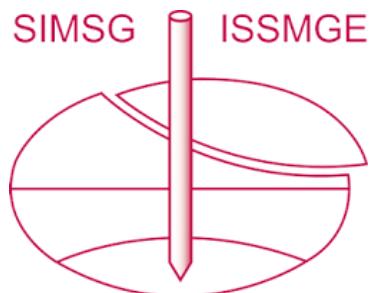


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Fondations superficielles

Shallow Foundations

Sujets de discussion : Influence de la dimension et de la forme des Fondations. Cas des sols non saturés. Expansion et retrait.

Subjects for Discussion: Influence of the dimensions and the shape of the foundation. Non-saturated soils. Expansion and shrinkage.

Président / Chairman :

J. VERDEYEN, *Belgique.*

Vice-Président / Vice-Chairman :

M. SALMON, *France.*

Rapporteur Général / General Reporter :

N. TSITOVTCH, *U.R.S.S.*

Co-Rapporteur / Assisted by :

R. TOKAR, *U.R.S.S.*

Membres du Groupe de discussion / Members of the Panel :

L.F. COOLING, *Grande-Bretagne*; O.K. FRÖHLICH, *Autriche*; P. HABIB, *France*, H. PEYNIRCIOLU, *Turquie*; J.G. ZEITLEN, *Israël*.

Discussion orale / Oral discussion

G. D. Aitchison, *Australie*

E. de Beer, *Belgique*

J. Coleman, *Grande-Bretagne*

L. J. Cooling, *Grande-Bretagne*

O. K. Fröhlich, *Autriche*

V. L. Granger, *Rhodésie*

P. Habib, *France*

J. E. Jennings, *Afrique du Sud*

A. R. Jumikis, *U.S.A.*

A. Kezdi, *Hongrie*

D. Krsmanovic, *Yougoslavie*

V. Mencl, *Tchécoslovaquie*

G. G. Meyerhof, *Canada*

U. Nascimento, *Portugal*

H. Neuber, *Allemagne*

H. Peynircioğlu, *Turquie*

K. Russam, *Grande-Bretagne*

J. A. J. Salas, *Espagne*

C. Szechy, *Hongrie*

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W. Ward, *Grande-Bretagne*

J. G. Zeitlen, *Israël*

Contributions écrites / Written Contributions

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J. Brinch Hansen, *Danemark*

J. B. Burland, *Afrique du Sud*

B. O. Corbett, *Grande-Bretagne*

E. de Beer et E. Lousberg, *Belgique*

A. Eggestad, *Norvège*

M. I. Gorbunov-Possadov, *U.R.S.S.*

H. Grasshoff, *Allemagne*

A. R. Jumikis, *U.S.A.*

A. Lazard et G. Gallerand, *France*

D. M. Milovic, *Yougoslavie*

E. Schultze, *Allemagne*

H. U. Smoltczyk, *Allemagne*

N. M. Sokolov et E. A. Sorochan, *U.R.S.S.*

R. Tokar, *U.R.S.S.*

Le Vice-Président :

Mesdames, Messieurs, en qualité de délégué du Comité d'organisation, j'ouvre la séance des discussions techniques concernant la Section 3A.

Au cours d'une réunion préparatoire les membres de ce groupe ont, d'une part retenu à l'ordre du jour les deux sujets de discussions suivants, relatifs aux fondations superficielles :

— Influence de la dimension et de la forme des fondations ;

— Expansion et retrait des sols non saturés ;
d'autre part convenu des conditions dans lesquelles la discus-



N. TSITOVTCH

Rapporteur Général, Division 3A / General Reporter. Division 3A

sion aura lieu, afin de la discipliner et de la rendre aussi fructueuse que possible. C'est ainsi qu'après la présentation de son rapport, par le Rapporteur Général, qui mettra en évidence, pour les sujets en question, les progrès réalisés en matière de Mécanique des sols et analysera les études récentes, des communications seront faites par les auteurs dont les noms vous seront donnés tout à l'heure par le président de séance, et qui ont apporté sur les questions traitées une contribution particulièrement importante. Ensuite, les discussions interviendront sur ces communications et en général sur toutes les questions que vous poserez, à condition bien entendu qu'elles se rapportent à l'ordre du jour.

La confrontation des connaissances des participants à la discussion, par les enseignements qu'elle procurera, permettra ainsi de faire progresser sur ces sujets une science en constante évolution.

Je recommande aux personnes qui participeront aux discussions de parler lentement afin de ne pas rendre trop difficile la tâche des interprètes et d'éviter de lire des papiers trop longs afin que la discussion reste vivante et animée.

Je passe la Présidence de la Séance à M. Verdelyen.

Le Président :

Mesdames, Messieurs, je remercie tout d'abord M. le Vice-Président Salmon pour son excellente introduction et j'aborde directement notre ordre du jour.

En premier lieu M. le Rapporteur général va donner lecture de son rapport. La parole est à M. le Professeur Tsitovitch.

Le Rapporteur Général :

My report is on *General Subjects of Theory and Practice of Foundation Engineering on Natural Soils*. In order to emphasise the most important ideas contributed by each paper to our knowledge in this field it was necessary to combine all the 53 papers received into six different groups.

The first group — bearing capacity of soils :

The papers on the bearing capacity of soils cover (a) theoretical investigations, (b) description of foundation tests and (c) particular problems.

The second group — stress distribution in soils, including the contact problem :

Six papers on stress distribution in soils have been presented to this Conference. Four of them cover theoretical problems and two papers are on the experimental determination of pressures over foundation basis.

The third group — settlement of structures :

For convenience all the papers of this group have been combined under the following headings : (a) the determination of complete stabilised settlement of foundations; (b) problems of the consolidation theory and (c) the results of observation of structure settlement.

The fourth group — foundation calculation and design :

The analysis of the papers, as well as the references, shows clearly that foundation calculation and design in respect to admissible pressures on the soil does not always guarantee the strength and the stability of the foundations, so that often it is necessary to take into account settlements of compressible bases and their non-uniformity.

The papers to be considered in this group are combined into the following sub-groups : (a) flexible foundations, (b) rigid foundations and (c) the peculiarities of designing foundations for certain types of structures.

The fifth group — foundations in regional soil conditions.

This group includes eight papers covering the problems of building foundations on regional soils : subsidence on wetting under load (highly porous loesses, eluvial soils formed as a result of soil weathering and others), considerable swelling on wetting, shrinkage on drying and made filling grounds.

It is interesting that two authors recommend measures to counteract the deformation of foundations built on swelling soils similar to those applied in counteracting the frost-heaving.

The sixth group — special problems :

This group includes papers covering the following problems : (a) stability, (b) dynamics of soil bases and foundations and (c) special cases in foundation engineering.

The main problem in the theory and practice of foundation engineering consists in the calculation and design of foundations. All the other groups of papers cover only particular aspects of this general problem.

Summary : The study of the papers submitted to the Fifth Conference and the publications issued in the period between the two Conferences clearly shows that the correct evaluation of the limits of applicability of individual solutions of soil mechanics problems and the improved methods for determining the characteristics of soils enable us to apply the solutions of the theory of soil mechanics in practice with great confidence.

The reports to Division 3A of the Fifth Conference, although they do not contain great discoveries, still advance a number of useful suggestions that can be used in the developing of the engineering design. It has been established that as a rule the magnitude of the bearing capacity of bases, as determined experimentally, by far exceeds the calculated one, which points to the insufficiently accurate evaluation of the boundary conditions and to the necessity of improving theory. The elaboration of solutions to the problem of stresses and deformations of a finite soil layer (resting on a rigid base) is a substantial contribution to the theory of stress distribution.

An important achievement in the field of foundation settlements consists in the further elaboration of the problem of the deformation of two-layer base and the successful application of vertical drainage for the acceleration of the consolidation of clay soils. However, the problems of secondary soil consolidation, the calculation of bases for creep and the limits of applicability of the filtration theory of consolidation have been largely neglected.

It may be considered as an established fact that flexible foundations should be designed taking into account the interaction between the structure and the compressible base, while rigid foundations should be designed proceeding from the settlement of soil bases determined by the limit deformations of superfoundation structures.

Although a number of papers submitted to the Fifth Conference are concerned with the problem of foundations in peculiar soil conditions the general theory of the mechanics of peculiar soils has not been elaborated to any satisfactory degree.

Proposals for discussion have been published in Vol. II of the Proceedings of this Conference and in Bulletin № 2. They may be combined as follows :

1. The limits of applicability of the solutions of soil mechanics, including influence of the dimensions on the shape of the foundations on their settlements.

2. Problems of the general theory of non-saturated soils, of the mechanics of swelling and expansive soils, subsidable loess soils and the mechanics of organic masses.

3. The design of foundations of limits states.

Le Président :

Je remercie vivement M. Tsitovitch pour la lecture de son rapport. Vous avez pu constater qu'il est particulièrement clair et que c'est un document de travail qui nous servira lors de l'examen détaillé des diverses communications. En votre nom je félicite M. Tsitovitch pour son travail; vous pourrez tous apprécier combien il a dû faire d'efforts

pour arriver à présenter un rapport aussi rapidement et aussi clairement.

J'aborde maintenant l'objet de nos discussions. Comme vous l'a rappelé M. le Vice-Président Salmon, le groupe de discussion a retenu deux sujets : l'un, l'influence de la dimension et de la forme des fondations ; le second est relatif aux sols non saturés. J'aborde donc le premier sujet.

Influence de la dimension et de la forme des fondations

Je donne la parole à M. Habib qui va, je crois, vous parler des coefficients de forme.

M. HABIB (France)

M. le Président, Messieurs, mes chers collègues,

La forme et la dimension des fondations peuvent intervenir de différentes façons, mais c'est essentiellement leur influence sur la valeur de la force portante limite et du tassement qui est importante. Je me limiterai à quelques remarques relatives à la force portante.

L'influence de la dimension des fondations sur la résistance à la rupture est à peu près clairement établie à l'heure actuelle. A la surface d'un milieu pulvérulent des fondations homothétiques portent des charges dont le rapport est égal au cube de l'échelle des longueurs. Autrement dit, si un cube d'une certaine densité est en équilibre limite sur un sable, tous les cubes en même matière seront en équilibre limite. Ce résultat est d'autant plus intéressant qu'il n'est pas vrai pour les structures que ce soient celles qui sont créées par l'homme, ou celles du monde animal. Il n'est pas vrai non plus pour les fondations sur les massifs purement cohérents : cette fois la pression limite est constante et indépendante de la dimension. L'influence de la forme, par contre est encore à ce congrès même l'objet de nombreux travaux sans qu'un accord complet se soit établi.

Mais avant d'aller plus avant il faut peut-être préciser le mot coefficient de forme car le langage n'est pas toujours le même d'un auteur à l'autre. Pour cela je vais comparer deux fondations, l'une en forme de bande de longueur infinie, l'autre rectangulaire et de même largeur que la première : j'appelle coefficient de forme le rapport du taux de travail à la rupture de ces deux fondations. Il est évident que pour des fondations très allongées ce coefficient est voisin de l'unité et qu'il s'en écarte le plus pour la forme carrée. Avec la définition que je viens de donner on constate que pour les sables le coefficient de forme mineure le taux de travail de 20 à 40 pour cent : pour les argiles, il le majore de 20 à 40 pour cent. Si je donne des valeurs extrêmes, c'est pour encadrer les résultats de chacun, car effectivement les valeurs proposées ne sont pas les mêmes selon les auteurs. Voici donc un sujet pour lequel nous disposons de résultats expérimentaux nombreux et récents : je pense en particulier à ceux de M. Meyerhof, qui est à cette table, de M. de Beer, de M. Kezdi et de bien d'autres encore. Ce sujet devrait être clair, or, il ne l'est pas; il y a donc quelque chose qui masque les conclusions : c'est la dispersion des résultats.

Mon opinion sur cette question est la suivante : dans l'état actuel, il est parfaitement inutile de s'occuper du coefficient de forme sauf peut-être pour les argiles, et voici pourquoi. Nous ne disposons pas, pour l'instant d'une théorie complète et satisfaisante pour les problèmes à trois dimensions. Dans de nombreux cas et en particulier pour celui des fondations superficielles, nous sommes donc obligés de nous appuyer sur l'expérience pour obtenir certains coefficients caractéristiques. Or si nous regardons les résultats expérimentaux nous constatons que la dispersion des résultats est de l'ordre de 1 à 2, peut-être même de 1 à 3. La Fig. a représente l'influence de l'élancement d'une fondation sur le taux de travail. En abscisse j'ai porté le rapport de la longueur à la largeur de la fondation et en ordonnée la pression limite divisée par la largeur.

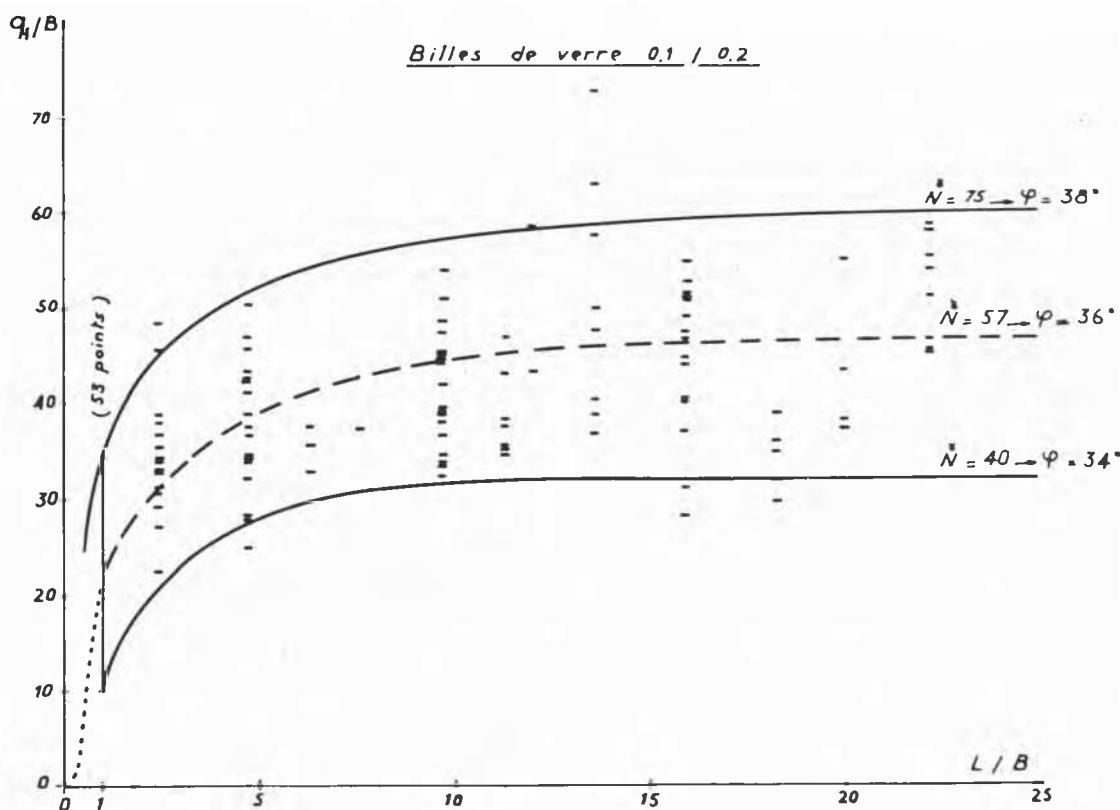


Fig. a

Diagramme de fréquence de la portance relative

Billes de verre 0.1 / 0.2

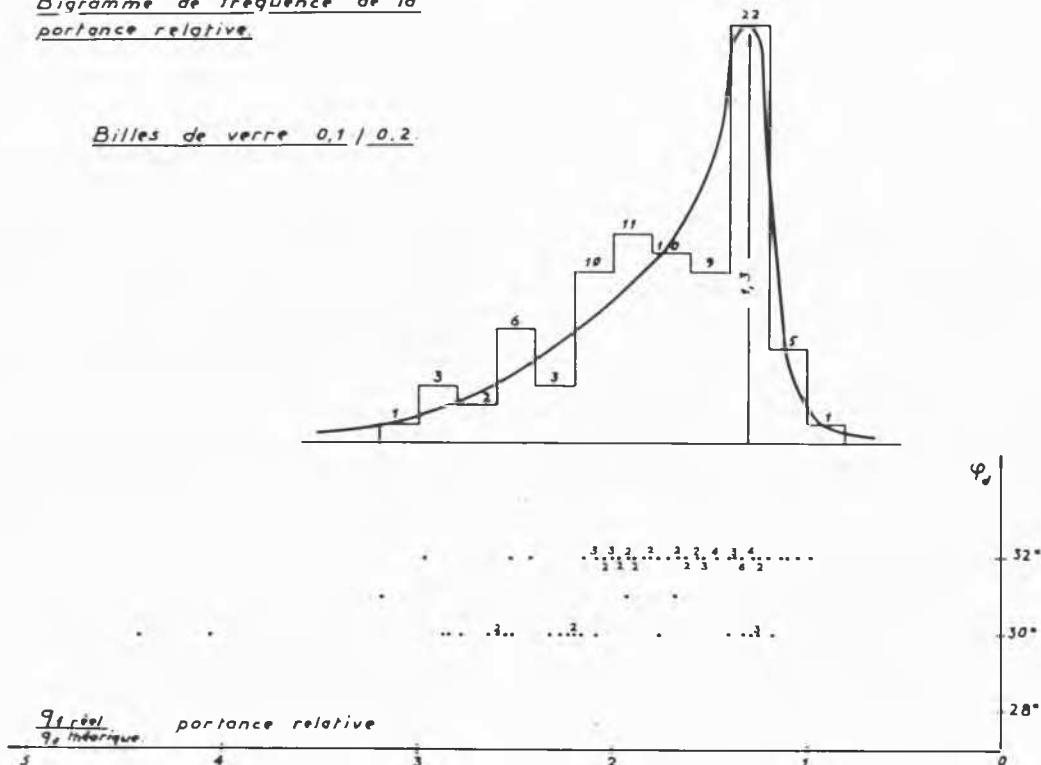


Fig. b

On voit l'énorme dispersion des résultats que l'on obtient dans des essais de cette nature.

La Fig. b représente un diagramme de fréquence. Il a été établi à partir d'une partie des résultats qui figuraient sur la figure précédente, mais porte encore sur un nombre considérable d'essais : je note occasionnellement d'ailleurs qu'il est bien clair sur cette figure que la répartition des écarts ne correspond pas à une distribution de Gauss. Ainsi, même en laboratoire, dans des conditions tout-à-fait satisfaisantes, avec une expérimentation soignée, il est nécessaire de répéter les essais un nombre considérable de fois pour pouvoir obtenir une précision à peine satisfaisante, celle que j'indiquais tout à l'heure. Si maintenant nous revenons aux problèmes pratiques et en tenant compte de l'incertitude qui règne sur les caractéristiques réelles des sols, incertitude due au fait que nos enquêtes sont toujours parcelaires, on est en droit de se demander ce que représente la recherche d'une précision de l'ordre de 10 pour cent.

Les remarques précédentes ne tiennent pas compte de la dispersion naturelle des propriétés mécaniques des sols dont nous n'avons pas en général une connaissance très précise car les études statistiques sont malheureusement très rares encore. J'ai dit tout à l'heure que le seul cas où la recherche du coefficient de forme pouvait à la rigueur se justifier était celui des argiles. En effet lorsque la rupture se fait dans des conditions telles que le frottement interne soit nul on peut atteindre, au laboratoire, une précision de l'ordre de 5 pour cent et sur des produits artificiels de l'ordre de 1 pour cent lorsque les manifestations visqueuses ne sont pas trop importantes : il peut être utile de donner alors une formule de rupture avec un terme correctif, mais notre optimisme doit être tempéré même dans ce cas, car la fissuration des argiles, la dispersion naturelle etc... sont encore des causes d'erreur nettement plus importantes. Dans tous les autres cas, je pense qu'il est illusoire d'introduire un coefficient de forme pour le calcul du taux de travail admissible.

Je n'ai envisagé pour l'instant que le terme de cohésion et le terme d'appui. Mais le calcul d'une fondation superficielle contient presque toujours un terme de profondeur et l'influence de la forme doit exister là aussi. Il est bien évident que l'étude expérimentale de ce terme correctif est particulièrement délicate et que le problème général est extrêmement ardu.

Le Président :

Je vous remercie, Monsieur Habib, je vais maintenant donner la parole à M. Fröhlich qui a l'intention de nous parler de la dispersion des résultats.

M. O. K. FRÖHLICH (Autriche)

This question relates to shallow foundations :

(1) of relatively small dimensions (so-called footings and of square, circular or rectangular shape);

(2) on infinitely long strips;

(3) on foundation mats or rafts of relatively great dimensions.

1. As regards the influence of dimension and shape *on the bearing capacity* of the foundation (that means the critical load, under which rupture of the soil occurs), this influence is described and computed in "Theoretical Soil Mechanics" by Terzaghi (1942).

The difference between a circular, square or regularly polygonal shape is negligible — all that matters is the size of the contact-area between foundation and soil, as long as the soil is homogeneous. This method of judging the influence of size and shape of the foundation *on the bearing capacity* leads to the conception of the *critical ground pressure*.

To find the allowable pressure in the contact surface, we must decide on a *factor of safety* say : 3 or 4 or still higher.

2. There is another possibility to determine the allowable soil pressure for the design of the foundation.

We may define the allowable pressure as *that* pressure which produces the *first small plastic deformations* at the rim of the contact area of the foundation.

The theoretical computation of this value requires the following data :

- (1) the foundation depth;
- (2) the unit weight of the soil :
 - (a) above the contact area and
 - (b) underneath it;

(3) the highest level of the groundwater, which may be expected;

(4) the Coulomb-parameters c and ϕ of all layers underneath the foundation.

The formula to be used for the calculation of the above-defined allowable soil pressure can be found in literature on soil mechanics (Fröhlich 1934). I should not like to trouble you with further details in this respect.

After having found the allowable pressure in the contact area for the design it is necessary to estimate the settlement of the structure to be expected. For this purpose we have to know *compressibility* and *permeability* of all layers underneath our foundation.

Further we have to decide on the distribution of the soil pressures underneath the foundation and over the contact area itself. In general, the distribution of the soil pressures is taken from the theory of elasticity (of the so-called elastic-isotropic half-space), even for stratified underground.

As regards the distribution of the pressures over the contact area, this depends on the degree of the rigidity of the foundation itself. For the first estimation *it is sufficient to suppose even distribution of pressures over the contact area.*

The further steps of the settlement analysis may be taken from literature, where circular, square, rectangular, elliptical and infinitely long contact areas are dealt with.

The result of the settlement analysis will show, if the calculated *admissible* soil pressure leads to an *acceptable* settlement or not. It is much easier to make this decision for the designer than to choose the correct factor of safety against rupture.

With regard to the *increasing settlement with time* by pore pressure dissipation, Soil Mechanics literature since the first International Conference in Cambridge (Mass.) gives the required data to the designer.

Le Président :

La parole est à M. Meyerhof.

M. G. G. MEYERHOF (Canada)

I should like to make a few remarks on the influence of shape and size of foundations on the ultimate bearing capacity of spread foundations on cohesionless soils. For shallow foundations on such soils the theoretical bearing capacity can be expressed by Terzaghi's well-known equation

$$q = \gamma D N_a + \frac{\gamma B N_y}{2} \quad \dots \quad (1)$$

where B = width of foundation, D = depth of foundation and γ = effective unit weight of soil. The theoretical bearing capacity factors for horizontal strip foundations under vertical load depend only on the angle of internal friction ϕ and are

$$N_a = e^{\pi \tan \phi \tan^2 (45^\circ + \phi/2)} \quad \dots \quad (2)$$

according to Reissner (1924) and, very nearly,

$$N_y = (N_a - 1) \tan (1 \cdot 4 \phi) \quad (3)$$

The latter equation is in good agreement with the theoretical factors derived by Messrs Caquot and Kérisel (1956), Lundgren-Odegaard (1953-1961) and Meyerhof (1955) all of which give practically identical results. For circular footings the theoretical bearing capacity factors for sands ($30^\circ < \phi < 45^\circ$) are roughly twice as great as for strip footings as shown by Berezantzev (1952) and Meyerhof (1951 and 1955).

As indicated by the speaker in this paper to this Conference (3B/16), the soil under strip foundations is in a state of plane strain and the corresponding angle of internal friction ϕ has to be determined by plane strain compression tests at the same density of the sand, if loading tests on strip foundations are to be compared with the theory on a proper basis. Moreover, the normal pressure on the shear plane in such compression tests must be the same as the average pressure on the failure surface in the loading tests so that the shearing strength is comparable in both cases (Meyerhof, 1948). Thus it has been found by Messrs Bishop and Janbu (1957) that the angle of internal friction, especially for dense sand, in plane compression tests is roughly 10 per cent greater than in triaxial compression tests. This increase in the friction angle is due to the effect of the intermediate principal stress on the shearing strength, as has also been shown by tests of M. Habib (1953) and is also obtained for dense sands in direct and ring shear tests. It should further be remembered that the angle of internal friction of sands increases, as the normal pressure on the shear plane decreases, which is particularly important for loading tests on small-scale footings (Meyerhof, 1948). The results of model tests on surface strip footings by Messrs De Beer and Ladanyi (3A/4) are in excellent agreement with the theory for strips if the angle of internal friction under strip footings is taken to be about 10 per cent greater than the triaxial angles at the same density given by the authors, as shown in Fig. 1.

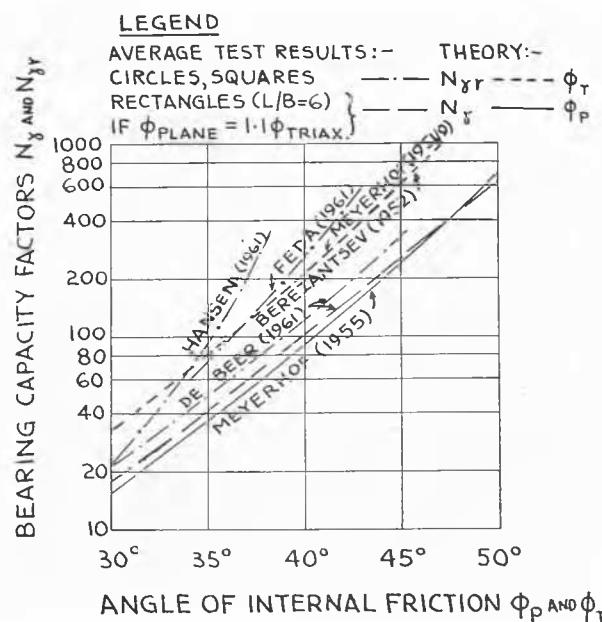


Fig. 1 Comparison of experimental and theoretical bearing capacity factors for footings on surface of sand.

On the other hand, the soil under circular (or square) foundations is in a state of axial symmetry, at least near the centre of the footing, while near the edges of the footing the plane state of stress is approached. As a first order of approximation, the angle of internal friction in this case can therefore be determined by triaxial compression tests on the sand at the same density and average normal pressure on the shear plane as under the footing. If the present loading tests by Messrs De Beer and Ladanyi on circular model footings on the surface of sand are compared with the theory for circles using the triaxial friction angles, the observed ultimate loads are somewhat smaller than predicted, as shown in Fig. 1. Consequently, the empirical shape factors are found to be about $1\frac{1}{2}$ compared with theoretical values of about 2. While the tests on circular and square footings by Messrs L'Herminier, Habib et al. (3A/26) give a similar result, the loading tests by Messrs Bent Hansen (3A/17) and Feda (3A/13) give empirical shape factors of the order of 2 to 4, as estimated (see Fig. 1). The differences between the empirical shape factors for circular footings by various authors may be due to variations in the methods of packing the sand, the procedures and interpretation of the loading tests and the triaxial compression tests.

Full-scale loading tests on sand in the field by different authors (Muhs, 1954, Meyerhof, 1951, 1953) give substantially the same general relationship of the ultimate bearing capacity as model tests in the laboratory. These tests show that the ultimate bearing capacity increases as the width and depth of spread foundations increases, the increase being somewhat less than estimated theoretically. This difference is due to a reduction of the effective angle of internal friction of sands, which in loose material may be explained by the effect of compressibility, while in dense material the friction angle decreases as the normal pressure on the failure surface increases, as mentioned before.

It may therefore be concluded from recent research that we now have a sufficiently accurate theoretical approach, which together with the appropriate soil test data enables us to estimate the ultimate bearing capacity of spread foundations on sands, as well as on clays, for most, if not all, practical purposes.

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Le Président :

Je vous remercie, M. Meyerhof. M. Habib demande la parole pour intervenir dans la discussion.

M. HABIB (France)

M. Meyerhof, je voudrais vous poser deux questions.

La première est relative à l'influence de la contrainte principale intermédiaire. Nous savons que cette influence est très faible sur la résistance au cisaillement de l'argile. Nous savons aussi que l'influence de la forme des fondations sur argile est différente et opposée à celle des sables. Pensez-vous qu'il s'agisse là d'un effet théorique normal et que l'influence de la contrainte intermédiaire sur la résistance au cisaillement des sables joue en somme un effet double puisqu'il y a à enlever une majoration, puis à prendre une minoration ?

La deuxième question est relative aux corps qui ont à la fois de la cohésion et un angle de frottement interne. Nous avons à l'heure actuelle énormément de travaux expérimentaux sur des corps qui ont soit l'un, soit l'autre. En particulier sur les sables beaucoup de choses ont été faites. Par contre, nous sommes beaucoup moins sûrs pour des corps plus généraux.

Pour le calcul d'un taux de travail lorsqu'un sol présente à la fois de la cohésion et du frottement interne, on majore le terme de cohésion et on minore le terme d'appui, mais, à ma connaissance on n'a aucune raison théorique de procéder ainsi et c'est presque un aspect sentimental. Malheureusement dans un tel cas, le raisonnement peut difficilement s'appuyer sur l'expérience. En effet les taux de travail sous la bande infinie et sous le carré deviennent très voisins car les deux corrections s'opposent et la dispersion masque le phénomène d'une façon de plus en plus complète. C'est pour cela que je continue à penser que la recherche d'un facteur correctif est tout à fait illusoire car la dispersion naturelle des caractéristiques des sols est hors de proportion avec la précision cherchée.

Le Président :

Voilà deux questions précises posées par M. Habib, je suppose que M. Meyerhof va lui répondre.

M. G. G. MEYERHOF

The shape factors are theoretically greater than unity not only for clay but also for sand, and thus that there is no fundamental difference between them. When we carry out triaxial compression tests on clays and compare the results with plane strain compression tests, the difference may be small; but I understand that such tests have not yet been completed. Possibly such results may show a difference of only about 10 per cent; in that case the corresponding theoretical bearing capacity would only differ by about 10 per cent, which is well within experimental errors and can therefore hardly be detected.

When we are dealing with sands, on the other hand, a difference of a few degrees in the angle of internal friction immediately shows up in the results loading tests. So far as strip foundations are concerned, we can judge them only by comparison with plane strain compression tests, in which the intermediate principal stress is perhaps of the same order of magnitude as under the strip foundation. If we have a soil which has both cohesion and internal friction, then presumably we should also carry out plane strain compression tests on such soils if we wish to design strip foundations, and the intermediate principal stress will have a similar effect, possibly of the order of 10 per cent, on both cohesion and internal friction. We then use the theoretical bearing capacity factors with these shear strength parameters for strip foundations. On the other hand, if we design circular foundations we would carry out triaxial compression tests on such soils to obtain the corresponding shear strength parameters and use these in connection with the theory for circular foundations. The corresponding theoretical bearing capacity

factors for cohesive soils have been obtained by Mr Berezantzev and by myself. There is no need therefore to use any empirical shape factors.

However, if we compare the results of loading tests on strip foundations directly with loading tests on circular foundations, we obtain completely different shape factors for frictional materials because we are not using any theory or soil tests but we are merely making a comparison. Since the intermediate principal stress raises the angle of internal friction, the bearing capacity of the strip is raised to such an extent that it exceeds the bearing capacity of a circular foundation, and the empirical shape factor is thus exactly opposite to that obtained theoretically.

Le Président :

Je vais donner maintenant la parole à M. Peynircioglu qui va nous parler du problème des tassements.

M. H. PEYNIRCIOLU (Turquie)

The main factors responsible for the discrepancies between calculated and observed total settlements may be summarised as follows :

- (i) the approximate nature of the influence of the depth and shape of the foundation;
- (ii) assumptions made regarding the rigidity of the foundation and superstructures;
- (iii) the difference between the real and assumed contact pressure distribution;
- (iv) the difference between the calculated and actual stresses in the soil mass;
- (v) ratio between shear stresses and shear resistance;
- (vi) the difference between the geotechnical properties determined in laboratories or in the field and their actual values *in situ*.
- (vii) the assumed averaged geotechnical properties of the foundation soil and the real values;
- (viii) the difference between the simplified and actual geology of the site and assumptions made regarding the history of the layer.

For the regular layers of definite and clear geology the last two points are of secondary importance. Here I want to draw attention to the sites which are underlain by layers which cannot be regarded as uniform, or not suitable for laboratory testing, namely, (i) soft or medium clays containing erratically sandy seams and lenses, or layers overconsolidated by drying, or peaty materials, or both; (ii) hard and fissured clay layers; (iii) gravel and boulders embedded in a sandy and silty soft or medium clay matrix; (iv) artificial or semi-artificial deposits composed of all kinds of debris, refusal and remnants of materials such as brick pieces, shells, bones, city refusals, etc., and with a variable content of sand, mud and peat.

