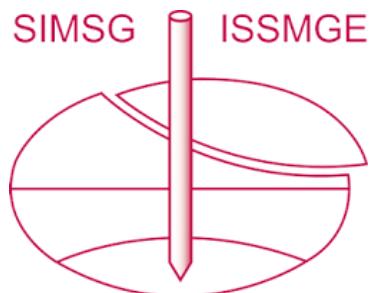


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Foundations of Structures

Fondations de Constructions

General Subjects and Foundations other than Piled Foundations—Sujets Généraux et Fondations autres que Fondations sur Pieux

Chairman / Président:

H. PEYNIRCOGLU, Turkey

Vice-Chairman / Vice-président:

O. MORETTO, Argentina

Assistant Reporter / Assistant Rapporteur: BENT HANSEN, Denmark

Oral discussion / Discussion orale:

Á. Kézdi	Hungary
A. W. Skempton	U.K.
J. K. Alderman	U.K.
H. Grasshoff	Germany
S. Chamecki	Brazil
D. Kršmanović	Yugoslavia
G. F. Sowers	U.S.A.
O. Moretto	Argentina
H. B. Sutherland	U.K.
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J. E. Jennings	South Africa
L. E. Collins	South Africa
I. M. Litvinov	U.S.S.R.
I. Ordemir	Turkey
A. Beles	Rumania
A. R. Jumikis	U.S.A.

B. J. Prugh	U.S.A.
P. J. Alley	New Zealand
V. G. Bereantzev	U.S.S.R.
R. V. Whitman	U.S.A.
O. Moretto	Argentina
E. de Beer	Belgium
A. Beles	Rumania
P. Habib	France
A. Dvořák	Czechoslovakia
P. E. Raes	Belgium
J. A. J. Salas	Spain
Bent Hansen	Denmark

Written contribution / Contribution par écrit:

E. de Beer and J. Wallays Belgium

Assistant Reporter

At this session the discussion of session 4 will be continued, and as the majority of the contributions this morning were concerned with bearing capacity problems I suppose that settlements will be the main topic this afternoon.

The standard procedure of estimating the settlements of a foundation can be criticized on many points. For example, laboratory samples are generally not undisturbed, oedometer tests do not represent the state of stress in nature, and the assumed stress distribution and drainage conditions are not in accordance with the actual conditions, although much work has been done in contributions to this conference to obtain a better agreement on this point.

It is, therefore, rather surprising to note that the methods normally adopted give reasonable agreement with the observed settlements, at least for normally or slightly over-consolidated clays. On the other hand, for heavily over-consolidated clays the deviations between calculated and observed settlements are generally excessive, so that the calculation method will have to be modified.

In Denmark, where such clays are frequently encountered as stiff, glacial clays, the following method is used: (1) the settlements are computed from the re-compression branch of the oedometer curve; (2) the re-compression branch is made after pre-loading and unloading to the overburden pressure. This procedure will normally reduce the ratio between the calculated and the observed settlement from 5 or 10 to 1·5 or 2.



Bent Hansen

Assistant Reporter, Division 3a / Assistant Rapporteur, Division 3a

The Chairman

I declare open the fifth session, which is a continuation of the fourth session, and I call on the Assistant Reporter, B. Hansen.

Sometimes model tests are made on these clays in order to check the calculation methods and to give a more accurate estimation of the settlements. If model foundations are used with diameters of 30 to 50 cm, it has generally been found that settlement predictions are reasonably accurate. It is also here necessary to use the re-compression branch in order to eliminate bedding effects due to disturbances while preparing the test site. The calculated settlements may be 20 to 50 per cent higher than the observed settlements.

It therefore seems possible to obtain in most cases a good estimation of the settlement in the described way. In order to obtain a better understanding of how this is possible it is proposed to discuss the current methods of settlement calculations and the theories of two- and three-dimensional consolidation. Other points of interest to practical application are the actual stress distributions and the allowable settlement of the superstructure.

A. KÉZDI (Hungary)

The costs of reliable settlement calculations are so great that in most practical cases they cannot be performed—we are dependent on good guesses. Nevertheless, improvements in

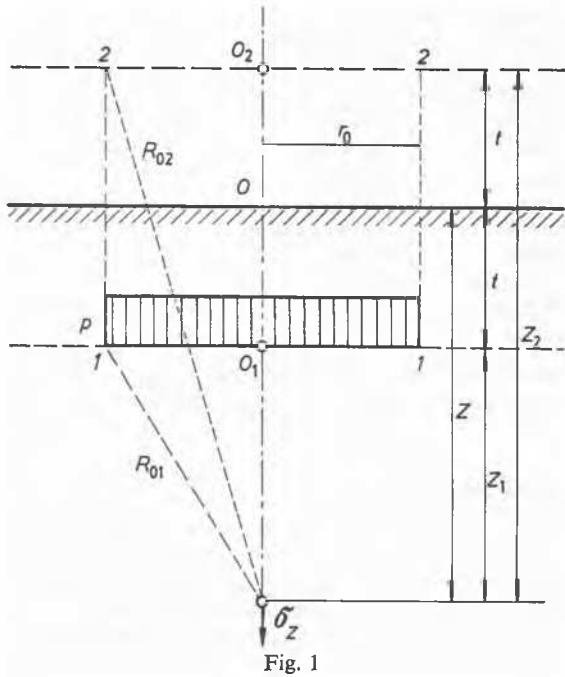


Fig. 1

the methods of calculation are very necessary in order to assess the importance of factors which affect the value of the settlement so that we can make a much better guess.

Stresses in the soil are computed, for instance, on the basis of the classical theory of elasticity, which is an approximation in itself, but at least real boundary conditions of the given case have to be considered. Thus, the fact that forces are acting in the interior of the semi-infinite body, in the depth of foundation, must not be disregarded. Stresses beneath a circular, uniformly loaded plate, embedded in the soil to a depth t (Fig. 1) are to be computed with the following formula:

$$\sigma_2 = -\frac{1}{1-\mu} \frac{3p}{2} \left[\frac{1-2\mu}{6} z_1 \left(\frac{1}{R_{02}} - \frac{1}{R_{01}} - \frac{1}{z_2} + \frac{1}{z_1} \right) + \frac{1}{6} - \frac{z_1^3}{6R_{01}^3} \right] + \left[\frac{3-4\mu}{2} z_2 - t(1-2\mu) \right] \left(\frac{1}{3z_2} - \frac{z_2^2}{3R_{02}^3} \right) - tz(z-t) \left[\frac{z_2^2}{R_{02}^5} - \frac{1}{R_{02}^3} \right] \quad \dots(1)$$

where μ is Poisson's ratio.

Fig. 2 gives curves in dimensionless terms for quick determination of these stresses. The deeper the foundation body lies the smaller are the stresses, and this affects considerably the value of computed settlements as demonstrated in Fig. 3.

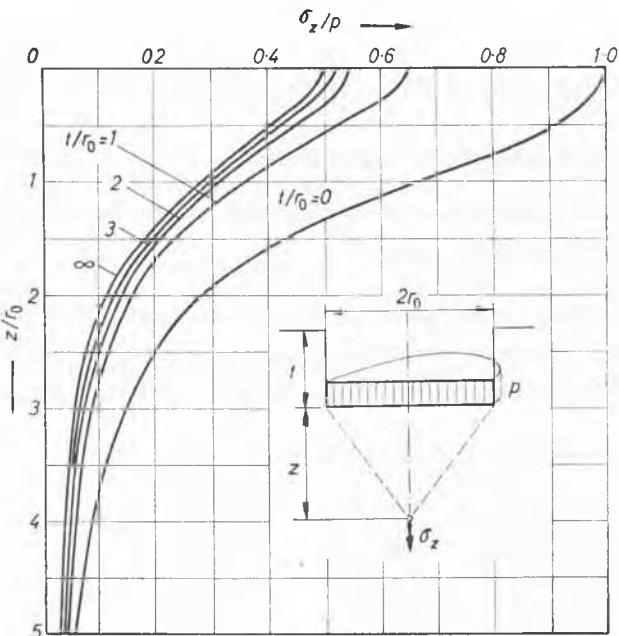


Fig. 2

Here the settlements of a circular plate are shown as a function of the depth of foundation. The curve designated 'usual method' has been calculated with stresses computed with the formula valid for surface loading; in the case of the

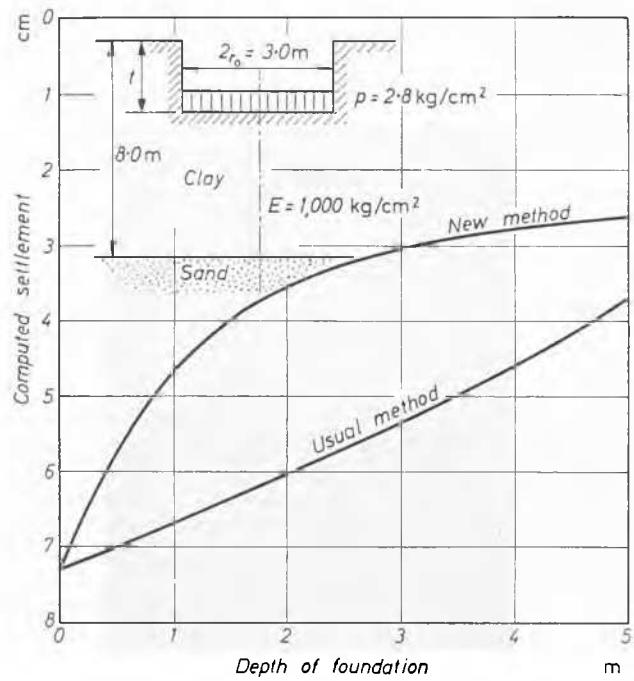


Fig. 3

curve 'new method', the stresses have been computed with formula 1.

In layered soils, when the upper layer is the stiffer—this condition being often artificially achieved by replacing a

certain thickness of the soft layer with well compacted sand or gravel—stresses on the lower layer decrease. There are graphs and tables for computing stresses acting in the boundary plane. To determine the vertical stresses along the whole

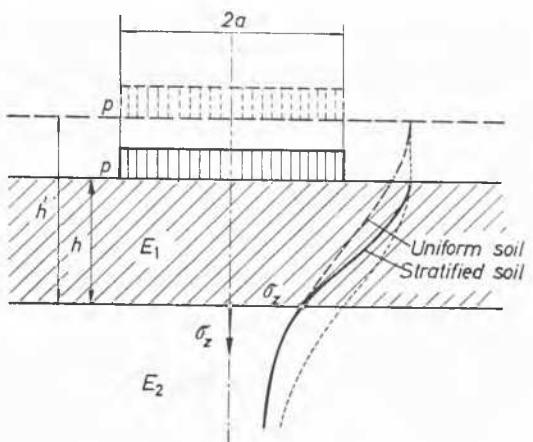


Fig. 4

centre line in the lower layer, however, the use of a uniform soil is recommended by increasing the thickness of the upper layer in such a way that the load acting on this elevated surface causes the same stress in a uniform soil at a depth h' , as the

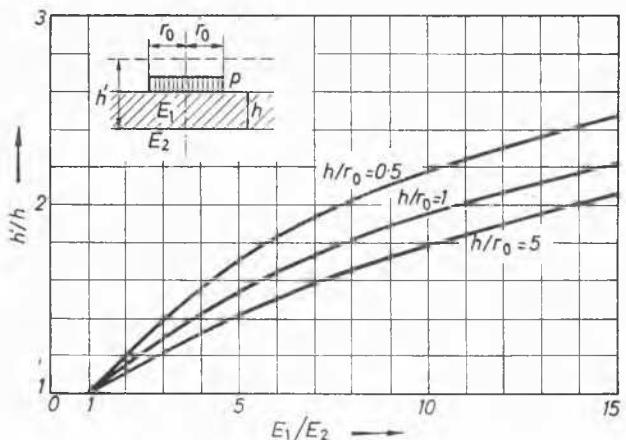


Fig. 5

load on the original surface in the layered soil, on the boundary plane, at a depth h . This principle is illustrated in Fig. 4; Fig. 5 furnishes curves for quick determination of the equivalent layer thickness h' as a function of the ratio of Young's moduli.

A. W. SKEMPTON (U.K.)

I should like to amplify some of the remarks concerning the settlement analysis of foundations on clay which were made by the Assistant Reporter, and I shall be talking only about those cases where the foundation is resting on a comparatively thick bed of clay.

