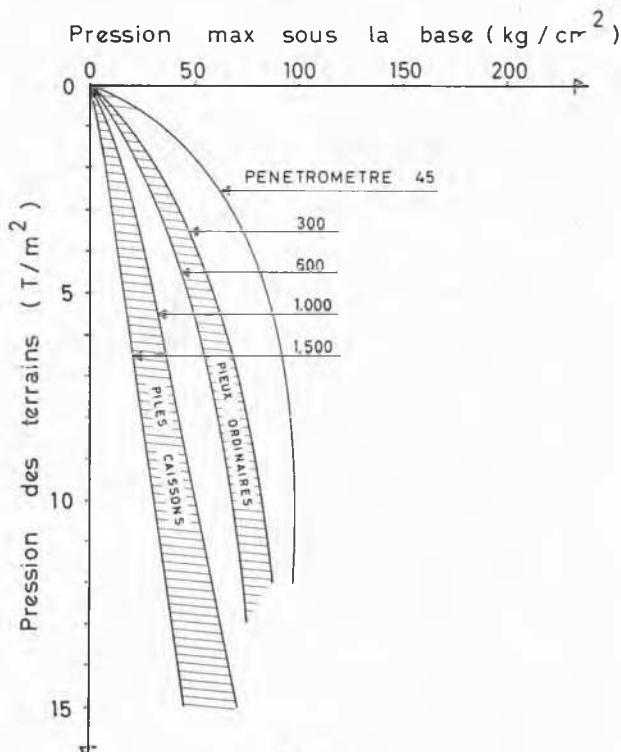


Fig. 2

availability of data that is necessary and these tests I therefore rejected. I considered those tests only :

1. In which the test on the pile was performed at such a high load that it was possible to determine the ultimate bearing resistance by the method explained in a paper presented to the Zürich Conference.

2. In which penetration tests were available within the strictest vicinity of the pile e.g. 5 m at most.

3. In which the measured cone resistance was evaluated correctly, as explained in my London Conference paper.

If this is done there appears to be a very good agreement between the cone resistance and the point resistance of the pile.

M. E. GEUZE

One of my comments has already been dealt with by Mr Van der Veen.

I still maintain my objection to Prof. Kérisel's statements, that the bearing capacity of a large diameter pile is lower than that of a small diameter pile in identical, homogeneous soil environments.

I still think that Prof. Kérisel has not taken the ratio between the displacements and the diameter into account and that is the only possible basis for a comparison when the yield limit of failure cannot be properly defined.

I would also strongly recommend to avoid the range of loads, which by the nature of the phenomenon of failure around the pile point causes progressively increasing displacements both by the increase of the volume of soil involved in the failure and by the gradual transition from time independent strains to indefinitely increasing strains.

The fact that during this stage of the process the displacements show a time dependency proportional to the applied load is sufficient proof of the undetermined character of the yield value for penetration at depth.

In my opinion the yield value of the penetration resistance is much more clearly defined by the range of time-independent displacements. This limit has been shown to be about 0.5 times the resistance obtained at conventional rates of penetration and proved to be fairly independent of the pile diameter.

This represents also the limit of the safe load, as has been found by a comparison of field data on piles with penetrometer tests.

It is evident that the above mentioned relationships do not hold any longer in the case of a layered soil medium, when the ratio between the pile diameter and the layer thickness has to be taken into consideration.

M. KÉRISEL

Je voudrais remplir un devoir fort agréable, celui de vous présenter le Vice-Président qui siège à cette table, M. Henri Courbot. Sa présence nous est agréable à un double titre; d'abord, parce que c'est un ingénieur des Sols puisqu'il dirige une excellente entreprise de fondations qui est bien connue dans tous les milieux français; ensuite, parce qu'il est Président de la Fédération Nationale des Travaux Publics. Il est significatif pour nous, ingénieurs des Sols, de voir M. Courbot, entrepreneur spécialisé en fondations, présider depuis bientôt dix ans la Fédération française de tous les entrepreneurs à quelque spécialité qu'ils appartiennent. C'est grâce à lui, grâce à l'intérêt qu'il a toujours apporté aux problèmes de fondations et grâce à l'appui financier de sa fédération, que nous avons pu mettre sur pied l'organisation de ce Congrès. Je tenais à le dire devant vous.

Je voudrais également répondre deux mots au Prof. Geuze à propos de la détermination de la charge limite.

Cette détermination offre effectivement une difficulté lorsque l'on fait un essai isolé de charge limite. Mais si l'on connaît même approximativement l'enveloppe des essais, c'est-à-dire

la courbe de poinçonnement continu en fonction de la profondeur, la charge limite se détermine comme étant le point de raccordement du diagramme de l'essai de charge avec cette courbe. Pratiquement, il suffit de connaître approximativement la pente de cette courbe au voisinage de la profondeur à laquelle se fait l'essai.

M. H. PETERMANN (Allemagne)

Au Quatrième Congrès International de Mécanique des Sols et des Travaux de Fondations de Londres en 1957, il a été fait référence à des essais importants dans la région de Brême, pour la détermination de la force portante des pieux en béton armé d'une faible épaisseur de 20/20 et de 25/25 cm.

Actuellement la deuxième série d'essais en vraie grandeur, alors annoncée, est terminée. (Fig. 3).

Un chantier d'essais à Brême a été organisé au moyen de forages, d'essais de battage et d'essais de pénétration statique, et 48 pieux en béton armé de 20, 25, 30, 35 et 40 cm d'épaisseur ont été battus et chargés. Il s'agissait de pieux de 7 à 11 m de longueur avec des enfoncements dans le sol portant de 1,30 à 5,90 m. Les mesures obtenues pendant le battage

des pieux ainsi que pendant les essais de charge ont été comparées avec celles données par des essais de sondage.

En général les résultats d'essais correspondent à ceux de la première série d'essais, effectuée quelques années auparavant, concernant l'utilité des pieux en béton armé d'une petite épaisseur pour la construction d'habitations.

La nature géologique du sous-sol du terrain d'essais à Brême est caractéristique pour toute la côte de l'Allemagne du nord-ouest. Sous des couches de limon de quelques mètres d'épaisseur se trouvent des couches de sable et des lits gravereux d'une plus grande épaisseur. En général ces sols non cohérents ne possèdent qu'un degré de compacité très moyen ou même inférieur à la moyenne.

Dans la deuxième série d'essais, on disposait de 5 différentes méthodes d'essais pour comparer les recherches et les mesures, à savoir : le forage, l'essai de battage, l'essai de pénétration statique, ainsi que le battage et la charge des pieux. En ce qui concerne les deux dernières méthodes d'essai les résultats proviennent d'essais effectués avec des pieux de 5 épaisseurs différentes.

Les résultats des essais effectués sur différents endroits du chantier d'essai ont été comparés. La situation des couches du

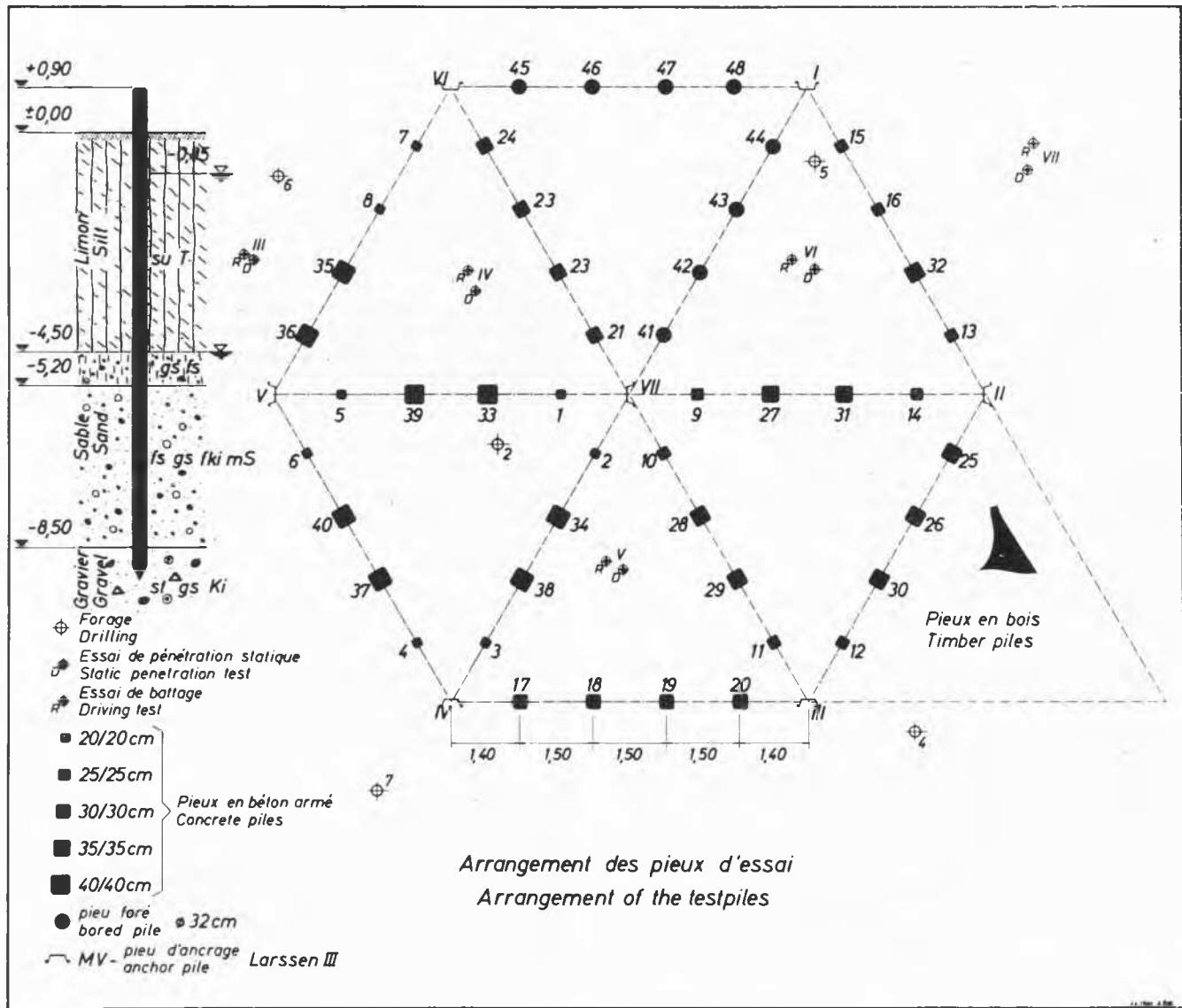


Fig. 3

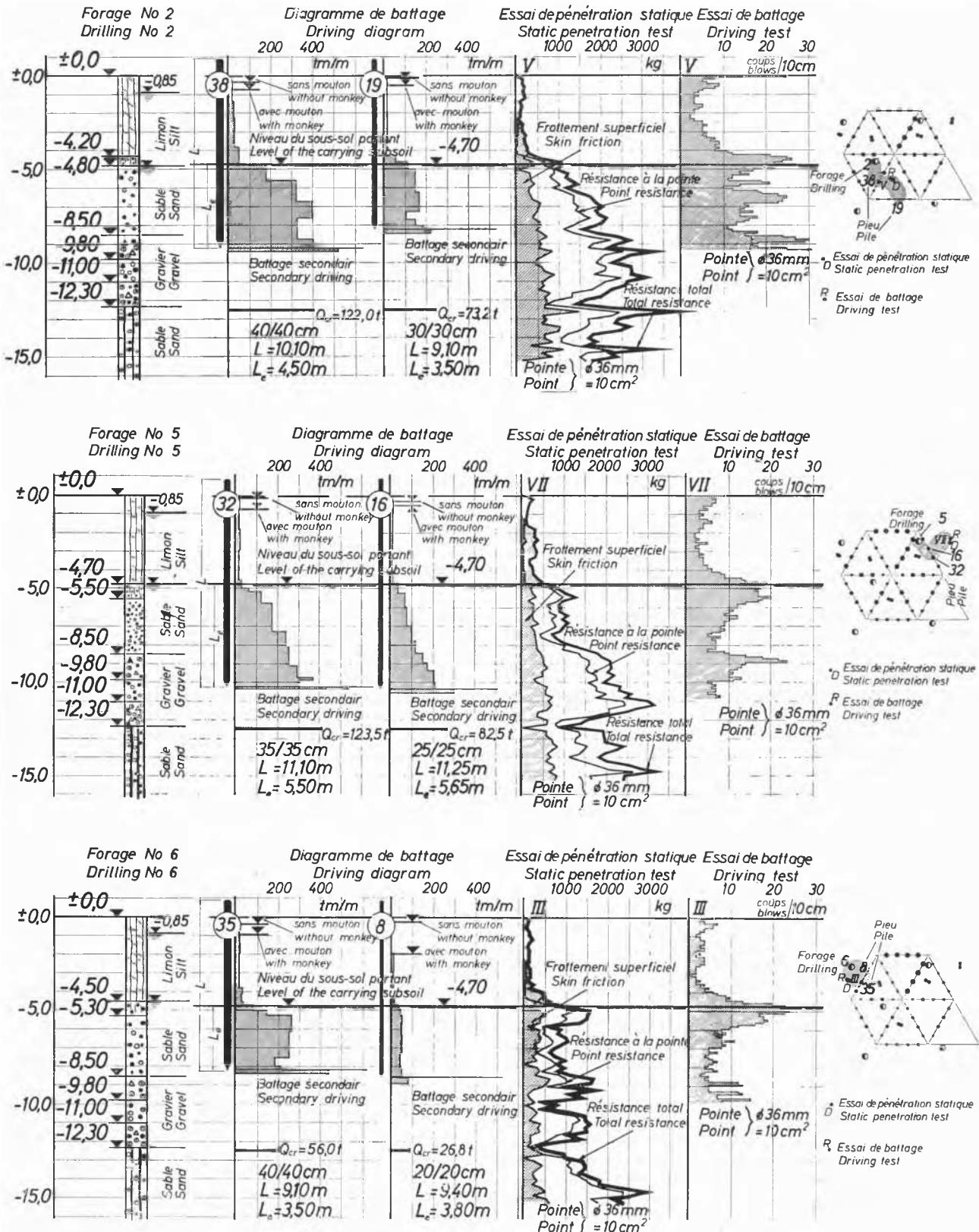


Fig. 4

sous-sol ne montrait que des déviations peu considérables.

L'essai de pénétration statique effectué avec une pointe de 10 cm² montre bien clairement la transition de la couche de sol tendre à la couche de sable. De plus la transition de la couche de sable à la couche graveleuse est bien visible. Plus profondément il y a encore une couche de sable dont la compacité moindre est indiquée par le pénétromètre.

En général ces différences ne sont indiquées que par la résistance de pointe. Le frottement superficiel augmente un peu avec l'enfoncement ou reste constant. Dans le sol non portant le frottement superficiel reste voisin de 10 kg/cm². La résistance de pointe reste voisine de 50 kg/cm², dans les couches tendres.

Le diagramme de sondage indique une grande sensibilité du pénétromètre pour les différences locales de compacité dans le sous-sol. (Fig. 4).

L'essai de battage (effectué lui aussi avec une pointe de 10 cm²) est d'une sensibilité comparable, ce que montrent les irrégularités dans le diagramme. Mais ici aussi le passage de la couche non portante au sol portant est bien visible. De même la transition de la couche de sable à la couche graveleuse sous-jacente est clairement indiquée. En cas d'un battage de moins que 5 coups pour un enfoncement de la pointe de 10 cm, le sol apparaît non portant pour des pieux.

Pour chacune des deux méthodes de sondage susmentionnées les transitions d'une couche à l'autre sont toujours bien clairement indiquées. L'essai de battage d'une sonde indique cette transition presque brusquement.

Le sondage à l'aide du pénétromètre, de son côté, l'indique par une augmentation relativement plus grande de la résistance. En plus de ces essais, on disposait d'autres essais avec des pieux en béton armé.

Le diagramme de battage des pieux montre toujours l'énergie de battage moindre dans la couche tendre. L'énergie de battage augmente brusquement quand on arrive aux couches portantes.

Environ 15 mois après le battage et à la fin de tous les essais on a effectué un battage secondaire des pieux. Dans tous les cas il y a eu une plus ou moins grande augmentation de l'énergie de battage.

Les résultats d'essais ont été interprétés de telle sorte qu'on puisse compter atteindre une couche portante, si, en cas de pieux d'une épaisseur habituelle, c'est-à-dire de 30, 35 ou 40 cm, une énergie de battage de 50 tm/m est nécessaire. Pour les pieux d'épaisseur moins considérable, (à savoir de 20 et 25 cm) on pouvait fixer la mesure de l'énergie de battage à 40 tm/m à l'endroit de transition à la couche portante.

La justesse de cette estimation du passage au sol portant est confirmée par les résultats de forage et les essais de sondage.

Malgré les différences dans l'épaisseur de pieu les diagrammes de pilotage indiquent toujours le même critère. Au forage no. 5 on a trouvé pour l'énergie de battage des pieux une augmentation proportionnelle à l'enfoncement dans le sol portant. Ce caractère de l'augmentation de la résistance se vérifie aussi dans le cas de l'essai de pénétration statique. Au forage no. 2 cependant l'augmentation de l'énergie de battage dans le sol portant est moindre. Au forage no. 6 l'énergie de battage est le plus souvent constante, ce qui se vérifie aussi dans le cas de l'essai de pénétromètre statique par insensibilité à l'enfoncement.

L'essai de chargement des pieux a été interprété en supposant que la charge critique était atteinte sous la charge d'essai, chaque fois, par un certain tassement du pieu. Pour les pieux d'une épaisseur habituelle, (à savoir 30, 35 et 40 cm) un tassement permanent de 6 mm a été fixé comme mesure critique pour la charge limite. Cependant, pour les pieux de petite épaisseur on ne peut — selon les résultats d'essais — fixer qu'une mesure de tassement moindre. En ce qui concerne la définition de la charge critique pour des pieux de 25 cm on a fixé un tassement permanent de 5 mm et pour des pieux de

20 cm ce tassement a été fixé à 4 mm. Cette supposition est basée sur le plus grand déplacement pour le développement de la résistance d'enfoncement dans le cas des pieux minces.

De cette hypothèse résultent les valeurs pour la charge critique des différents pieux, à partir de laquelle on peut déterminer la valeur réduite de la charge admissible en prenant en considération un degré de sécurité de 1,5.

Dans la région du forage n° 6 le sol est moins portant qu'aux autres endroits. Cette caractéristique était rendue également apparente par d'autres observations pendant la série d'essais. On reconnaît la différence des charges critiques pour les deux pieux de 40 cm d'épaisseur et d'un enfoncement de 3,5 et 4,5 m dans la couche portante. En cas d'un enfoncement d'environ 5 1/2 m dans le sol portant, il résulte de la comparaison que les pieux minces — en ce cas-là le pieu de 25 cm — sont des éléments de fondation absolument utilisables. De même, le pieu susmentionné de 20 cm d'épaisseur avec une longueur d'enfoncement de 3,80 m dans le sol portant permet encore de compter sur une charge admissible d'environ 18 tonnes.

Les exemples expliqués ne prêtent pas à des relations exactes entre les différents modes d'essais pour fixer plus précisément les rapports fonctionnels. Les essais indiquent cependant que les essais de battage et de pénétration statique constituent un moyen important pour la détermination de la force portante des pieux de fondation.

Le Président :

Je remercie M. Petermann, et je donne maintenant la parole au Prof. de Beer.

M. E. DE BEER (Belgique)

C'est avec grand intérêt que j'ai pris connaissance des essais effectués par le Prof. Kérisel et de l'étude statistique faite par M. Menzenbach. Je voudrais y ajouter les constatations suivantes :

1. Nous avons effectué, au cours des années, un assez grand nombre d'essais comparatifs avec des cônes de 36 mm et de 50,5 mm de diamètre. En général, nous avons obtenu les mêmes résistances à la pointe pour les deux cônes. Nous avons cependant aussi rencontré des couches, assez rares d'ailleurs, où la résistance à la pointe, mesurée avec le grand cône, est nettement inférieure à celle mesurée avec le petit cône. C'est ce qui est illustré par la Fig. 5.

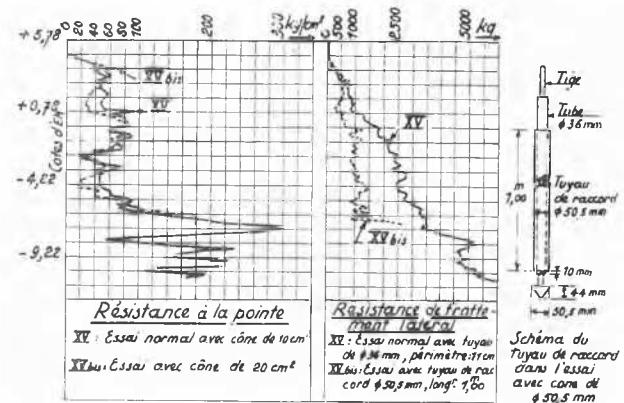


Fig. 5 Essais de pénétration à Langerbrugge.

2. Un autre point sur lequel je voudrais attirer l'attention est la profondeur considérable à laquelle on doit foncer la pointe d'un pieu dans la couche résistante, pour obtenir la même résistance que celle enregistrée avec le petit cône de l'appareil.

A ce sujet, on peut remarquer que dans les essais de péné-

Résistance à la pointe

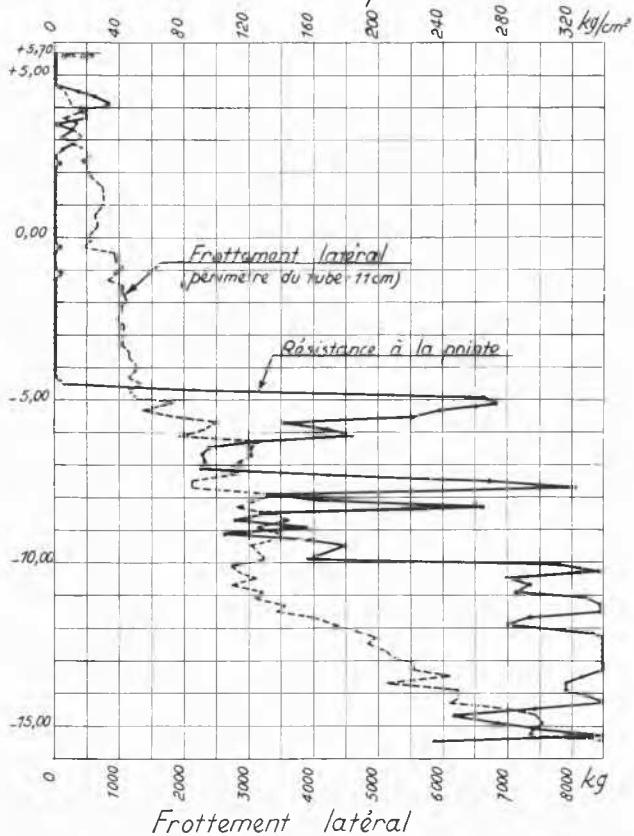
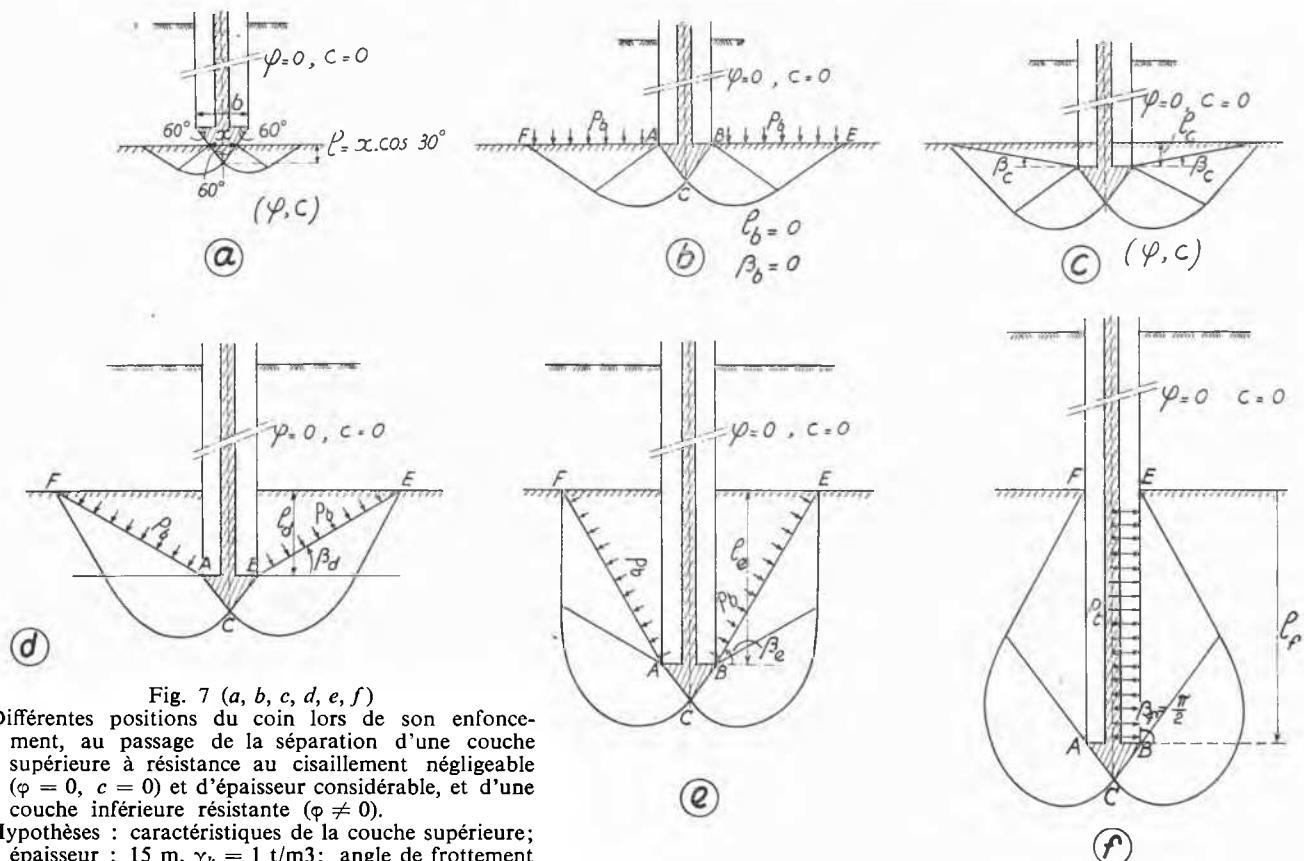


Fig. 6 Essai de pénétration à Anvers.



tration réels, les résistances à la pointe se situent souvent sur des alignements plus ou moins droits de 40 à 60 cm de longueur et à accroissement très rapide de C_{kd} (voir par exemple le diagramme de la Fig. 6).