Along the shores of the Golden Horn such layers cover large areas and their thickness is up to 40 metres. They are underlain mostly by devonian formations or sandy and silty and clayey layers. The total settlements of the structures on such areas are composed of immediate and consolidation settlements, settlements due to secondary time effect and plastic flow of the soil under the buildings, and settlements due to other earth movements which are caused by factors other than the weight of the structure, such as complete or partial slides, creep, earthquake shocks, etc. The average over-all compression modulus for such layers can best be determined from settlement observations of old buildings.

In cases of new areas of this kind, where there are no buildings or no observation data, test loading might be of great help. Settlements calculated for structures on such layers with the routine methods need to be multiplied by

correction factors determined on the basis of observation or test loadings, geology and professional experience.

Now I will show you some slides showing some photographs of these shores :

Fig. 2 : Here you see a building built in 1880 on raft foundation and tilted towards the sea. The building itself is not so heavy, but still tilt has occurred. To stop the tilting a new building, 1953, has been built on wooden piles. But the creep of the soil towards the sea continued and the building began to push the new building, so they had to demolish the two upper storeys.

Fig. 3 : This photograph shows the tilted building and the one built in front of it.

Fig. 7* : This is an old fountain built about 200 years ago. It has settled more than 180 cm. Recent borings and laboratory tests and recalculation have given a much smaller settlement. The cause of this excessive settlement and tilt may be the movement of the shore towards the sea. The water surface on the photograph is at the sea level.

Fig. 5* : This is an example of the geology of the shores of the Golden Horn. The bed rock is very steep towards the sea and of devonian formations, shales, greywacke, etc., and the geotechnical properties of the matrix of the debris and refusals are given on the lower part of the figure.

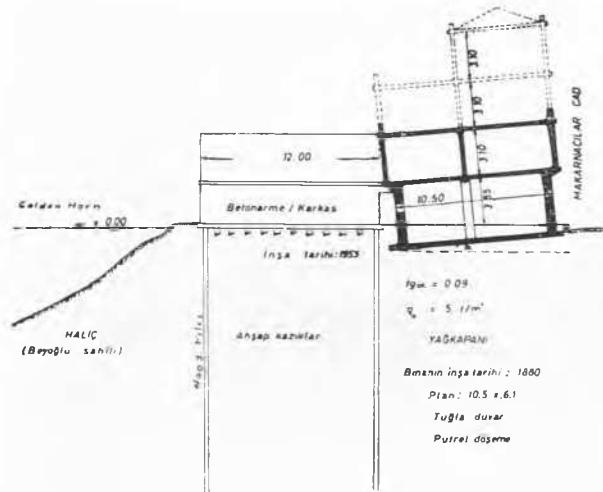


Fig. 4 : This is one of the many settled buildings near to the sea shore. Considering the lowest sea level this building should have been settled more than 0,5 meters. Every 20 years they have been obliged to fill the ground floor of the building and the surrounding area. Here the streets are also settling.



Fig. 4

Fig. 5 : This not a flood. This is also due to settlement, not due to consolidation settlement, but due to the movement of the earth beneath the buildings.



Fig. 5

Fig. 6 : This photograph shows two old buildings, built about 120 years ago and now they will be demolished. The gap between the two buildings is filled with brick. Notice the bricks. After filling the movement continued. The inhabitants say that approximately every twenty years 4 cm horizontal movement is observed.

Fig. 7 : These are window arches. As you see the windows are now under street level which is + 0,60 m.

Fig. 10 : This comparatively new 3 storey building also tilted. But not towards the sea — back from the sea. This shows that the tilt and settlement is due mostly to the sliding towards the sea.

Fig. 8. : On this slide you see a tilted building. Originally it was a three storey building. Due to excessive tilt the upper



Fig. 6



Fig. 7

two storeys had been taken down. Differential settlement is about 60 cm. Average settlement is nearly 200 cm. See Fig. 9. Street level is at the sea level.

Fig. 9. : This is a very old mosque. Notice the height of this column. This was originally proportional. But now it looks square. The settlement of this structure also was of the order of more than one metre.

Fig. 10 : The new and the older buildings. The sea is at the back of these buildings. The old building has tilted back-towards the sea. The tilt can easily be observed by comparing the old building with the new one.



Fig. 8



Fig. 9



Fig. 10



Fig. 11

Fig. 11 : These buildings show irregular settlements and horizontal movements. Each tilted in a different direction. All these buildings are about 130 years old.

Fig. 12 : This chimney table belongs to the electric power station of Istanbul. This chimney, as well as the other buildings, requires periodical adjustments.

Fig. 13 : This is a test loading arrangement in southern Anatolia on hard fissured clays. This loading test had a duration of 27 days.

Fig. 14 : This is a footing for the test loading shown in Fig. 15.

Fig. 15 : This is also a long term loading test on layers composed of small boulders and gravel with some clayey admixture. Test duration is 44 days.



Fig. 12

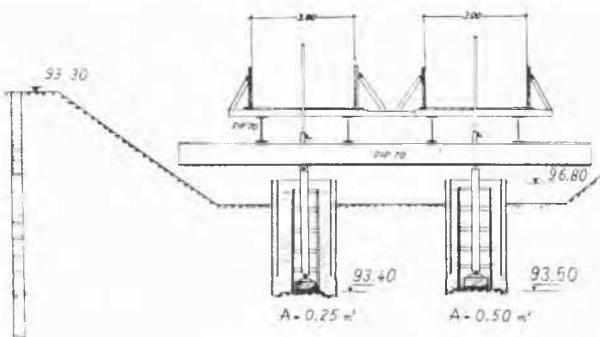


Fig. 13



Fig. 14

Le Président :

Je remercie M. Peynircioglu de son intéressante communication. Il nous a montré en effet des tassements exceptionnels et la grande difficulté qu'il y a pour les éviter dans la région d'Istamboul. D'ailleurs, plus que des tassements, ce sont de véritables affaissements et enfoncements de la construction dans le sol.

M. Tsitovitch désire intervenir dans la discussion.

M. N. TSITOVTCH

Prof. Peynircioglu rend les résultats de l'observation de la tassement des bâtiments dans leur région. Of great



Fig. 15

interest and value are the author's data on the magnitude of settlement of ancient buildings on the shores of the Golden Horn Bay, reaching 180 cm, which is a very good demonstration of the fact that entire attention was wrongly centred on the admissible pressure rather than on the possible cause of settlement and its non-conformity.

Le Président :

Je vous remercie, la parole est au Prof. Zeitlen.

M. J. G. ZEITLEN (Israël)

In respect to the influence of size and shape of foundation, I would particularly like to comment on the effect of size.

We should not hesitate to go properly into soil engineering problems simply because our present test results are erratic or are not obviously simple to correlate. After all, soil mechanics was difficult for many engineers to accept as an engineering science because they had personally experienced erratic behavior in foundations. Not knowing the influence of the various factors involved, they did not deem foundation problems would be ordinarily susceptible to mathematical analysis.

The effect of size is particularly of interest to those of us who are concerned directly with furnishing advice on foundations of engineering projects, and try to provide numerical estimates of settlements to be expected. We have available sound theoretical approaches, checked by field experience, for predicting settlements in layers of saturated clays, where such approaches as the elastic methods of stress analysis and Terzaghi's theory of consolidation provide useable tools. Ultimate bearing capacity may be predicted with reasonable safety, for many various types of soils, using the formulae

and criteria developed by a number of investigators. We are indeed fortunate that we have attained a stage where Dr Meyerhof can discuss the reasons for a 10 per cent difference between predicted and actual bearing capacity.

Although with present knowledge we may feel safe that we will not be likely to have actual ruptures or failures, we have much to learn about the stress-strain relations of footings on various soil types. The tendency for differential movement is always present, and we must strive to better predict actual settlements under various conditions of soil and flexibility of the superstructure. The structural designer must be provided guidance as to movements, and as to differential loadings possible if rigidity of the structure is maintained for minimum differential settlement. We must be able to provide a sound basis for integrating foundation with structural design.

It is admitted that, with the variations in natural soil conditions, settlement predictions may not be exact, but we must still be able to analyze the problem on a sound basis after making the necessary assumptions. The results of analysis should be at least correct in order of magnitude, and in any event should be presented with the necessary qualifications as to accuracy or the range of conditions within which the predictions may be useable.

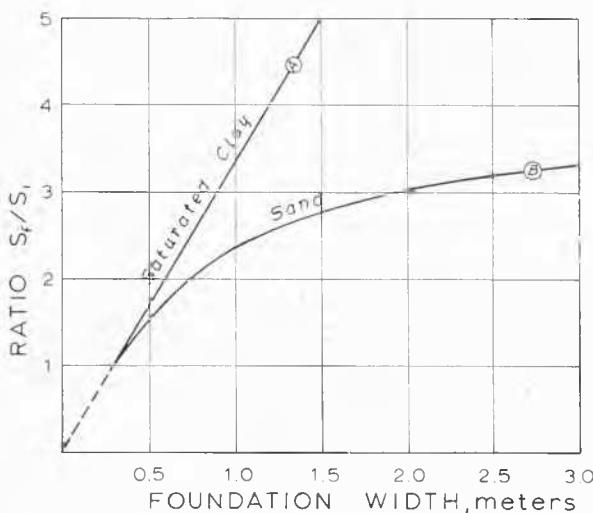


Fig. 16

Present design criteria for estimating the effect of size on settlement are often based on the functions shown graphically in Fig. 16. Settlement of the actual foundation, s_f , is shown by the ordinate as a ratio to the settlement of a unit foundation, s_1 (30 cm. square) which has presumably been tested on the foundation soil under similar conditions to those expected for the structure. The two curves shown for saturated clay and for sand may presumably be used for comparing anticipated settlements of various foundation sizes by comparison of their respective s_f/s_1 ratios.

For equal unit foundation pressures, the settlement of a plate on saturated clay is assumed to vary directly with the width of the plate, as in line A of Fig. 16 in accordance with elastic theory. This approach checks well with field experience, and it may be noted, for example, that Skempton correlated field performance of clay foundation soils to develop constant settlement factors as coefficients for plate width, indicating again a linear function.

Line B, for sands, is based on the work of Kögler, and it should be noted that this function also served as a basis for the design charts furnished by Terzaghi and Peck and

Peck, Hansen, and Thornburn, where the effect of foundation size is considered on allowable bearing value and/or settlements.

It is believed in order at this time, in view of the increasing importance of deformation predictions, to point out that :

(a) The work of Kögler should be further checked and expanded.

(b) Study need be made of the behavior of materials intermediate between sand and saturated clay, to develop the area between lines A and B.

(c) Bearing tests in the field could be used more extensively for prediction of the stress-deformation function of the structural foundations, and not merely for check of conformance to building codes, or for determining ultimate bearing capacity.

(d) Further theoretical, laboratory, and field studies are required to establish stress-strain relations for various plate sizes and soil conditions, and actual measurements of foundation stresses and movements for structures of various rigidities.

It is believed of interest to point out that in some cases stress-strain relations may be more consistent than the actual rupture values. A research just completed under my direction in Israel by Mr Y. Negri was aimed at studying the use of lateral loading tests in comparison with vertical loading tests on sand. The purpose of the study was to develop a more economical type of field test, since by running the horizontal test in a pit or trench it should be possible to save the expense of the relatively large reaction loads required by vertical loading tests.

It was found that for vertical loading tests in small scale laboratory tests, and also for field tests on 30 cm. square plates, sand in a medium-dense and dense state had a relatively constant value of subgrade reaction, compared with somewhat erratic values for rupture conditions. Loose sand was far less consistent, showing variations in stress-strain functions and in rupture values.

Under field conditions, when lateral loading tests were made with surcharge load acting on the stressed zone, the stress-strain functions found were also relatively consistent. It may be noted that field vertical loading tests were made for design investigations at the same site, and the sand showed comparable values of the coefficient of subgrade reaction.

Le Président :

Je vous remercie. La parole est au Prof. Meyerhof.

M. G. G. MEYERHOF

I should like to reply to the statement made by Mr Zeitlen about the reference to the 10 per cent variation of test results, which he attributed to me. This variation applies only if we carry out a number of repeat tests, and particularly on a fairly large scale. On large diameter plates in the field the scatter of results is usually smaller than in the laboratory or for small-scale tests because a larger plate size seems to average out the variability of the soil over a greater area. But even so I should like to stress that we have to carry out repeat tests. As we have seen in the curves shown by M. Habib, there is a fair amount of scatter when we deal with small-scale footings due to the variability of the soil itself.

Le Président :

Je remercie M. Meyerhof pour cette précision. Puisque personne ne demande plus la parole sur ce premier sujet de discussion, nous abordons le deuxième sujet relatif aux sols non saturés, aux argiles gonflantes, aux sols à structure; à l'étude des retraits et de leurs limites et des tassements.

Expansion et retrait des sols non saturés.

Je donne la parole à M. Aitchison qui a demandé à intervenir dans ce débat.

M. G. D. AITCHISON (Australie)

I wish to speak on the question of the engineering properties of unsaturated soils. My comments will refer initially to natural soils, although a discussion of this type could extend to cover compacted soils. In the sense in which I use the term at this moment, an unsaturated soil is one in which the pore water pressure is less than atmospheric pressure. This negative pore water pressure is the dominant feature influencing the whole of the pattern of behaviour of unsaturated soils.

Variations in the negative pore water pressure may take place as a result of applied stress, but more often the variation in the negative pore water pressure may be caused by external factors — rainfall, evaporation, evapo-transpiration, and so on — and all of these are in general not under the control of an engineer. Therefore we tend to talk of unsaturated soils as soils with peculiar properties, simply because these soils are not responding to obvious applied stress conditions.

It is apparent from a study of the physics of unsaturated soils that a condition of negative pore water pressure in the soil corresponds to a condition of available free energy for the uptake of water, and this in turn, of course, can cause changes of volume or changes of shear strength.

Now the magnitude of the possible range of negative pore water pressures in many natural soils is quite high : it often will exceed 10 kg./cm.² and in some cases, in arid climates, may exceed by a substantial amount 100 kg./cm.². The actual value of the negative pore water pressure, of course, in any soil is clearly related to climatic factors, and there now appears to be a first possibility that the actual value of the negative pore pressure may be predicted from climatic data. Papers presented to this Conference, and earlier in a South African Conference, have suggested some relationships between climate and the actual value of the negative pore water pressure.

In the light of the quite large magnitude of pore pressures which have been shown to exist in soils, the very large question which has faced those engineers concerned with unsaturated soils is this : Can we insert these values of negative pore water pressure into an effective stress equation in order that we can define soil behaviour in quantitative terms ? And if we can so insert this value of negative pore water pressure into an effective stress equation, what are the limits to the range of validity of such an effective stress law ?

This question has been with us for a very long time, of course, and there have been reactions, more or less intuitively, to this question for very many years. Terzaghi, of course, mentioned this point in his original text on soil mechanics, but a great deal of progress has been made since the last International Conference towards providing a quantitative answer to this question. Evidence has been presented at most of the regional conferences which have taken place since the Fourth International Conference — conferences in Australia, South Africa, London and Israel (to mention only some) have all devoted considerable attention to this question of an effective stress law in unsaturated soils, and it is possible to suggest that we are now quite close to an agreement on the form of this law.

Such an effective stress law is necessarily much more complicated than that which can be used in saturated soils, since we must deal with a 3-phase system rather than a 2-phase system. The law which has been proposed as a basis for general agreement is :

$$\sigma' = \sigma - u_a + \gamma (u_a - u_w)$$

where u_a is the pressure in the pore air
 u_w is the pressure in the pore water

and γ is an empirical factor with values between 0 and 1.

If the pore pressure u_a is zero as is the case normally in a natural soil, the effective stress law is very similar to the law which applies in saturated soils, recognising that of course pore water pressure now is negative and that we have introduced the empirical factor γ . The values of γ are predictable theoretically in some cases, in idealised soils, and values which can be determined experimentally in other cases.

This form of the effective stress law is now one on which at least there is a substantial measure of agreement.

The next problem is how engineers may give quantitative expression to the actual effective stresses which exist in the soils under consideration. To achieve this purpose it is necessary to know both the initial and the final condition of negative pore water pressure and to know the values of the effective stress parameters which operate throughout the range of applied stress and range of pore water stress. It is not suggested that at this stage all of these measurements and all of these predictions can be made, but it does seem that so much progress has been made in the past few years towards this ultimate aim that in the course of just a few years we should be able now to utilise an effective stress law of this general form. If this is so, we should be able, I believe, to express the whole pattern of behaviour of unsaturated soils in terms of an effective stress system, and so be as rational in dealing with unsaturated soils as we are when we deal with saturated soils.

M. J. E. JENNINGS (Afrique du Sud)

I would like to start my discussion by considering the problem of heave as we observe it in the field. We have a considerable number of records of movements of structures on these subsoils and we find that the distortions are greatest when the applied loads are smallest, i.e. structures, such as roads or single storey houses, are the most severely affected. Heavier and more rigid structures are also affected but to a smaller degree. Hence structural loads and stiffness are factors which must also be taken into account.

I agree with Dr Aitchison when he says that the problem can be handled in terms of changes in effective stresses, providing one recognizes that there will be limitation to this procedure if the soils exist in nature at a degree of saturation which is lower than a particular critical value which is a property of the soil in question.

Referring to Fig. 17, which is derived from a Fig. taken from Vol. III of the proceedings of the last conference, we can see the pattern of effective stress changes. These occur as a loaded desiccated soil becomes wetted under the influence of a surface covering, which has changed the evaporation or evapo-transpiration in the soil. To the right, or positive side of the vertical line, the overburden pressures and additional pressures due to structural loading have been plotted. This shows the effect of structural loading on the subsoil. The most interesting curves, however, are those on the left or negative side of the diagram. These show the negative pressures in the porewater, both initially and finally after heaving has occurred. We must remember that these dotted curves represent pressures in the porewater and that the pores also contain air which will generally exist as continuous voids, open to the atmosphere. Hence the porewater pressure no longer acts across the whole area of any plane through the soil. To use these water pressures in conjunction with the total pressure (σ) or the effective pressure (σ'), both of which are considered to act across the whole of any plane through the soil, we must somehow adjust the pressures in the porewater of the partially saturated soil to the same common

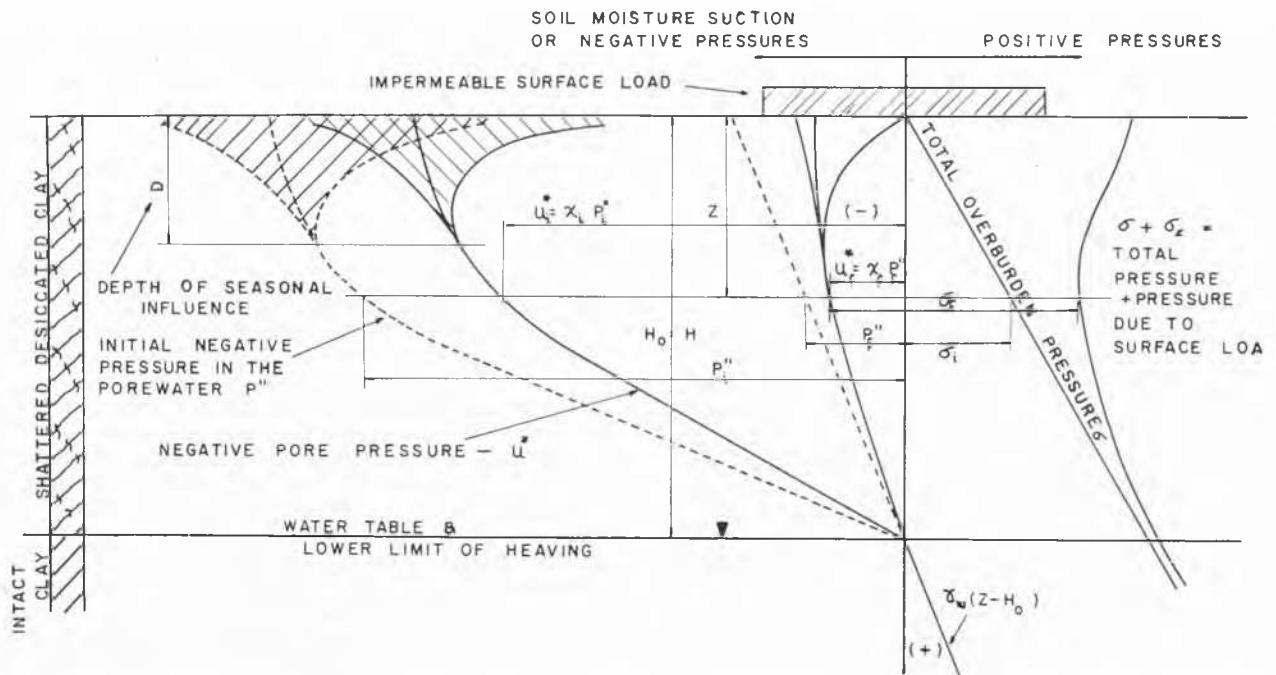


Fig. 17 Water Pressure and Stresses in Subsoil under a Building on an Expansive Clay Subsoil.

basis. We must think of the pore pressure as the contribution to effective stress made by the pressures of the fluids in the pores. This may be done quite easily by writing an equation :

$$\begin{aligned}\sigma' &= \sigma - u^* \\ &= \sigma - \{u_a + \chi(u_w - u_a)\}\end{aligned}$$

This is identical to the equation used by Dr Aitchison and the term χ , which replaces previous β by Jennings and Croney and ψ by Aitchison, is used now as a statistical parameter to determine the equivalent pore pressure, u^* , for a partly saturated soil. If u_a , the air pressure, is atmospheric and if all other pressures are referred to atmospheric pressure, the equation reduces to $\sigma' = \sigma + \chi p''$ where p'' is the negative pressure in the porewater with respect to atmospheric pressure, i.e. the soil suction. The u^* curves initially and finally, after heaving has taken place, are also shown on Fig. 17 and the reduction in effective pressure which gives rise to the heaving process can be clearly seen.

Fig. 17 has some other interesting features. Firstly, the zone of seasonal influence can be clearly seen. This seasonal movement is reflected as rise and fall of the natural unaltered ground surface during the annual rainfall cycle. It has been frequently observed and the depth of seasonal influence can be found by observing the movements of pegs buried at varying depths. But this depth of seasonal influence has little connection with the depth to which heaving will occur if the surface is covered by a structure. Heaving will often take place at considerably greater depths showing that a zone of permanent desiccation exists below the depth of seasonal movements.

The existence of this zone of permanent desiccation is connected with a continual small upward flow of water from the water table. This introduces some interesting thoughts : is the water table dropping continuously or is it being supplied laterally ? This problem may have considerable significance.

The processes shown on Fig. 17 suggest that we have two effects namely a general upward heaving movement plus a cyclic seasonal movement. These two effects are shown on Fig. 18 and the actual values of the general heave A and

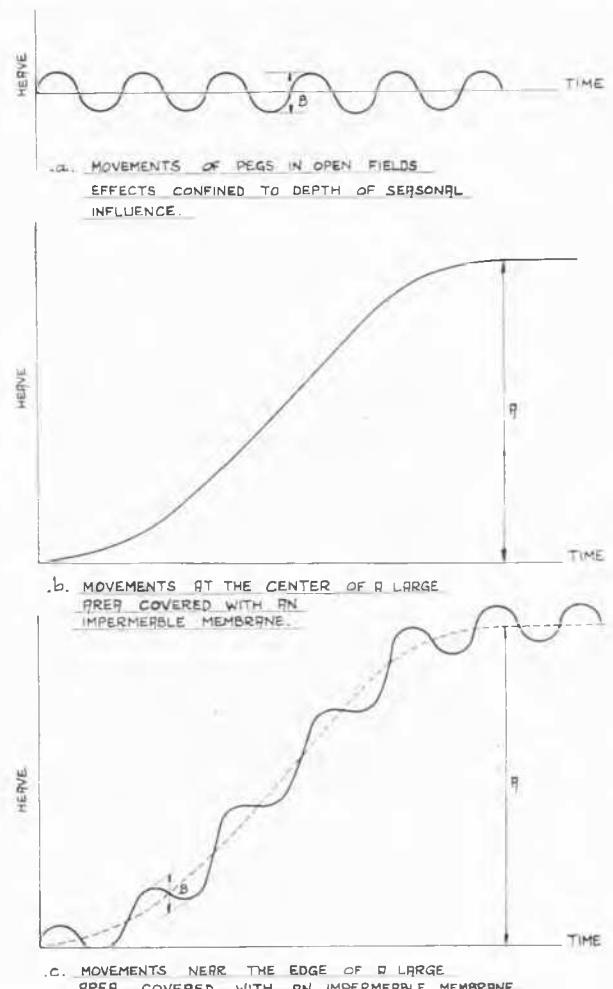


Fig. 18 Surface Movements on a Desiccated Subsoil.

the cyclic heave B will probably depend upon site conditions. Atmospheric climate is only one of these conditions and not the sole criterion as has been accepted by some writers. There may be other and very much more important factors, depending upon the local circumstances.

Fig. 19 shows an actual heaving record taken on a building at Odendaalsrus in the Orange Free State. All of the features of Fig. 18 will be observed. The general heaving of Fig. 19 has also been developed statistically for this area from the costs of maintaining buildings which have cracked due to foundation movements.

Predictions of total heave can now be made using the double oedometer test which is referred to in my paper in Vol. I of the Proceedings. This procedure has been checked very satisfactorily against records of actual observed heave. It also shows that heave decreases with depth and it permits the effects of surcharge loads to be calculated. It takes account of all the known factors in heaving except seasonal variations and effects of lateral pressure in the subsoil.

The double oedometer test is based essentially upon the effective stress principle which may be stated as two propositions : firstly, that the effective stress may be defined by the equation already given and, secondly, that positive changes in effective stress will be accompanied by decrease in void ratio and increase in shear strength and vice versa. In certain partially saturated soils, namely with the collapse of grain structure phenomenon, a decrease in effective stress is accompanied by a decrease in volume which contradicts the second proposition given above. There is some evidence now to point out that this can also occur even with clayey desiccated soil when the degree of saturation is below a certain critical value. This may be the reason for the one case of overprediction of heave but there may also be other aspects of the problem such as the influence of lateral pressures.

Le Président :

Je donne la parole à M. Zeitlen.

M. J. G. ZEITLEN

It was very worthwhile to note the emphasis which both Dr Aitchison and Prof. Jennings gave to environmental factors in their descriptions of the conditions obtaining during their studies of volume changes associated with soil moisture changes. However, recognition of these factors implies the need for caution in the application of methods which might work very well in one area to other areas, particularly in reference to South African experience.

The profiles existing in South Africa, where much of the work was done, include a sand cover which acts as an insulating layer and reduces the volume changes of the upper zone. From the suction curves which were shown by Prof. Jennings there is a feeding of ground water into the upper strata and a tendency for the seasonal moisture changes to be eclipsed under a covered area by the general increase of moisture content and reduction in suction forces.

In our experience in Israel we have had severe damage to buildings similar to that reported by Prof. Jennings. But the phenomena of general swelling has not yet been observed. As far as we know at the present time we are getting a general moisture increase only under the actual structure, such as a wide airfield pavement, which can be explained by the constant introduction of water into the side drains during the wet weather and its infiltration under the pavement itself, as well as by the inhibition of evaporation from the soil as provided by the impervious cover.

In addition I would like to point out that the analyses and explanations so far have been concerned with vertical movement, and we have experienced a great deal of trouble with

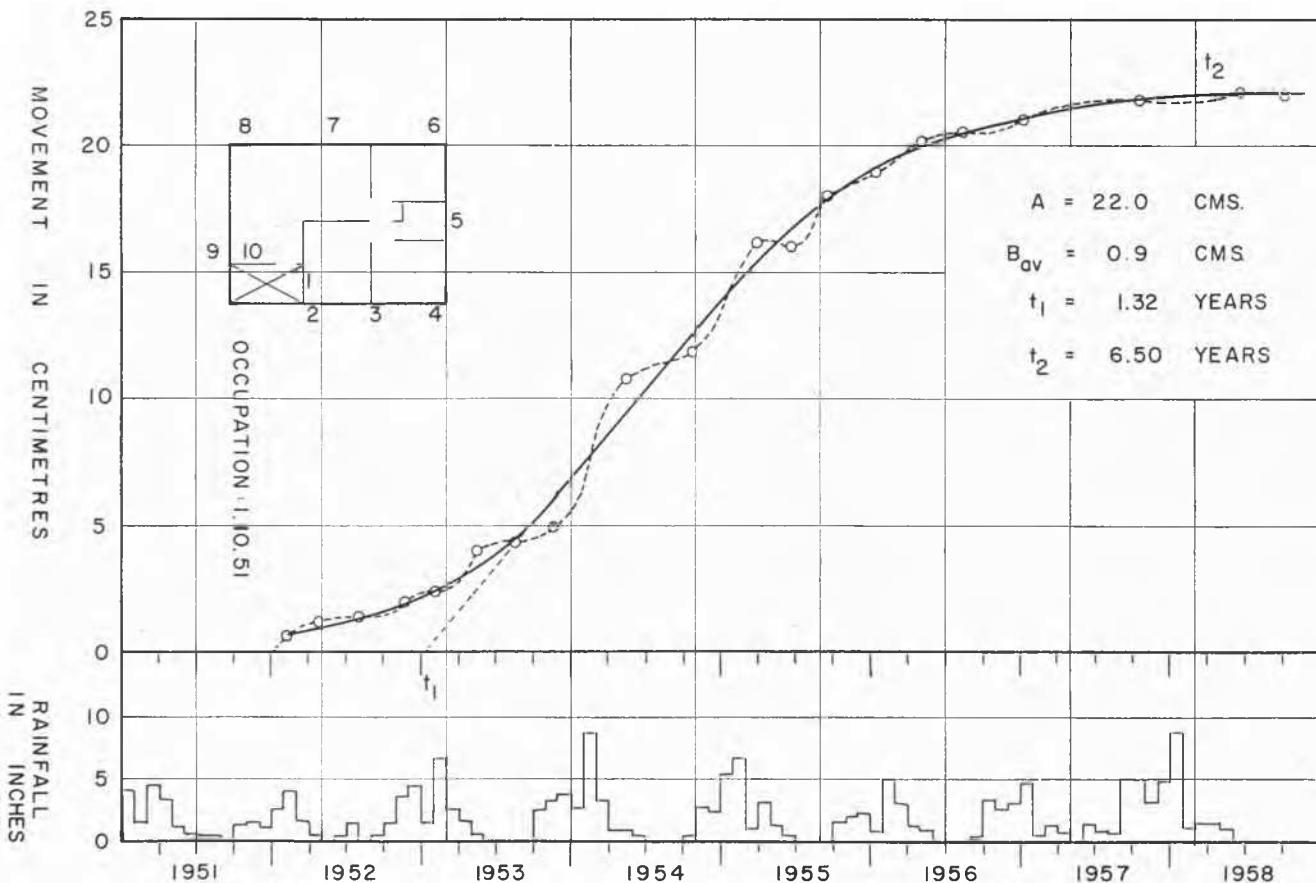


Fig. 19 Observed Heave of Single Storey Building in Odendaalsrus.

the lateral movement of the clay soils. That is due to a large extent because we usually have fat clays extending completely up to the ground surface. After a structure is erected, a general moisture increase would take place under the building, greater in the center, and tending to push to the outside the exterior piles or columns. Correspondingly, lateral movement is inward where a structure is built in a relatively dry season and then either rain occurs or the grounds around the structure are irrigated, to produce an expansion of clay around the building.

Hence we should not concentrate only on the vertical movement, forgetting the lateral forces which can be exerted by the clay.

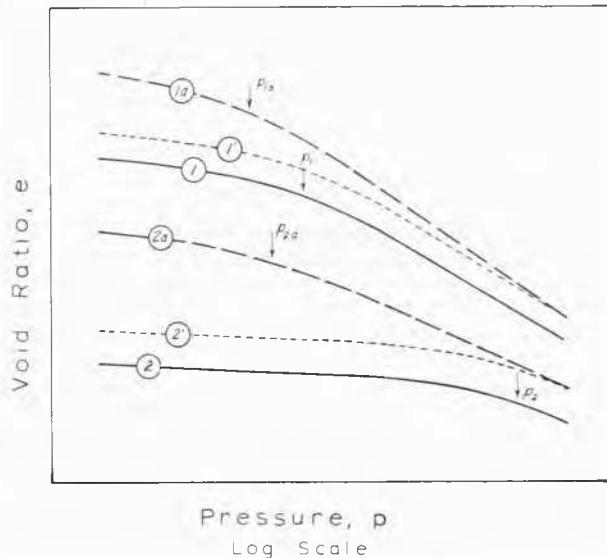


Fig. 20 Consolidation at Natural Moisture and after Wetting at Low Pressure.