The equations to which I refer are:

Final settlement

$$(1) \rho_{\infty} = \rho_i + \rho_c$$

$$(3) \rho_{\infty} = \rho_{oed}$$

$$(5) \rho_{\infty} = \rho_i + \mu \cdot \rho_{oed}$$

Settlement at time t

$$(2) \rho_t = \rho_i \times U \cdot \rho_c \quad \rho_i + U \cdot \rho_c$$

$$(4) \begin{cases} \rho_t = U \cdot \rho_{oed} \\ \rho_t = \rho_i + U(\rho_{oed} - \rho_i) \end{cases} \dots (a)$$

$$(6) \rho_t = \rho_i + U \cdot \mu \cdot \rho_{oed}$$

The application of a load from the foundation first of all causes deformations which we can consider as taking place at constant volume, it being granted that the clay is saturated; and the vertical deformation is known as the immediate settlement ρ_i . But the same stresses which have caused that deformation set up pore pressures in the clay, and in the course of time these pore pressures gradually dissipate, causing a corresponding gradual increase in the effective stresses and a corresponding compression or volume decrease in the clay, which leads to the well known long-term settlement ρ_c . There can be no question that equation 1 is the correct expression for the final settlement of a foundation on clay and that at any given time after construction the settlement expression is given by equation 2, with the consolidation component multiplied by the degree of consolidation, which can be obtained from the theory of consolidation in its various forms. There is also, of course, the secondary time effect, but I will not discuss that now.

Unfortunately, however, both terms in these equations present certain difficulties. The immediate settlement term requires, above all, the reliable measurement of the Young's modulus of the clay, and, as I think we are all quite well aware, that can in many cases be a difficult matter to achieve with any accuracy. However, I think it is true to say that until such time as we can obtain a reliable measure of E we shall not be able to get a reliable estimate of the immediate settlement. There is no short-cut.

Concerning the consolidation component, however, I think that in general the position is at least as difficult. The test which is usually carried out is the oedometer test, but this does not directly give us a measure of either the consolidation settlement or the final settlement. Of course, the oedometer test originated, I think I am right in saying, primarily for solving the problem of the layer of clay sandwiched between two beds of sand; but once we come to the case of a foundation resting directly on clay, the oedometer test is not directly applicable in principle. However, it is the test which we commonly carry out, and therefore it is obviously a matter of the utmost interest to know to what extent this test as normally interpreted gives us a reliable estimate of the settlement. We have recently made an examination of a considerable number of records, and we find that as a rough approximation—by no means in all cases, but in the majority of cases—the oedometer test just by itself does give at least a guide as to the final settlement; that is to say, the oedometer test appears to measure something like the sum of the two components ρ_i and ρ_c (equation 3). It is wrong, however, to multiply this by the degree of consolidation (equation 4a) and hope to get anything approaching a reasonable time settlement curve. If you wish to get a more reasonable time settlement curve you ought to make an estimate of the immediate settlement, subtract it from the oedometer settlement and apply the degree of consolidation to the difference, as indicated by equation 4b. This is, however, only a rough and ready method, and it is not satisfactory in principle.

I think that in due course we shall probably be carrying out settlement analyses for the case I am considering by carrying out a sort of triaxial test in which we apply the principal stresses first of all under undrained conditions, and then allow the dissipation of pore pressure to take place; but we have not got that far yet, and perhaps it is best for the present to consider an intermediate stage which is more promising than the rough and ready method expressed by equations 3 and 4b. This new method is expressed by equations 5 and 6. So far as the immediate settlement is concerned we have to face up to the necessity of measuring the Young's modulus of the clay as best we can—perhaps as a matter of experience one can make suitable corrections for sample disturbance; but I want to direct attention particularly to the method of obtaining the consolidation component.

As I have mentioned, the consolidation component is a direct consequence of the dissipation of the pore pressures which are set up by the foundation stresses, and these pore pressures will be very different, even for the same stresses, in different clays. In normally consolidated clays there will be rather high pore pressures; whilst they will be much smaller in over-consolidated clays. On the other hand, the oedometer test always sets up a pore pressure in the clay exactly equal to the applied increment of load. In other words, the oedometer test does not differentiate between the clays; consequently, we have to apply a correction to the oedometer test. This will depend, as I have indicated, on the structure of the clay—on its geological history—and there is evidence that for normally consolidated clays this multiplying factor is not very far short of unity—perhaps about 0.9. On the other hand, for the over-consolidated clays—for which we have most data on London clay—this multiplying factor is known to be more in the region of 0.5. L. Bjerrum and I have been working on this problem and we have developed an approximate theory which does relate this factor to the pore pressure coefficient, A , which is I think to be expected. This is a fairly promising line of approach, at least for the next few years until a more logical system of testing can be devised, and we shall shortly be publishing a paper on the theory.

J. K. ALDERMAN (U.K.)

I wish to address my remarks to the method of calculation for the term called the 'immediate settlement' as given by A. W. Skempton.

In dealing with glacial deposits of either lake or boulder form it was found that the method given by A. W. Skempton for estimating the settlement of a structure at any time, t (i.e. $\rho_t = \rho_i + U(\rho_c - \rho_i)$), could not be used because the immediate settlement, ρ_i , was actually greater than the calculated consolidation settlement, ρ_c . In view of this we have studied the method for determining the immediate settlement.

Referring to the equation

$$\rho_i = \frac{qn \cdot B(1 - \mu^2)I\rho}{E}$$

the only soil property in this equation is the value of E and it is therefore assumed that this value is lower than in the field, due to disturbance during sampling. This factor, however, is not of importance in our glacial clay deposits as they are absolutely insensitive and have the same value of E in the remoulded or undisturbed state.

It is, therefore, essential to study the method for determining E , which, as you will know, is obtained from the stress-strain curve derived from the standard undrained triaxial compression test. This is a strain control test and not a stress control test.

Does this apply in practice? Values of E have been determined for different rates of loading and, as an example, for a rate of strain of 0.3 in. per min. the value of E was 23 ton/sq. ft. On decreasing the rate of loading to 0.0022 in. per min. the value of E decreased to 13 ton/sq. ft., i.e. 100 per cent reduction in E is obtained simply by decreasing the rate of loading.

In view of these facts it is as well to consider whether the strain control method is applicable to field loading conditions. We have, therefore, carried out tests using stress control methods by applying direct loads to samples placed in the triaxial cell, and it was found that the E value obtained differed greatly from the standard method. This is demonstrated by the stress-strain curve shown in Fig. 6.

For London clay the ratio between the E value obtained by direct loading (stress control) and the E value obtained in the standard triaxial test (strain control) was 7, and for glacial clays the ratio was 3. Furthermore, if this direct load was applied for

a period of time there was immediately a small deflection and then a gradual movement with time, so that there was a tendency for a reduction in the E value. This, I feel quite certain, is associated with the shear creep movement and as such is connected with the viscous flow effect in clays.

It can be seen from these tests that the immediate settlement is very small and often negligible, but associated with this movement there is a shear creep in the clay whose magnitude is a

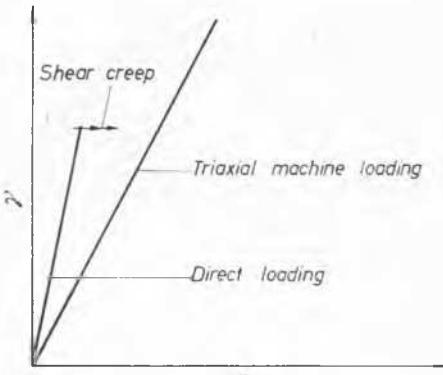


Fig. 6

function of time: therefore, in any settlement calculations we have to consider both the shear creep movement and the pure compression effect, both of which are functions of time. A theory for the above phenomena is not at present available but it is possible that the theory given by T-K. Tan this morning could be developed to produce a method for determining both the shear creep and the pure compression at any given time.

H. GRASSHOFF (Germany)

I should like to make some remarks on point (a) of the proposals for discussion which J. Brinch Hansen has stated in his General Report—namely, the influence of the rigidity of the superstructure on the pressure distribution.

For several years a Committee of the German Society of Soil Mechanics and Foundation Engineering has existed, under the guidance of E. Schultze, to investigate all these questions by means of extensive team work. Some recent results of this work will be announced.

It is a remarkable fact that D. Krsmanović (3a/18) has come to nearly the same results in his determination of an elastic foundation beam with three symmetrical loads as we have done, namely, that the curves of soil pressure under a perfectly flexible and a rigid superstructure do not essentially deviate. D. Krsmanović has evaluated some greater differences than I have; this may be because of his selection of a foundation beam with cantilevers and his dividing the beam into more sections than I have done. In the derivation of the bending moments of the beam it is very important to decide whether the columns are stiffly fixed with the beam or not, because thereby the distribution of the moments changes very much.

D. Krsmanović and myself have made the investigations on a slender beam. It would be interesting to know whether the results change under an extended plate. If a plate is stiff in the direction of one axis it may be evaluated in the so-called characteristic cross-section as a plane problem.

For the dimensions in plan of the plate which are to be seen in Fig. 7 I have evaluated several examples, the proportion of the sides being 1:2; 1:1½; 1:1; 1:1/10 and 1:1/20.

Fig. 8 shows the results of a flexible superstructure: only one-half of the symmetrical loaded beam is shown. The

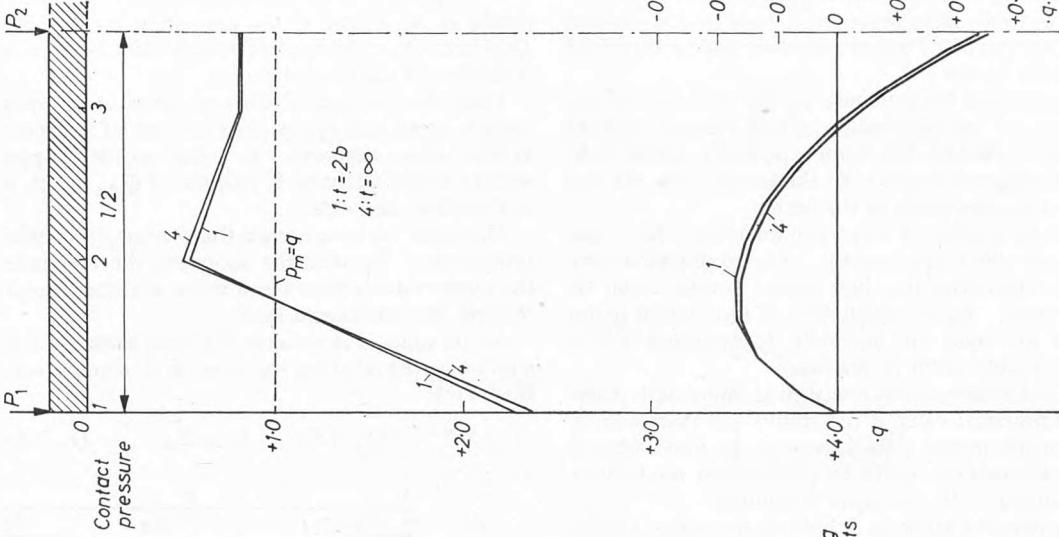


Fig. 9

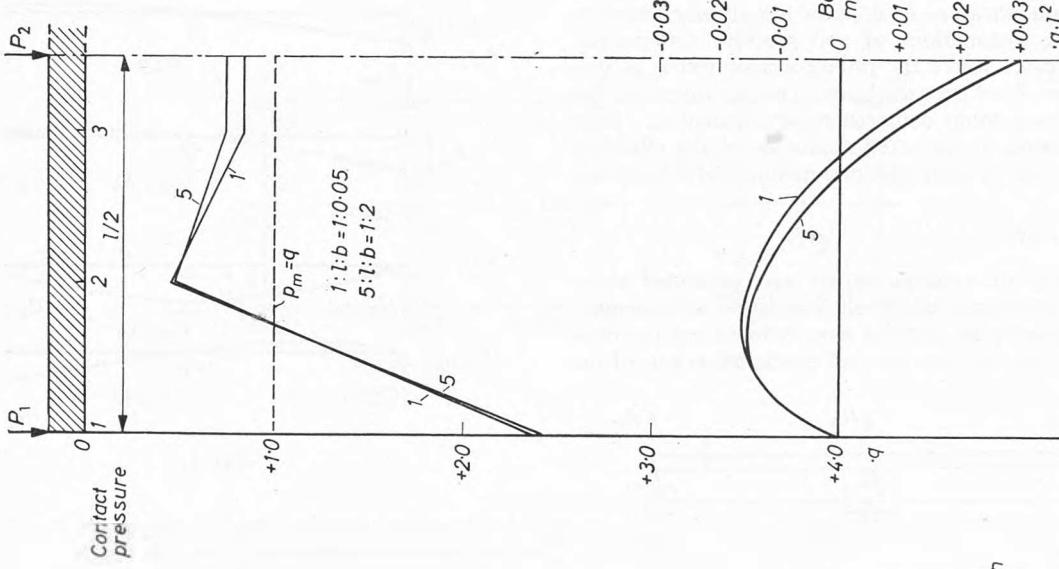


Fig. 8

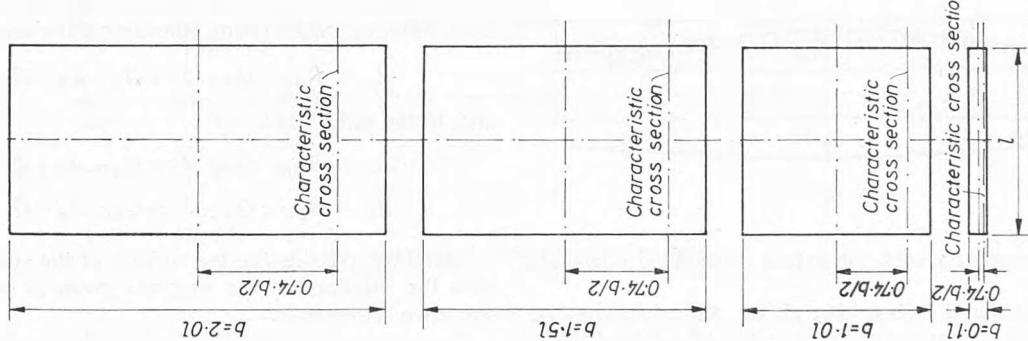


Fig. 7

upper curves indicate the soil pressure distribution; the lower ones the bending moments. The results are very interesting; only small differences occur in the soil pressure curves, in the curves of the bending moment there are practically no differences. Under a rigid superstructure I have found similar results. The computation of beams and plates will be simplified very much by these results.

