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Foundations of Structures

Fondations de Constructions

Piling and Piled Foundations—Les Pieux et Fondations sur Pieux

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D. Lazarević	*Yugoslavia*
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The Chairman

I declare open the sixth session of the conference, and I call first on the General Reporter, P. C. Rutledge.

General Reporter

To introduce the subject of piled foundations I should like to refer to K. Terzaghi's classifications of engineering made in his opening address to the conference. Unlike K. Terzaghi, who divided engineering into two parts, that of the head and that of the hips, I should like to divide engineering into four parts. First, there is theoretical engineering, which is based strictly on the results of theoretical analyses; secondly, there is rational engineering, which is based on rational procedures, but is not based strictly on the results of theory; thirdly, there is intuitive engineering, which is sometimes called engineering art, or in America we say 'seat of the pants' engineering; and fourthly, there is what I call naïve engineering, which proceeds on the basis of over-simplification and elementary formulae, frequently with inadequate basic information, to illogical and sometimes absurd conclusions.

Fortunately for me, as General Reporter, piled foundations present a completely practical subject which is exclusively engineering and which can be discussed in terms of the preceding classifications. I am happy that the more abstract theoretical problems which, as J. Brinch Hansen said yesterday, are becoming too difficult for the soil mechanics engineers to understand fall entirely within the domains of the other General Reporters. In piled foundations, I doubt seriously that any theory will help a pile to support its load.

To begin with an optimistic note, I believe that the trend in design of piled foundations is in the direction of rational engineering, and that it is proceeding in this direction from the two extremes of theoretical engineering and of naïve engineering. This trend seems to me to be demonstrated by many recent papers, including those presented to this conference. As an example, a growing body of published field observations seems to demonstrate that the capacity of piles embedded entirely in clays, even in the stiff and hard clays, is limited to relatively small loads in comparison with theoretical loads computed either from the shear strength of the clay or from the large driving resistances that are frequently encountered in such soils. Further, there is a growing realization that time changes the distribution of load transfer from a pile embedded partially or wholly in soft clay. While it has been largely neglected in theories for load transfer, it seems quite obvious that whenever load is transferred from a pile to soft clay, the resulting stresses in the clay will cause consolidation. The settlements in the clay caused by consolidation with respect to the pile, which does not consolidate, will inevitably cause changes in magnitude and location of load transfer. These changes in distribution of load transfer continue over a long period of time, until finally an equilibrium condition is achieved. Unfortunately, these time effects have been largely ignored in most of the published reports on tests on the so-called instrumented piles in which distribution of load transfer from piles to soft clays has been measured.

Piled foundations are somewhat unique in our general field of foundation engineering in that here we have many commercial organizations with large financial interests in supplying and installing piled foundations. With all due respect to my many friends in these organizations, I believe that they have, to say the least, not discouraged the naïve engineering approach to piled foundations, particularly when they are trying to sell patented types of piles or very large loads on piles. I regret also that some engineers still seem to set aside their normal rational processes when it comes to piles and to assume that the simple act of pounding a pile into the ground automatically assures a safe foundation.

Getting back to rational engineering in the design of piled foundations, I believe that there are three points of major importance.

(1) In order for piles to carry foundation loads satisfactorily the piles must terminate at or in a bearing stratum which is capable of taking the full load out of the piles by skin friction, by point resistance or by both. The determination of satisfactory bearing strata should be made from the results of an adequate number of good exploratory borings. Parenthetically, it is your Reporter's opinion that it is futile even to discuss piled foundations without first having available good subsoil profiles from borings. In general, it is my personal opinion that it is becoming more and more evident that the only truly satisfactory bearing strata for piles are granular soils or sound rock.

(2) With adequate transfer of loads from the piles to a bearing stratum, the performance of a piled foundation is controlled by the materials in the soil profile below the pile tips. It is obvious that, if compressible soil strata exist below the bearing stratum in which the piles terminate, settlements will result. Such compressible strata are not changed in any way by the act of driving piles, and the piles can only be beneficial in minimizing settlements due to compressible strata which they penetrate completely.

(3) Most theories for bearing capacity of piles and practically all load tests on full-scale piles are for single piles. In practice, however, piles rarely occur as individuals and are almost always used in closely spaced groups or clusters. These usually are not tested because of the large loads involved and the rather large cost. Further, load tests are almost always of short duration, whereas the actual loading of pile foundations covers long periods of time. Thus the results of both theories and pile load tests can be used only as guides to engineering judgment and in themselves rarely have any direct significance for foundation design.

I hope that those taking part in the discussion in this Division will keep those three points in mind.

In closing, I wish to mention that the art of engineering comes into the driving of piles. In nature, the bearing strata for piles are rarely uniform in depth, thickness or consistency. Selection of driving equipment and driving procedures which will secure adequate and uniform resistance of piles for a foundation in a bearing stratum without damage to the structural integrity of the piles or to the supporting capacity of the bearing stratum is an art based on experience and judgment. Here I believe that dynamic formulae may sometimes be used as a guide for control of pile driving, even though they may have little relationship to the design load on the piles.

I shall not take your time by repeating the recommended topics for discussion as listed in the General Report for this Division, because I am sure that such repetition would have no effect whatsoever on the many discussions that are already prepared.

G. G. MEYERHOF (Canada)

The interesting paper (3b/6) by M. P. P. DOS SANTOS and N. A. GOMES shows that a reasonable estimate of the ultimate bearing capacity of piles can be obtained from soil tests and simple static theory if the piles are embedded entirely either in clay or in sand of fairly constant relative density. However, where piles are driven through clay into sand or through loose sand into dense sand, the theoretical point resistance in the sand will be too high unless the actual embedment ratio of depth/width of piles within the lower stratum or denser portion is taken into account. From an analysis of published results of pile loading tests, I previously suggested (MEYERHOF, 1951) a minimum embedment ratio of 10, which is supported by Paper 3b/10 by Á. Kézdi. For smaller embedments the theoretical point bearing capacity factor was shown to be reduced roughly in direct proportion to the embedment ratio. Moreover, the average soil properties to be used in estimating this point resistance are those within the failure zone, which theoretically depends on the friction angle ϕ and extends from three or four times the pile diameter above base level to a depth of about one pile diameter below the base (MEYERHOF, 1952). The importance of these limits of the governing soil strength near pile points is fully borne out by the analyses contained in Paper 3b/15 by C. VAN DER VEEN and L. BOERSMA and Paper 3b/16 by A. F. VAN WEELE and should lead to a closer estimate of the static bearing capacity of piles resting on sands.

References

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B. HANSEN (Denmark)

I should like to raise a few points concerning the bearing capacity of single piles in sand.

It can easily be agreed that a proper understanding of this problem can only be obtained if a reliable method of computing statically this bearing capacity can be found. In this connection it is especially interesting to note that the results of H. O. IRELAND (3b/9) indicate much higher values of skin friction in sand than have previously been assumed. This is in accordance with recent loading and pulling tests which have been made in Denmark in connection with the consultative practice of the Danish Geotechnical Institute. We have found factors of skin friction of the same order of magnitude as those given in the paper by H. O. Ireland, but I do not think it is desirable to express them by means of a Rankine passive earth pressure multiplied by tan ϕ. This can be confusing because Rankine's earth pressure is computed under the specific assumption that the wall is smooth. The passive earth pressure on a rough wall is much higher, so that the results cannot properly be interpreted in this way.

The scatter of the test results round the bearing capacities computed statically seems to be of the same order of magnitude as that found by the dynamic pile driving formulae. Although the determination of bearing capacities by means of a pile driving formula is not theoretically a very satisfactory method, it seems to be one of the most reliable. The accuracy of this method can be further increased if a loading test is performed so that a correction factor can be introduced in the dynamic formula. The standard deviation for the piles on a fairly homogeneous site will then be appreciably smaller than indicated in Paper 3b/13.

It seems to be that almost every comparison between loading tests and pile driving formulae until now have resulted in the proposal of a new pile driving formula. Perhaps it is not surprising, therefore, that T. SØRENSEN and myself (3b/13) have found it expedient to do exactly the same thing. It was our intention to find a formula which was as accurate as the best of the existing ones and as easy to use in practice as possible.

The dimensionless considerations in our Paper 3b/13 make it possible to obtain very simple driving criteria in order to avoid over-driving and crushing of the pile. From the graphs it can be seen that a pile ought not to be driven harder than corresponding to $Q/Q_0 = 0{\cdot}9$, where

$$Q_0 = \left(2\alpha WH\frac{AE}{L}\right)^{\frac{1}{2}}$$

To Q/Q_0 greater than 0·9 correspond very small settlements per blow, so that the driving is uneconomical and in extreme cases impossible. If Q/Q_0 is less than about 0·9 it can also easily be shown by considering the shock waves in the pile that the maximum compression stress in the pile is given by $\sigma = Q_0 W_p^{\frac{1}{2}}/AW^{\frac{1}{2}}$. If σ becomes greater than the strength of the pile material the pile top will be crushed. For smaller values of σ crushing can also occur if Q/Q_0 is too great. The shock waves can then be reflected from the pile tip so that crushing can occur there. Taken together, the two criteria give for each separate case upper limits for W_p/W and H, which have to be obeyed if it is to be possible to drive the pile to the desired bearing capacity without crushing it during the process.

R. L'HERMINIER (France)

Le pénétromètre et le pouvoir portant des pieux.

Pour évaluer le pouvoir portant des pieux, l'ingénieur géotechnicien dispose de deux méthodes générales: ou bien prélever des échantillons intacts dans les différentes couches rencontrées; ou bien utiliser un pénétromètre à cone.

Dans le premier cas, on détermine les caractéristiques mécaniques, et en particulier l'angle de frottement interne et la cohésion de chaque sol échantillonné, et à partir de ces renseignements précis, on calcule le pouvoir portant du pieu en utilisant les formules qui apparaissent les plus appropriées. Or ces formules sont en pleine évolution. Jusqu'à ces temps derniers, l'effort en pointe était fixé à l'aide des formules établies pour les fondations quasi-superficielles. Depuis la publication faite par G. G. Meyerhof dans la revue *Géotechnique* de Décembre 1951, on a reconsidèré complètement le problème et les formules nouvelles actuelles donnent pour la pointe une contrainte de rupture beaucoup plus élevée. D'autres formules apparaissent d'année en année, de telle sorte que si le point de départ (caractéristiques mécaniques du sol) est certain, le point d'arrivée (pouvoir portant des pieux) peut preter à discussion. D'autre part, les pieux sont en général utilisés pour asseoir une fondation, par leur intermédiaire sur une couche résistante plus on moins profonde. En France, cette couche résistante est souvent constituée par des alluvions anciennes sablo-graveleuses. L'angle de frottement de ces alluvions est élevé et difficilement déterminable en laboratoire. Il suffit que l'erreur soit de 2° à 3°, pour que le pouvoir portant évalué à l'aide des formules de pointe, varie du simple ou double.

Enfin, le sol est presque toujours hétérogène et plus ou moins stratifié. Et aucune formule, à ma connaissance, ne donne le pouvoir portant d'un pieu dans un tel milieu.

L'utilisation d'un pénétromètre à cône supprime ou presque les difficultés que je viens de signaler. Le pénétromètre garde le secret des formules qu'il utilise pour déterminer l'effort en pointe et le frottement latéral, mais les résultats sont exacts.

Toutefois une autre difficulté se présente. Comment passer du pouvoir portant d'un petit pieu de 3 à 6 cm, de diamètre, à celui d'un pieu ordinaire, normalement utilisé dans les travaux de fondation?

J. Kérisel nous a montré hier que le terme N_q (je ne reviendrai pas ici sur sa définition que tout le monde connait), était à la fois fonction du diamètre du pénétromètre et de la profondeur de fiche dans la couche résistante. Ces résultats n'ont rien d'étonnant. Et déjà, au cours des journées de *Mécanique des sols* à Paris en juillet 1952, j'avais donné une formule de pointe où le terme N_q était fonction du rapport D/H, D étant le diamètre du piue ou du pénétromètre et H la fiche. Mais j'obtenais, si j'ai bonne mémoire, des chiffres inférieurs à ceux présentés, par J. Kérisel pour les petits diamètres. Les considérations qui précédent indiquent que le terme N_q est effectivement une fonction, non pas seulement de l'angle de frottement interne, mais également du rapport D/H, et il n'est pas exclu que d'autres paramètres apparaissent ... Il doit résulter de la mise au point de facteurs correctifs une plus grande précision dans l'emploi du pénétromètre.

Quoi qu'il en soit, les expériences présentées par J. Kérisel montrent également que l'eventail des valeurs de N_q correspondant aux differents diamètres, se referme très nettement lorsque la fiche H croit.

Il arrive même que pour une valeur suffisamment élevée du diamètre du pénétromètre (60 mm par exemple), les différences entre le N_q des pieux et celui mesuré au pénétromètre sont négligeable à l'échelle de nos connaissances en matière de pouvoir portant des pieux, sous réserve qu'une certaine profondeur d'ancrage des pieux dans la couche résistante soit atteinte: c'est cette valeur particulière de H, que nous avons appelée 'hauteur critique', en 1952.

A. Caquot et J. Kérisel donnent d'ailleurs dans l'édition 1956, de leur *Traité de Mécanique des Sols*, les valeurs que l'on peut adopter pour la hauteur critique, en fonction d'une part de l'angle de frottement de la couche résistante, où est fichée la pointe des pieux, et d'autre part, du diamètre des pieux.

Les considérations générales qui précédent sont illustrées par des essais effectués en 1956 au Bec d'Ambès, pres de Bordeaux.

La coupe du terrain est la suivante: tout d'abord 14 m de vase très peu cohérente; puis 7 m d'alluvions anciennes sablo-graveleuses assez résistantes; enfin des marnes compactes.

Les ouvrages envisagés devaient être fondés sur des pieux ancrés dans la couche sablo-graveleuse.

A la suite d'une campagne de reconnaissance effectuée au pénétromètre de 60 mm de diamètre, l'emploi de pieux battus fût decidé. L'angle de frottement de la couche sablo-graveleuse fût évalué à 31°, et la 'hauteur critique' correspondante estimée à 1 m 60. Il en résulta pour les pieux une fiche totale de 15 m 60.

En accord avec le maitre de l'œuvre, ces conclusions, déduites des seuls essais de pénétration, furent vérifées par des essais de battage et des essais de fonçage sur pieux réels de $0{\cdot}30 \times 0{\cdot}30$.

Un pieu fût donc battu au voisinage d'un essai de pénétration jusqu'à la cote $-15{\cdot}60$. Le raccourcissement élastique du pieu et du sol fût mesuré à l'aide de fléximètres et l'interprétation du battage réalisée par application de la formule de Hiley, dont B. Hansen et T. Sørensen, ont montré tout l'interêt dans leurs interventions au cours du présent Congrés. La force portante du pieu ainsi évaluée fût de 125 à 130 tonnes.

Après battage le pieu fût foncé à l'aide d'un vérin prenant appui sur des fers I.P.N., fixés à des charges voisines. La courbe efforts déformations, rectifée compte tenu du raccourcissement élastique du pieu et du sol, montra que la rupture pouvait être considèrée comme atteinte après un enfoncement du pieu dans le sol d'environ 2·5 cm. Cette courbe, interprétée à l'aide de la méthode exposée pas C. van der Veen au cours du 3 ème congrès international de Zurich, donna un pouvoir portant de 123 tonnes.

Or, on prenant l'enveloppe des valeurs minima données par l'essai de pénétration dans la traversés de la couche sablo-graveleuse (il s'agit de l'essai de pénétration éxécuté au voisinage immédiat des essais sur pieu de $0{\cdot}30 \times 0{\cdot}30$) la résistance de pointe à 15·60 m de profondeur était de 128 kg cm², ce qui rapporté à la section du pieu réel fournissait 115 tonnes environ. D'autre part le frottement latéral était d'environ 1 tonne, à la même profondeur de 15·60. En admettant des frottements latéraux proportionnels aux diamètres, on arrivait ainsi par le seul éxamen du diàgramme de pénétration à une force portante globale de 120 tonnes, tout à fait comparable à celle obtenue soit par battage soit par fonçage.

En conclusion, nous pensons que l'essai de pénétration doit donner avec une approximation très satisfaisante le pouvoir portant des pieux ancrés dans une couche résistante sous réserve toutefois que d'une part, le diamètre du pénétromètre soit suffisant, et que d'autre part les pieux soient ancrés dans la couche résistante, d'au moins la 'hauteur critique'.

Toute autre méthode basée sur l'emploi de formules apparait à l'heure actuelle beaucoup moins sûre.

D. Lazarević (Yugoslavia)

Je voudrais donner quelques remarques sur le problème de variations des modules de déformations du sol avec la profondeur.

Il semble légitime, en tenant compte de l'influence des poussées du sol au repos sur la déformabilité des couches, de prendre en considération la possibilité des distributions curvilignes des poussées — non seulement hydrostatiques, comme on fait habituellement. Le sol sollicité, qui se tasse, se trouve dans l'état cinématique et le coéfficient de poussée du sol au repos ne correspond pas à l'état physique du phénomène.

Si on peut supposer existence de la similitude au point de vue de comportement statique des sols rocheux et des bétons, on peut former une simple loi de variation des modules de déformation en fonction de la profondeur.

La liaison, connue comme la plus simple au point de vue d'algèbre, parmi la resistance de rupture et de module d'élasticité, est éxprimée par la loi deduite des résultats statistiques

$$E_o = A(\sigma_n)^{\frac{1}{2}} \qquad \ldots . \; (1)$$

Dans cette expression A est une constante, qui peut être déterminée en mesurant sur place $\bar{E}$ et $\bar{\sigma}_N$ à la condition correspondante, d'où on obtient $A = \bar{E}/(\bar{\sigma}_n)^{\frac{1}{2}}$.

La courbe intrinsèque du béton est représentée par l'enveloppe des cercles de Mohr. An exploitant les résultats des nouvelles recherches dans le domaine des résistance des sols rocheux, on peut conclure que ces matériaux aussi ne suivent pas rigouresement le diagramme Coulomb–Mohr. Si on peut être permise la simplification, pour pouvoire obtenir une idée de la variation du module de déformation en fonction de la profondeur — on peut calculer les valeurs σ_n, expression 1, de la condition de rupture, deduite du diagramme Coulomb–Mohr

$$\sigma_n = 2C\frac{\cos\phi}{1-\sin\phi} + K_o z\gamma\frac{1-\sin\phi}{1+\sin\phi} \qquad \ldots . \; (2)$$

Dans cette expression toutes les notations sont bien connues. La valeur K_o est toujours considereé constant et hyperstatique. Dans les cas des sols au repos la valeur K_o peut être considereé constante. En exploitant les conditions que les sections verticales planes, sous l'action des forces de pesanteur restent verticales planes, on obtient l'expression connue

$$K_o = \frac{\mu}{1-\mu} \qquad \ldots . \; (3)$$

Sous l'action des forces extérieures, provenant des fondations, la valeur K_o ne peut pas être calculée de l'expressions 3,

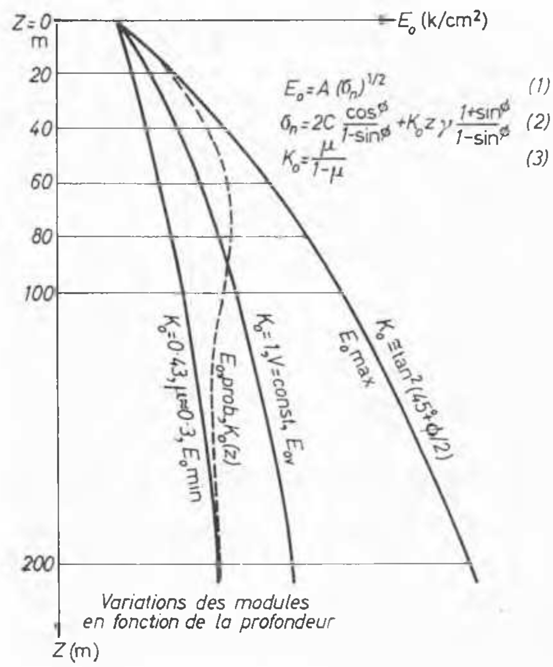


Fig. 1

parceque les sections verticales ne resteront plus planes, et les valeurs K_o se rapprochent vers le coéfficient de la butée.