Oedometer tests have been used by many investigators for predicting the vertical movement, and it is believed useful to discuss this test, particularly in reference to Prof. Jennings' comments. From the oedometer, or consolidation, test, we plot e vs $\log p$ functions, as in Fig. 20. For example, a soil tested at natural moisture may have a typical consolidation curve as in 1, which may be recognized as showing a particular pre-consolidation pressure p_1 . If allowed to saturate and swell under a light load, as in the double oedometer test method that Prof. Jennings explained, it will increase in volume and then when reconsolidated will have a curve such as 1a. For similar soil found at a higher initial density (that is, a lower initial) a higher preconsolidation pressure, p_2 , is found either because of surcharge effect or because of desiccation. Desiccation pressures, as we know, can reach very high values, so the a can be relatively very low. Here again, a swelling action occurs when the soil is saturated and a reconsolidation will give a curve such as 2a with a corresponding preconsolidation pressure, P_{2a} . Prof. Jennings proposes that, by transferring curves such as 1 and 2 into positions shown by 1' and 2', the anticipated swelling would be shown by the difference in the ordinates of 1' and 1a or 2' and 2a. The question is whether a sample which has become saturated at a light load and then reconsolidated under a greater pressure will arrive at the same void ratio as if the soil had been consolidated at its natural moisture, and had then been saturated under the greater pressure. Under a structure the soil is usually wetted, either under its overburden pressure, or if it lies directly under

a pile or footing it would be wetted under the stress corresponding to the bearing pressure which is exerted on it by the footing. Since swelling phenomena for soils are not fully reversible and stress history influences soil properties, there undoubtedly exists a difference in resulting void ratio, depending on the sequence of loading and saturation. If the double oedometer test is used without properly appreciating this fact, error may be brought into the application of laboratory test results to problems in the field. It will be observed that the transferred natural moisture content curve may cross the saturated consolidation curve, as in the curves which Prof. Jennings showed. It is believed that such behavior is explained by the effect of pre-consolidation pressure on the soil structure and hence on the shape of the curves. Soils at higher void ratios have relatively lower pre-consolidation pressures, which are less effected by the specimens being allowed to swell and then reconsolidated. However, in respect to the denser, more highly desiccated materials very high stresses have been acting on the soils, as indicated by the preconsolidation pressure on Curve 2, and when allowed to swell under light load its structure may be sufficiently disturbed so that it shows a lower pre-consolidation pressure, P_{2a} , and a curve of different shape is obtained.

If Prof. Jennings' oedometer curves were examined, it will be observed that entirely different pre-consolidation pressures are shown for what were originally similar samples, which indicates that something has happened to the structure of the soil because of the swelling.

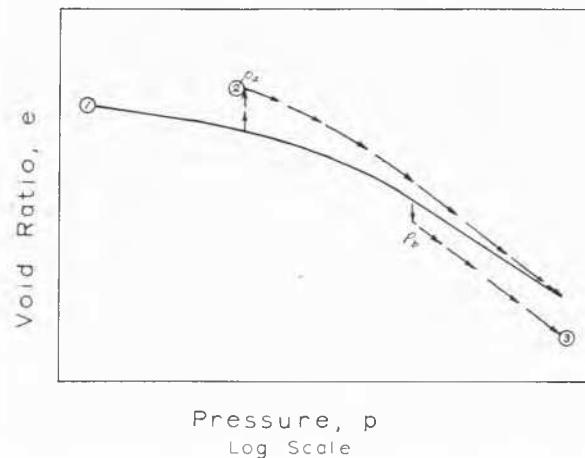


Fig. 21 Consolidation of Specimens Saturated at Various Pressures.

Hence it is considered more justified to compare samples of the same material which are wetted while under various pressures, as p_a and p_b in Fig. 21. In Fig. 21 a consolidation function is shown for a test at natural moisture, as indicated by curve 1. If it is wetted at a low pressure, p_a , it will swell and then consolidate as in curve 2. However, if it is saturated at a higher load, p_b , it will immediately settle, and then consolidate as in curve 3. Should this particular sample have been saturated under a lesser pressure than p_a , it will have higher void ratios than curve 2 until high pressures are attained.

By making a series of oedometer tests on samples saturated at different pressures with the same initial moisture and repeating for different initial moistures, it is possible to get a complete picture of the amount of swelling to be expected for any initial conditions of moisture and load. Consolidation data need not be obtained for all tests once the specimens have been saturated. Typical behavior is shown in Fig. 22, with a very high swelling at low pressure and when specimens

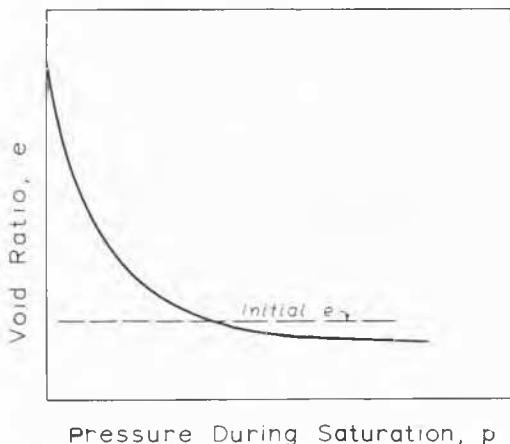


Fig. 22 Change in Void Ratio upon Saturating Similar Specimens at Various Pressures.

are tested at higher loads, the amount of swelling or change in void ratio becomes smaller and smaller, and finally consolidation is obtained at the highest loads. In practice, it may be necessary to obtain a function such as this only for a particular initial moisture corresponding to that of field conditions. In particular design problems it may be sufficient to test only under the overburden pressure and the footing pressure conditions to be expected in the field.

In connection with complete studies of swelling behavior, reference should be made to a paper in this conference by Dr G. Kassif and myself (5/11) on the pressures exerted by clay soil on conduits. A thorough study was made of a clay which was saturated under various pressures for various initial moistures. Referring to the plots of Fig. 2a and 2b, with the initial moisture as the abscissa it will be noted that ordinates are selected to show either the swelling pressure which would be exerted when various swells are allowed, or the amount of swell which would result from a particular swelling pressure.

It may also be observed that if swelling is not permitted, such plots indicate, for a particular initial moisture condition, the swelling pressure which is obtained. This type of study is particularly useful in problems involving a correlation between the structural design and the soil conditions, where it is necessary to predict the amount of deformation which might be obtained for particular pressures, or the reverse. It will then be possible either to design against differential movements through taking account of the rigidity of the structure, and making the structure sufficiently strong, or to plan for joints so that the structure may accommodate itself to the differential movements to be expected.

Le Président :

Je donne la parole à M. Jennings.

M. J. E. JENNINGS

It is quite clear from Prof. Zeitlen's remarks that we had a most stimulating meeting in Israel, just prior to this Conference.

I must emphasize the fact that if you propose measuring structural movements, it is very important that the bench marks be established below the zone of movement. This means placing the pegs in boreholes which may be up to 100 ft. deep.

Prof. Zeitlen has compared the swelling pressure tests conducted in Israel and the U.S.A. with the results of double

oedometer tests. Essentially both tests are the same but the double oedometer gives more information and is easier to carry out. The difficulty with the swelling pressure is that of getting water into the soil when it exists under load in the consolidometer. We have tried tests of this type and compared the predicted heave with that observed in the field. The predicted value was very much less than the observed field heave and I now blame this on the fact that we did not wet up our soil samples fully in the laboratory. I intend to repeat those tests and take much more trouble and time to secure neater entry into the soil.

We must of course appreciate that we are dealing with overconsolidated soils and settlement in such soils leads to difficulties, to say the least. It would be reasonable to expect some over-prediction of the heave and I feel that the fact we have come within 20 per cent with the double oedometer is very good indeed. It is true there has been one case of severe over-prediction but this was a minor case in which movements were not precisely observed. It will be investigated further.

M. L. F. COOLING (Grande-Bretagne)

It is evident that the movement of clay soils which accompanies changes in moisture content and its effect on structures built on such clays, is a world-wide problem and manifests itself in different ways according to the climatic conditions in different parts of the world. As I see it, the basic problem is as follows.

The moisture condition in the ground before construction starts, represents a dynamic balance between the water absorbed into the soil from rainfall and the water removed by evaporation and transpiration of plants and trees. Now this balance varies throughout the climatic season and hence, conditions at the time when construction is carried out can have an important influence on subsequent behaviour. When you put the structure on a site you make radical changes in the conditions. On the one hand, the vegetation is cleared away and this removes a potent source of depletion of soil moisture. On the other hand, the structure shelters the ground from direct rainfall and thus reduces ingress of moisture. The new moisture equilibrium established will be different from the original and either swelling or settlement may result depending on which factor has the predominant effect. Either of these two processes can occur — even in the same country.

In England where the clay soil is usually more or less saturated at the time of construction, most trouble is experienced due to shrinkage. The major problems are due to trees and shrubs which can deplete the soil moisture down to a depth of 8-10 ft. during the summer growing season. During dry summers, the extension of root systems is common and roots spreading horizontally under house foundations may cause local movements up to four inches. Recently, however, the Building Research Station has made observations on two structures which exhibited long-term heave under the foundations due to wetting-up of desiccated clays. Both of these sites has previously supported the growth of large forest trees.

One of these was a terrace block of cottages, single storey, brick-built of 11" cavity wall construction. It was built in 1952 and cracks had appeared after two years and had continued to increase in size over the subsequent four years. In 1958 we were asked to investigate what the architect thought was a stubborn case of soil shrinkage. Site borings and trial pits indicated that the soil was a stiff fissured clay (L.L. 75 per cent) with moisture content 26-31 per cent, and dead tree roots were found. A study of the site plan and aerial photographs taken before construction, showed that two large elm trees had existed on the site near one end of

the cottages. Reference points were therefore fixed in the building and observations of movement made with reference to two datum points 20 ft. deep about 50 ft. from the building. In the course of the year, the observations showed that points near where the trees had been, had risen 1 cm while points at the far end of the terrace remote from the trees, had scarcely moved. A survey made, assuming the damp course to be level when it was constructed, indicated that the movements might have been as much as 10 cm in seven years. The actual observations during the last years have shown a movement of the same order.

The second structure was a larger one, a three-storey office block, 140 ft. long, 37 ft. wide built on a shrinkable boulder clay. Several well-established forest trees of about 8 ft. girth were felled and removed in November 1958, the building was completed in July 1959 and levels had been taken since the commencement of construction. These have shown that while in other parts of the building slight settlement had occurred, in the region where the trees had been, there was a rise of about 1.25 cm; it is still rising and the movement is about 1 mm every two months.

These two examples indicate that where an area of clay has been dried out at a depth beneath the surface, the rewetting to come to a new equilibrium may take several years. This also fits in with other experiences where deliberate attempts have been made to wet up a volume of dried-out clay beneath a boiler furnace — the water seems to pond on the surface which becomes impermeable and penetration into the mass of clay takes a very long time.

Turning to the general problem I think the suggested approach outlined by Aitchison and Holmes in Division 4, paper 1, entitled 'Suction profiles in soils beneath covered and uncovered areas' is a useful one. They suggest that the vertical distribution of water in the soil should be expressed in terms of the 'suction profile'. By measuring the initial 'suction profile' before construction starts and by assessing the probable final 'suction profile' the soil at the site will attain during the subsequent life of the structure, the question of whether heave or settlement will occur can be resolved.

As I see it the initial 'suction profile' can be obtained by taking undisturbed samples from different depths in the ground and measuring the capillary pressure of each sample by one of the methods indicated by Prof. Skempton in paper 61 Vol. 1, p. 351 of the Proceedings of the Conference. By comparing the values so obtained with theoretical values derived from the position of the water table and assuming zero flux, it is possible to assess whether there exists in the ground an upward flow of water to supply evaporation at the surface or a downward flow due to drainage.

The probable effect of placing the structure over the site in producing heave or settlement can then be assessed qualitatively. To make a quantitative estimate the following approach may be made. The initial 'capillary pressure' profile represents the 'effective' stress in the ground before construction. The effective stress profile finally attained after construction may, as a first approximation, be obtained by adding to the theoretical value for zero flow, the effective stress due to the loads from the building. From the change in effective stress profile between the initial and final condition and a knowledge of the compression characteristics of the samples, a theoretical estimate of the quantitative movement of the ground can be made in the same way as in settlement analysis. This approach assumes that the water table would remain constant in position and an estimated correction would need to be made for the probable change in ground water level. However before this approach can be accepted there is need for much more evidence derived from practical examples where comparisons have been made between actual and predicted movements.

(Suspension de séance)

Le Président :

Nous allons reprendre nos travaux. Nous avons 13 orateurs inscrits ce qui signifie que le temps de parole doit être limité à 3 à 5 minutes. J'insiste donc pour qu'on soit aussi bref que possible si nous désirons terminer dans les délais qui nous sont impartis. Nous avons 7 orateurs inscrits sur le premier sujet et 6 pour le deuxième.

Je donne la parole à M. le Prof. de Beer.

M. DE BEER (Belgique)

Les essais expérimentaux montrent systématiquement que pour les sols pulvérulents, les forces portantes limites du sol sous des semelles superficielles sont supérieures aux valeurs calculées avec les formules de Prandtl et les formules dérivées en y introduisant les caractéristiques de cisaillement obtenues au moyen d'essais triaxiaux normaux ou cellulaires normaux.

Afin de pouvoir rechercher les causes de la différence, il faut prêter attention à trois éléments essentiels du problème :

1. La signification théorique des formules utilisées.
2. La signification physique des paramètres que l'on introduit dans ces formules.
3. La signification de la grandeur mesurée dans les essais.

Je ne puis évidemment pas épouser le sujet dans les quelques minutes qui me sont accordées :

1. En ce qui concerne la signification théorique des formules utilisées, il faut constater que les formules de Prandtl et les formules dérivées se rapportent à un milieu rigido-plastique, et qu'elles ont pour but de déterminer la charge sous laquelle l'écoulement plastique commence. Pour de nombreux sols il peut y avoir encore une augmentation considérable de la capacité portante en fonction des déformations.

2. En ce qui concerne les paramètres à introduire dans la formule, il faut tenir compte du fait :

(a) Que la résistance au cisaillement d'un sol pulvérulent augmente fortement avec sa compacité et en est une fonction très sensible.

La Fig. 23 donne, par exemple, la variation de l'angle de cisaillement en fonction du pourcentage des vides n , avec le rapport de la contrainte sphérique moyenne σ_m à un module d'élasticité E pris comme référence. Cette figure a été établie par le Dr Ladanyi pour le sable de Mol.

(b) La résistance au cisaillement d'un sol pulvérulent pour une même compacité diminue lorsque la contrainte sphérique moyenne σ_m augmente. Si la plupart des expérimentations tiennent compte de la variation de la résistance au cisaillement en fonction de la compacité, elles négligent par contre la variation avec σ_m . On peut évidemment se poser la question qu'elle est encore, eu égard à cette variation, la signification physique de l'angle de cisaillement ϕ .

(c) Si dans un exposé récent que j'ai fait en Allemagne, je n'ai pas fait mention de la variation possible de l'angle de frottement avec la contrainte principale moyenne, c'était parce que des essais qui ont été effectués il y a plusieurs années déjà à Delft au moyen de l'appareil cellulaire, avaient indiqué que l'influence de la contrainte principale moyenne dans les cas extrêmes était fort limitée.

C'est avec intérêt que nous avons pris connaissance des essais que Bishop vient d'effectuer dans un appareil à déformations planes et qui indiqueraient que pour des fortes compacités, on pourrait avoir des différences de l'ordre de 4° par rapport à la résistance au cisaillement mesurée dans l'appareil triaxial.

En ce qui concerne la déduction que l'on en fait pour l'influence de la contrainte moyenne, je voudrais cependant, avec Roscoe et Rowe, conseiller la prudence, car il faut,

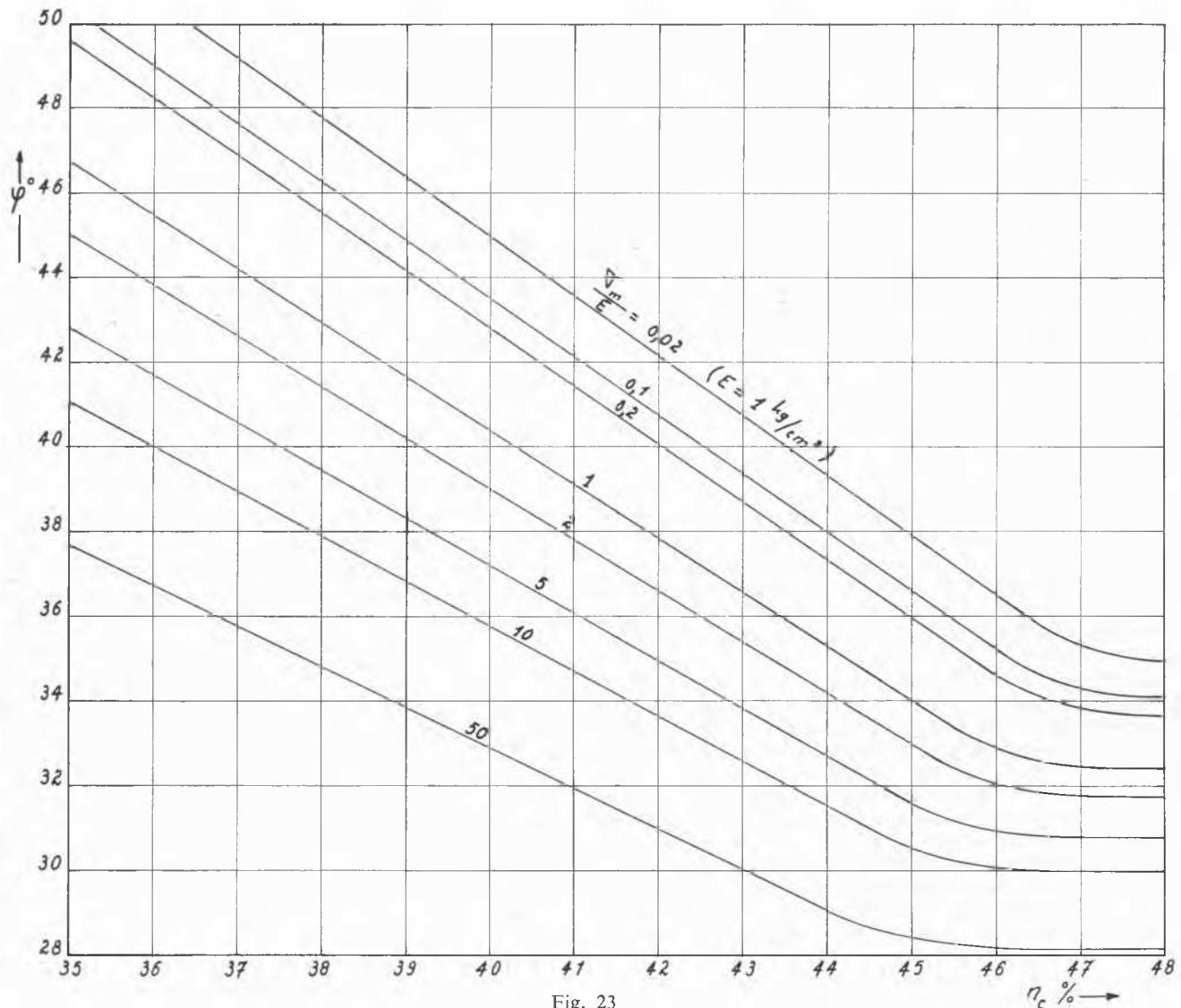


Fig. 23

pour arriver à des conclusions valables, pouvoir obtenir un état de déformation complètement homogène. C'est d'ailleurs la raison pour laquelle les essais de Delft n'ont jamais été publiés. Cela n'empêche que Bishop a trouvé que pour certains systèmes de cisaillement, la résistance au cisaillement peut être plus forte que celle enregistrée dans l'appareil triaxial.

Il restera à confronter les résultats de ses essais avec ceux d'autres chercheurs, avant de pouvoir en tirer des conclusions finales.

Pour le moment on ne peut donc inscrire l'influence de la contrainte principale moyenne et notamment l'état de déformation plane que comme une cause supplémentaire possible et probable de la divergence entre les essais et les formules.

3. En ce qui concerne la signification des grandeurs mesurées, il faut constater que, pour des sols à compacité faible et moyenne, les courbes enfoncements/charges ne présentent pratiquement plus d'asymptote, et qu'avant d'atteindre un état de rupture complet, on peut enfoncer la semelle à une profondeur dont l'ordre de grandeur atteint celui de la largeur de la semelle. Pour lever cette difficulté on peut passer à la définition d'une charge de rupture conventionnelle, fonction du rapport de l'enfoncement e à la largeur b de la semelle.

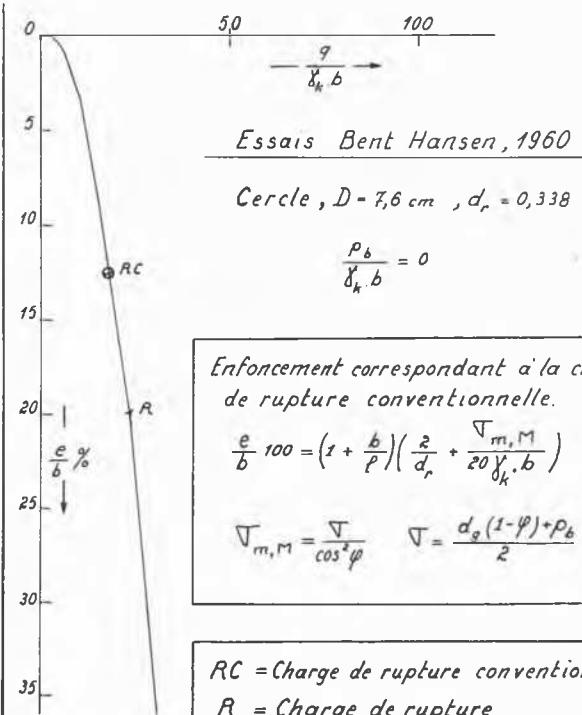
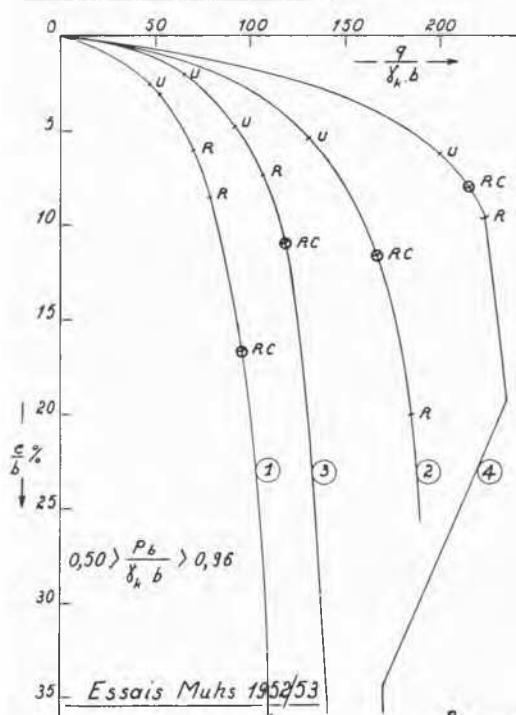
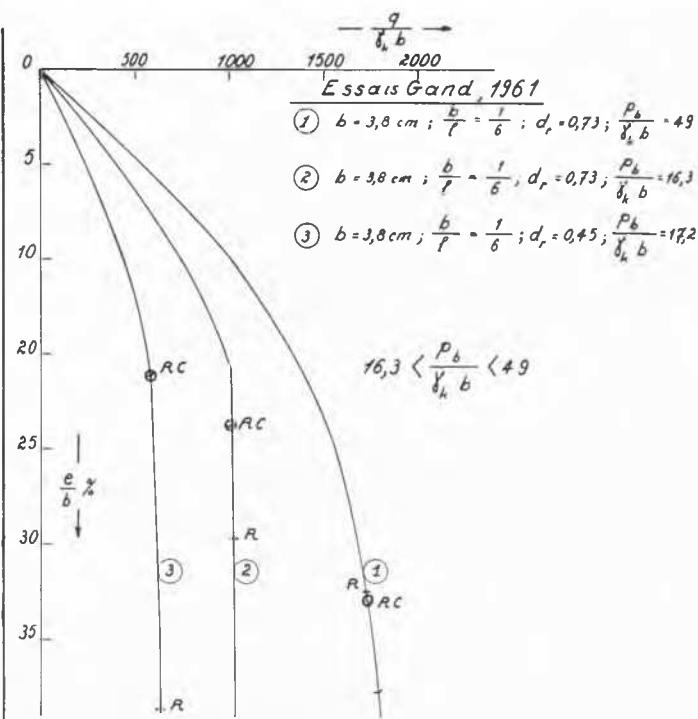
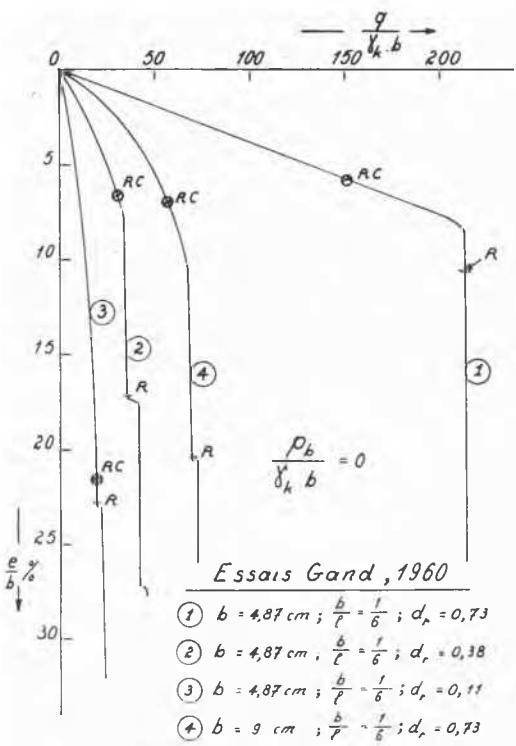
A la Fig. 24 sont données quelques courbes reliant les enfoncements aux charges appliquées, ainsi qu'une formule définissant le rapport $e : b$ correspondant à la charge de rupture conventionnelle.

On ne peut aussi perdre de vue l'influence de l'anisotropie de sollicitation qui peut exister dans le sol avant la mise en charge de la semelle; or, les formules basées sur l'hypothèse d'un milieu rigido-plastique ne peuvent tenir compte de l'influence de cette anisotropie. Celle-ci peut e.a. provenir d'un damage en couches successives ou de surcharges géologiques antérieures... etc., qui ont comme conséquence que les contraintes horizontales soient supérieures aux surcharges verticales latérales p_b .

En ce qui concerne l'influence de la dimension des fondations, la Fig. 24 indique que surtout au cas de faibles largeurs, pour une compacité et une forme de semelle données le

coefficient $V_g = \frac{N_g}{2} = \frac{d_g}{\gamma_k \cdot b}$ diminue lorsque la largeur augmente, conformément à l'influence de la contrainte sphérique moyenne sur la résistance au cisaillement.

M. Habib a dit qu'il ne faut plus traiter du problème du coefficient de forme. Il est possible que dans un avenir plus ou moins rapproché cela soit le cas, lorsque nous connaîtrons



RC = Charge de rupture conventionnelle
R = Charge de rupture
U = Charge d'inversion („Umkehrlast“
 d'après Muhs)

Fig. 24

mieux le problème à 3 dimensions. Mais entretemps il faut tout de même savoir comment doit se calculer la force portante de rupture des fondations autres que celles de longueur infinie. C'est pourquoi nous pensons que les essais en ce domaine gardent leur utilité.

Nous avons trouvé que le coefficient de forme pour le cercle au cas de fondations superficielles sur sable non damé $p_b : \gamma_k \cdot b = 0$, est pratiquement indépendant de la compacité et vaut environ 0,6, sauf pour les très faibles compacités où il se rapproche de l'unité.

Des essais sont actuellement en cours à l'Institut Géotechnique de l'Etat pour déterminer le coefficient de forme ζ_b par lequel il faut multiplier le terme en N_n , donc valable pour

les cas où $\frac{p_b}{\gamma_k \cdot b} = \infty$. Les premiers résultats trouvés par

le Dr Ladanyi semblent indiquer que pour des sables non damés en couches successives, le coefficient de forme ζ_b pour le cercle serait de l'ordre de 1,2 à 1,3 pour les fortes compacités, pour diminuer graduellement à 1 pour les moyennes et faibles compacités.

Par contre au cas de sables damés en couches successives avec de grandes valeurs du serrage latéral, les premiers essais effectués indiquent que le coefficient ζ_b pourrait en certains cas s'approcher de la valeur 2.

Nous voudrions simplement conclure pour le moment, que les coefficients de forme à introduire dans les formules ne dépendent pas uniquement de la compacité relative mais aussi de l'état de serrage latéral. Ce dernier effet pourrait expliquer les divergences qui sont trouvées pour les coefficients de forme par différents expérimentateurs.

M. A.R. JUMIKIS (Etats-Unis)

Please permit me to demonstrate by way of a short color film photographed at Rutgers University the process of the formation of rupture surfaces in soil obtained in my studies, and as presented in my paper 3 A/23, pp. 693-698, vol. I of the Proceedings of this Conference.

The film depicts the formation of three systems of rupture surfaces :

1. One-sided rupture surface brought about by a centrally applied inclined load.
2. Two-sided rupture surface brought about by a centrally applied vertical load, and
3. Rupture surfaces in dry sand formed underneath the edge of a vertically loaded caisson model.

1. One-Sided Rupture Surface.

The base area of the model is 15 cm \times 15 cm. First a vertical load is applied. At the end of vertical loading the settlement is fairly uniform. The spread of pressure may also be clearly seen. Then the horizontal load is gradually applied to the model (through the base of the model). Because of the horizontal load, the model translocates gradually in the direction of the action of the horizontal load. Upon further increment of the horizontal load the model tilts in a clockwise (in this case) rotation. As the vector of the resultant load on the model declines and becomes flatter and flatter, and when the shear strength of the soil is exceeded, a spirally shaped soil wedge, resembling a solid body, is sheared off and slid out along the rupture surface from underneath the model (Fig. 25). Please note in the film that no pronounced "wedge" underneath the base of the footing can be observed. Also it can be seen that the shearing off of the logarithmically spiralled sand wedge takes place suddenly.

The film shows where the rupture surface starts, how deep it reaches below the "ground surface", the nature of its

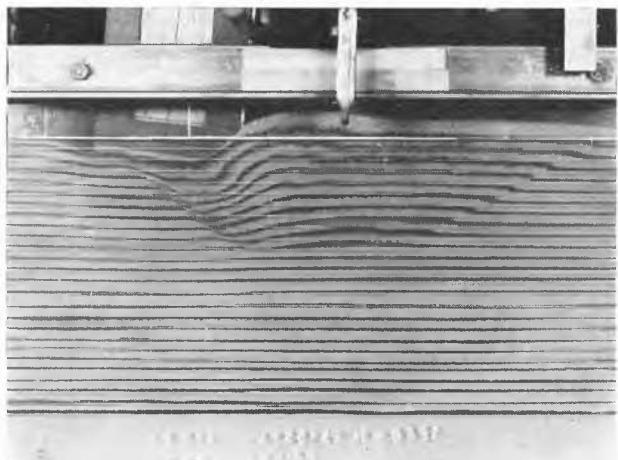


Fig. 25 One-sided rupture surface.
 $V = 281 \cdot 25 \text{ kg}$; $H = 92 \cdot 63 \text{ kg}$
 $b = 15 \cdot 0 \text{ cm}$; $h = 0 \text{ cm}$.

curvature, and what is its extent laterally. It may be noted that the formation of the rupture surface is a continuous process. Therefore it is not appropriate to describe the rupture curve by a mathematical equation which is a continuous one and is not composed of a compound curve, such as the sum of a circle plus spiral plus a tangent. Such a sum of geometrical curves and lines implies discontinuity at the points of contacts and tangencies of these different geometric elements, discontinuities which were not observed in the experiment.

The "creased" end of the logarithmic rupture spiral may partly be explained by the unconfined boundary condition at the ground surface.

The angle of internal friction of the sand used was $\phi = 35^\circ$.

At this point please permit me to express my appreciation to Dr Fröhlich, under whom these experiments were started many years ago at the Technical University in Vienna, and also to the Reviewing Committee of Papers who found these experiments to be of an original nature. The studies on this topic are now continued by the author at Rutgers. The State University, New Brunswick, New Jersey, U.S.A.

2. Two-Sided Rupture Surfaces

This is a 7.5 cm wide model loaded centrally and vertically on a sand whose angle of internal friction is $\phi = 35^\circ$. This experiment shows clearly the formation of a conspicuous

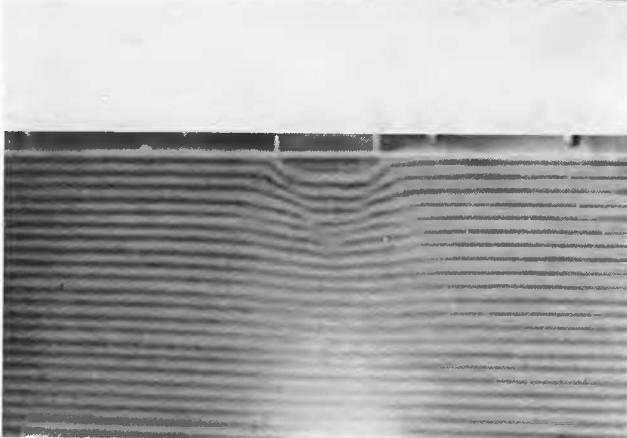


Fig. 26 Soil wedge.

triangular sand wedge directly underneath the base of the footing (Fig. 26). The downward-pointing wedge is first short in height, having an obtuse slope. As the vertical load increases the height of the wedge increases somewhat, resulting in an acute wedge. Note from the undisturbed black horizontal reference lines that the wedge acts as a solid body. After the shear strength of the soil is exceeded, the soil ruptures two-sidedly (Fig. 27). Because, during the rupture, the model continues to settle, the triangular wedge is destroyed. If the soil density is truly uniform, the two-sided rupture surfaces are symmetrical. In the case of a non-uniform density, the two-sided rupture surfaces are asymmetrical. Note again that the mass of the ruptured soil wedges appears virtually undisturbed. The shapes of the rupture surfaces appear to be particular logarithmic spirals.