I have also examined the influence of the thickness of the consolidated layer on the distribution of soil pressure and the bending moments. Besides the infinite perfectly elastic half-space I have investigated layers with thicknesses ten, six and two times larger than the width of the beam.

Fig. 9 shows the results of these computations: here also the curves do not differ appreciably. The examples shown refer to flexible superstructures but similar results could be shown for rigid ones. In the calculation of foundation plates the thickness of the layer can normally be neglected if it is thicker than the double width of the beam.

All mathematical investigations concerning foundation plates only give a safe practical value if the results are examined by tests. As large-scale model tests appear to be impossible, a control of the mathematical results by photoelastic model tests is meanwhile planned by the German Committee.

In reply to the remarks made by J. Feld on my paper, I agree with his opinion that we should not refine the theoretical computation to too great an extent. On the other hand, it is definitely not economical, as A. B. Vesic has already stated, to over-simplify the assumptions of soil pressure distribution. There are even cases where the primitive assumption of uniform distribution of soil pressure leads to results which are less conservative than a more comprehensive calculation. Even the smallest increase of the safety factor or of the efficiency would justify the use of more accurate methods of calculation.

S. CHAMECKI (Brazil)

Very interesting and valuable papers were presented about the influence of structural rigidity on foundation settlements.

In all of the papers the authors have pointed out the difficulties in taking into account the real elastic behaviour of the

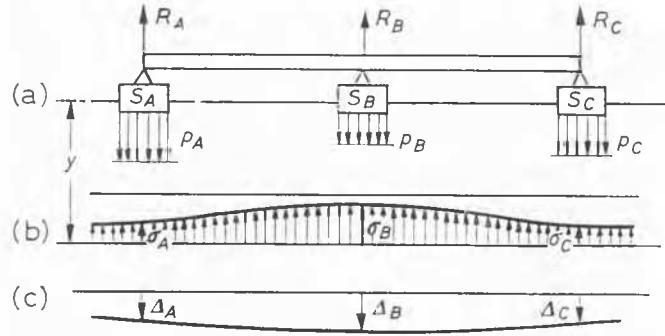


Fig. 10

structure in connection with the actual rheological constants of the soil.

I beg to state that I believe that all the difficulties may be overcome by using, what I called the 'coefficients of load transference' in my work entitled Structural Rigidity in Calculating Settlements, published in January 1956 as *Proceeding Paper No. 865 of the American Society of Civil Engineers*.

Let us consider the simple example of a two-span continuous beam resting on a foundation soil containing a compressive clay layer responsible for the settlements.

Let us assume that the real settlements of our beam are Δ_A ,

Δ_B and Δ_C (Fig. 11a), that will now be considered separately in order to determine their influence on the reactions of the supports.

If support A undergoes a settlement of unit value, it will create at A , B and C the respective vertical reactions Q_{AA} , Q_{AB} and Q_{AC} , the sum of which must be zero to satisfy the condition of static equilibrium.

Thus, the reaction of the support A undergoing unit settlement is equal and opposite to the sum of the reactions created at the other supports. In other words: support A , when settling a unit quantity, is relieved of Q_{AA} which is transferred to the other supports.

Therefore we have named these terms Q 'coefficients of load transference' between the supports, the first index indicating the support that settles a unit value, and the second the one that receives the transferred load.

As the support A undergoes a total settlement Δ_A , instead of a unit one, the resulting reactions in A , B and C are, respectively (Fig. 11c):

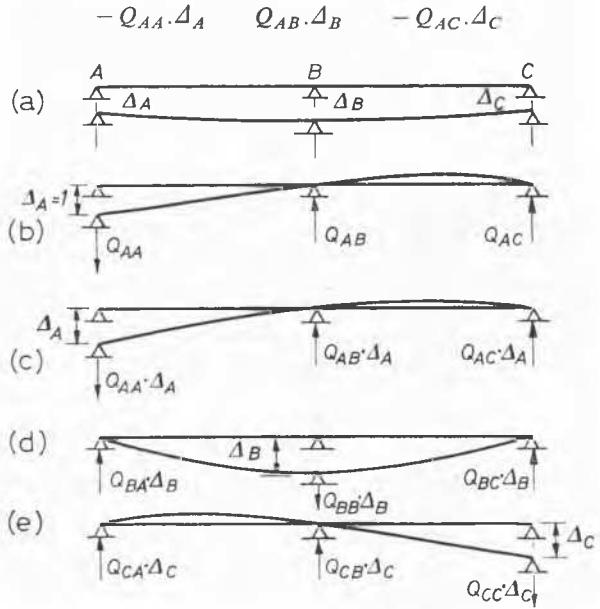


Fig. 11

In the same way, in Fig. 11d and e, the reactions created by the separate settlements Δ_B and Δ_C appear.

Representing by R_{OA} the reaction of the support A due to the load, we have, in our beam, considering the settlements:

$$R_A = R_{OA} - Q_{AA} \cdot \Delta_A + Q_{BA} \cdot \Delta_B - Q_{CA} \cdot \Delta_C$$

and, in the same way:

$$R_B = R_{OB} + Q_{AB} \cdot \Delta_A - Q_{BB} \cdot \Delta_B + Q_{CB} \cdot \Delta_C$$

$$R_C = R_{OC} - Q_{AC} \cdot \Delta_A + Q_{BC} \cdot \Delta_B - Q_{CC} \cdot \Delta_C$$

Therefore, considering the rigidity of the structure, we now have the reactions of the supports given as functions of the unknown settlements.

The reactions R_O are known and the 'coefficients of load transference' Q are elastic constants of the whole structure which can easily be calculated by the common analytical processes of statically indeterminate structures. As we have seen, they are the reactions of the supports due to settlements of unit value and can easily be supplied by the structural engineer by a small extension of his normal task of performing the analysis of the structure.

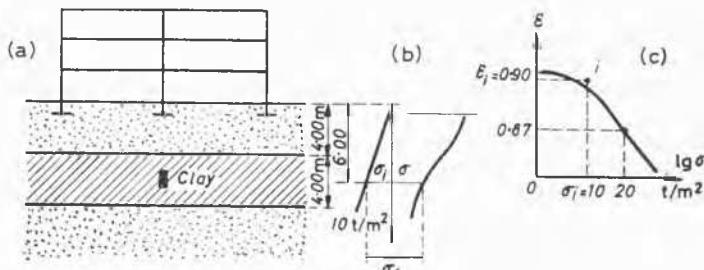
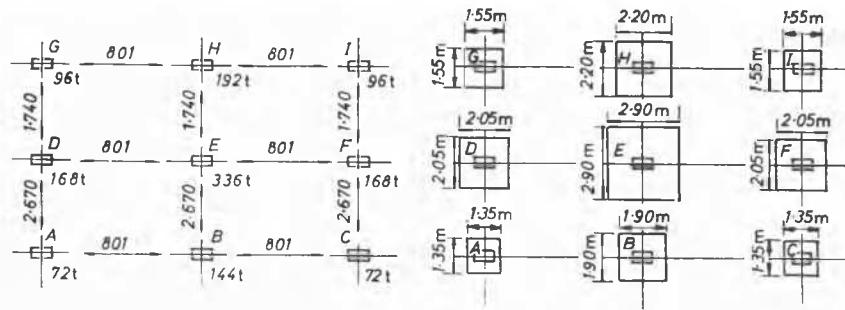


Fig. 12

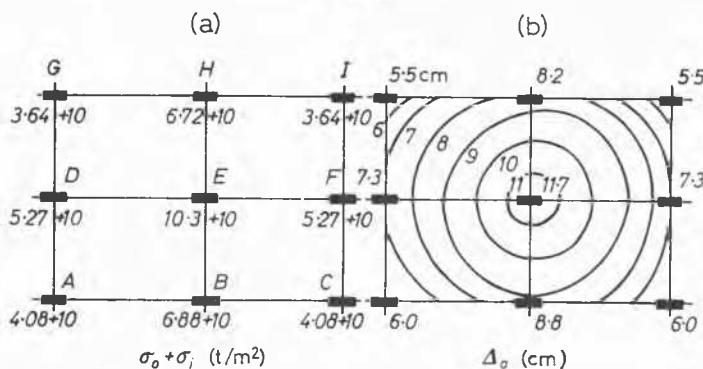


Fig. 13

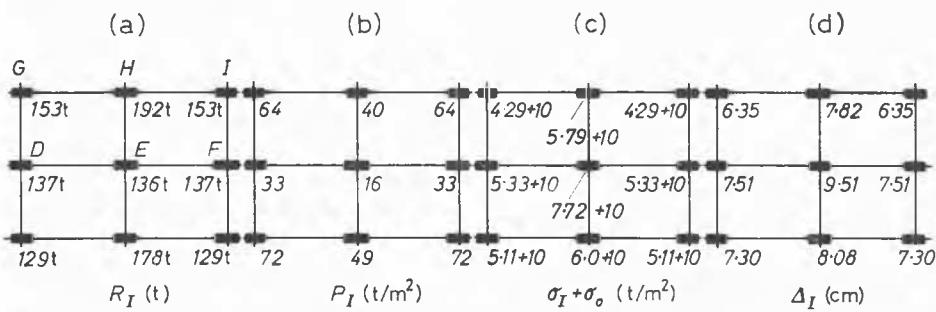


Fig. 14

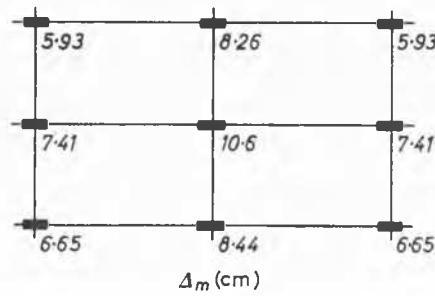


Fig. 15

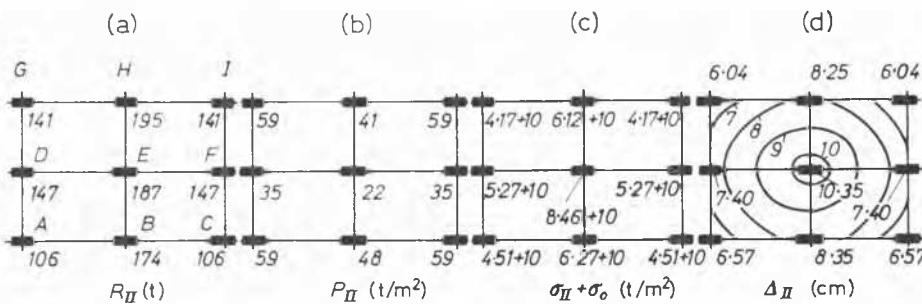


Fig. 16

In order to avoid the tedious solution of a laborious set of simultaneous equations we substitute for it, as is shown in the mentioned paper, a sequence of simple operations reaching the final result by successive approximations which can be done, in ordinary cases, in only two steps.

To conclude, let me show briefly those operations that do not differ from the routine used in Soil Mechanics, taking into account the real conditions of the underlying soil.