Dans une conférence que nous avons donnée en Yougoslavie en 1955, nous avons analysé plus en détails ce phénomène, et nous avons indiqué qu'il est dû au fait que l'état de sollicitation du cône se modifie graduellement d'une sollicitation en surface en une sollicitation en profondeur.

Les Figs. 7 et 8 sont simplement projetées pour vous schématiser l'explication donnée au phénomène.

La Fig. 9 indique les valeurs considérables de la profondeur relative $l_2 : b$ pour que le phénomène de profondeur puisse pleinement se développer. Pour un sable compact caractérisé par $\varphi = 40^\circ$, on obtient des valeurs de $l_2 : b$ de loin supérieures à 20.

Si en dessous d'une couche très peu résistante AB (Fig. 10) on trouve une couche résistante, dans laquelle le cône de $\varnothing 3,6$ cm du pénétromètre éprouve des résistances BCD en fonction de la profondeur, rien ne serait plus faux que de croire qu'un pieu de $\varnothing 36$ cm, par exemple, éprouvera la résistance C'C au niveau du point C. Puisque le cône de l'appareil a eu besoin d'un enfoncement BC' (par exemple $BC' = 60$ cm) pour ressentir l'effet de profondeur, il faudra une profondeur BC'_p dans le rapport des diamètres de la base du pieu et du cône, pour obtenir la même résistance que celle éprouvée par le cône. A cette profondeur, à cause de la surcharge accrue, on aura finalement la valeur C'_p D.

Surtout dans les sables compacts à très compacts, les profondeurs d'enfoncement requises sont tellement importantes qu'elles ne sont pratiquement jamais atteintes dans la pratique.

La Fig. 11 montre comment les grandes profondeurs de pénétration requises dans des sables très compacts, ainsi que le fait que les pieux sont généralement arrêtés dans ces couches résistantes à des profondeurs ne dépassant pas 2 à

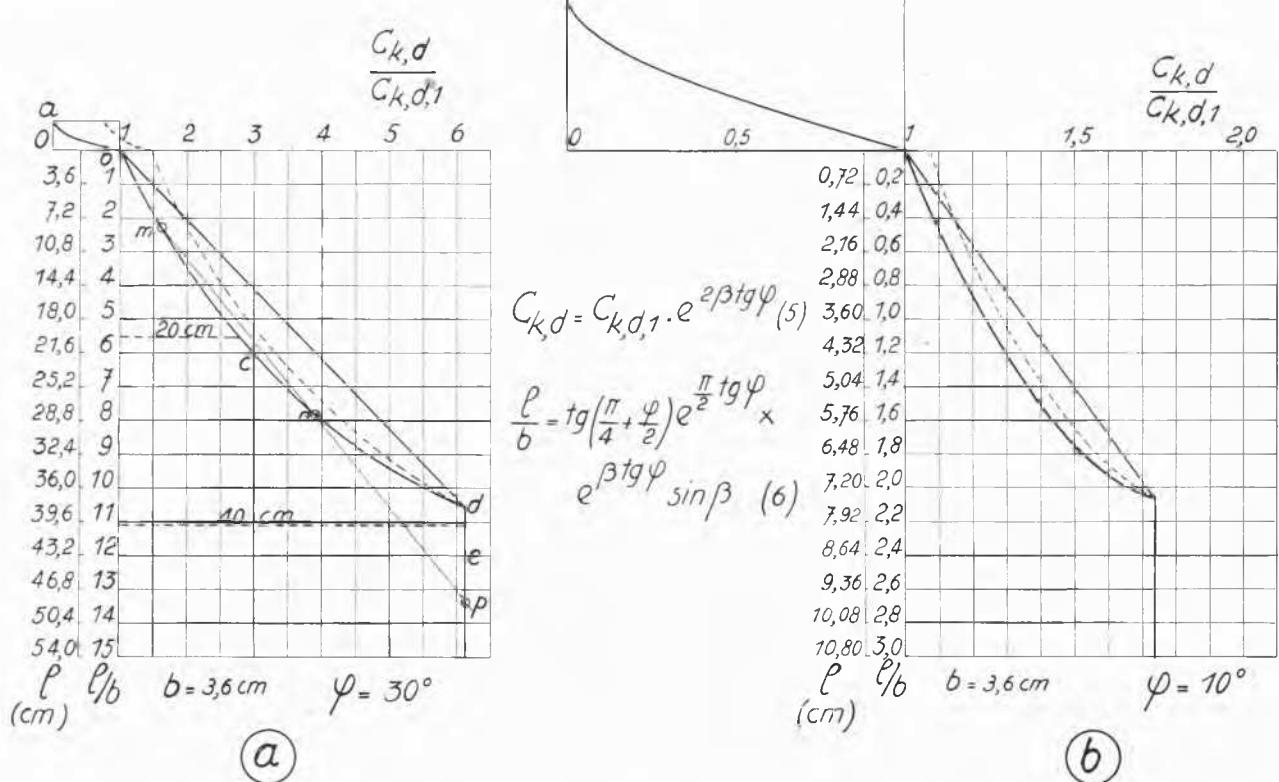


Fig. 8 a et b Variation du rapport de la résistance à l'enfoncement $C_{k,d}$ à la résistance $C_{k,d,1}$ éprouvée dans la position indiquée à figure 3 b en fonction de la distance de la pointe du coin au niveau de séparation des 2 couches, ou du rapport de cette distance à la largeur b du coin.

Fig. a angle de frottement de la couche inférieure : $\varphi = 30^\circ$.

Fig. b angle de frottement de la couche inférieure : $\varphi = 10^\circ$.

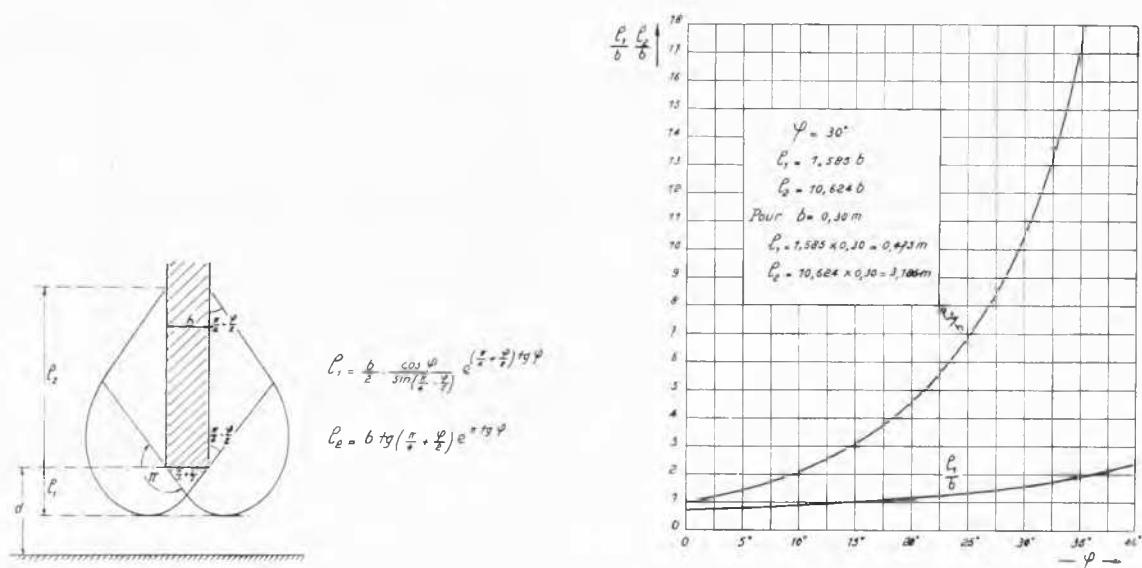


Fig. 9

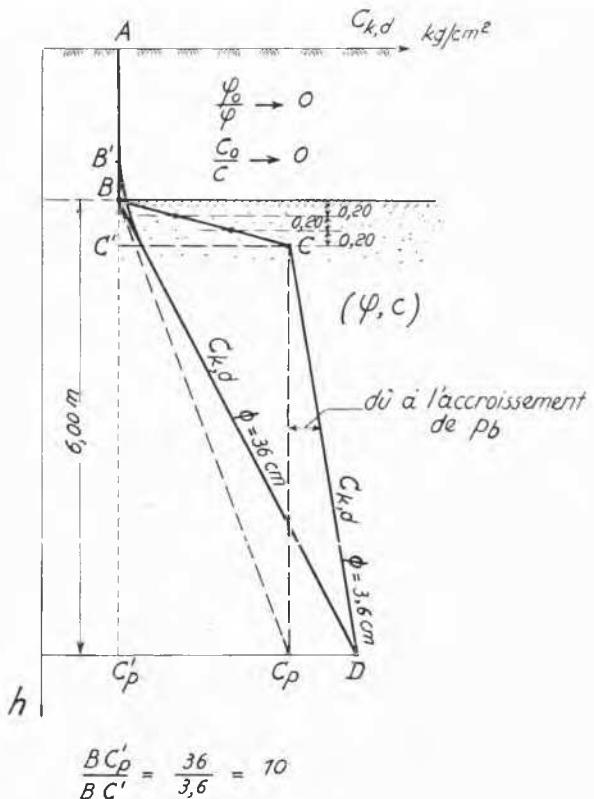


Fig. 10

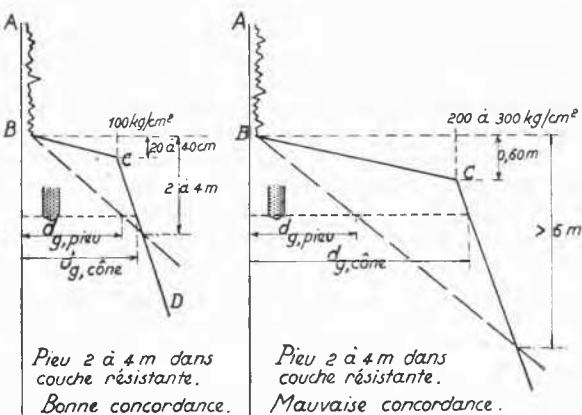


Fig. 11

3 mètres, expliquent les divergences entre les résistances au cône et les résistances à la base du pieu. La même figure explique, pour des raisons analogues, la bonne concordance obtenue dans des sables à compacité moyenne.

3. Il existe aussi des sols où la résistance à la pointe augmente pratiquement linéairement avec la profondeur. La Fig. 12 en donne un exemple. Dans de tels sols, la détermination de la force portante limite du pieu par résistance à la base n'inclut pas la difficulté mentionnée ci-dessus. Il s'agit en ce cas de sols ayant une compacité légèrement supérieure à la compacité critique.

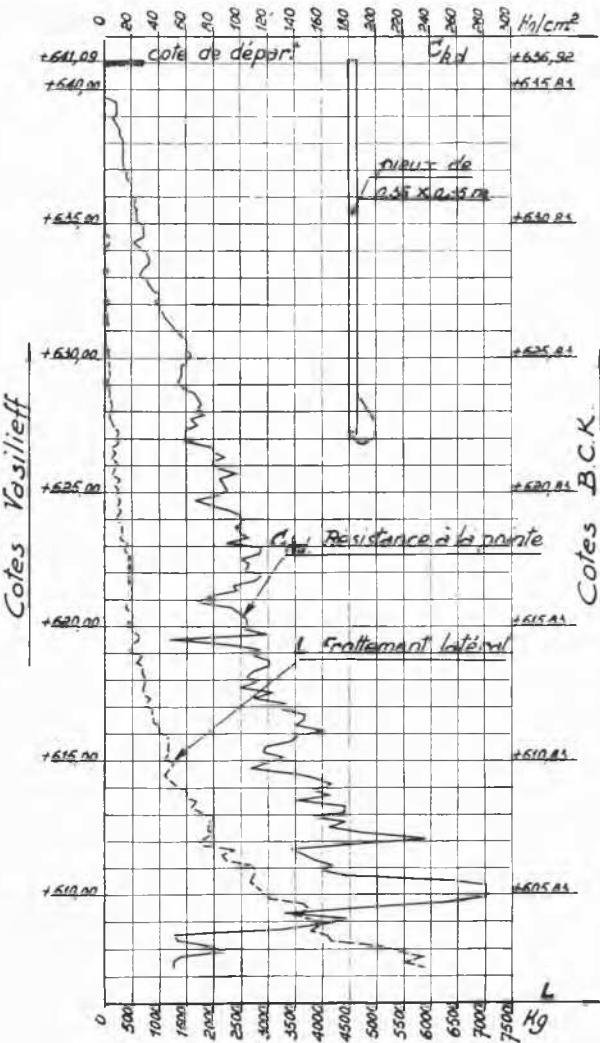


Fig. 12 Essai de pénétration VI.

4. A la Fig. 7, nous avons supposé qu'à partir du point C les résistances à la pointe augmentent linéairement avec la profondeur. En général, dès que les résistances à la pointe sont grandes (par exemple de l'ordre 200 à 300 kg/cm²), pour une compacité donnée, elles resteront souvent constantes. Il n'y a plus un phénomène de refoulement pur, mais un phénomène influencé par l'érassement des grains. Un essai à pression triaxiale effectué par M. Ladanyi, sous une contrainte latérale de 50 kg/cm² produit déjà une attrition importante comme le montre la détermination des courbes granulométriques avant et après essai (Fig. 13). Il doit en être ainsi *a fortiori* lorsque l'on enfonce un cône sous des pressions de 200 à 300 kg/cm².

Je crois que toutes ces constatations ont leur utilité pour l'interprétation des résultats expérimentaux du Prof. Kérisel et des études statistiques de M. Menzenbach.

COURBES GRANULOMETRIQUES

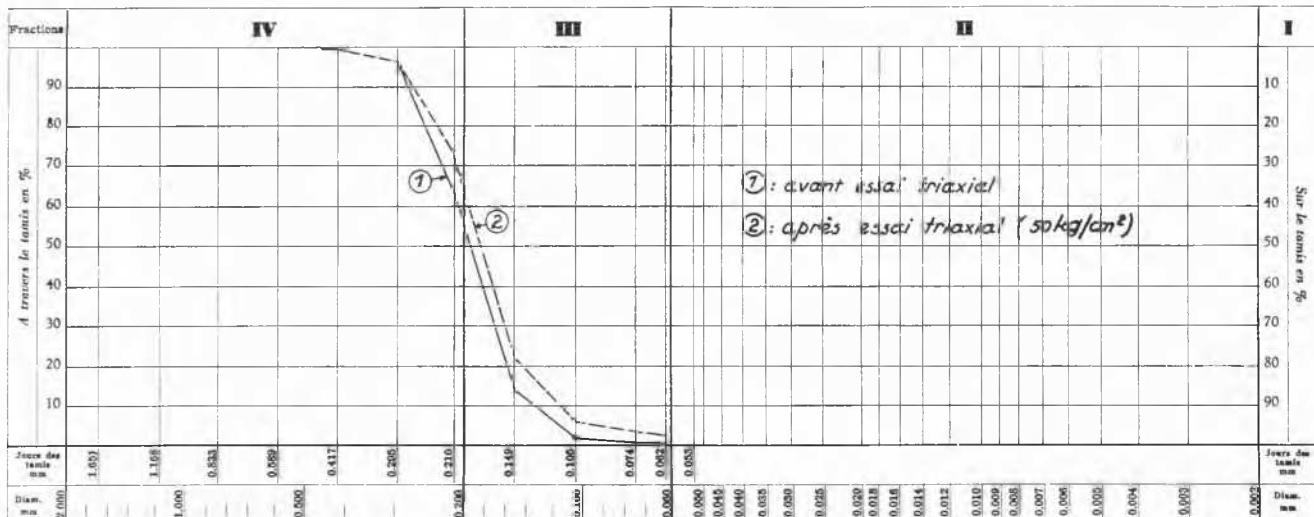


Fig. 13

M. W.G. HOLTZ (Etats-Unis)

There has been considerable discussion on the use of the cone penetrometer for estimating load bearing and pile bearing capacities. I think it would be well, at this time, to point out some of the correlations which can be obtained between pile driving resistance and the driving resistance of the standard penetration spoon. The thick-walled, split-spoon sampler has been used for several years by the Bureau of Reclamation to estimate the working load capacity of timber friction piles. These estimates have worked out exceptionally well on several pile driving contracts, provided the soil types and moisture conditions were properly evaluated. It is the practice of the Bureau of Reclamation to require that timber piles for small structures be driven to a specified working load value based on the Engineering News Formula (Chellis, 1951). These small structures include transmission towers, canal bridges, and small pumping plants. When piles are required for large structures with complex soil foundation conditions, test piles are usually driven and load tested for determining safe load-bearing capacities.

The correlation of spoon penetration resistance and pile driving resistance is based on the fact that both the spoon and the pile are driven by impact methods and the driving resistance is related to the firmness, or density, of the soil and the overburden pressures. While the pile resistance to driving reflects the entire depth driven, the spoon resistance to driving reflects only a 1-foot length when values are given on blows per foot. Therefore, the results of the spoon penetration test must be multiplied by the depth interval of the soil represented by the test and these test data for each depth interval summated for the entire depth being studied. In this manner, the spoon penetration-resistance values are used to estimate P , the approximate bearing resistance in pounds by the following empirical formula :

$$P = K_1(d_1 N_1) + K_2(d_2 N_2) + \dots + K_n(d_n N_n)$$

where N_1, N_2, \dots, N_n . Number of blows per foot, in corresponding increments of depth d_1, d_2, \dots, d_n in feet, obtained from the penetration test, and

K_1, K_2, \dots, K_n . A factor in pounds per linear foot of pile depending on the type of soil in the corresponding increment of depth.

Examples of values of K which have been determined from field test data of pile driving and penetration testing are :

Plastic clay = 100

Low plasticity saturated clays to silts = 50 to 65

Sand = 65

Loess : Low water content = 200

High water content = 100.

The spoon penetration test is performed by the Bureau of Reclamation (Earth Manual, 1960) in conformance with Standard Method of Test, D 1 586-58 T, of the American Society of Testing and Materials (ASTM Standards, 1958). The pile driving data for the above factors were obtained on treated and untreated timber piles with approximately 8-inch-diameter tips and 12- to 16-inch-diameter butts.

Figs. 14 to 17 are examples of some of the correlation data which have been accumulated over a period of several years. The piles shown as examples were load tested in conformance with ASTM Standard Method D 1 143-57T without significant movements. The maximum loads applied to the piles of Figs. 14, 15, and 16 were 100 tons, with the load on the Fig. 14 piles remaining for 4 days. The pile shown on Fig. 17 was tested under a maximum load of 200 tons.

It has been found that the spoon penetration test can be used effectively to estimate timber pile lengths in various types of soil foundations. The "K" values listed in this discussion are normally on the safe side for estimating working loads. It has been our practice to use these values directly for preliminary estimates. When using them for final designs or construction control of transmission lines or canal structures, test piles may be driven adjacent to a penetration test hole in representative soils along the alinements to check the values. The spoon penetration data can then be used with greater confidence at the numerous structure sites along the alinements. When pile driving and loading tests are made at large and critical foundation sites, the penetration tests made throughout the site area provide a good means for extending the interpretation of the pile test data over the entire site area.

Références

- [1] CHELLIS, R.D. (1951). *Pile Foundations*, McGraw-Hill Book Company, Inc., First Edition, p. 530.
- [2] *Earth Manual*, Bureau of Reclamation, Denver, Colorado, 1960, p. 574.
- [3] *Standards of the American Society for Testing and Materials*, 1958, Part 4, p. 1110.

TEST PILE B

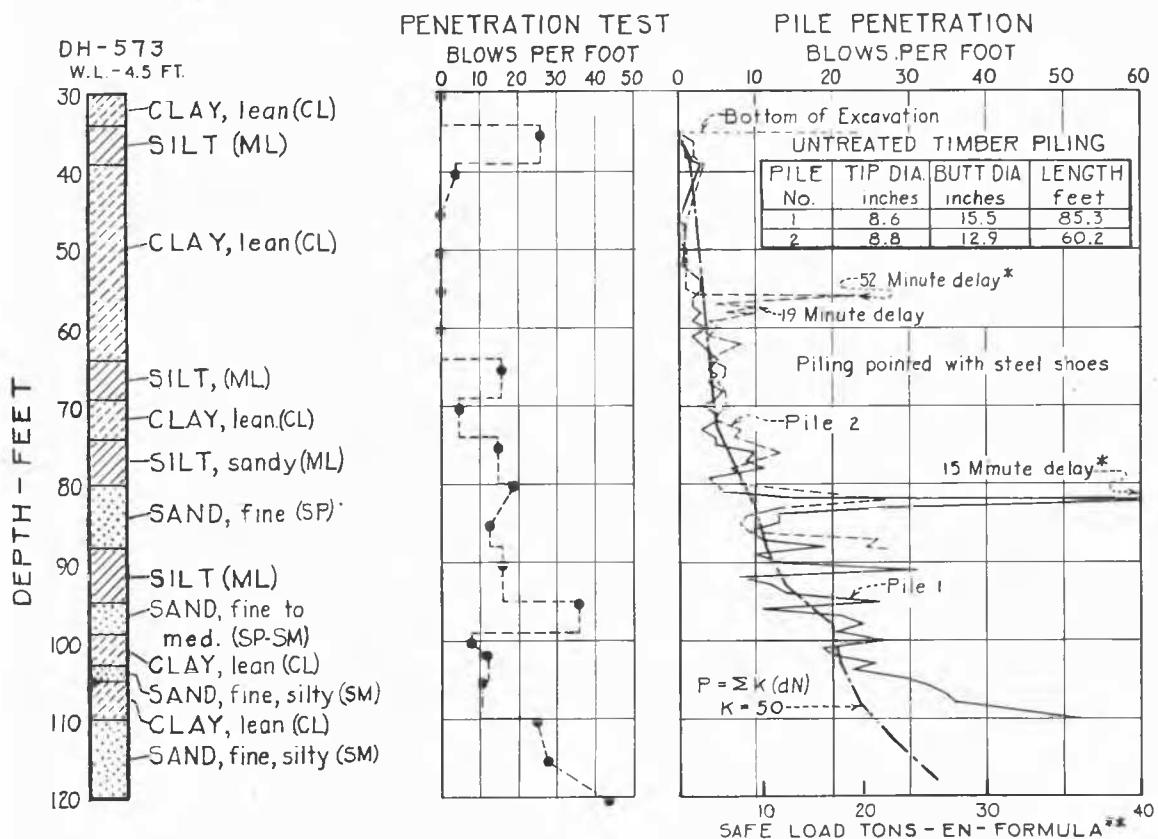


Fig. 14 Penetration resistance & pile driving records. Willard Pumping Plant No. 1.

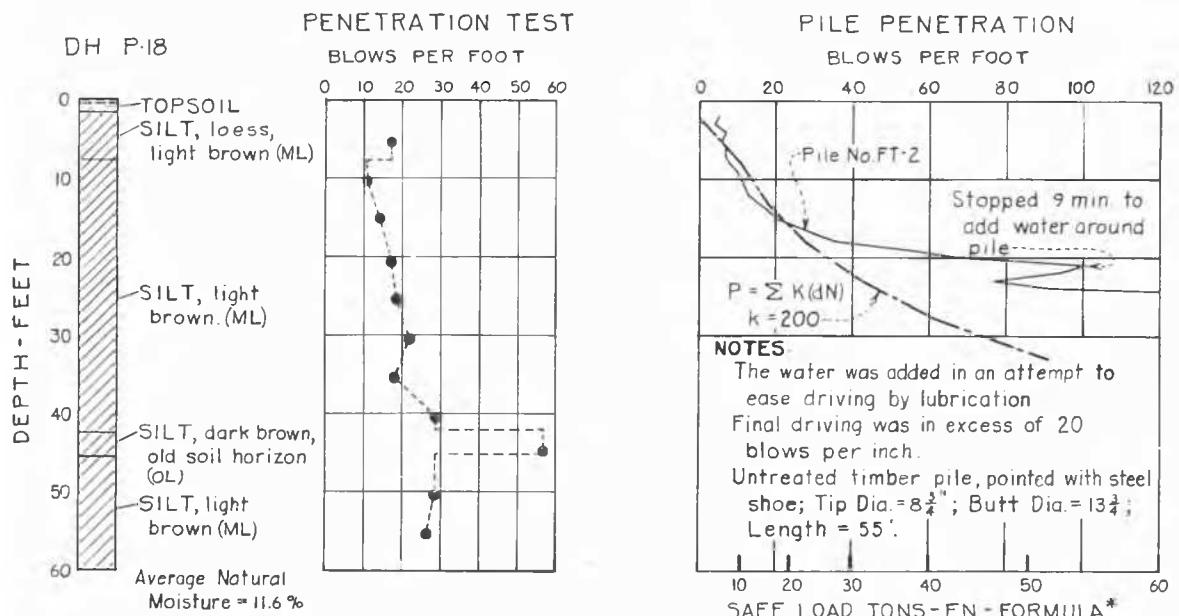


Fig. 15 Penetration resistance & pile driving record. Jamestown-Fargo 230 KV transmission line.