Dans le cas donné les valeurs: C, ϕ, γ, μ, $E_{z=o}$ peuvent être mesurées sur place. En exploitant les expressions 1 à 3, on peut déterminer la ligne qui exprime la dépendence du module de déformation et la profondeur (z). Sur la Fig. 1 sont représentés en traits pleins trois différents modules de déformation, calculés dans l'hypothèse: $\mu = 0{\cdot}3$, $K_o = 0{\cdot}43$; $\mu = 0{\cdot}5$, $K_o = 1$ et $K_o = \tan^2(45° + \phi/2)$; $\gamma = 1{\cdot}8$ t/m², $\phi = 30°$. La ligne interrompue représente la variation probable du module de déformation sous l'action de surcharge provenant des fondations et de la pesanteur du sol même. Cette courbe se trouve entre la courbe $E_{o\max}$, quand il est $K_o = \tan^2(45° + \phi/2)$, et $E_{o\min}$ calculé dans l'hypothèse $K_o = a(1-\mu)$, en prenant pour le coèfficient de Poisson la valeur mesurée sur place. La courbe entre ces deux représente le cas idéalisé, qui concerne les matériaux incompressible, dans l'état des tensions hydrostatique: $\sigma_x = \sigma_y = \sigma_z$.

La ligne pointillée représente E probable. L'allure de cette ligne dépend des circonstences physiques dans chaque cas spécial, et surtout dépend du jeu de tensions horizontals hyperstatiques. A présente époque il semble qu'on peut obtenire une idée d'évolution du jeu des forces internes seulement en mesurant sur place les déformations qui caractérisent le phénomène.

Pour pouvoir suivre quelques déformations intéréssentes, nous sommes au cours d'étude de deux appareils speciaux. Dans le même but nous exploiterons notre *presse radiale*, en augmentant leur capacité de 2000 à 3000 t. Tous ces trois appareils vont être décrits une autre fois. Les noms des appareils donnent l'idée de leurs fonctions. Nous avons surnommé un d'eux *dilatomètre du sondage*. Il est dans son essentiel une presse radiale, qui se met en fonction dans un trou foré de petit diamètre, et fonctionne à distance. Nous avons surnommé l'autre *pénétromètre expansif*.

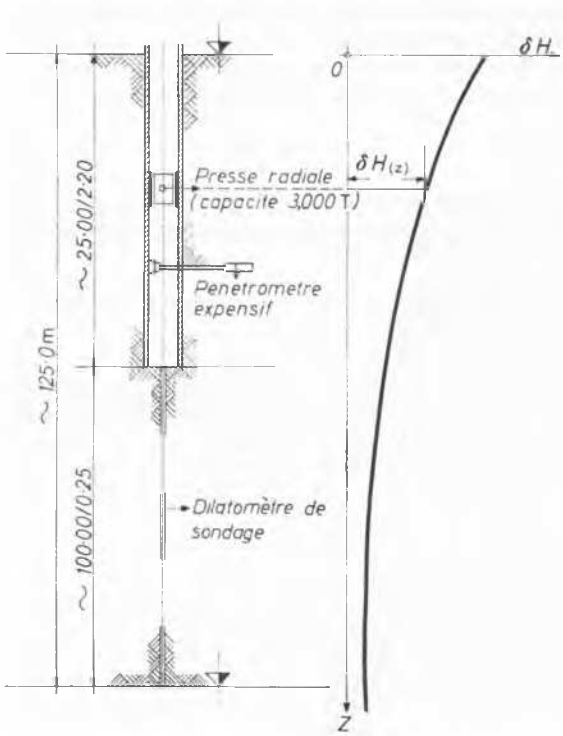


Fig. 2 Dispositif des appareils differants pour essayer les modules
Test arrangements for the determination of moduli

La presse radiale et le pénétromètre expansif sont prévus pour les mesurements dans les zones les plus intéressantes — une vingtaine des mètres sous les fondations du barrage. La dilatomètre du sondage est prévu pour les profondeurs jusqu'à cent mètres. Il nous semble probable à obtenir, par mesure appropriées, la variation pratiquement réele des modules des couches du sol à la profondeur oscultée. Par éxtrapolation, nous pourrions estimer les valeurs dans le domaine intéréssant pour le comportement du barrage.

Sans tenir compte de la variation du module de déformation avec la profondeur — on obtient par le calculs habutels, quelque fois, les tassements qui gênent non seulement les élements géometriques du barrage, mais aussi les bilans des eaux. Il arrive, aussi, quelque fois que les tassements mesurés ne sont pas même semblables aux tassements calculés.

Il semble possible de donner une explication logique sur quelques problèmes de force portante des pieux, en tenant compte des variations non linéaires des poussées de terre latérales. Ces poussées augmentent par frottement sur la pourtour les résistances à l'enfoncement, surtout dans les régions des pieux où se manifeste vivement la valeur de $K_{o(z)} \approx K_p$.

Les recherches mathématiques sur ce problème ne sont pas encore très poussées. L'idée des mathématiciens à classer ce problème comme un des multidimensionels, du domaine des problèmes de Riemann, nous semble pour le moment une très intéréssante spéculation intelectuelle. Les ingénieurs doivent chercher les sollitions, quoique brutalement approximatifs, qui éviteront evidemment les inconvenients des choses imprévues.

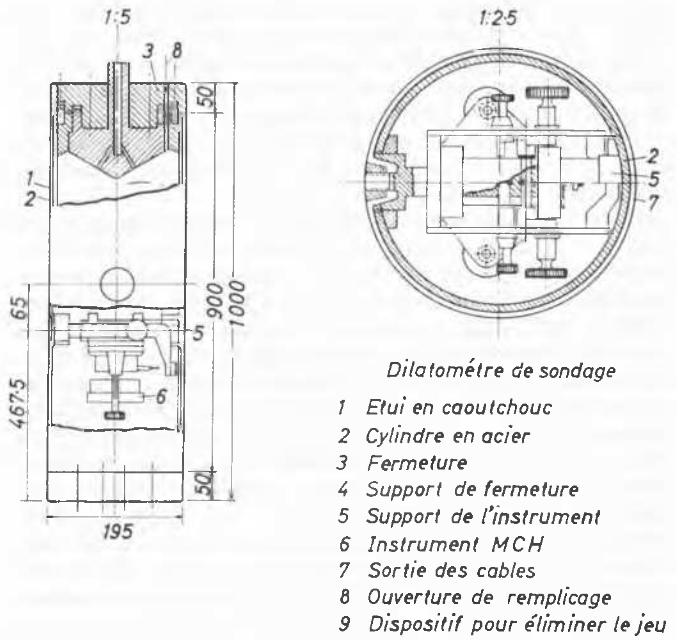


Fig. 3

J. Feld (U.S.A.)

My remarks will relate to Papers 3b/10 by Á. Kézdi and 3b/11 by G. Plantema and C. A. Nolet. The author's reported conclusion that a group of friction piles can safely support more than the individual pile value per pile with equal settlement when pile spacing is less than six times pile dimension cannot be accepted without further proof. It is contrary to present-day practice, based on many full-scale load tests, summarized in Chellis' *Pile Foundations*, that the experiments described by the author must be considered as a special case not typical of actual practice.

The statement made by the author: 'The less the distance between pile centres, the smaller will be the settlements under a given load' is also contrary to recognized action and, if true, would void all specifications and codes limiting minimum pile spacing (in friction piles) to twice the pile diameter, or 30 in. with reductions in pile values if such spacing is not provided. The large-scale tests by Frank M. Masters of wood pile clusters in soil of very similar gradation to the fine silt-sand used by the author showed results just the opposite to the conclusions given in this paper.

The paper does not indicate whether the pile group loadings were on single piles or on the entire group. If the results are the summation of individual pile loadings, such technique may be a clue to the unexpected conclusions.

One explanation is probably the small scale of the tests. The silt-sand in the space of from 10 to 50 cm between pile faces can be small enough to wedge the fill and prevent differential movements of adjacent piles. Normal gap between piles is of the range of 40 to 60 cm. Another factor may be the small pile loadings, 2 to 5 tons per pile.

Until further tests substantiate the author's conclusions, it would be unwise to use them in actual foundation designs.

Reference is also made to Paper 3b/11 to the conclusion that around the pile at a distance of 1·00 m the influence of the pile driving is less marked in increasing the cone resistances of the soil layers at and below the pile tip level.

The results in sand consolidation at and below the pile tips are reasonable and conform to expected soil action. However, the time factor must also be considered and some experience in similar tests indicates that some if not all of the soil strengthening dissipates quite rapidly.

In a large supported highway project 18 in. steel pipes, ½-in. thick, with cast steel conical points were driven with a No. 0 Vulcan hammer to an indicated safe bearing of 80 tons, to a depth of 50 to 60 ft. The soil was a varved silty sand of great depth covered with some 10 ft. of medium sand. Static load tests places on several piles within days after driving showed all piles to be satisfactory under 120 and 160 ton loadings. Upon further driving, some erratic depths of resistance penetration indicated a necessity for re-evaluation of previously accepted piles. Load tests on the piles previously found satisfactory both by driving resistance and by static load test were now (at an age of about a month) unsatisfactory. The net settlements under static loading applied in increments to a maximum of 80 tons were far beyond permissible limits. The piles were then re-driven with a Vulcan No. 00 hammer and accepted upon driving resistance requirements only.

Borings taken close to driven piles generally indicated spoon resistance greater than reported in the borings made before work started. Similar indicated loss in bearing value of steel *H* piles driven in fine marly sand has been noted on a recent project where the exterior piles of a group were found to carry static test load with less net settlement than interior piles, and load tests at 24 hours after driving were better than on piles already subjected to vibration from subsequent pile driving.

L. van der Veen (Netherlands)

I should like to draw attention to Paper 3b/11 by G. Plantema and C. A. Nolet concerning the influence of pile driving on the resistance of the soil as measured, for instance, by the Dutch cone penetration test. Undoubtedly, in the prediction of the bearing capacity of a pile to be expected after such a penetration test, the influence of the pile driving on the bearing capacity of the soil is of great importance. The paper is interesting as it gives some data on this influence.

As we have to deal with a very complicated problem, I doubt whether it will be possible to predict the influence of the pile driving on the cone resistance which is measured before the piles are driven. I have dwelt on this subject in paper 3b/15, in which are compared loading tests on piles with cone penetration tests. I am sorry that L. Boersma and I were not clear enough in that paper, as the General Reporter states that it was not made clear whether the cone measurements were made before or after pile driving. Actually, all penetration tests were made before pile driving. The influence of the pile driving is to be derived statistically from the comparison of the actual bearing capacity determined by pile loading tests and the bearing capacity which is predicted from the cone penetration test. It appears that nearly all the test results are contained within the limits of 50 and 150 per cent of the bearing capacity predicted from the cone penetration test, according to the method described in my paper. That means that the influence of the pile driving on the bearing capacity of the pile is included in the measured accuracy of 50 per cent more or less. As the average of the test results is about 100 per cent of the bearing capacity to be expected from the cone penetration tests, it seems that the influence of the pile driving is not of such great importance as is to be derived from the test results of G. Plantema. Nevertheless, it seems to me essential that more test results of this kind should be made available, so as to give a better understanding of this important phenomenon.

The second comment that I should like to make concerns the paper of N. B. Hobbs (3b/8), who describes the influence of artesian flow on cast-in-place concrete piles without shells. As a result of this artesian flow, necking of a large number of the piles was observed in a loose sand stratum near the top of the piles. I think that N. B. Hobbs draws our attention to a very important thing—the failures which can occur in the concrete of cast-in-place piles.

In this connection I might give a short comment on the statement of the General Reporter, who concludes: 'The remedy is, of course, to cast the piles in shells with sufficient strength to maintain the original open hole and to support fully the concrete of the pile.' I agree that, by doing so, no influence of artesian flow is to be feared. I must emphasize, perhaps superfluously, that the application of such a shell does not necessarily guarantee in every case a good concrete pile, as the artesian flow is not the only circumstance which can cause weak spots in the concrete pile.

Recent investigations in the Netherlands have pointed out that when casting concrete in a tube, even when this tube does not exceed a length of 10 to 15 m, and especially when reinforcement has been put in it, partial segregation of the concrete and even the forming of holes in the pile, can occur. To avoid this, the results of these investigators seem to indicate that some vibration of the concrete, in one way or the other, is necessary.

E. C. W. A. Geuze (Netherlands)

I have been asked by H. K. S. P. Begemann and Mr. Heynen, of the Delft Soil Mechanics Laboratory, to give you a brief summary of their comments on Paper 3b/16 by A. F. van Weele. Those conclusions are:

(1) Fig. 6b (right-hand side) shows the total resistance along the sounding tubes (circumference 11·2 cm) and the sum of the local skin friction, the latter having been measured with the friction jacket cone (see *Proc. 3rd Conf.*, Vol. I, p. 213).

The curve of the sum of the local skin friction is correct down to the sandy layer, since it has been found that the angles of internal friction between clay and iron, and clay and concrete are practically identical. However, this is not the case in sand. For smooth concrete the angle of internal friction fluctuates around 20 degrees; for iron and tightly packed fine sharp sand the angle of internal friction may increase to 33 degrees. In connection with this and the speed of sounding, the local skin friction values found in the sandy layer ought to have been reduced to about 55 per cent of the values measured.

(2) It is important to point to the close agreement between the *maximum* friction *measured* under the test load and the maximum friction calculated from the *local* skin friction. At the level of extensometer II, a total maximum friction of 14·8 ton was measured at a pile load of 175 ton, whilst the sum of the local skin friction gives 15 ton.

At the level of extensometer III (still in the clay layer) 33 ton was found for both, whilst for the total pile length these amounts are 60·8 ton and 66·8 ton respectively (only this latter amount was calculated with the corrected local skin friction values in the sand).

(3) After correction η is found to become 0·27 for the curve showing the sum of the local skin friction, whilst from the load test, after *rectilinear* interpolation between the measuring points, an η value of 0·255 is obtained. For the measured total friction curve $\eta = 0{\cdot}247$ was found.

In this connection it may be pointed out that in this case the difference between η calculated from the measured total friction and η calculated from the sum of the local skin friction is not so very much. However, for other cases the differences may be considerably greater. For instance, values of respectively 0·22 and 0·34 have already been found. In A. F. van Weele's final equation these differences in η may give deviations in the calculated skin friction of, for instance, 10 to possibly 18 per cent, or more.

(4) The method of calculation developed in the article can only be used if no residual stresses remain in the pile and the pile point after the load has been removed. With the test loads applied in Amsterdam these stresses proved to be almost absent, but this will not necessarily be the case in all circumstances. A noteworthy point is that, according to Fig. 3, a pile point settlement of 3·1 mm was needed for the development of the maximum skin friction along the entire pile, whilst the elastic bounce of the point was practically as large, viz. 3·4 mm.

(5) In connection with the above remarks, it seems desirable to start from an η value calculated from the sum of the local skin friction measured with the friction jacket cone.

Furthermore, measurement of the local skin friction makes it possible to investigate whether the results obtained with the equation developed by A. F. van Weele have led to incorrect results, perhaps as a result of residual stresses in the pile. If these are major differences this will definitely have to be the point for investigation.

I should like now to proceed to make some remarks in connection with that contribution and also those made earlier by C. van der Veen and other members of the conference on the effects of residual stresses in the soil on the behaviour of a pile during a loading test.

After pushing a pile into the soil down to its ultimate depth, residual stresses will have been developed acting between the pile surface and the surrounding soil. Their magnitudes will be determined mainly by the elastic properties of the pile and those of the soil. Dividing the resultants of these stresses as usual into a part acting on the pile point and another part acting on the lateral surface of the pile, it is obvious that before a loading test is carried out the sum of the resultants will equal the weight of the pile. The fact that by the penetration of the pile the radial effective stresses in the cohesionless material around the pile point will be greatly increased is of considerable interest in the case of the settlement of end-bearing piles in sand layers of low to medium density. In Paper 3b/11 by G. Plantema and C. A. Nolet this fact is clearly demonstrated by the results of subsequent penetration tests.

In a paper to the Paris Conference in 1952 I gave a series of results obtained by loading tests on piles of different diameter. The results showed that the settlement curves of the pile points in relation to the forces acting on those points all had the same shape, and that their average slopes increased in direct proportion to their diameter.

In taking into account the residual stresses at the pile points, that is, the stresses which did not produce any settlement under subsequent loading, I arrived at two conclusions:

First, the settlement curves proved to be perfectly reproducible, that is, any of the curves of a series belonging to different diameters could be obtained from another by the relationship:

$$S = a\frac{P-P_R}{P_L}D \qquad \ldots. \ (1)$$

where S represents the settlement of the pile point, $P_R < P < P_L$ represents the vertical force acting on the pile point, P_L represents the same force related to some arbitrarily chosen constant speed of penetration, D represents the diameter of the pile point, and a represents a constant, representing the stress deformation characteristic of the pressure bulb surrounding the pile point.

Secondly, the validity of these relationships is limited to the range of pile point forces producing instantaneous deformations in the sand layers and not to the range of forces involving subsequent partial breakdowns in the stressed area, which has been so aptly described by G. de Josselin de Jong as the 'hesitant' part of the deformation process. The range of validity has been indicated by the symbol P_E. It proved to be roughly equal to one half of the magnitude P_L with a slight tendency to decrease with increasing diameter.

We may write equation 1 as:

$$\epsilon = \frac{S}{D} = a\left(\frac{P-P_R}{P_L}\right)$$

In this way we may expect that the model law, which apparently can be applied to deep foundations, may provide us with a working basis for a prediction of settlements of piles within the range of allowable deformations, in which we are mostly interested, if the behaviour of the model is perfectly known and if the soil conditions warrant their application. Needless to say, many checks and further research will be needed before this stage will be reached. Therefore, the study of the behaviour of piles in the field should be encouraged, but at the same time the results should be checked with the predictions furnished by the available applied theoretical methods and the results of investigations on a semi-technical scale obtained *in situ*. If not, the value of these case records might be greatly reduced.

H. Petermann (Germany)

I should like to give the results of tests made by a research group consisting of my colleagues E. Lackner, H. Schenck and myself on the bearing capacity of thin reinforced concrete piles for house foundations.

In the North-west German coast area there are often diluvial and alluvial sands covered with geologically young, still soft, deposits of clay, mud and peat. In the Bremen area these compressible layers have a thickness of between 2 and 6 m: underneath are fine and medium sands. If with these soil conditions prevailing we are not willing to choose a footing foundation because of the settlements which must be expected due to the compressibility of the upper layers, we must arrange for a piled foundation. In Bremen the length of the piles in such cases is often only 6 m or less.

The loads are quite small in housing; for example, in the case of medium sized buildings the foundation load is only 6 to 8 t/m. With pile foundations only relatively small pile loads result as the distance between the piles cannot be of any size one chooses. It would not be economical to employ the usual piles, for example of 30 × 30 cm, for such small loads since their bearing capacity could not be fully utilized. Therefore, in Bremen, we use for the housing foundations reinforced concrete piles of 20 × 20 and 25 × 25 cm. No standards exist for the admissible load on these thin piles. Such piles are scarcely known in other areas as foundation elements.