After having obtained the column load chart together with the 'coefficients of load transference', the geotechnical survey with the results of the consolidation test and the design of the foundation (Fig. 12) we perform the following operations:

(1) due to the column loads R_O , without consideration of the structural rigidity, we calculate the settlements Δ_O (Fig. 13);

(2) using those values we calculate the new column loads R_I , with the aid of the 'coefficients of load transference';

(3) due to those column loads we obtain the new settlements Δ_I (Fig. 14), this step representing the first correction due to the rigidity of the structure;

(4) considering that the successive approximations oscillate around and get nearer the final result, we make a second correction calculating the new column loads R_{II} taking the average settlements $\frac{1}{2}(\Delta_O + \Delta_I)$ (Fig. 15).

In most practical cases there will be no need of further corrections, as we can see in Fig. 16.

That solution given to the problem of interaction between structure and foundation soil is consistent, in all respects, with what is orthodox in both structural and foundation engineering. Yet the relations established by means of the 'coefficients of load transference' will hold even in the case of evolution and modification of the assumptions and processes for calculating soil deformations.

As I believe that the method may be applied to any type of frame-structure with any type of foundation resting on any soil, I do suggest that H. GRASSHOFF (3a/9) should try to use it for the solution of the general and actual case that lies between the limiting cases solved by him, and that K. E. EGOROV, P. G. KUZMIN and B. P. POPOV (3a/7) should try to use it in order to compare the results with the measured differential settlements performed by them in their country.

D. KRSMANOVIC (Yugoslavia)

Cette contribution examine, conformément à la proposition du Assistant Rapporteur, l'influence de la rigidité de la superstructure (S_k) sur la répartition des pressions sous la poutre de fondation continue (S_t), et ceci pour les constructions traitées dans le rapport sous 3a/18.

Si nous considérons uniquement l'influence de la rigidité de la superstructure sur la répartition sous la poutre de fondation, nous constatons que cette influence (quand les autres paramètres qui influent la répartition sont constants) peut être différente dans l'ordre de grandeur en fonction du mode de

chargement de la construction. Tandis que l'influence de la rigidité de la superstructure est très petite dans certains cas, elle est dans d'autres cas très importante.

Les influences sont petites quand les différences de tassement des divers appuis — lorsqu'on traite séparément le système S_t chargé d'un certain chargement ($J_k=0$) — sont égales à zéro ou quand elles sont minimales (chargement favorable). Dans ces cas la pose de la superstructure avec un chargement correspondant sur le système S_t ne provoque pas de changement

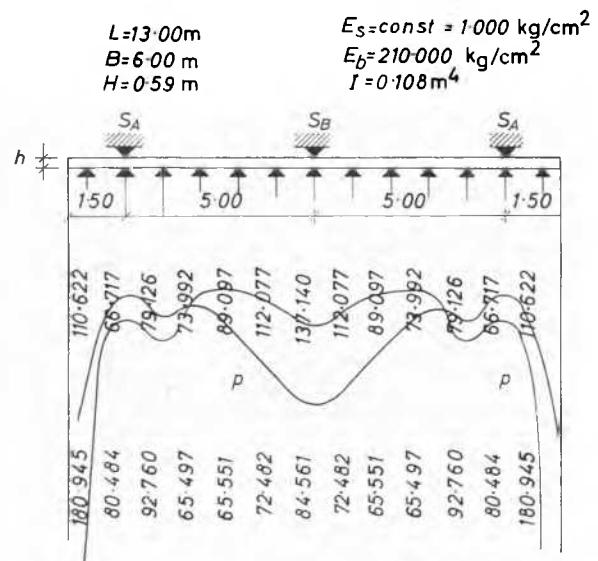


Fig. 17 Poutre continue 1·5+5·0+5·0+1·5
Lignes de répartitions pour divers rapports entre les forces S_A et S_B

$$\Sigma S = 1,200 \text{ t}$$

Répartitions p $S_A:S_B:S_A = 1:2:1$; $S_A = 300 \text{ t}$; $S_B = 600 \text{ t}$

Répartitions p' $S_A:S_B:S_A = 1:0.81:1$; $S_A = 427 \text{ t}$; $S_B = 346 \text{ t}$

Continuous beam 1·5+5·0+5·0+1·5
Distribution curves for various ratios of forces S_A and S_B

$$\Sigma S = 1,200 \text{ t}$$

Distributions p $S_A:S_B:S_A = 1:2:1$; $S_A = 300 \text{ t}$; $S_B = 600 \text{ t}$

Distributions p' $S_A:S_B:S_A = 1:0.81:1$; $S_A = 427 \text{ t}$; $S_B = 346 \text{ t}$

dans la répartition des pressions, et de ce fait, ne provoque pas non plus de changement dans les tensions de la construction.

Dans le second cas, quand les différences d'affaissement des appuis sont importantes — considérant séparément le système S_t , sous certain chargement (chargement défavorable) — les influences de la pose de la superstructure sont importantes et provoquent des changements dans la répartition des pressions et dans les tensions de la construction entière.

Pour illustrer l'influence de la rigidité de la superstructure sur la répartition des pressions, observons premièrement deux

cas extrêmes I_k égal à zéro et I_k égal à l'infini, et ensuite les cas avec les valeurs finies I_k .

Les répartition de chargement pour deux cas I_k égal à zéro

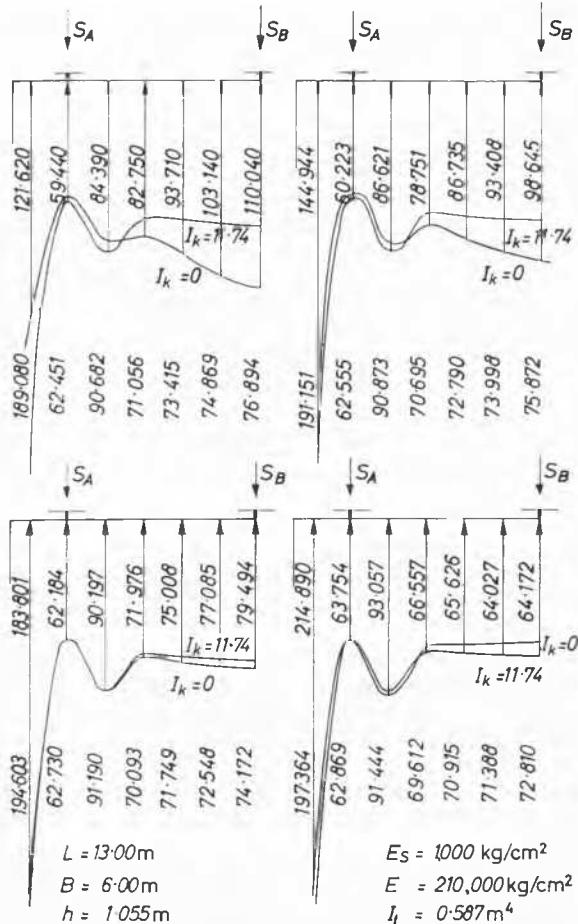


Fig. 18 Poutre continue $1 \cdot 5 + 5 \cdot 0 + 5 \cdot 0 + 1 \cdot 5$
Lignes de répartitions pour divers rapports entre les forces S_A et S_B
et valeurs $J_k = 0, J_k = 11 \cdot 74 \text{ m}^4$
 $\Sigma S = 1,200 \text{ t}$

Continuous beam $1 \cdot 5 + 5 \cdot 0 + 5 \cdot 0 + 1 \cdot 5$
Distribution curves for various ratios of forces S_A and S_B and values
 $J_k = 0, J_k = 11 \cdot 74 \text{ m}^4$
 $\Sigma S = 1,200 \text{ t}$

et I_k égal à l'infini sont données pour la poutre de fondation continue examinée et représentée par la Fig. 17, sous condition que la somme des forces S soit toujours constante. La ligne

p représente la répartition sous la poutre chargée des forces S avec rapport entre $S_A : S_B : S_A = 1 : 2 : 1$, si I_k est égal à zéro.

La différence d'affaissement des appuis A et B se monte dans ce cas à $4 \cdot 21 \text{ mm}$ (voir Fig. 3, Contribution 3a/18).

Dans le cas quand I_k est égal à l'infini, nous avons obtenu la répartition p' qui effectivement correspond au rapport entre les

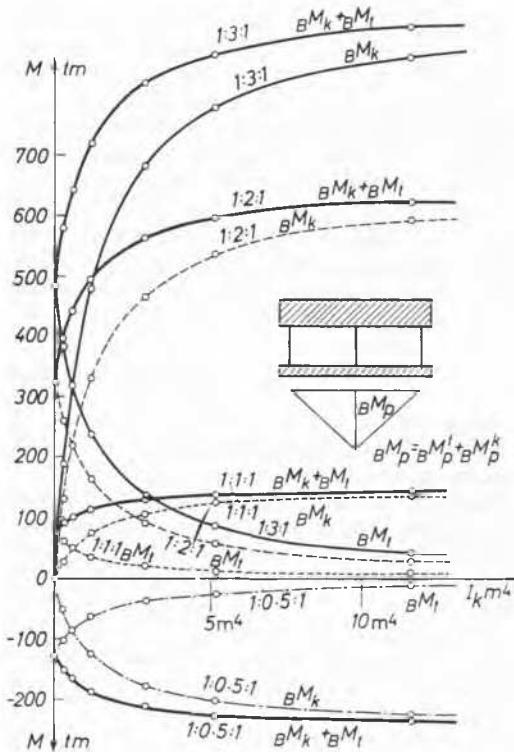


Fig. 19 Poutre continue $1 \cdot 5 + 5 \cdot 0 + 5 \cdot 0 + 1 \cdot 5$
Moments fléchissants en fonction de la rigidité du système S_k .
 $I_t = \text{const.}$

Continuous beam $1 \cdot 5 + 5 \cdot 0 + 5 \cdot 0 + 1 \cdot 5$
Bending moments as a function of the rigidity of the system S_k .
 $I_t = \text{const.}$

forces $S_A : S_B : S_A = 1 : 0.81 : 1$. Dans ce cas les différences dans les affaissements des appuis A et B sont égales à zéro.

Dans le premier cas, la pose de la superstructure rigide sur le système I_t égal zéro et avec le rapport $S_A : S_B : S_A = 1 : 2 : 1$ influera beaucoup la répartition et elle passera de la répartition p à celle de p' . Mais dans le second chargement, la pose d'une superstructure rigide avec le rapport des forces $S_A : S_B : S_A = 1 : 0.81 : 1$ ne provoquera aucun changement dans la répartition

Tableau 1

Poutre continue $1 \cdot 50 + 5 \cdot 00 + 5 \cdot 00 + 1 \cdot 50$. Valeurs des moments fléchissants pour les cas quand $I_k = 0$ et $I_k = \infty$
Continuous beam $1 \cdot 50 + 5 \cdot 00 + 5 \cdot 00 + 1 \cdot 50$. Value of the bending moments for $I_k = 0$ and $I_k = \infty$

$L = 13 \cdot 00 \text{ m}$ $B = 6 \cdot 00 \text{ m}$ $h = 0 \cdot 59 \text{ m}$	Rigidité	Manière du chargem.	Moments dans le syst. S_t				$S_k \text{ tm}$	Remarques	
			AM^t	BM^t	AM_a^t	BM_a^t			
$I_k = 0$	1 : 2 : 1		118.96	342.61	—	114.60	—	—	Syst. S_t charge défavorablement
	1 : 0.81 : 1		191.00	134.04	—	—	—	—	Syst. S_t charge favorablement
$I_k = \infty$	1 : 2 : 1		191.00	134.04	—	—	635.00	—	Syst. S_k charge défavorablement
	1 : 0.81 : 1		191.00	134.04	—	—	—	—	Syst. S_t et S_k charge favorablement

$$E_b = 210 \cdot 000 \text{ kg/cm}^2$$

$$E_s = 1 \cdot 000 \text{ kg/cm}^2$$

$$I_t = 0 \cdot 108 \text{ m}^4$$

Tableau 2

Poutre continue $1 \cdot 50 + 5 \cdot 00 + 5 \cdot 00 + 1 \cdot 50$
 Valeurs des moments fléchissants pour divers rapport entre les forces S_A et S_B en fonction des changements de rigidité du système S_t

Continuous beam $1 \cdot 50 + 5 \cdot 00 + 5 \cdot 00 + 1 \cdot 50$

Value of the bending moments for various ratios between forces S_A and S_B as a function of changes in the rigidity of system S_t

Rapports des forces	I_k m^4	AM_t	BM_t	$AM_a' = AM_p'$	$BM_a' = BM_p'$	AM_p^k	BM_p^k
1 : 3 : 1	0·00	129·050	585·120	0·000	491·510	—	—
$L = 13\cdot00\text{ m}$ $B = 6\cdot00\text{ m}$ $h = 1\cdot055\text{ m}$	1 : 0·901 : 1*	11·740	196·886	186·037	0·000	43·398	0·000
	1 : 2 : 1	0·00	114·940	520·360	0·000	334·590	0·000
	1 : 0·865 : 1	11·740	198·970	169·760	0·000	29·780	0·000
	1 : 1 : 1	0·00	191·574	95·560	0·000	78·190	0·000
	1 : 0·805 : 1*	11·740	202·444	142·653	0·000	6·950	0·000
	1 : 0·5 : 1	0·00	222·859	131·910	0·000	126·935	0·000
	1 : 0·759 : 1*	11·740	205·223	120·955	0·000	11·275	0·000
							125·490

* Forces dans les piliers.

des pressions. Celle-ci restera telle qu'elle était quand I_k était égal à zéro (répartition p').