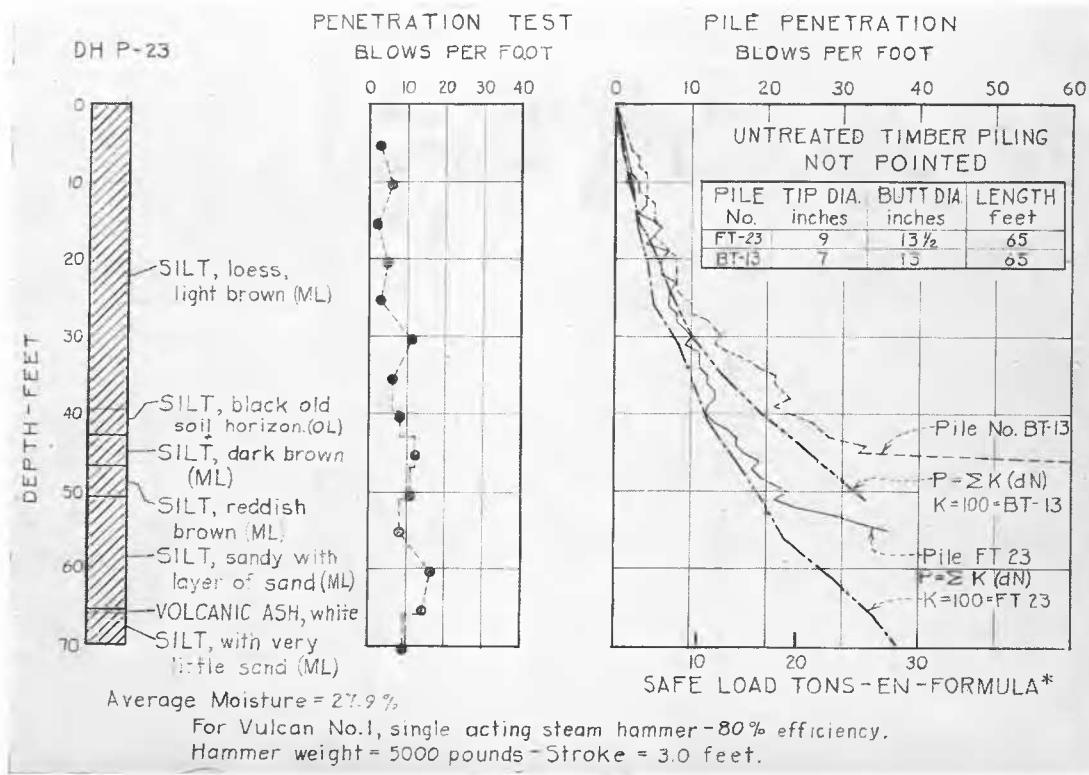
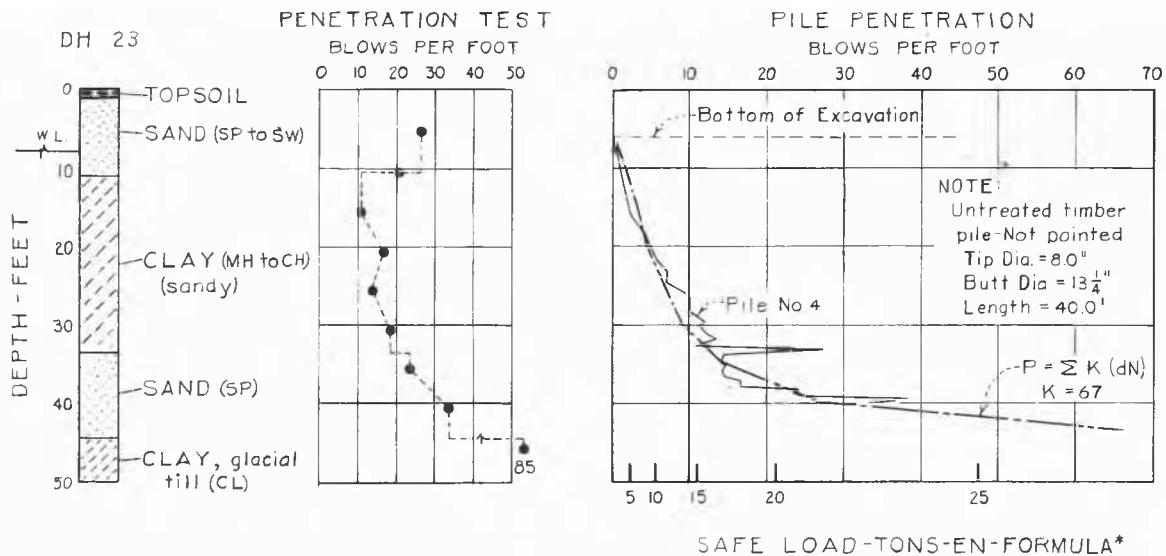


Fig. 16 Penetration resistance & pile driving records. Ashton, Nebraska - pile testing program. Tests in prewetted area.



* For 3130 pound gravity hammer - 100% efficiency - 10 foot fall

Fig. 17 Penetration resistance & pile driving record. Ashton, Nebraska - pile testing program. Tests in natural moisture area.

M. A.J.L. BOLOGNESI (Argentine)

There is no simple method available for determining the necessary length of piles embedded in saturated hard silts and clays, and the difficulties of this problem indicate that it is improbable that penetrometers could be a solution.

Theoretically, point bearing is practically negligible in saturated silts and clays, and most of the load is taken by lateral friction, but the fact is that there are certain saturated

silts and clays that behave practically as frictional materials.

This is not exceptional, but common experience. It requires considerable experience in local conditions to predict the way that saturated silts and clays will carry the pile loads. It is taken mostly by lateral friction in certain types, but very frequently there is a very important point bearing. The prediction of this behaviour is made more confusing because it is very difficult to obtain, under economical conditions, satisfactory undisturbed samples of hard soils at the depth

where they are found when pile foundations are required.

The behaviour of saturated hard silts and clays seems to be very much influenced by their structure. This structure controls the drainage characteristics. This might be a possible explanation of the fact that certain saturated hard silts and clays behave as frictional materials, developing both dynamic resistance during driving and static resistance under load tests to be safely used with short lengths of embedment, similar to that necessary in frictional materials.

Le Président :

You have heard this morning from Prof. Kérisel about the bridge of Maracaibo. Now we will hear from Dr Simons some discussion about this very interesting project.

M. H. SIMONS (Allemagne)

I would like to speak concerning questions *a*) and *d*) about the very extensive load pile tests performed for the construction of the Maracaibo Bridge which is 8·6 km long; I worked on the foundation of this bridge with Prof. Kérisel.

For the foundation of the bridge two types of piles have been used : driven piles and bored piles. The working loads required were between 250 tones for driven piles and 750 tons for bored piles.

The driven piles have an external diameter of 92 cm, wall thickness of 5 inches and they are prestressed in the longitudinal direction. The point of the piles are closed. They have a weight of 0·8 tons per meter, so that the weight of a 56 m long pile was 45 tons. So we need very heavy hammers to get a good relation between the weight of the hammer and the weight of the pile. Therefore hammers of 15 and 20 tons and the corresponding driving equipment have been specially constructed.

The 15 and 20 ton steam hammers are semi-automatic so that the steam-exit and therefore the drop of the hammer was regulated by hand with a rope. According to the soil-conditions the piles were driven with drop varying between 15 and 75 cm. The worst relation between the weight of a pile to the weight of the hammer was 0·4. Up to that relation piles of this type can be driven very hard.

During the first tests with the hammer of 20 tons, the fixing bolts of the pile cap at the rail of the hammer broke after a short time, despite these bolts having a breaking resistance of 80 tons. This happened because the usual pile-cap is fixed at the rail of the hammer and so it has an asymmetrical cross-section. Therefore a perfect symmetrical pile-cap has been built and is loosely placed on the head of the pile. The pile guide is fixed to the hammer rail just below the pile cap, but is independent of the hammer itself. The hammer hits a flat steel plate, without any ball formed centering guide. Our experience has shown that these guides are not necessary. However the greatest care is necessary for the exact centering of the falling weight. A few mm's wear of the hammer weight guides can cause sufficient eccentricity of driving, for the piles to be destroyed. We have driven test piles by the 20 ton hammer in sand and stiff clay up to 2 000 blows for drops of between 15 and 75 cm. They have shown the following results : Independent of the hammer drop, the piles were absolutely undamaged up to 1 200 blows, but above 1 500 blows they often get horizontal cracks. To reduce the number of blows, we have always used the largest possible drop. However, when driving piles through soft soils, the piles can easily develop tension cracks. The drop was limited so that the pile did not penetrate deeper than 1 to 2 cm per blow. For this the semi-automatic hammer was found to be better than the automatic, because it was easier to regulate the hammer drop according to the measured penetration per blow.

Many test drivings, checked by calculations with different driving formulae have shown that only 13 blows of the 20 ton hammer were equivalent to 100 blows of the 15 ton hammer for the same drop. The piles were driven until the 15 ton hammer needed more than 100 blows for the last 3 cm or alternatively the 20 ton hammer needed more than 25 blows for the same penetration. One test load was on a driven pile carrying 600 tons at which the load was only stabilised with two auxiliary hydraulic jacks.

To determine the required length of the driven piles in advance, we used a special static cone penetrometer, type Heerema, fixed at the end of a telescopic steel pile. We found that the depth which could be reached by driving the piles corresponded to a cone resistance of 100 kg/cm².

The bored piles are also hollow cylinders with 1·35 m external diameter, prefabricated and prestressed; they were put down the borehole, then fixed in the soil with cement injection between the outside of the pile and the wall of the borehole. When the lateral injection had hardened, we made a second injection under the point of the pile. The point injection is announced to be patented.

We have two ways of performing load tests on bored piles :

1. The classical way, when we load the pile on its head.

2. When we injected below the base of a pile we observed simultaneously the beginning of the pile uplift and the increase of the pressure of the injection cement grout; when the force of the injection was equal to or bigger than the working load and the pile had not come up, we knew that the safety factor was greater than 2. This technique greatly reduced the settlement under the working load, but we have observed that the ultimate bearing capacity is more or less the same with an injection or without one. But with point-injection we have a permanent settlement of only 1 to 2 mm after an 850 ton testload. Without point-injection, we get permanent settlement of 10 to 20 mm after the 850 ton testload.

We have also performed many other tests on pile groups and horizontal load tests on single piles. Time is too short to speak about the result of our tests, but Prof. Kérisel and I will be publishing the results of this bridge foundation investigation in the near future.

M. A.J. DA COSTA NUNES (Brésil)

Je voudrais discuter le problème de la force portante des fondations sur pieux à partir de résultats d'essais au pénétromètre hollandais et d'essais de charge, en discutant, chemin faisant, quelques travaux présentés à ce Congrès.

En ce qui concerne l'essai au pénétromètre hollandais, le « diepsondering », nous l'avons utilisé plus de dix ans avec de très bons résultats au Brésil.

Nous avons comparé les résultats de l'essai avec le comportement de la fondation en charge, au moyen de mesures de tassement et aussi au moyen d'essais de charge de pieux isolés. Nous sommes arrivés, d'une façon théorique interprétative et aussi selon l'analyse statistique, à la conclusion que les pieux — dont la base n'est pas au-dessus de couches compressibles profondes et est assise sur un terrain qui supporte une charge de pointe du diepsondering de 8 à 10 fois la pression de travail —, se comportent très bien sous le poids de la construction. On a eu des difficultés avec des coefficients plus bas que ça. Si bien qu'une investigation par le calcul dans chaque cas particulier est souhaitable; il faudrait, à notre avis, dans l'état actuel de nos connaissances, établir des coefficients ou bien des courbes d'utilisation des données du pénétromètre comme celles que M. Kérisel nous a montrées, afin de ne pas être dépassés dans la pratique. Cette recommandation s'impose, à notre avis, du fait que le fonçage des pieux modifie en général, d'une façon très appréciable, les paramètres de résistance du sol, en plus du fait que

le comportement d'un groupe de pieux diffère du comportement d'un pieu isolé, non seulement par suite des superpositions de pressions, mais aussi parce que le sol sous le groupe de pieux se trouve modifié, quelquefois d'une façon décisive.

Les études de Meyerhof sur la capacité de charge des pieux à base élargie dans le sable, présentées au Journal of The Soil Mechanics and Foundations Division et au Symposium de Stockholm sur les pieux, ont montré cette influence d'une façon théorique et expérimentale.

Le projet de révision du Code de Boston, en divisant les pieux moulés dans le sol en pieux forés et pieux compactés, l'a reconnu.

Dans notre Congrès, le travail théorique de Nishida 3B/19, a montré que pour un sol sans cohésion le compactage atteint une distance de 8 diamètres autour de la base d'un pieu. Des investigations réalisées au Brésil, en Belgique, Hollande et dans plusieurs autres pays, avec l'utilisation d'essais au pénétromètre avant et après le fonçage de pieux, ont donné des résultats concordants.

Nous présentons, à la Fig. 18, le résultat d'essais réalisés à Amsterdam pour la Nederlandse Franki Maatschappij.

le travail présenté par MM. Folque et Castro, 3B/9, sur un essai horizontal des pieux et qui constitue une contribution importante à l'étude de la résistance des pieux aux efforts horizontaux, comme d'ailleurs importantes sont les six contributions du LNEC au Congrès.

L'étude, étant donné le nombre de pieux considéré, les dimensions record de ces pieux et du fait qu'on a obtenu des résistances horizontales beaucoup plus grandes que prévues, résistances très bien expliquées par les auteurs comme dues à la préconsolidation du sol, est très intéressante. Un facteur important dans ce type d'investigation, est le point de rotation du pieu, dont la profondeur conditionne les pressions du sol et les contraintes dans les matériaux du pieu.

Dans un essai semblable, que nous avons fait au Brésil en collaboration avec M. Grillo (Société Geotecnica S.A.), on a chargé simultanément le pieu avec une charge verticale et une horizontale et on a mesuré la déformation de l'axe du pieu à l'aide d'un trou central aménagé dans le pieu et d'une lunette mesurant le déplacement horizontal de chaque point de l'axe, afin de pouvoir déterminer l'élasticité et le centre de rotation.

Nous présentons, à la Fig. 19, le schéma des dispositifs de chargement horizontal et de mesure des déplacements horizontaux du pieu.

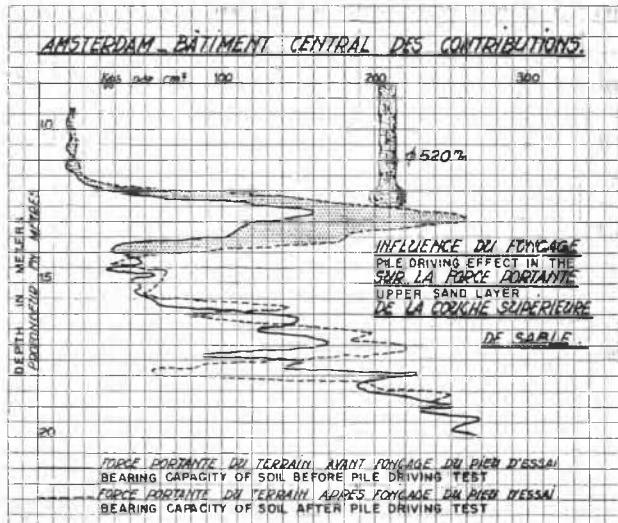


Fig. 18

A notre avis, les conclusions du Dr Széchy, dans son intéressant travail 3B/25 sur le même sujet, sont liées au type de fonçage qu'il a adopté et ne s'appliquent pas aux pieux avec pénétration forcée du béton dans le terrain, lors de la constitution de la base (pieux compactés du nouveau Code de Boston).

Le travail très intéressant de Van Welle, 3B/26 donne des graphiques de l'influence du compactage du sol provoqué par le battage de pieux préfabriqués, analogues à ceux que nous avons déterminé pour les pieux moulés dans le sol; cependant, l'influence dans ce dernier cas est beaucoup plus marquée. En plus, nous avons constaté, au cours de l'exécution de fondations en pieux Méga, donc foncés au vérin hydraulique, que même ce type de pieu produit un compactage appréciable du sol et dans un groupe de pieux on a beaucoup plus de difficultés à foncer les derniers pieux que les premiers.

Cela limite d'une certaine façon, à notre avis, l'étude très complète de M. Bogdanovic 3B/3, dans le cas d'une semelle de plusieurs pieux.

Finalement, je voudrais faire quelques considérations sur

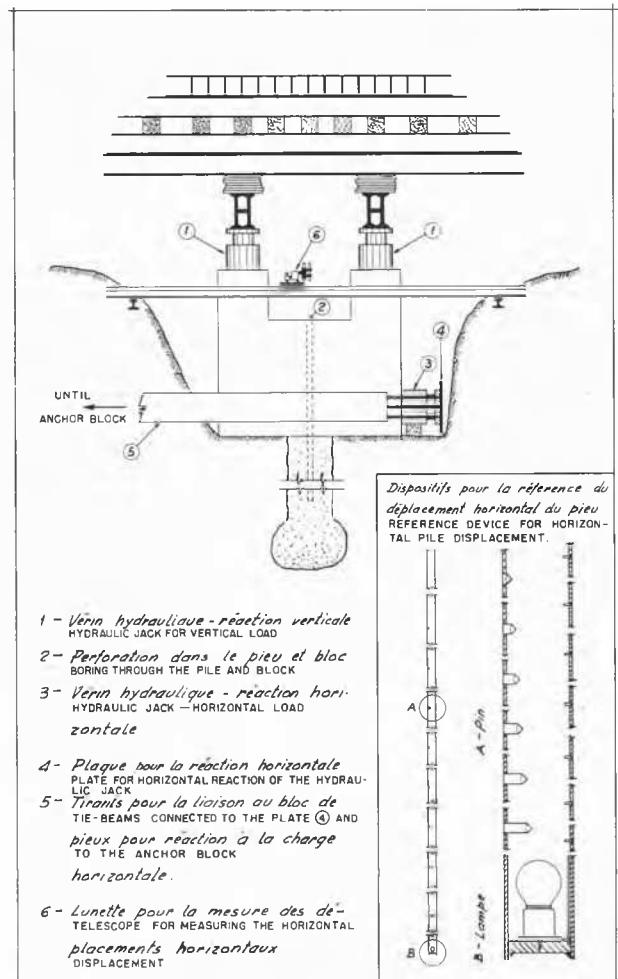


Fig. 19

Le Président :

We come now to point (c) and I ask Prof. Meyerhof to speak.

I should like to make a few remarks on the ultimate bearing capacity of pile groups although I am fully aware that much more information on the behaviour of pile groups in the field is required before this difficult problem in foundation engineering is adequately understood. When considering pile groups one has to distinguish between two types, namely pile groups in which the caps or foundations rest on load-bearing soil (piled foundations) and pile groups with caps clear of the ground or with piles driven through soft strata into dense soil (free standing pile groups).

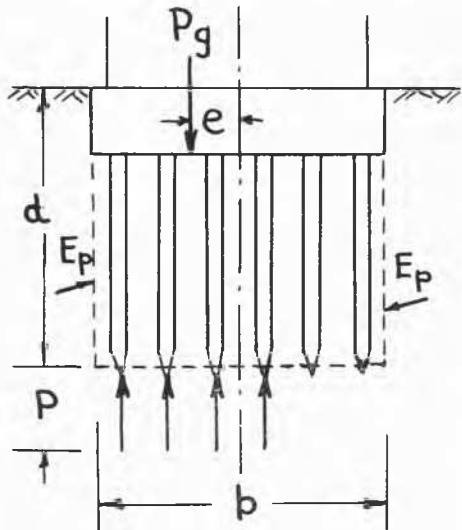


Fig. 20

A piled foundation (Fig. 20) can be considered as a pier foundation with its base at the depth of the pile points, and the total bearing capacity is practically independent of the pile spacing. Although at present only model tests with a central vertical load on piled foundations in clay are available (Whitaker, 1960), loading tests on free standing model pile groups with a close pile spacing in sand (Kezdi 1957 and Stuart et al., 1960) indicate that similar conditions probably obtain generally for piled foundations in uniform soils. For a central load on such foundations with vertical piles the total bearing capacity is the sum of the base resistance of the equivalent pier and the shearing strength of the soil along the perimeter of the group less the weight of the enclosed soil. For eccentric or inclined loading the foundation tilts and the earth pressure on the sides of the group should generally be taken into account. However, if the foundation width is of the same order of magnitude as the pile length, the lateral earth pressure can be neglected; the calculation is then similar to that of a spread foundation in which only compression piles are loaded and tension piles are ignored by using an effective foundation area (Meyerhof, 1953).

The total bearing capacity of a free standing pile group (Fig. 21) is the smaller amount of either the bearing capacity of an equivalent pier foundation or the sum of the bearing capacities of the individual piles. It is found that for a pile spacing of less than about two or three times the pile diameter the individual failure zones in the soil around the piles overlap and thus produce soil arching between the piles leading to pier action of the group. For a greater pile spacing the individual action of the piles governs although the deformation of the soil near the piles has to be considered in estimating the total bearing capacity. As shown by model tests on free standing pile groups in clay with central vertical load (Whit-

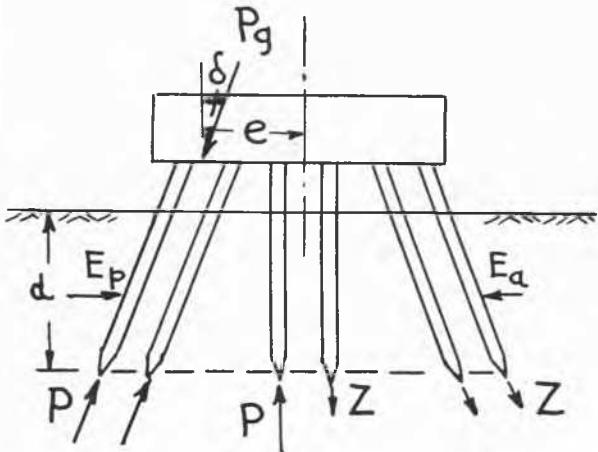


Fig. 21

aker, 1957 and Sowers and Martin, 3B/24) and with eccentric vertical load (Saffery and Tate, 3B/20), the total bearing capacity for a pile spacing of about two pile diameters is only about two-thirds of the full bearing capacity. The latter is only reached at about 7 pile diameters, which can be explained by overlapping of the individual zones of shearing deformation in the soil. Similarly it has been found for free standing pile groups in loose sand (mentioned above) that for a pile spacing of about two pile diameters the total bearing capacity is more than twice the full bearing capacity, which is reached at about 6 or 7 pile diameters. This result is supported by theoretical estimates of the compaction and deformation of loose sand by pile driving as well as an overlapping of the individual soil failure zones for a small pile spacing (Meyerhof, 1959). For eccentric and inclined loads the bearing capacity of free standing pile groups with a small pile spacing can be estimated as for a pier foundation. For a greater pile spacing, however, the total bearing capacity depends both on the compression and tension piles and on the lateral earth pressure for which simple methods of analysis have recently been suggested for groups of vertical and inclined piles (Meyerhof, 1960).

Références

- [1] KEZDI, A. (1957). "Bearing Capacity of Piles and Pile Groups". *Proc. Fourth Int. Conf. Soil Mech.*, London, vol. 2, p. 46.
- [2] MEYERHOF, G.G. (1953). "The Bearing Capacity of Foundations under Eccentric and Inclined Loads". *Proc. Third Int. Conf. Soil Mech.*, Zurich, vol. 1, p. 440.
- [3] (1959). Compaction of Sands and Bearing Capacity of Piles". *Proc. Am. Soc. Civ. Engrs.*, vol. 85, No. SM 6, p. 2295.
- [4] (1960). "The Design of Franki Piles with Special Reference to Groups in Sands". *Proc. Symp. Design of Pile Foundations*, Stockholm, p. 105.
- [5] STUART, J.G., HANNA, T.H. and NAYLOR, A.H. (1960). "Notes on the Behaviour of Model Pile Groups in Sand". *Proc. Symp. Design of Pile Foundations*, Stockholm, p. 97.
- [6] WHITAKER, T. (1957). "Experiments with Model Piles in Groups". *Geotechnique*, vol. 7, p. 147.
"Some Experiments on Model Piled Foundations in Clay". *Proc. Symp. Design of Pile Foundations*, Stockholm, 1960, p. 124.

M. C. DJANOEFF (Grande-Bretagne)

My comments are based on the experience of West's Piling & Construction Company Limited — specialist piling contractors of the United Kingdom.

Dealing with the Question (a). We are daily dealing with a large number of records of bore-holes, which are often supported by mostly dynamic penetrometer tests, and we are faced with the forecast of the carrying capacity of deep piled foundations from the information available.

The penetrometer tests alone, I feel, are sometimes unreliable, mostly because of size, and it would be imprudent to derive the carrying capacity from that information alone — which I trust the question does not suggest. Unless, of course, the ground is comparatively well known, as for instance in Holland.

There is another aspect of approach to this question which is concerned with the altered characteristics of the soil through the treatment of pile driving, aspect of which there is still very rarely any information. There is also disturbance to the piles through driving large numbers at close centres.

The Company has recently carried out the driving of 4 sizes of piles for loads ranging from 40 to 150 tons each, at the Spencer Steelworks, Newport, England. Some 35 000 piles were driven in 9 months and a large number of test loads were done. The piles were founded in Red Marl and averaged some 40 ft. in length.

The West's Shell Pile is in fact a dynamic penetrometer and the efficiency of this apparatus has, on this vast project, been checked by high speed photography, at some 8 000 frames per second, during driving operations, with subsequent projection on to a screen one frame at a time. Hence, velocities, compressions, coefficients of restitutions were actually visually measured or calculated from observations. It proved to be a very reliable method of checking impact phenomena and formulae. There was an opportunity to assess the relation between penetrometer tests and test loads under comparatively uniform conditions. There, the message from some 110 test loads done on the aforesaid project is that smaller diameter piles capable of penetrating deeper to the harder marl gave the better results.

Now, turning to Question (b). I shall not deal with the first part of this question which has been extensively dealt with by others.

Dealing with the second part of this question — the conclusion from comparatively few, only four tests on groups of 8 to 12-100 ton medium sized piles spaced at approximately 3 dia., is that the settlement at working load was 100 per cent greater than that for single piles and 3 times at 50 per cent overload.

Time does not permit me to deal with the other questions, other experiences at home and overseas and many other aspects, such as extraction tests, shoe shapes, heave, its measurements and effects, and ways of overcoming it.

I will only conclude by saying that the two main objects in piled foundations are first to forecast for the designer, who often does not know well enough his allowable limits of differential settlement and secondly to verify the forecast on completion.

More emphasis, I feel, should be brought to bear on the correlation between these two essential elements of a successful scheme.

M. L. BJERRUM (Norvège)

The following is a brief comment on that part of the bearing capacity of piles driven into clay which is due to adhesion developed between the surface of the pile and the surrounding clay.

Loading tests on piles driven into the normally consolidated Norwegian clays indicate that after a few weeks the adhesion approaches or might even exceed the undrained shear strength of the undisturbed clay. The tests show furthermore that

the ratio of adhesion to undrained shear strength is related to the liquidity index of the clay, and the maximum values appear to occur for piles driven in clays having liquidity indices between 0.5 and 1.0 which are typical values for the Scandinavian marine clays. It is thus common practice in Norway to design pile foundations on the basis of the undrained shear strength of the undisturbed clay.