When estimating the bearing capacity of these thin concrete piles we have relied up till now on practical observations and on experience with foundations that have been carried out up to the present. The economics of housing construction today make it necessary to make full use of building material and building elements while at the same time guaranteeing sufficient safety. Now, if we want to make economic use of thin concrete piles it is necessary to be fully informed of their characteristic qualities which concern handling, piling technique and their static and dynamic strength. Determinations can be made regarding handling, and the reaction of the piles to dynamic driving. However, on the question of admissible load, the necessary length of embedment in the bearing sand,

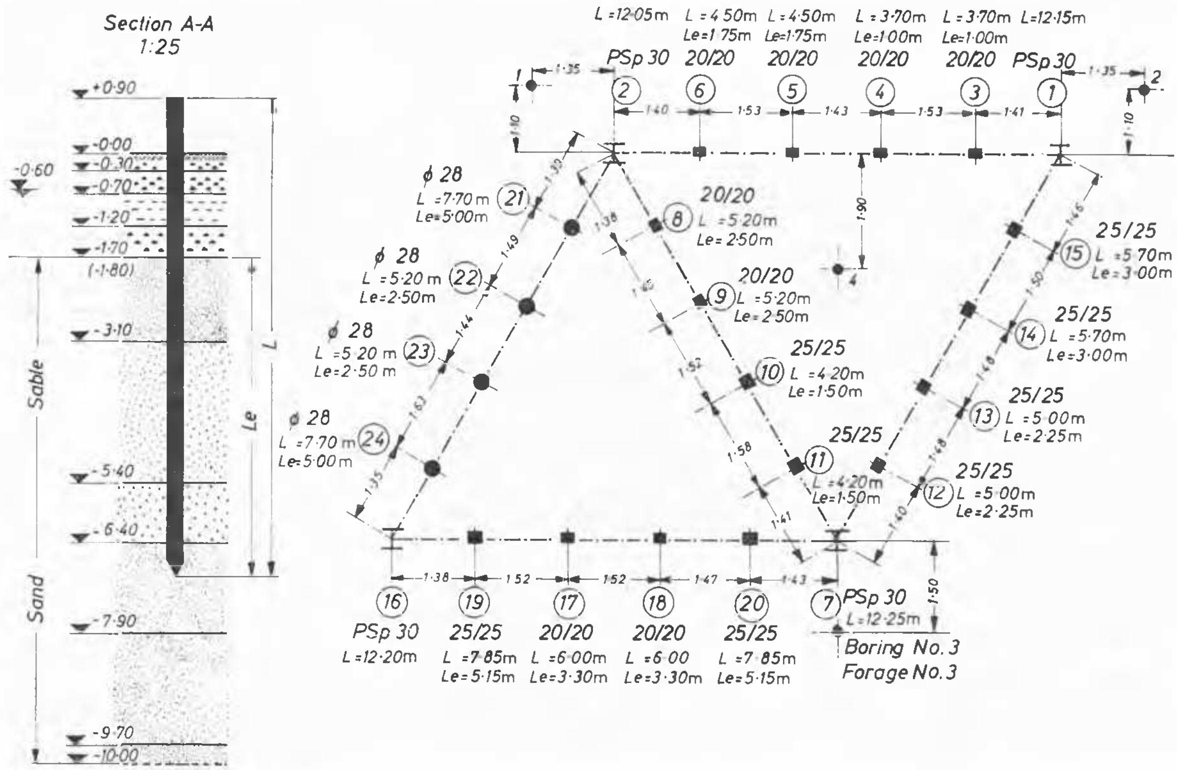


Fig. 4 Arrangement of the test piles
Arrangement des pieux d'essai

and the amount of acceptable settlement under load, exact recommendations can be gained only from tests on a large scale.

There is need for guiding directions for the estimation of bearing capacity and admissible load on thin concrete piles of 20 × 20 and 25 × 25 cm, and an extensive series of tests with such piles has therefore been carried out. The arrangement of the test piles is shown in Fig. 4. Piles of 20 × 20 and 25 × 25 cm, 4 to 8 m length, and with embedded lengths in the bearing sand of 1 to 5 m, were first driven and controls thereby obtained,

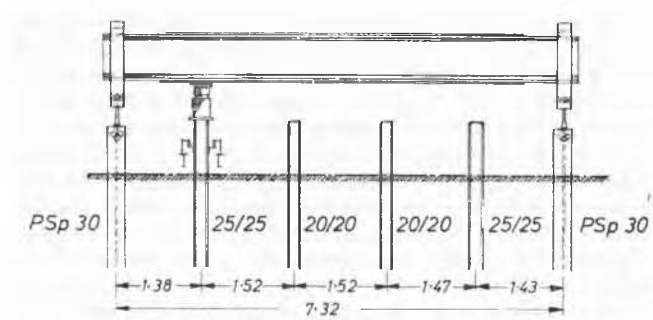


Fig. 5 Test equipment
Equipment d'essai

then test loadings were made and finally they were pulled out again. Fig. 5 shows the arrangement for testing. Test loadings were made shortly after driving, and then 3 months, 6 months and one year later. The ground water level was normal, that is to say about ½ m below the surface. At a time of low ground water 4 piles were separately driven and test loaded: these piles were pulled out when ground water was normal, as a test. Drillings, sounding tests and trials of density and shearing resistance of the bearing sand were made.

No influence of time on the bearing capacity of these piles was established. Thus in practice test loads made shortly after driving, as is customary, would actually suffice. A typical load-settlement diagram is shown in Fig. 6. The critical loads Q_{crit} in the tests were fixed at values for which the settlement of the 20 × 20 piles reached 5 mm, and that of the 25 × 25 cm piles reached 6 mm. The maximum loads Q_{crit} at the end of the tests were noted, but not taken into account.

For the characteristic form of diagram the admissible load was fixed with $Q_a = 2/3 Q_{crit}$, giving a safety factor 1·5.

The critical load Q_{crit} increased nearly proportionately with

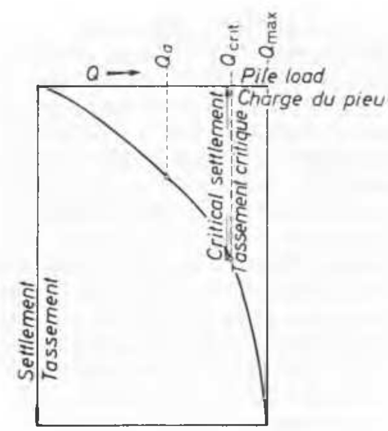


Fig. 6 Load/settlement diagram
Diagramme charge/tassement du pieu

increasing length of embedment in the bearing sand L_e as shown in Fig. 7. In a zone of sand of lower density which had been located by other tests the critical load was proportionately smaller. For a length of embedment of 3 m the resulting admissible load for the 20 × 20 cm pile was 13 ton and, for the 25 × 25 cm pile, 21 ton. For only 2 m length of embedment

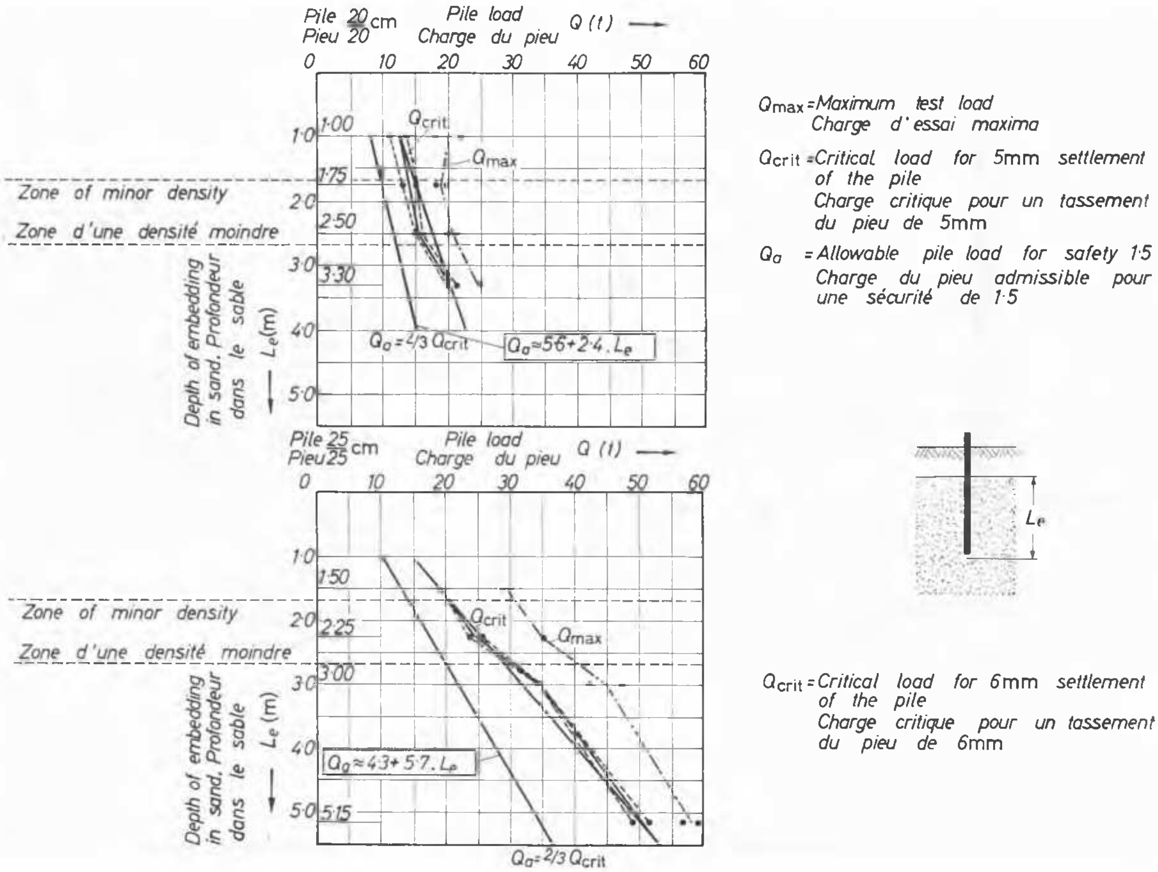


Fig. 7 Critical pile load Q_{crit} and allowable pile load Q_a as function of the depth of embedding L_e in the bearing sand

Charge critique du pieu Q_{crit} et charge admissible Q_a en fonction du profondeur de l'enforcement L_e du pieu dans le sable portant

the admissible load would be reduced to 11 and 16 ton, respectively. For an embedded length of less than 1 m, the empirical relation found in the tests cannot be used. Measurement of the uplift force in the pulling tests enabled the separation of point-resistance and skin-friction; the latter also increased with the greater length of embedment in the sand. A maximum value of skin friction of 0·25 kg/cm² was found for both piles. The application of these test results for fixing the admissible load pre-supposes that the bearing sand layer received a pressure from overlaying soils of 2 ton/m². At a smaller overlaying load the bearing capacity of the piles decreases.

Relationships were also established between the static critical load Q_{crit} and the dynamic resistance to penetration Q_{dyn} of the piles during driving. The critical load was always the smaller. With increasing length of embedment of the piles in the sand the static and dynamic values became similar, and the same occurred when the density of the sand was reduced. The penetration of the thin piles under the last 10 blows was proportionately greater than customary in the case of thicker piles. Thus the values as stated under the German standard for the thicker piles cannot be used here.

In accordance with the knowledge gained, a piling code has been developed which, besides the weight of the monkey, pile-weight and height of fall also takes into account length of embedment of the pile in the sand.

In order to reach a certain critical load $Q_{crit} = 1{\cdot}5\ Q_{ad}$ for a pile it does not suffice just to issue instructions on a certain penetration of the pile during the last 10 blows at a certain height of fall and certain relationship of the monkey weight to the weight of pile + pile-hood; a fixed length of embedment of the pile in the bearing sand must be stipulated. If this is known the limit which should be given to the penetration of the pile under the last 10 blows to enable the necessary critical load to be reached can be calculated in conformity with the piling code.

If the piling code is applied to piles which are embedded in sand of low density then the calculation is on the safe side.

The results of these tests cannot be applied to other conditions without further investigations. We hope that soon control tests with both thin and thick concrete piles will be started. A publication on the tests made up till now will soon be made.

I would like to take this opportunity to mention Á. Kézdi's interesting publication. He has been concerned with thin piles, by using model piles of 10 × 10 cm, but these are hardly usable in practical foundations.

H. Zweck (Germany)

With regard to the third point proposed for discussion, I should like to report on settlement observations of a pile foundation and of a slab foundation, both performed in soft clay under practically the same conditions. The settlement of the slab foundation proved to be much smaller than that of the pile foundation.

The construction site was situated near the lake of Constance, in Southern Germany. The subsoil is soft clay which reaches down to depths of 50 m, and sometimes more.

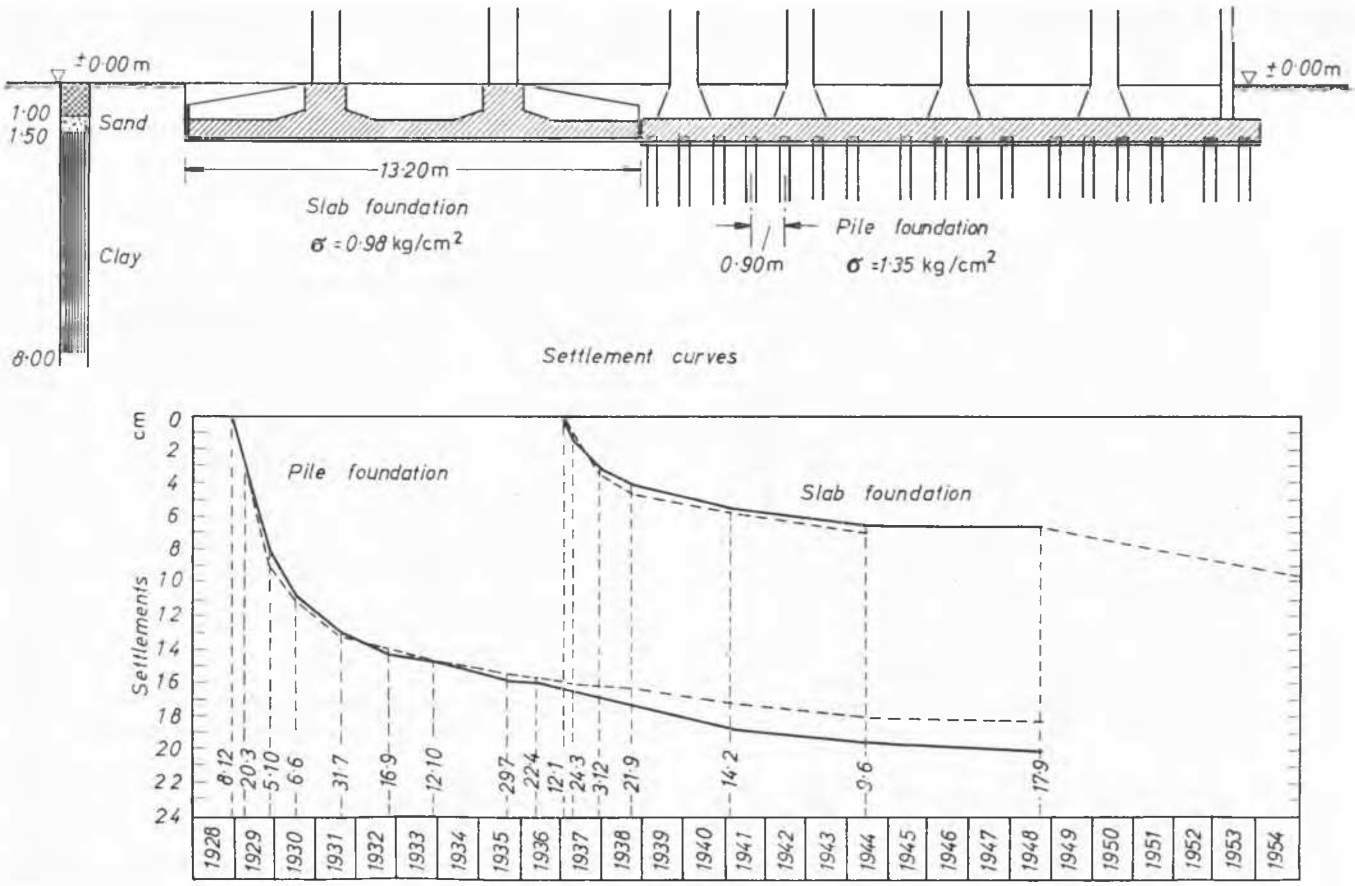


Fig. 8

The clay fraction lies between 40 and 60 per cent, the LL between 45 and 54 per cent, the PL between 21 and 24 per cent, and the consistency between 0·5 and 0·4. The unconfined compression strength was about 0·5 kg/cm². By a laterally confined compression test the coefficient of compressibility was found to be 0·05.

In 1928 the construction of a gas oven was started on this soil. During the driving of the piles the resistance did not increase; furthermore, on account of their close spacing, the piles caused the clay between them to bulge. For these reasons it was decided to cover the pileheads with an 80 cm thick concrete slab, thus transforming the pile foundation into a slab foundation. Then the reinforced concrete building was erected. The foundation pressure of the 18·0 × 21·4 m slab was 1·35 kg/cm². During construction considerable settlement occurred. Observations continued for several years showed that after $11\frac{1}{2}$ years settlement ranging between 11·5 and 18·5 cm and after 19 years between 12 cm and 20 cm had occurred.

In 1936 the plant had to be enlarged. The new building was not erected on piles but on a slab 16·4 m long and 13·2 m wide. The foundation pressure was 0·98 kg/cm² due to the somewhat lighter construction of the new building. The settlement after $11\frac{1}{2}$ years amounted to only 6·5 cm.

In order to get comparable results the settlement to be expected under the new building was calculated on the assumptions that its slab had the same outline and the same foundation pressure as the slab of the building erected first. Thus a settlement of 10 cm was obtained. It is 50 per cent greater than the actually measured settlement of the slab but still considerably smaller than the observed 18 cm settlement of the previously built pile foundation.

Therefore, the piles did not diminish the settlement but made it increase. This result can only be explained by the assumption that the clay was disturbed by pile driving and had thus become more disposed to settlement.

Á. Kézdi (Hungary)

I should like to make a few remarks by way of addition to my paper, 3b/10.

Paper 3b/10 deals with three special cases: (a) rigid pile with skin friction only; (b) compressible pile, fixed at the pile point; and (c) rigid pile, with point resistance and skin friction. I have, since then, succeeded in writing down the differential equation for the general case (compressible pile with point resistance and skin friction) and giving a solution by successive approximation. It would take up too much time for me to present the solution in detail now, but the result is shown in Fig. 9, which presents the force in the pile at different depths. The character of the curves fits very well with the experiments of M. Kaufmann and H. Zweck. With the aid of these curves the load settlement diagram could be constructed as shown on the illustration. The constants in the equations are determined by a pile loading test.

My next remarks refer to pile groups. Fig. 10, constructed on the basis of tests with model piles, 2 m in length, shows clearly that the group effect ceases at pile spacing $d = 6D$. Fig. 11 shows the test results of a group with five piles: the central pile has been loaded and the settlements of unloaded piles measured. The group effect ceases at spacing $d = 6D$. I agree with our General Reporter that with a greater number of piles in a group the group effect comes into display even at greater distances. Some new tests support this suggestion.