Les valeurs des moments fléchissants dans les systèmes S_t et S_k sous les appuis et dans les différents cas (conformément aux signes dans 3a/18) sont données au Tableau 1.

données pour différents cas de rapport entre les forces S_A et S_B . On peut constater que les différences dans les répartitions peuvent être très grandes quand les rapports des forces S sont défavorables.

Nous avons aussi calculé pour tous ces cas les valeurs des moments fléchissants au dessous des appuis du système S_t , ainsi que les valeurs des moments M_a' , M_p' et M_p^k . Ces valeurs sont indiquées sur le Tableau 2.

Les changements des valeurs des moments M_a' et M_p' en fonction des changements de rigidité du système S_k et des rapports des forces S sont représentés par la Fig. 19 pour les cas donnés.

Fig. 20 montre les différences de tassement des appuis A et B dépendant des valeurs I_k , mais pour une valeur constante de I_t .

Conclusion — L'influence de la rigidité de la superstructure sur la répartition des pressions peut varier dans une grande mesure quand on la considère en fonction du chargement. En tant que la construction est chargée avec un rapport favorable entre les forces extérieures les influences sont petites et *vice versa*.

G. F. SOWERS (U.S.A.)

The problem of the allowable differential settlement of structures has been under study at the Georgia Institute of Technology. The results of part of the work can be given here.

Theoretical studies were made of the amount of differential settlement required to produce structural failure in typical multi-storied rigid frame buildings. It was assumed that the buildings were loaded to the full design live and dead loads, and then differential settlement occurred instantaneously. The deformations required to produce structural failure are as follows: 0·002 ($\frac{1}{5}$ of 1 per cent) of the column spacing for steel frame; 0·0025 ($\frac{1}{4}$ of 1 per cent) of the column spacing for concrete frame.

If the settlement should take place slowly the structures would adjust themselves plastically. In such cases somewhat greater deformations would be required for failure.

These values are within the range given in Paper 3a/31 by D. E. POLSHIN and R. A. TOKAR.

O. MORETTO (Argentina)

Small, light buildings founded on certain types of clays, called either swelling or active clays, are frequently damaged

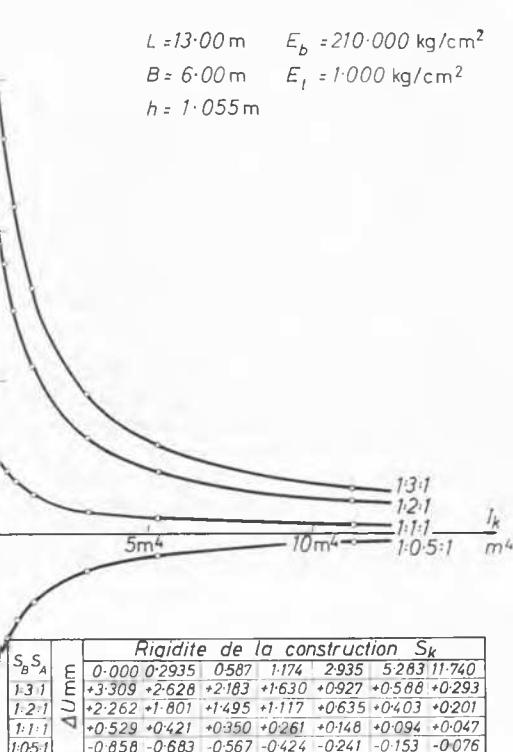


Fig. 20 Poutre continue $1 \cdot 5 + 5 \cdot 0 + 5 \cdot 0 + 1 \cdot 5$
 Tassement relatif des appuis en fonction de la rigidité du système S_k . $I_t = \text{const.}$

Continuous beam $1 \cdot 5 + 5 \cdot 0 + 5 \cdot 0 + 1 \cdot 5$

Relative settlement of the supports as a function of the rigidity of the system S_k . $I_t = \text{const.}$

Pour les cas quand I_k a une valeur finie, les différences dans les répartitions pour un cas examiné sont indiquées à la Fig. 18, tout en changeant I_k de zéro à une valeur déterminée ($I_k = 11 \cdot 74 \text{ m}^4$) et I_t restant constant. Les répartitions sont

by cracking of walls and by uplifting and cracking of floors. This damage results as a consequence of the disturbance produced, by the construction of buildings, on the natural cyclical up and down movement of the earth surface. It is most conspicuous in arid and semi-arid regions, where usually it results in an uplifting of the building, provoked by swelling forces that in these regions may acquire considerable values. The mechanism of this phenomenon is well known. It has been described in many papers, including one by myself presented at the Brazilian Conference on Soil Mechanics and Foundation Engineering held in Porto Alegre in 1954.

Normally, at first, the soil expands and the building is lifted and stretched. Later, with seasonal changes, the soil may shrink and swell alternately to a certain depth below surface. With these shrinkages and swellings the building moves up and down in a non-uniform fashion, and walls are distorted and stretched, as well as floors. Cracks resulting in walls have a very distinct pattern, comprising the normal 45 degree crack and also horizontal and vertical cracks.

Solutions proposed to solve this problem are not many. They are reviewed in Paper 3a/36 by J. A. J. SALAS and J. M.

produced by soil expansion is small, and whatever may come is resisted by the beam itself.

The floors are designed following another line of thought. As you all know, expansion of active clays is a sporadic event; it may or may not take place, depending on factors which we do not as yet understand. Because of this, a calculated risk can be taken with floors, given that any solution that will surely take care of floors is usually too expensive and out of the question. Besides, a floor failure is usually much less serious than a wall crack, at least from the psychological point of view! We also discovered that the number of floor failures is very sharply reduced if the floor is integrated with a slab of rather poor concrete, reinforced only with the purpose of avoiding the cracks opening and eventually leading to the break-up of the floor. The floor makes with the reinforced slab a very flexible structure capable of sustaining rather big deflections without breaking apart, thanks to the ties provided by the reinforcement.

The solution as described is frequently cheaper than a normal wall footing foundation and occasionally only a little more expensive.

H. B. SUTHERLAND (U.K.)

J. Feld has discussed the question of the normal design assumption of uniform pressure distribution under isolated footings. I should like to describe briefly the measurements of pressure distribution which are at present being made under two of the footings of the new engineering extension which is being built at the University of Glasgow.

The footings are adjacent to each other and their design loads are 230 and 30 ton. The subsoil is a very stiff boulder clay which extends to a considerable depth, the permissible bearing capacity of which has been taken as 3 ton/sq. ft.

At the larger footing, 16 pre-cast concrete blocks 2 ft. 6 in. square were placed close to each other to form four rows each of four blocks. On each of these 16 blocks is a cylindrical vibrating wire load gauge. These gauges carry the reinforced concrete footing which is 9 ft. square and 2 ft. 6 in. thick. The distribution of column load can therefore be measured at 16 points below the footing. The column transmitting the load to the footing is a built-up steel section encased in concrete. A temporary hinge point has been provided at the base of the column to eliminate the transmission of bending to the footing.

The column load is also being measured by vibrating wire strain gauges attached to the flanges of the steel. The column load can therefore be checked against the sum of the loads registered on the 16 cylindrical gauges under the footing.

Construction of the building is still proceeding, but the measurements made to date show the edge pressures to be about twice the pressure at the centre of the footing. The pressure distribution takes the form to be expected theoretically for a rigid type footing on a clay soil.

L. MENARD (France)

Monsieur le président, messieurs, à la suite de diverses interventions relatives aux mesures de sol *in situ* il convient de signaler tout l'intérêt de la méthode de R. HAEFELI et H. B. FEHLMANN (2/4) concernant la mesure de la compressibilité du sol en profondeur, méthode qui consiste à interpréter à l'aide de formules de Boussinesq l'enfoncement de la pointe d'un pénétromètre. En fait, les équations de Boussinesq ne sont rigoureusement applicables qu'au seul calcul des contraintes et déformations produites par une charge, à la surface du sol. Aussi, nous permettons-nous de signaler que l'analyse de ce type d'essai serait facilité par l'utilisation directe des formules de Mindlin qui sont une généralisation des formules de Boussinesq pour les essais de chargement en profondeur.

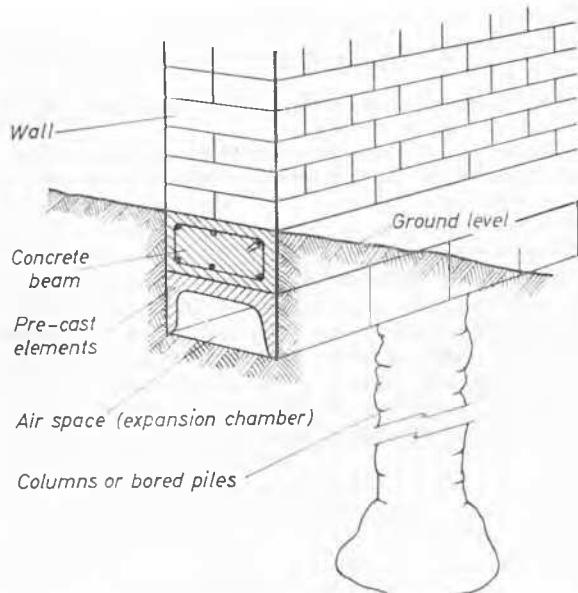


Fig. 21

SERRATOSA and are based on one of the following two lines of thought.

(1) One line of thought avoids the problem of swelling clays completely by placing the whole building in the air so that no swelling stresses will act on it.

(2) The other line of thought meets the problem by making a structure strong enough to resist distortions. To take care of floor damage this solution has to be provided with an adequately reinforced floor slab.

Both solutions are very expensive and can be used only exceptionally, on either very unusual soils or on expensive houses.

Due to these circumstances an intermediate solution was devised and has been used for some time now in Argentina, giving satisfactory results. This solution takes care of wall cracking by placing all walls, including partitions, on reinforced concrete beams, sustained on columns or bored piles of sufficient length. The beams are separated from the ground by chambers of expansion which permit swelling of the clay without lifting the walls, as indicated in Fig. 21. Since these beams are placed at or near ground level the stretching effort

Il est d'ailleurs démontré dans le traité de mécanique des sols de A. Caquot et J. Kérisel que dans le cas d'une charge uniforme appliquée sur une aire circulaire, le coefficient de réduction à apporter aux résultats de Boussinesq, varie de 1 à 0.50 environ, au fur et à mesure que la profondeur augmente.

Je saisais l'occasion de cette intervention relative à des essais de compressibilité '*in situ*' pour compléter ma brève intervention au cours de la troisième séance.

Contrairement à l'appareil de Köbler, et je pense que W. Aichhorn est entièrement d'accord à ce sujet, le pressiomètre est constitué de trois cellules superposées, dilatables radialement et gonflées à la même pression; on obtient ainsi des surfaces isostatiques rigoureusement cylindriques en regard de la cellule médiane de mesure et il est ainsi possible de mesurer avec précision les augmentations de diamètre correspondantes.

Des essais ont montré que si l'on utilisait une seule cellule sur trois, on était conduit aux résultats les plus fantaisistes au point que l'on obtenait des résultats de compressibilité de 2 à 6 fois supérieurs à ceux obtenus à l'aide du pressiomètre ou par des essais classiques.

Il a été objecté que la pression naturelle du terrain sur la paroi devait conduire à une diminution importante du diamètre du sondage après les opérations du forage. Or, des essais systématiques ont montré que cette diminution de diamètre — qui est de l'ordre de 1 pour cent pour les argiles raides et de 8 pour cent pour les argiles molles et les sables — n'intervenait en rien dans les mesures pressiométriques.