Now, on the other hand, loading tests carried out in England have indicated that the adhesion developed between the tested piles and the surrounding soft clay was considerably smaller than the undrained shear strength of the undisturbed soil and was, under certain conditions, as small as the remoulded shear strength of the clay.

Obviously, for a pile driven into clay, there are a number of factors which influence the magnitude and the rate of increase of bearing capacity. Further progress can be made from a study of a great number of loading tests, and the Norwegian Geotechnical Institute has therefore decided to publish a collection of loading tests on piles in clay. The data will be classified according to the type of pile and the properties of the clay. More than 200 pile tests have already been collected, but we would be happy to receive more test data and to include them in the publication.

M. A. CASAGRANDE (Etats-Unis)

I should like to present a few comments on the *bearing capacity of friction piles in soft clays*, supplementing the important information which Prof. Peck presented in his talk, and also Dr Bjerrum's remarks.

First, I should like to emphasize that agreement between a theoretical bearing capacity computed from the undisturbed shear strength and the actual bearing capacity cannot be expected for overconsolidated clays. Frequently it is assumed on the basis of geological evidence that a stratum of sensitive clay is normally consolidated, whereas one finds later, from consolidation tests on satisfactory samples, that it is substantially overconsolidated. In such instances the actual bearing capacity could not be larger than that based on the strength of the clay after it is reconsolidated to the present overburden pressure, i.e. for an equivalent normally consolidated clay. (Note : A striking example is presented in the ASCE Proceedings Volume 81, Separate No. 657-13, March 1955, in my discussion of Prof. R. B. Peck's paper on Foundation Conditions in the Cuyahoga River Valley.)

My second point concerns the difference in the bearing capacity that one obtains when testing a single pile, with no other piles driven in the vicinity, and the bearing capacity of a pile in the midst of hundreds of piles already driven for a foundation. In the case of a soft, but brittle clay, the undisturbed clay surrounding a single pile would develop effective arching, so that reconsolidation of the zone of remolded clay will cause a drop in the horizontal stress acting against the pile. Thus, the magnitude of earth pressure which would control the skin friction, may remain well below the magnitude of the earth pressure that existed before the pile was driven. Therefore, the bearing capacity would be much smaller than a theoretical value based on the undisturbed strength of the clay. However, the result of such a single load test would not be representative for the bearing capacity of the same pile, after many other piles have been driven in the vicinity. The brittle mass of clay is then thoroughly fissured which, together with the substantial volume displacement of a large number of piles, causes the build-up of horizontal pressures that may reach or even exceed the horizontal earth pressure which existed within the clay mass before any piles were driven. Hence, the skin friction that would build up some time after all piles are driven, could be much greater than the skin friction measured on an isolated test pile.

As third point, I should like to mention the possibility that progressive failure may cause the bearing capacity to be less than if the skin friction could be mobilized simultaneously over the embedded length of the pile. In a number of investigations it was found that most of the test load on a friction pile is transmitted through the upper portion of the pile into the soil. A very small differential strain between the pile and the soil is sufficient to mobilize the skin friction. When conducting a load test to failure in a highly sensitive, brittle clay, it is possible that at the failure load strain in the upper part of the pile has already exceeded the maximum skin friction and that it has dropped to a small fraction due to remolding, while in the lower part it is just fully mobilized. This phenomenon can develop during the relatively fast load application when test-loading a pile. On the other hand, during the normal, very slow increase in loading of a pile in a foundation of a structure, a substantial reduction in failure load due to progressive failure is much less likely to happen. Furthermore, in slow loading the full friction angle corresponding to a fully consolidated condition, and the strength as obtained from S tests, would be mobilized; whereas during load testing the clay may be consolidated only very little under the applied shear stresses, and only the strength as obtained from R tests might apply. For sensitive clays the strength values from S and R tests may differ in the ratio of 2 : 1. The difference in behaviour of wood and steel piles may perhaps also, at least in part, be due to differences in the degree of progressive failure, for the modulus of elasticity of wood is only about one-twentieth of that of steel. When comparing steel and wood piles, there is usually involved also a considerable difference in pile length which would further aggravate differences due to progressive failure.

As fifth point, I should like to comment on the question of standardization of pile load tests. We all realize that the practitioners who are not specialized in soil mechanics, need guidance in the form of standards which should be relatively simple and which should give results that are on the safe side. But I am always afraid of such efforts, because I have seen again and again that standardization in soils engineering has done more harm than good. From the discussions we have heard today, and from my comments, it should be fairly clear that we are dealing with many more variables than is usually realized, and that their effect on bearing capacity is only vaguely known, so that by means of standardization we could easily inhibit progress. Instead of standardization, I would favor dissimilation of the description of pile tests which are considered good examples and representative for various soil conditions and various types of piles. If standards are created, they should not be rigid, but merely a set of minimum requirements, and which will permit the investigator substantial deviations if he desires to make his tests more comprehensive.

M. V. G. BEREZANTZEV (U.R.S.S.)

Modern foundation engineering technique makes possible the construction of foundations not only with a great number of ordinary sized piles but also foundations with a few long piles of large cross-section. In both the above-mentioned cases methods of determining the foundation load bearing capacity are different.

With a great number of piles, located at a distance equal to 3-4 diameters, the whole group of piles, together with the ground between them, acts like a deep foundation which has a considerable load-transmitting area. So great is the stability of the subsoil under such a foundation, that there is no need to check its bearing capacity from this point of view. The foundation load-bearing capacity is determined by the value of the ultimate deformation of the subsoil.

To do this, it is necessary to compute the foundation settlement under the pressure, acting on the level of pile ends.

When there are few piles of large dimensions, placed a great distance apart, the bearing capacity of each pile may be determined from the point of view of soil stability. This statement refers mainly to sands. When a pile is sunk into the sand, a compacted zone is formed around it. In this case, the curve, indicating the dependence between settlement and load, has a characteristic point, corresponding to the load, beyond which the settlement grows more intensively. This load corresponds to the beginning of displacement of considerably developed sliding zones in compacted soil.

The theoretical value of this critical load may be correctly determined with the help of the limit equilibrium theory, as shown in the paper presented to this conference.

M. PIETKOWSKI (Pologne)

Je veux rappeler en quelques mots les journées consacrées à la force portante des pieux organisées ici à Paris en 1952. Avec M. Czarnota-Bojarski j'ai présenté alors un rapport où j'ai proposé quelques coefficients pour prévoir la force portante des pieux selon les conditions des sols. Naturellement, ces coefficients étaient basés sur plusieurs dizaines d'essais de charge. Bien sûr, ces essais n'étaient pas semblables à l'étude faite par l'IRABA; l'étude de l'IRABA est une étude scientifique exceptionnelle et unique dans le monde.

Nous avons trouvé plusieurs coefficients que j'ai essayé ensuite de comparer avec le rapport de MM. Eide, Hutchinson et Landva d'Oslo. J'ai évalué, d'après ces coefficients, la force portante de leur pieu, laquelle, pour le tassement de 5 mm, s'élève à 20,6 tonnes. La force portante réelle, relative au tassement de 5 mm, était après 71 journées de 19 tonnes; après 800 jours la même force portante était de 22 tonnes. La différence de 10 pour cent, en pratique, est très satisfaisante.

Je prends encore le second rapport de MM. Haefeli et Bucher, de Suisse. Il porte sur l'examen d'un pieu Benoto, et dans ce cas je trouve, d'après mes coefficients, la force portante de 258 tonnes, alors que l'observation réelle pour 5 mm de tassement donne 220 tonnes. Cependant, pour 10 mm de tassement, le résultat est très différent : 375 tonnes. Ici, pour 5 mm de tassement la différence atteint 20 pour cent. Je suis convaincu, qu'en pratique, nous ne pourrons pas arriver à une meilleure exactitude.

M. G. F. SOWERS (Etats-Unis)

For some years, soil engineers have accepted the idea that the bearing capacity of a friction pile in clay can be computed as the sum of the end bearing on the pile tip and the termed "skin friction".

However, the assumptions made in the theoretical analyses and the conditions imposed by field and laboratory tests are not always compatible with the real behavior of the soil-pile system. In such cases the computed bearing capacity may differ substantially from the observed capacity, a fact which has been reported in a number of papers to this Conference.

The first uncertainty is the development of plastic equilibrium at the pile tip. The general theories of bearing capacity failure of deep foundations (such as that of Meyerhof, 1951) assume an ideal material in which a zone of plastic equilibrium develops spontaneously and which extends from the tip of the pile downward, outward, upward and then back to pile like an inverted heart.

This leads to a theoretical bearing capacity factor N_c of about 9 which has been confirmed by a limited number of field tests in a small range of soils.

Laboratory tests on model piles made under the writer's direction indicate that appreciably lower N_c values, 5 to 8,

can occur (Paper 3B/24). A study was undertaken to determine the character of the zone of plasticity for this condition. A bentonite clay was placed in layers 1/2 in. thick, with every other layer darkened by carbon. A model pile 2 in. (5 cm) in diameter was forced into the soil 12 diameters after which the model was cut in half longitudinally. There was no evidence of the development of the heart-shaped shear zone. There was a well developed cone of shear beneath the pile, a broad zone of gentle upward bulging around the pile shaft, and sharp downward deflection immediately adjacent to the pile as shown in Fig. 22. This is explained by the large strains required for failure by the soil employed in these tests whereas the theory of general shear failure is based on the assumption of an ideally plastic material with very small strains at failure. The lack of general shear was responsible for the lower value of N_c .

A second complicating factor is the development of skin friction. It is customarily assumed to be uniformly distributed along the pile surface in a uniform soil. The author's tests reported to this conference (Fig. 1, Paper 3B/24) show that the skin friction is greatest near the pile tip and becomes less toward the ground surface. It becomes more nearly uniform as failure is approached but is never entirely uniform. The authors explained this by the non-uniform distribution of strains in a homogeneous elastic medium where the interior of the mass is more rigid than the surface.

A third uncertainty lies in the addition of the computed end bearing to the computed skin friction to obtain the total bearing capacity. The mechanical processes of skin friction



Fig. 22

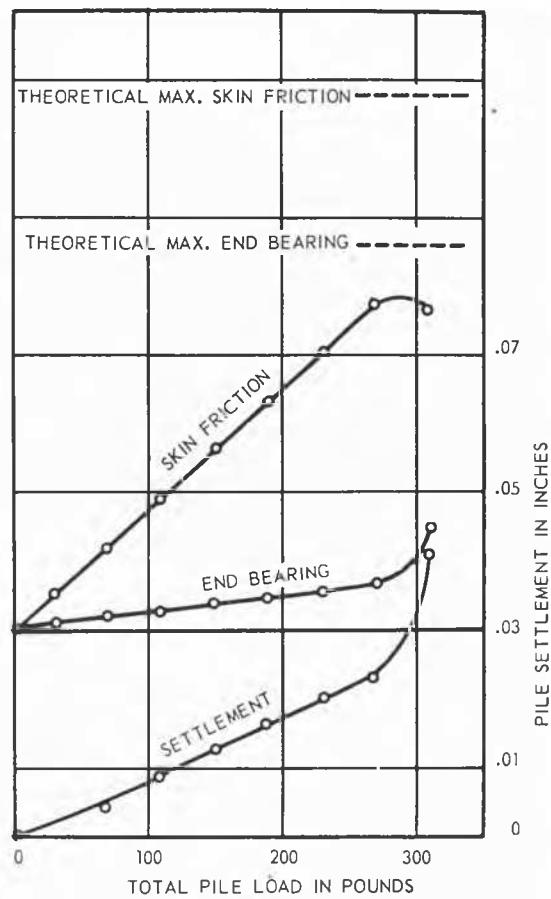
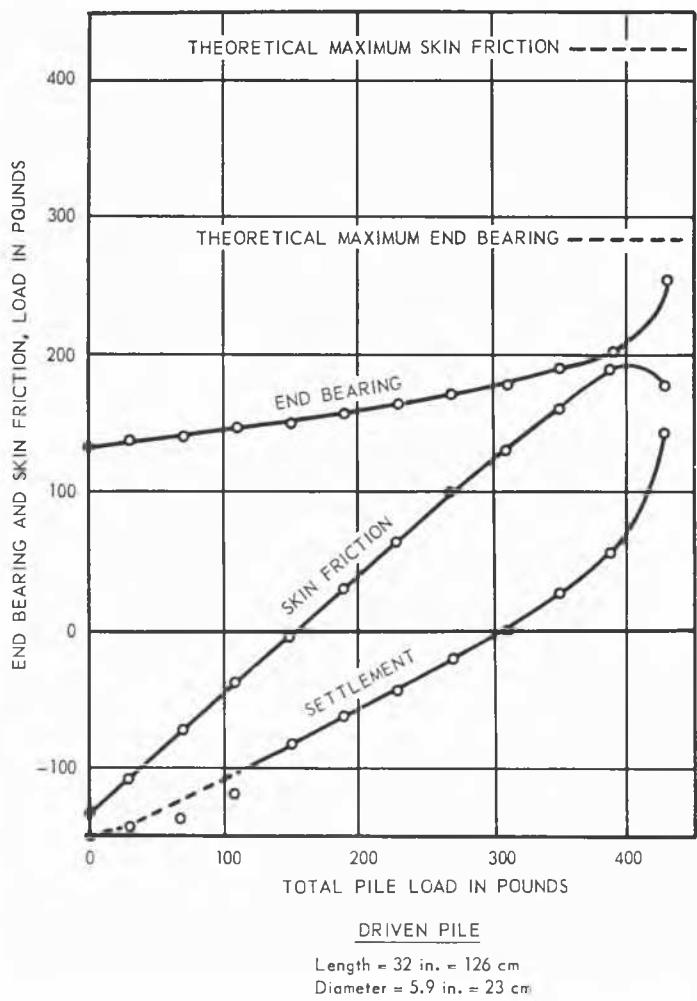


Fig. 23

failure and end bearing failure are not similar; therefore, it should not be expected that these two different types of failure should occur simultaneously. Tests of a 6 in. diameter model pile approximately 30 in. long in soft clay bear out the importance of non-simultaneous mobilization of shear.

Fig. 23 shows the end bearing and total skin friction (total pile load minus end bearing) of a pile forced or driven into the soil. After driving and with no applied load, end bearing equal to about half the ultimate end bearing remained. This can be attributed to the elastic rebound of the soil that was forced aside by the cone of shear during driving. The upward force on the pile tip was resisted by negative skin friction. During loading both skin friction and end bearing increased linearly. Failure, evidenced by excessive deflection, began with end bearing failure at a load equivalent to $N_c = 8$. Skin friction by this time had reached only 45 per cent of its theoretical maximum value. This is comparable to the "Reduction Coefficients" reputed in Paper 3B/28. When the same pile was installed by drilling the behavior was different. Both end bearing and skin friction began at zero with no load. Failure (rapidly increasing deflection) took place when the skin friction reached 51 per cent of its theoretical ultimate. The end bearing had reached only 29 per cent of its theoretical ultimate which is equivalent to $N_c = \text{about } 3$. In other words, starting with no residual strain the skin friction reached the point of failure well before the end bearing. This can be explained by the greater deflection required to mobilize end bearing than skin friction.

Référence

- [1] G. G. MEYERHOF, (1951). 'Ultimate Bearing Capacity of Foundation, *Geotechnique*, Vol. 2, p. 301.

M. O. EIDE (Norvège)

Concerning friction piles in clay I wish to comment on Prof. Geuze's remark about the effect of the thickness of the remoulded zone on the earth pressure built up against the pile.

The construction of the Oslo subway provided us with the possibility of driving sheet piles instrumented with earth-pressure cells and pore-pressure cells. The remoulded zone along the piles would be only 1 or 2 cm thick. At one job there was a time interval between pile driving and excavation during which excess pore pressures could dissipate and here we were able to measure directly the effective pressure built up against the piling and which corresponded to a k -value of about 0.8. In this case the soil profile was rather thin.

In another case which has been reported in a paper to this conference, we made long-term loading tests on a cylindrical timber pile over a period of two years; these tests were purposely done slowly to provide drained conditions. Here the thickness of the remoulded zone was of the order of 20 cm.

If we now take the measured long-term bearing capacity of the pile and if we use the ϕ' -value for undisturbed clay, we obtain a k -value of 0.5. If there should be a different ϕ' -value for remoulded soil it would be expected to be higher than that for undisturbed clay and this would lead to an even lower value of k .

I have taken these two cases as an indication that the thickness of the remoulded zone has some influence on the earth pressure built up against a pile.

In the near future we will drive a long steel pile having a large cross section for the purpose of measuring negative skin friction built up over a long period of time. This pile will also be instrumented with earth-pressure and pore-pressure gauges. Because the remoulded zone around this

pile will be greater than for those cases mentioned previously, we will also take this opportunity to investigate the effect of the thickness of the remoulded zone on the effective earth pressure against the pile.

M. C. SZÉCHY (Hongrie)

I am making reference to our Chairman, when he stated that it is largely desirable to arrive at a certain uniformisation in the evaluation of pile test loadings, and I shall show you a procedure which might lead to a univocal determination of the safe bearing capacity of piles.

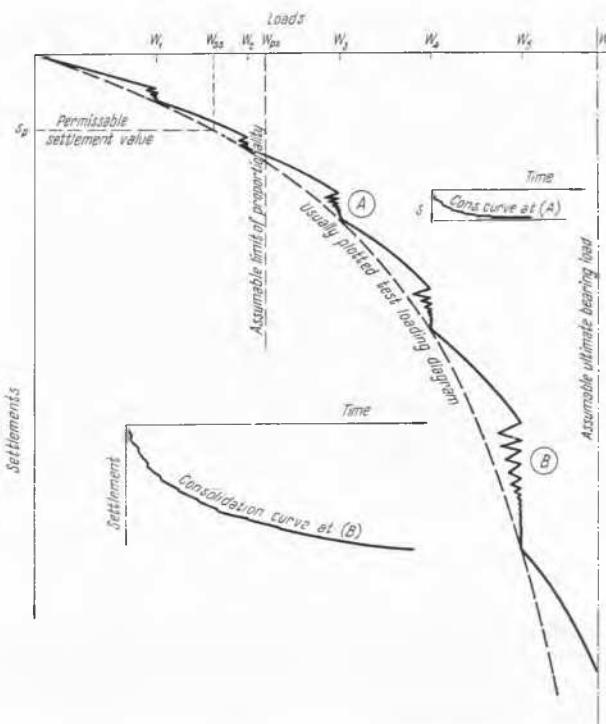


Fig. 24

If we are looking upon the usual shape of a load settlement curve of a pile test loading (Fig. 24) we can see that there is a lot of ambiguity when plotting the curve itself. Partly one cannot determine safely in cohesive soils the consolidation-settlement, and partly one cannot keep the load proper constant during the period of settlement only by repeated adjustment of the piston of the hydraulic-jack if loading is performed against an anchorage and not in the form of a kentledge placed on a platform. Each newer readjusting impulse on the piston will involve a little additional settlement. These are the main reasons why we cannot fix exactly the final settlement at each loading state and consequently the plotting of the load settlement curve will be also uncertain. Still more uncertain however is the evaluation of the obtained curve.

This is usually effected in one of the following three ways :

1. Determination of the safe bearing load [W_{ss}] as a function of the permissible settlement [S_p] of the building.
2. Find out on the curve some limit of proportionality (mostly in cohesive soils) and determine the safe load [W_p] by dividing it with an appropriate safety factor [n_p].
3. Define the ultimate load as that at which the rate of settlement continues undiminished without further increment of load (unless this rate is so slow as to indicate that settlement may be due to consolidation) and determinate the

safe load by dividing it (W_u) with another appropriate safety factor (n_u).

The difficulty is now that it is a rather rare exception when either the limit of proportionality or the ultimate load appear distinctly on the curve. The heterogeneity of soil stratification on one hand and increase of load bearing capacity bound together with progressive settlement and the changing load-distribution between mantle friction and toe resistance on the other hand, render this problem especially difficult.

Based on laboratory and field observations the author has developed a procedure for the direct determination of the safe pile-load as follows :

The pile transmits its load upon the soil and therefore we have to consider first the deformation process of the soil proper under progressive loading quite up to failure (Fig. 25).

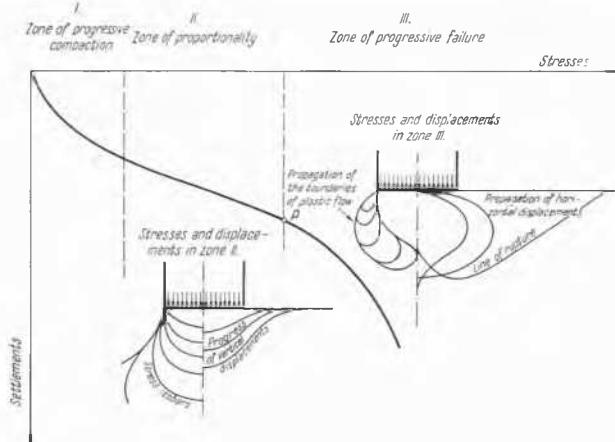


Fig. 25

At the beginning the soil particles will undergo a progressive compaction (Zone I) and vertical displacement will be preponderant. In this phase the relative density of soil will be gradually reduced and its elastic properties — and as a consequence also its load-carrying capacity — gradually improved (Zone II). Consequently the residual part in the total amount of deformation (settlement) will be gradually decreased and the elastic part increased. But after a certain limit, with the further increase of loading, plastic zones gradually develop on the edges and an irreversible lateral displacement of soil particles will set in again until at last these plastic zones will conflux under the loading plate and — accompanied with rapidly increasing residual deformations — will give rise to the development of a continuous sliding surface along which bearing failure will set in.

It is obvious from this figure, that the limit of the safe bearing load lies at the boundary of zones II and III (Point P). Thus it may be concluded, that this limit may be characterized by the phenomenon that the increments of plastic (residual) deformations get bigger from here again than those of elastic deformations. With regard to this the stress-strain relations of the typical load-settlement curve of soils could be reflected by the change in the ratio of elastic deformation increments Δs_e to residual deformation increments Δs_r , i.e. by

the ratio : $\epsilon = \frac{\Delta s_e}{\Delta s_r}$. In the first phase of loading it must

become always bigger because the elastic part of the deformation is increasing as compared to the residual part, owing to the gradual increase of soil bearing strength brought about by the compacting effect of vertical loading and displacements. But when the boundary between zones II and III is attained this tendency will be changed. The boundary and point P

must be thus represented by a maximum of the ϵ values which may be obtained in a diagram by laying a horizontal tangent to the plotted ϵ curve (Fig. 26 b).

For the separation of the two components : residual settlement and elastic settlement, the best way is to apply alternate loading and load relief cycles. The elastic set (S_{e1}) will appear here as the heave measured under the load relief-cycle and the plastic or residual part (S_{r1}) as the difference between total settlement (S_1) and the elastic part. If there were no other influencing factors, disturbing the evaluation of surface load-test on soils (dimensions, shape, etc.) this procedure might be generally used for the evaluation of any test-loading of soils. But it may be used with proper accuracy for the evaluation of bearing tests on piles, where these disturbing factors are absent, and dimensions and shape of test specimens are the same as that of the real structure.

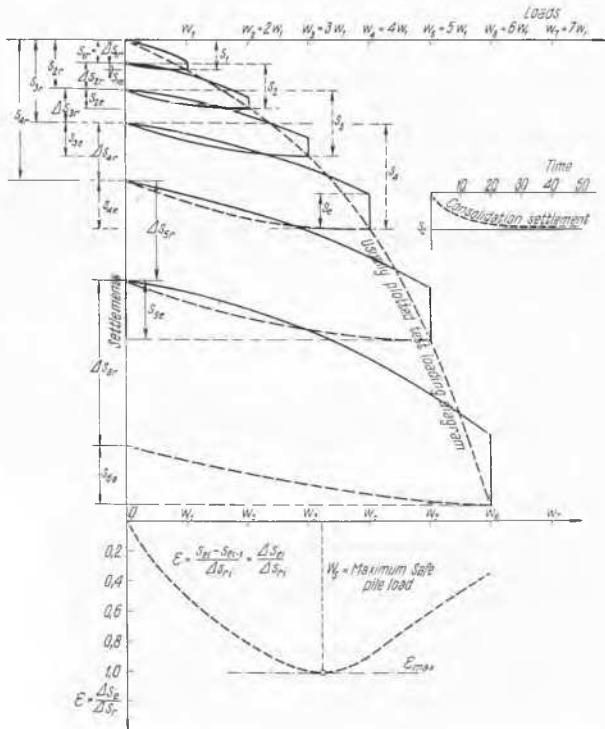


Fig. 26

In conclusion Fig. 26 shows the way to evaluate the load-settlement curve of a pile according to these principles.

The test load should be imposed by equal increments. The first load (W_1) must be applied and the settlement (S_1) measured (including the determination of consolidation settlement by plotting a time-settlement curve at each loading stage). Then the elastic rebound (S_{1e}) must be measured under the following relief cycle. By this we get also the residual settlement as $S_{1r} = S_1 - S_{1e}$. The ϵ_1 value may be computed as $\epsilon_1 = \frac{S_{1e} - 0}{S_{1r}} = \frac{\Delta S_{1e}}{\Delta S_{1r}}$. Then the loading must be raised

up to the next loading stage (W_2) and after the determination of the total settlement (S_2) again a total load relief applied, the total elastic rebound (S_{2e}) measured and the respective residual settlement gained as $\Delta S_{2r} = S_2 - S_{2e}$, or $S_{2r} - S_{1r}$. The differential ϵ_2 value, appertaining to the load increment $W_2 - W_1$ may be obtained now as

$$\epsilon_2 = \frac{S_{2e} - S_{1e}}{S_{2r} - S_{1r}} = \frac{\Delta S_{2e}}{\Delta S_{2r}}$$

Proceeding in this way we can determine all ε values until we see that they reveal a definitely diminishing tendency. Now drawing the horizontal tangent to the plotted ε curve we can determine the *appertaining safe bearing load* : W_s .

It is to be seen, that the ε values were determined by repeated full loading and full relief, although the direct determination of the change of elastic and residual Δ settlement values between the single loading stages might have been more accurate. This would however be not feasible practically owing to the relatively small differences to be measured, to the inexactitude of test mechanism and to the uncertain behaviour of soils under relatively small load relief decrements.

Beyond the distinct and univocal determination of the safe pile bearing load another advantage of this procedure is that we must not apply loads to obtain the ultimate bearing capacity, but when it is obvious that the ε values are beyond their maximum and show a definite decreasing tendency, then no further load increment should be imposed, which means a considerable saving in time and cost, considering the ever increasing duration time of consolidation settlement.