I agree completely with the General Report that the results shown in Fig. 12 in my paper, with the increasing ratio of point resistance to skin friction, are a consequence of the test procedure. The real value of point resistance cannot be determined with this method; it develops only with the loaded mantle. Here the theoretical results are satisfactory.

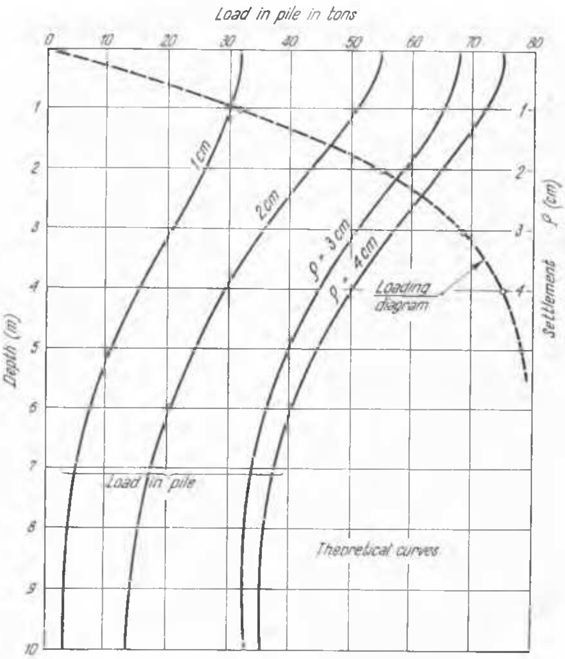


Fig. 9 Force in a compressible pile embedded in uniform soil with point resistance and skin friction, at various settlements; theoretical loading diagram

L'effort dans un pieu compressible enfoui dans un sol uniforme avec résistance à la pointe et frottement superficiel, à différents tassements; graphique de chargement théorique

Finally, I should like to point out an interesting result from the model tests. We investigated the case where in the sand around and beneath the pile the water level rises at a certain rate. This rising causes, on the one hand sudden sagging; the apparent cohesion disappears; the seepage pressure on the other hand causes neutral stresses and the bearing capacity decreases. Fig. 12 shows the degree of this in the case of a model pile.

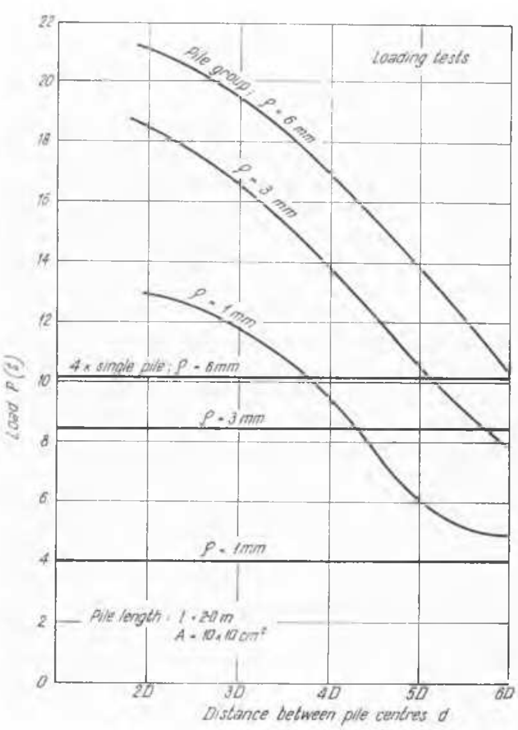


Fig. 10 Settlement of pile groups in sand

Tassement de groupes de pieux dans du sable

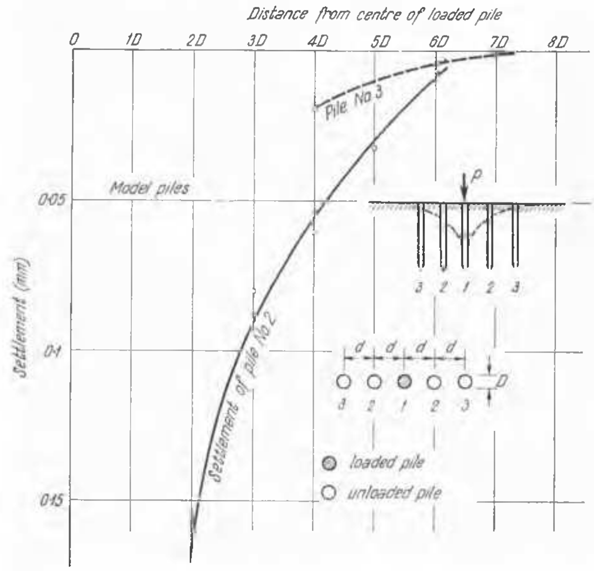


Fig. 11 Settlement of unloaded model piles

Tassement de modèles pieux sans chargement

With regard to the remarks of J. Feld, I should like to point out that my statements referring to pile groups refer only to a certain grouping of the piles where the inner earth core is surrounded by piles resulting in an increased compaction. Furthermore, it should be emphasized that the experiments were made in a non-cohesive sand with driven piles. The tests first made with model piles were fully verified in the field, also in the case when only the lower third of the piles were embedded in sand, the upper layer being softer. The fact that up till now we did

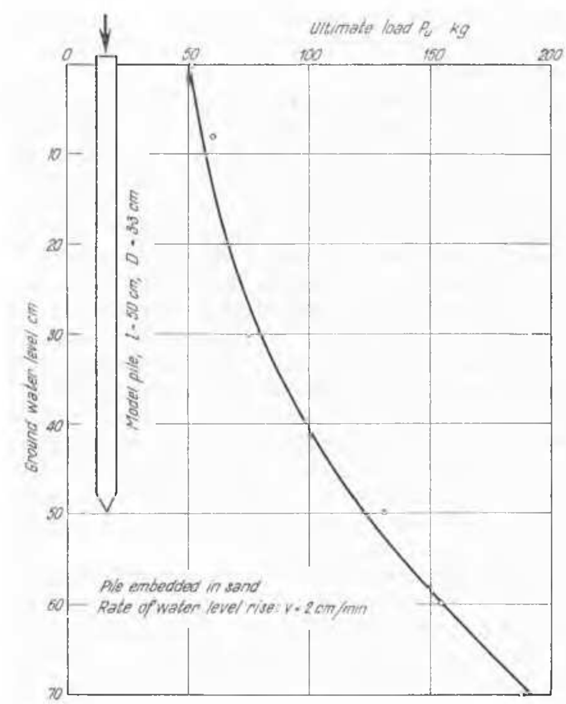


Fig. 12 Effect of rising water level on ultimate load of pile embedded in sand

Effet d'un niveau d'eau montant sur la charge de rupture d'un pieu enfoui dans le sable not take this group effect into account in the practical design cannot invalidate the test results, supported by theoretical considerations.

J. G. Stuart (U.K.)

My contribution is prompted by the last part of Paper 3b/10 by Á. Kézdi, and my results may also be of some interest to J. Feld.

I wish to say a few words about some tests on groups of model piles in sand which have been carried out at Queen's University, Belfast.

The model piles are flat-ended wood dowels $\frac{3}{8}$ in. (about 9 mm) and $\frac{5}{8}$ in. (about 16 mm) diameter which are pushed down as a group into a clean dry fine sand contained in a cubical box of about $2\frac{1}{2}$ cu. ft. capacity. The dry density of the sand, which is compacted in thin layers, has been kept constant at 100 lb./cu. ft. for most of the experiments.

The experiments, which were carried out by Mr. Scarlet and Mr. Fleming, were first directed towards obtaining the load-settlement relationship for single piles, and these results have been used as a basis for the comparison of the efficiency of the groups.

Square groups of piles consisting of from 4 to 25 piles have been tested under axial loads and the spacings of the piles have been varied to investigate this effect on the bearing capacity and settlement behaviour of the groups.

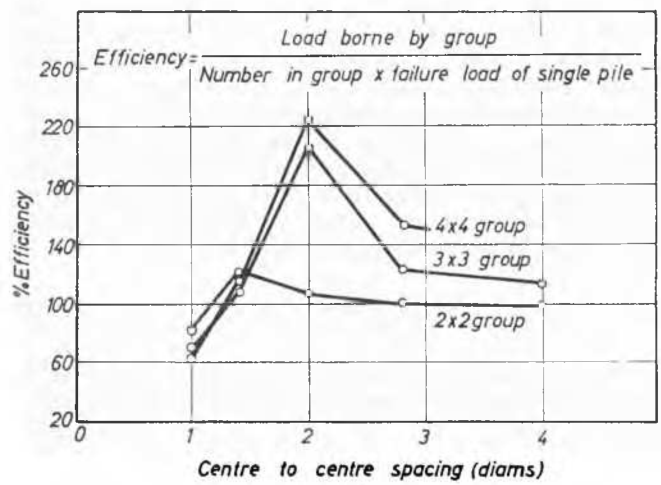


Fig. 13 Spacing/efficiency graphs for groups of piles in sand
Graphiques distance/efficacité pour groupes de pieux dans le sable

Fig. 13 shows some of the results for three of the groupings used. It shows the plot of centre to centre spacing against the efficiency, which is defined as the load borne by the group divided by the number of piles in the group multiplied by the failure load of a single pile at the same depth. It will be seen that at a spacing of about 2 diameters a critical condition is reached in the larger groups. Furthermore we find that below the critical spacing the soil is carried down by the piles as if it were a solid block. We have made experiments with solid piers of similar overall end area to the pile groups below the critical spacing, and these give load-settlement curves similar to the pile groups at close spacings.

Fig. 14 shows the load-settlement curves for a group of 9 piles at a depth of 7·8 in. at several spacings. The curve marked ∞ is for a single pile with the load multiplied by a factor of 9 to simulate a group of 9 piles with very wide spacing.

A few tests have also been carried out in loose sand and the results show the same general pattern.

The distribution of load among the piles in a group is also being studied, and although this part of the work is in its early stages it has been found that the centre piles carry more load than the others. This suggests a similar type of distribution to that which exists under stiff surface foundations on sand.

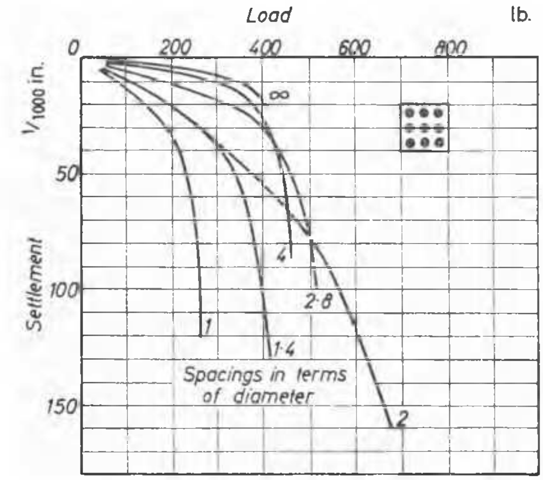


Fig. 14 Load/settlement graphs for a group of 9 piles of $\frac{3}{8}$ in. diameter at depth 7·8 in. with various spacings
Graphiques charge/tassement pour groupes de 9 pieux avec un diamètre de 9·5 mm à une profondeur de 19·8 cm et à des distances différentes

We hope shortly to be able to enlarge the research programme to include some field tests on larger models, and it has been very encouraging to see that Á. Kézdi's tests on a much larger scale show a similar trend to the small models that I have just described.

T. Whitaker (U.K.)

Work on piles and piling at the Building Research Station has included a certain number of experiments on the load distribution among model piles in groups.

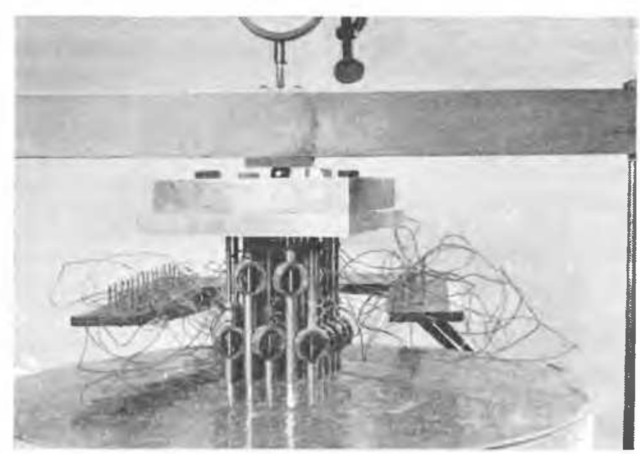


Fig. 15

Fig. 15 shows a square group of 25 model piles each having a load gauge. The top ends of the load gauges are cast into a solid cap, and the load is applied through a central ball. These model piles give results which are shown in Fig. 16. The ordinate gives the load per pile (the loads have been averaged for similarly placed piles) and the abscissa the total load. The total load exceeds the failing load by reason of more load being added while the group was sinking into the soil, that is after failure had been reached.

The interesting point is that the corner piles exhibit plastic failure. They reach a maximum load bearing and then proceed to go steadily into the ground without taking more load. At that time the piles in the centre of the group begin to take proportionately more load.

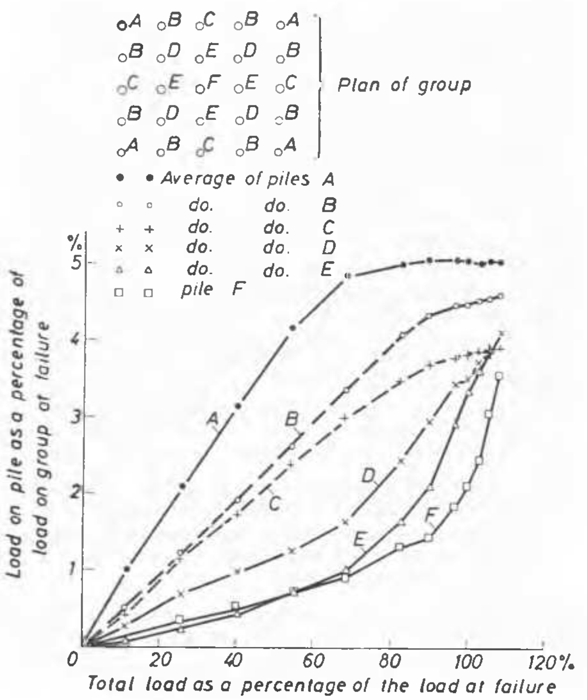
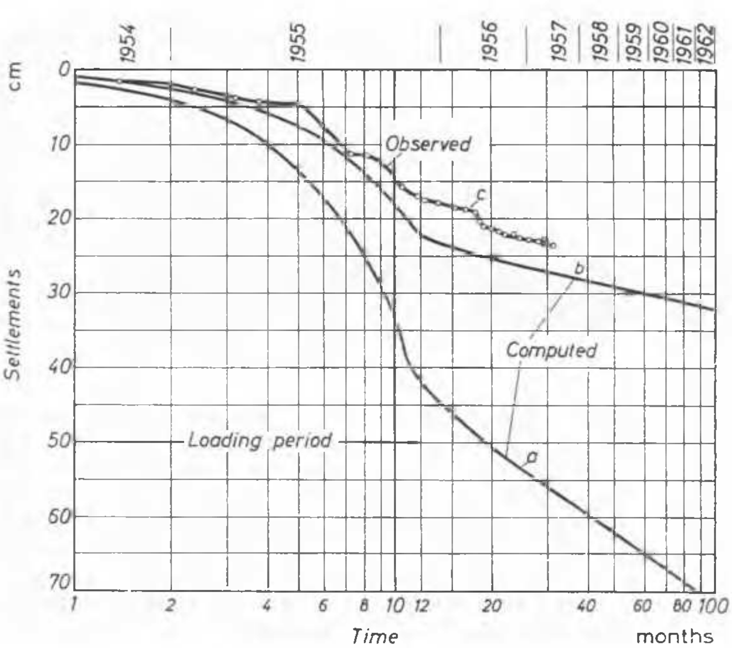


Fig. 16 Load distribution in a 5^2 group of piles, 2 diameters spacing. Centre pile driven first, corners last

Répartition des charges sur un groupe de 5^2 pieux à deux diamètres de distance. Le pieu du milieu est foncé en premier lieu, ceux des coins en dernier lieu

If such a group of piles is unloaded after failure, there is every indication that on reloading the regime of load distribution changes. The corner piles appear to take very little load during the second loading, while the centre core of piles take most of the load.



L. Zeevaert (Mexico)

Following the request of the General Reporter, I should like to present up-to-date settlement records of the compensated raft-friction pile foundation which I reported and discussed in Paper 3b/17.

The settlement records have been extended to one year more of levelling observations. As may be seen from Fig. 17, corresponding to Fig. 9 in my paper, the observations show a fairly good agreement with computed settlement curve b for which the building was designed.

The settlement computations of curve b were performed with parameters obtained from consolidation curves showing distinctly the secondary time effect. The tests were carried out in the oedometer on good undisturbed samples. The parameters refer to those mentioned in settlement analysis procedure which I summarized in my contribution in *Proc. 3rd Conf.*, Vol. III, p. 129.

One problem that has not been reported in the Proceedings or suggested in the excellent General Report of P. C. Rutledge is that concerning the negative friction in piles, and particularly that connected with the phenomenon of the reduction in bearing capacity of piles because of negative friction: however, this problem may enter into points 4 and 5 suggested by the General Reporter. This phenomenon takes place when compressible soil deposits consolidate with respect to firm layers where the piles are supported, thus creating an additional load on the piles and at the same time reducing their bearing capacity. In order to approach the philosophy of this problem, assume Fig. 18 to represent a large group of piles driven into the subsoil to point bearing resistance in a sand layer. The initial effective overburden pressure is p_o. Now assume that the compressible deposits are consolidating to depth h under the initial pressure p_o. Therefore, during this vertical movement, and as negative friction is induced on the piles, the vertical pressure gradually reduces from p_o to a value equal to p_v. The rate of load transfer with depth may be computed by limiting equilibrium condition expressed by formula 1:

$$\partial\left(\frac{p_o - p_v}{\partial z}\right) = \bar{n}\bar{w}s \quad \ldots\ (1)$$

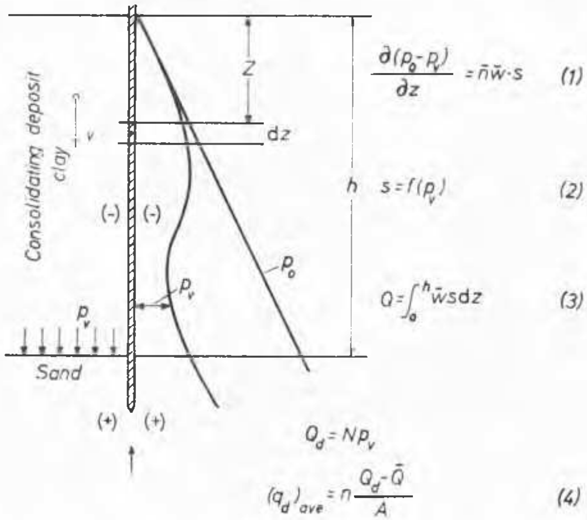
in which $\bar{n}$ = number of piles per unit area, $\bar{w}$ = the perimeter of the pile, and s = shearing resistance along the shaft of pile.

Fig. 17

The value of the shearing resistance s may be expressed as a linear function of p_v, and equation 1 may be solved for different specific cases concerning a large group of piles.

The total negative friction $\bar{Q}$ may be computed knowing the approximate value of p_v as a function of depth, and using formula 3, Fig. 18.