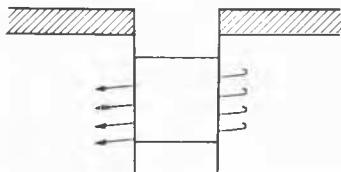


Fig. 22

Notre intention au cours de ce congrès n'était que de signaler, sans plus, l'existence de ce nouvel appareil de sol. En fait, des milliers d'essais ont été réalisés au cours de ces derniers mois par nos expérimentateurs sur plus de cinquante sites différents et accompagnés de mesures de comparaison à l'oedomètre, l'appareil triaxial au seismomètre et à l'appareil de module de réaction.

Ces résultats nous permettent d'adopter systématiquement le pressiomètre pour les mesures de la résistance au cisaillement et de la compressibilité. Un rapport complet sera soumis au prochain congrès.

A la suite de l'intervention de Y. TCHENG et A. LAZARD j'aimerais seulement vous présenter un résultat souvent obtenu dans les sables compactés: Un essai pressiométrique effectué à faible profondeur constitue une expérience de butée. Or, dans les sables qui, au triaxial ont une cohésion nulle on trouve parfois des résistances de butée pouvant être de l'ordre de 100 t/m². Ce résultat ne peut s'expliquer que par l'existence d'une pseudo cohésion due à l'enchevêtrement des grains. Cette pseudo cohésion qui peut atteindre 1 kg/cm² disparaît au moindre remaniement de l'échantillon.

J. E. JENNINGS (Union of South Africa)

M. S. Youssef and others who have contributed papers on heaving foundations will be interested in recent developments in South Africa which show that it may be possible to estimate heave from a slightly modified consolidation theory.

Considering heave as a process of change in effective stress in the soil, the problem may be represented in Fig. 23 as a

process of changing the negative pore pressure in the soil from the curve *ABaC* to the curve *ABbD*, i.e. from the conditions as they exist in the ground in nature to the conditions which result after the surface evaporation has been eliminated and a new equilibrium moisture regime is established.

The study of this effective stress change may be done in the normal oedometer by considering the change from applied pressure $\sigma = cd$ to effective pressure $\sigma' = bd$ in Fig. 23. Fig. 24

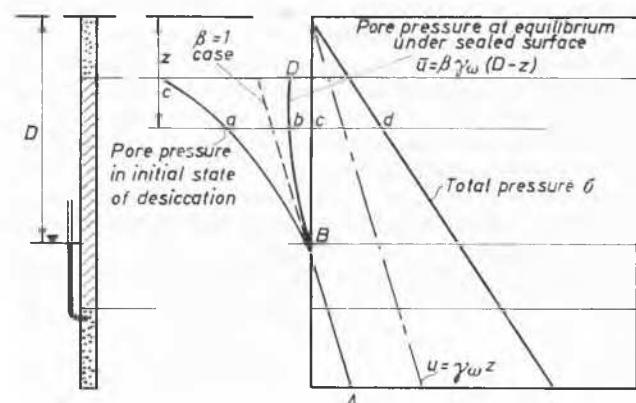


Fig. 23 Pore pressure changes due to wetting up of a desiccated clay under surface coverage

Changements dans les pressions interstitielles causée par le mouillage d'une argile desséchée sous une couverture en surface

shows the results of such a double oedometer test. The dashed curves represent the compression under applied pressures on a sample of soil at its natural water content while the continuous line represents the compression of an identical sample under effective stresses only, i.e. when the negative pore pressures in the soil are completely relieved by soaking the sample in water for a sufficiently long time. An important fact is the coincidence of the straight C_c sections of these curves and adjustment

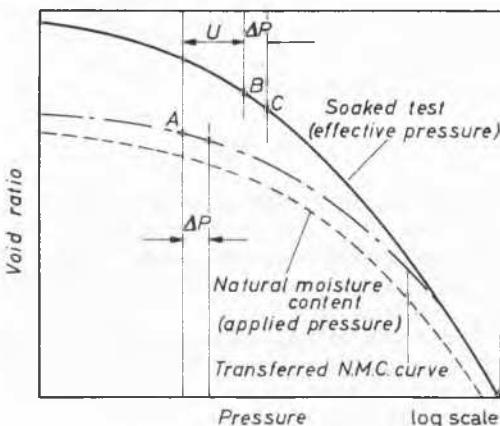


Fig. 24 Oedometer curves for estimating heave of desiccated clay Courbes oédométriques pour estimer le soulèvement d'une argile desséchée

of slightly dissimilar curves is necessary in most cases to take account of natural variations in the soil.

For free heave conditions, i.e. with no surface loading, the movements are calculated from the void ratio changes from point *A* to point *B*. Where the soil is loaded by a structure, the movements are calculated from point *A* to point *C*.

This method takes account of surcharge, overburden, degree of desiccation and clay activity in a single operation. Com-

parison of results of predictions and field observations has given very satisfactory results up to date.

Further details of this method together with other aspects of heaving are being dealt with at a symposium to be held in Johannesburg in September and will be published in the *Transactions of the South African Institution of Civil Engineers*.

L. E. COLLINS (Union of South Africa)

There are some points that I should like to mention in connection with Paper 3a/36 by J. A. J. SALAS and J. M. SERRATOSA and with the comments of O. Moretto.

Frequent observations carried out to determine the movements of a small single-storey brick house founded on expansive soil in Vereeniging, South Africa, over the past six years have shown that the time-heave pattern is as follows:

(1) A 'general' heave occurs which, when plotted against time, gives rise to a 'logistic' type of curve with a slope which increases for about two years and then decreases again, apparently tending to an asymptote at about 8 cm: after about five years, the average heave was 7 cm.

(2) Superimposed on the general curve is a seasonal fluctuation showing sharp increases in heave which seem to correlate well with the rainfall which normally occurs in summer. The amplitude of the seasonal variation is initially about 1 cm but it decreases with time.

(3) The difference between the average movements of observed points on the periphery of the house and those of points near the centre of the house show that a 'doming' of about 1 cm has so far occurred: this value is still increasing.

(4) Observations on the movements of the soil at different depths indicate that by far the greatest heave has taken place in the upper portion of the expansive soil profile. The damage to the building is fairly severe, but not sufficient to make it uninhabitable.

On this house, as on many others built on expansive soil in South Africa, the cracking pattern indicates that 'hogging' as well as 'sagging' deflections have taken place, so it is necessary to take both possibilities into account when designing buildings with reinforced walls on expansive soils.

Present methods of reinforced brick design used in South Africa are not aimed at preventing cracking but rather at minimizing it in an economical way.

These observations are dealt with more fully in a paper to be published in the September *Transactions of the South African Institution of Civil Engineers*.

O. Moretto said that one of the solutions of founding buildings on expansive clays was to avoid the expansive nature of the clay by founding at considerable depth, and that this method tended to be somewhat expensive. In practice one does not avoid the problem of the expansive nature of the clay. One has to deal with it in rather a complicated way, because considerable tensile stresses can occur in such a pile as a result of the shear forces exerted on the pile by the expanding clay. In South Africa cases have occurred where the piles have been broken in tension by these forces. As regards the matter of expense, there are few data, but some fairly reliable figures indicate that the cost need not exceed 12 per cent of the total cost of the building which, when balanced against the savings in future maintenance, is not exorbitant.

I. M. LITVINOV (U.S.S.R.)

In the U.S.S.R. there have been developed and introduced in building practice two methods of thermal consolidation for use on permeable clayey soils.

The first method, due to Ostashov, consists of forcing hot air

at a temperature of 600° to 800° C. through bore holes into the soil. The second method, developed by myself, depends on the burning under pressure in the soil itself of various fuels.

Both methods are recommended although the second is considered to be more effective and economical; it enables large volumes of soil to be treated at the same time.

In the second method the heating of the soil mass to temperatures of between 800° and 1,000° C. is achieved by the penetration of hot air or incandescent products of combustion into the voids of the soil. In order to facilitate this penetration the pressure of the hot gases is kept above that of the atmosphere by pumping cold air into the bore holes.

The temperature of the products of combustion in the second method must not exceed the fusion temperature of the soil to be consolidated: this is easily ensured by regulating the amount of cold air supplied. The amount of fuel is determined according to the permeability of the soil.

Each bore hole, 4 in. in diameter and 30 to 35 ft. deep, produces a brick-like mass 7 to 10 ft. in diameter and 30 to 35 ft. deep. The burning of the bore holes is carried out simultaneously in groups of 10 to 20 and more.

These methods of thermal soil consolidation are being applied in the U.S.S.R. on many sites in the construction of foundations for new buildings, in underpinning existing buildings which show unequal settlement to a dangerous degree and in some other cases.

I. ORDEMIR (Turkey)

In connection with J. A. OSTERBERG's very useful paper (3a/28), I should like to inform the conference that we have similar contributions which have not yet been published, but preparations are being made for their publication in the very near future.

As part of a thesis presented to the Technical University of Istanbul in 1955, I calculated graphs to give the pressure areas for trapezoidal and triangular contact pressures under strip foundations. These graphs give the values of pressure areas between two arbitrary levels.

During my stay at MIT, U.S.A., I had occasion to prepare further graphs for strip and circular foundations. Now there are pressure area graphs for 21 different cases, which facilitate the computations in the settlement analysis considerably.

A. BELES (Rumania)

Le problème de la réalisation de rideaux souterrains étanches se pose très souvent aux ingénieurs et de nombreuses méthodes ont été imaginées à cet effet.

A notre connaissance la solution que nous voudrions exposer n'a pas encore été appliquée mais, par les résultats favorables obtenus, par sa simplicité d'exécution et par son économie elle mérite d'être relevée.

Il s'agit d'une construction située dans une localité balnéaire — Ocna Sibiului en Roumanie — sur l'emplacement d'anciennes salines exploitées du temps des Romains. Dans les anciennes exploitations, des lacs d'eau salée s'étaient formés, jouissant dans le pays d'une grande renommée pour leur action curative.

Au bord d'un de ces lacs, un groupe de constructions a été construit en 1905. Il comprenait 3 édifices dont un le principal, était l'hôtel ayant un rez de chaussée, 3 étages et combles. Il était relié aux deux autres ayant seulement un rez de chausée et destiné l'un au restaurant et l'autre à l'administration et aux bains chauds.

L'hôtel a été fondé sur un radier en béton armé, formé de cloisons étanches de 2 mètres de hauteur.

Après un certain temps l'hôtel a commencé à se tasser et à

présenter des fissures peu importantes d'ailleurs et qui étaient réparées chaque année.

Au raccordement de couloir se produisirent des crevasses plus importantes qui marquaient l'affaissement de l'hôtel.

Vers 1948, les crevasses devenant plus inquiétantes, nous avons été appelé pour examiner la situation. Les recherches entreprises indiquèrent un tassement de 80 cm sur la façade donnant vers le lac et une déviation par rapport à la verticale de 40 cm au niveau de la corniche.

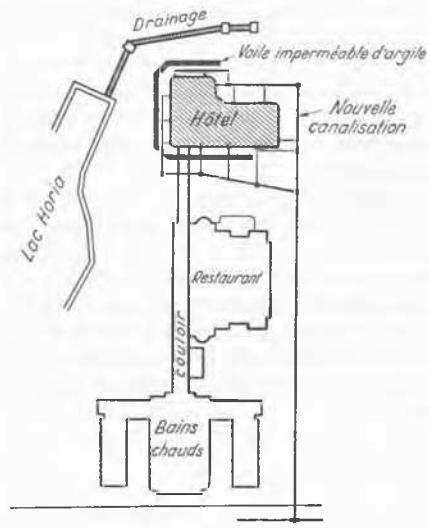


Fig. 25

L'étude du sous sol montra que l'édifice reposait sur le massif de sel qui entourait le lac dont le niveau supérieur présentait, du côté du lac, une arrête située à, environ, 6 mètres sous le niveau du terrain, tandis qu'à l'autre extrémité du bâtiment, le sel se trouvait à quelques 12 mètres de profondeur.

Une nappe d'eau douce qui se trouvait à environ 2 mètres de profondeur et qui s'écoulait des collines du voisinage produisait la dissolution du massif de sel, dissolution plus accentuée au

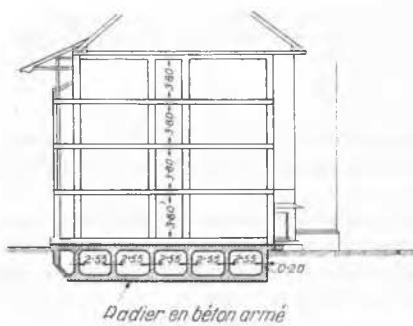


Fig. 26

niveau de l'arrêté du massif, ce qui expliquait l'inclinaison de la construction.

Des forages effectués indiquèrent que le terrain était formé par des couches très irrégulières composées de sable, de gravier et d'argile, et donnant des zones de perméabilité très variables.