Le Président :

I think, gentlemen, that was a very fruitful discussion. We are all sorry that we cannot hear more speakers, but the time is stronger than ourselves, and I personally am very sorry that I had to shorten the time of some speakers.

I would like to thank all the speakers and all who intended to speak this morning for their very interesting contributions to the topics of the day, and, last but not least, all of you for the interest you have shown concerning the subjects treated this morning.

La Séance fut levée à 13 h. 10

Interventions écrites / Written Contributions

M. H.K.S.P. BEGEMANN (Hollande)

Dr Menzenbach has compared statistically the results of pile-loading tests with the predicted ultimate pile point loads from the results of static penetration tests. For this purpose he used the calculation method of Messrs Van der Veen and Boersma (Proceedings 1957).

If one considers these results, compiled on pages 103 and 104, carefully and compares them with the corresponding complete penetration-graphs, the following points come to mind :

(a) In about 50 per cent of the cases dealt with, the calculated point resistance exceeds the ultimate resistance as measured during the load test; in several cases by even more than 100 per cent.

(b) These large deviations correspond with typical shapes of the penetration-graphs as great alternations of the cone resistance below the pile points to a depth of ± 4 times the pile-diameter.

With regard to this I will draw attention to the results of the pile-sounding of Plantema (Proceedings 1948), also used by Menzenbach (page 103). This test concerns the results of *one* penetration test and *one* pile only. Here the penetration-graph shows in the sandlayer a very gradual increase and decrease of the penetration resistance. As soon as the pile point is embedded in the sand layer to a sufficient depth (15.4 m — G.L.) the calculated and measured values are in agreement. However, this is also the case if the penetration resistance at pile point level is compared with the ultimate pile point resistance at the concerned depth.

Such a shape for a penetration graph, however, is rather exceptional for Holland.

(c) The shapes of the penetration-graphs for which the calculation method of Van der Veen gives far too high cal-

culated pile point bearing capacities, may easily be present under one whole building or structure.

In that case the safety-factor for all these piles may drop easily to values equal to or less than one and failure of the structure will occur.

This shows that, when using the recommendations of Menzenbach with respect to the safety factors in connection with the calculation method of Van der Veen and Boersma the probability of occurrence of failure for a structure can be very much higher than 5 per cent.

This is because the Van der Veen method evidently cannot be used for all types of shapes of penetration-graphs.

(d) It is my opinion that first the reliability of a calculation method should be checked by determining the deviations for several typical shapes of penetration-graphs.

These deviations should be about the same for all types of penetration-graphs, and as small as possible. But then use should be made of fully reliable test data; the contribution of pile friction to the ultimate pile resistance should be measured in one way or another and not calculated by means of an unproved theory. The penetration test should be carried out on the exact place of the pile and before pile driving (it is a tricky business to use the results of a penetration test carried out close to a driven pile).

(e) Another point Menzenbach has not indicated is the possibility of deviations from the test results at one building site in between the test points.

I am convinced that further studies along the line indicated by Menzenbach, and taking into account the remarks mentioned before, will lead to a better judgment of a pile-bearing calculation and determination of the safety-factor.

M. BUISSON (France)

Communication 3B/12 de M. Kérisel

Les réseaux de courbes — enfoncement — pression — obtenues par M. Kérisel en un milieu sableux serré ou demi serré et résument les résultats obtenus sur des pénétromètres, des pieux et des piles caissons de différents diamètres (communication 3/B, Fig. 9 et 10) permettent de mettre en évidence la forme des courbes joignant, sur les courbes précédentes, les points de *même profondeur relative*, c'est-à-dire ceux pour lesquels le rapport $\frac{\text{profondeur}}{\text{diamètre}}$ est le même. Ce rapport $\frac{\text{profondeur}}{\text{diamètre}}$ est le même que le rapport $\frac{\text{pression du terrain}}{\text{densité} \times \text{diamètre}}$.

On constate (Fig. 27, établie au moyen de la Fig. 9 de M. Kérisel) que ces courbes présentent la propriété remarquable de faire apparaître que, quelle que soit la profondeur relative, la *pression de rupture sous la base passe par un maximum lorsque le diamètre est voisin de 30 cm*.

La Fig. 28 montre avec plus de netteté que, pour une *même pression de rupture*, le minimum de profondeur relative est atteint, lorsque le diamètre est de 30 à 40 cm.

Il en résulte donc que, à un rapport donné, on peut faire correspondre deux diamètres de pieux ou de pénétromètres ayant la même résistance de pointe aux deux profondeurs relatives correspondantes.

Cependant, il arrive le plus souvent que les pieux atteignent des profondeurs relatives très supérieures à celles des courbes données par M. Kérisel. Leur prolongation permettrait d'obtenir des courbes valables pour des rapports $\frac{\text{profondeur}}{\text{diamètre}}$ plus élevés et plus voisins de la pratique.

On disposerait alors d'un réseau de courbes pouvant permettre l'analyse plus poussée du phénomène. On pourrait peut-être ainsi transformer les forces portantes, calculées

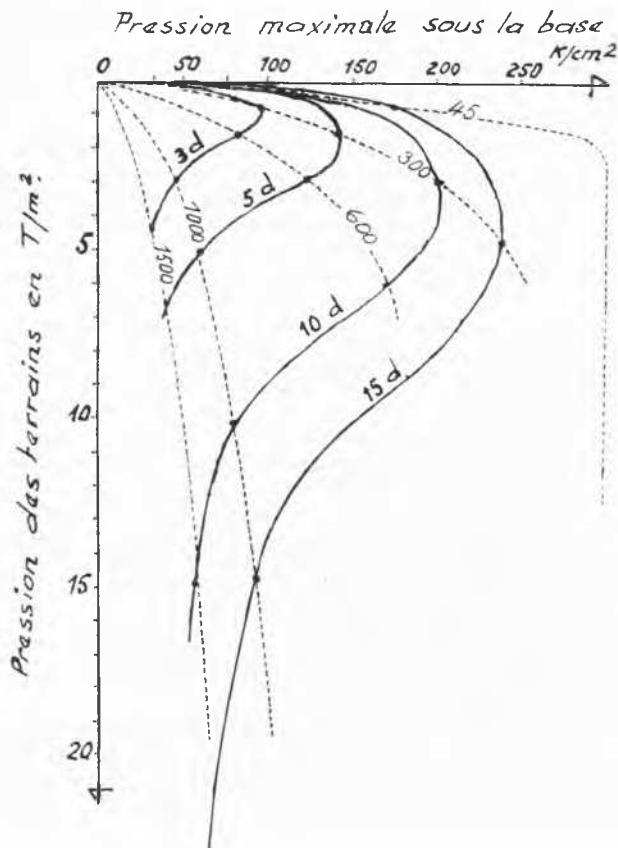


Fig. 27 (D'après la figure 9 de la communication de M. Kérisel)
Courbes d'égale profondeur relative.

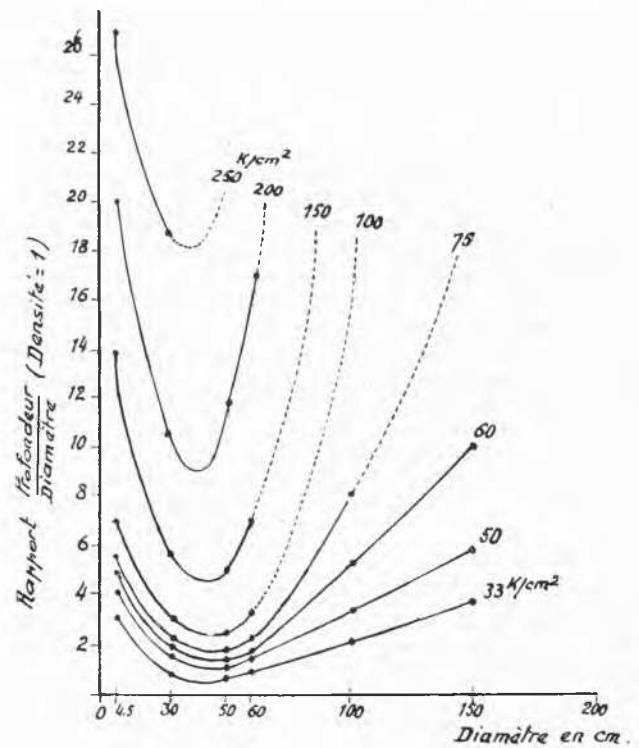


Fig. 28 (D'après la figure 9 de la communication de M. Kérisel)
Courbes isobares.

par simple proportionnalité au diamètre et à la profondeur, en partant de valeurs adaptées de N_y et de N_q . Toute la théorie de la rupture est donc à reprendre entièrement.

Bien entendu, ce travail de prospection pourrait être effectué également dans les sols de moindre densité (Fig. 10 et 11 de M. Kérisel) et, par conséquent, de frottement inférieur, ce que M. Kérisel a amorcé.

On peut rapprocher la forme des courbes de la Fig. 28 de celles obtenues par Kögler et Scheidig, au cours d'essais superficiels. Ces auteurs avaient montré que le tassement sous une contrainte donnée, passait par un minimum qui avait lieu lorsque le diamètre était de 25 cm environ. L'expérimentation employée avait été fort discutée à l'époque (1930). Cependant, il semble bien que, depuis cette époque, des résultats équivalents aient été observés par d'autres expérimentateurs.

Remarquons que la valeur du minimum de tassement (pour 25 cm de diamètre) est transformée ici en minimum du rapport profondeur / diamètre des courbes équibares qui tiennent compte à la fois de N_y et N_q . Il est possible que la valeur du diamètre critique change en fonction de la densité du sol; il semble qu'il en soit ainsi d'après la Fig. 11 de M. Kérisel.

On remarquera enfin que les courbes isobares de la Fig. 28 présentent une certaine symétrie et paraissent se déduire les unes des autres par translation verticale lorsque la pression est supérieure à 100 kg/cm² environ, alors que les courbes inférieures s'ouvrent vers la droite au fur et à mesure que la pression limite diminue. La courbe de 100 kg/cm² présente un minimum dont la profondeur relative est de l'ordre de 2,2 lorsque la densité du sol est de 1. Cette profondeur relative diminue lorsque la densité croît. Or, ce minimum de pro-

fondeur relative doit être celui que M. Terzaghi désignait comme profondeur critique dès 1925 dans «Erbaumechanik». On peut constater que cette profondeur critique diminue rapidement lorsque la densité augmente puisque la Fig. 27 montre que lorsque la densité double, les courbes 5d prennent la place des anciennes courbes 10d.

Au point de vue pratique, des graphiques du genre de ceux de la Fig. 28 pourraient être facilement utilisés si l'on effectuait des essais assez poussés sur un nombre suffisant de sols de nature et de densité différentes.

Répartition des densités au moment de l'équilibre limite en milieu serré

Là encore, les essais devraient être poursuivis et notamment étendus à des rapports de profondeur relative plus importants. Sans que l'on puisse en tirer des conclusions certaines, le fait constaté maintes fois sur les chantiers de pieux que le soulèvement des pieux déjà battus se fasse sentir à une distance égale à la moitié de la fiche du dernier pieu battu, en milieu serré, semble prouver que les déplacements, devant accompagner des changements de densité, se font très vraisemblablement sentir dans un cône ayant la pointe du pieu comme sommet et de rayon égal à la moitié de la fiche. L'examen de la figure 15 de M. Kérisel, montre que, lorsqu'on s'enfonce de 20 cm, le rayon de la surface soulevée augmente de 30 cm en moyenne. Cependant, il s'agit là d'un phénomène constaté pour une faible profondeur, puisque l'essai n'a été poursuivi que jusqu'à une fiche de 1,046 m ce qui est tout à fait insuffisant pour un pieu de 320 mm de diamètre.

La poursuite de ces mesures serait là encore extrêmement

instructive, si on les étendait à une profondeur relative voisine de celles effectivement obtenues dans la réalité, dans les fondations profondes.

Enfin, l'étude fort importante et intéressante de M. Kérisel devrait être complétée par celle de la pénétration de couches de densité et de consistances différentes. Sans doute, les puits de Maracaibo entrent-ils déjà dans ce cas, mais n'est-ce pas en raison de la faible résistance du sol d'appui?

Conclusions

Il est hautement souhaitable que des moyens soient fournis à M. Kérisel pour poursuivre et compléter ses importants essais.

De toutes façons, les essais actuels font apparaître le fait important que les diamètres compris entre 30 et 40 cm semblent être les plus économiques, *dans les sables compacts et moyennement serrés* puisque ces diamètres permettent de supporter les plus fortes contraintes.

Les courbes isobares de la Fig. 28 permettent d'ailleurs de répondre aux questions posées à la discussion de la section 3/B en ce qui concerne les profondeurs à atteindre par un pieu de diamètre donné pour qu'il ait la même contrainte de rupture que celle du pénétromètre dans la couche d'assise. Or ces courbes ne peuvent, avec les données actuelles, fournir de réponse dans beaucoup de cas intéressants au point de vue pratique.

Il y a le plus grand intérêt à poursuivre ces essais.

M. W. G. K. FLEMING (Grande Bretagne)

I wish to refer in particular to some work presented to the conference, concerning the interference of foundations in groups, namely paper 3B/23 by Hanna and Stuart.

In the paper reference has been made to some work by myself on pile groups in cohesionless materials, prior to 1958, and reported in a thesis to the Queen's University of Belfast of that date.

In these experiments, model piles of 3/8" and 5/8" diameter were tested in groups of up to 25 piles, in dense and loose sand conditions.

Although it is recognised that the results obtained at that stage were subject to some distortion, due to the limits of the apparatus then available, improved larger apparatus was prepared by Dr Hanna, who has since obtained many interesting results, using the same techniques. The results of his work are also available in a thesis to the Queen's University of 1960. I wish merely to refer briefly to some of the effects which have been observed.

In dense sands it was found that at close pile spacings, the piles of a group behaved as a block foundation, and in the circumstances considerable increases in the bearing efficiency were obtained. These increases persisted to slightly greater spacings than those necessary to give block behaviour, and were dependent upon the constraints provided to prevent lateral movement of the points of the piles. Such constraints may of course be provided by the natural passive pressure of the soil, or by other artificial means.

Likewise, in loose sand, considerable increase in efficiency of bearing was recorded, but in this case the natural forces provided by the soil to prevent spreading of the pile points, were more effective.

In considering the theoretical implications of this behaviour at that time, I found it necessary to return to the simpler cases of surface and deep strip foundations, and the mechanisms, upon which the paper of Stuart and Hanna to this conference is based, became evident.

Of course at that time experimental results were not available to justify reasonably the mathematical application of the method, but since then the calculations and experiments

performed by Dr Hanna suggest that in principle it is substantially correct.

In attempting to extend the theory to piles in groups, a number of difficulties arise. There is no exact method by which the slip line fields for circular isolated bases may be determined and such approximate methods as do exist, for example that of Prof. Meyerhof, involve the selection of various parameters which in fact are liable to error, and which may also vary in accordance with the process of driving or placing a group. It was therefore not possible to give more than a qualitative explanation of the observed behaviour of the pile groups by the mechanisms proposed.

It may perhaps be worth mentioning some of the points which could be of importance in practical application to foundation design.

For example, it appears likely that the later driven piles in groups have better load/settlement characteristics than those first driven, and therefore when groups are rigidly capped and loaded, some individual piles may bear considerably more load than others. This could lead to unexpectedly high stresses in the pile caps.

Also, with regard to surface foundations, it is not yet known at what stage in the process of loading, the eccentric and inclined base loads begin to develop. If this takes place in the working load range, the columns above closely spaced bases could be subject to bending moments not normally considered.

These problems may not in fact cause much difficulty but, in view of the general trend towards less conservative design of foundations on cohesionless soil, and the increased loads resulting from higher buildings, they are worthy of attention.

M. R. HAEFELI (Suisse)

La discussion que j'ai suivie avec le plus grand intérêt ne laisse pas de doutes que davantage de mesures et d'observations systématiques sont nécessaires pour mieux comprendre et prévoir les comportements des différentes sortes de pieux dans les divers types de sols. Dans ce but il me semble indispensable que des recommandations exactes soient établies en indiquant un certain minimum d'observations et de données, qui sont nécessaires pour pouvoir juger et interpréter le comportement des pieux observés. Ceci peut être fait sans aller jusqu'à l'extrême d'une « standardisation » dont les dangers ont été savamment soulignés par A. Casagrande. C'est la condition *sine qua non* pour obtenir un matériel d'observation assez homogène et valable qui pourrait faire l'objet d'un livre, comme le suggère l'Institut Géotechnique de Norvège (Bjerrum).

En ce qui concerne la nature des observations, je me permets d'attirer votre attention sur deux éléments qui jusqu'à maintenant ont été fortement négligés. En ignorant ces deux éléments, il est pratiquement impossible d'interpréter le tassement des pieux d'une manière approfondie.

Le premier point se réfère à la déformation du pieu lui-même. La connaissance de cette déformation propre du pieu, soit indirectement à base d'essais des matériaux qui forment le pieu, soit à base de mesures directes, est absolument nécessaire pour analyser la relation changeante entre le frottement latéral d'une part et la charge de la pointe d'autre part (en % de la charge totale). Un exemple numérique sur une telle analyse qui me paraît fondamental pour pouvoir juger du comportement soit d'un pieu isolé, soit d'un groupe de pieux, est donné à la page 70 du Vol. II des Comptes Rendus de ce Congrès (Rapport 3B/11, Fig. 7).

Un deuxième point important se réfère à l'installation et au procédé appliqués pour charger le pieu d'essai. Trop souvent des essais de pieux très coûteux sont exécutés de manière que la courbe de tassement est partiellement faussée par

l'influence de la friction négative provoquée par les appuis latéraux de la charge, avant que celle-ci s'appuie sur le pieu. Pour contrôler cette influence, il est indispensable de déterminer le tassement non seulement du pieu, mais aussi des appuis latéraux sous la charge. Les petits tassements observés pour les faibles charges du pieu (qui ne tiennent pas compte de la friction négative mentionnée) sont souvent dus au fait que l'augmentation progressive de la charge visible du pieu va parallèlement à une diminution de la friction négative. Il s'ensuit que la charge réelle du pieu progresse beaucoup plus lentement que la charge visible, qui figure dans le diagramme de tassement. Pour éviter des complications de ce genre, il est préférable d'appliquer des systèmes de charges qui rendent impossible la formation de frictions négatives dues à des charges voisines.

M. T. H. HANNA (Grande Bretagne)

I wish to make a few comments regarding Paper 3B/23 by Mr J. G. Stuart and myself. With reference to Fig. 7 in this paper, the spacing values should read twice the values indicated. This error was overlooked during the checking of the final draft.

Concerning the basic theory on which the calculations are based, it is apparent that the original ideas suggested by Fleming (1958) are correct and only slight modifications were necessary to enable the calculations to be performed. An examination of Fig. 9 and 10 shows a considerable discrepancy between experiment and theory. This is caused mainly by the restriction of the centre of the logarithmic spiral failure surface to the corner of the foundation. Such an approximation is justified only for deep foundations which derive the greater portion of their bearing capacity from soil surcharge. If the spiral centre be unrestricted and a minimum value calculation method used then good agreement will be found for the shallow foundation case. Unfortunately the volume of computation work is greatly increased and it was not found possible to include this refinement in the calculations.

This work is being extended to the case of any number of parallel strips. Complications arise whenever a third strip is introduced (Hanna 1960) but it appears that calculations may be performed with some confidence if a modified failure pattern is used.

These calculations and experiments demonstrate beyond doubt that a considerable portion of the increase in group foundation loads in sands is due to the interaction and distortion of the failure zones around the individual foundations in the group. The pattern of behaviour displayed in these tests has also been found with model pile group tests and although there are detailed modifications of the above trends due to the greater importance of some variables in the case of piles, these strip tests have given a very clear insight into the basic mechanics of foundation grouping in sands.

A detailed account of these pile group results is in preparation with a view to publication.

Références

- [1] FLEMING, W.G.K. (1958). "The Bearing Capacity of Pile Groups". *Ph.D. Thesis*, Queen's University, Belfast.
- [2] HANNA, T.H. (1960). The Behaviour of Pile Groups in Sand. *Ph.D. Thesis*, Queen's University, Belfast.

M. E. P. KHALIZEV (U.R.S.S.)

The modern technique of sinking pneumatic caissons and open caissons into water-saturated soils provides control of the value of hydrodynamic pressure in the zone of the cutting

edge for ensuring the highest possible difference between the non-reduced head near the caisson and the head inside. However, increase of the difference in heads is expedient only up to a certain limit which, when exceeded, may cause inadvertable large water inflow or may lead to violation of soil stability and to influx of the soil into the caisson. For finding this limit, investigations were performed on the problems of water inflow and soil influx into caissons, the contents and results being briefly described below.

The analysis of *in situ* observations of water inflow into caissons verified the correctness of the premises of Forchheimer-Kamensky on the insignificant influence, which is caused by the modification of the distant limits of the filtration zone, on the inflow of water into wells with open bottoms. This permitted the approximate solution of the general problem of water inflow into round pneumatic caissons and open caissons, studying only the two most typical cases : the case of pressure filtration into a strata of finite thickness and the case of sinking floating caissons into strata of finite thickness.

The problem of the value of inflow was investigated and also its variations with increase of sinking. Investigations were performed by the EHDA method with a three-dimensional model in the form of a bath of organic glass with horizontal dimensions of 600×600 mm. The field of filtration was modelled with a cylindrical sector of electrolyte with an angle of 10° , height and radius of the sector corresponding with the dimensions of the bath. The caisson was modelled by a mobile cylindrical sector with the same central angle of 10° and a radius of 50 mm (Fig. 29).



Fig. 29 General view of experimental installation.

Experimental determination of inflow into shore and floating caissons was performed with the corresponding position of the electrode-model of the feeding field and with different positions of the electrode-model of the bottom of the caisson (Fig. 30) for each of these two cases.

The first aim of the experiments was determination of the value of coefficient A in the formula $Q = AkrS$, where Q — water inflow, m^3/hr ; K — coefficient of permeability, m/hr ; r — radius of caisson, m ; S — difference between non-reduced head near caisson and head inside, m . Coefficient A was determined empirically, on the basis of values of the difference of potentials of the electrode-models, amperage and specific resistance of the electrolyte.

The results of experiments (Fig. 31) allowed one approximately to estimate the maximum value of inflow, considering in this case $A = 7.0$ for floating and $A = 5.7$ for shore caissons. This allows one with sufficient accuracy to determine the required delivery of pumping plants for sinking caissons and mainly substantially to determine the allowable difference in heads on condition of water inflow.

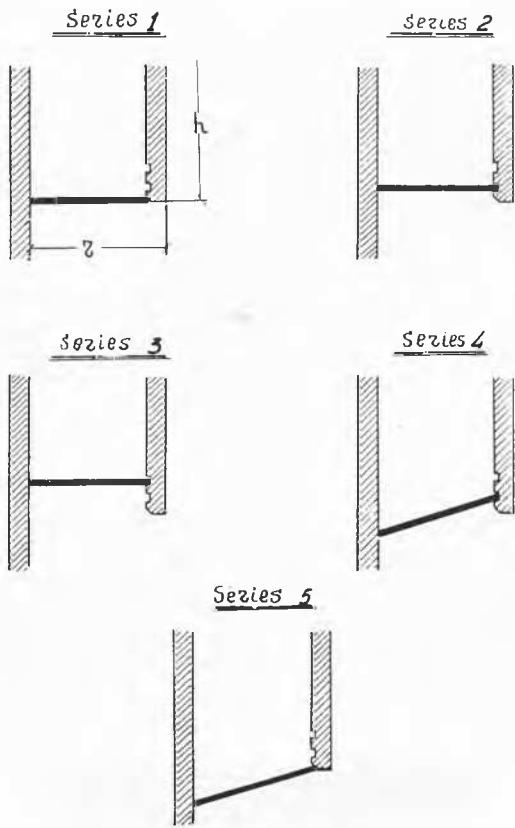


Fig. 30 Position of electrode-model of bottom of caisson for five test series.

The problems of soil influx into caissons as well as the problems of inflow of water into them was first studied *in situ* and then in laboratory conditions.

The analysis of *in situ* observations of influx of different soils with lowering head in caissons allowed one to set the approximate values of allowable differences of heads on conditions of soil influx (see Table).

Soil	Allowable difference of heads on condition of soil influx m
Sands (silty, fine, medium size)	1 to 2
Sands (coarse and gravel-type)	3 to 5
Sandy clay	3 to 6
Loam	5 to 10
Clay	10 and over

The data on the basis of which the Table was composed were obtained only in shore caissons and for only a comparatively low depth, in the majority of cases for a depth of up to 10m.

As the grounds for considering the tabulated data for floating caissons, and also for caissons sunk to greater depths, investigations were performed by the EHDA method with the same model which investigateded the problem of water inflow (Fig. 29).

The problem was considered of finding the general variation of the hydrodynamic pressure in the zone of the cutting edge with increase of the depth of sinking of shore and floating caissons. For the solution of this problem, points were plotted according to experiments by the bottom of the cais-

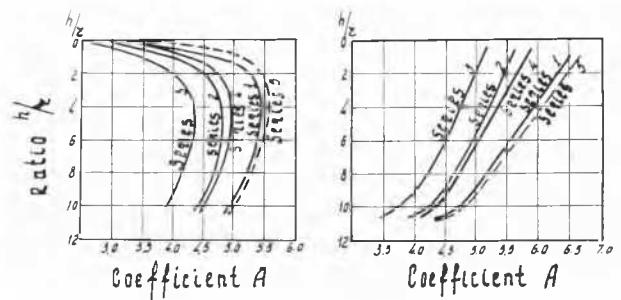


Fig. 31 Curves of function $A = f\left(\frac{h}{z}\right)$

- a) Shore caisson; b) Floating caisson
 — Curves plotted according to test data
 - - - Curves plotted according to test data and by extrapolation.

son and equal head lines were plotted for six values of depth of sinking of the model of the caisson. Each line was plotted according to 12-16 experimental points (Fig. 32). Experiments were carried out of series 2 and 4 (Fig. 30) parallel for the shore and for the floating caissons.