The average unit bearing capacity of the group of piles given by formula 4 is largely reduced; first because of the negative friction $\bar{Q}$ acting on the piles; and secondly as Q_d becomes smaller because of the inevitable reduction in the confining



As $\bar{Q}$ increases, Q_d decreases

n = Number of piles in the group

A = Total area of the group

Q_d = Total bearing capacity of one pile in the group

$\bar{Q}$ = Total negative friction

Fig. 18

pressure at the supporting layer as the initial effective pressure p_o reduces to p_v.

In Mexico City, the well-known ground surface subsidence induces negative friction in long piles, with the result that, in some cases, the point bearing capacity of the piles is strongly reduced and therefore undesirable and non-uniform penetration of the piles takes place in the supporting deposits.

This theory may be used to demonstrate that the positive friction of piles should not be separated from the total by measuring the resistance to pulling the pile.

J. W. Alexander (U.K.)

A. F. van Weele (3b/16) will be interested to know that there is in this country a specialist piling contractor who is able not only to calculate but also actually to measure the frictional resistance of a patented shell pile. This method of measurement is unique.

The pile is a reinforced concrete tube made up from prefabricated sections (or shells) threaded on to a steel mandrel, the whole being assembled upon a solid concrete driving shoe. The tube, mandrel and shoe are driven, new sections being added as required, until the calculated pre-determined set is obtained, when the steel mandrel is withdrawn. A cage of reinforcement is then suspended in the tube and concrete is poured in to form an unstressed solid core. Measurement of the final set gives a value for the total driving resistance of the shell pile. If the driving head, which normally apportions the hammer blow between the shell and mandrel, is disconnected from the shell by removing the fibre rings between the head and shell, or by extracting the top shell, then the hammer blow transmits all its energy to the shoe. If a set is now taken, the resistance to driving of the shoe only can be calculated, and by subtraction from the total resistance the frictional resistance of the pile is obtained. This test can be applied to each and every shell pile without interrupting progress or causing additional cost. Thus, we have a method of determining in the field, to an accuracy sufficient for practical purposes, the skin frictional values of soils.

To quote just one example, the company referred to have carried out extensive pile loading tests at Crossness on the southern bank of the River Thames, where the 17½ in. diameter shell piles were driven to a depth of 31 ft. 6 in. to a set required for the proposed working loads. This contract included two uplift tests for which each pile was capped with an encastrè beam, this being jacked up against two anchor piles. Upward displacements of the pile were measured for 1 ton increments of load until yield of the subsoil occurred. The test was then stopped and the pile completely extracted for laboratory testing. The total ultimate driving resistance of a typical pile was 188 ton of which 139 ton was end-bearing and 49 ton frictional. The actual force required to produce a yield of the subsoil in the uplift test was 48 ton. From detailed site investigations carried out on an adjacent site, theoretical skin friction values gave 6½ ton/ft. of 12 in. × 12 in. pile in the London clay and 1½ ton/ft. of pile in the overlying gravel. These results when converted to apply to the 17½ in. diameter shell piles at Crossness would indicate a theoretical frictional resistance of approximately 45 ton. Thus complete agreement was reached.

Not all the tests undertaken at Crossness compared to this degree of accuracy, but, nevertheless, it was found that there is more similarity between the values obtained by such test driving and test loading than there is between normal site investigations and test loading.

Repetitive loading and unloading cycle tests were also carried out at Crossness. Although the main purpose of these 24-hour-cycle tests was to indicate the fatigue likely to be induced in the subsoil by constant water changes in the aeration tanks supported by the piles, the field measurements taken bore a relationship to those mentioned in A. F. van Weele's paper, and I think reasonable results would have been obtained if I had had the courage to assess a correct value for the modulus of elasticity for the concrete.

The tests at Crossness were concluded by test loading a single 20 in. diameter pile of length 65 ft. with 300 ton dead load. The vertical displacement was not measurable and the residual horizontal settlement was 0·13 in.

I have found that some penetration tests are apt to be misleading to the piling contractor as they often either ignore or loosely interpret the three following conditions:

(1) The effect of subsoil consolidation caused by the driving of a large number of piles at close centres, or conversely the reconsolidation or tightening up of the soil after driving. In this connection it would be useful to know whether any penetration tests have been made before and after concentrated piling as a comparison of the results would be most enlightening.

(2) Allowance for ground heave, as mentioned in M. J. Tomlinson's paper, 3b/14.

(3) Measurement of negative skin friction.

It will be appreciated that these factors should be made known to the piling contractor if he is to be expected to guarantee the foundation. I hope, therefore, that the technicians in this country will be able, before the next conference, to devise a penetration testing apparatus that will be able to assess these factors.

Finally, I should be most grateful if our President would say whether the remarks he made yesterday on the usefulness of the penetration tests for spread footings also apply to piled foundations, with particular reference to the three factors stated above, as these three factors alone provide the piling contractor with too large a percentage of his problems.

K. V. HELENELUND (Finland)

I should like to make a few comments on the settlement and horizontal movement of structures on pile foundations. Friction piles are used to some extent in the south-western parts of Finland where the thickness of the soft, normally consolidated clay layers may exceed 30 m. Observations of the settlement of structures on friction pile foundations in clay show, however, that the settlement in most cases is quite large. Buildings in these low areas have often been surrounded by gravel or sand filling, and this filling affects the settlement to a very large extent.

Among the cases with large settlements there are also pile foundations designed under the assumption that only a part of the load will be taken by the piles, the other part being carried directly by the subsoil, that is, the same assumption which, according to Paper 3b/17 by L. ZEEVAERT, has been used recently in designing a pile foundation on the highly compressible clay in Mexico City. As an example, I can mention a five-storey brick building in the city of Turku, where the thickness of the soft, partly organic clay layers varies between 20 and 30 m. The building, which was founded on timber piles with a length of less than 10 m, settled about 60 to 120 cm and had to be replaced by a new building resting on point bearing piles.

Observations on the movements of bridge foundations on compressible soil layers show that they undergo rather large horizontal movements. In some cases the distance between the abutments decreases to such an extent that the bearings of the bridge constructions have to be moved several times. This may in some cases be due to a tilting of the abutment caused by differential settlements, as stated by L. BJERRUM, W. JÖNSON and C. OSTENFELD in Paper 3b/3, but in many cases the movement is caused by horizontal displacements of the soil layers under the abutment and the embankment filling behind the bridge. In this connection I refer to a paper by PECK, IRELAND and TENG (1948) concerning retaining wall failures in the U.S.A. Since the direction of the principal stresses under the end of the embankment is not vertical but is inclined towards the opposite abutment, the compressible soil layer under and between the abutments will undergo consolidation deformations resulting not only in settlements but also in horizontal movements of the abutments and the embankment filling behind the bridge. In soft soils there are also considerable horizontal movements caused by shear displacements under the embankment filling. Large horizontal movements have been observed not only on bridges founded directly on compressible soil layers and on friction piles in clay, but also on bridges resting on point bearing piles surrounded by clay.

Fig. 19 shows the relationship between the observed movements and the soil conditions at 50 railway bridges in Finland (HELENELUND, 1951). The largest movements are observed at bridges on piles embedded in soft organic clay layers and at bridges placed directly on such compressible cohesive soil layers. Bridges on frictional soil layers show much smaller movements. The pile foundations are generally designed according to the methods of Vøkkentved or Culmann assuming that the piles can take loads only in the direction of their axis. In reality, the piles will get considerable transverse loads when the soil layers consolidate and move in a horizontal direction, It has been found impracticable to provide the abutments with such a strong pile foundation that no movements would occur; instead, the most practical method for eliminating the movements has been found to be to provide not only the abutments but also a certain part of the embankment filling behind the bridge with a pile foundation.

In Paper 3b/3, The Settlement of a Bridge Abutment on Friction Piles, the authors state that the observed settlements and horizontal movements have been greater than expected and also greater than the calculated values. In calculating the settlement the authors have not taken into consideration the influence of the embankment load, which causes considerable stresses and displacements not only in the soil layers under the pile points or under two-thirds of the length of the piles but also in the upper soil layers. Because of the thickness of the compressible soil layers all the consolidation deformations cannot have occurred during the overloading period as no sand piles or other means for speeding up the consolidation process have been used. A large part of the secondary time effect observed may be due to slow shear displacements caused by the embankment filling. As the secondary settlements observed are very interesting I suggest that the authors take a new series

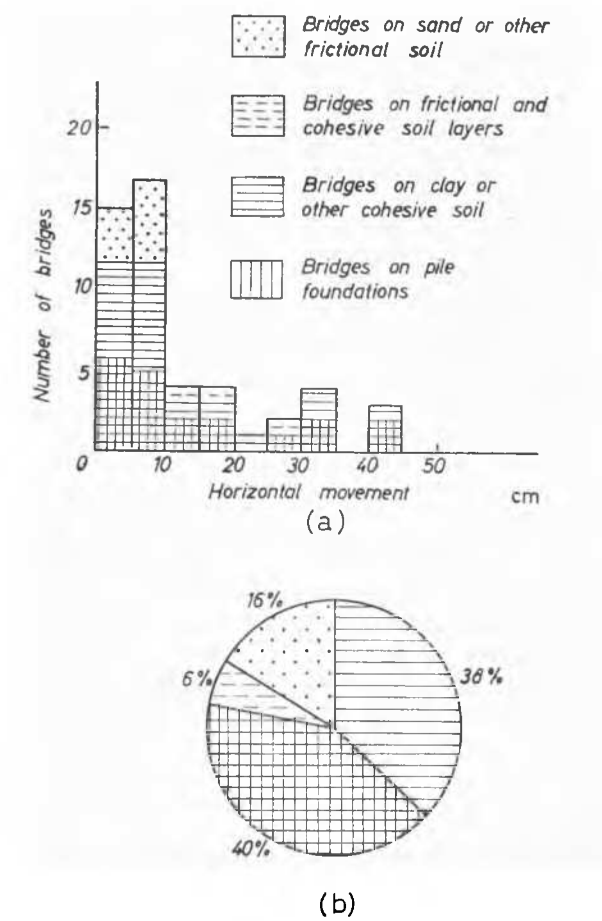


Fig. 19 Horizontal movements observed at 50 railway bridges in Finland. (a) Number of bridges and the observed movements as a function of the soil conditions. (b) Number of bridges on frictional and cohesive soil in per cent of the total number of the observed bridges. Forty per cent of the bridges are provided with pile foundations

Mouvements horizontaux observés sur cinquante ponts de chemin de fer en Finlande. (a) Nombre de ponts et mouvements observés en fonction des conditions des sols. (b) Nombre de ponts bâtis sur sols cohérents et mon coherents, comme pourcentage du nombre total de ponts. Quarante pour cent des ponts sont bâtis sur pieux of soil samples from the site in order to perform long-term consolidation and long-term undrained and drained triaxial tests.

The observations at this bridge seem to confirm the conclusion drawn from other observations, that bridge abutments on compressible soil should not be provided with friction ('floating') pile foundations unless the influence of the embankment load has been eliminated through embankment filling, long-term overloading, overloading in connection with sand piles or other effective method. In the design of point bearing pile foundations, where the piles are surrounded by soft compressible soil layers, the horizontal displacements of the soil and the resulting lateral forces on the piles have to be taken into consideration.

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A. LAZARD (France)

Monsieur le président, messieurs, je voudrais parler très rapidement des communications présentées à ce congrès concernant le flambement des pieux dans l'argile molle. Il s'agit des communications 3b/2, de A. BERGFELT, de la communication 3b/4, de A. BRANDTZAEG and E. HARBOE auxquelles il convient de joindre un article récent de L. Bjerrum, paru dans *Géotechnique* de juin, ainsi que l'article de 3b/7 H. Q. GOLDER et B. O. SKIPP. B. O. Skipp doit prendre la parole après moi pour donner des renseignements plus précis sur l'état d'avancement de ses essais. Je ne peux donc pas en faire état.

Je voudrais présenter des remarques d'ordre théorique pour finir par une contribution d'ordre pratique.

Théorie — (a) La plupart des cas cités ont montré un flambement avec déformations permanentes, alors que les contraintes semblaient rester dans le domaine élastiqué des pieux essayés. Il y a là une grave lacune que la théorie ne semble pas actuellement capable de combler. (b) Les essais 3b/4 ne semblent pouvoir s'interpréter correctement qu'en admettant, en plus d'une résistance au déplacement plus on moins élastiqué, une résistance contre des rotations aux extrémités des zones flambées, car le pieu sortait d'une couche dure de gravier pour entrer plus bas dans une couche d'argile plus dure que celle où s'est produit le flambement. Pour calculerces cas on pourrait s'inspirer de quelques graphiques que j'ai donnés dans les *Annales de l'Institut du Bâtiment et des Travaux Publics*, Septembre 1949 (Paris) (Flambement d'unetige posée sur supports équidistants).

Pratique — (a) Les résultats obtenus, en particulier en 3b/4, ne peuvent s'expliquer que par une grande hétérogénéité du terrain: conduisant, par exemple, à des charges variant de 9 t. à 16 t. soit de 1 à 1·8; où à des longeurs flambées de 1·30 m et 1·60 m pour la même charge de 9 t. Il est de même pour de nombreux résultats rapportés en 3b/2. (b) En conséquence l'auteur propose de suivre A. Bergfelt et d'adopter une formule de la forme

$$\sigma = c(z \times EI)^{\frac{1}{2}}$$

où Caurait la valeur centrale g mais pourrait varier entre $g(2)^{\frac{1}{2}}/2$ et $g(2)^{\frac{1}{2}}$ en admettant une dispersion générale de l'ordre de 1 à 2 (comme il l'a constaté pour le Renversement des Fondations, communication 3a/19).

B. O. SKIPP (U.K.)

Following the indications in Paper 3b/7 (H. Q. GOLDER and B. O. SKIPP) that the failure of end bearing piles in various soft clays was elastic-plastic, it has been possible to develop an analysis of elastic-plastic collapse. The basis is a contribution of W. Marchant to the Symposium on the Plastic Theory of Structures, held in Cambridge, in 1956.

Before any plasticity develops in either pile or clay the load deflection curve is governed by the relation:

$$\delta = \frac{\delta_o}{1 - \dfrac{P}{P_{cn}}}$$

where P_{cn} can be taken as Granholm's critical load. This is a simplification of some work by R. V. SOUTHWELL in 1932 (*Proc. Roy. Soc. A*, **135**, 606). If the argument is confined to a single curvature deformation of a slender pin-ended pile, initially deformed as a half sine wave of amplitude δ_o, it can be supposed that a plastic hinge develops at $l/2$, at failure. Another relation between P and δ can then be defined by examining the moments

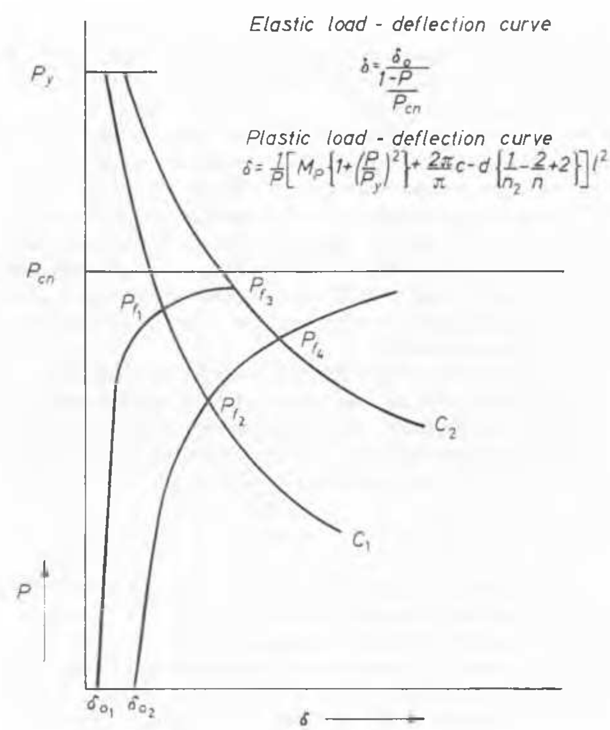

Fig. 20 Collapse of slender piles in clay. Load-deflection curves for elastic and plastic conditions

Rupture de pieux minces dans l'argile. Courbes de charge/deflection pour les conditions plastiques et élastiques

associated with the mechanism surrounded by a yielding clay. This mechanism now consists of two rigid members hinged at the centre. The term M_p is the pin load at upper and lower extremities. A collapse condition is indicated by the intersection of the two load-deflection curves.

From Fig. 20 it can be seen that there are values of initial deflections E/c, c, for which a collapse may take place well below the critical value. The clay is assumed to fail when the stress upon it at $l/2$ exceeds $2(2+\pi)c$.

This elastic-plastic analysis has been extended to cases where the elastic deformation of the pile is in the form of a damped half sine wave, by assuming a plastic hinge at the maximum amplitude of the first half sine wave.

This work will be published in full shortly.

A. J. DA COSTA NUNES (Brazil)

Je voudrais discuter trois points du rapport général du P. C. Rutledge.

D'abord je crains que la conclusion presentée aujourd'hui par le rapporteur général de qu'il faut limiter l'utilization des pieux au cas de terrain de base pulverulant est peut être trop générale. Je voudrais à ce propos mentionner ce que a dit A. Cummings, ancien président du Comité American de Mécanique des sols à la Purdue Conference.

Il a remarqué qu'il y a au monde des milliers de fondations sur pieux avec leurs bases sur argile et qui se sont comportés très bien. Ce que serat arrivé avec un autre type de fondation restera toujours matière de spéculation.

Dans sa discussion de l'article de N. B. Hobbs (3b/8), on lit dans le rapport général que: 'The remedy is, of course, to cast the piles in shells with sufficient strength to maintain the original open hole and to support fully the concrete of the pile'. Je pense que l'effet est provoque par la force ascensionelle de la vibration employée pour l'execution du pieu et qu'on pourrait aussi eviter l'accident en damant le beton au monton ou bien en le comprimant à l'air comprimé au encore en faisant un trou préalable dans le terrain.

Finalement, dans son appréciation de l'article 3b/5, P. C. Rutledge a dit qu'il n'apparait pas clairement en quoi le flambage est un problème.

Nous voudrions eclaircir le fait que, dans ce cas particulier, comme il est indiqué dans le titre même de l'article, il s'agit de pieux avec grande hauteur libre au dessus du sol.

L'élamement, rapport entre la longueur libre et le rayon de giration atteint, comme on peu deduire de l'article, environ 130 pour un pieu de 50 cm de diamètre. Se on tient compte de la longueur dans le sol jusq'au point de rotation on aura encore des rapports plus grands, donc une réduction radicale de la change admissible.

Au dessous de la côte d'érosion le sol est très résistant. On a du sable assez compact, du sable argileux aussi assez compact et de l'argile sabloneuse dure. Le nombre de coups du carottier normalisé est d'environ 20 à 30, en moyenne et la résistance à la pénétration du cône hollandais est d'environ 80 à 150 kg/cm^2.

T. E. Mao (China)

With reference to Paper 3b/1, Foundation Engineering and Drilling by the Vibration Method, by D. D. Barkan, our General Reporter made this statement which I heartily support: 'Manufacturers of construction equipment and Contractors in all parts of the world would do well to investigate the possibilities of driving and extracting sheet piles with vibratory equipment'. I would like, however, to go one step further by recommending investigations of the possibilities of vibratory equipment not only in the field of sheet piles but also in the wide domain of tubular piles or columns with ordinary sizes as well as extraordinary sizes such as sinking wells. I would even suggest exploring the possibility of sinking caissons by the vibratory method.

It was stated in 3b/1 that 'More powerful vibrators of the Tatarnikov type were used simultaneously with water jets for driving reinforced concrete cylinders when building bridge piers'; I wish to add that such is the case with bridge building in China. In fact, we have just finished using the vibratory equipment with great success in the sinking of unusually big reinforced concrete cylinders on an unprecedentedly large scale, both in regard to the depth of sinking and the number of cylinders sunk. I refer to the foundation work of the Yangtze River Bridge at Hankow, China.