Le niveau du lac se trouvant à environ 2 mètres au dessus de la crête du massif de sel et la nappe d'eau souterraine étant très riche, ni son rabattement, ni sa suppression n'étaient possible.

La seule solution, à notre avis, était de créer pour l'eau une zone de repos par l'exécution d'un voile étanche autour du bâtiment. L'eau immobilisée dans l'enceinte ainsi formée

atteignant le degré de saturation maximum n'aura plus d'action sur le massif de sel, et par suite les tassements devaient s'arrêter.

Vu les possibilités dont nous disposions à cette époque la solution recommandable était la création, à l'aide de forages encastrés dans le sel et injectés de ciment, d'un voile étanche. La solution fut ajournée jusqu'en 1954, quand, les tassements ayant pris une allure rapide, de fortes crevasses se produisirent.

Cependant, ayant eu l'occasion de faire certaines réparations aux sous-sols d'une usine de cellulose, et ayant observé que le

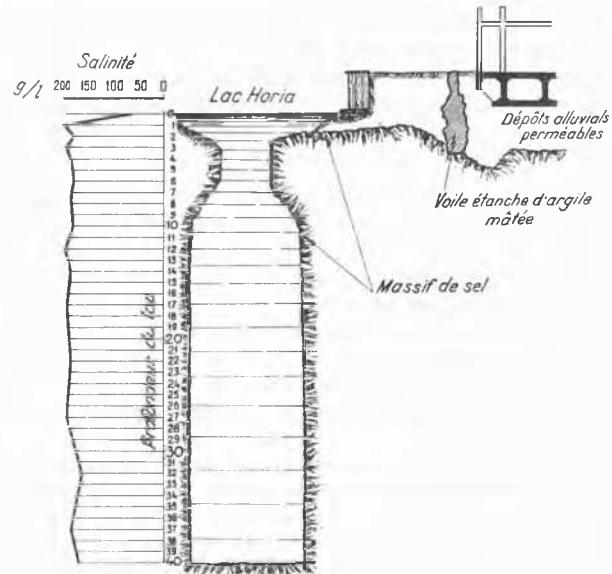


Fig. 27

terrains argileux imbibés de lessive de bisulphite de soude se révélaient imperméables, je me décidai, avec mon collègue et collaborateur l'ingénieur J. Stanculesco, d'examiner la possibilité d'utiliser un mélange d'argile et de lessive de bisulphite de soude pour réaliser l'étanchement des terrains perméables. Des études de laboratoires continuées pendant une année, nous montrèrent qu'un mélange d'argile fine, bien broyée et de lessive

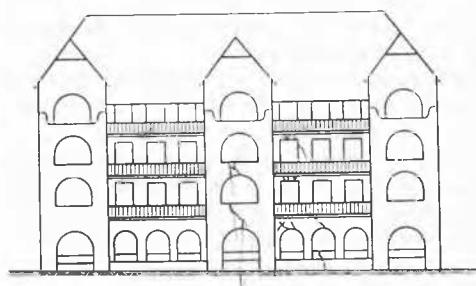


Fig. 28

de bisulphite de soude additionnée d'eau donnait un matériau très facilement injectable, étanche et résistant à l'entrainement par les courants d'eau, même salée.

Vu les résultats favorables obtenus au laboratoire et le coût réduit nous décidâmes d'employer ce matériau comme moyen d'étanchement.

Les travaux furent commencés en automne 1955 par l'exécution de forages disposés en quinconce et pénétrant solidement dans le massif de sel. Au fur et à mesure de leur exécution les forages furent remplis du mélange préparé au préalable et par matage, comprimé dans le sol.

Le projet prévoyait l'exécution d'un voile continu autour de

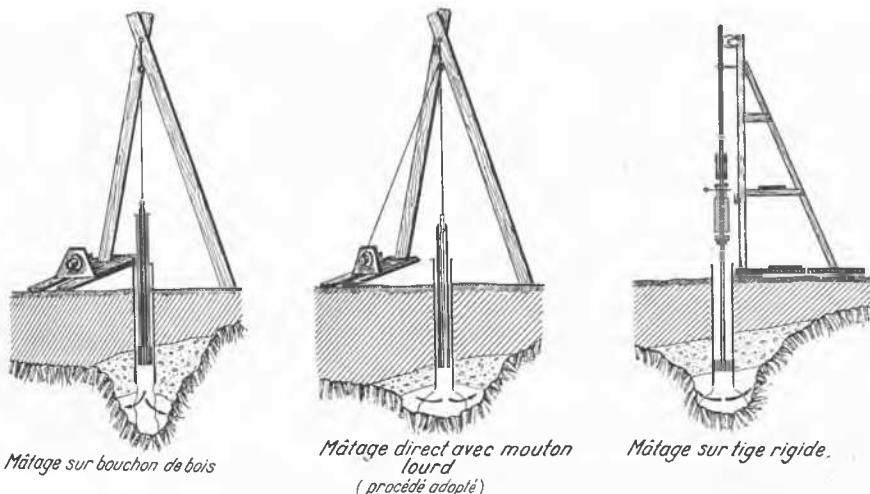


Fig. 29

l'hôtel, mais une partie seulement fut réalisée. Dès que la route de l'eau vers le lac fut barrée, les tassements se réduisirent, pour cesser ensuite complètement. Dans les derniers mois un niveling de précision n'indiqua plus de tassements perceptibles.

Pendant l'opération on put constater une grande pénétration du matériau injecté dans les différentes couches rencontrées et la réalisation d'un étanchement complet de la zone injectée. En ce qui concerne les matériaux utilisés nous remarquons que l'argile était du type utilisé sur les chantiers d'exploitation du pétrole et la lessive de bisulphite de soude était un déchet résultant de la fabrication de la cellulose; tous deux sont d'un coût très réduit.

Je crois avoir indiqué une méthode économique facile à réaliser et très simple pour l'exécution de voiles étanches et j'espère que le matériau indiqué ci-avant sera utilisé, après expérimentation, pour la réalisation de voiles étanches souterrains et aussi pour d'autres ouvrages.

A. R. JUMIKIS (U.S.A.)

The bulk of Paper 3a/30, 'Danger of Frost Heaving on Skating-rink Foundation', by R. PIETKOWSKI deals with thermal calculations; it contains very little about the danger of frost heaving mentioned in the title. From practical experience and from theoretical considerations the non-uniform heaves are more dangerous than the uniform ones. The author, calculating heaving as 4.4 cm, apparently assumed a uniform heaving throughout the entire freezing area. If the area is large in extent and the soil is of non-uniform structure and texture, then differential heaves may occur which would cause trouble not so much to the skating surface itself (unevennesses can be somewhat corrected by spreading and freezing more water on the surface) but rather to the freezing coils and the layers where the coils are installed.

At the end of the paper the author writes:

... Taking into consideration that capillary suction of water will not appear in clay in any remarkable degree owing to its very small permeability, we shall obtain a frost heaving of the soil. This heaving can be evaluated as . . . 4.4 cm. The phenomenon of frost heaving limited only to this figure would not present a menace to the skating-rink foundation, but the soil there was not uniform. If any water suction could take place, the freezing process would be slower, but heaving would increase.

There are a few comments I should like to contribute in connection with the above quotation.

(a) The author does not give any relative criteria as to when the heaving is dangerous and when not, or what is the magnitude of permissible heaving. It is difficult to see why the cal-

culated 4.4 cm (not quite 2 in.) heave would not present a menace to the foundation. On highways, a 2-in. uniform heave would not be considered dangerous to high-speed traffic, whereas a 2-in. differential heave may be disastrous. Now, if the freezing coils are made and installed so as to allow, say, a 10 cm dislocation, would such a heave be dangerous to the ice-skaters?

(b) As to the author's statement that capillary suction of water will not appear in clay in any remarkable degree owing to its very small permeability, I should like to indicate that although the process of soil moisture migration is slow, a considerable amount of soil moisture can flow upwards during a relatively long period of time under a thermal potential (a skating-rink would probably be in operation from December through February, or even all the year round). However, it is in the slow process of soil moisture flow, which is often overlooked and forgotten, where the danger of damage to highways, runways, and skating-rinks lies.

(c) The author does not indicate the position of the ground water table. If there is no ground water present or it is inaccessible, or the soil moisture transfer mechanism is ineffective, then the migration of soil moisture to the cold front is a drying process of the soil, and usually no frost heave troubles are experienced. However, if ground water is present, then, depending upon the method and rate of freezing, the following can happen. Upon quick freezing, no ice segregation takes place; upon slow freezing, or repeated freezing and thawing, considerable ice segregation takes place, resulting in moisture re-distribution (ice layers in soil may under some circumstances also act as an insulator). Heaving, however, is considerable in both instances, and if, in a large skating-rink, differences in soil types, texture, other soil properties and varying climatic conditions are encountered, differential heaves may occur. Differential heaves are causing pipes and coils to break, and can impair traffic and/or skating. The magnitudes of the heaves depend upon the depth of the ground water table from which water is supplied to the growing ice lenses.

The presentation of the thermal calculations for a dry layered foundation system where the freezing coils are installed are clear and appealing and may be of considerable practical value to those engaged in this kind of construction.

B. J. PRUGH (U.S.A.)

Paper 3a/39, Foundation of the Hampton Roads Tunnel, by S. STEUERMAN and G. J. MURPHY, gives an excellent description of an adequate subaqueous foundation. This was accomplished

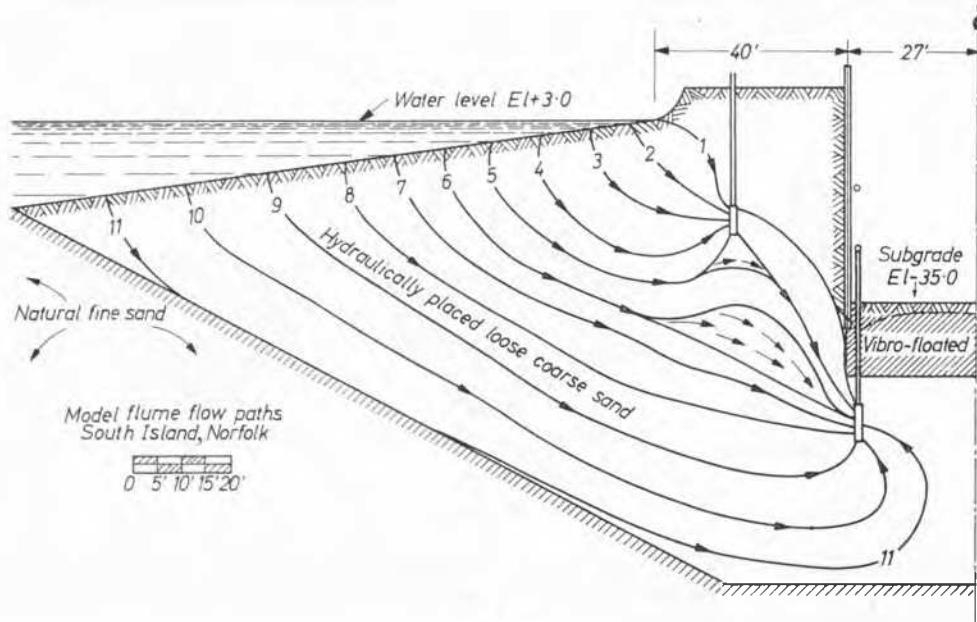


Fig. 30

by densification of the loose saturated sands under the tunnel portals by means of vibroflootation.

While densification by vibroflootation provided a compact sand with an average relative density of approximately 85 per cent, it simultaneously created, especially at the south portal, an unusual dewatering problem. Even without vibroflootation the dewatering problem was a serious one. An artificial island, created by hydraulically placed loose coarse sand, in an exposed position in practically the centre of Chesapeake Bay, was to be excavated, by the open cut method, 38 ft. below water level. Construction or working area was limited and protection against storm and wave action was provided by a single concrete breakwater and large-size rip-rap.

Boring investigations of the hydraulically placed coarse sand fill revealed that the permeability was in the range of 1,300 to 1,600 microns per sec with open water some 40 ft. from the edge of the excavation. Flow nets constructed in a model flume revealed that a conventional wellpoint dewatering system, with the screen sections located below subgrade and inside the sheeting line, would provide a suitable drawdown curve with a factor of safety if interlocking steel sheet piling was installed.

The contractor, however, elected to use vertical steel 'H' beams with horizontal wood plank lagging. The densification by vibroflootation produced an artificial vertical plug of compacted sand of low permeability exactly where the wellpoint screen sections should have a soil of high permeability.

Placing the wellpoint screen sections below the vibrofloated area in the loose sand eliminated the disadvantage. Flume models revealed that in this case the drawdown curve intercepted the line of sheeting above subgrade.