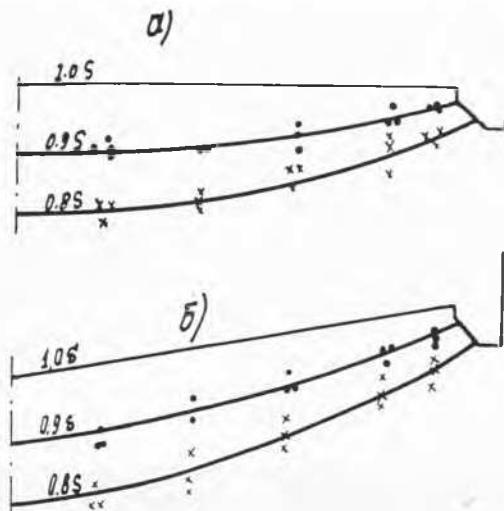


Fig. 32 Example of plotting lines of equal heads by test data:
 a) shore caissons; series 2; $h/r = 8$,
 b) shore caissons; series 4; $h/r = 8$.

The obtained data allowed one to determine that the hydraulic gradients in the zone of the cutting edge at different depths of sinking of the shore caissons change insignificantly and, consequently, the data on the allowable difference of heads given in the table may be employed independent of the depth of sinking of the shore caissons.

Comparison of the values of the gradients for the same depth of sinking of the shore and floating caissons shows that the gradients in the zone of the cutting edge for both these cases differ insignificantly, not over 18 per cent. This allowed the use of the data of the table obtained for shore caissons also for floating caissons.

Thus, the results of investigations allowed, even though only approximately, but with sufficient accuracy for practical aims the determination of the allowable difference in heads both for the condition of water inflow and for soil influx, thus allowing with the highest efficiency to regulate the value of hydrodynamic pressure in the soils during the sinking of pneumatic and open caissons.

Lors de la discussion de la section 3B, une question importante a été évoquée par M. le Prof. Kérisel, notamment de savoir si la valeur théorique du facteur de portance N_q valable pour un pieu doit être recherchée à partir d'une extension de la théorie de Prandtl ou bien à partir d'une théorie élasto-plastique.

Pour pouvoir répondre à cette question il sera nécessaire, à notre avis, d'étudier plus en détail les déformations du massif provoquées par l'enfoncement dans celui-ci d'un pieu ou d'une fondation profonde.

Nous avons pensé qu'il serait intéressant à ce sujet de présenter quelques résultats d'une étude expérimentale que nous avons effectuée à l'Institut Géotechnique de l'Etat à Gand en 1957 [1], ayant comme but principal d'étudier l'allure des déformations provoquées dans le sol par l'enfoncement d'une fondation circulaire profonde. Cependant, la méthode d'essai utilisée est essentiellement différente de celles habituellement employées.

En effet, un inconvénient des méthodes d'essai habituelles réside dans le fait que la base de la fondation enfonce dans le sol, et qui est l'origine des déformations à observer, est mobile. De ce fait il est difficile d'observer le développement complet du phénomène, tout en reliant les déplacements constatés à un système fixe de coordonnées.

Pour éviter cette difficulté, nous avons conduit nos essais de manière à provoquer, à partir d'un point fixe du sol, une déformation semblable à celle qui se produit lors de l'enfoncement de la pointe d'un pieu.

Les essais ont été exécutés de la manière suivante : Un mélange sec, composé de sable et de ciment, est enfoncé, par l'intermédiaire d'un tube fixe et au moyen d'un piston, dans la masse du sable sec. Le mélange est enfoncé graduellement par petites doses, alternativement colorées en blanc et en gris. Après que toute la quantité prévue de mélange a été enfoncée, la masse est humidifiée. Après durcissement du mélange remplissant la cavité formée dans le massif de sable, on retire le bulbe durci, puis, après séchage, on le scie suivant un plan vertical de symétrie, ce qui permet de constater les emplacements occupés par les masses de sable qui ont été successivement introduites.

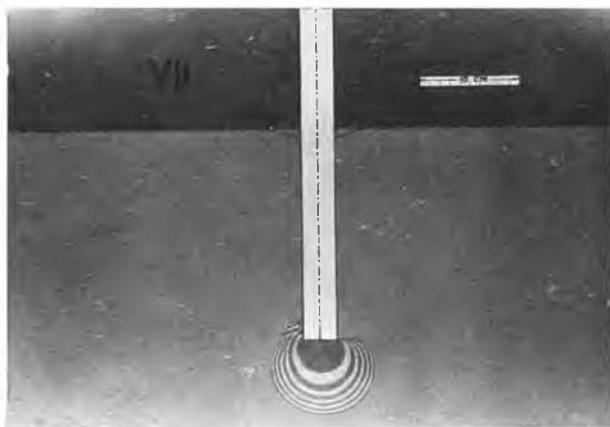


Fig. 33

La Fig. 33 montre la section du bulbe de déformation obtenu dans un sable peu compact. On constate que, dans un sable peu compact, la méthode d'essai utilisée provoque une déformation radiale continue, et cela même lorsque le volume du mélange enfoncé est considérable. Cette déformation plastique continue est fort semblable à celle prévue par la

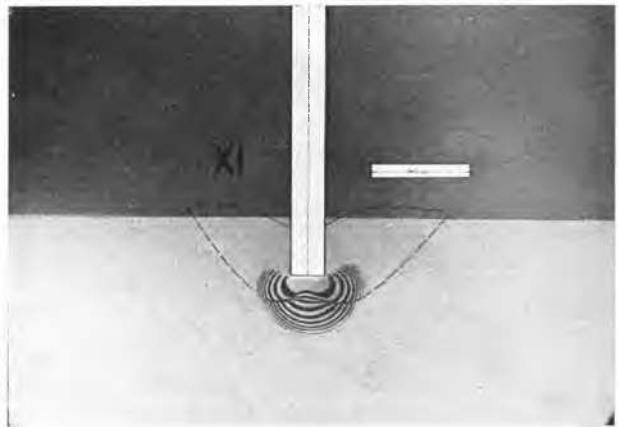


Fig. 34

théorie de l'expansion d'une cavité sphérique provoquée en un point d'un milieu indéfini.

Cependant, lorsque l'orifice du tube est situé relativement à proximité de la surface libre du sable, une fois que le volume du bulbe formé devient considérable, comme le montre Fig. 34, une rupture par refoulement se produit, suivant une surface de glissement bien déterminée conformément à la théorie généralisée de Prandtl.

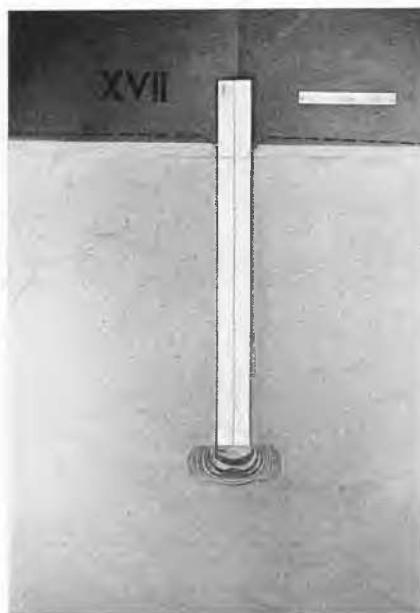


Fig. 35

La forme des bulbes de déformation obtenus dans un sable compact, Fig. 35, conduit à la conclusion, que dans ce cas se sont produits successivement deux phénomènes distincts : d'abord une déformation plastique radiale semblable à l'expansion d'une cavité sphérique, et ensuite une rupture par refoulement, suivant une surface de glissement bien définie.

Cependant, les essais montrent, que la phase de déformation plastique radiale ne se produit dans un sable compact que durant l'introduction d'un volume beaucoup plus limité que dans un sable peu compact. A la Fig. 35 on constate, qu'au moment où le phénomène de refoulement s'était déjà complètement développé, du premier phénomène, analogue à celui de l'expansion d'une cavité sphérique, il n'est resté

visible sur le bulbe de déformation qu'une calotte plus ou moins sphérique, située à la partie inférieure de celui-ci.

Les bulbes de déformation obtenus lors de ces essais montrent qu'au début de l'enfoncement du matériau dans le sable par l'intermédiaire d'un tube fixe, et jusqu'à un certain moment de l'essai, la cavité se développe en gardant une forme approximativement sphérique.

Ce phénomène de l'expansion d'une cavité se poursuit jusqu'au moment où du fait de la proximité relative de la surface libre du sol, sont réalisées les conditions, permettant qu'une rupture par refoulement se produise vers la surface libre du sable. Or, pour que les conditions de la rupture par refoulement soient satisfaites, les essais montrent qu'il faut que les dimensions de la cavité atteignent des valeurs d'autant plus grandes que la profondeur du centre de la cavité en dessous de la surface libre du sable est plus grande, et que la compacité initiale du sable est plus faible. En d'autres termes, des essais du même type, exécutés au moyen du même appareillage, mais dans une cuve présentant des distances beaucoup plus grandes entre le centre de la cavité et les parois, le fond et la surface libre du sable, que celle utilisée, conduiraient, pour les mêmes compacités du sable, à produire des cavités approximativement sphériques, mais de dimensions beaucoup plus grandes, avant qu'apparaisse le phénomène de la rupture par refoulement.

On peut se poser la question de savoir si le poinçonnement continu du sol au moyen d'un pieu cylindrique ou d'un pénétromètre ne peut être assimilé à l'enfoncement continu d'un tube et d'un piston de même diamètre que le pieu, dont les enfoncements alternatifs et égaux sont très faibles ? A notre avis, pour ces deux genres de phénomènes, les déformations dans le sol seront pratiquement les mêmes. Si on admet l'analogie proposée, les conclusions tirées des essais par introduction des doses successives à un niveau fixe, seront applicables au cas de l'enfoncement d'un pieu cylindrique. Comme le montrent les résultats des essais ci-dessus, ce n'est qu'à proximité de la surface libre du sol et jusqu'à une profondeur limitée, que la déformation provoquée par l'enfoncement d'un pieu peut conduire à une rupture par refoulement. Par contre, lorsque le pieu pénètre suffisamment profondément dans le sol, le rapport de la profondeur atteinte au diamètre du pieu est suffisamment important, pour que l'enfoncement du pieu ne provoque qu'une déformation semblable à celle définie dans la théorie de l'expansion d'une cavité située à l'intérieur d'un massif pulvérulent indéfini.

Cette conclusion n'exclut pas, cependant, que dans certains cas particuliers, l'enfoncement d'un pieu puisse provoquer, à une profondeur relativement grande, une rupture par refoulement, notamment lorsque malgré la grande profondeur relative, la résistance au déplacement du sol est beaucoup plus faible dans certaines directions que dans les autres. Un tel cas se présente, par exemple, lorsque le sol comporte des couches de compacité très différente, ou lorsqu'on enfonce dans le sol un cône au moyen d'un tube, ayant un diamètre inférieur à celui du cône.

Référence

- [1] LADANYI, B. (1959). Etude théorique et expérimentale du problème de l'expansion dans un sol pulvérulent, d'une cavité présentant une symétrie sphérique ou cylindrique. *Thèse de doctorat*, Université de Louvain.

M. A. MAYER (France)

La communication 3B/9 de MM. J. Folque et G. de Castro décrit un essai de chargement horizontal réalisé au moyen d'une semelle en béton armé de 25×7 m de surface, de 2 m de hauteur, reposant sur 18 pieux également en béton armé, système Benoto, de 1 m de diamètre et à peu près 55 m de

longueur. Ces pieux, traversant trois niveaux successifs de vases, d'abord molles, puis sablonneuses, puis consistantes et sableuses en profondeur, sont ancrés dans une assise résistante constituée, dit le rapport, par du gravier et du calcaire. La semelle était coupée en son milieu; un vérin avait été placé entre les deux sections de la semelle et permettait de les écarter en appliquant des forces horizontales. L'essai avait pour objet de contrôler la tenue de la semelle sous l'effet des sollicitations temporaires telles que le vent ou les tremblements de terre.

Le rapport donne un diagramme d'essai de chargement horizontal. Celui-ci comprend un déplacement fonction linéaire de la charge appliquée jusqu'à une valeur très importante de la charge, égale à 250 t, de nature, je pense, à donner tout apaisement aux auteurs du projet. À partir de là, la résistance cesse pratiquement de croître. Les auteurs de la communication ont eu l'impression que la seconde branche de la courbe était rectiligne et correspondait à une chute brusque du module d'élasticité du terrain s'opposant au déplacement des pieux.

L'objet de la présente note est de leur suggérer une autre interprétation qui me paraît bien plus vraisemblable et conforme à ce que j'ai vu dans tous les cas où des pieux qui n'avaient pas été conçus pour résister à des contraintes horizontales avaient été amenés à en subir. Dans ce cas les pieux se fissurent. Ceci ne veut pas dire d'ailleurs que brusquement leur résistance aux efforts verticaux s'annule. Mais il se forme des fissures que l'on peut constater chaque fois que l'on dégarnit des pieux qui ont été amenés à travailler à la flexion, alors qu'ils n'avaient pas été calculés pour cela.

Je donnerai quelques exemples. Je citerai d'abord un cas dont j'ai eu connaissance pendant la dernière guerre. Un abri à sous-marins devait être construit dans un port français dans une zone où le rocher se trouvait à une douzaine de mètres de profondeur et était surmonté d'une vase molle. Plusieurs centaines de pieux en béton armé avaient été battus à l'emplacement des murs porteurs de la superstructure et, en vue du bétonnage, un énorme tas de sable avait été déposé à proximité immédiate du chantier, sur la vase. Lorsqu'on ouvrit les souilles destinées à constituer les garages pour les sous-marins, tous les pieux étaient cassés. Les contraintes horizontales provoquées par le tas de sable, transmises par la vase sous-jacente, avaient fait travailler les pieux à la flexion. Ils étaient tous cassés à la base, au contact entre le rocher et la vase et durent être remplacés.

Je voudrais également rappeler deux cas plus récents : dans le premier, un bâtiment d'une dizaine d'étages et d'une centaine de mètres de longueur avait été construit dans l'estuaire d'un grand fleuve, face à la direction des vents dominants. À la suite d'une forte tempête des fissures apparaissent en fondation en même temps qu'une nette inclinaison du bâtiment. Celui-ci fortement armé s'était légèrement incliné sans se fissurer. Les mouvements furent arrêtés au moyen de pieux en sousœuvre qui déchargèrent les premiers, lesquels furent trouvés fissurés. Le vent avait fait travailler à la flexion les pieux arrière du bâtiment, qui se fissurèrent.

Dans le second cas des bâtiments de trois étages et un sous-sol avaient été construits à 3 m en contrebas du quai d'un fleuve sur pieux de 18 m allant jusqu'au sol résistant. Les remblais du quai avaient été arrêtés à une certaine distance des bâtiments qui purent être terminés sans faire apparaître aucun désordre. Ensuite on remblaia, sur toute la hauteur des caves, l'intervalle compris entre les constructions et le quai. On constata des désordres très importants auxquels on ne put remédier qu'en encadrant chaque pieu de fondation entre deux pieux exécutés en sousœuvre dont un certain nombre inclinés pour recevoir la poussée du terrain.

Si maintenant nous revenons aux fondations des semelles de MM. Folque et de Castro, nous rappellerons qu'il s'agit de pieux Benoto, c'est-à-dire de pieux forés, probablement in-

suffisamment armés pour résister à la flexion, surtout en raison de la longueur du bras de levier, de 55 m. Chacune des deux moitiés de la semelle n'a subi que des déplacements élastiques jusqu'à une charge horizontale de 250 t. La semelle étant supportée par 18 pieux, chacune des deux moitiés tenait sur 9 pieux, qui ont donc chacun supporté, en ne subissant que des déformations élastiques, des contraintes horizontales allant jusqu'à près de 28 tonnes, ce qui est considérable. Après quoi certains d'entre eux ont dû se fissurer, ce qui explique sans difficulté la forme de la courbe relevée.

Je ne crois malheureusement pas que l'on se trouve en présence d'un phénomène nouveau d'une modification du module d'élasticité du terrain, mais d'une constatation absolument classique, que j'ai pour ma part faite en de nombreuses occasions au cours de ma carrière.

Il est rare cependant que l'on ait l'occasion de mesurer l'intensité des efforts horizontaux que peut supporter un groupe de pieux sans se rompre et c'est à ce titre que les essais de MM. Folque et de Castro nous paraissent présenter un intérêt incontestable.

M. G. MEARDI (Italie)

We have heard how difficult it is to find the actual relations between the penetration resistance of the penetrometer and the resistance of the piles — which have a much greater diameter — when a static penetrometer is used to calculate the bearing capacity of foundation piles.

I shall explain how I have planned bored piles for several years using a dynamic penetrometer of 51 mm in diameter, a 73 kg (160 lb.) hammer, 75 cm height of fall, with a Ø 48 mm casing.

As I have already related in Section 2, with sands, silty sands and gravelly sands, with the indication of the dynamic penetrometer (considering it equivalent to the Standard Penetration Test of Terzaghi and Peck) following Meyerhof's experiences I have obtained the friction angle of the soil, applying to this angle the reductions of Terzaghi and Peck when the soils are loose or of medium density. I have compared the results of the loading tests of the bored piles with the calculated bearing capacity : I corrected the friction angles to make them agree, and I have obtained a diagram with friction angles versus N as shown in the Fig. 36.

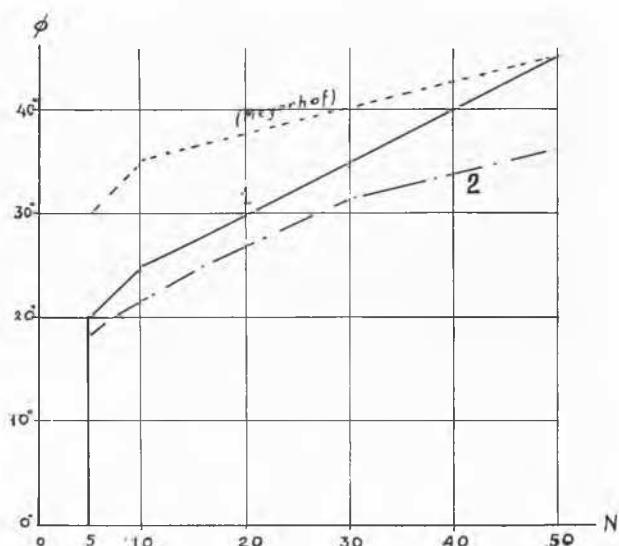


Fig. 36 Diagram of Φ versus N : 1. Silty sand and gravel, other than saturated fine silty sand; 2. Saturated fine silty sand.

The diagram is double : for ordinary sands and gravels, and for very fine sands in water. For these soils the angle ϕ is minor for the well known reasons. So far the diagram proved satisfactory. The calculation of the friction bearing capacity of the bored piles is made with such angles applying Mayer's old formula

$$P_f = \pi d \gamma_e \frac{h^2}{2} \operatorname{tg} \phi$$

when the depth h does not exceed 10 m. When the depth is more than 10 m. I correct the formula, supposing that the pressure of the concrete against the surrounding soil cannot exceed $10 \gamma_e$ because of the time necessary to execute the bored piles.

For the friction bearing capacity I adopt Mayer's formula because it is based on the active pressure exerted by concrete against the soil, and we can evaluate it better than the pressure (active or passive) of the soil; besides it enables us in a very simple way (even if somewhat by estimation) to keep into account the various pressures exerted by the concrete against the earth according to the methods of construction of the piles : concrete compressed with a hammer — concrete compressed with air (Wolfsholtz pile) — fluid concrete simply poured into the boring tube moving it up and down during extraction.

When we deal with piles in clayey soils (and such soils are generally indicated as I have already shown in Section 2 by the diagram of the casing resistance) I estimate the friction bearing capacity on the ground from the shear strength obtained with the Vane Test or other means.

If the clay layer is not fundamental I assume $\tau = \frac{N}{5}$ kg/cm²

with soft soils; $\tau = N : 10$ with stiff soils up to a maximum of $\tau = 1$ kg/cm². For the base I adopt the total value of the shear strength.

With tension piles the results are not so concordant; generally the bearing capacity of the piles is inferior to the expectation. Sometimes in fine sandy soils in water, it was less than a half of the bearing capacity calculated; but I have not sufficient data on this pile to be able to draw any conclusion.

M. H. MUHS (Allemagne)

The subsoil conditions in Northern Germany where in many cases old riverbeds with organic soils are embedded in thick layers of diluvial sands, rather often necessitate pile foundations. In connection with the desire of the contractors to increase the permissible loads of the different pile-types, quite a lot of loading tests were carried out in recent years. I want to tell you about some results of loading tests with cast *in situ* bore piles which, in my opinion, are of general interest.

When you perform a loading test with a usual pile-type of 30 — 50 cm, whose point is embedded in loose or medium dense sand, generally you do not get a load-settlement-curve with a typical failure point. The same happens when you perform a test in dense sand, but with a pile with a specifically enlarged base, because generally you do not reach the high loads under which the failure would occur. In both cases the failure load must be introduced as a load which, in connection with the chosen factor of safety, leads to a limited settlement without any danger for the structure.

Preparing a standard for the construction and the use of cast *in situ* bore piles for Germany — the DIN 4014 — a number of about 60 to 80 loading tests was evaluated in the same way. It was found that a total settlement of 20 mm corresponds very well with that load which — according to other failure criteria — would give the failure load for relatively slender piles, with approx. 30 - 50 cm diameter.

For thicker piles, with diameters of more than 50 cm, the failure load would be higher of course, but it would be connected with a higher settlement, which is normally not wanted for pile foundations in sand. Therefore it seems to be quite useful to determine the load for a settlement of 20 mm as failure load in such cases, where the curve does not yield a typical failure point.

The evaluation of the rather high number of 60-80 loading tests shows furthermore that — when you introduce a factor of safety of 2 — the settlement under the permissible load lies in the range of 5 mm — say 3 — 7.5 mm. That means that the piles will show a practically equal and very limited settlement and that they possess a factor of safety of at least 2, when the allowable load is determined as described. That the total and not the net settlement is used for the failure criterium, is another advantage, because usually loading tests are carried out very late, often not before the construction work begins. In such cases it is very helpful if one can perform the test in the shortest way, that is without needing to interrupt the test by repeated unloading to obtain the net settlement curve.

M. H. MUHS

I want to give some information about experience gained from penetrometer investigations in connection with the determination of the permissible load of cast *in situ* bore piles with special wide bases, ranging from 65 to 105 cm. With such piles, twelve loading tests were performed in different parts of Northern Germany. The bases of the piles were always located in dense sand, in which the measured point-resistance of the penetrometer ranged from 175 to 250 kg/cm². In the same sand four other loading tests with piles without a special wide base were carried out.

In his contribution to this Conference, Dr Menzenbach has stated that the allowable pressure at the point of a pile can be determined by dividing the penetrometer resistance by two figures : First by the factor of safety, which is chosen with 1.7, then by a figure which lies between 1.5 and 1.75. This relationship had already been proved by Van der Veen. Menzenbach only confirmed it by the statistical evaluation of 88 loading tests. Most of these tests were carried out in loose or medium dense sand, for which Menzenbach announces the penetration resistance with 25—150 kg/cm².

Through our tests in dense sand we found the figure of 1.5—1.75 much too low. These 12 tests led to relation values between about 8 and 12, the lower values for the relatively smaller bases, the higher ones for the relatively larger bases. The loading tests with piles without a special wide base gave figures between 4 and 6.

According to these test results the figures of Van der Veen and Menzenbach respectively should not be used in dense or very dense sand with a penetration resistance of more than approximately 180 kg/cm², especially for piles with large bases. In my opinion the figures of 1.5—1.75 are also too low in loose sand, when cast *in situ* bore piles with larger bases are used.

M. A. PELLEGRINO (Italie)

L'objet de mon intervention est la détermination de la force portante d'un pieu à partir des indications du pénétromètre statique. Je voudrais d'abord souligner l'influence des deux facteurs expérimentaux suivants :

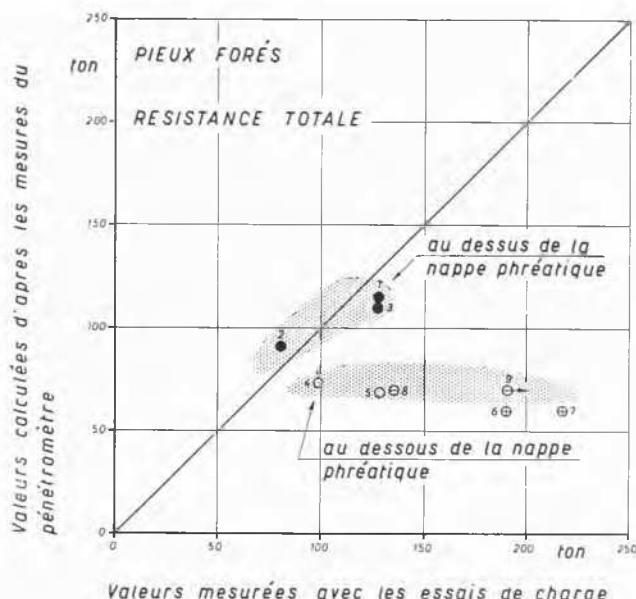
- (a) Nature des sols;
- (b) Type de pieu.

Les articles publiés jusqu'à présent se rapportent pour la plupart à des essais dans des sables et dans des graviers et à des pieux battus.

Je pense donc qu'il peut être utile de communiquer les résultats d'essais réalisés dans des argiles et dans des sols d'origine volcanique, avec des pieux forés [1].

Quelques-unes de nos expériences ont été effectuées dans la ville de Naples, dont le sous-sol est constitué surtout par des sols volcaniques : ces sols sont formés par des fragments de magma à texture plus ou moins vitreuse et spongieuse (cendres, ponces), ou bien par des fragments de lave avec parfois des cristaux et avec des fragments de roches non volcaniques (sables volcaniques); en tout cas il s'agit de produits non altérés [2]. D'autres expériences ont été faites dans les environs de la même ville : c'est là qu'on rencontre des sols de transport fluvial constitués surtout par des argiles; près de la mer ces sols sont recouverts par des sables de dune.

Les résultats obtenus jusqu'à présent sont représentés sur la Fig. 37.