This bridge has a total main span length of 1,156 m, consisting of three units of three spans each, of steel continuous trusses of 128 m span. It is a double-decked structure with a double-track railway on the lower deck and a six-lane highway on the upper deck.

Both the nature of the Yangtze River and the geological condition of its bottom at the bridge site make the foundation work of the bridge extremely difficult. The difference between the highest and lowest water levels is 19 m, and the duration of the high water season in a year is usually seven to eight months. The deepest water to be expected in building the bridge pier is much over 40 m, and this is the water depth that has to be constantly dealt with during the construction. Pneumatic caissons are therefore not much to our advantage. The bedrock at the bridge site, composed of limestone, marl or shale, is not very deep but its top surface has so steep a slope that under the base of the foundation of a pier there is a difference in elevation of over 5 m within a circle of 16·5 m. The open caisson process is, therefore, not to our advantage either, and we cannot use piled foundations for most of the piers because the soil deposit of a maximum depth of 27 m is subject to a maximum scour of 10 m, at the mid-channel and, for one or two piers, the bedrock may even have no overlying deposit at all. As a final resort and out of sheer necessity we developed the 'Colonnade' foundation process after much experimenting, in which the principal structural part is a reinforced concrete tubular column anchored into the bedrock at bottom and cast into the base of the pier at the top.

The column, 1·55 m in outside diameter, is first sunk as a tube, in sections of 9 m length, by vibratory equipment in conjunction with water jetting. There are 30 to 35 tubular columns under each pair. These tubes are sunk and the sheet pile cofferdam surrounding them are driven with the help of a cylindrical guiding and bracing frame which is dismantled after the pier shaft is built.

All the tubular columns for the entire bridge are sunk through soil deposits of fine sand to coarse sand and gravel by vibratory equipment, together with water jetting. The vibrator is of the type designed by Tatarnikov, as mentioned in D. D. Barkan's paper, but is much enlarged and made at the bridge site. The vibrator is rigidly connected to the tubular column below and the electric motor above so that all the three vibrate at the same frequency.

The specifications of the various vibrators used are as follows: (1) vibrating force: 17·5 to 120 ton, (2) load carrying axles: 4 to 8, (3) moment of eccentrics: 10,000 to 38,000 kg/cm, (4) frequency, rev/min.: 408 to 1,000, (5) capacity of electric motor: 60 to 220 kW, (6) weight of vibrator: 4·5 to 11·25 ton.

These vibrators are very effective, sinking the tubes into 10 m of soil deposit without the help of water jetting. Combined with water jetting we can sink two to six of these tubes through sand deposits of up to 25 m depth in every 24 hours.

There are altogether 224 tubular columns of 1·55 m outside diameter for seven piers, and 116 tubular piles of 0·55 m outside diameter for one pier sunk by vibrating and water jetting for the main spans of this bridge. The deepest pier thus constructed is 64 m, down to the bedrock.

Actual construction work on the Yangtze River Bridge was not started until July 1955 and a report has just been received that a railway train passed over the bridge, as a trial run, yesterday, 15 August 1957. This unusual speed of construction is due, to no small extent, to the great success achieved in sinking the tubes by vibrating and water jetting.

To call your attention to the future development of the vibrating process, I wish to furnish some more information derived from several tests we have recently made at the bridge site and elsewhere concerning the sinking of reinforced concrete cylinders of 1·55 m, 3 m, and 5 m diameters into different soils by vibrating, with or without water jetting. The primary purpose of these tests was to explore the potentiality of this type of tubular column in bridge foundations of whatever requirements and in foundations of buildings and hydraulic structures.

The vibrator used has a vibrating force of 120 ton and the load axles have two frequencies, 500 and 1,000 rev/min.

(1) In clay (without water jetting): 1·55 m tube sinks through 17·9 m after vibrating for 177 minutes, 3·00 m tube sinks through 17·76 m after vibrating for 207 minutes, 5·00 m tube sinks through 16·5 m after vibrating for 180 minutes.

(2) In fine sand (with water jetting): 1·55 m tube sinks through 35 m after vibrating for 56 minutes, 3·00 m tube sinks through 23 m after vibrating for 14 minutes.

(3) In sand and gravel, with 40 cm boulders (without water jetting): 3·00 m tube sinks through 16·31 m after vibrating for 113 minutes.

F. R. BULLEN (U.K.)

It is refreshing to find acknowledged today the fact that a pile in the ground obtains support against settlement from two factors, (a) point bearing and (b) friction. This fact was not always recognized, but time is showing and science is proving that piles gain support from both these factors. I would venture to comment, however, that the relative values of these factors depend entirely upon the ground conditions where the pile is installed, and sometimes the friction value varies according to the proportion of the strength at the toe. Because of this joint support to a driven pile I find it difficult to understand why dynamic formulae such as those referred to by T. SØRENSEN and B. HANSEN (3b/13) still receive so great popularity.

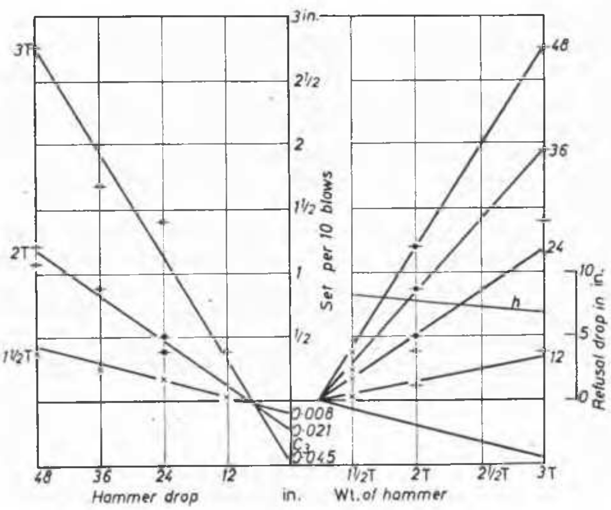

Fig. 21 Test pile No. 2, Silvertown, about 40 ft. penetration
Pieu d'essai No. 2, Silvertown, pénétration: 12 m environ

Paper 3b/16 by A. F. VAN WEELE suggests that the value of the friction support is approximately constant. This seems reasonable once the pile has begun to move, since the friction then may partake of the nature of a resistance to sliding. If, therefore, additional load be added to the pile it might seem that the toe load is increasing, but I do not understand what this means. I should have thought that the value of the support at the toe would be reasonably constant, depending as it must do upon the properties of the ground at the toe. It would follow, therefore, that the addition of more load to the top of the pile is probably causing the pile to accelerate into the ground, and I have an idea that if the settlement diagram be constructed for such loading the curve will be found to have turned over and to have become nearly parallel to the settlement axis. Indeed, I have plotted the loading diagram for the pile mentioned by A. F. van Weele, and I find that for a settlement of about 10 per cent of the pile size the diagram in fact runs almost parallel to the settlement axis. On this basis I submit that the ultimate value of this pile is something of the order of 155 ton. In a paper read before the American Society of Civil Engineers in 1955 Seed and Reese showed that a pile driven into a uniform clay soil had a reasonably constant toe value at failure. Further, they found that the friction support had an ultimate value as well, although their experiments showed that the friction resistance increased appreciably with age.

Referring again to the paper by A. F. van Weele, I should like to ask him whether he was able to measure the value of the temporary compression or quake at the toe of the pile during driving. Furthermore, would he be prepared to say whether the quake during driving is in numerical relationship to the elastic compression of the subgrade referred to in his Fig. 5. According to his Fig. 5, the elastic recovery of the toe of the

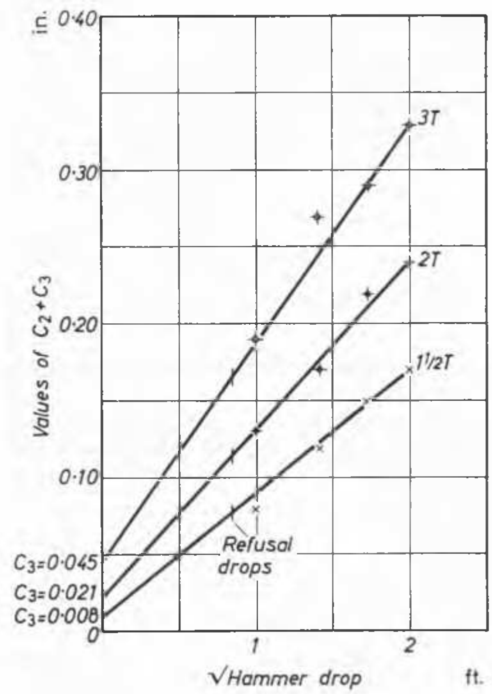

Fig. 22 Test pile No. 2, Silvertown
Pieu d'essai No. 2, Silvertown

pile under a load of 100 ton is 4 mm, that is, 0·15 in. This figure is about equal to that which is quoted by Hiley and others, although according to my own measurements it is about twice what I should expect.

Regarding Paper 3b/16 by M. P. P. DOS SANTOS and N. A. GOMES, I should like to ask whether the temporary compression referred to in the paper is actually the total of the temporary compression of the pile and of the ground at the toe, that is, C_2 plus C_3, using the usual symbols.

A method of measuring the value of the quake at the toe of a pile is given in Fig. 21 which shows the result of some experiments on a pre-cast reinforced concrete pile 14 in. × 14 in. × 45 ft. long driven into gravel in East London. I have found that if the sets for different hammer drops are plotted the resulting diagram is a straight line. If this straight line be produced through the horizontal axis of hammer drop to cut the vertical axis, the intercept on the vertical axis is equal to the temporary compression or ground quake C_3. This statement is supported by Fig. 22, where the values of C_2 plus C_3 for the pile are plotted against the square root of the hammer drop. It will be seen that again the diagram is linear and a comparison of the results indicates a good support for this contention. Perhaps I might ask A. F. van Weele, M. P. P. Dos Santos and

T. Sørensen whether they have found anything similar to this.

In passing, it might be noted that these tests have been carried

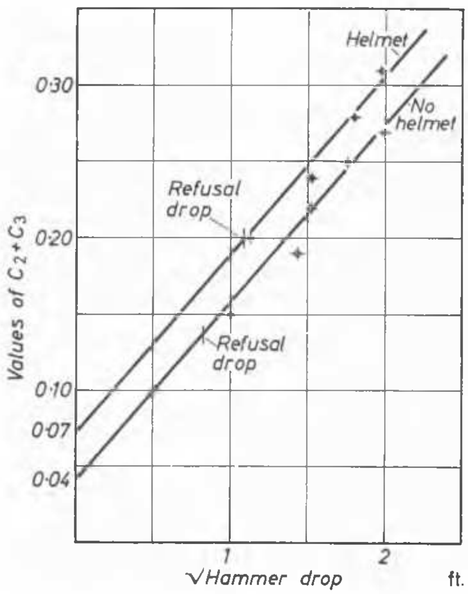

Fig. 23 Test pile, No. 3, Wandsworth
Pieu d'essai No. 3, Wandsworth

out using varying weights of hammer and in all cases the same conclusions have been drawn.

A development of this idea for a pile driven into clay in West London is given in Fig. 23. In this case the pile was driven first with a helmet and then without a helmet. The temporary

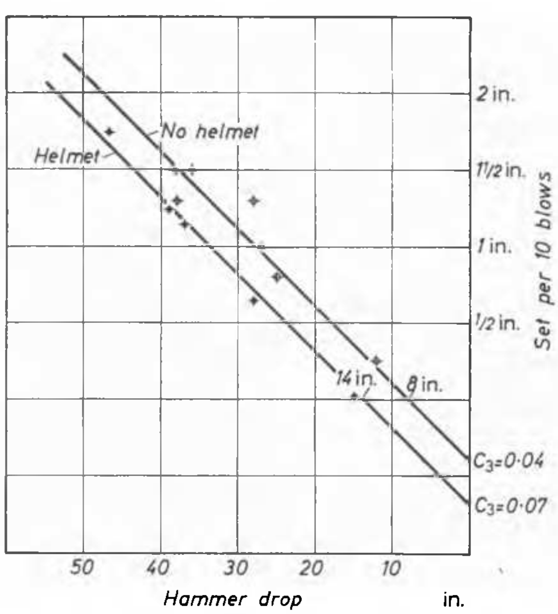

Fig. 24 Test pile, No. 3, Wandsworth
Pieu d'essai No. 3, Wandsworth

compressions or quakes at the toe of the pile under both conditions are equal to the intercept of the set/drop line upon the vertical axis.

In the light of these facts I would suggest that when piling operations are being undertaken it would be of great benefit if engineers could arrange to measure and publish the varying sets with the varying hammer drops, since if my work is correct the value of the toe quake will then become apparent.

One of the interesting results which arises is that the value $S+C_3$ appears to be more important than $s+C_3/2$ and it has occurred to me that this may contain one reason why the Hiley and other dynamic formulae do not always give satisfactory results. May I appeal, therefore, to engineers here today to endeavour to include this additional information when they publish their records as I believe it would be of tremendous benefit in pursuing research into the bearing capacity of driven piles.

C. Széchy (Hungary)

T. Sørensen and B. Hansen have stated in Paper 3b/13 that driving formulae are based upon the assumption that the energy of a blow is consumed by the elastic compression of the pile, soil and driving cap and by the plastic deformation of the soil. It is well known that criticism as to the reliability of the computation of these two factors is justified. I would like to draw attention to other items which are still neglected in these formulae although they have an essential influence upon the bearing capacity of the pile.

First, the plastic deformation of the soil is generally considered equal to the plastic set of the pile. It must be stated however that a considerable part of the impact energy is transformed into lateral thrust at the point of the pile, which consumes energy entirely different from the penetration proper. It is only the part coinciding with the direction of the driving force which may be regarded as useful work, therefore the second term in equation 11 of Paper 3b/13 ought to be multiplied by a factor $(1+\beta)$. Considering that this lateral compaction solidifies the soil in the immediate vicinity of the loaded pile shaft it might be regarded as a kind of overloading over the neighbouring surfaces starting from below the point of the pile in the stage of failure. The increase of this overloading stress may be computed from the change in void ratio due to the penetration of the pile shaft.

In the case of a circular pile of radius r, penetrating into the soil by a distance s the lateral earth masses are compressed to an extent varying with the distance Δr from the pile shaft (Fig. 23). The degree of compression may be expressed by the specific volume change: ϵ.ϵ may be expressed by the help of the compression-modulus M, by the void ratio e and by the increase of pressure Δp

$$\epsilon = \frac{\Delta p}{M}$$

On the other hand the increase of volume as compared to the total volume is also ϵ and may be expressed as:

$$\epsilon = \frac{\Delta V}{V} = \frac{r^2.\pi.s}{(r+\Delta r)^2.\pi.s} = \frac{r^2}{(r+\Delta r)^2}$$

From these two expressions:

$$\Delta p = \frac{r^2}{(r+\Delta r)^2}.M$$

from which we can conclude that the greater the distance Δr from the pile shaft as compared to the radius of the shaft the less will be the increase of inner stress Δp (Fig. 25). It cannot be denied that in addition to the weight of the overlying layers this exerts a beneficial influence upon the bearing value, but there is no doubt that losses in driving energy by lateral loading diminish the effectiveness of direct compression resistance underneath the pile shaft itself and therefore this member of the energy equation cannot be simply expressed as $P.s.$ but $(1+\beta)P.s.$

Another factor which is neglected in the driving formulae is the cross-sectional shape of the pile. It is generally known that the driving resistance is largely dependent upon the shape of the pile, e.g. the small driving resistance of the thin steel sheet piles as compared to the much thicker R.C. or wooden sheet piles is one of their acknowledged advantages. The bearing capacity of sheet piling is not usually important but 'box sections' give a good bearing capacity because of the formation

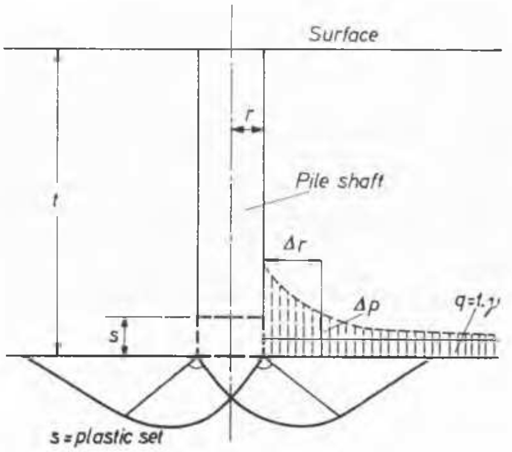

Fig. 25

of a compacted earth core. To a smaller extent this occurs in the case of sinusoidal steel sheet piles (Larssen-Klöckner, Inland, etc.). These phenomena are evidence that the diminution of driving resistance does not necessarily incur a proportional diminution of bearing resistance, and that the shape may

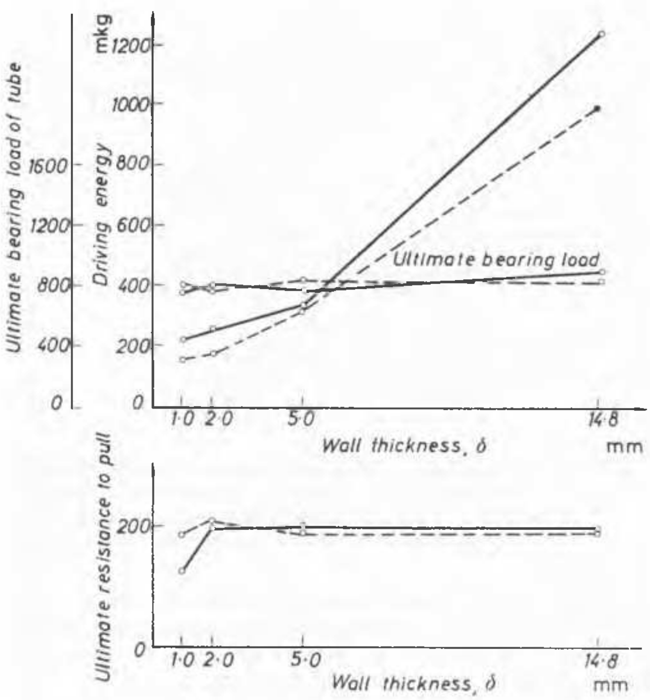

Fig. 26

have a very important influence upon the bearing capacity of a given pile.

This fact was clearly demonstrated by a series of tests carried out with tubular piles in the laboratories of the Technical University of Budapest. Steel tubes 50 cm long were driven with a 5 kg drop hammer into a steel case of 80 × 130 × 60 cm deep, filled with uniform fine sand. The external diameter of the tube ranged from 14·7 to 70·5 mm and at diameter 29·6 mm the wall thicknesses were varied from 1 mm up to 14·8 mm (solid pile). The bearing capacity was determined by a piston jack.

It was found that within the limits of height/radius ratio tested there was practically no difference in the bearing capacity of a tubular pile and a solid pile of the same diameter; but there was a difference in the energy necessary to drive the tubes to the same depth. Two series of tests were carried out with tubes of the same external diameter but of various wall thickness.

Fig. 26 shows the bearing capacity of the tubes as a function of wall thickness. (A wall thickness of 14·8 mm represents a solid steel rod.) It is to be seen that both the ultimate bearing capacity and the ultimate resistance to pulling are practically the same for all the tested wall thicknesses. On the other hand, an increase in wall thickness incurs a progressive increase in the driving energy required.