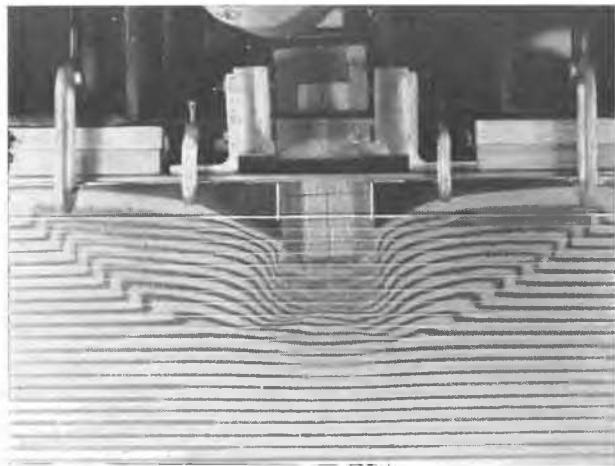


Fig. 27 Two-sided expulsion.
 $V = 240 \text{ kg}$; $H = 0 \text{ kg}$; $b = 7.5 \text{ cm}$.

3. Rupture Surfaces under a Caisson Model

The rupture surfaces underneath the knife-edges of a caisson model form jerk-wise and asymmetrically under the influence of a vertical, central load, thus demonstrating the effects of non-uniform density, some eccentricity and other factors, as they would actually be encountered in nature. The film shows clearly that for each set of load there occurs a corresponding deformation in the soil (Fig. 28).

The film was shown on Friday, July 21, 1961, at 15.00 hours in the main auditorium, UNESCO building.

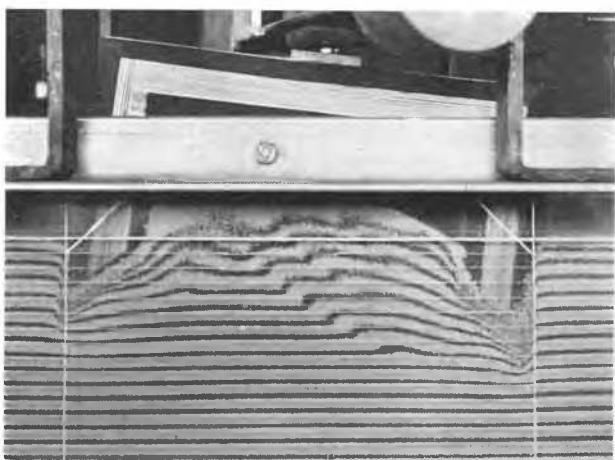


Fig. 28 Deformations in sand under knife edges of a model of caisson.

Le Président :

Je donne la parole à M. Kezdi.

M. A. KEZDI (Hongrie)

In his General Report Mr Tsitovitch pointed out that my paper giving some notes on bearing capacity in the case of inclined forces could well be rounded up by presenting some related test results. Since writing the paper which, then, has been stimulated actually by the results of field tests seeking to determine the safety against sliding of anchoring blocks of a chain bridge, we carried out extensive tests. I would like to show the principal results on a slide.

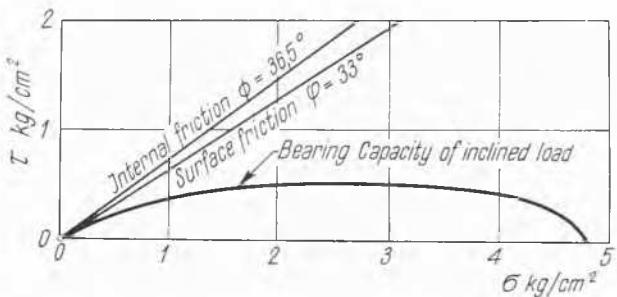


Fig. 29

The upper line in the figure shows the Coulomb-line giving the angle of internal friction of the material in question. It was a gravel sand. The second line has been furnished by surface friction tests. And now the relation between normal and tangential stresses in the state of limit equilibrium could be represented by the curved line. If horizontal forces are acting there occurs always a rupture and never a sliding. Therefore the safety cannot be calculated on the base of the angle of surface friction. The latter gives values always on the unsafe side. Taking a given value of τ and σ far enough from causing rupture, rupture can be introduced either by increasing or decreasing the value of σ .

Plotting the results as function of the inclination of the resultant force we get with very good approximation the same line as furnished by the theory presented in my paper.

I really hate to bring mathematics into an oral discussion just like this. I only mention that the theory presented in my paper has been developed and refined in the meantime and I believe that it gives now a sufficient explanation of the phenomena. It will be published shortly.

M. C. SZÉCHY (Hongrie)

In the panel discussion it was generally stated that experimental values of ultimate bearing capacity under any footing or strip foundation always exceed theoretical values. In reasoning this deviation I should like to call attention to the influence of contact-pressure distribution. As we know, at ultimate load this must be of a parabolic shape with a maximum value in the middle, whereas most theories assume a uniform distribution of the ultimate pressure. There is no doubt that the parabolic shape of pressure distribution will lead to a bigger ultimate load than the uniform one, and this may also be one reason for the discrepancies experienced.

In addition, this contact pressure distribution gives some evidence that, in contradiction to Mr Habib's opinion, there must be a favourable shape-factor for square and circular footings also in sands. In the case of strip foundations the average value of uniformly distributed contact pressure is $2/3$ of the actual maximum value of the middle ordinate of the parabolic

contact pressure. If we assume the same parabolic law of contact pressure distribution under a circular footing it will constitute a paraboloid. The height of the cylinder of equal volume will now be $1/2$ of the maximum middle ordinate. This difference in the ratios of the maximum and average values also demonstrates that a shape factor

$$\left(\frac{2}{3} : \frac{1}{2} = \frac{4}{3} \right)$$

must exist also in sands, if our generally accepted

concept (after Terzaghi, Schultze, etc.) — namely that the contact pressure under ultimate load will have some parabolic form — is correct.

M. D. KRSMANOVIC (Yugoslavie)

According to the proposal of the general reporter concerning the problems of pressure distribution, I wish to present some results of the examination dealing with the influence of rigidity on pressure distribution in multilayer soils.

We investigated the problem of structures resting on soil in an area where superficial terrain subsidences occur due to deep underground workings. Such subsidences occur gradually, since the sediments are clays, of tertiary age, more or less in a plastic state. There are alluvial sediments covering the former which consist of disintegrated clay and gravel.

Since the thickness of layers in the city differs in places, we have selected four typical cases for purposes of investigation. The variations in soil layers with the accepted average modulus of deformation and thicknesses are shown in Fig. 30 a, No. 1 to 4.

The foundation examinations covered some cases of settlements as well as those of irregular subsidence. A reinforced concrete structure was selected as a typical example, while the foundation structure was designed with a slab strengthened by beams in two prependicular directions (Fig. 30 b).

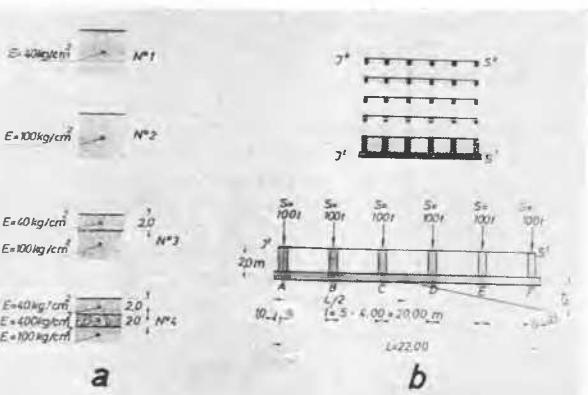
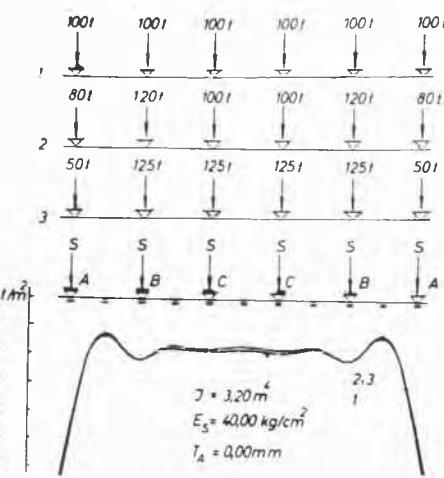


Fig. 30 Types of soil layers. Examined system of structure.

The foundation was examined under different cases of loading. Since the rigidity of the foundation compared with the one of the superstructure was very great, it was possible to neglect the influence of the rigidity of the superstructure. Further, we wanted to protect the superstructure construction from irregular settlements provoked by compressibility of soil or subsidence of terrain. The continuous system of the foundation was examined in detail in this way :

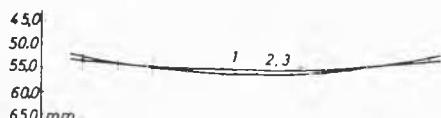
1. For different cases of loading with equal forces and also for other possible cases of construction loading. The examination was made under the assumption that there were no subsidences. The results obtained for the case of soil No. 1 are shown in Fig. 31.



$$J = 3.20 \text{ m}^4$$

$$E_S = 40.00 \text{ kg/cm}^2$$

$$T_A = 0.00 \text{ mm}$$



c

d

Fig. 31 Results without terrain subsidence; a) loading cases; b) pressure distribution lines; c) elastic lines; d) bending moments.

The values of bending moments shown in Fig. 31 d range within small limits; the influence of possible loading changes within the limits examined was not very high.

2. In the case of subsidences caused by the workings, it was assumed, in the first case, that the terrain under one half of the structure is *subsiding in a straight line* (Fig. 32 a). The results shown in Fig. 30 refer to cases without any subsidence ($T_A = 0.0$), or with the subsidence of 20 and 60 mm. In the figure the examination results are given for No. 1 and 2 types of soil.

The kinds of pressure distribution for different cases are shown in Fig. 32 a. It can be seen that the pressure distribution changes vary in large limits and that — if the value T_A increases — very great increases in pressure intensity occur under the middle of the foundation. In Fig. 32 c the values of bending moments are shown depending on the values of subsidence and of the modulus of deformation. It has been found that in the same subsidence conditions the values of bending moments depend very much on the modulus of deformation and that much higher bending moments occur in soils with less compressibility.

3. In Fig. 33 some results are shown when it was assumed that the soil was subsiding under one half of the foundation system and that the subsidence line was not straight, but had the form of an arch turned now upward and now downward by its concave side. In this case the pressure distributions were nonsymmetrical (Fig. 33 a). We can see that the form of curvature is a factor of no great importance.

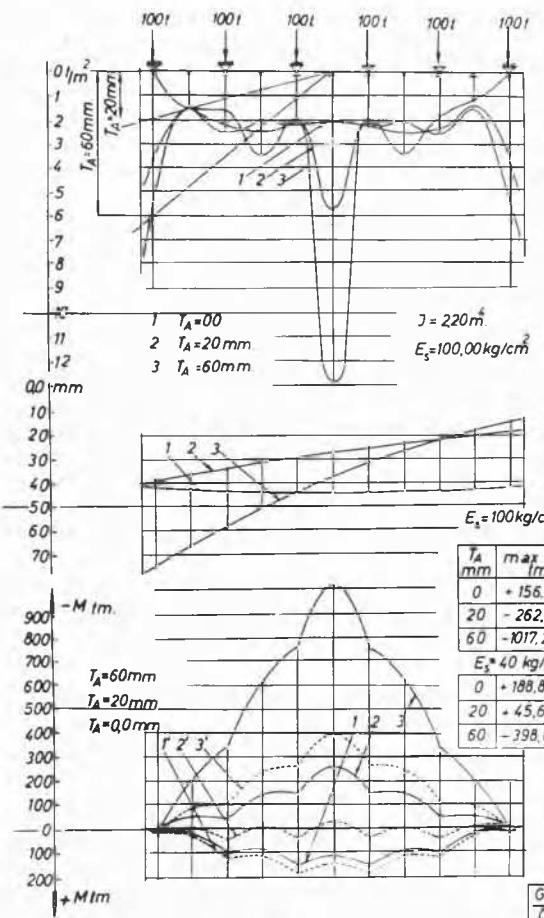


Fig. 32 Results with a straight subsidence line: a) pressure distribution lines; b) settlement lines; c) lines of bending moments.

The results of all the examinations concerning the bending moments that depend on the values of subsidence and relate to different kinds of soil structures (according to types of multilayer soils, Fig. 30, No. 1 to 4) are shown in a diagram, in Fig. 34.

It can be stated with regard to the foundations in question that the lesser the modulus of soil deformation the more favourable are the conditions for the foundation structure. In this case, however, the absolute settlement values of the whole structure will be higher.

The most unfavourable condition with regard to the bending moments arises when the structure rests directly on soil with the highest value of modulus of soil deformation (case No. 2). The values of the two remaining cases are between those two mentioned above. In this connection it can be said that the presence of a layer with the lowest value, considerably influences the decrease of bending moments (case No. 3, Fig. 30 a), while the presence of a layer with much higher modulus at a certain depth is of no particular significance.

In order to be able to compare the obtained values of bending moments in particular cases with an absolute maximum value, when the modulus of soil deformation tends to infinity and when subsidence occurs under one half of the length of the structure, that maximum possible value has also been computed. We see now that the values of bending moments obtained for the case $T_A = 60 \text{ mm}$ and $E_s = 100 \text{ kg/cm}^2$, are of a range of about 50 per cent of the values obtained when the modulus of deformation tends to infinity.

It would seem that, with regard to similar rigid structures with subsidence occurrences, it would be most advisable to insert under the foundation a layer of material with a low modulus of soil deformation in order to obtain the fewest possible changes in the lines of distribution. However, the layer should be thick enough, if we wish to obtain an economical foundation. The insertion of the above mentioned layer would result in a "less hard" resting of the structure in soil, and the settlement in this case would not be considerably greater.

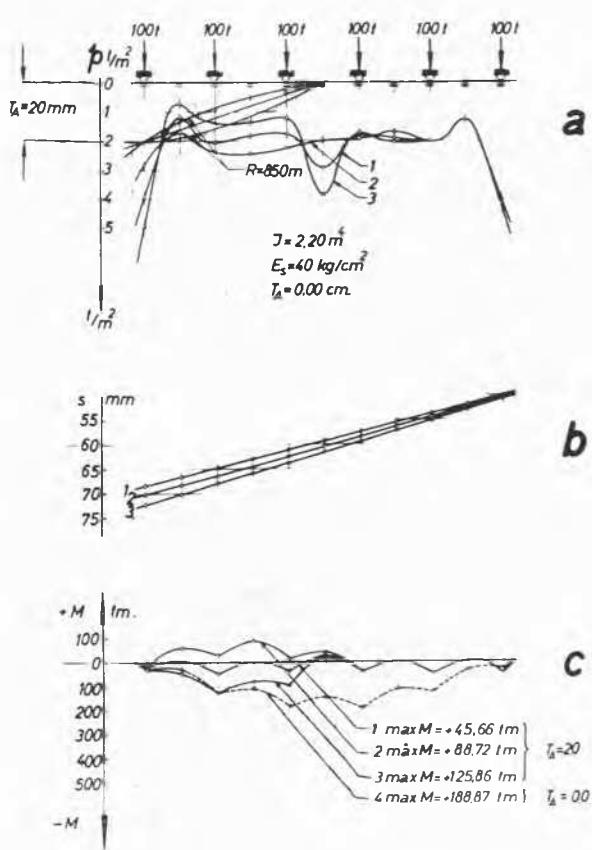


Fig. 33 Results with arched subsidence line: a) pressure distribution lines; b) settlement lines; c) bending moments.

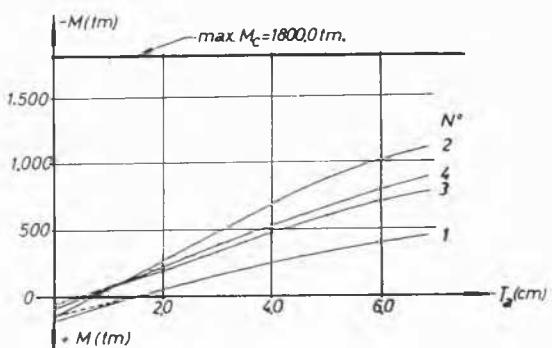


Fig. 34 Relation between values of bending moments and subsidences T_A (layers with different kinds of soil).

Ma communication concerne une fondation superficielle bien connue, peut-être la plus connue, car elle représente à la fois un exemple fameux d'insuccès et de succès de la technique. Il s'agit de la fondation de la Tour de Pise.

C'est bien une fondation superficielle, puisqu'elle fut placée à moins de deux mètres de profondeur, ce qui est très peu si l'on considère la hauteur de la Tour (60 mètres), sa largeur à la base (20 mètres) et surtout la pression moyenne, qui est d'environ 5 kg/cm².

Dans l'étude des conditions statiques de la Tour de Pise, l'évolution du sol sous-jacent a été jusqu'ici négligée; l'étude de cette évolution nous paraît au contraire essentielle pour éclairer les causes du mouvement, le degré de sûreté actuel, les moyens pour arrêter le mouvement.

La Tour de Pise a subi un effondrement général moyen d'environ deux mètres et une rotation correspondante au dénivellement actuel de 1,80 m entre les bords sud et nord de la base. La vitesse de la rotation et celle de l'effondrement moyen ont beaucoup diminué avec le temps, ainsi qu'on peut le constater dans la partie inférieure de la Fig. 35 déduite des inclinaisons actuelles des paliers (construits horizontaux) et des observations précédentes. Le bord septentrional est pratiquement stable; le mouvement de rotation, au contraire, n'est pas prêt de s'arrêter. L'inclinaison, environ 10 pour cent au total, s'est accrue de 1,5 pour mille au cours des cinquante dernières années [1].

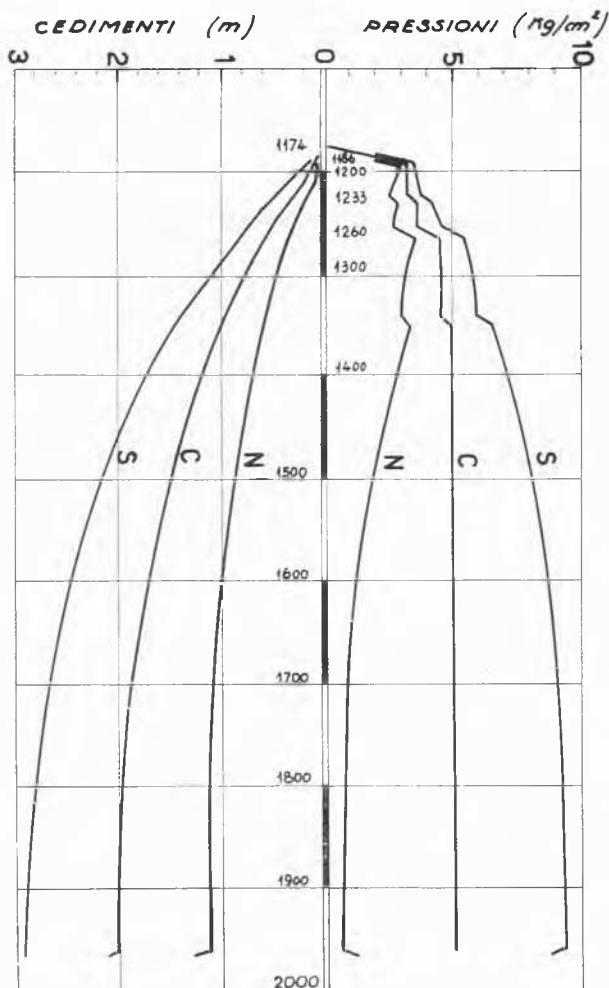


Fig. 35

Les caractéristiques du sol, la situation hydrogéologique, l'évolution des mouvements, imposent l'hypothèse que se soit vérifié dans le sol un tassemement du matériau fin saturé du type classique de Terzaghi ainsi que l'a signalé Terzaghi lui-même [2]; des phénomènes d'entraînement des grains de la part d'eaux courantes ne sont pas au nombre des causes essentielles du mouvement et de son manque d'uniformité, contrairement à ce que l'on a cru longtemps.

Quoique les effondrements irréversibles soient imposants, il n'y a pas eu de *résistance insuffisante* dans le sens technique du terme pour les sols, c'est-à-dire dans le sens de résistance insuffisante au glissement.

Nous avons essayé ailleurs [3] une vérification de la stabilité du sol; faute d'une connaissance positive, nous nous sommes référés à des résistances au cisaillement déduites d'hypothèses simplifiées, peut-être discutables, avec des résultats rassurants; à cet égard, la circonstance décisive est la haute rigidité conférée à la surface de la base par la structure massive et symétrique du bâtiment ainsi que sa grande hauteur; cela permet de considérer comme possibles seules les déformations du sol compatibles avec la conservation de la forme de la base.

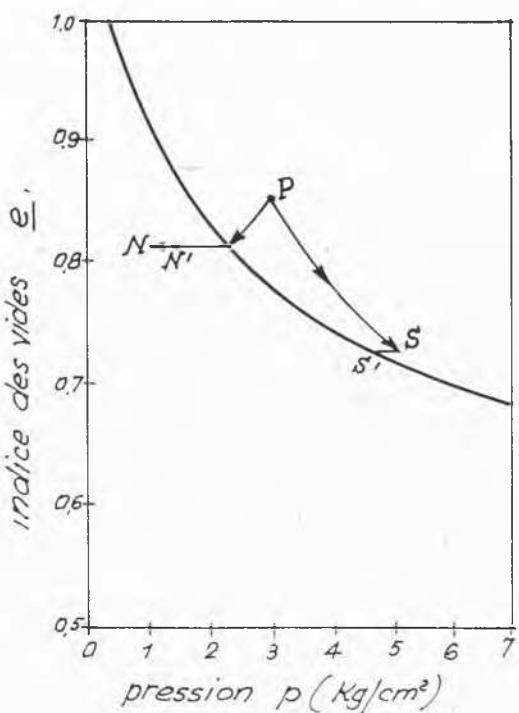
L'inclinaison de la Tour est d'ailleurs reliée à la dissymétrie de la situation des deux côtés du plan vertical Est Ouest; il convient cependant de considérer séparément deux circonstances :

(a) La cause originale, c'est-à-dire la compressibilité plus grande du sol, côté sud:

(b) La cause dérivée, c'est-à-dire la pression plus forte, toujours du même côté.

La deuxième cause s'est toujours accentuée et elle ne peut que s'accroître dans l'avenir; l'évolution de ce phénomène est montrée par celle des pressions aux bords Sud et Nord, calculées d'après l'hypothèse usuelle du diagramme linéaire; ces pressions, ainsi que leur valeur moyenne, sont représentées dans la partie supérieure de la Fig. 35; bien entendu, elles doivent être considérées comme de simples *indices* de la dissymétrie des pressions, car il est bien connu que sous les fondations rigides les pressions de contact sont bien loin d'être linéaires; à part une zone annulaire plus ou moins ample proche du contour, elles ont de préférence une marche croissante vers la périphérie. D'ailleurs, celles qui comptent dans notre cas seraient les pressions très variables dans tout l'espace intéressé. Le diagramme cité montre en tout cas la différence entre l'histoire du sol au Sud et au Nord. De ce diagramme et du fait que nous ne sommes pas loin de la consolidation complète, il ressort clairement que si, à l'époque de la construction, le sol était plus compressible au Sud, la situation est inversée aujourd'hui. La Tour continue à s'abaisser du côté Sud, quoique le sol soit moins compressible parce que, de ce côté la consolidation n'est pas encore atteinte pour les pressions précédentes et parce que les pressions croissent toujours.

La situation est représentée schématiquement dans la Fig. 36 qui met en évidence la seule cause b., c'est-à-dire que l'on suppose le sol tout à fait homogène. Dans le plan *p, e* (pression, indice des vides), *m* est la ligne du sol consolidé, ainsi qu'on peut la déduire des résultats des essais oedométriques exécutés sur des matériaux remaniés à partir de la limite de liquidité; on peut alors dire, comme première approximation, que les points au-dessous de la ligne représentent des positions d'équilibre, les points au-dessus de la ligne des états non consolidés. Etant donné la marche différente des pressions superficielles et des pressions profondes des deux côtés de la Tour, l'évolution de l'état du sol après la construction de la Tour pourrait être représentée (à titre indicatif, bien entendu) par le trait *PN* pour un élément typique de la zone Nord, par le trait *PS* pour un élément placé symétriquement dans la zone Sud.



Tassement typique du sol soujacent à la Tour de Pise du côté N et du côté S

Fig. 36

On peut alors concevoir que le bord Sud arrêterait son mouvement si la pression de l'élément considéré était par exemple réduite dans la mesure indiquée par SS' ; dans ce cas, en effet, elle deviendrait inférieure à la pression d'équilibre correspondant à la densité actuelle.

Une réduction des pressions au côté Sud peut être obtenue en réduisant l'inclinaison de la Tour; il est très probable qu'une réduction de 1 pour cent de l'inclinaison (actuellement 10 pour cent) en reportant les valeurs des pressions au niveau de celles d'il y a deux cents ou trois cents ans, serait suffisante pour arrêter le mouvement de la Tour.

Le redressement peut être obtenu de la façon la plus simple et la plus sûre, en enlevant du matériau du côté Nord au moyen de sondages successifs disposés, par exemple, comme il est indiqué à la Fig. 37.

L'opération devrait être précédée de nombreuses vérifications de l'état du sol et de ses résistances; elle pourrait être d'abord tentée sous des constructions sans valeur historique; en tout cas, en réglant le nombre, la position, le diamètre et surtout la vitesse d'extraction, des sondes, on peut satisfaire toutes les exigences de prudence que le respect envers ce Monument exceptionnel impose.

Une proposition dans ce sens est actuellement examinée par le Ministère des Travaux Publics italien.

Références

- [1] SANPAOLESI (1959). "Il Campanile di Pisa", Pisa.
- [2] TERZAGHI, K. (1934). ".Die Ursachen der Schiefstellung des Turmes von Pisa". *Bauingenieur*.
- [3] TERRACINA, F. (1960). "Sui problemi di meccanica delle terre relativi alla Torre di Pisa, etc.". *Rendiconti Accademia Nazionale dei Lincei*, Roma.

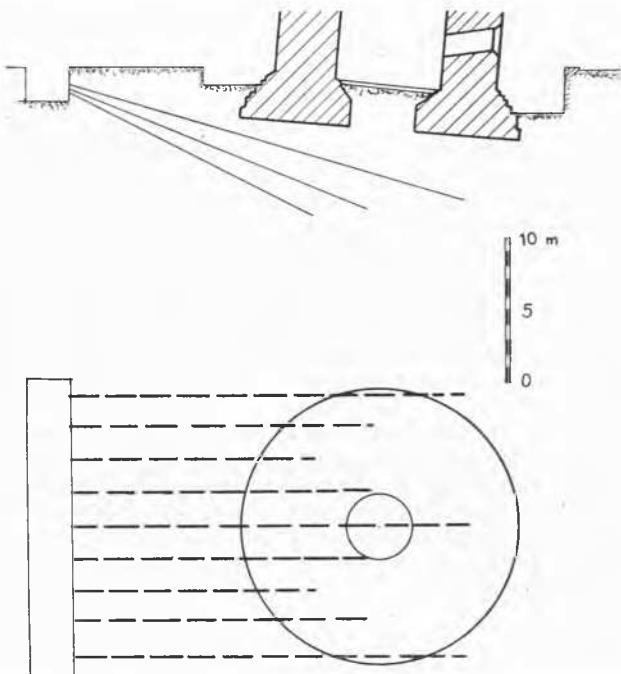


Fig. 37

M. H. NEUBER (Allemagne)

I would like to make some very brief remarks concerning the prediction of settlements of shallow footings due to slow statical loading. The main topics against conventional settlement analysis are inhomogeneity and anisotropy of the soils, difficulties in the testing procedure, the over-emphasizing of the influence of normal stresses, and the assumption of fully saturated soils, the strain-time relationship of which is said to be governed by the flow of pore water only.

In my opinion soils are separated from fluids by the possibility that shear stresses can exist, and I would like to propose that the shear stresses are as essential in deformation problems as in stability problems, both being continuously connected. As a matter of fact, the stress systems calculated from elastic theory in order to be used for settlement analysis could not be borne by the soil in many cases because of its limited shear strength. This difficulty is commonly overcome in the usual calculations by only taking notice of the normal stresses.

Furthermore, pore water pressures do not play an important role in settlement problems with a loading time of several months in most cases, sparing out very short construction periods and extremely soft young deposits.

Bearing these aspects in mind one comes to the conclusion that the original one-dimensional consolidation theory and the use of oedometers coupled to it are to be restricted to cases in which the underlying rather special assumptions are at least approximately true, namely, very soft widespread deposits loaded uniformly over a large area.

Furthermore, due to the above statements, one is to restrict the applicability of three-dimensional consolidation theory to a few cases with extremely soft deposits, too.

Concerning the collection of settlement records on the other hand one would doubt at first glance whether such an accumulation of data can be taken as a self-contained method for predicting settlements, but in fact it can. Some years ago I had been to find out when I was confronted with a number of case records collected mostly in Germany. In trying to extract something useful from a large heap of otherwise nearly waste paper, the following procedure was used :

As a first step, the complex relation between the time dependencies of load and settlement, was expressed by three parameters. To define these the most simple rheological model has been used, namely, a Hooke-Body (Spring) and a Kelvin-Body (Spring with dashpot across it) in series. The second step was to compare these parameters calculated for the different cases, and fortunately it turned out that there are quite simple and useful correlations. The parameters are not soil constants by definition : they depend on the size of the foundation and therefore cannot be determined in the laboratory. — Fort he stiffness modulus for final settlement in the middle of a foundation a graph elapsed from the field data which enables a prediction for new cases within reasonable limits if only rough information is given on the geological age and type of material as well as the size of the contact area.

For two further parameters defining the relationship between settlement and time which depends on the rate of loading, rather narrow correlations of the above-mentioned modulus for final settlement were found. Using these correlations, settlements can be predicted in a most direct way. Details are laid down in a paper which is at present coming out of the press. [1].

Référence

- [1] NEUBER, H. (1961). Setzungen von Bauwerken und ihre Vorhersage". *Berichte aus der Bauforschung*, H. 19, Berlin.

M. V. MENCL (Tchécoslovaquie)

Please allow me to present a very simple explanation of the phenomenon which has been shown by Dr Kezdi, who has given the correlation of the T and N forces on a foundation concrete block at failure.

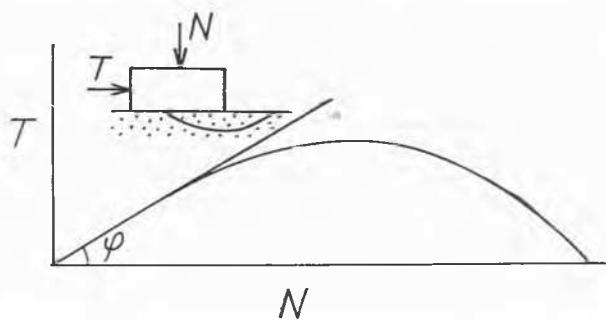


Fig. 38

If the block is strained by an oblique force and if the components of this force are N and T Dr Kezdi got such a diagram for the correlation of the N and T forces, but the explanation of this diagram is a very simple one : if a concrete block is strained by an oblique force, and if the normal component of this force is great, then failure does not occur at the base of the block but inside the soil, and the analysis must be carried out along this surface of failure, using the ϕ circle method, and not along the base of the block.

Le Président :

Je remercie M. Mencl pour sa remarque. Nous en avons fini avec les orateurs qui se sont inscrits pour la discussion de la première question; j'aborde donc la deuxième question en donnant la parole à M. Nascimento qui va nous entretenir des argiles gonflantes.

M. NASCIMENTO (Portugal)

Monsieur le Président, Messieurs, au sujet des sols saturés de surface ou non, je veux dire quelques mots sur les recherches portugaises.

Le problème se présente à propos des sols argileux expansifs dans la région de Lisbonne, surtout dans les argiles basaltiques et les argiles oligocéniques; dans la région de Luanda, Angola et en Mozambique, dans les argiles alluvionnaires des estuaires des grands fleuves comme l'Incomati, le Zambezé, etc., mais il se présente aussi dans les sols sableux de formation récente des régions arides, surtout en Angola. Le phénomène se fait sentir dans les petites maisons, par des fissures avec des mouvements saisonniers, ainsi que dans les chaussées des routes et dans l'équilibre des talus.