Valeurs mesurées avec les essais de charge

PIEU	SOLS
1-2-3	cendre volcanique
4-5	sols volcaniques (cendre et ponces) mixtes à des sols d'autre origine
6-7	sables volcaniques
8-9	argiles recouvertes par des sables de dune

Fig. 37

Il en résulte que s'il n'y a pas de nappe (pieux n. 1, 2, 3) il y a un bon accord — du moins pour les cendres volcaniques — entre les valeurs de la résistance totale calculées d'après les mesures effectuées avec le pénétromètre et celles qui ont été mesurées avec des essais de charge sur pieux. Ce qui doit être rapporté au fait que, dans ces conditions, les parois des forages restent bien stables et l'on peut présumer qu'il n'y a pas de variations dans la résistance des sols à la suite du forage.

Avec la présence de la nappe phréatique, la résistance totale calculée d'après les mesures effectuées avec le pénétromètre est beaucoup moins grande que celle qui a été

mesurée avec l'essai de charge. Cela a été constaté non seulement dans les sols volcaniques, mais aussi dans les argiles et sur ce point nous nous trouvons d'accord avec les expériences de Golder [3].

Si l'on compare les pieux forés avec les pieux battus pour ce qui concerne la prévision de leur résistance totale d'après les indications des pénétromètres, pour les pieux battus [4] les résultats des calculs sembleraient être satisfaisants dans l'ensemble, tandis que pour les pieux forés, selon les expériences de Golder et les nôtres, la différence entre prévision et réalité semblerait être parfois considérable.

* * *

Dans notre recherche nous nous sommes proposé un autre but de caractère un peu plus général, c'est-à-dire de mettre en rapport la résistance sur la pointe mesurée au moyen du pénétromètre avec les propriétés mécaniques des sols.

Les essais ont été effectués dans les mêmes sols illustrés ci-dessus, c'est-à-dire dans des dépôts fluviaux à grain fin (argiles) et dans des sols d'origine volcanique. La résistance au cisaillement pour les argiles a été déterminée par des essais de compression triaxiale « Q ». Pour les sols volcaniques, et spécialement pour les cendres, la résistance au cisaillement ne varie pas beaucoup avec les modalités de l'essai.

Pour les argiles (voir Fig. 38) les valeurs de la résistance sur la pointe calculées selon la théorie de Meyerhof sont inférieures à celles qui ont été mesurées. Il faut tenir compte que les calculs ont été développés d'après la théorie proposée par Meyerhof pour des fondations profondes à base plane; c'est à des valeurs peu différentes que l'on parvient en appliquant la théorie pour des fondations profondes à base conique, présentée par Meyerhof à ce Congrès.

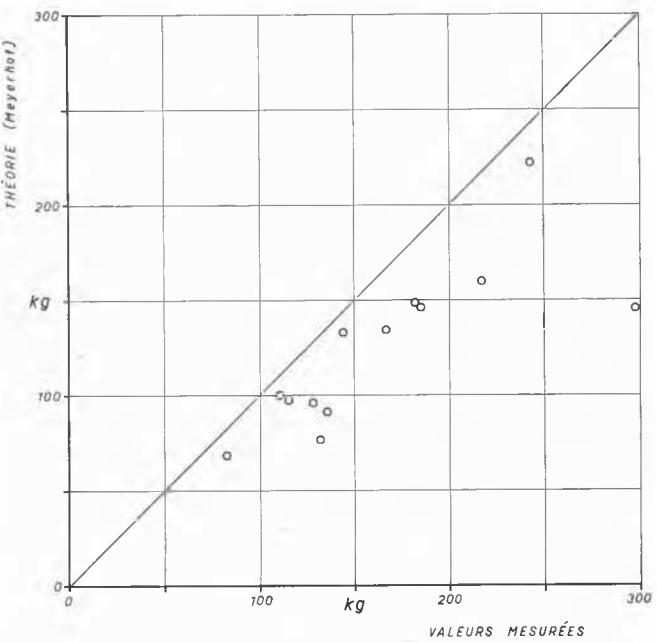
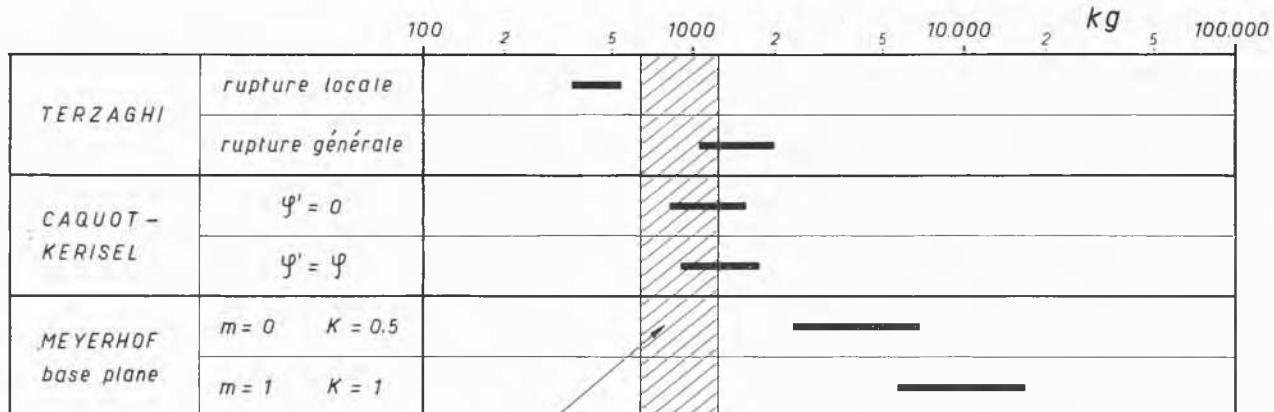


Fig. 38 Résistance de pointe du pénétromètre.
Argiles ($c \neq 0$; $\varphi = 0$)

En appliquant d'autres théories, on aboutirait à des résultats encore plus éloignés de la réalité, comme on peut facilement s'en rendre compte par l'observation des valeurs du coefficient N_c :



Valeurs mesurées avec le pénétromètre

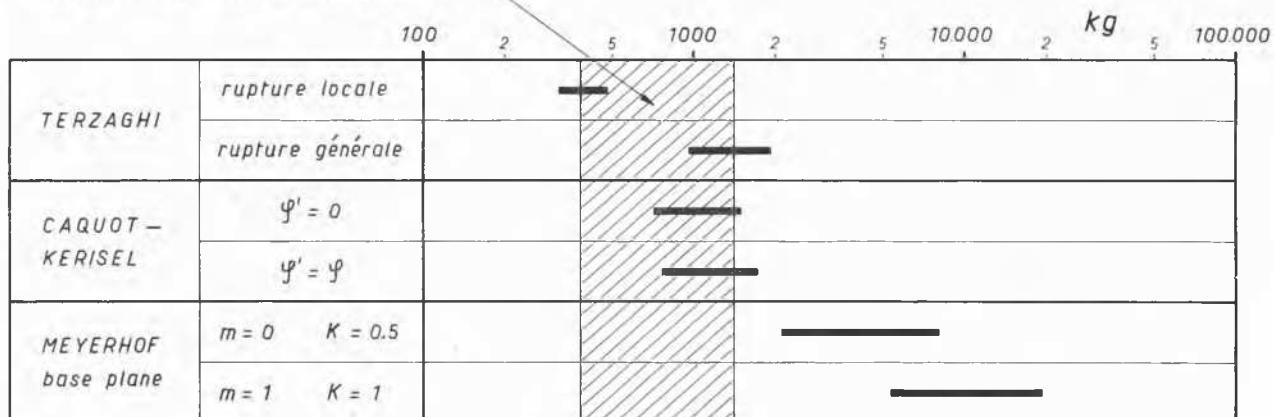


Fig. 39 Résistance de pointe; Cendre volcanique ($c \neq 0$; $\varphi \neq 0$).

Meyerhof	base plane	9,5
	base conique	10
	rugueuse	7,5
Caquot-Kérisel		5,15
Terzaghi	rupture locale	4,9
	rupture générale	7,4

Quant aux cendres volcaniques, (Fig. 39) il arrive tout le contraire : suivant les théories de Caquot-Kérisel et de Terzaghi on obtient des résultats peu différents entre eux et correspondant à l'expérience mieux que ceux que l'on obtient suivant les théories de Meyerhof.

Selon nous, les différences qu'on remarque en passant d'un type à un autre de sol confirment l'observation du Rapporteur Général sur la nécessité de prolonger la recherche sur l'influence des divers paramètres sur les coefficients de portance.

Il ne faut pas seulement considérer que les conditions du sol autour de la pointe conique pourraient être différentes de celles des échantillons intacts sur lesquels on exécute les essais de laboratoire — comme le démontrent les récentes expériences de Kérisel — mais on peut même présumer que les hypothèses qui sont à la base des différentes théories sont satisfaites seulement dans quelques sols et non pas dans d'autres ; par conséquent, dans un sol donné on pourrait se rapporter de préférence à une de ces théories plutôt qu'aux autres.

Références

- [1] Istituto di Tecnica delle Fondazioni e Costruzioni in Terra dell'Université di Napoli. Une Communication à leur sujet a été présentée au Ve Congrès de Géotechnique de Palerme, mars 1961.
- [2] PENTA, CROCE, ESU (1961). "Engineering Properties of Volcanic Soils". Proc. V Int. Conf. Soil Mech. Found. Eng., Paris.
- [3] GOLDER, H.Q. (1953). "Some loading Tests to Failure on Piles". Proc. III Int. Conf. Soil Mech. Found. Eng., Zurich.
- [4] HUIZINGA, T.K. (1951). "Application of Results of Deep Penetration Tests to Foundation Piles". National Research Council of Canada, Building Research Congress.
- [5] MENZENBACH, E. (1961). "The Determination of the Permissible Point-load of Piles by Means of Static Penetration Tests. Proc. V Int. Conf. Soil Mech. Found. Eng., Paris, 1961.

M. T.E. PHALEN (Etats-Unis)

The purpose of this report is to point out the practical and theoretical consideration that must be made when placing added dead loads or additional surcharge adjacent to an existing stable wood friction pile foundation.

The problem as it presented itself in the field consisted of an "L" shaped fifteen foot wood friction pile foundation that was established some forty years ago. The tops of pilings were established well below the existing ground water table and subsequent investigation indicated that the water table currently is still well above the top of pilings and that the physical evidence indicated that no serious bacterial action was affecting the wood pilings. Immediately after construction to the site, some four years prior to this writing, differential settlement began to manifest itself to such an extent that at the time of writing this paper the maximum differential settlements were in the magnitude of one and three quarter inches. This settlement created distortions and cracking through the entire four storeys of the structure.

The technique utilized to examine the differential settlements consisted of determining the effective stresses added

to the various soil layers by means of Boussinesq and Westergaard equations and utilizing the increase in stress as the ΔP in determining settlements at various points.

This problem and the technique applied to predicting trends of settlements point out several important factors.

1. Extreme care should be utilized when adding any surcharge in the vicinity of existing shallow friction pile foundations.

2. Boussinesq and Westergaard equations when properly applied predict the future trend of differential settlements and stress conditions of various stratas quite accurately.

3. The application of these equations can be utilized to determine the locus of new construction so as to minimize differential settlements.

4. Minor increases in effective stresses can create severe differential settlements to existing friction pile foundations.

5. Stability of wood piles from bacterial action is very good provided the water table is maintained above the top of the piles.

6. The magnitude of the surcharge will affect the differential settlements provided the effective stress envelope as described by Boussinesq and Westergaard equations comes within the area of the foundation.

7. Once increases in effective stress patterns are determined, it is extremely important to have knowledge of soil properties and compressibility properties, to more accurately forecast future settlements utilizing Terzaghi's Theory of Consolidation.

M. H.M. RAEDSCHELDERS (Belgique)

Dans la communication N° 3B/12 Kérisel annonce que les systèmes de pénétration dans lesquels on procède par avancement successif de la pointe et l'enveloppe latérale conduisent à des valeurs exagérées de la pression à la pointe.

L'Institut Géotechnique de l'Etat en Belgique a effectué en 1949-1950 quelques essais avec deux cônes différents, le cône classique de l'appareil Hollandais et un cône avec mesure de pression à l'aide de « strain-gages » permettant la mesure continue. Quelques diagrammes permettant la comparaison des deux méthodes ont été publiés en 1951 par le Prof. De Beer dans les « Transactions of the South African Institution of Civil Engineers » comme discussion à un article de M. Kantey.

Les essais ont été effectués suivant la méthode discontinue avec le cône représenté à la Fig. 40 a, et suivant la méthode continue avec le cône à strain-gages représenté à la Fig. 40 b.

Les Figs. 41, 42, 43, 44 et 45 permettent de comparer les résultats obtenus avec les mesures discontinues (traits interrompus) et les mesures continues (traits pleins).

Les essais correspondants ont toujours été effectués à moins de 1 m de distance. Les figures renseignent également la nature des couches rencontrées. On peut constater que globalement la résistance à la pointe est plus grande lorsque le cône et les tubes sont enfoncés simultanément.

Nous sommes convaincu, et M. De Beer l'a indiqué aussi dans l'article mentionné, que cette augmentation de la résistance à la pointe est due au fait que le frottement provoque autour de la base du pieu des surcharges latérales qui sont supérieures à celles qui existent en l'absence d'une sollicitation sur le fourreau. La Fig. 46 montre schématiquement le phénomène. Ceci correspond d'ailleurs à la remarque du Rapporteur général lorsqu'il dit que le frottement positif lors de l'enfoncement des pieux fait accroître les pressions verticales effectives.

Echelle : 0 1 2 5 cm

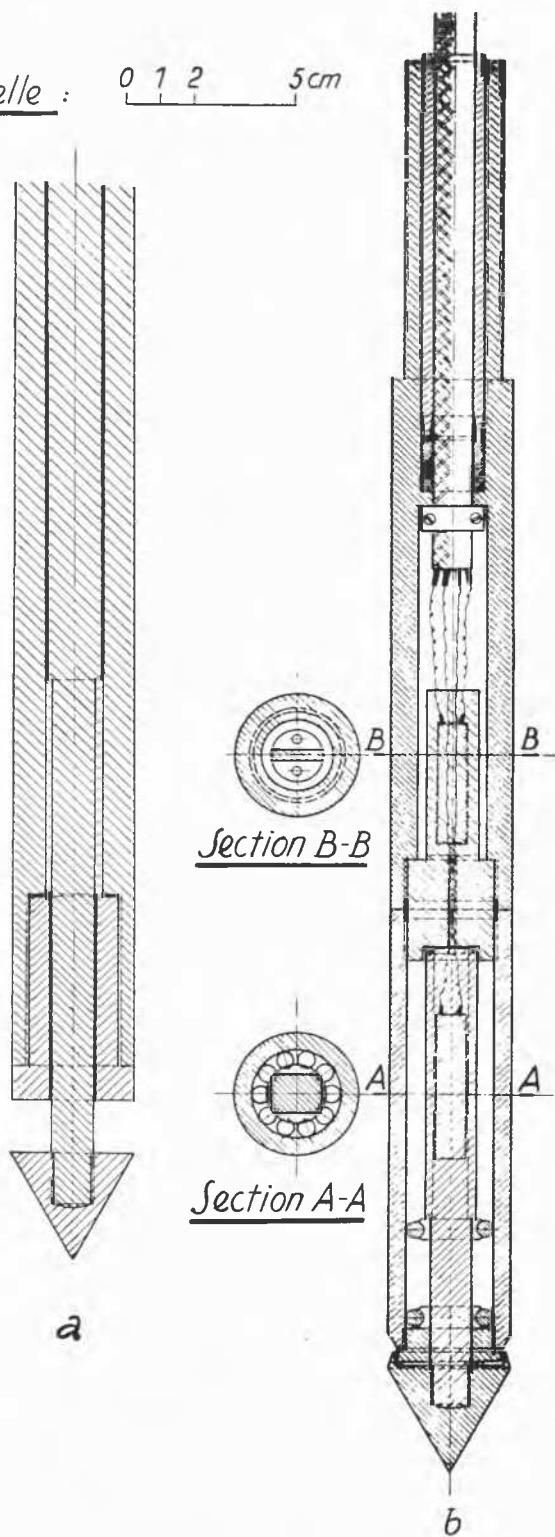


Fig. 40

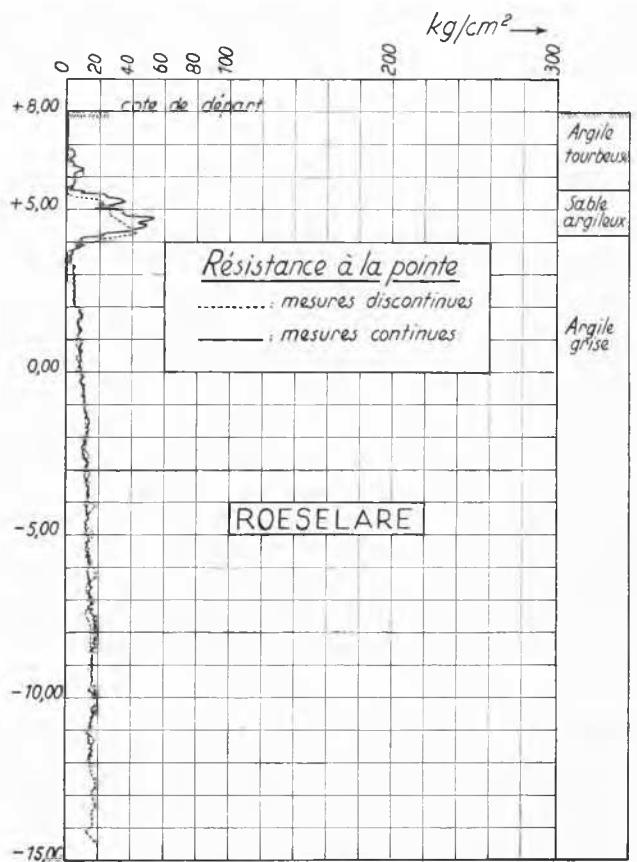


Fig. 41

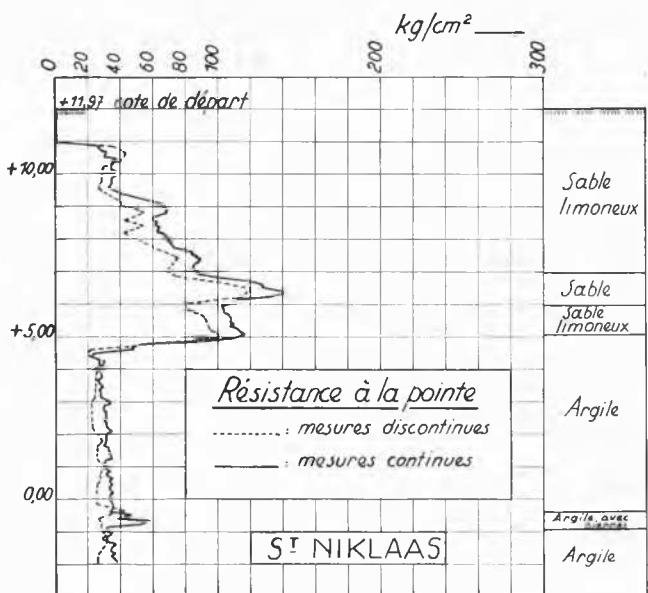


Fig. 42

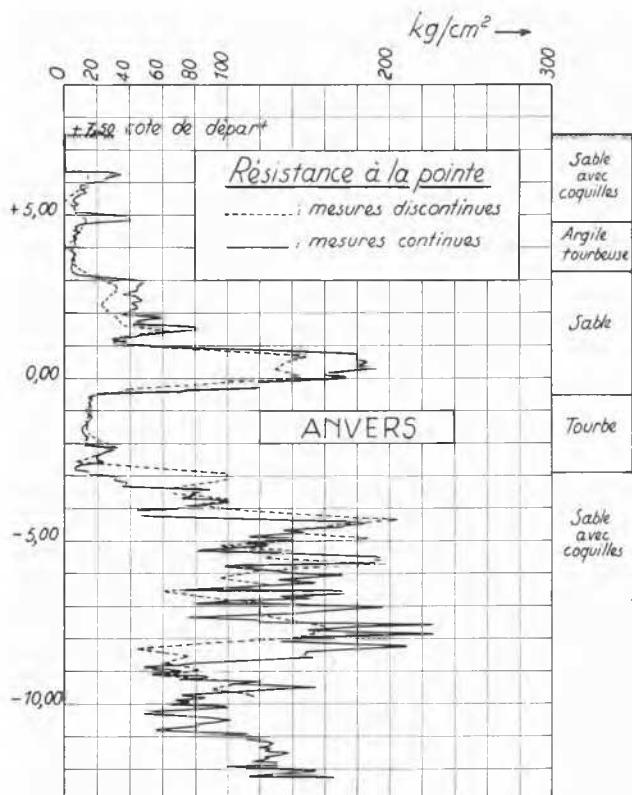


Fig. 43

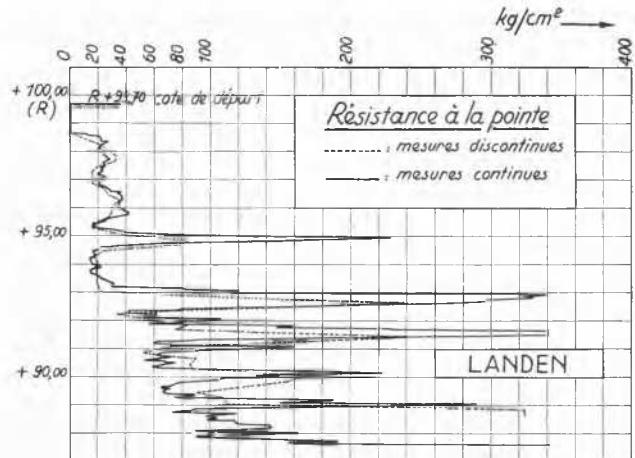


Fig. 45

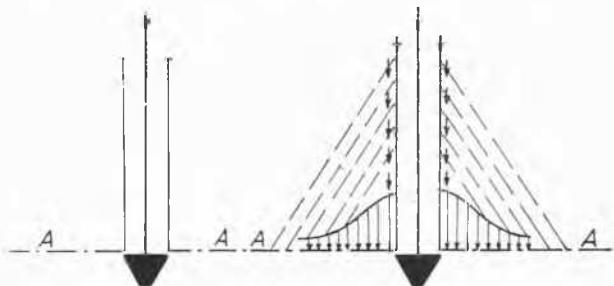


Fig. 46

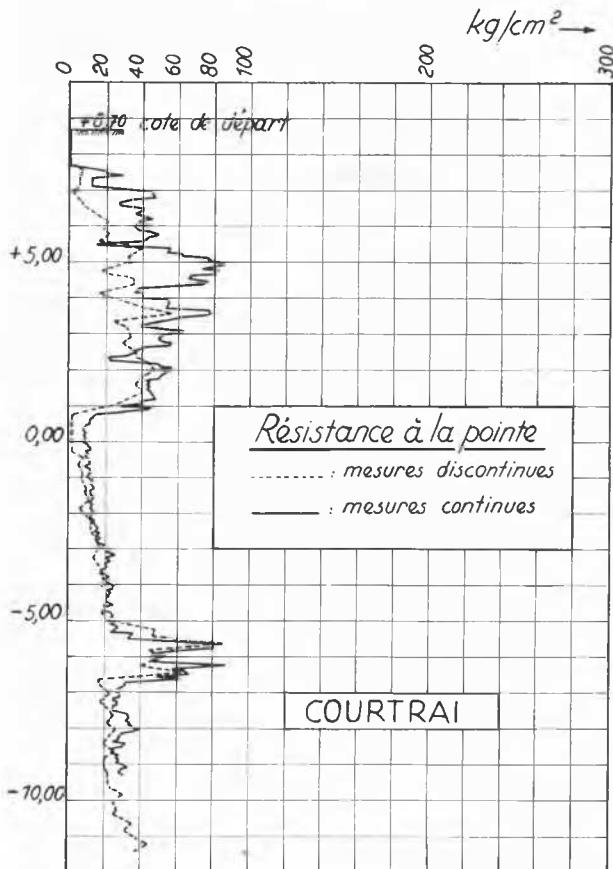


Fig. 44

M. S. SCHIFF (France)

L'étude de groupe de pieux dans le sable fait l'objet de plusieurs communications, notamment celles de Stuart, Hanna (3B/23) et Nishida (3B/19). Des essais effectués sur pieux modèles réduits à la Section Recherches de Mécanique des Sols du C.E.B.T.P., permettent de compléter les données présentées et de préciser le comportement des petits groupes de pieux fichés dans le sable.

Les pieux utilisés sont constitués par des ronds ou des tubes en acier lisse, de diamètre compris entre 25 et 100 mm et de 1 m de longueur environ. Les essais ont été effectués avec du sable de Seine tamisé à 2 mm dans une cuve de 2,5 m de diamètre et de 2,5 m de profondeur, dans laquelle le sable a été compacté à des densités comprises entre 1,65 et 1,70, soit au-dessus de la densité critique. Quelques essais ont été réalisés sur du sable lâche *in situ* aux Vaux de Cernay, dans la formation du sable de Fontainebleau.

Des groupes de pieux de différentes configurations et avec des écartements entre axes de pieux différents ont été utilisés, pour deux fiches initiales. La puissance disponible était de 12 tonnes et le plus grand groupe de pieux essayé comportait treize unités. Les pieux étaient équipés de jauge à fil résistant permettant de connaître l'effort total par pieu et la charge en pointe.

Ces essais ont mis en évidence le comportement suivant :

1. Dans des sables denses l'efficacité des pieux dans un groupe augmente en général avec le nombre de pieux, et il y a un écartement entre axes de pieux optimum pour lequel, pour une fiche donnée, l'efficacité est maximale. Cet écartement est de l'ordre de 5 à 6 fois le diamètre des pieux pour des groupes de 2 à 5 pieux. L'efficacité est en général supérieure à 2,00 et nous avons enregistré pour des groupes de 5 pieux

des valeurs supérieures à 3. L'interaction entre pieux disparaît pour des écarts de l'ordre de 13 fois le diamètre.

2. Dans le cas des sables lâches, d'après le nombre limité d'essais dont nous disposons, il semble que l'efficacité peut être inférieure à l'unité mais assez voisine de 1,00.

3. Les essais effectués sur le sable dense ont mis en évidence une augmentation de la charge portée par frottement latéral et une diminution de la charge en pointe, toujours par rapport aux charges sur un pieu isolé. Par contre, dans les sables lâches, le phénomène de serrage qui accompagne la majoration de la charge latérale des pieux ne se manifeste pas.