We can conclude that these results are in agreement with the previous statements of W. Aichhorn and B. Fellenius. They have found for footings that the least bearing resistance is yielded by annular surfaces and the greatest by solid circular ones. Driving resistance is chiefly due to the bearing resistance of the surface at the toe of the pile and therefore tubular piles will need less driving energy than solid rods.

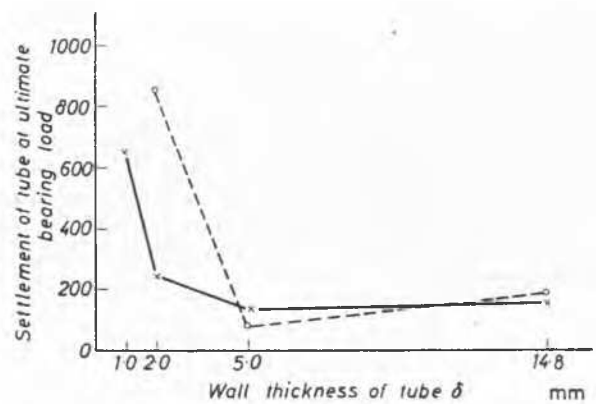

Fig. 6/23/c

The former statement concerning the equal bearing capacity in spite of the varying driving energy can be explained only by the development of a compacted inner earth-core within the tube which augments the resistance of the annular surface of the pile toe. The gradual compaction of this earth core was observed during the tests and it was noted that a decrease of 6 per cent in the original void ratio ($n = 0{\cdot}335$ per cent) of the sand was sufficient to ensure the full point resistance. This resistance will naturally develop only with larger displacements of the pile shaft owing to the greater compressibility of the earth core. This fact is demonstrated by Fig. 27 where the settlement under ultimate load is indicated as a function of wall thickness for tubes of the same diameter when their bearing capacity was practically the same. It is to be seen that at 1 mm wall thickness, when the diameter of the inner earth core was the biggest, this settlement was about four times as great as that in the case of a solid rod.

This phenomenon is similar to that of the steel sheet piles and box sections referred to above. An approximate theory verifying the greater settlement of a tubular pile is as follows:

Under a solid circular pile point the given load P will set up a specific point resistance of σ_t and it may be written that $P = R^2.\pi.\sigma_t$ and the settlement $(\Delta s) = (\sigma_t . t)/M$ approximately where M denotes the compression modulus of soil and t the depth of the influence ($t \cong 4R$).

Under a tubular pile with the same settlement the same load P cannot be exerted because underneath the smaller annular toe surface only a force $P' = (R^2 - r^2)\pi . \sigma_t$ can be transmitted to the soil. The soil-mass encased within the tube will not penetrate into the soil to the same amount Δs, because it will principally slide further into the tube itself and the force acting on the subsoil will be only that fraction P'' which can be transmitted to the inner surface of the tube by friction. This may be expressed: $P'' = \sigma_x . 2r\pi . \Delta h . \tan\delta$, where Δh is the height of the earth cylinder in the tube (Fig. 28), and $\Delta h = \Delta s - dh$ where dh

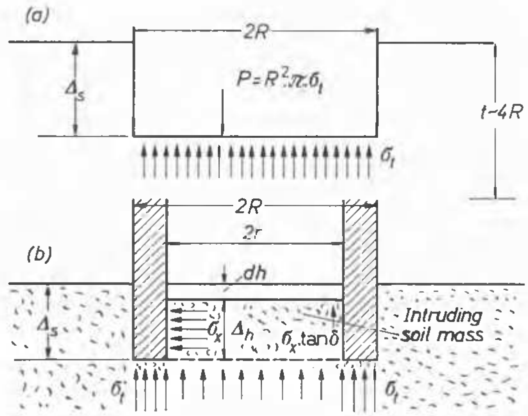

Fig. 27

is the compression of the intruding soil mass under the effect of the force P'', approximately $dh = P''/(r^2 \cdot \pi)$. t'/M' where t' and M' are the somewhat modified values of influence depth ($t' \cong 4r$) and of compression modulus which will be also functions of the change of void-ratio (Δe). According to Jamieson's silo-theory

$$\sigma_{x\max} = \frac{F}{U} \cdot \frac{\gamma}{\tan\delta} = \frac{r^2.\pi}{2r\pi} \cdot \frac{\gamma}{\tan\delta} = \frac{r}{2} \cdot \frac{\gamma}{\tan\delta}$$

and when substituting this value to the equation of P'' we get $P'' = r^2.\pi.\gamma.\Delta h$, which is the weight of the intruded soil mass and considering that $\gamma.\Delta h$ is essentially less than σ_t; $P - P'$ will be at any rate greater than P'', and if we want to transmit the same force P a difference of $P_o = P - (P' + P'')$ must be put in addition on the pile, which will undoubtedly set up a further ds settlement under the annular pile, which will be added to that (Δs) of the solid pile.

A. Vesić (Yugoslavia)

In Paper 3b/2 A. Bergfelt has presented some valuable observations on the behaviour of pile bents in soft clay. Special attention should be called to the fact that, when his piles were sufficiently long, the failures occurred by breaking of the piles, although the axial bearing capacity was not reached.

This points once more to the importance of the lateral resistance of piles which is still often disregarded in the analysis of foundations with batter piles. We wish to emphasize that the author's observations are in full agreement with recent theoretical investigations we have made on this subject.

In order to demonstrate the influence of lateral resistance we have examined different types of foundations with batter piles, studying the variation of axial reactions, transversal reactions and bending moments in the individual piles as a function of the relative lateral resistance of piles $\lambda = k_t/k_n$.

One of the typical foundations examined is shown in Fig. 29. The piles were reinforced concrete, 30 × 30 cm square; the exterior ones being at a batter of 3:1 or 4:1. Their axial coefficient of reaction was supposed to be constant and equal to $k_n = 10$ t/mm, while their lateral coefficient of reaction k_t varied from $k_t = 0$ to $k_t = 2$ t/mm. The cases of loading examined were: a horizontal load $X = 20$t in three positions X_1, X_2, X_3; a vertical load $Y = 100$ t in two positions: Y_1 (eccentric) and Y_2 (central). The axial reactions N, transverse reactions T and bending moments M in the piles were evaluated.

Fig. 30 shows the variation of $N(X)$ and $T(X)$; Fig. 31 the variation of $N(Y)$ in the batter piles 1, as a function of $\lambda = k_t/k_n$. In Fig. 32 the rotations ϕ of the foundation caused by the loads X and Y are given. In the abscissae are plotted also the corresponding values of Young's modulus of the soil E_s (after a theory assuming piles as beams in an elastic medium).

The type of curves shown in the illustrations is characteristic

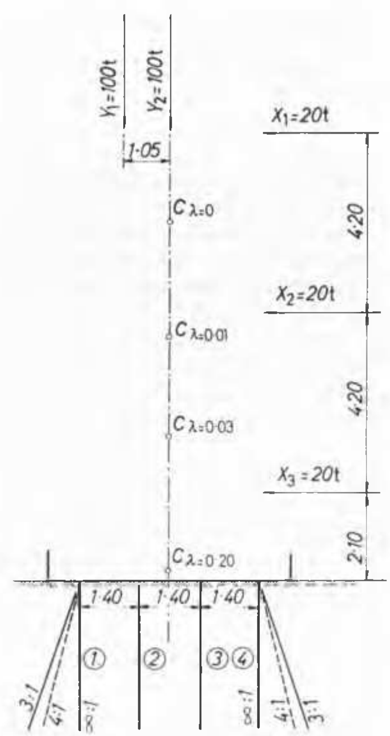

Fig. 29 The foundation. The shown positions of the elastic centre C correspond to the batter 4:1 of the exterior piles

Fondation. Les positions indiquées pour le centre élastique C correspondent à une inclinaison 4:1 des pieux extérieurs

for all the analyses: when the load is vertical and central the influence of the lateral resistance of piles is negligible; with the increase of inclination and eccentricity of the loads this influence becomes more and more important.

In such cases a relatively small lateral support of the soil is sufficient to change thoroughly the distribution of the reactions between the individual piles, and to decrease considerably the displacements of the foundation. Thus, neglecting the lateral resistance of piles we can often enter into considerable errors.

The conclusion that can be inferred from these computations, as well as from the observed facts, is: that in the analysis of stability of pile foundations, both the axial and the lateral resistances must be considered, although the latter are small compared to the former.

A very simple method for such an analysis has been developed recently (Vesić, 1956, 1957).

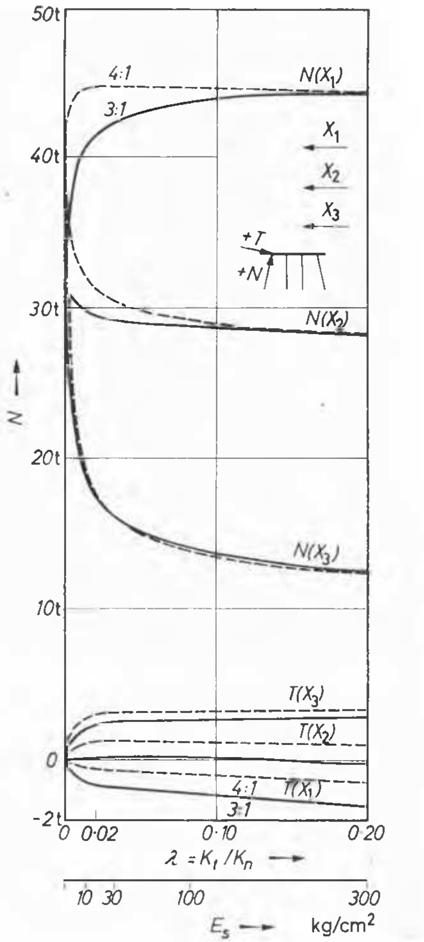

Fig. 30 Variation of the axial reactions N and the transversal reactions T in the exterior battered pile 1 as a function of the relative lateral resistance $\lambda = k_t/k_n$. Full lines correspond to a batter of 3:1; dotted lines to 4:1. Horizontal load $X = 20$ t in different positions: X_1, X_2, X_3

Variation des réactions axiales N et transversales T dans un pieu extérieur incliné 1, en fonction de la résistance latérale relative $\lambda = k_t/k_n$. Les lignes représentent une inclinaison de 3:1, les pointillés une inclinaison de 4:1. Charge horizontale $X = 20$ t dans différentes positions X_1, X_1, X_3

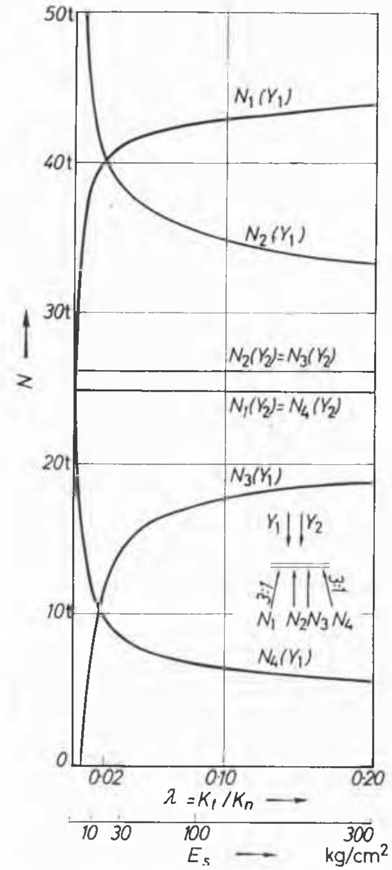

Fig. 31 Variation of the axial reactions N caused by a vertical load $Y = 100$ t in a central (Y_2) or eccentric (Y_1) position

Variation des réactions axiales N causées par une charge verticale $Y = 100$ t dans une position centrale or excentrique (Y_1)

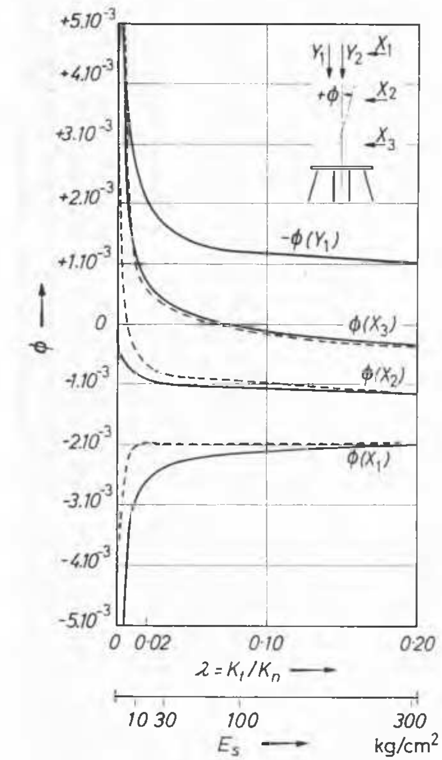

Fig. 32 Rotations ϕ of the foundation, caused by horizontal loads X or by an eccentric vertical load Y_1. Full lines correspond to a batter of 3:1; dotted lines to 4:1

Rotations ϕ de la fondation causées par les charges horizontales X on par une charge excentrique verticale Y. Les lignes correspondent à une inclinaison de 3:1, les pointillés à une inclinaison 4:1

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Vesić, A. (1956). Fundamental problems of the theory and analysis of pile groups. *Dr. Sc. Thesis, University of Belgrade* (in press)

— (1957). Contributions à l'étude des fondations sur pieux verticaux et inclinés. *Ann. Trav. publ. Belg.* (in press)

M. Buisson (France)

Les renseignements donnés par N. B. Hobbs (3b/8) sont précieux en ce qui concerne la pathologie des pieux moulés dans le sol.

Toutefois, nous pensons que le phénomène observé est plus général. Nous avons eu l'occasion de rencontrer de tels rétrecissements de section et délavage du béton en dehors des zones où l'eau est artésienne.

Ce phénomène peut aussi bien se produire dans tous les pieux bétonnés à sec dans le sol noyé lorsqu'on ne se protège pas contre l'intrusion de l'eau de la nappe, et ceci est d'autant plus difficile que la profondeur d'eau est plus grande.

Jusqu'à ces dernières années, je pensais qu'il suffisait de vérifier:

(1°) que le béton ne remonte pas dans le tube, au moment où on relève celui-ci;

(2°) que l'eau n'apparaissait pas à la surface du béton (ce qui se traduit par un mouillage du mouton servant au pilonnage du béton pendant le relevage du tube).

Toutefois, j'ai eu l'occasion de constater que, si ces conditions sont en effet nécessaires, elles ne sont pas suffisantes. Très souvent, les pieux traversent de l'argile assez molle, ou de la vase, avant de rencontrer une couche portante de gravier et de sable perméables. Ces phénomènes se produisent souvent dans les estuaires, là où la hauteur de marée entraîne des mouvements d'eau dans le sol. Ils se produisent aussi au voisinage de rivières ou de fleuves sujets aux crues. Il en résulte que dans tous ces cas, et surtout lorsque la profondeur d'eau dépasse quelques mètres, ce genre de pieux doit être écarté, du fait du risque trop grand. Ce risque de délavage et de rétrecissement du béton existe malheureusement aussi en ce qui concerne les pieux bétonnés dans le sol sous charge d'eau, pour éviter justement l'effet de l'eau artésienne. Les causes de déficience sont alors:

(1°) un accrochage du béton au tube en cours de relevage;

(2°) un mouvement de l'eau dans le sol, susceptible d'entraîner le ciment. Ce mouvement peut être dû à l'excès de pression d'eau dans le tube. L'eau remonte alors le long du pieu dans l'espace peu compact libère par l'épaisseur du tube. S'il est nécessaire d'équilibrer la pression d'eau dans le tube, il est donc aussi nécessaire de ne pas l'augmenter indûment. De fausses manœuvres peuvent conduire à une injection de ciment dans le sol très perméable (cas des pieux exécutés sous pression d'air comprimé ou d'eau comprimée). Même si ces causes disparaissent, l'entraînement des particules de ciment par l'eau du sol peut se produire avant prise de celui-ci. Pour l'éviter, il est bon, avant de bétonner le pieu, de mélanger de l'eau et de l'argile, et de colmater les pores de la zone perméable par injection sous faible pression. Il est bon également d'employer des accélérateurs de prise pouréviter une durée trop grande de prise de béton. Tout cela doit être contrôlé de très près.

Mais indépendamment des faits observés et reportés par N. B. Hobbs, et de ceux qui précèdent et s'y relient, je signale aussi les ruptures qui se produisent dans les pieux moulés dans le sol, lorsque d'autres pieux sont battus à côté. Si la prise n'est pas encore faite, il semble que des rétrecissements se produisent là où la résistance est moindre. Si la prise a déjà eu lieu, les ruptures observées sont de traction. Les surfaces de rupture sont horizontales. Un vide se produit entre deux tronçons du pieu. Ces phénomènes se produisent lorsque le terrain traversé est compact; ils sont observés aussi bien pendant le battage que pendant le bétonnage, si l'on essaie d'augmenter le diamètre du fût ou de la base d'une façon excessive eu égard à l'incompressibilité du sol (soit sous la pointe, soit dans la hauteur du fût). Ces coupures, que nous avons pu observer à plusieurs reprises, peuvent être decelées par la mesure du soulèvement de la partie supérieure du pieu dans lequel on place une barre de scellement au moment du bétonnage de la tête. On a observé des soulèvements de plusieurs millimètres à 7 m de distance d'un pieu battu de 12 m de longeur. Le soulèvement total est donc la somme des soulèvements dus au battage de tous les pieux battus dans un certain rayon.

Ce rayon croît avec la longueur des pieux. Nous avons constaté ainsi que ce soulèvement peut atteindre dans certains cas 7 cm et même plus. Il est certain que l'on peut s'attendre alors à un grand nombre de coupures qui en général ont quelques millimètres de hauteur, et à un affaissement important et inégal des pieux en cours de construction. Le critérium que nous avons retenu est un soulèvement total de 7 à 8 mm.

Dans tous les cas douteux, la mesure de relèvement de la tête du pieu s'impose. Dans les deux cas envisagés (celui de N. B. Hobbs et le cas signalé) il est nécessaire de prendre des précautions qui peuvent être différentes suivant les constructeurs: dans le premier cas, le chemisage est le meilleur remède; dans le deuxième cas, il peut en être un également. On peut aussi préparer la cavité du pieu soit en décarottant, soit en transformant le pieu battu en pieu foré avec les précautions indiquées précédemment.

R. Pietkowski (Poland)

I want to bring to your attention some interesting attempts to drive piles and sheet piles by vibration. I have found many descriptions of this in Russian literature, but no mechanical explanations of the phenomenon. I tried to do it myself, and arrived at some interesting results.

I take the numbers according to Russian literature. Assuming a = the amplitude of vibrations equal to 0·5 cm, n = vibration frequency equal to 25 c/s, I get the acceleration conforming to the well-known formula of dynamics:

$$p = a4\pi^2n^2 = 12{,}500 \text{ cm/sec}^2$$

e.g. about 13 times the terrestrial acceleration. If we take the weight of the pile as 2 ton, the superimposed weight 4·5 ton according to Russian recommendations, we get a force corresponding to statical force 89 ton, which is quite sufficient to drive in a pile in conditions of soil which are not difficult.

This computation explains the mechanical sense of the vibration method of driving in piles.

The Chairman

I now ask the General Reporter to summarize the discussion.

General Reporter

We have had today a large number of interesting contributions, and I shall not even attempt to summarize them. I should like to remind you that piles are a very practical subject. The many experiments and tests and theories that have been described for the bearing capacity of an individual pile are applicable to the taking of the load out of the pile into the bearing strata. However, as a number of the discussions on group loadings of piles have shown, the performance of a group of piles does not necessarily have very much relationship to the load capacity of an individual pile, and the distribution of load amongst the group of piles is not necessarily related to the loading capacity of an individual pile. Therefore, in practical engineering we are concerned with a judgment as to what we can do with a group of piles. Zeevaert also mentioned negative skin frictions in clays. This is, of course, the time phenomenon to which I referred, and is an important factor whenever piles penetrate through soft clays.