L'étude du problème a commencé au Portugal en 1948, par la mesure systématique des déformations et des prélèvements d'échantillons pour avoir l'allure du phénomène. Ensuite, nous avons fait des études théoriques et expérimentales afin d'essayer d'en établir une théorie.

Au Congrès de Zurich, nous avons présenté une communication, dans laquelle nous proposions la généralisation de l'équation de Terzaghi : $\sigma = \bar{\sigma} + u$.

En additionnant la valeur de la succion ω , $\sigma = \bar{\sigma} + u + \omega$; si, dans les couches de surface nous avons la tension neutre et la tension totale presque nulles nous obtenons $\bar{\sigma} = -\omega$. Les phénomènes des variations de volume sont dus à la succion, et la succion fonctionne presque comme une tension effective.

Afin de vérifier la validité de cette généralisation nous avons fait des essais de consolidation pour une dizaine de sols de Lisbonne.

Nous avons tracé le diagramme de consolidation-tension dans les mêmes sols et dans les mêmes conditions, et nous avons tracé aussi les relations succion-variation hygrométrique par séchage et mouillage. Nous avons vérifié que l'allure des courbes des deux diagrammes était semblable. Ces résultats ont donné lieu à une publication du laboratoire de Lisbonne en 1954 et à une communication au Congrès de Londres en 1957. Cependant, il faut avoir présent à l'esprit que la consolidation provoquée dans les charges par les succions, est une consolidation tridimensionnelle, alors que la consolidation des charges est une consolidation unidimensionnelle.

Sur l'influence des variations volume sur les talus, nous avons établi une petite théorie qui explique les mouvements descendant des couches de surface, que nous avons vérifiée expérimentalement au moyen d'un petit bloc d'argile dans un plan incliné glissant.

Nous faisons maintenant la mesure systématique des variations d'humidité en profondeur et pendant diverses saisons. Pour cela le laboratoire de Lisbonne a mis au point deux méthodes : une méthode hygrométrique qui a été l'objet d'une communication au Congrès de Londres de M. Roche, de Mme de Castro et de moi-même. La deuxième méthode utilise une cellule électrique à quatre électrodes. Elle a été l'objet d'une publication de M. Ressurreição Neto dans la revue *Técnica* de Lisbonne.

Nous faisons aussi des mesures systématiques *in situ* de gonflements; et des recherches de laboratoire pour mettre au point des essais simples de caractérisation des sols expansifs. Pour cela, nous avons développé un essai d'expansibilité sur les sols qui passent dans les tamis n° 40 comme les essais d'Atterberg.

Les résultats déjà obtenus ont été publiés par les Laboratoires du Génie civil à Lisbonne, à Lourenço-Marques et Luanda en 1959. Les sols non expansifs ont des expansibilités inférieures à 5 pour cent; les sols normaux ont des expansibilités de 5 à 10 pour cent et les sols expansifs ont des expansibilités supérieures à 10 pour cent,

Un autre essai a été fait sur les mesures des pressions d'expansibilité. Mme Dalva Nou du Brésil, a réalisé au laboratoire de Lisbonne des essais dans lesquels sont intervenues des pressions supérieures à 1 kg/cm².

M. SALAS (Espagne)

Je veux faire quelques observations rapides, parce que le temps est très limité, sur les constructions en Espagne sur les argiles gonflantes. Ces argiles sont très répandues en Espagne, mais avec des caractéristiques très différentes. Par exemple, dans le sud, en Andalousie, nous trouvons des terres noires identiques aux « tirs » marocaines et, je suppose, aux black-cottons de l'Amérique et des Indes. Dans le centre de l'Espagne, vers Madrid, par exemple, nous trouvons de l'argile grise, très rigide, avec une dessiccation qui ne dépend pas seulement du climat mais aussi des grandes pressions géologiques qu'elles ont subies.

Le comportement de ces deux types de terrain est absolument différent; le premier gonfle avec régularité et permet quelques prédictions. Le second a seulement quelques minces couches gonflantes de nature bentonitique ou d'atapulgite qui produisent des gonflements différentiaux très importants et très difficiles à prévoir.

Une deuxième remarque tient aux déplacements latéraux, semblables à celui qu'a remarqué M. Zeitlen. En Espagne les mouvements latéraux du sol sont très importants et ont produit des dégâts même dans des bâtiments de 6 étages avec des fondations relativement profondes.

Troisième remarque : dans le sud de l'Espagne où les mouvements sont plus systématiques, il semble que le mouvement est à peu près sinusoïdal, c'est-à-dire reversible. Il ne semble pas qu'il y ait un mouvement de gonflement progressif général.

Passons maintenant au sujet des essais. Nous travaillons beaucoup avec des courbes de gonflement. Nous préparons plusieurs consolidomètres et nous les soumettons à différentes pressions. Ensuite, nous mettons l'échantillon en contact avec l'eau et nous mesurons le gonflement. Il est nécessaire de prendre beaucoup de précautions pour assurer une saturation acceptable — que l'on mesure *a posteriori*. Je n'ai pas le temps de détailler ces mesures mais nous publierons prochainement un article sur cette question. Nous insistons sur l'utilisation d'un échantillon frais pour la détermination de chaque point de la courbe de gonflement. La structure d'une argile gonflante est si délicate qu'après chaque expérience elle se transforme d'une manière définitive. Nous avons commencé, naturellement, en déterminant la courbe complète de gonflement avec un seul échantillon, mais nous avons constaté qu'il y a une grande différence avec notre méthode actuelle.

Je veux indiquer que nous travaillons aussi avec des courbes de rétraction. Nous prenons divers échantillons et en les plaçant dans des récipients avec des tensions de vapeur contrôlées, nous mesurons le changement en humidité et en volume. Les résultats pratiques de ce procédé peuvent être considérés comme encourageants. Nous continuons quelques recherches dans cette direction, mais nous ne faisons presque pas d'application pratique, pour le moment.

Finalement, je veux dire quelques mots sur la fondation, but unique de toutes ces recherches.

Nous n'avions jusqu'à ce jour que des procédés très grossiers et je n'avais pas cru qu'ils méritaient la peine de gaspiller l'attention du Congrès et c'est pour cela que je n'avais pas fait une communication; mais comme j'observe que rien n'a été dit dans cette séance à ce sujet, je ne veux pas que l'on pense qu'il n'y a aucune méthode valable pour cela. Très rapidement, je vais décrire notre méthode assez grossière.

Quand nous avons prédit un gonflement possible du sol, nous devons aussi supposer un certain gonflement différentiel.

Nous supposons un gonflement de cette forme :

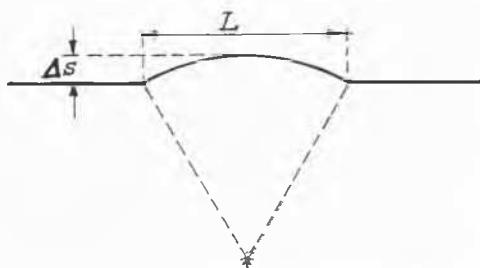


Fig. 39

L est une longueur arbitraire. Nous calculons les pressions produites sur la fondation avec l'aide de la courbe de gonflement et de la théorie générale. Nous tirons le coefficient d'élasticité des essais de compression simple sur le sol avec son humidité naturelle. C'est faux, naturellement, parce que le sol humide a un coefficient d'élasticité moindre, mais l'erreur est du côté de la sécurité et nous espérons la diminuer avec quelques développements que nous faisons actuellement. Il faut faire des tâtonnements et trouver quelle valeur de *L* donne la valeur maximum pour le moment de flexion de la structure. Finalement il faut contrôler que la pression maximum ainsi calculée n'excède pas la pression maximum que le sol peut supporter. Si elle l'excède, il y aura une fluxion localisée dans la zone où la pression est la plus grande; et cette fluxion améliore les conditions de travail de la structure. La fondation se calcule dans ce cas par un autre procédé qui a été décrit par moi-même à l'occasion du Congrès de Londres et que je ne veux pas répéter ici.

M. V. L. GRANGER (Rhodésie)

I wish to direct my remarks towards a single soil condition and a single solution to a particular foundation swelling problem in Southern Rhodesia. To describe the conditions under which this solution has been applied I would say that the annual rainfall in this area of Southern Rhodesia, in the City of Salisbury, is between 36 and 42 in., that there is virtually no rain between May and November. The condition I describe is a particular one, as I have mentioned, and only applies under these particular conditions, but it is a condition where the dry season water table is within 8 ft. of the surface in a condition of heavy clay, heavy expansive clay, montmorillonitic in nature.

There is a considerable history of cracking in the area under which these conditions apply and the heave which takes place is purely a seasonal one so far as we can determine. There is a considerable sinusoidal flap, and the foundations of the older houses were very shallow. It has been found that there is virtually no movement in foundations below about 5 ft. below the surface.

Therefore several solutions were possible, but all of them require that the floors and the suspended beams between the piles or columns should be entirely free of the ground.

The particular solution to which I want to refer is the placing of the house on ordinary aluminium scaffold tubes. These tubes in an experimental house, 23 such tubes, were driven to refusal with a standard penetrometer hammer and the house built on these tubes with suspended beams and pre-stressed planks for floors. This solution has been entirely successful : there has been no measurable movement in the last two years in such a house, and the seasonal movements have been entirely eliminated. The total time required to put in such a foundation was approximately six hours.

The panel speakers have taken part in a most interesting discussion of the factors controlling the moisture distribution in soils under sealed surfaces, and it appears to be accepted that in most circumstances the depth to the water-table is the major factor controlling the moisture distribution. I would like to present evidence which shows however that as the depth of the water-table becomes greater, seasonal changes in the moisture content of the exposed verges at the sides of a pavement become increasingly important. Even where the water-table is deep or non-existent seasonal variations in moisture content are usually confined to within a few feet of the edge of the pavement, but it is the availability of water at the pavement edges which is the main factor controlling the ultimate moisture condition under the pavement and the rate at which this ultimate distribution is approached.

In order to illustrate this point I should like to give three examples of moisture distribution beneath pavements. First in clayey sand adjacent to and under a 50 ft. wide taxiway at Khartoum airfield. Measurements taken over a period of several years showed that the moisture content of the 2-3 ft. of exposed soil varied between 3 and 7 per cent according to season. It is therefore supposed that the moisture content of the top foot of the subgrade was within this range when construction of the taxiway was completed in early 1952. The changes in moisture distribution which then took place in the top foot of the subgrade are given in Fig. 40.

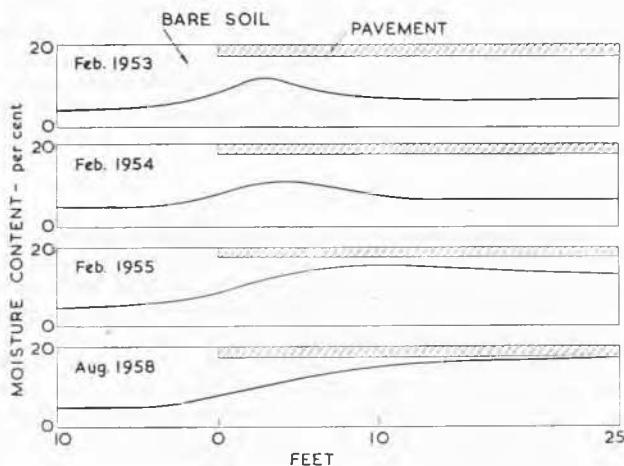


Fig. 40 Moisture in Clayey Sand Subgrade, Khartoum.

Although the annual rainfall at Khartoum is only about 8 inches, this falls as heavy showers and the water, concentrated by surface run-off to the edges of the pavement, soon resulted in wet conditions in the subgrade in this position. With succeeding years the moisture moved further under the pavement. The movement apparently takes place as a series of pulses, each pulse occurring during the wet season and followed by a period of extremely slow and slight drying of the subgrade from the exposed verge. The resulting moisture distribution six years after construction was such that the subgrade was much wetter than the comparable unpaved soil and became wetter with increasing distance from the edge of the pavement. Apparently less significant sources of moisture were from the water-table at a depth of 23-32 ft., and through the surfacing, but the moisture condition measured under the centre of the taxiway in 1958 approximated to the estimated moisture content for the soil in equilibrium with this water-table.

The Road Research Laboratory has, with the help of the Ministry of Works Kenya, been studying conditions under roads which have generally been bituminised within the last ten years. In the majority of cases the subgrade near the edge of the road was found to be wetter and weaker than the subgrade nearer the centre of the road. Typical results are given in Fig. 41 for a road 33 miles from Nairobi which was bituminised in February 1957. The soil is a red clay (L. L. 62, P. L. 32).

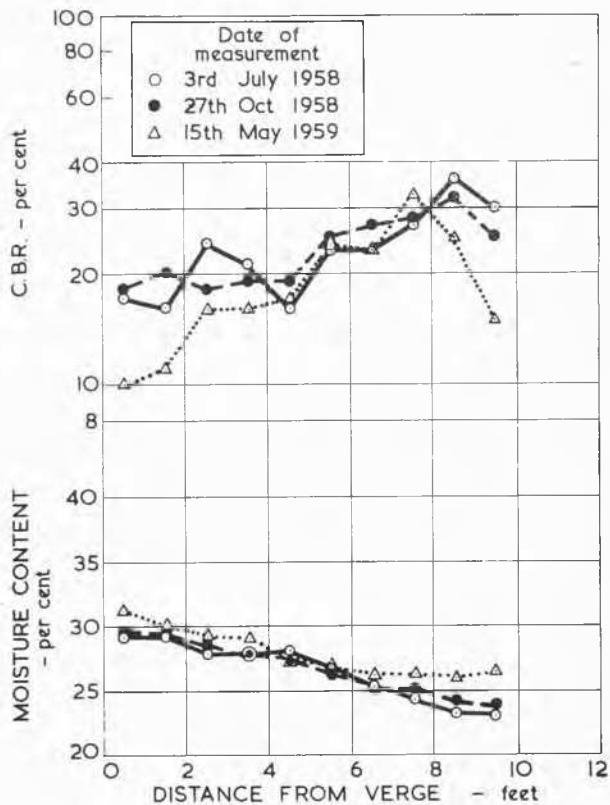


Fig. 41 C.B.R. and Moisture Content of Subgrade - Thika Sagana (Level).

The amount of water penetrating into the subgrade each year is influenced by the shape and type of construction of the road and verge as well as the soil permeability. On part of the Nairobi-Mombasa road for example, an undrained stone base extends a short distance into the verge and collects run-off water from the road surfacing. The relatively large amount of water held at the edge has produced a large "edge effect" or variation in moisture content and strength across the road as shown in Fig. 42. The road was bituminised in 1950 and the subgrade is a very heavy black clay of low permeability (L. L. 79, P. L. 33).

The results given above emphasise the dependence of the subgrade moisture distribution of the verge conditions.

While the final moisture distribution under the two roads cannot be accurately predicted at the present time the gradient of soil moisture across the road will represent a dynamic equilibrium where vertical moisture flow is balanced by the net inflow of moisture from the verges. In very dry climates, such as that of Khartoum, when equilibrium is established the net horizontal flow is from the subgrade to the verge and the subgrade near the edge of the pavement is drier than on the centreline. In wet climates the "edge effect" may be reversed.

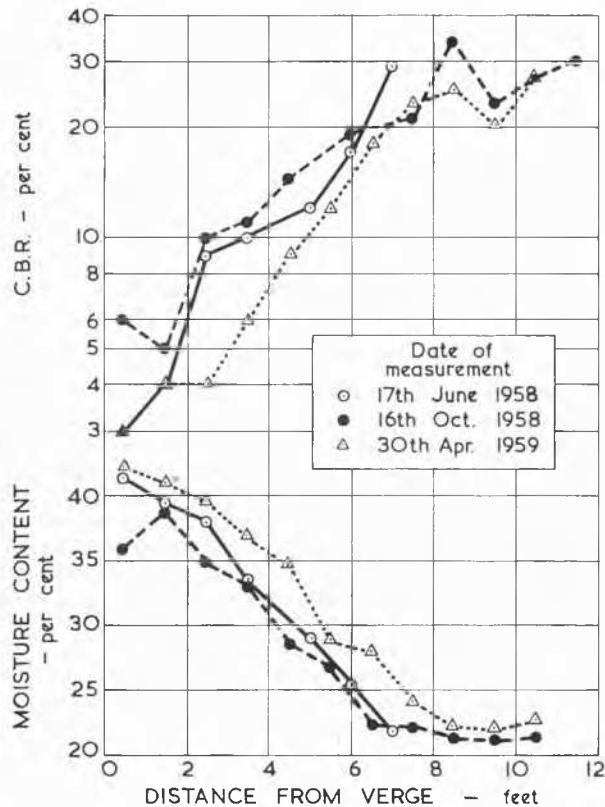


Fig. 42 C.B.R. and Moisture content of Subgrade - Mombasa Road Mile 12.

M. J. D. COLEMAN (Grande-Bretagne)

I wish to comment on the suggestion of Prof. Jennings that the effective stress law for unsaturated soils fails below a certain critical degree of saturation. As I understand it, if a certain dry sand carrying a load is wetted, he finds it to collapse always. A dry clay, on the other hand, may collapse at high loads and expand at low loads when it is wetted.

In March this year (Coleman and Russam, 1961) we have given the following expression for the effective stress in unsaturated soil :

$$\sigma'_i = -\beta' \cdot S + (1 - \alpha \cdot \beta') (p - \pi)$$

where σ'_i is the mean principal effective stress for failure of soil in shear

$$\beta' = \left(\frac{\partial(p - \pi)}{\partial(u - \pi)} \right)$$

at constant shear strength. It lies between zero and one. u = measured pore water pressure in the soil

S = the value of the measured pore water pressure when the total stresses are all reduced to zero, the shear stresses being removed first

p = mean principal total stress

π = measured pore air pressure in the soil

$\alpha = \left(\frac{\partial(u - \pi)}{\partial(p - \pi)} \right)$ at constant water content. It lies between zero and one.

The phenomena of the collapse of sand upon drying (at moisture contents above the optimum) was noticed at the Road Research Laboratory in 1943, and reported by Maclean (1955); it was interpreted in terms of the compaction curve

for soils. Dr Jennings' observation of the collapse upon wetting at moisture contents below the optimum is similar.

At low values of the $(p - \pi)$ term, the above expression indicates a low value for the effective stress in a dry sand, since β' will be zero. Thus, upon wetting the sand the term $-\beta' \cdot S$ rises, reaches a maximum, and then falls again to a low value as the sand approaches saturation, a conclusion supported by CBR results on sand (Croney, Coleman and Black, 1958). This behaviour is consistent with that observed by Prof. Jennings, the rise in effective stress as the dry sand is wetted causing it to decrease irreversibly in volume.

For heavy clays the term $-\beta' \cdot S$ is large at low moisture contents, and falls as the clay is wetted. (Croney and Coleman, 1960.) The reversible expansion of dry clay upon wetting at low loads is therefore expected. The contraction of dry clay upon wetting under high loads is more problematical, since the expression $(1 - \alpha \cdot \beta')$ also falls as the clay is wetted, and so the effective stress (under high loads) calculated from the above equation would again indicate a reversible expansion of the dry clay upon wetting. The possibility that the decrease in effective stress as the clay is wetted may simultaneously cause the clay to shear irreversibly under the high load, thus filling voids in the material and producing a nett settlement at the surface, may be suggested.

In this discussion, reversible and irreversible changes of volume in the soil are not regarded as equivalent. In particular, the coefficient β' employed in the discussion is strictly applicable only to irreversible changes of volume and shape in the soil, the appropriate stress point lying upon either a yield or the failure surface in principle effective stress space (depending upon whether plastic flow or ultimate shear of the soil is envisaged). For reversible changes of volume in the clay, a further coefficient β was put forward in the original thermodynamic generalisation of the effective stress laws for unsaturated soils (Croney, Coleman and Black, 1958) the stress points for such reversible changes lying within the appropriate yield surface in stress space. The discussion now given of the problems suggested by Prof. Jennings has not assumed, however, that β and β' are equal, but merely that they show the same qualitative variation with moisture content. Subsequent proposals for a standardised notation (Bishop and Aitchison, 1960) have attempted to use a single symbol for both reversible and irreversible changes in unsaturated soils.

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- [5] MACLEAN, D.J. (1955). "The application of soil mechanics to roads and engineering foundations". *J. Instn. Munic. Engrs.*, 81, (7), pp. 323-339.

M. W. H. WARD (Grande-Bretagne)

I would like to ask Dr Aitchison a very short question in relation to one of the examples which Dr Cooling described earlier in this discussion. We have a heavy clay site in England which under normal winter conditions is fully saturated and the water table lies at the surface. Over the centuries a large

tree has caused a local deficiency of water in a zone about 15 ft. deep and 200 ft. diameter. The tree was felled and the ground has been heaving for about 7 years and is still heaving. The dried zone is full of cracks but in the process of water entering the zone the cracks become sealed and prevent easy access of water. What is the quickest and most practical way of estimating the total amount of ground heave and how long will it take to reach a state of equilibrium?

Le Président :

Je remercie M. Ward pour sa communication.

Il n'y a plus d'orateur inscrit sur la question. Je tiens à remercier tous ceux qui sont intervenus dans notre discussion. Je remercie également tout particulièrement les nombreux auditeurs de leur attention.

(La séance est levée à 13 h 05)

Interventions écrites / Written Contributions

M. R. A. ASHBEE (Grande-Bretagne)

Design of uplift foundations

I would like to draw attention to two matters concerning the design of foundations to resist uplift forces. These relate to suction forces beneath the base and to the factor of safety in relation to the method of design.

(a) Suction Forces beneath the base

Suction and tension forces are generally ignored in the design of foundations subject to uplift forces. These forces act below the base of the foundation and they can be considerable. Any engineer who has witnessed the extraction of a 4 inch diameter sample tube from a clay soil has seen how powerful is the suction straining against withdrawal. As soon, however, as the vacuum below the sample is broken the extraction of the sample becomes relatively easy.

For uplift foundations, particularly in clay, there appears to be a good case for using this suction during design, as a force directly resisting shock loads; such as the transient forces following breakage of a conductor in an overhead transmission line.

It may be considered that suction forces beneath a foundation are inseparable from those due to tensile adhesion of the soil. Further observations are necessary.

In some uplift tests near Newark, Nottinghamshire, carried out by the Central Electricity Research Laboratories in November 1960, the tension or suction forces beneath the base of foundations 4 feet 6 inches square on Keuper Marl (a stiff clayey silt) were very nearly $\frac{1}{2}$ ton/sq. ft. This was before any significant movement of the foundation had taken place. Such a force, nearly 10 tons in this case, contributed up to 25 per cent of the ultimate failure load and cannot economically be neglected.

(b) Factor of Safety

Nearly all methods of design against uplift forces, such as that by Dr Balla in Volume 1 of these proceedings (3A/3) and those methods in common use in the electrical supply industry, determine the working load of a foundation as a proportion of the rupture load, this meaning rupture of the soil.

These methods are often unrealistic. Most structures are out of service at deflections well below those corresponding to complete rupture of the soil, and such deflections cannot be predicted from strength tests alone.

It would be valuable if investigators in this field, whilst ensuring an adequate reserve against rupture, could show how an economic design can be based on specified deflections.

It is a matter of conjecture whether Lazard's mast overturning tests, on some 197 masts, would not give more acceptable results if based on the elastic or elastic-plastic properties of the soil. His conclusion, from his tests, that the results were independent of soil properties (presumably strength) is surely a strong challenge to any member of this Society.

MM. J. BERNEDE et K. PAKDAMAN (France)

Mesures des contraintes dans un massif pulvérulent

Pour les mesures de contraintes dans les sables, le C.E.B.T.P. a adopté le dynamomètre à friction. Un prototype de cet appareil avait été présenté par Habib (1957) à l'occasion du Congrès de Mécanique des Sols de Londres; depuis, il a été à la fois perfectionné et largement utilisé.

L'appareil se compose de trois lames d'acier accolées et protégées par une gaine très mince. On mesure le frottement de la lame centrale coulissante. Deux sections de garde encadrent la partie utile et permettent ainsi d'éliminer les effets perturbateurs des parois (Fig. 43).

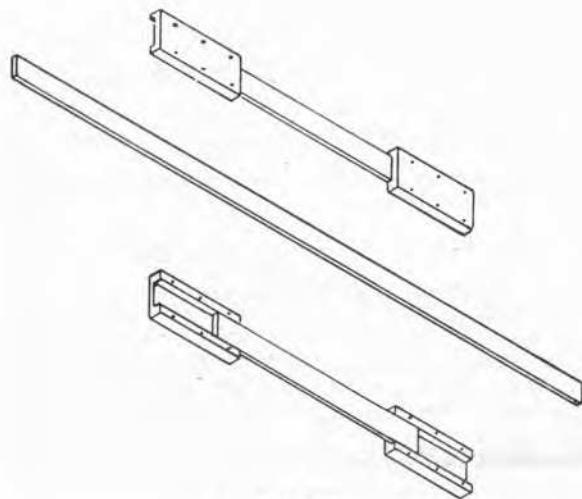


Fig. 43

Cet appareil a été jusqu'à présent utilisé dans des systèmes bidimensionnels. Il convient bien aux mesures à la paroi mais aussi au sein du massif moyennant certaines précautions, car il ne modifie les propriétés du milieu que de façon négligeable (Fig. 44).

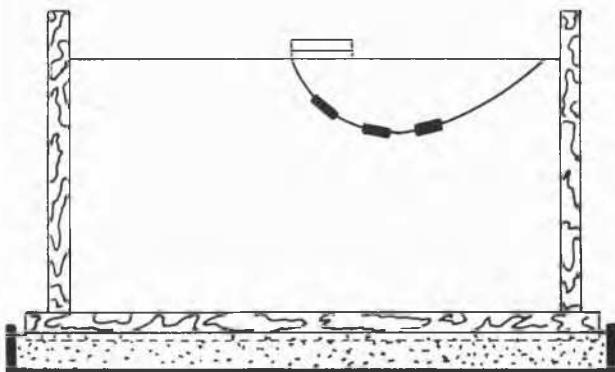


Fig. 44

Il a été possible notamment, de reconnaître la répartition des contraintes sous une bande superficielle chargée sur du sable et en particulier la concentration des tensions qui s'accroît avec la charge et devient très importante quand on approche de la rupture.

Dans le cas du poinçonnement d'une couche pulvérulente relativement mince, sus-jacente à un massif cohérent, Tcheng a montré en 1958 que la rupture se produit selon deux plans pratiquement verticaux passant par les arêtes de la semelle. Nous avons placé des dynamomètres à friction le long des lignes de glissement pour établir le bilan des forces. Cet essai a permis de déterminer la répartition des contraintes sur un modèle de dimensions importantes.

Depuis l'appareil a été encore perfectionné et de nombreux essais ont été réalisés. Nous avons étudié notamment l'incidence du rapport de la dimension de grain à la largeur des lames ainsi que l'erreur résultant du fléchissement possible du témoin dans le milieu. Pour ce dernier point, il est difficile dans les conditions habituelles des expériences que l'erreur dépasse quelques pourcents [1].

Nous avons vérifié en particulier de façon expérimentale, l'application de l'équation de Kotter le long d'une ligne de glissement, en y plaçant un certain nombre de dynamomètres à friction (Fig. 45). Ces appareils ont également permis de suivre en cours d'accroissement de la charge sur une bande superficielle, la variation du rapport de la tension verticale à la tension horizontale, et de reconnaître le passage du système à un équilibre nettement plastique. Ce rapport tombe en effet rapidement de 0,5 à 0,2 environ.

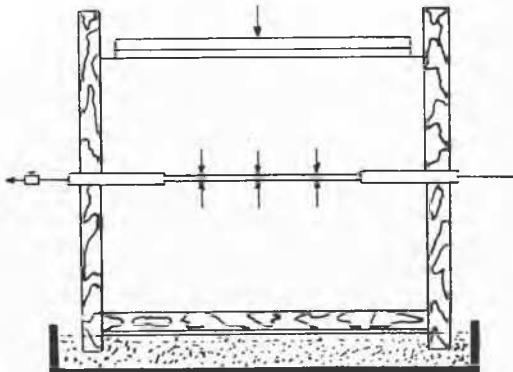


Fig. 45

Ce même rapport mesuré sous le seul effet de la gravité (coefficient au repos) a été trouvé voisin de 0,9 ce qui apporte une justification expérimentale de l'hypothèse de Boussinesq : le coefficient de Poisson d'un milieu pulvérulent reste très voisin de 0,5 dans le domaine des petites déformations.

Référence :

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M. J. BRINCH HANSEN (Danemark)

Discussion on « The Shape of Rupture Surface in Dry Sand » (A.R. Jumikis) (3 A/23)

Prof. Jumikis states that in his tests with inclined foundation loads the experimental sliding surface coincides very closely with a logarithmic spiral. However, both from his Fig. 4 and from his photographs it is evident that, at least under the foundation proper, considerable deviations exist.

I have, first in 1955 [1] and also recently [2], indicated and calculated kinematically possible figures of rupture for the considered case of an inclined, central load. As examples can be shown Fig. 46 (weightless earth) and Fig. 47 (heavy earth). The Rankine triangle (to the right) may actually move as a rigid body without internal plastic deformations. It will be seen that this type of rupture-figure agrees considerably better with Prof. Jumikis' experimental results than a single logarithmic spiral.

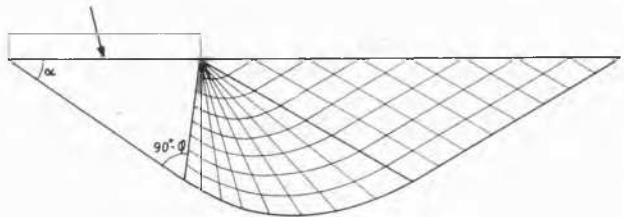


Fig. 46

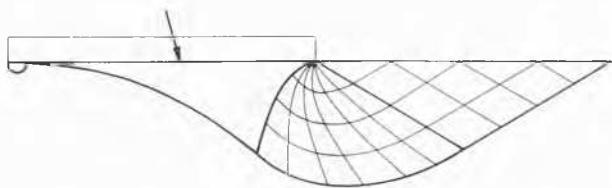


Fig. 47

The results of my calculations (for a strip foundation) can approximately be expressed in the following formula for the vertical component of the unit bearing capacity [2] :

$$b_v = \frac{1}{2} \gamma B N_\gamma \left(1 - \frac{H}{V}\right)^4 + q N_q \left(1 - \frac{H}{V}\right)^2$$

I suggest that Prof. Jumikis try to evaluate his experimental results by means of this formula. The second term, corresponding to a unit surface load q , must be included, putting $q = \gamma \delta$, where δ is the level difference (at failure) between the surface of the Rankine wedge and the underside of the foundation.

Similar remarks can be made to paper 3A/52 : *Sur la stabilité des fondations rigides (E. Zaharescu)*.

Références

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[2] — (1961). "A general formula for bearing capacity". *Ingeniøren*, International Edition, June, Copenhagen, and *Bulletin No. 11*, The Danish Geotechnical Institute, Copenhagen.

M. J.B. BURLAND (Afrique du Sud)

The phenomenon of collapse of soil structure on wetting is one that is now receiving some considerable attention. Papers 3A/8, 3A/20 and 3A/36 deal with this problem. In 1957 Jennings and Knight discussed collapsing sands in some detail and presented a simple test to identify and predict collapse.