4. Le tassement d'un groupe de pieux dans le sable dense est, à charge unitaire égale, moins important que celui d'un pieu isolé. Par contre, à la rupture un groupe de n pieux portant plus que n fois le pieu isolé, le tassement du groupe est plus élevé. A tassement égal, la charge unitaire dans un groupe (par pieux) est supérieure à 2 fois celle du pieu isolé.

5. D'une façon générale, on peut dire que la densité relative du sable est un paramètre fondamental dans le comportement des groupes de pieux fichés dans le sable.

M. F.A. SHARMAN (Grande-Bretagne)

During the discussion nobody remarked that among all the uncertainties and unknowns concerning penetrometer methods there is one system that will give entirely reliable results every time — I mean the constant rate of penetration method applied to the pile itself by the loading method proposed in Mr Whitaker's paper 3B/27 "A new approach to pile testing". It may be thought to be rather unfair to claim that a pile test is a penetrometer method a tall, and of course it is true that prediction after the event is easier and rather less useful than forecasting, but the point I want to make is that the general adoption of this method of testing would have at least two excellent results :

1. It would enormously enlarge the amount of field data available for correlation between cone and penetration tests of different diameters and the behaviour of fullscale piles.

2. It would be a positive move in the direction of the standardization of testing procedures for which our general reporter has so rightly appealed. Anyone who has attempted to summarize and make deductions from the testing records of a large number of jobs, will know how impossible it is to compare performances under test while there is no general agreement on the definition of ultimate load, and very often the criteria used in the tests are not adequately recorded.

I should like also to refer to comments by Prof. Peck, and of the General Reporter (in his written report) on my tentative suggestion for a "skin friction reduction coefficient" for clay, as both these comments were a little misleading about the conclusion stated in paper 3B/21.

I found just as Prof. Peck found in Dakota, that one cannot rely on mobilizing even the re-moulded shear strength of soft clays as shaft support. My proposal was, in fact, the use of a factor 0.4 (in the case of straight sided concrete piles), applied to the mean of re-moulded and undisturbed cohesions, as determined by *in situ* vane tests. With clays of medium sensitivity this does in fact give an answer less than the re-moulded cohesion would produce. Of course the proposal is empirical, but the disturbance pattern in a soft clay near a driven pile is certainly very complex, and it is important to remember that in real cases there will certainly have been some lateral movement of the sides of the pile however well it has been guided and however accurately the hammer has hit it. It seems to me just as likely that the two strength parameters obtained under vane test conditions should bear some sort of limit relationship to failure conditions round

a pile, so that a consistent relationship should be deducible between small diameter penetrometers and fullscale piles.

M. H.U. SMOLTCZYK (Allemagne)

The question of from which critical centre distance the bearing capacity of a friction pile group is different from the sum of the bearing capacities of the single piles, is twofold; as the skin friction depends on the possibility of the soil of giving space to the pile volume either by compaction or by displacement.

The influence of compaction may be evaluated by assuming that the void volume n_0 before pile driving is changed within a limited "influence area" of radius R as following :

$$\Delta n(r) = (n_0 - n_{\min}) \left[1 - \left(\frac{r - r_0}{R - r_0} \right)^2 \right] \quad \dots(1)$$

where r_0 is the pile radius and r the variable distance. With $r_0 = 1$ one gets by integration from r_0 to R the equation

$$\frac{1}{n_0 - n_{\min}} = R^2 - 1 - \frac{R + 1}{R - 1} - \frac{(R^2 + 1)(R + 1)}{2(R - 1)} + \frac{4}{3} \frac{R^2 + R + 1}{R - 1} \quad \dots(2)$$

which is graphically shown by Fig. 47.

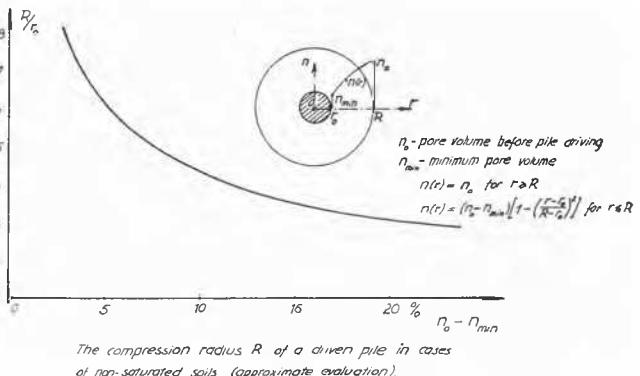


Fig. 47

The underlying assumption (1) is, that the void volume change decreases proportionally with the influenced area. Eq. (2) therefore is applicable only in cases of non-saturated soils which are not already too compacted. In other cases, where displacement of soil is predominant compared with compaction, R will be smaller than computed by (2). So, this formula may give an upper limit for the critical distance beyond which no interdependence of the piles is to be expected.

As I am presently involved in the problem of inducing large horizontal loads by means of a piled foundation into the ground, I was very interested to learn through paper 3B/14 of Messrs Matlock and Reese the American approach to the problem. I wonder about the authors' opinion about the nature of the soil reaction close to the ground level. They use a modulus of soil reaction which might be expected to be something mid-way between confined and unconfined strength. At small loads, it will be almost like the confined strength. Later on, it will decrease, which means that the comparative length T will gradually increase. At the beginning the soil reacts predominantly by compaction, later on by displacement. So, the soil reaction is not only a function of depth but also of lateral displacement of the pile. Are there any attempts to extend this valuable method to this additional parameter ?

M. G.F. SOWERS (Etats-Unis)

For convenience in design, the charts of optimum pile spacing and group efficiency in Paper 3B/24 by Sowers et al, have been approximated by mathematical expressions. The optimum spacing, S , center to center in pile diameters for a group of N piles is given by

$$S = 1.1 + 0.4 N^{0.4}$$

The group efficiency, e , at the optimum spacing is

$$e = 0.5 + \frac{0.4}{(N - 0.9)^{0.1}}$$

M. U. W. STOLL (Etats-Unis)

On Paper 3B/27 "A New Approach to Pile Testing" by T. Whitaker and R.W. Cooke

The authors are to be commended for re-examining the question of pile load test procedure. It has been a general experience that conventional load tests, which depend on achieving negligible rates of settlement before proceeding to successively higher loads, are often time consuming and lead to uncertain results in the case of high capacity end bearing piles. Although the C.R.P. method described gives a definitive bearing capacity for pure friction piles in clays, the writer does not find evidence that the procedure will

give an equally definitive test capacity for essentially end-bearing piles. Specifically, Fig. 8 of the paper, showing results of a load test for a pile bearing on sand, does not reveal an identifiable limit load falling within the initial 3 inch settlement range. Further, the suggested arbitrary settlement limit of 20 per cent of the pile width is well beyond that required to induce plastic displacement in most soils. For example, this would involve more than 2 inches of movement for a conventional 10-3/4 inch diameter pile.

The writer wishes to describe briefly a pile test method which has been successfully used on upwards of 100 piles over a 30 year period, and present two examples of test results: the first, obtained from an H pile supported in side friction; the second, an essentially end-bearing pipe pile. The method was developed by Prof. William S. Housel, of the University of Michigan, and is presently included as an alternate method by the American Society for Testing Materials [1], is incorporated in the Detroit Building Code [2], and has been described in previous technical papers. [3, 4]

The basis of the method is to obtain a series of precise settlement readings at 10 minute intervals for a succession of one hour periods, during which the load is held constant. By selecting an estimated load increment of, say, one-eighth the available reaction, one definitely sets the upper limit for the test period as 8 hours plus 2.5 hours for unloading, or 10.5 hours total. Testing is actually continued until either definite progressive movement is achieved or the capacity of the loading apparatus is reached.

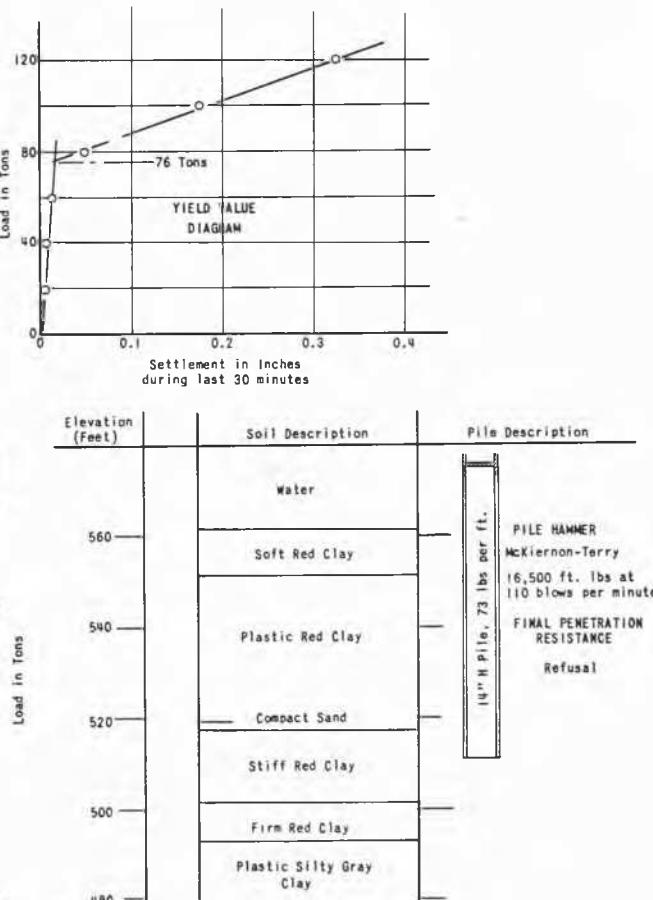
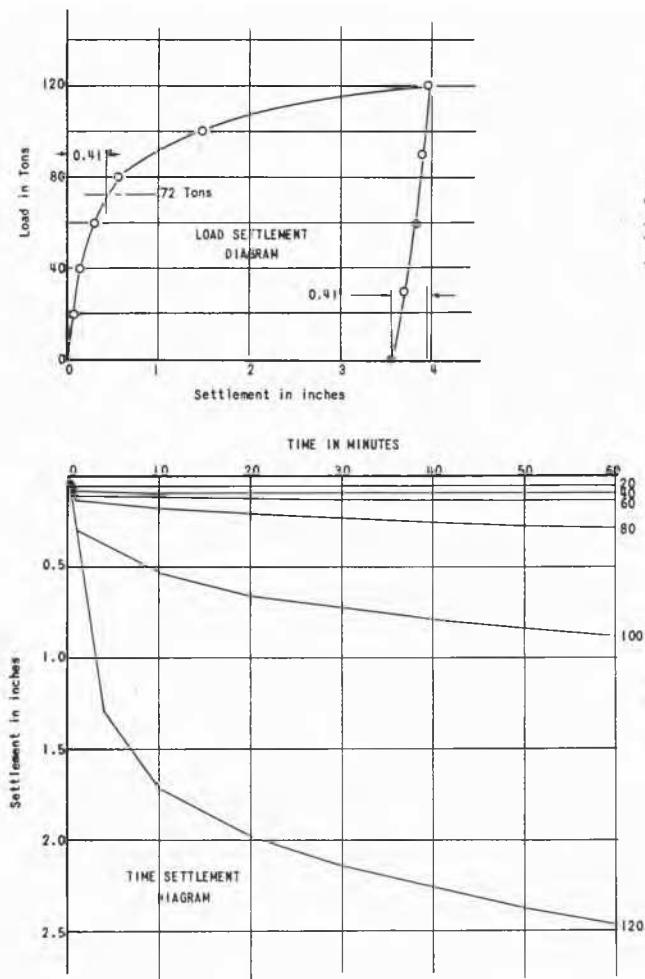


Fig. 48

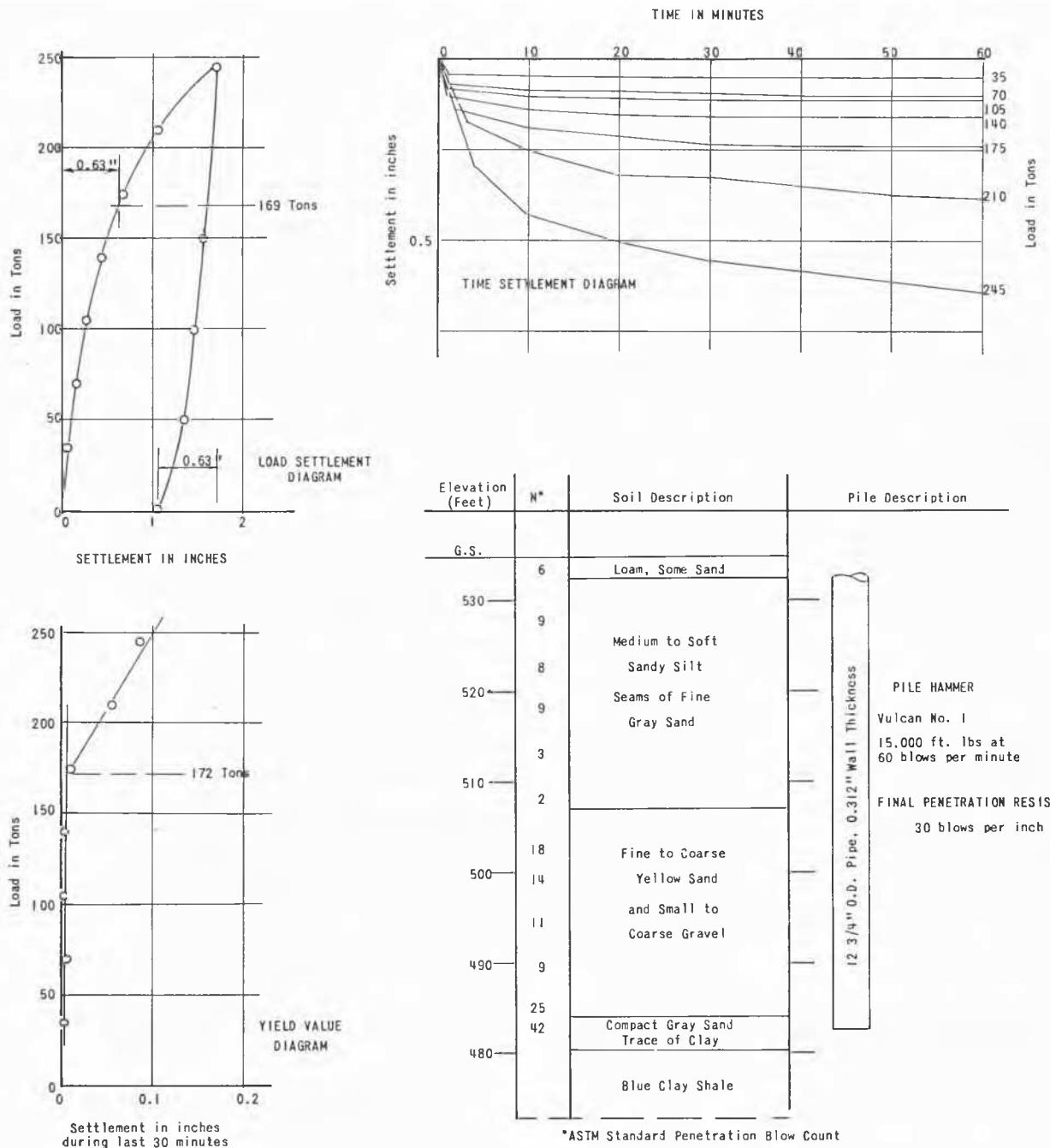


Fig. 49

The load-time-settlement data are summarized on three related charts. Referring to the example graphs, the one designated "Load-Settlement" shows the total settlement at the end of one hour for each load increment. The "Time-Settlement" diagram shows the settlement vs. time, for each of the load increments. On the "Yield Value Diagrams" are plotted values of terminal rates of settlement for each of the increment loads; specifically, the settlement for the last 30 minutes for each load. It will be noted that the terminal settlement rate values are essentially equal to zero until a certain limit load is reached. Thereafter, the settlement rate increases as a direct function of the applied load. By extrapolating backward, through these points on the "Yield Value Diagram", one can determine the load at which yielding commences; this is, the yield point. For the examples shown, these are 76 tons for the pure friction pile and 172 tons for the essentially end-bearing pile.

It is noteworthy that the values in these cases check closely with the elastic limit values obtained from the "Load-Settlement" curves at a settlement equal to the elastic rebound.

Because of limitations on space available, it will not be possible to carry the present discussion beyond this point. The reader is referred to the papers cited for explanation of the load-time control method in all phases of soil testing.

Références

- [1] "Load-Settlement Relationship for Individual Piles under Vertical Load", ASTM Designation D 143-57 T, American Society for Testing Materials, Part 4, 1958.
- [2] "Allowable Pile Loads", City of Detroit Building Code, Article 7, Section 739, 1956.
- [3] HOUSEL, W.S. (1956). "Field and Laboratory Correlation of the Bearing Capacity of Hardpan for Design of Deep Foundations", *Proceedings, American Society for Testing Materials*, vol. LVI; pp. 1320-1 350.
- [4] — (1959). "Dynamic and Static Resistance of Cohesive Soils 1846-1958", *American Society for Testing Materials*, Special Technical Publication No. 254.

M. Y. TCHENG (France)

Le sujet de la discussion proposée est le suivant : « Peut-on déterminer le pouvoir portant des pieux à partir des essais au pénétromètre ? » Ma réponse est oui. En effet les pénétromètres sont des pieux de petit diamètre puisque les pieux ont un diamètre dépassant généralement 30 cm, alors que celui des pénétromètres est de l'ordre de plusieurs centimètres.

M. Kérisel à l'IRABA constate que la résistance en pointe devient constante à partir d'une certaine profondeur qui est de un mètre pour un pénétromètre de 4,5 cm et pour le sable très serré utilisé à Saint-Rémy. Il est encore prématûr de vouloir expliquer ce phénomène. M. Kérisel en déduit que N_q serait une fonction du diamètre du pénétromètre ou des pieux. Mais peut-être le sable sous la pointe, pulvérisé par la charge, perd-il une partie de sa résistance au cisaillement ?

Quoi qu'il en soit, l'interprétation des pénétromètres devient plus compliquée, ainsi que la formule des pieux, quand on aura trouvé cette dernière formule ou plus exactement la juste valeur de N_q , on saura déterminer la force portante des pieux.

Mais comment, à l'heure actuelle, déterminer cette force portante ? Il existe en principe plusieurs possibilités, telles que :

1. Essais de pieux. C'est une solution toujours valable mais onéreuse;

2. Essais sur prélèvements intacts. Dans ce cas, la détermination du pouvoir portant des pieux, à partir des caractéristiques des sols, nécessite la connaissance de formules qui ne sont pas encore établies, comme le montre l'essai de Saint-Rémy;

3. Essais *in situ*, notamment, le pénétromètre. D'après les Fig. 9, 10 et 11 du compte rendu de M. Kérisel, à partir d'une surcharge latérale du terrain, au niveau de la pointe de 4,5 tonnes par m^2 , ce qui est pratiquement un minimum, la résistance en pointe des pieux de diamètre ≤ 60 cm, dépasse 150 kg/cm^2 . On pourra donc faire travailler sans difficulté ces pieux à 60 kg/cm^2 environ. Il en sera toujours ainsi chaque fois que la pointe du pénétromètre accusera une résistance égale ou supérieure à 300 kg/cm^2 .

Si le pénétromètre indique une résistance en pointe de 200 kg/cm^2 la pression des terrains sus-jacents doit varier de 10 à 20 tonnes par m^2 pour obtenir le même taux de travail du béton.

On notera d'ailleurs tout l'intérêt d'utiliser des pieux de même diamètre en pareil cas.

Si la résistance en pointe du pénétromètre atteint au maximum 100 kg/cm^2 , et si l'on désire avoir un certain coefficient de sécurité pour les pieux, il suffira de prendre par rapport à la résistance en pointe du pénétromètre, ce coefficient de sécurité majoré de 25 pour cent seulement.

Dans tous les cas, et compte tenu des résultats d'essais obtenus à Saint-Rémy-les-Chevres, le pénétromètre sera donc un guide très sûr dans la détermination de la force portante des pieux.

M. P. R. TIKUNOV (U.R.S.S.)

Function of elements of total set of pile and their dependency on driving conditions

Experiments have been performed on driving and re-driving of reinforced concrete piles of a length of 9·30 to 14·50 m, cross-section of 30×30 and $35 \times 35 \text{ cm}^2$ into clay soils.

The elastic and final set were noted on a drawing table fastened on the pile. The notes shown in Fig. 51 were made by the setmeter, an instrument where the pencil fastened to a nut moves horizontally along a screw turned by an electric motor.(Fig. 50).

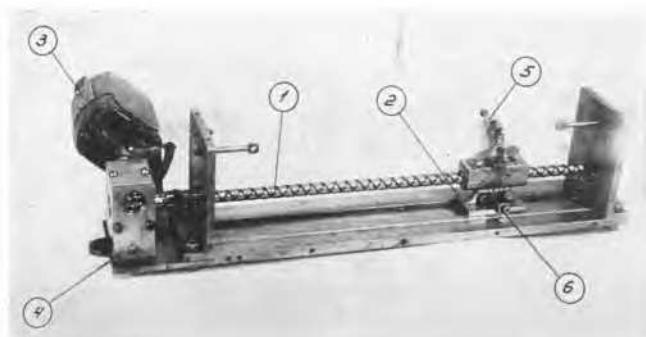


Fig. 50 Setmeter. (1. Screw, 2. Slide-block, 3. Motor 100 W, 4. Reducer, 5. Switch of movement slide-block, 6. Place of fastening pencil to slide-block.)

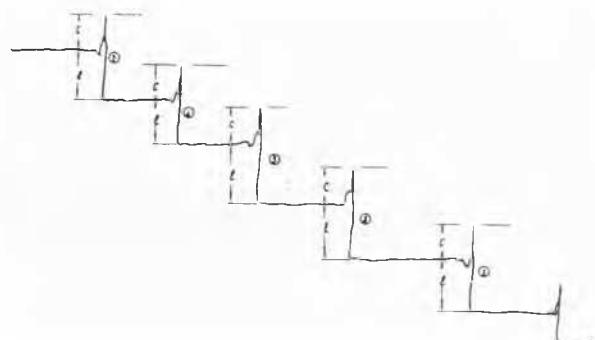


Fig. 51 Diagram of set (final and elastic pacts) of reinforced concrete pile.

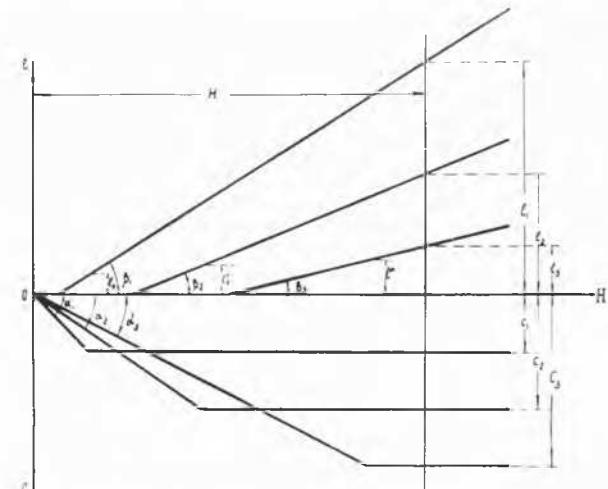


Fig. 52 Curves of final and elastic set for one and the same pile depending on drop of hammer and resistance of pile.

1. Function of e and c on H

Testing of a pile driven to a depth of l_1 , having a resistance of R_1 has shown that increase of the drop of hammer H , leads to linear increase of the elastic part of the set from zero to some value of c_1 . With further increase of H , the value c , remains practically constant.

The final part of set e increases in direct ratio with H .

At the moment when the elastic part of the set reaches maximum value of c_1 the final set reaches a value of e_0 , characterizing the motion of the pile as a result of surpassing the elastic forces of soil resistance below the tip and on the side surfaces.

In the performed tests $e_0 = 0.5$ to 1.5 cm depending on the depth of pile driving.

According to Fig. 52 the function between e_1 , c_1 , e'_0 , and H :

$$\frac{c_1}{\operatorname{tg}\alpha_1} + \frac{e_1 - e'_0}{\operatorname{tg}\beta_1} = H$$

2. Function of e and c on R

If the same pile, with all other conditions equal, is driven to a depth of $l_2 > l_1$ and, consequently, $R_2 > R_1$, we obtain the same function, only with different values of parameters : c_2 , e_2 , α_2 and β_2 , corresponding with the same height of hammer dropping H .

In this case, it was found that for increase of resistance R :

1. Elastic part of set increases ($c_2 > c_1$);
2. Final set decreases ($e_2 < e_1$),
3. Total $e_2 + c_2 < e_1 + c_1$ and $e_2 + \frac{c_2}{2} < e_1 + \frac{c_1}{2}$,
4. Angles of inclination of elastic and final set to axis H decreases : $\alpha_2 < \alpha_1$ and $\beta_2 < \beta_1$.

Similar results were obtained in tests of redriving one and the same pile for different durations (from 18 to 332 hours) after driving, i.e. in conditions of increased pile resistance by time.

3. Function of e and c on Q and $m = \frac{Q}{q}$

Testing by impacts of one and the same pile by weight q with other conditions equal by hammers of different weight of Q_1 and Q_2 , with $Q_2 > Q_1$ have shown that :

1. The maximum value of the elastic part of set c corresponding with the given resistance of the pile R , determined by driving conditions (soil, pile dimensions, etc.) does not depend on Q and does not depend on $m = \frac{Q}{q}$ and, consequently, also does not depend on QH (on observing $e > e_0$).

2. During increase of Q and $m = \frac{Q}{q}$ the final part of set e sharply increases. Angles of inclination α and β simultaneously increase.

3. The higher R , the higher the ratio of values of final set, measured during testing with hammers of high (Q_2) and low (Q_1) weight.

Conclusions

1. The actual characteristic of pile resistance in the soil should not be considered the final set as taken at present, but the elastic part of set should be considered.

The final set, exceeding a value of $e_0 = 0.5$ to 1.5 cm, shows that the elastic part of set has reached maximum value corresponding with resistance of the pile in the given conditions of driving or testing.

2. The following function exists between final (e) and elastic (c) set :

$$\frac{c}{\operatorname{tg}\alpha} + \frac{e - e_0}{\operatorname{tg}\beta} = H$$

where α and β — angles of inclination of lines of elastic and final set to axis H on curve of function e and c on H ,

H — drop of hammer,

e_0 — final set corresponding with reaching of elastic part of set of its maximum value.