The Chairman

I am sure that we shall all be glad to hear some remarks from our President.

K. TERZAGHI, President (U.S.A.)

The General Report which has been presented by P. C. Rutledge has given us a very clear picture of the progress that has been made since the last conference in our understanding of the interaction between pile and soil, and we are certainly all grateful for the time and labour he has spent on preparing his Report.

The application of the procedures and the principles which have been described require, as a matter of course, first of all that the soil conditions at the site of the pile to which they refer are thoroughly known and, secondly, that the soil conditions are reasonably uniform over the entire area to be covered by the pile foundation. However, these conditions are by no means always satisfied, therefore it is necessary in an early phase of construction to find out by observation whether or not the assumptions on which the design was based are satisfied and to take steps to prevent the consequences of errors in our assumptions before it is too late. If that is not done the consequences can be very embarrassing, and I wish to illustrate them by the following examples.

About 11 or 12 years ago a large school building was to be erected. The main tract of the building was located on bedrock and one of the wings above a valley which was carved out of the bedrock and subsequently filled with sediments. The site was investigated by means of borings spaced 50 ft. both ways. The boring records indicated that the bottom of the rock valley and the slope located beneath the wing were covered with a layer of dense gravel with a thickness of about 10 ft. The gravel stratum was buried beneath a stratum of soft clay with a thickness up to 30 ft. overlain by a thin blanket of peat. The uppermost part of the subsoil consisted of recent fill. On the basis of this information it was decided to establish the wing on cast-in-place concrete piles to be driven to refusal into the gravel.

When the building was almost completed, settlement cracks developed in the sidewalls of the wing, whereupon settlement observations were started. The readings showed that the settlement of the structure increased during the following six months by amounts ranging between zero and 3½ in. The owners claimed that the contractor had failed to drive the piles to refusal into the gravel and filed a lawsuit against him. At that stage I was asked by the contractor to investigate the causes of the subsidence.

When I examined the contractor's construction records I noticed that the depth at which the piles supporting one individual cluster had met refusal varied by amounts up to 16 ft. In order to find out whether the piles had really met refusal, the uppermost portions of the piles supporting the footings at the site of maximum settlement were cut out, one by one. A loading test was performed on each of the cut-off piles and then the connection between the pile and the base of the footing was re-established. The tests showed that none of the piles started to move under a load of less than 100 tons and only two piles yielded under a load of less than 200 tons. Yet the dead load per pile was less than 40 tons, and while the loading tests were being performed the footing continued to settle. Thus it became evident that the gravel stratum contained large pockets of soft clay which had escaped the attention of the drillers. Most of the piles had met refusal in the gravel above clay pockets and the pile-supported footings settled on account of the gradual consolidation of pockets of soft clay located beneath the points of the piles, in spite of the fact that the gravel was dense enough to prevent the penetration of the piles into the clay.

Two years after this incident I had to design the pile foundation for a pulp and paper mill on Vancouver Island. The test borings showed that the subsoil consisted of soft silt and clay, resting at an average depth of about 40 ft. on a thick stratum of sand underlain by glacial till. I recommended using wood piles to be driven to refusal into the sand. When I received the first pile-driving records I noticed with alarm that the depth at which the piles met refusal varied within the same cluster by amounts up to 16 ft. This information reminded me of the situation I had encountered two years earlier in connection with the school building which I have described. I rushed to the site and requested that borings should be put down at the four corners of the cluster on which the penetration observations were made. To my pleasant surprise all the borings encountered the sand a short distance above the elevation of the shortest pile, and to a depth of 40 ft. below this surface, nothing but clean, coarse sand was encountered. Thus it became evident that the variations in the total depth of penetration were only due to erratic variations in the density of the sand.

On the strength of this fact, and the results of a subsequent survey of the topography of the sand surface, I decided that every pile should be considered satisfactory provided it met refusal below the surface of the sand. Settlement observations on the completed structure extending now over a period of 14 years show that none of the pile-supported footings settled more than 0·4 in. in spite of the fact that some of the footings support strongly vibrating loads.

N. JANBU (Norway)

In the excellent General Report for Division 3b, P. C. RUTLEDGE has called attention to the importance of being able to identify the conditions leading to under- or over-driving of piles. In this connection a brief review will be given of the investigations carried out at the Norwegian Geotechnical Institute for the purpose of establishing a pile driving criterion to avoid under- and over-driving, particularly of concrete piles.

Principle of criterion—The criterion is in principle based on the formulae which I derived in 1951. The formulae and the necessary notations are given in Fig. 33.

When introducing the equivalent static stress $\sigma_s = Q/A$, and solving formula (a) with respect to WH one obtains the following simple relationship:

$$WH = \epsilon' V \sigma_s \qquad \ldots. \ (1)$$

The abbreviation ϵ' is defined in Fig. 33, while V=pile volume.

For simplicity C_d and σ_s/E are introduced as constants in the formula for ϵ', which alters the meaning of σ_s to σ_n, herein called nominal stress. For reinforced concrete piles the following values have been adopted, $C_d = 1$ and $\sigma_s/E = 1/2{,}500$. With these alterations equation 1 reads:

$$WH = \epsilon_n V \sigma_n, \text{ where } \epsilon_n = \frac{1}{2{,}500} + \frac{2s}{L} \qquad \ldots. \ (2)$$

The value given above for ϵ_n is used for concrete piles only. (For timber and steel piles more information is still needed.)

Study of case records—On the basis of 37 cases records for reinforced concrete piles an attempt has been made to obtain

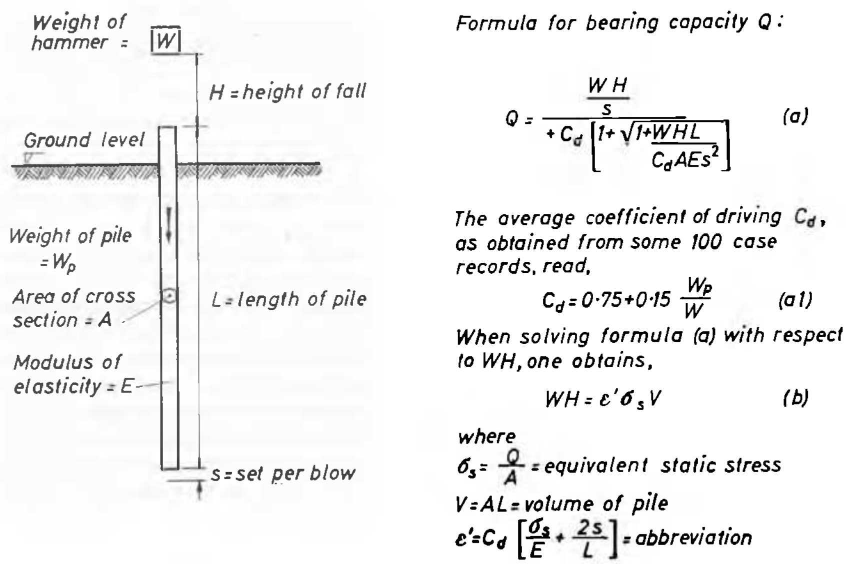

Fig. 33 Notations and formulae for bearing capacity during criterion
Notations et formules pour la capacité portante

the limiting values of the nominal stress σ_n which will ensure an adequate applied driving energy, *WH*, so that both under- and over-driving may be prevented.

To indicate the variation in the collected records the range of some essential data is summarized below:

Pile volume, V	$= 0{\cdot}6$–$5{\cdot}1$ m^3
Pile length, L	$= 6{\cdot}7$–$27{\cdot}7$ m
Driving energy, WH	$= 0{\cdot}9$–$6{\cdot}5$ tm
Ratio, $s/L \,.\, 10^4$	$= 0{\cdot}4$–$16{\cdot}7$
Test load, Q	$= 32$–204 ton

The soil conditions near and at the pile tip varied from silty sand to firm base (hard moraine or rock) except for three piles which were driven in clay.

From the available driving records the bearing capacity, Q, and the nominal stress, σ_n, have been calculated and the ratio between observed (load tests) and calculated bearing capacity is plotted *versus* σ_n in Fig. 34, from which the following observations are made.

For a nominal stress of say 70 to 100 kg/cm² the difference between calculated and observed bearing capacity is roughly ± 15 per cent for single piles, and less than ± 10 per cent for the average of several piles at the same foundation site. For σ_n greater than 100 kg/cm², however, the load ratio is always less than 100 per cent and it decreases rapidly with increasing σ_n.

The latter observation leads to the conclusion that the pile driving formula yields too high values of Q when $\sigma_n > 100$ kg/cm². I believe that this may be due to damage of the concrete around and between the reinforcement, as was actually observed on one occasion by withdrawing the pile. In some other cases it was found that the test load at failure when $\sigma_n > 100$ kg/cm² corresponded merely to the yield point of the vertical steel bars (neglecting the concrete).

To obtain an approximate idea of the actual stresses in the piles during driving, reference is made to Fig. 35, from which it is first observed that the nominal and static stresses differ only by some ± 20 per cent up to about 150 kg/cm². For higher nominal stress $\sigma_s < \sigma_n$.

On the basis of strain gauge measurements on steel piles driven to refusal (see R. C. Vold, *Norw. Geot. Inst. Publ.* 17) it has been found that the dynamic peak stress during driving was roughly 50 per cent higher than the average static stress using formula (a) Fig. 33 for $s=0$, and $C_d=1$, whence

$$\sigma_s = \left(\frac{WHE}{V}\right)^{\frac{1}{2}} \qquad \ldots (3)$$

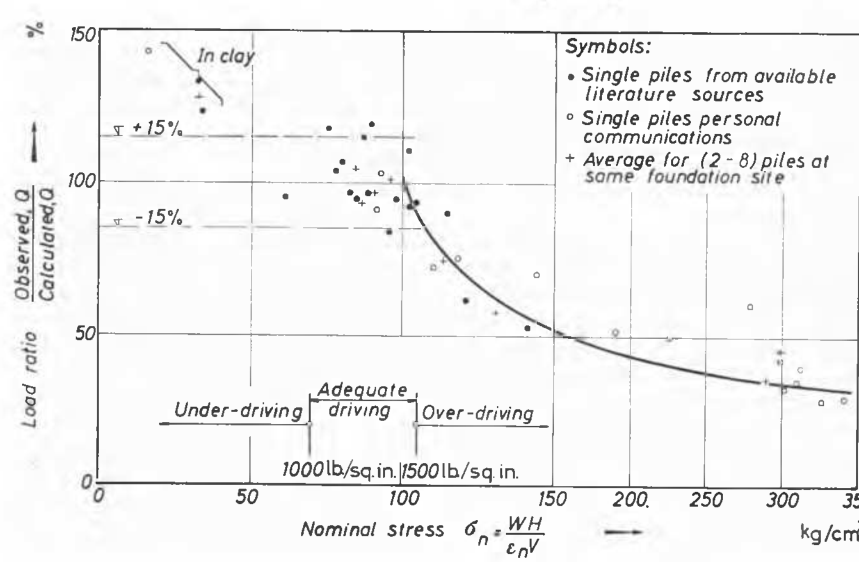

Fig. 34 Load ratio *versus* nominal stress for 37 reinforced concrete piles
Relation entre le rapport de charge et l'effort nominal pour 37 pieux en béton armé

Applying a stress concentration factor of 1·5 also for concrete piles and taking account of the scattering of the observation points in Fig. 35 the approximate range of possible peak dynamic stress will be as indicated by the shaded area, Fig. 35. If this picture is true, one may obtain dynamic stresses as high as 200 kg/cm² for a nominal stress just above 100 kg/cm².

In my opinion it is by no means surprising that repeated dynamic stresses changing from a small tension to a high compression of the order of magnitude of 20 kg/cm² may lead to severe damage of the complex, composite material in a reinforced concrete pile.

For pre-stressed concrete piles and for cylindrical piles with spiral reinforcement it is quite possible that the limit for overdriving may be taken higher than indicated by Fig. 34, but sufficient information is not yet available.

Tentative pile driving criterion for concrete piles—On the basis of the study above it has been suggested that the condition of safe, adequate driving of concrete piles be determined by

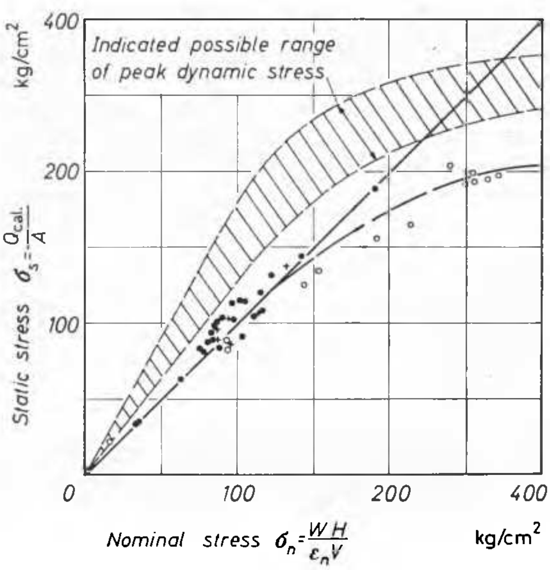

Fig. 35 Relationship between nominal and static stress with indicated peak stress range

Rapport entre l'effort nominal et l'effort statique pour la gamme des efforts dynamiques les plus élevés

requiring that the applied ram energy *WH* should be kept within the over-driving (U_o) and the under-driving (U_u) limits, that is:

$$\left.\begin{aligned} &U_o \geqslant WH \geqslant U_u \\ \text{where}\quad &U_o = \epsilon_n V\sigma_o = \text{energy limit for over-driving} \\ &U_u = \epsilon_n V\sigma_u = \text{energy limiting for under driving} \end{aligned}\right\} . \quad (4)$$

For concrete piles ϵ_n is calculated from equation 2 while $\sigma_o \sim 105$ kg/cm² $\sim$ 1,500 lb./sq. in., $\sigma_u \sim 70$ kg/cm² $\sim$ 1,000 lb./sq. in., and V= volume of pile.

It is believed that if $WH < U_u$ the pile may be under-driven so that the low bearing capacity of the ground will not permit full utilization of the allowable stress of the pile material, except perhaps in sensitive clays; and if $WH > U_o$ it is feared that the pile may either be damaged, or if not the higher bearing capacity of the ground may not be adequately utilized even with full utilization of the allowable pile stress. Both extreme conditions may lead to uneconomical design.

Example—Reinforced concrete piles with dimensions A= 0·125 m², L= 18 m and V= 2·25 m³ are to be driven through soft clay into a bed of gravelly sand underlain by rock. A drop hammer weighing 3 ton (metric) is used for driving. To prevent over-driving the applied energy *WH* should be limited according to equation 4. When using σ_o= 105 kg/cm² one can calculate the corresponding $WH = U_o$ for different values of s down to zero (refusal), see table below:

Table

Set per blow, s mm	*Maximum WH= U₀* tm	*Maximum drop H* cm	*Calculated bearing capacity Q* ton
3	1·33	44·5	111
2	1·20	40·0	117
1	1·08	36·0	123
0	0·95	31·5	131

By means of the table a field control during driving is readily performed. As the pile tip penetrates deeper into the sand bed and approaches the rock the set per blow decreases. According to the table the drop of the hammer must be decreased for decreasing s, and particularly if it is possible to drive the pile to refusal the height H should not exceed 30 cm, or 1 ft, for this example.

R. P. Milner (U.K.)

In reading the many useful contributions to this Division I find that only slight reference is made to the effect of negative skin friction on piled foundations, a phenomenon which in certain circumstances can be of major importance. This reference is made in Paper 3b/17 by L. Zeevaert.

In a recent case in the U.K., with which my Department was concerned, a building on pile foundations was enclosed by a 25 ft. high earth mound, through which an access tunnel of reinforced concrete, also on piles, was constructed. The internal face of the mound was supported by a retaining wall about 17 ft. high. The retaining wall end of the tunnel had settled excessively, and caused serious cracking of the tunnel. A brief description of this case is I feel of value, as apart from drawing attention to the effect of negative skin friction on piles it does also indicate the close agreement that exists between the practical case and the current theoretical pile design methods, which are as yet not generally accepted with complete confidence.

The soil profile in the area is a soft alluvium deposit approximately 25 ft. thick overlying boulders and soft to hard shales with limestone bands.

The reinforced concrete tunnel is founded on 15 in. diameter bored piles penetrating generally into about 5 ft. of the shale.

The piles were sunk about a year before construction of the mound was started. A gap was left in the mound for the tunnel itself which was started about two months after work on the mound had commenced. When about 3 ft. of soil had been placed on the top of the tunnel, 2 in. of settlement occurred suddenly at the retaining wall end. Settlement continued and in a period of about 6 months 7½ in. had occurred, thereafter the rate of settlement reduced considerably. Settlement of the base of the mound itself was of the order of 18 to 24 in.

Pile tests carried out in the vicinity of this site, where the soil profile was similar, indicated an ultimate load on a single pile of 90 to 100 ton. From a site investigation that was carried out on an adjacent area for another project, the results of undrained triaxial tests carried out on the soil deposits were used to determine the theoretical ultimate bearing load, including the negative skin friction due to the highly compressible soils, on a single pile. The result obtained was slightly in excess of 100 ton. The proportion of the negative skin friction in this figure was of the order of 80 per cent. The calculated ultimate bearing capacity of a single pile was found to be just under

100 ton. It was thus evident that the serious settlement that had occurred was due to the piles having been overloaded by negative skin friction.

Some of the remedial measures that could have been taken were as follows:

(a) To let settlement continue and after it had ceased to reinstate the tunnel.

(b) To take off the load of soil above the tunnel and thus partially reduce the pressure on the piles, and to replace this soil after sufficient consolidation of the compressible soils had taken place.

(c) To underpin the tunnel with additional piles.

In fact, a few months ago, method (b) was adopted and so far no further settlement has occurred.

It is clear from this case that where highly compressible soils are concerned the sequence of construction operations is of importance, as if the piles had been sunk after consolidation was significantly complete or partially complete, overloading due to negative skin friction may not have occurred; this was in fact the case on a neighbouring site where this procedure was adopted. Alternatively if speed is of importance, a sufficient number of piles should be incorporated in the design to take account of the negative skin friction.

R. A. Simpson (New Zealand)

The theoretical and experimental work carried out on the supporting power of individual piles and groups of piles offer the engineer a basis of design for piled foundations, but it remains for the engineer to undertake tests in the field to satisfy himself of the integrity of the design theories adopted and of the piles themselves for the duty required of them.

Unless test loading of piles is carried out there can be no certainty that construction is adequate to the service required, and unless such test loading is carried to the stage of ground failure little information is available to the research worker to enable him to check his theoretical and experimental studies.

The Institution of Civil Engineers, London, has set up a Committee to assemble data on pile loading tests and to study the information obtained. This Committee has requested the support of associated engineering institutions and Government departments in Commonwealth countries in assembling data on pile loading tests for study by the central Committee. The information requested is a full record of the pile, the site materials and of the test loading, and a questionnaire for the guidance of the contributors is available from the Committee. Investigations are intended to cover both pre-cast and cast *in situ* types of piling.

It is considered that any contributions which members of this conference can forward to the Institution of Civil Engineers on the subject of test loading of piles would add to the sum of knowledge on a most important and difficult subject, and would be of mutual benefit to the Institution and to this conference.