Recent work by Burland (1961) indicates that, below a critical degree of saturation, collapse can take place not only in granular materials but also in clayey soils. The discussion

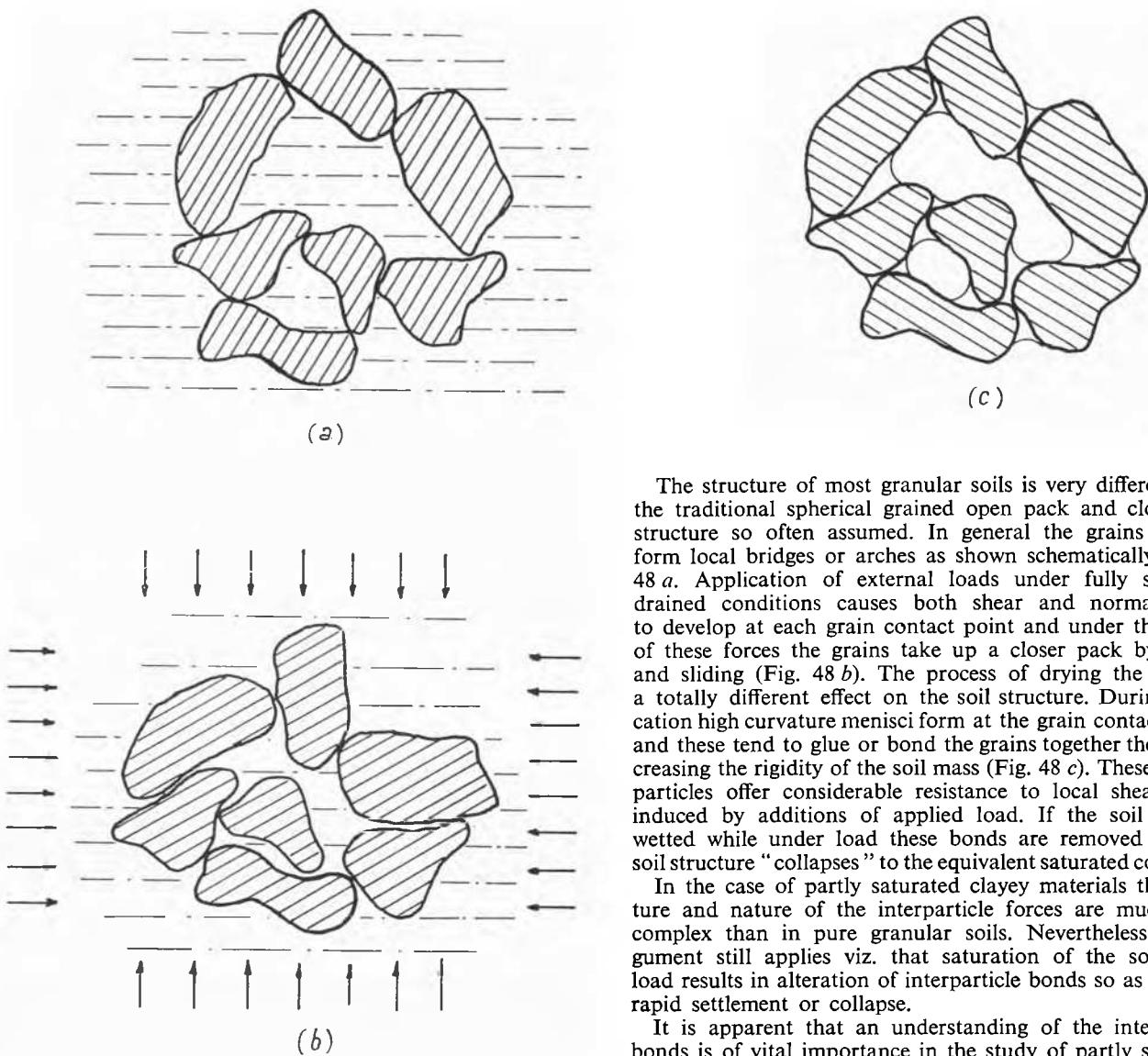


Fig. 48 Structural changes in a granular soil: a) Initial loosely packed soil; b) Displacement of grains resulting from an increase in applied load; c) Particles bonded together by the action of high curvature menisci at grain contact points.

presented by Prof. Zeitlen earlier this session confirms this idea. The critical degree of saturation just mentioned is usually below about 50 per cent for granular soils but in clayey soils it can be as high as 90 per cent.

Holtz and Hilf (paper 3A/20) suggest that the collapse phenomenon results from the decrease in shearing resistance of the soil on wetting. However tests have shown that collapse occurs in equal all round compression and that this collapse is very nearly as large as that occurring in laterally confined compression. Clearly such behaviour is not explained by the action of shear stresses developing from differences in the principal stresses as inferred by Holtz and Hilf.

The phenomenon of collapse can rather be attributed to alterations in soil structure resulting from changes in the nature of the bonds between soil particles. In the case of partly saturated granular soils these bonds result from high curvature menisci at grain contact points whereas in clay soils the nature of the bonds are probably far more complex.

The structure of most granular soils is very different from the traditional spherical grained open pack and close pack structure so often assumed. In general the grains tend to form local bridges or arches as shown schematically in Fig. 48 a. Application of external loads under fully saturated drained conditions causes both shear and normal forces to develop at each grain contact point and under the action of these forces the grains take up a closer pack by rolling and sliding (Fig. 48 b). The process of drying the soil has a totally different effect on the soil structure. During desiccation high curvature menisci form at the grain contact points and these tend to glue or bond the grains together thereby increasing the rigidity of the soil mass (Fig. 48 c). These bonded particles offer considerable resistance to local shear forces induced by additions of applied load. If the soil mass is wetted while under load these bonds are removed and the soil structure "collapses" to the equivalent saturated condition.

In the case of partly saturated clayey materials the structure and nature of the interparticle forces are much more complex than in pure granular soils. Nevertheless the argument still applies viz. that saturation of the soil under load results in alteration of interparticle bonds so as to cause rapid settlement or collapse.

It is apparent that an understanding of the interparticle bonds is of vital importance in the study of partly saturated soil behaviour. Serious thought must also be given to whether such soils can be treated generally in terms of effective stresses or whether the individual components of effective stress are not more important in determining the soil behaviour (see discussion to section 1 by J.B. Burland).

Références

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M. B.O. CORBETT (Grande Bretagne)

Settlement of Turbo-Alternator blocks

In recent years there has been a significant increase in the power output from turbo-alternators and because of the shortage of suitable sites power stations may be built at places where the soil conditions render a total settlement of several inches in magnitude inevitable.

To study the behaviour of turbo-alternators under working conditions in an effort to improve the design of the foundations, a number of measurements are being made at a power station in England. This site is particularly interesting since the Turbine Hall is founded on granular material. The geology consists of dense sandy gravel (River Gravel) overlying the Lower Keuper Sandstone. Vibration might be a factor contributing to the settlement.

The programme of work consists of

- (a) precise levelling to settlement points fixed into the lower part of the turbo-blocks.
- (b) differential levelling with a manometer system around the lower part of the blocks with a correlated survey on the operating decks.
- (c) concrete shrinkage measurements using 4 in. linear electrical resistance strain gauges mounted on the concrete surface near the settlement points.
- (d) measurement of the vibrations during the operation of the sets.

- (e) theoretical settlement analysis under static and dynamic loads.

The German specification DIN 4024 "Stützkonstruktionen für Rotierende Maschinen" covers the design of the foundations for turbo-alternators. It is worth noting that the foundations would not conform with this code in all respects. In particular the columns of the turbo-blocks are integral with the main raft of the Turbine Hall.

Turbo-alternator No. 1 with an output of 120 MW at 3 000 r.p.m. was commissioned in December, 1960. Before this time levelling commenced and subsequently vibration measurements have been made. At the time of the last measurements the eccentricity of the H.P. turbine shaft was about 0·0031 inches and of the I.P. shaft about 0·0024 inches. The vibration displacements at the bearings were between 0·0001 and 0·0012 inches. Levelling has revealed an increase in settlement of less than 0·1 in., probably all due to the additional weight of the stator and turbine casings which were fitted after measurements began.

The vibration measurements are summarised below :

Summary of vibration measurements

(Peak displacements in inches).

Position	Vertical		Lateral		Longitudinal	
	Max.	Min.	Max.	Min.	Max.	Min.
Top deck - turbine end .	0·000330	0·000050	0·000110	0·000045	0·000165	0·000145
Top deck - alternator end.	0·000380	0·000045	0·000170	0·000165	0·000110	0·000090
Base of columns	0·000070	0·000030	0·000055	0·000010	0·000037	0·000010

The significance of these measurements is that damping in the block has reduced the vertical displacements (which would be mainly responsible for settlement due to dynamic forces) by an order of five to six between the deck and ground floor level. The lateral and longitudinal vibrations at the ground floor level have been similarly reduced by an order of about three. These vibrations of the foundation are well within the safe limits suggested by Richart.

Records taken on a cathode ray oscilloscope have shown that the vibration is basically sinusoidal at a frequency of 50 cycles/second. Consequently it should be possible to analyse this dynamic system theoretically. A recent paper on machine foundations deals with this type of problem (Alpan 1961). (Similar dynamic measurements have been made on a 50 MW turbo-alternator with a piled foundation in sand and sandy clay by Slooff, Roorda and Van der Veen).

It is hoped that it will be possible from the outcome of this work to adopt a more rational approach to the design of turbo-alternator foundations on granular soils.

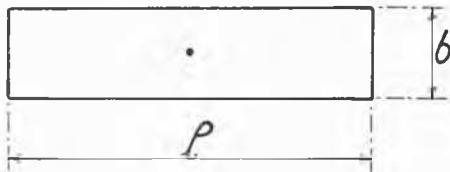
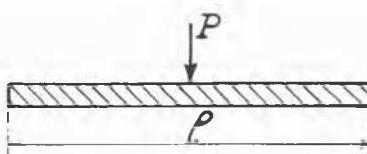
The programme of measurements is part of a research programme by the Central Electricity Generating Board with the writer's company, Soil Mechanics Limited, assisting the Board in this case where a granular soil offers a promising field of study.

MM. E. DE BEER et E. LOUSBERG (Belgique)

Par sa contribution (3A/48) le Prof. Vesic a notamment dégagé le domaine d'applicabilité de la méthode du coefficient de raideur K pour le calcul des réactions du sol sous une poutre.

En 1948 [1] le premier auteur a montré que l'écart entre le moment maximum M_E fourni par la méthode du module d'élasticité E_s constant et celui M_K obtenu par la méthode du coefficient de raideur K , est maximum pour une poutre rectangulaire chargée d'une force centrale (Fig. 49) quand son inertie est infinie.

En fonction de $\delta = \frac{b}{l}$ et pour $I = \infty$, la Fig. 50 donne ces écarts qui varient entre — 19 pour cent et — 12 pour cent



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- [2] ALPAN, I. (1961). "Machine foundations and soil resonance". *Geotechnique*, vol. 11, June.
- [3] SLOOFF, F., ROORDA, W. and VAN DER VEEN (1954). "Berekeningen op meetresultaten betreffende trillingsverschijnselen van een turbinefundament in de Electrische Centrale Hemweg te Amsterdam". *De Ingenieur*, May.

Fig. 49

et sont à comparer à la valeur de — 14,5 pour cent indiquée à la figure 3 de la contribution de M. Vesic pour $\lambda = 0$. Les résultats obtenus concordent avec ses conclusions.

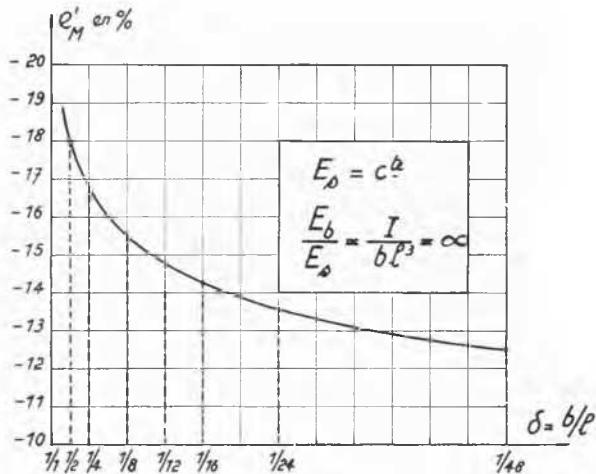


Fig. 50

Les écarts entre les moments M_E et M_K ne sont donc jamais très importants pour une force isolée appliquée au centre d'une poutre. On pourrait donc se contenter de la valeur obtenue en utilisant le coefficient de raideur si la détermination du moment exact s'avérait laborieuse.

Par ailleurs nous voudrions souligner que l'hypothèse consistant à remplacer les couches réelles par un matériau caractérisé par un E_s constant ne fournit pas automatiquement de résultats plus exacts que si on remplace ces couches par un liquide caractérisé par le coefficient K car les couches réelles sont souvent caractérisées par une compressibilité qui diminue avec la profondeur Z . Ainsi E_s peut croître linéairement avec la profondeur

$$E_s = C \gamma_k Z \quad [2]$$

Pour $I = \infty$ nous avons comparé (Fig. 51) les répartitions correspondant aux cas $K = \text{cte}$ (a), $E_s = \text{cte}$ (c) et $E_s = C \gamma_k Z$ (b). La répartition b est intermédiaire entre les répartitions a et c.

L'hypothèse E_s constant ne conduit pas nécessairement à des résultats situés plus près de la réalité.

Si le sol réel se compose d'une couche compressible d'épaisseur e plus faible que la longueur l de la construction et reposant sur une couche beaucoup moins déformable (Fig. 52), les résultats obtenus avec l'hypothèse E_s constant pour un sol constitué d'une couche unique peuvent s'écartez plus fortement de la réalité que dans l'hypothèse du coefficient de raideur K .

Nous nous rallions dès lors à la conclusion du rapporteur général lorsqu'il constate qu'au cas de fondations à faible rigidité relative prenant appui sur une couche plus épaisse de sol compressible recouvrant un substratum rocheux, la méthode du coefficient de raideur donnera des résultats plus exacts que celle obtenue en considérant un massif indéfini caractérisé par un E_s constant; nous voudrions néanmoins nuancer un peu sa conclusion lorsqu'il dit que dans tous les autres cas et spécialement pour les sols compacts uniformes, les calculs effectués en partant de l'hypothèse d'un espace semi-infini élastique fourniront des résultats plus proches de la réalité. Nous voudrions notamment ajouter la restriction « pour autant que les propriétés de déformabilité réelles du sol puissent être caractérisées par un E_s constant ». En effet si tel est le cas pour de nombreux problèmes, par contre pour d'autres les propriétés de déformabilité sont mieux caractérisées par un E_s dont la valeur croît avec la profondeur,

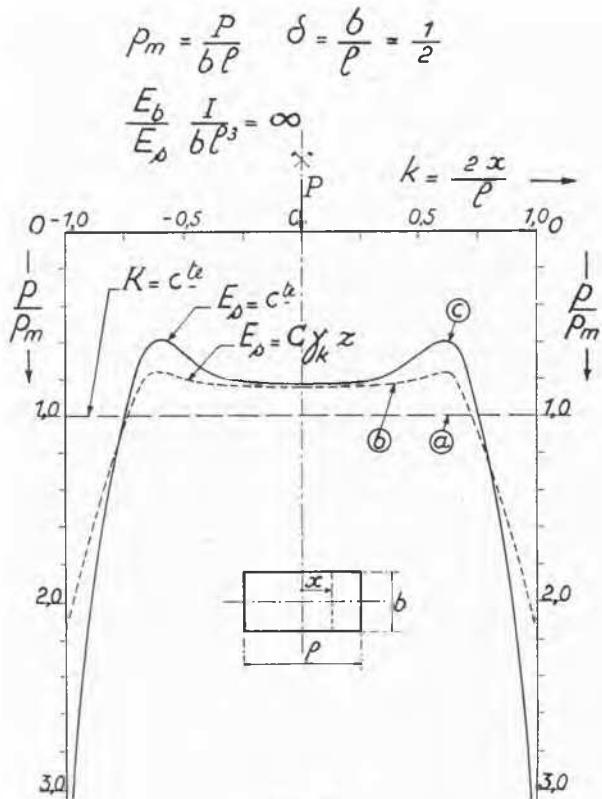


Fig. 51

ce qui nous rapproche alors à nouveau dans une mesure plus ou moins importante de la solution du coefficient de raideur.

La configuration des couches, leurs épaisseurs relatives par rapport aux dimensions de la construction, leurs propriétés de déformabilité intrinsèque ainsi que leur succession doivent nous guider en définitive dans le choix de la méthode.

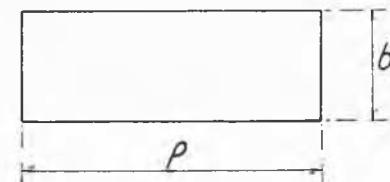
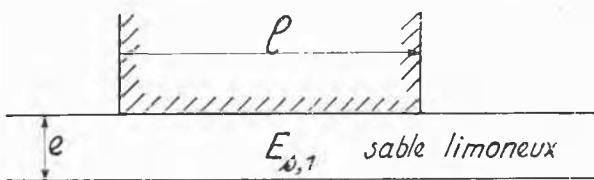


Fig. 52

Références

- [1] DE BEER, E. (1948). « Calcul de poutres reposant sur le sol, le coefficient de raideur K du sol ». *A.T.P.B.*
- [2] — LOUSBERG, E et VAN BEVEREN, P. (1956). « Le calcul de poutres et plaques appuyées sur le sol ». *A.T.P.B.*

M. A. EGGESEN (Norvège)

In connection with the paper 3A/43 by H.U. Smoltczyk dealing with a theoretical calculation of the shear deformations beneath a shallow foundation, I would like to describe some model tests with foundations on sand which have been carried out at the Norwegian Geotechnical Institute during the last year.

The only purpose of these tests was to investigate the mode of deformation of the sand below a footing. By use of small

variable inductance gauges the vertical and horizontal strains were measured at various depths below the model footing. The vertical strain was measured at 6 points along the vertical centreline. The horizontal strain was measured at four different depths. The model footing was 20 cm in diameter, the thickness of the sand layer was 50 cm, and the diameter of the container was 1,13 m. The tests were carried out with a dry fine to medium sand and different porosities were used. The porosity limits were according to Kolbuszewski, 36 per cent and 47 per cent.

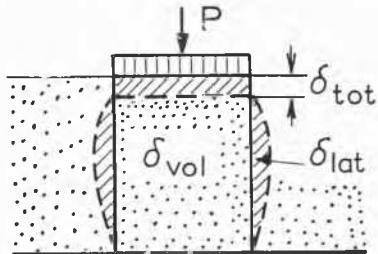
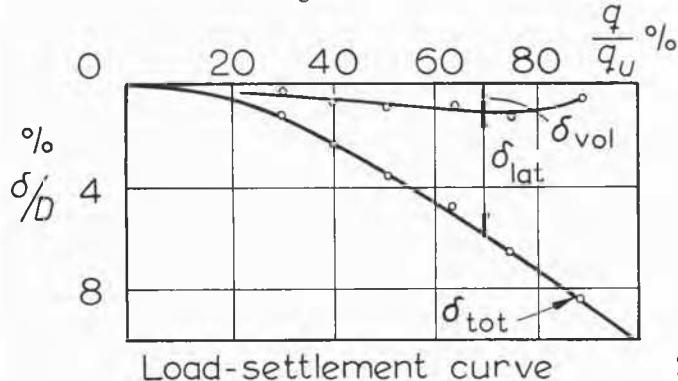


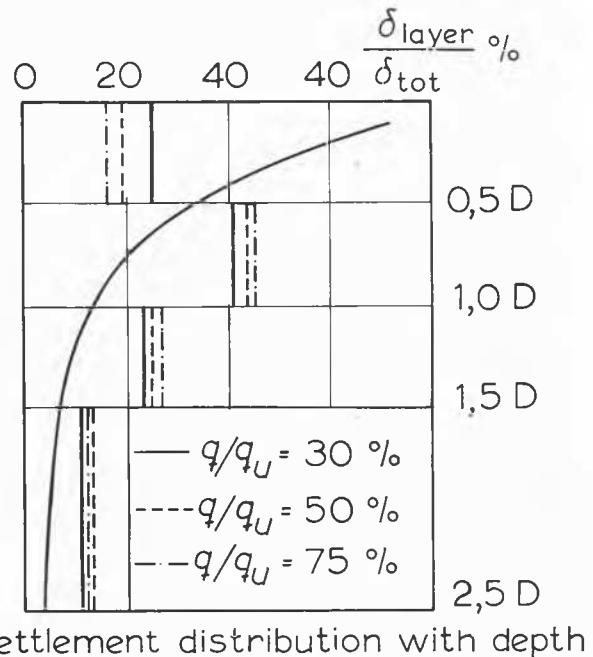
Fig. 53.



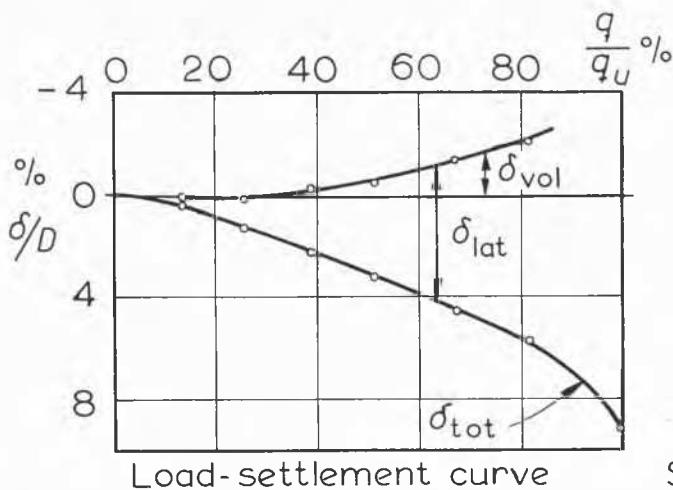
Load-settlement curve

RELATIVELY LOOSE SAND ($D_r = 40\%$)

Fig. 54



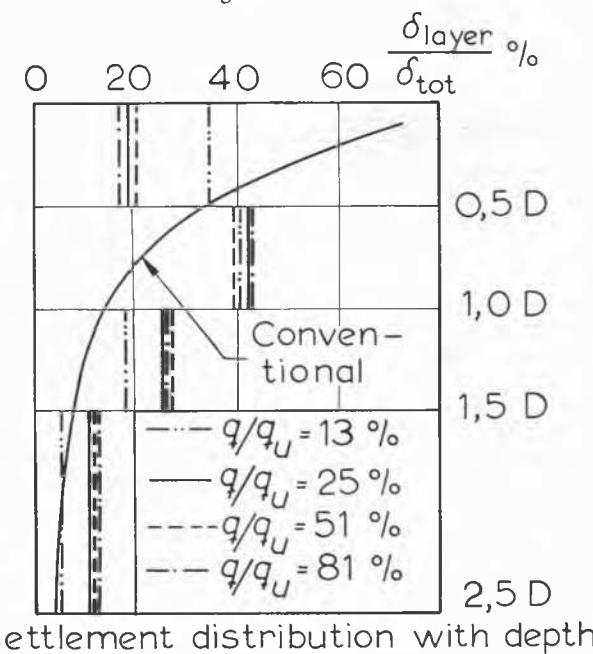
Settlement distribution with depth



Load-settlement curve

Fig. 56

DENSE SAND ($D_r = 85\%$)



Settlement distribution with depth

Fig. 57

From the observations it is possible to calculate how great a part of the total settlement is due to lateral displacement and how much is caused by a change in volume. The principle of this method of presenting the test results is illustrated on Fig. 53. Figs 54 and 56 show load-settlement curves in terms of the q/q_u ratio versus the δ/D ratio where q is the applied foundation pressure, δ is the settlement, D is the diameter of model footing, and q_u is the pressure at $\delta/D = 10$ per cent. In the diagrams two curves are plotted, one for the total settlement and one for the settlement caused only by volume change.

Furthermore, from the vertical strain measurements it is possible to determine the distribution of settlement with depth, which is shown on Figs 55 and 57 for different load ratios. In the same diagrams are shown the distribution which can be assumed from a conventional settlement prediction based on consolidation tests.

The test results show that the settlements of shallow footings on sand are mainly due to lateral strain. In the case of relatively loose sand that part of the settlement which is due to lateral strain is about 50 per cent for q/q_u ratios smaller than 20 per cent, increasing to nearly 100 per cent at failure. For dense sand the volume increases for q/q_u ratios greater than about 30 per cent, which is due to the dilatancy of the material.

The settlement distribution diagrams, Figs 55 and 57, confirm this conclusion as the main contribution to the settlement originates from lateral displacement in the depth from $1/2 D$ to D , where the shear stresses very likely are maximum. The settlement distribution with depth is very different from that assumed in a conventional settlement prediction based on consolidation tests, and it seems to be independent of the porosity of the sand and only slightly dependent on the loading ratio, q/q_u .

For sands, these test results may perhaps contribute to a more careful use of methods of settlement analysis based on standard consolidation tests, and lead to greater efforts to develop methods based on the shear strain characteristics of the material.

M. M. I. GORBUNOV-POSSADOV (U.R.S.S.)

The most modern methods of calculation of finite settlement and stability of foundations, as well as beams and slabs on elastic foundations employ the theory of elasticity or the theory of limit-stressed condition (plasticity) of soils. The actual settlement, the value of the ultimate load, distribution of pressures at the toe of the foundation frequently significantly differ from the results of these calculations. This divergence is explained not only by incorrect reflection by the mentioned theories of the complex mechanical properties of the soils, but to a high degree by incorrect application of both theories. In many important cases both elastic and plastic fields simultaneously exist in the foundation of the structure. These fields interact along their borders with equality of normal and tangential stresses at both sides of the borders. These features determine not only the stressed condition but also show the borders of the elastic and plastic fields.

This problem of calculation of foundations is known as the mixed problem of the theory of elasticity and plasticity. Its solution is of great economic importance. Thus, for example, estimation of the origin of plastic deformations in the soil under the foundation edges should very much lower the rated values of the bending moments. When calculating the stability of foundation estimation, the formation of an elastic core below the rough toe of the foundation leads to a sharp increase of the rated ultimate load in comparison with the load determined on the basis of the theory of plasticity of soils. Introduction of a rigid core into the calculations instead of the elastic core, is not justified both by experimental data on distribution of pressures under the stamp and because choice of its shape is arbitrary. The shape of the core should be determined on the basis of solution of the mixed problem; the shape of the elastic core predetermines the shape of the adjoining compacted core, which is in a plastic condition, and the entire prism of yielding soil (Fig. 58).

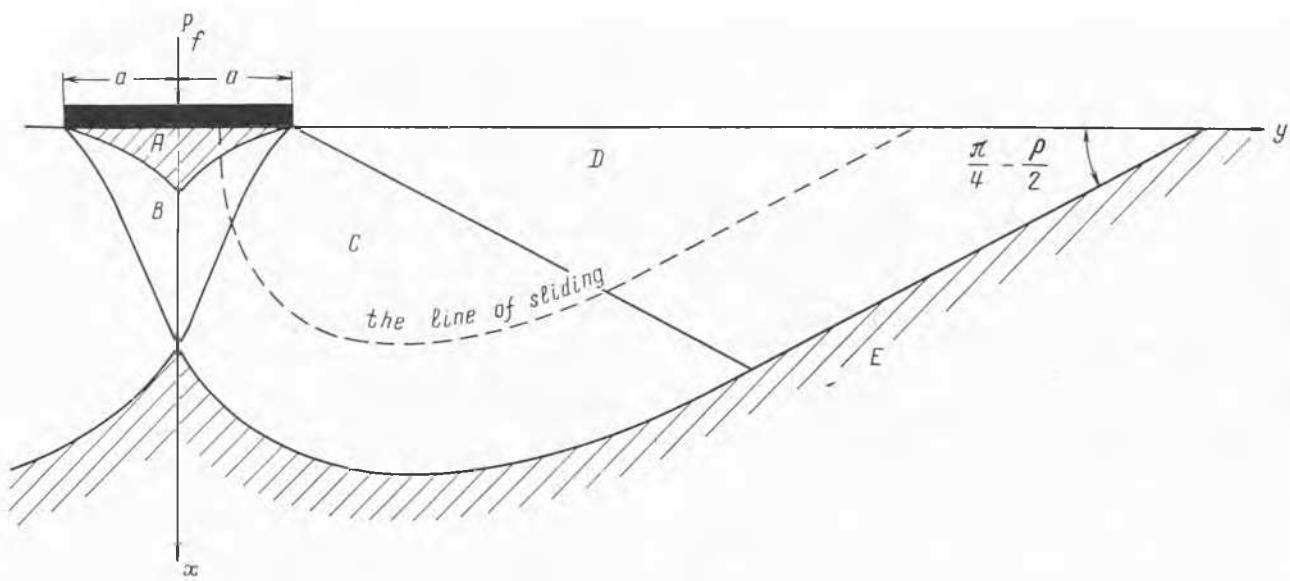


Fig. 58 General scheme of solution of combined problem on stability of stamp on sand foundation: A. elastic core; B. compacted plastic core; C. transient plastic Prandtl zone; D. zone of Renkin simple stressed condition; E. semi-infinite elastic zone.

We have solved the plane static problem of determination of the borders of the elastic core under a rigid central-loaded stamp placed on the surface of a sand foundation proceeding from the following conditions :

1. Strain and stress in the core are determined on the basis of the elastic theory.

2. The following is adhered to inside the core:

$$A \equiv \frac{\sigma_1 - \sigma_2}{\sigma_1 + \sigma_2} \leq \sin \varphi \quad (1)$$

where σ_1, σ_2 = main normal stresses and φ = angle of internal friction of sand. The lower sign in (1) relates only to the lower border of the core.

3. The foundation is not shifted along the line of contact in respect of the stamp.

4. The core border for the plastic zone is not one of the lines of sliding and is not the intrinsic curve of these lines; it is similar with a back facet retaining wall with variable angle of friction δ between the wall and plastic soil ($\delta \leq \varphi$).

5. It is considered that elastic displacement in the core is significant in relation to the rectilinear vertical displacement of the core downward with the stamp under the load approaching the critical value. In a plastic compacted core the trajectory of displacement of sand particles is identified with the lines of sliding. On the basis of these premises and experimental data showing that there is no breaking on the displacement trajec-

tories when passing from the elastic core to the compacted plastic core, it is considered that the direction of one family of lines of sliding in the plastic core is vertical when passing to the border of the elastic core.

6. In the upper corners all three stress components equal zero, as there is no surcharge outside of the stamp. At the lower corner the stresses are also equal to zero, as the elastic core is wedged into the lower sand, while the tensile stresses cannot arise in sand.

7. The core is considered imponderable, this taking into account its insignificant size leading to small errors.

For solving this problem the stress function, consisting of a biharmonic double polynomial of the 7-th degree and of 6 biharmonic functions of the $C_i r_i^2 \ln r$, where r = distance from pole "i", was introduced. Points were chosen as the poles on the axis of symmetry below the toe of the stamp at distances reduced to the half-width of the stamp

$$x_0 = 0.6, \quad x_1 = 0.75, \quad x_2 = 1, \quad x_3 = 1.25,$$

as well as the points with coordinates $x = 0.595, y = \pm 0.1$. The coefficients in the stress function were taken, considering precise performance of all above-mentioned conditions, except for the condition of $A = \sin \varphi$, which is taken approximately.

As a result of successive approximations, the solution is found for the case of $\varphi = 40^\circ$, characterized in Table 59 and Fig. 60.

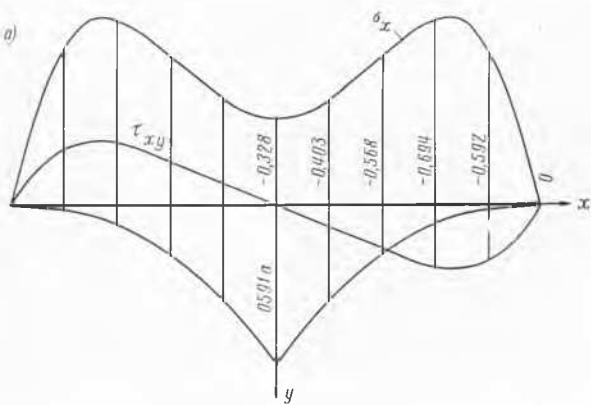
Table 59

y	0.0	0.2	0.4	0.6	0.8	1.0
x	0.5910	0.3613	0.1826	0.0700	0.0152	0
σ_x	0	-0.6334	-0.6647	-0.7020	-0.5854	0
σ_y	0	-0.1479	-0.2743	-0.3205	-0.2676	0
τ_{xy}	0	0.0779	0.2305	0.2553	0.2098	0
A	—	0.653	0.643	0.623	0.617	—

Table 59 shows the reduced coordinates for a row of points at the lower border of the core and the values of stresses at these points in fractions of P_f/a , where P_f — critical load. The last line of the table indicates the degree of accuracy of the solution. With a precise solution $A = \sin 40^\circ = 0.643$.

The solution may be found more precisely by increasing the number of biharmonic functions.

The value of critical load P_f is set during the following stages of solution of the problem.



M. H. GRASSHOFF (Allemagne)

I should like to make some remarks regarding the contribution 3A/48 of Prof. Vesic.

There are two main methods of calculating elastic foundation beams :

1. The so-called Winkler's hypothesis (theory of the first degree).

2. The method of the elastic-isotropic subsoil (theory of the second degree).

Vesic made very interesting investigations to find out up to which length of foundation beams the so-called Winkler's hypothesis is still valid and when the method of the elastic-isotropic subsoil gives more precise values.

In this contribution the possibilities of the application of these two methods were very clearly limited in a theoretical way. But I do not believe that these refined differences are of essential, practical importance, because the subsoil reacts neither like a system of independent springs nor like an elastic-isotropic medium. The so-called Winkler's hypothesis has been preferred mostly because it leads to a complete solution if a constant coefficient of subgrade is applied. The method of the elastic-isotropic subsoil demands much more work. Levinton developed an approximate method which allows the use of a coefficient of subgrade of any desired distribution. I extended this method to that of the elastic-isotropic subsoil with the possibility of using any variable

Fig. 60 a) Stress diagrams under stamp toe in fractions of value P_f/a : σ_x vertical normal stresses (reactive pressure); τ_{xy} tangential stresses. b) Border of elastic core.

modulus of deformation of the subsoil. I had the two methods programmed for an electronic computer which, with only few parameters, can calculate a great number of examples in a short time. In a series of nearly 80 examples in which we stated the limiting cases of a rigid and an absolutely flexible superstructure we have found that the influence of the stiffness of the superstructure on the distribution of contact pressure is not so important. The stiffness of the foundation beam itself — and most of all the more or less stiff connection of the columns with the foundation beam — shows a more important influence.

Today computation methods are highly developed by using electronic computers. Without making it more difficult they permit the use of any variability of the coefficient of subsoil or modulus of deformation of the subsoil. Only one cannot say anything about the distribution. One can only come nearer to the solution of the question in an empirical way. The methods of calculation are now already sufficiently perfected for all possibilities of variability of the coefficient of subsoil or the modulus of deformation of the subsoil to be treated. They have widely over-rounded the measurements. To develop the theory of flexible beams on elastic subgrade more and more, it is above all necessary to examine the calculation results by measurements.

M. A. R. JUMIKIS (Etats-Unis)

On "Deformation in dry sand observed in laboratory experiments with cantilever sheetpile models"

Qualitatively, the extent of the deformation of the passive zones in dry sand brought about by a horizontally loaded

cantilever sheetpile model depends very much, among other things, on the flexibility of the sheetpile material. Laboratory experiments show that the more rigid the sheet pile, the less extensive the deformed passive zone of soil of the opposite side of the applied load to the sheetpile. Also, the less restrained is the sheetpile in the sand, the less are sheared-off bodies of the soil mass in question (Fig. 61 and 62).

The more flexible the sheetpile, the more it can store up energy (like a bow : the more it is loaded, the more energy it stores up), and the larger are the sheared-off sand wedges (passive zone) on the opposite side of the applied horizontal load to the sheetpile (Fig. 63). On this passive zone there rides a relatively small active zone.

These figures also show the existence and position of a pivot point about which the sheetpile rotates. The pivot point in the sand (point of rotation), if the pile does not rest on rock, is a mechanical necessity for balancing the forces in question (externally applied load, active and passive earth pressures).

These features as described above and the kinematics of the formation of the rupture surface were also demonstrated to those attending the congress by a short Rutgers University film on Friday, July 21, 1961, at 15.00 hours, Auditorium I. The film shows that the formation of the rupture surfaces is a continuous process. Hence an attempt should be made to describe each of the rupture curves by corresponding continuous functions as closely as is practically possible.

Also, the size of the passive zones depends upon the rate of horizontal loading of the sheetpile. Knowledge of the sizes and shapes of the active and passive zones is of importance in stability calculations of sheetpile-soil systems.



Fig. 61



Fig. 62

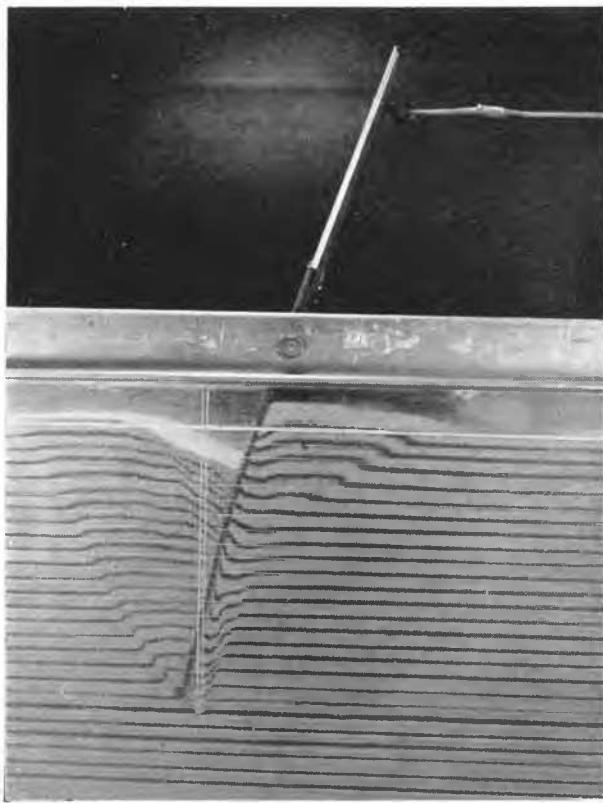


Fig. 63

M. A. R. JUMIKIS

On Dr J.B. Hansen's discussion on Jumikis' paper 3A/23 entitled "the Shape of Rupture Surface in Dry Sand"

The author's Figs. 64 and 65 show a comparison of theoretically calculated and experimentally observed spirals.

These figures indicate clearly that for all intents and purposes the experimentally obtained rupture surfaces agree practically very well with logarithmically spiralled surfaces. Besides, as the experiments show, the formation of the rupture surface is a *continuous process*, and so the rupture surface is also continuous. Introduction of compound rupture surfaces would mean mathematical and physical discontinuities at the points where the individual curves and/or tangents join each other. Such discontinuities were never observed in the experiments.

Besides, the application of a logarithmically spiralled rupture surface to stability calculations possesses a great advantage over the assumed compound rupture surfaces, namely, that in the case of the spiral it is not necessary to assume nor to be concerned as to how the reactions in soil are distributed along the rupture surfaces and what their magnitudes are. This is so because the logarithmic spiral possesses an important natural property, namely, all radius-vectors (viz. reactions) pass through the pole of the spiral (moment arm for reactions is zero). Hence all soil reaction moments are automatically excluded from our stability calculations by comparing active (driving) and reactive (resisting) moments. This makes the problem statically determinate.

Furthermore, the logarithmically spiralled rupture surface has the advantage in that the moment of the rupture surface wedge can be integrated by means of one single equation

between limits of amplitude ω from ω_1 to ω_2 , automatically taking care of the driving and resisting moments about the pole caused by the self-weight of the soil rupture wedge, giving a resultant moment [2, 3]. This is accomplished by the cosine function, $\cos \omega$, entering into the expression of the arm, which is $2/3 r \cos \omega$ (Fig. 66).

When ω varies between 0 and $\pi/2$, then the cosine varies from +1 to 0, and from $\omega = \pi/2$ to $\omega = \pi$ the cosine varies from 0 to -1. For example, if the plane of the horizontal ground surface passes through the pole, then the resultant moment $M\gamma$ of the self-weight of the ruptured soil wedge may be calculated as

$$M\gamma = (1/3) \gamma r_0^3 \int_0^\pi e^3 \omega \tan \varphi \cdot \cos \omega \cdot d\omega,$$

where γ = unit weight of soil

r_0 = reference radius-vector

$e = 2.718282\ldots$ = base of the natural logarithm system

ω = amplitude of any radius-vector

φ = angle of internal friction

$\tan \varphi$ = coefficient of internal friction.

Note that here the increasing radius-vector, from r_0 to r_π , sweeps the ruptured soil wedge area, viz., weight of wedge, in the counterclockwise direction from $\omega = 0$ to $\omega = \pi$.

In the photos, the left end of the spiral underneath the foundation model is somewhat depressed, indeed, thus deviating at that localized region somewhat from the spiral. However, this deviation is an experimental "after-effect". This is because of the inertia of the model and load which continue to displace in the horizontal direction upon shearing off of the spiralled soil wedge from the rest of the soil mass, and from settlement due to compression.

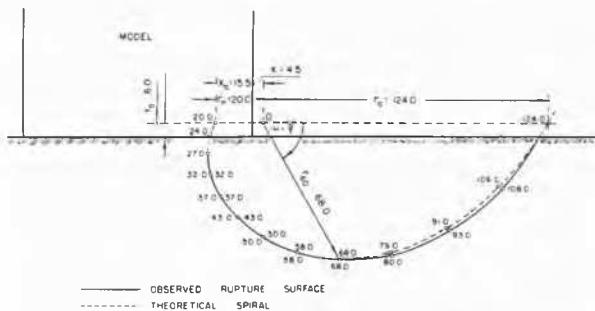


Fig. 64 Comparison of theoretical and observed spirals. Oblique loading. Contact pressure = $1.0'/ft^2$; $h = 0$. All measurements in millimeters.

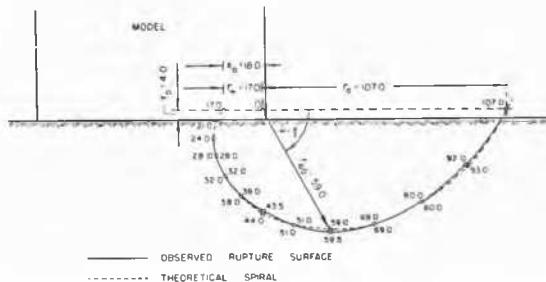


Fig. 65 Comparison of theoretical and observed spirals. Oblique loading. Contact pressure = $0.75'/ft^2$; $h = 4'$. All measurements in millimeters.

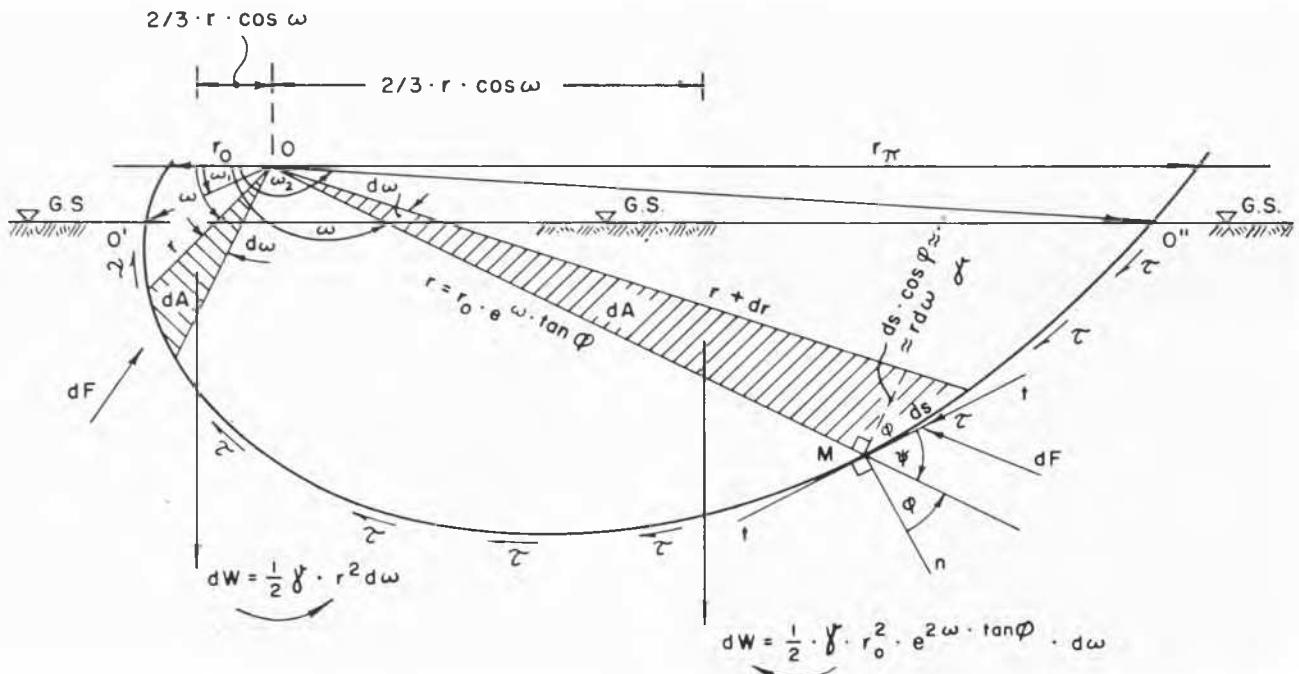


Fig. 66 Driving and resisting moments of weight of a spiraled soil wedge.

Likewise, the almost vertical shear line in sand connecting the right-hand edge of the model and the spiral is an after-effect : upon shearing off the soil wedge, the model tilts about its front edge, thus cutting into the sand and creating a secondary or after-shear plane after the spiral has already been formed. The author's film, which was shown at the Conference on July 21, 1961, brings this out clearly. Therefore it was thought that it is justified to consider the whole rupture surface as a *continuous spiral*.

If this is a small approximation, then it should be at least as good as the approximate condition with which Dr Hansen likes to work. It is, of course, Dr Hansen's privilege to have his own opinion.

The many advantages of the continuous logarithmic spiral, however, are obvious, and should not be underestimated.

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- [3] — (1960). "Some Properties of a Logarithmic Spiral; Rutgers. The State University, New Brunswick, New Jersey, p. 20.

The author has been engaged in these studies since 1936. The experimental studies were made by the author in the Soil Mechanics Laboratory at the Technische Hochschule Wien, in Austria, under Dr O.K. Fröhlich in 1943, and are now continued by the author at the Bureau of Engineering Research, College of Engineering, Rutgers. The State University, New Brunswick, New Jersey, U.S.A.

MM. A. LAZARD et G. GALLERAND (France)

Dans une communication présentée au 4^e Congrès de Mécanique des Sols et des Fondations à Londres, en Août

1959 [1] il a été proposé 2 formules (A) et (B) permettant de prévoir avec une approximation suffisante la valeur du moment limite de renversement d'une fondation isolée.

La formule (A), obtenue en corrigeant une formule proposée en 1950 par Ramelot et Vandepierre [2] après des essais en laboratoire sur des fondations à petite échelle, est une formule empirique mise au point après plus de 200 essais de fondations normales le long des voies des Chemins de fer français dans des terrains divers mais de nature argileuse dans les 2/3 environ des cas.

La formule (B), bien connue des Electriciens, suppose que toutes les contraintes sont horizontales, réparties linéairement, et que le maximum en terrain plat est voisin de la valeur de 6 kg/cm^2 (85 psi).

Les essais ont montré que le phénomène limite est essentiellement un phénomène de butée (ou poussée passive) s'exerçant sur la partie supérieure de la face avant et sur la partie inférieure de la face arrière de la fondation.

Nous avons voulu voir dans quelles proportions on augmente la stabilité et la valeur du moment limite en disposant transversalement à l'effort de tirage des traverses horizontales enterrées, solidaires de la fondation et disposées l'une en tête et l'autre au pied (Fig. 67). Pratiquement, la mise en place de la traverse de pied étant impossible, on s'est limité à la traverse de tête.

Les premiers essais ont eu lieu à Villemomble (près de Paris) en mai 1960. Le terrain, en déblai, se compose d'une couche de vieux remblai de 0,50 m d'épaisseur sur du sable fortement argileux.

Les deux massifs de fondation étaient en béton. La traverse était saillante pour l'un (Fig. 68) et incorporée pour l'autre (Fig. 69).

Les résultats des essais ont été bons : par rapport à une fondation classique de même base et de même hauteur, l'augmentation du moment limite de renversement a été de 65 pour cent et de 42 pour cent.

La Fig. 70 donne la comparaison des résultats des essais avec les valeurs calculées par la formule (A) appliquée au parallépipède circonscrit et avec $K = 1,5$ et la formule

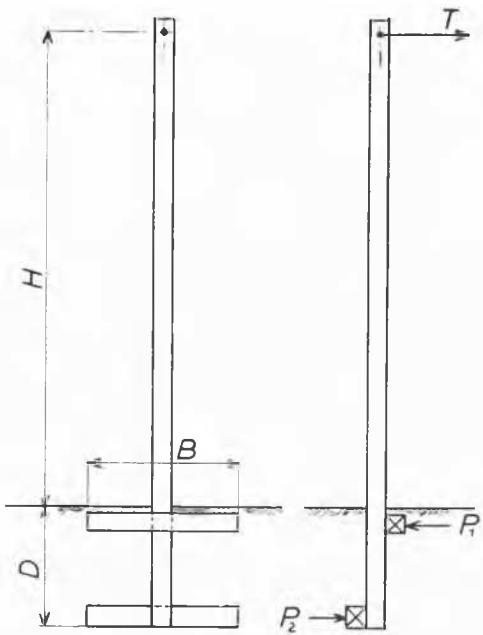
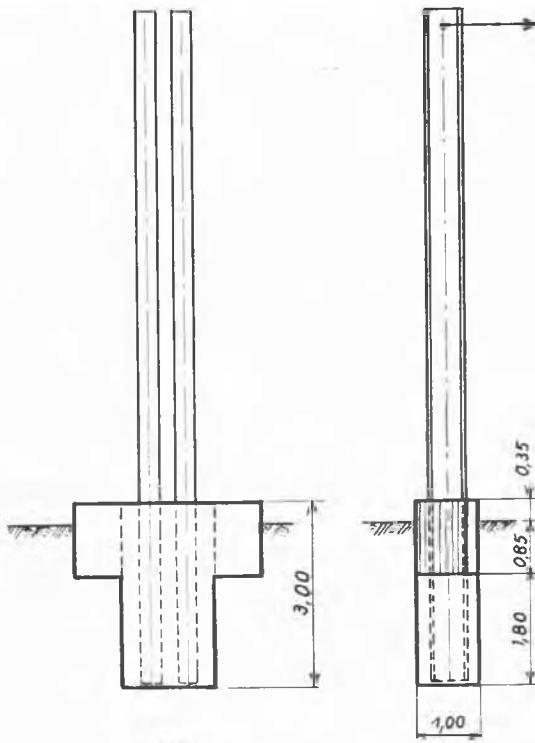
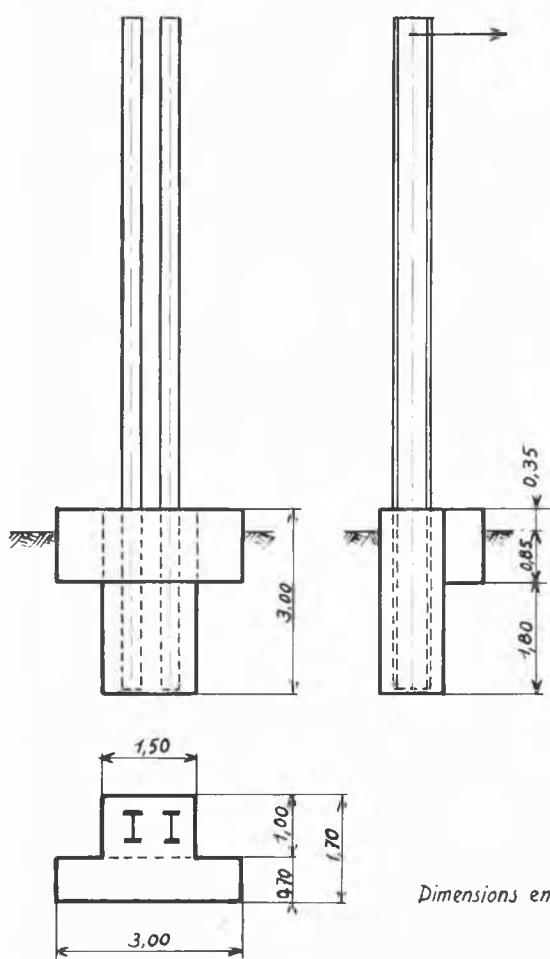


Fig. 67



Dimensions en m.



Dimensions en m.

Fig. 68

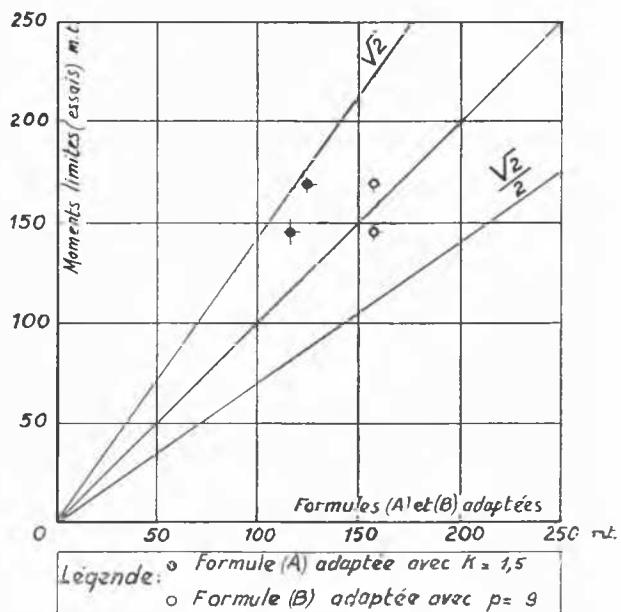


Fig. 70

(B) adaptée¹, et avec une pression limite $p = 9$. La concordance est bonne.

Les essais de Villemomble ayant été favorables, nous avons envisagé d'en faire d'autres dans la terre brute, sans fondation d'aucune sorte.

Deux terrains de nature différente ont été choisis :

— L'un, à Massy-Palaiseau (près de Paris), se compose de terre argileuse mélangée de sable sur 1 mètre, puis de terre végétale lourde fortement argileuse. La surface, à l'emplacement des essais, était garnie de très nombreuses racines.

— L'autre, à Villeneuve-Triage (près de Paris) est un remblai tout-venant de 2 mètres d'épaisseur, datant de 1945, sur un sable argileux humide.

Les poteaux d'essais étaient des poutrelles HN de 300 et 320 mm. Quatre ont été plantées dans le sol à 1 mètre de profondeur, quatre à 1,5 mètres et dix à 2 mètres.

A Massy (essais de mars 1961) les poutrelles ont été enfoncées au moyen d'un mouton auto-moteur. Le procédé s'est révélé très lent.

A Villeneuve (essais de juillet 1961) elles ont été descendues dans un trou de 0,45 m de diamètre, foré mécaniquement, puis comblé avec de la terre fortement damée.

Une poutrelle sur deux s'appuyait sur une traverse de 2,5 m de longueur, en HN de 320 mm, enterrée sous 0,15 m de terre (Fig. 71) l'autre poutrelle restant nue.

Les poutrelles avec traverse n'ont pas donné le gain escompté (Tableau I).

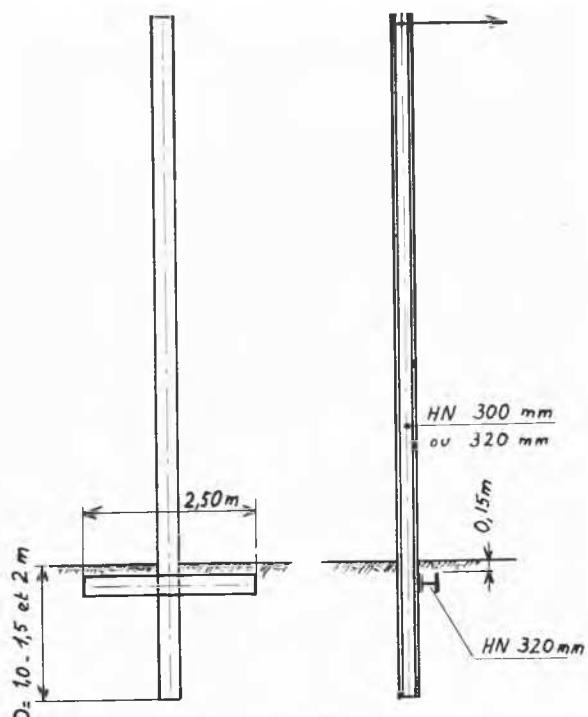


Fig. 71

Tableau 1

Profondeur de fiche D - (m)	Sans traverse				Avec traverse			
	Emplacement	Numéro	Moment limite (mt)	α limite	Emplacement	Numéro	Moment limite (mt)	α limite
1,00	Massy	1	2,4	4°36'	Massy	2	4,6	11°38'
	Massy	3	3,1	4°13'	Massy	4	3,6	5°14'
1,50	Massy	5	12,2	20°24'	Massy	6	13,5	12°6'
	Massy	7	> 11,8	> 11°30'	Massy	8	12,6	7°7'
2,00	Massy	9	23,4	31°10'	Massy	0	23,7	12°43'
	Massy	10	23,0	38°46'	Massy	11	27,2	8°8'
	Villeneuve	1	26,0	32°30'	Massy	12	22,4	18°40'
	Villeneuve	2	21,0	24°55'	Villeneuve	3	26,7	8°16'
	Villeneuve	4	25,6	31°44'	Villeneuve	5	22,0	7°35'

Nous citerons également, deux essais analogues de l'Électricité de France. Les poteaux étaient fichés à 1,80 m et butés sur des traverses identiques aux nôtres. Les moments limites obtenus ont été de 20,6 et de 29,5 mt, cette dernière valeur étant remarquable.

Dans tous les essais avec traverse nous avons constaté l'effet « cuiller », très rare en terrain plat pour les massifs en béton.

Les moments limites calculés par la formule (B) adaptée sont nettement au-dessous des résultats d'essais (Fig. 72 et 73). Les pressions maximales moyennes p_2 correspondant à ces derniers sont de :

15 kg/cm² pour les poutrelles nues;

10 kg/cm² pour les poutrelles avec traverse, de la SNCF;

12 et 17 kg/cm² pour les poutrelles avec traverse, de l'EDF.

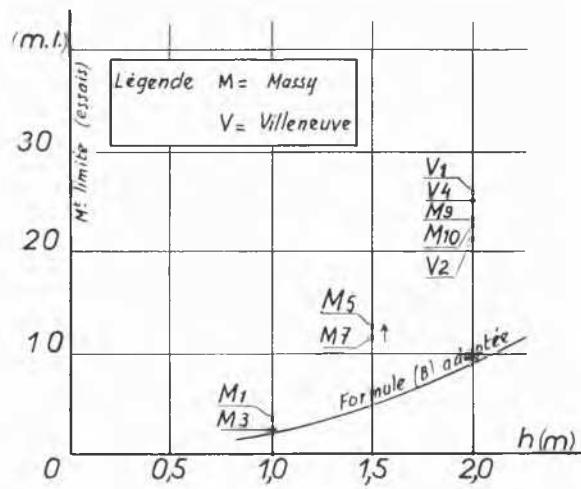


Fig. 72

1. On a continué à supposer une répartition linéaire des contraintes. Il a fallu déterminer la position de la fibre neutre et les modules de résistance des parties haute et basse (selon un calcul classique).

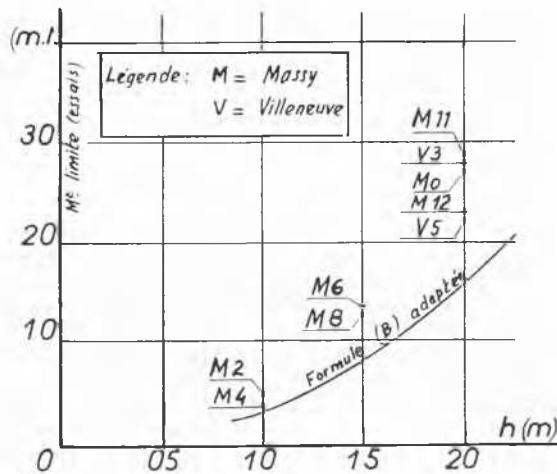


Fig. 73

Comparées aux 6 kg/cm^2 pour les massifs classiques en béton, en terrain plat, ces valeurs élevées peuvent surprendre. Elles s'expliquent sans doute par la faiblesse de b pour les

poutrelles (Fig. 74) : il est probable que les efforts ne sont plus proportionnels à b du fait de l'action dans les angles β .

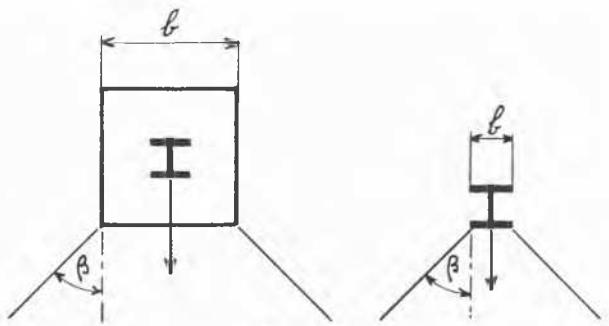


Fig. 74

La figure 75 donne des courbes : effort de tirage (T) en tonnes-angle de rotation (α) pour des poutrelles avec et sans traverse. On remarquera que, pendant une notable partie de l'essai, la poutrelle avec traverse se déverse beaucoup moins rapidement. Cette indication pourra être utile aux Administrations qui s'imposent, en service normal, un angle limite de rotation.

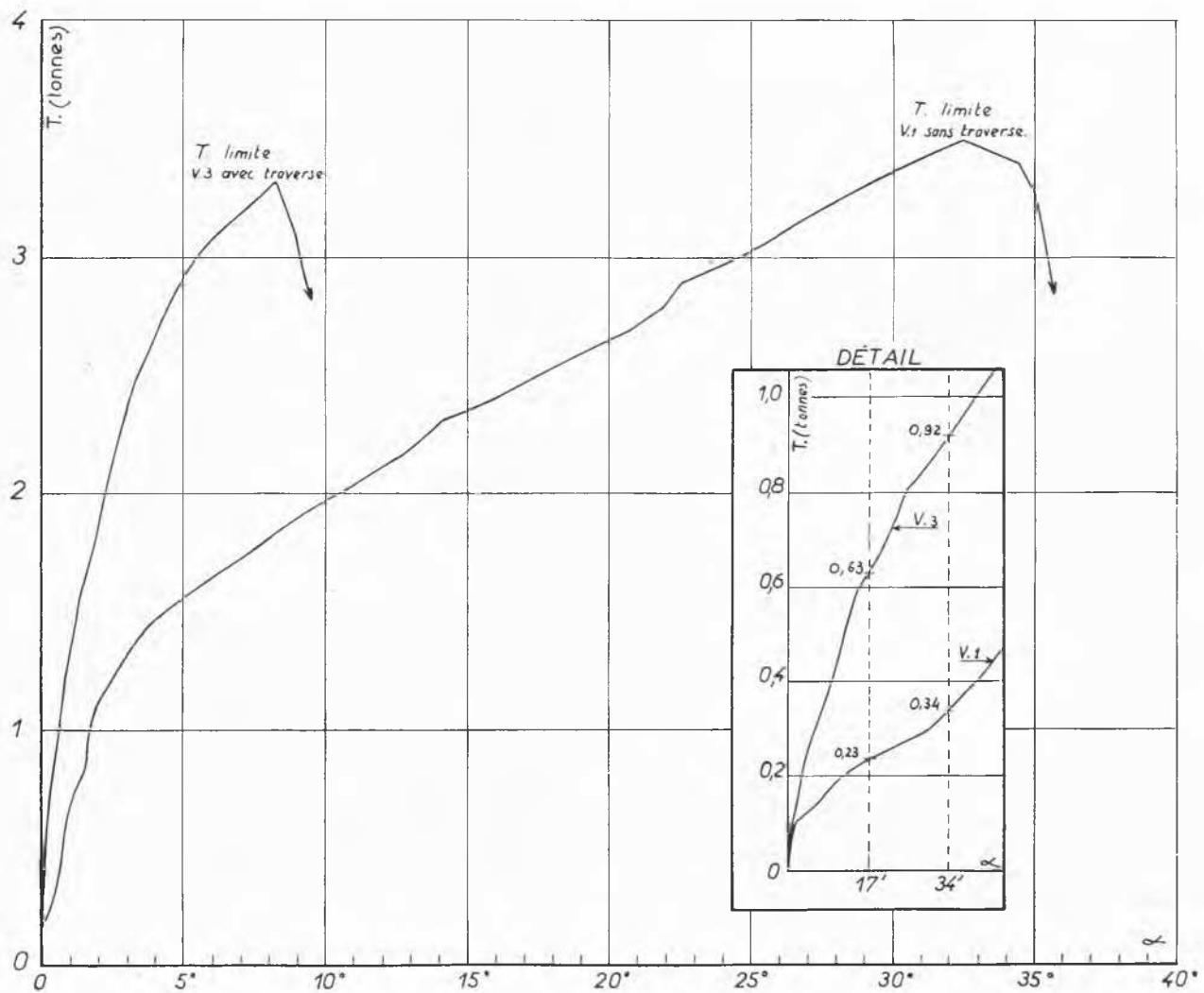


Fig. 75

Il convient de mentionner deux essais d'E.D.F. avec des fondations économiques où la plus grande partie de la fondation était constituée par des pierres de blocage coincées entre deux galettes de béton (Fig. 76).

Les essais ont été interrompus du fait de l'insuffisance des poutrelles, alors que le moment de renversement déjà élevé (37 mt dans les 2 cas) n'avait provoqué qu'un faible pivotement des massifs (déplacements angulaires de 22' et 29'). Ceci souligne l'excellente tenue de ce type de fondation, susceptible d'intéresser les constructeurs de lignes d'Electricité.

Pour terminer, nous mentionnerons la thèse de R. Baus sur la contribution au calcul à la rupture de poutres en béton armé et en béton précontraint [3].

Examinant plus de 2 000 essais effectués dans le monde entier sur des poutres de laboratoire très soignées et se don-

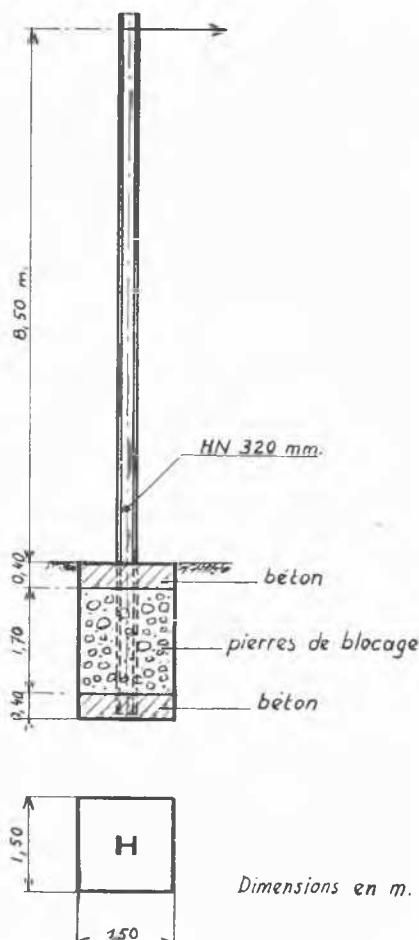


Fig. 76

nant un intervalle de confiance à 95 pour cent qui correspond pratiquement à prendre 2 écarts quadratiques de chaque côté de la médiane, l'auteur obtient un excellent groupement des points expérimentaux, entre environ 139 pour cent et 79 pour cent de la formule moyenne proposée.

On rapprochera ces valeurs des limites $\sqrt{2}$ ou 141 pour cent et $\frac{\sqrt{2}}{2}$ ou 71 pour cent que nous nous étions fixées à priori pour la détermination des formules (A) et (B).

On ne pourra manquer d'être surpris en constatant que les terrains français essayés, s'étendant des argiles de toutes sortes aux terrains graveleux, n'ont pas plus de dispersion que des poutres de laboratoire, en béton armé d'exécution particulièrement soignée (écart quadratique relatif voisin de 18 à 20 pour cent).

Références

- [1] LAZARD, A. (1957). *4^e Congrès de Mécanique des sols et des fondations*, Londres. Communication 3A/19; pp. 349-354.
- [2] RAMELLOT, Ch. et VANDEPERRE, L. « Les fondations de pylônes électriques : leur résistance au renversement, leur stabilité, leur calcul, étude expérimentale ». *Comptes rendus de recherches de l'I.R.S.I.A.*
- [3] BAUS, R. (1961). *Contribution au calcul à la rupture du béton armé*. Thèse défendue devant la Faculté des Sciences appliquées de l'Université de Liège.

M. D.M. MILOVIC

Concernant la Contribution de MM. W.G. Holtz et J.W. Hilf

J'ai lu avec grand intérêt une communication de MM. W.G. Holtz et J.W. Hilf (3A/20) sur le problème du tassement des fondations des sols de densité peu élevée.

Je voudrais faire quelques comparaisons entre les résultats présentés par MM. Holtz et Hilf et ceux que j'ai obtenus pour le loess en Serbie.

Par l'emploi de rayons X il était prouvé que le loess de Serbie contient une quantité considérable de montmorillonite. Par ailleurs, les courbes granulométriques aussi bien que les données de plasticité correspondant avec celles présentées par MM. Holtz et Hilf, il en résulte que le loess que j'ai examiné pourrait être classé comme le sol dénommé « A » dans la communication de ces auteurs.

L'examen de plus de 600 échantillons non remaniés du loess en Serbie, a permis d'établir que la densité γ_d et la teneur en eau W sont les paramètres les plus importants qui influent les caractéristiques du cisaillement et de consolidation du loess.

Pour évaluer l'influence de chaque paramètre séparément sur les caractéristiques mentionnées, des investigations systématiques étaient effectuées sur les échantillons non remaniés du loess. Les échantillons de la première série étaient de densité $\gamma_d = 1,25 - 1,27 \text{ g/cm}^3$, pour la deuxième $\gamma_d = 1,36 - 1,38$, pour la troisième $\gamma_d = 1,42 - 1,44 \text{ g/cm}^3$, pour la quatrième $\gamma_d = 1,48 - 1,50 \text{ g/cm}^3$ et pour la cinquième $\gamma_d = 1,53 - 1,55 \text{ g/cm}^3$.

Les échantillons de ces cinq séries étaient examinés avec des teneurs en eau différentes, variant de 10 à 28 pour cent.

Les résultats des essais de cisaillement des échantillons du loess non remaniés de la même densité initiale γ_d mais de teneurs en eau différentes sont présentés sur la Fig. 77. L'influence de la teneur en eau est évidente.

Sur la Fig. 78 sont donnés les résultats des essais de cisaillement pour une même teneur en eau W mais pour différentes valeurs de la densité.

La relation entre les valeurs de la cohésion C et de la densité γ_d , pour la même teneur en eau est donné sur la Fig. 79, tandis que la relation entre les valeurs de la cohésion et de la teneur en eau, pour une même densité, est présentée sur la Fig. 80.

Comme il a déjà été dit, les paramètres les plus importants sont la teneur en eau et la densité, ce qui est évident dans les résultats donnés.

L'importance de ces paramètres pour les caractéristiques de consolidation n'est pas moins remarquable.

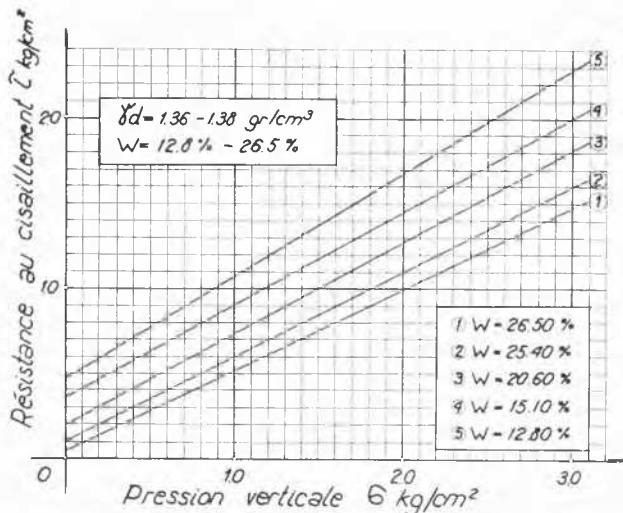


Fig. 77 Résultats de l'essai de cisaillement du loess de Serbie (échantillons de même densité).

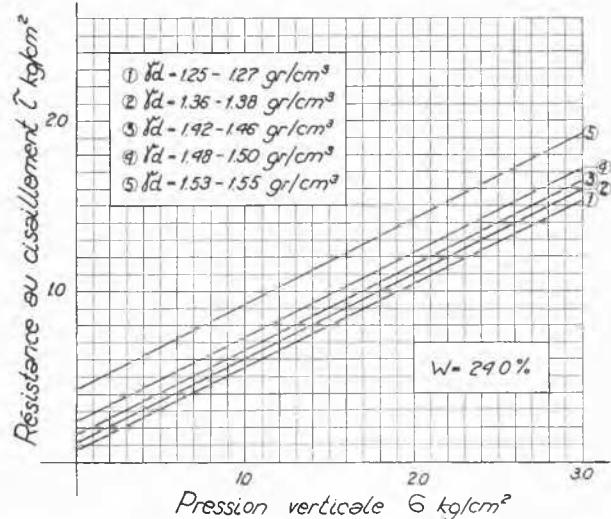


Fig. 78 Résultats de l'essai de cisaillement du loess de Serbie (échantillons avec la même teneur en eau).

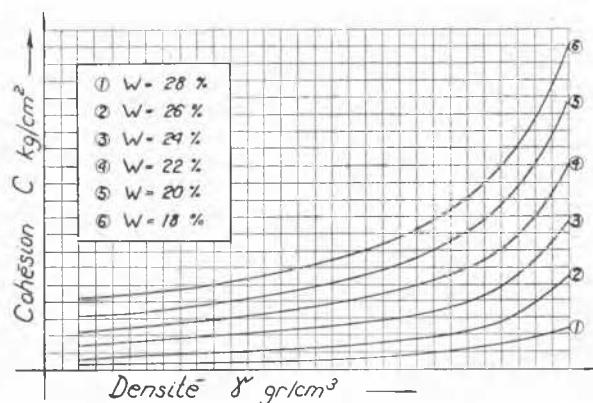


Fig. 79 Variation de cohésion en fonction des densités (échantillons non remaniés).

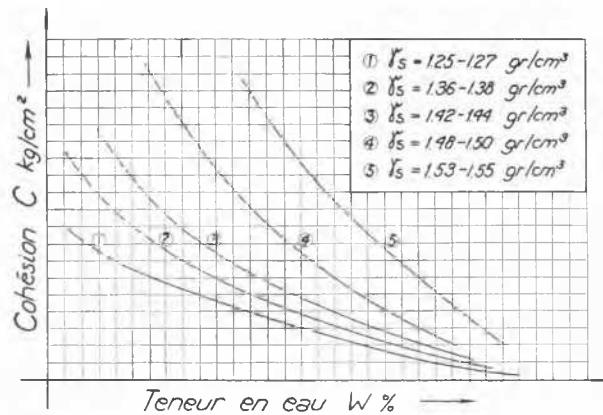


Fig. 80 Variation de cohésion en fonction de la teneur en eau (échantillons non remaniés).

Pour toutes les cinq séries des échantillons non remaniés du loess de densité $\gamma_d = 1,25 - 1,55 \text{ g/cm}^3$, les essais de consolidations étaient effectués avec des teneurs en eau différentes.

Sur la Fig. 81 sont présentés les résultats des essais de consolidation du loess les plus caractéristiques.

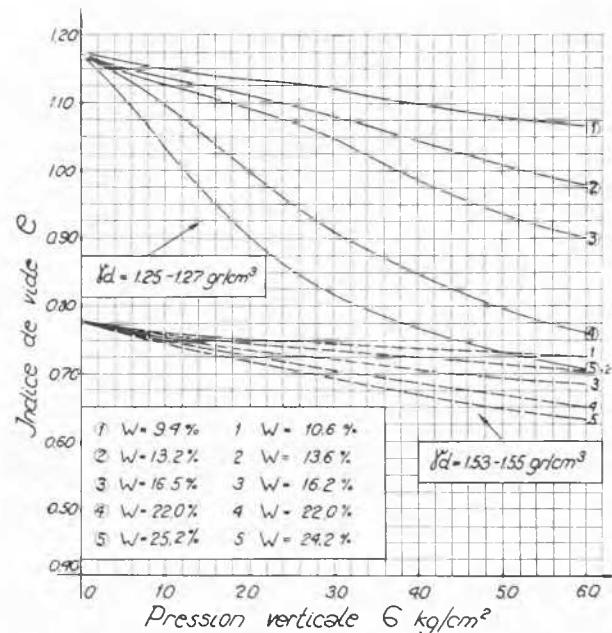


Fig. 81 Résultats d'essai de consolidation du loess de Serbie.

L'influence de l'augmentation de la teneur en eau sur la grandeur de la déformation est clairement montrée par les courbes de consolidation. En même temps on peut constater l'importance du deuxième paramètre, c'est-à-dire de la densité initiale, de laquelle dépend l'influence de l'augmentation de la teneur en eau sur l'augmentation de la déformation.

Il est très intéressant de souligner une très bonne concordance de ces résultats avec ceux présentés dans la communication de MM. Holtz et Hilf.

A mon avis, les critères du tastement pour le loess qui tiennent compte simultanément de la densité et de la teneur en eau, particulièrement en tenant compte du tastement potentiel, seront ceux correspondant le mieux à la réalité.

Intervention concernant les Communications de MM. A. Beles et I. Stanculesco (vol. I 3A/5) et de M. G. Stefanoff (vol. II, 7/7)

Une question parmi les plus intéressantes quand il s'agit des problèmes de fondation sur loess est sans doute la sensibilité du loess sous l'action de l'eau et la détermination de la grandeur du tassement supplémentaire dû à l'augmentation de la teneur en eau.

Très souvent cette sensibilité est caractérisée par le tassement spécifique sous une pression fixée à 3 kg/cm^2 (vol. I, 3A/5, et vol. II, 7/7).

Cependant, à la base des nombreux essais on peut constater avec certitude que le tassement supplémentaire dû à la saturation dépend de la densité et de la teneur en eau. La sensibilité est d'autant plus grande que la densité et la teneur en eau initiale sont plus petites.

En même temps, on peut dire que les valeurs du tassement supplémentaire augmentent avec le degré de saturation et la pression verticale.

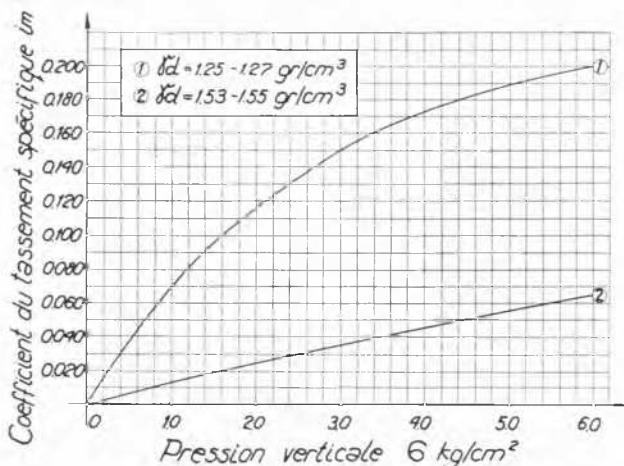


Fig. 82

Sur la Fig. 82 sont représentées deux courbes les plus caractéristiques du coefficient du tassement spécifique

$$i_m = \frac{\Delta e_m}{1 + e_m}$$

l'une pour les échantillons non remaniés de

densité très peu élevée $\gamma_d = 1,25 - 1,27 \text{ g/cm}^3$ et l'autre pour ceux de densité assez élevée $\gamma_d = 1,53 - 1,55 \text{ g/cm}^3$.

Dans ce cas présenté ci-dessus la teneur en eau de 5 pour cent est augmentée jusqu'à 25 pour cent. Les courbes montrent clairement la corrélation entre le tassement supplémentaire et la densité.

M. E. SCHULTZE (Allemagne)

In the General Report, paragraph 2 regarding 3A/41, it is said that the introduction of the safety factor into the calculations of edge pressures (as is done in the paper) seems to be unjustified because on the one hand this factor increases the divergence between the experimental and theoretical data, and on the other hand it is not generally included in the safety margin when calculating foundation strength.

There must have been a misunderstanding. No safety factor has been introduced into the calculation, but it has been based on the fact that actually foundations are not stressed by their failure load but only by a fraction of it,

estimated at 1/2 to 1/3. In case one wants to compare the observed contact pressure distribution with the theory, one has to take from Fig. 2 a pressure distribution regarding this circumstance. These are the distributions in Fig. 2 d or e. Indeed, they correspond quite well with measurements of structures; the distributions of Fig. 2 a do not match (complete efficiency of the soil failure load = safety). As the difference of the distributions for a safety of 2 or 3 is only small the exact efficiency determination of the maximum load is not necessary. Naturally this value of the different measured foundations varies.

The notice, that the previous results contain too high safety factors, since they neglect problems associated with the extention of plastic zones down to a certain depth and the origination of an elastic soil wedge under the foundation base, cannot be understood without any explanation. Actually a compressible wedge under the foundation does not change the soil failure load as long as this wedge does not differ too much from the Prandtl-Buisman plastic triangle under the foundation. There exists no reason for the above mentioned.

M. H. U. SMOLTCZYK (Allemagne)

The General Reporter, Mr Tsitovitch, has missed arguments in my paper 3A/43 for my approach to the problem of the bearing capacity of shallow foundations on sand. I therefore would like to apologize by referring him to the fact that it was necessary to keep to the limit ruled, two pages only : there was little space for arguing.

But, following on the General Reporter's proposals for discussion, I will give some comments which, as well, might explain the necessity and basic idea for an additional approach to the problem.

When I started to deal with the problem of bearing capacity I naturally found myself quite fascinated by the possibilities of the theory of plastic rupture. All those solutions which were and are being developed presuppose the existence of rupture lines which surround the area of unrestricted plastic deformation. However, they do not at all cover the wide range of possible states of deformation where no rupture line yet exists, but where that amount of plastic deformation that in each stage of deformation process is interconnected with elastic strain, far exceeds the admissible limit. So, a basic question arises : how can we transform the deformation experience which we state in any soil test to practical states of stress in the field ? Everybody quite agrees about the necessity of calculating settlements but very little is done to calculate the displacement of soil following shear stresses. To deal with this, the following points seem to me to be essential :

A body of soil which comes to a state of balance after some movements of its particles is, from the mechanical point of view, just as much an elastic body as any other material is. It is not necessary to consider an area of soil in which, prior to failure, some remarkable deformations occur, as being something mysterious between elasticity and plasticity. Maybe it is necessary to have more than one cycle of loading and unloading until the additional amount of deformation per cycle is small enough to take the state of the body balanced. In some cases we do not come to any balance at all. However, if such a state of balance is possible, the body has got an elastic potential and may be analysed by means of the theory of elasticity. This can be proved by a simple experiment.

On the surface of a soil body a vibration is excited (left side). Two pick-ups measure the amplitude. Between them, we put a loading plate. As soon as we load this plate considerably, we can observe a total extinction of the amplitude to the right. This means that the elastic energy which is led off in the soil cannot pass the area where there are plastic

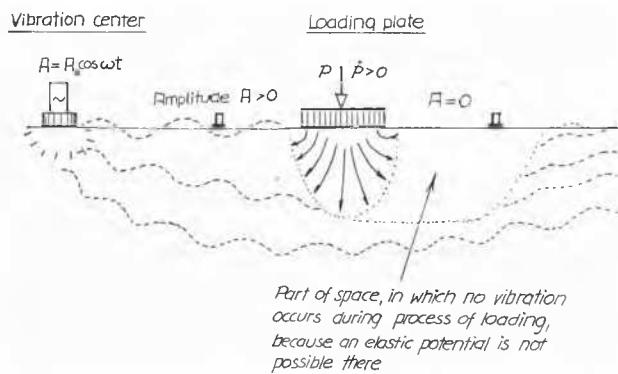


Fig. 83

deformations, because there is no state of balance and, subsequently, no elastic potential existent. When the loading plate stops its settlement, the amplitude gradually increases again. The left amplitude does not show any variation. This phenomenon is a very informative example for the mechanism of elasticity in a soil body.

Trying to treat the soil body by means of the theory of elasticity, it is not possible to take into consideration the limited deviatoric plasticity or displacement of soil just by using two different moduli of elasticity, i.e. a modulus for loading and another one for unloading. This can be done only where the state of stress is almost constant, as in a triaxial test for instance. In a general state of stress, changing from point to point, this would be possible only if the modulus were constant. We could then generally say that a certain percentage of the total deformation would always be of a permanent character. But the nearer we approach the state of failure the less constant is this modulus. So, it is conclusive that this problem cannot be separated from the other one, to introduce into our calculations in certain cases a modulus that is a function of stress. I should be very interested indeed to learn about possibilities of dealing with this problem in any way of reasonable approximation that makes one independant of the absolutely insufficient method of considering soil a Hookian body.

MM. N. M. SOKOLOV et E. A. SOROCHEAN (U.R.S.S.)

The principal direction in construction in the U.S.S.R. is the application of details and structures of buildings previously manufactured at factories. The precast elements are also used for erecting foundations — one of the most responsible and laborious parts of the building.

The working out of the problem of application of precast foundations in the Soviet Union was started as long ago as 1925. However, mass application of such foundations has been widely developed in recent years. For example, it is sufficient to note that over 90 per cent of dwellings being built at present in Moscow and Leningrad have precast foundations.

The previously existing point of view that in case of replacement of "strong" monolithic foundations by "cut" precast ones, deformations would inevitably appear in the building, was based on inaccurate ideas on the work of the structure. It was not taken into account that the forces arising during unequal settlement of the building are resisted not only by the foundations, but also by all other building structures, in direct ratio with their rigidity.

Besides, investigations have determined that changing of rigidity of the foundation has practically no influence on the total rigidity of the building.

However, this does not mean that precast foundations may be used everywhere. For highly compressible macroporous, settling, and expansive soils only such foundation structures are allowable which have sufficient strength and rigidity.

Depending on the sequence of arrangement below the structure, precast foundations are divided into three groups — strip, interrupted, and post.

Strip foundations are erected of solid reinforced concrete cushion blocks placed in a solid strip (Fig. 84 a) and of solid concrete or reinforced concrete hollow wall blocks (Fig. 84 b), placed on cushion-blocks in several rows by height "large-block" or in one row by height of the basement wall "panel".

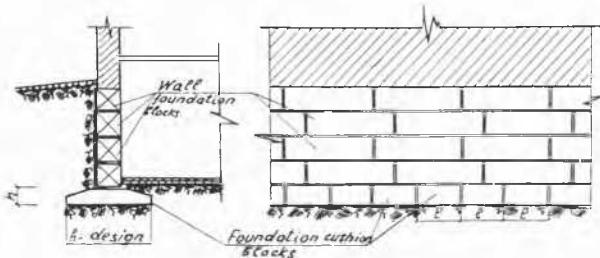


Fig. 84 a Precast strip foundations with solid concrete wall blocks.

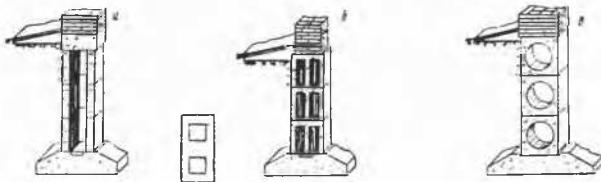


Fig. 84 b Precast strip foundations with hollow reinforced concrete wall blocks.

In the latter case, the foundation is separated by through vertical joints and, therefore, has no strength and rigidity longitudinally. The application of such foundations is allowable only on soils of low compressibility. The installation of such foundations becomes more complicated at different depths, as a larger number of standard sizes of panels is required.

For foundations of wall blocks of low height, these disadvantages fall away. Due to the possibility of placing blocks with jointing of the foundation, the latter may resist longitudinal tensile stresses arising during unequal settlement of the building.

For a depth of vertical jointing of the foundation wall blocks

$$t \geq \frac{m \cdot h \cdot R_p}{n \cdot T}$$

where : h = height of blocks, cm;

m = number of blocks, where destruction may appear;

n = number of rows of blocks;

T = shear resistance of mortar in joint, kg/cm²;

R_p = ultimate strength of block material, kg/cm²
[tensile resistance of large-block foundations (strength) will be for all other equal conditions higher than for monolithic rubble concrete].

Precast foundations are expediently made of a limited number of standard members, as this simplifies the technology of their manufacture and assembly.

However, this consideration leads to the fact that, in most cases, the design width of the foundation does not coincide with the width of the standard blocks and the foundation is usually made wider not using entirely the bearing capacity of the foundation soil.

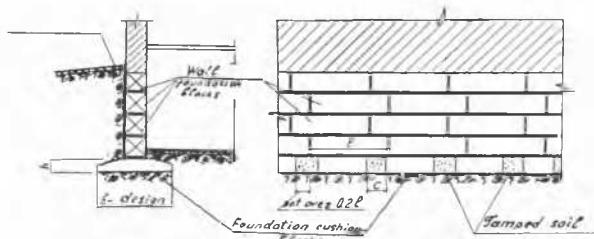


Fig. 85 Precast interrupted foundations.

Interrupted foundations are erected by placing cushion-blocks at some distance from each other (Fig. 85). In this case, it is possible to choose such a distance between the blocks (C), so that the pressure at the toe is equal to the design resistance of the foundation soil.

$$C = \left(\frac{l_{\min}}{l_{\text{des}}} - 1 \right) l$$

where : l_{\min} = width of standard cushion-block;

l_{des} = design width of foundation;

l = length of standard cushion-block.

Our investigations have determined that a strip foundation may be replaced by an interrupted foundation equivalent in relation to settlement by shifting apart the cushion-blocks for the required distance. As an example, Fig. 86 shows

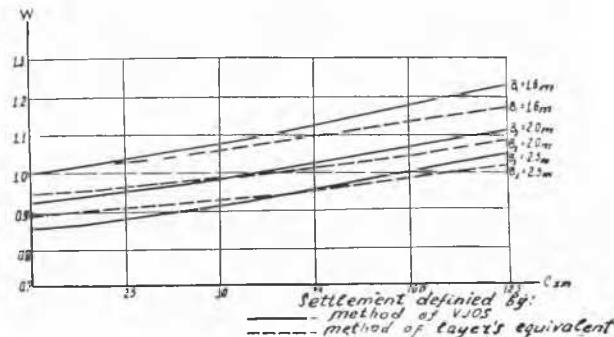


Fig. 86 a Function of reduced settlement of interrupted foundations of a width of 1.6 m, 2.0 m and 2.3 m, 1.6 m long on the distance between the cushion-blocks.

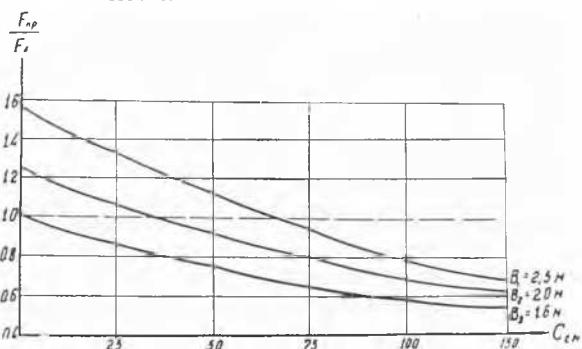


Fig. 86 b Function of supporting area of interrupted foundation on the distance between the cushion-blocks.

the function of settlement W (here the reduced settlement is the ratio of the settlement of the interrupted foundation to the settlement of the strip foundation) on distance C between the cushion-blocks for an interrupted foundation of a width of $b_1 = 1.6$ m; $b_2 = 2.5$ m replacing a strip foundation of a width of $b_1 = 1.6$ m.

The curve shows that such a strip foundation may be replaced by one with an equivalent settlement of the interrupted type (i.e. $W = 1.0$ m) of a width of $b_2 = 2$ m with the distance between the blocks $C_1 = 0.7$ m or by a foundation of a width $b_3 = 2.5$ m for $C = 1$ m.

On the contrary, it is possible to replace strip foundations of varying width by one interrupted foundation of greater width, but with different distances between the cushion-blocks. Such replacement is of great practical importance, as it allows limiting of the precast foundations with a small number of standard size cushion-blocks.

Let us consider how the supporting area changes when replacing continuous foundations by equivalent interrupted ones in relation to the value of settlement. For the above-mentioned case, a curve has been plotted (Fig. 86 b) with the ordinate axis showing the ratio of the supporting area of interrupted foundation to the area of the strip one $\frac{F_{\text{int}}}{F_{\text{con}}}$,

while the abscissa shows distance C between the blocks of the interrupted foundation. The curve shows that settlement of the interrupted foundation increases not in direct ratio with changing of the supporting area, but in a lower ratio, this causing economy when replacing strip foundations by interrupted ones. Thus, the area of an interrupted foundation 2 m width for $C = 0.7$ m equals 0.85 of the area taken up by the continuous foundation, 1.6 m width.

The distance between the cushion-blocks of the interrupted foundation equivalent, in relation to settlement with the strip one, is determined by the following formula :

$$C = l \left(\frac{W_l L_{bl}}{W_{hl} k_h} - 1 \right)$$

where : l = length of cushion-block;

w_l = coefficient depending on ratio of sides of strip foundation;

w_{bl} = coefficient depending on ratio of sides of cushion-block;

L_{bl} = ratio of sides of cushion-block;

k = coefficient taking into account the mutual influence of cushion-blocks in the interrupted foundation.

For simplifying calculations for standard cushion-blocks, auxiliary tables have been composed.

3. Post foundations consist of separate supporting columns and members of basement wall members in the form of thin slabs. In this case, the strength of the members is used to a higher extent. However, the large number of standard sizes of members, difficulties during assembly, lack of rigidity longitudinally, are still highly disadvantageous for post foundations.

When precast foundations are employed, the construction delay is lowered, as well as labour expenditure and expenses, while the labour of the workers is facilitated.

M. R. TOKAR (U.R.S.S.)

Dans ce complément au Rapport général, présenté par M. Tsitovitch, je voudrais attirer attention des membres du Congrès sur la dynamique des sols.

L'évolution de la théorie de la dynamique des sols est liée aux recherches et aux solutions apportées dans les deux problèmes principaux :

(a) La lutte contre les vibrations du sol.

(b) L'utilisation de ces vibrations pour les travaux de terrassements, de fondations et autres.

Les recherches effectuées dans différents pays et, à une échelle plus élevée, en U.R.S.S. (les plus nombreuses se rapportent à la période d'avant guerre) ont permis de bâtir une théorie approchée sur les oscillations des socles des installations mécaniques. Cependant, certaines parties de cette théorie ne sont pas suffisamment étudiées actuellement; notamment, l'influence de l'inertie du sol et de ses propriétés de dissipation. Cette première question a fait l'objet de nombreuses études dans différents pays. Ces études ont donné naissance à des schémas de calculs qui tiennent compte de l'influence de l'inertie du sol, mais il n'existe pas jusqu'à présent d'exemple d'essai qui soutiendrait avec sûreté un des schémas proposés. La même question fait l'objet, entre autres, du compte rendu présenté au Congrès par M. H.A. Balakrishna Rao (Inde). Le Rapporteur propose d'introduire un terme d'addition supplémentaire dans la masse de fondation pour évaluer l'influence de l'inertie du sol sur la fréquence des propres oscillations du socle. L'étude ultérieure de l'influence de l'inertie du sol non seulement sur ses propres fréquences mais aussi sur les amplitudes des oscillations forcées ou libres, exigerait des essais sur de grands modèles de plusieurs mètres carrés de surface et pesant des dizaines de tonnes. Ces essais doivent être réalisés sur un terrain et sur des sols dont les qualités dynamiques sont suffisamment bien étudiées. En outre il serait indispensable d'effectuer ultérieurement une étude approfondie de l'influence de différents facteurs sur l'inertie du sol (dimensions, forme, poids de fondation, propriétés du sol, fréquences et amplitudes des oscillations).

Une attention particulière doit être apportée à l'étude des propriétés de dissipation des sols qui déterminent les amortissements des amplitudes des oscillations forcées ou libres appartenant à la zone de résonnance, ainsi que l'absorption de l'énergie par le sol lors de la propagation des ondes dans le sol.

Il faut noter à ce sujet que la mise au point d'une théorie plus ou moins valable sur la propagation dans le sol des ondes provenant des installations industrielles et des moyens de transport se heurte à de grandes difficultés. Les recherches classiques dans le domaine de la propagation des ondes dans les corps solides conduisent à des résultats qui sont souvent en contradiction avec les résultats d'essais.

La solution de ce problème devrait donc suivre la voie d'exécution des schémas approximatifs de calcul, établis à partir de mesures suffisamment abondantes sur les ondes émises par des sources différentes dans les sols.

Cette méthode demande un appareillage spécial. Nous sommes actuellement en possession de dispositifs de vibrations de différentes provenances. Néanmoins, les appareils de prise de mesures décrits dans les Comptes-rendus du Congrès par MM. Bendel et D. Bovet (Suisse), (3A/6) méritent une grande attention de la part des spécialistes, et particulièrement l'appareil triaxial dynamique (stablolomètre) dont l'utilisation au laboratoire pourrait donner des résultats très intéressants.

Le Rapporteur donne des indications très intéressantes sur l'utilisation des appareils pour mesurer les vibrations et, en particulier, sur la propagation des ondes dans les sols, même gelés.

Le fait que deux communications seulement sur les vibrations des sols et fondations ont été présentées à la section

3 A du Congrès témoigne que ce problème n'a pas retenu suffisamment l'attention.

L'utilisation des vibrations pour les travaux de fondations et autres travaux de terrassement n'a pas cessé d'évoluer depuis les travaux du 4^e Congrès de Mécanique des Sols et des travaux de Fondations. Au cours de cette période les procédés utilisant les vibrations pour les travaux de terrassement se sont développés, non seulement dans l'Union Soviétique, mais encore dans nombre de pays; mais cette technique nouvelle a été principalement utilisée pour l'enfoncement et l'enlèvement des palplanches métalliques, le fonçage des puits en béton armé et pour le compactage des sols peu portants et de préférence sablonneux, en surface et en profondeur.

La pratique de ces travaux a amené la création de nouveaux types de vibrateurs.

Cependant, les travaux de recherches sur les procédés de vibrations manquent toujours d'envergure.

Dans son aperçu sur l'état de ce problème en U.R.S.S., M. O.A. Savinov montre que le domaine de l'utilisation des vibrations est loin d'être épousé par les procédés mentionnés ci-dessus.

Des recherches expérimentales (effectuées par excellence sur le chantier) ont prouvé que l'utilisation des vibrations permet d'obtenir des résultats techniquement et économiquement intéressants dans l'abaissement de la nappe d'eau, la pose de conduites sans creuser les tranchées, la destruction, la pulvérisation ou l'excavation de terres (même dures et gelées), l'exécution des fosses, la pose des dalles en béton armé pour les pistes d'envol, le refoulement du sol et autres travaux de terrassements.

L'utilisation à une grande échelle des vibrations pour divers travaux de terrassement exige des appareils de vibration sûrs et rentables. Les travaux effectués à ce jour, d'ailleurs nombreux, ne nous permettent pas d'affirmer que le problème est résolu complètement et dans tous les cas, ce qui est vrai surtout pour les vibrateurs et moutons-vibrants puissants et à grands moments d'excentricité et de fréquences.

L'étude des phénomènes résiduels dans le sol provoqués par de fortes vibrations, est d'une grande signification pour le développement des procédés utilisant ces vibrations. Ces phénomènes, de caractère évidemment plastique, entraînent la nécessité de développer la théorie dynamique de la plasticité ce qui veut dire la théorie des déformations plastiques résultant de l'action dynamique sur le sol. Certains aspects de cette théorie qui sont étudiés dans les travaux des savants soviétiques (Mme Bespalova, MM. Neumark, Kouchoul, Chekter, Barkan, Savinov, Golovatchev, etc.) et applicables à l'enfoncement dans le sol par choc ou vibration, ont permis d'établir et de mesurer l'influence de certains facteurs sur la pénétration dans le sol par vibration ou vibrochoc.

L'application pratique de cette théorie exige que soient définies certaines caractéristiques du sol à savoir : limite dynamique de plasticité, module d'élasticité dynamique, coefficient de consolidation par vibration, limites de rupture et coefficients de frottement sous vibration intense, etc. Aujourd'hui encore on n'analyse pas totalement les propriétés des sols ce qui freine non seulement le développement de la théorie des procédés vibratoires mais aussi l'application même de ces procédés.

Compte tenu du caractère particulier du problème de la dynamique des sols, un symposium spécial pour une large discussion des problèmes en question me paraît bien souhaitable.