An impervious blanket on the outside of the island would have greatly lengthened the upper boundary of the flow lines. The economical placing and maintaining of such a blanket in the face of wave and storm action was questioned, as well as its effectiveness after storms.

The solution was in a continuously operating two-stage wellpoint system with the top stage designed to penetrate and depress the upper boundary limit and also to carry 40 per cent of the total volume of water pumped. You will note from Fig. 30 that under the top stage of wellpoints, the actual flow paths widened.

In actual operation the upper stage pumped 500 gal. per min.

per ft. of drawdown against the 525 gal. per min. computed from the flow nets. When both stages were in operation the flow gradually shifted to the lower set-up with part of the upper stage going dry. Investigation revealed very steep water gradients and silting up of the top of the outside island slopes due to the continuous large quantities of water being pumped.

From the actual drawdown curves, individual wellpoint tests and total volume pumped, the actual permeability of the loose sand was computed at 1400 microns per second and of the vibrofloated densified sand, 1000 microns per second.

In conclusion: when designing densified areas by vibroflootation, due consideration should be given to dewatering problems in adjacent areas.

P. J. ALLEY (New Zealand)

With the old methods of treatment of foundations being gradually replaced by new, it is refreshing to see that some of the old methods give good results. The Fellenius slip circle for foundations and Terzaghi's formula for settlement are instances of this.

A certain oil company proposed to erect an oil tank at a place called Lyttelton, in New Zealand, and complete subsoil investigations were carried out. The subsoil consisted of 20 ft. of reclaimed silts overlying harbour silty clay and clays to a depth of 80 ft. The value of stress was 2 to 3 lb./sq. in. As the strata varied greatly it was found expedient to use the natural water contents and liquid limit to derive the settlement. The Fellenius method was used to arrive at the stability by taking a foot stress through the centre of the tank. The tank was 78 ft. in diameter and it was hoped to find how much water could be placed in the tank safely. Calculations showed that only 20 ft. of water could be put in the tank for a start. The tank was filled to 20 ft., and there was a settlement of about 8 in. From that stage onwards the company did not believe the predictions of the engineer who was employed, so they thought they would fill up the tank to 30 ft.; they did so and nothing happened, except that a bit more settlement took place. They then went on to 33 ft. when suddenly the tank tilted—so they rapidly emptied it and thought better of soil mechanics afterwards! The settlement after one year was 18 in. The factor of safety used was 1.5.

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

Experimental investigations of the bearing capacity of sand show that with depth ratios of more than 1 to 1·5 but less than 3 in medium and dense sands there is considerable settlement of the foundation before failure with upheaval. This settlement is not dangerous provided it is determined beforehand and allowance is made in the design or construction. In the present case the load causing failure with upheaval is of little practical interest in foundation design: however, in such conditions one can observe a particular load the increase of which leads to more intensive settlement. This particular load corresponds to the moment when the gradually developing plastic zones reach the level of foundation base. In-so-far as further increase of load causes deformation and the gradual development of plastic zones in the overburden, this load can be termed the critical load. This critical load is considerably greater than the value of the first critical load determined by the usual methods known in soil mechanics. With a depth ratio 3 to 4 there are no characteristic points on the settlement curve.

Thus, foundations of different depths can be classified into three groups. First, shallow foundations where the increase of settlement is sudden and is followed by failure with upheaval. That is for a depth ratio lying between 0 and 1 to 1·5. Secondly, deep foundations where the increase of settlement is not followed by failure with upheaval, but is characterized by gradual deformation of overburden. That is for the depth ratio lying between 1 to 1·5 and 3 to 4. Thirdly, very deep foundations, with the depth ratio more than 3 or 4, where settlement increases gradually but is non-linear.

For the first group the main point is the determination of limit load by the method of limit equilibrium theory. For the second group it is necessary to determine the critical load when the plastic zones reach the base level and also to determine the amount of settlement. For the third group, the main point is the determination of the amount of settlement. With such an approach to its evaluation it is possible to assume considerably greater values for soil bearing capacity. The most important problem in future theoretical investigations is the development of methods for the determination of settlement values with non-linear relationship between load and deformation.

R. V. WHITMAN (U.S.A.)

The question of the proper means for the determination of a modulus of deformation was raised at this afternoon's session, as it had been earlier in sessions 1 and 2. In this respect, one of the most important comments made by many of the speakers is, I believe, the necessity for some degree of pre-stress or pre-loading to be given to the sample if a correct or proper determination of this modulus of deformation is to be obtained.

I should like to mention two instances in which it has proved possible to correlate a modulus of deformation obtained from a triaxial or an oedometer test with measurements of the seismic wave velocity. The first of these examples involved a uniform deposit of silt at a moisture content below the shrinkage limit. In this instance the seismic velocity was determined both by the usual geophysical sounding methods and also by the observation of time lags in the transmission of some large intensity pressure waves resulting from blasting. I might add that these blasting waves were transmitted downwards through the soil in what might be called a one-dimensional pattern. In this case it proved possible to obtain a good correlation between the observed seismic velocity and the velocity computed from the results of oedometer tests using the slope of the pre-loading curve; in other words, using a modulus of deformation obtained after a pre-loading had been applied to the sample and removed.

Another instance of such correlation involved a dry sand

with very uniform rounded soil particles. In this case the seismic velocity was determined in a laboratory experiment upon a long sample where it was possible to observe the time lag for transmission of the wave from one end of the sample to another (see Paper 1b/15). Once again it proved possible to correlate this velocity with the value calculated from a modulus of deformation obtained in a triaxial test. Once again a considerable degree of pre-loading—several cycles of it, as a matter of fact—had been applied to the sample.

The fact that this correlation could be obtained with the modulus of deformation which resulted after the pre-loading or pre-stressing is to me a demonstration of the necessity of using some sort of pre-stress.

It has been suggested that the low values of the modulus of deformation obtained in laboratory tests without pre-loading may be the result of sample disturbance. I think that perhaps there is a particular kind of sample disturbance present; namely, the soil at the two ends of the sample, whether it be a triaxial sample or an oedometer sample, has a looser structure than the main bulk of the sample. If one is dealing with an artificially prepared sample, the upper layer where the compaction has ceased will not possess the dense properties of the remainder of the sample. If one is dealing with a sample of natural soil, I believe that the significant disturbance occurs right at those end surfaces where the sample has been trimmed before placing it in the testing machine. If this is true—that the disturbances are of this particular type, so that it makes no difference whether one starts with a bore tube sample or a sample cut from a chunk—it means that the errors are almost unavoidable if one attempts to measure a modulus of deformation during the first loading; hence some degree of pre-stress or pre-loading must be used. The choice of the proper magnitude for the pre-loading is a question that needs considerable further study.

O. MORETTO (Argentina)

L. E. Collins of South Africa has misunderstood what I said in relation to the type of foundation to be used to avoid swelling pressures. I did not say that one uses deep foundations; I said that one places the building in the air. The ground floor is placed on the air so that there is no contact between the building and the ground except through the columns which concentrate loads at certain points on the ground.

In the paper which I presented to the conference in Brazil, and which I mentioned in my contribution, I made a critical study of those solutions which pretend to solve the problem by taking the foundations to a relatively deep layer. I also had exactly the same experience as L. E. Collins of seeing piles broken by extension due to the pressure of the soil under the beams which sustained the building. That experience was the one which gave me the idea of placing an air chamber under the beams.

E. DE BEER (Belgium)

Pour la définition de la marge de sécurité, il faut distinguer entre le problème de l'équilibre limite de rupture et celui de l'équilibre limite de déformation. Je crois que le problème posé par le rapport général concerne essentiellement le problème de l'équilibre limite de rupture.

Si on se limite à ce dernier problème, il nous semble qu'il est préférable d'introduire la notion du calcul à la rupture telle qu'elle est en train de se généraliser dans les ouvrages en béton et en béton précontraint.

En principe, il y a deux possibilités d'introduire une marge de sécurité, notamment, soit par rapport aux forces résistantes, soit par rapport aux forces agissantes.

Les forces résistantes sont des fonctions de la cohésion et de

l'angle de frottement. Mais ces fonctions sont souvent loin d'être linéaires. Ainsi, dans de nombreux cas, ils sont une fonction exponentielle de l'angle ϕ . En appliquant un coefficient de sécurité à la cohésion ou à $\tan \phi$, on obtient une marge de sécurité qui est fonction de la valeur absolue de ces grandeurs. C'est pourquoi, je puis difficilement me rallier à une généralisation de la méthode consistant à appliquer un coefficient de sécurité S à la cohésion et à $\tan \phi$, par exemple, dans les cas d'ouvrages de soutènement.

Les grandeurs C et ϕ à introduire dans les calculs, doivent être déduites d'études statistiques et de probabilité comme A. Lazard l'a exposé.

En ce qui concerne la marge de sécurité de l'ouvrage lui-même, elle peut être déterminée par le rapport de la valeur des forces sollicitantes provoquant l'état de rupture par rapport à la valeur maximum qui doit être prise en considération pour la construction.

Comme je l'ai dit au début, la question du problème des déformations doit être résolu séparément.

A. BELES (Rumania)

En relation avec la communication de I. M. Litvinov de l'URSS, concernant le traitement thermique des terrains, je me permets de signaler au congrès que nous avons développé en Roumanie, une méthode simple et économique de traitement thermique du sol qui fera l'objet d'une communication à l'occasion des discussions de la IX^e séance, 6 ème section technique.

P. HABIB (France)

Dans les discussions des communications de la section 3a, l'accent a été mis à plusieurs reprises sur l'influence de la rigidité

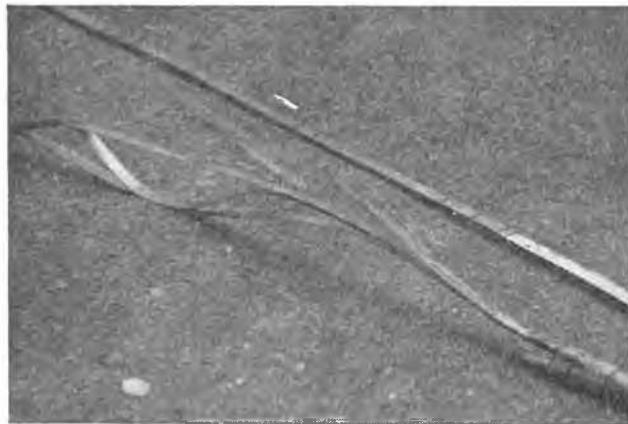


Fig. 31 Dynamomètre à friction
Friction dynamometer

de la structure sur la répartition des contraintes sous la fondation. Avec les instruments de mesure de contraintes dans le sol, les vérifications directes sont difficiles à obtenir car la mise en œuvre et l'utilisation de ces appareils sont délicates. Aussi je voudrais signaler un dispositif de mesure de contraintes que nous avons eu l'occasion de mettre au point pour des modèles réduits de grandes dimensions et qui semble directement utilisable pour les fondations.

Le principe de cet appareil est celui du dynamomètre à friction. Trois lames métalliques fines sont empilées les unes sur les autres: l'effort pour faire glisser la lame intermédiaire est fonction linéaire de la charge normale que supporte le dispositif.

De nombreux essais préparatoires nous ont conduits à lui donner les dimensions suivantes:

longueur active	60 cm
largeur	6 mm (pour éviter les effets de voute)
épaisseur totale	6/10 mm

La lame centrale en acier coulisse avec des mouvements de va-et-vient dans le sens de la longueur. L'ensemble est lubrifié avec de la graisse silicone et est placé dans une enveloppe en matière plastique fine pour assurer une étanchéité suffisante. A chaque extrémité un blindage de garde abrite le dispositif de traction et de mesure.

Les résultats que nous avons obtenus ont été très satisfaisants dans les sables mais comme nos essais n'ont pas duré plus de dix jours, nous craignons, avec le dispositif original, qu'une légère corrosion modifie le coefficient de frottement de la lame centrale. Nous réalisons actuellement le même appareil, en acier inoxydable traité en surface, pour le placer sous des fondations.

Je pense que ce dispositif, par sa simplicité, peut rendre de grands services pour la détermination des charges effectivement appliquées sur le sol.

A. DVOŘÁK (Czechoslovakia)

I should like to refer briefly to the contribution by R. V. Whitman (U.S.A.), concerning the seismic estimation of deformation modulus in soils.

If the seismic velocity in cohesionless soil such as sand is measured we may obtain very great differences depending upon the amount of water in the voids of the soil. In water-saturated sands the velocity of seismic waves is much higher than in dry sand, whereas the static compressibility is the same. In a construction we have to deal with static loads and this fact must be considered.

In clays saturated with water the velocity of seismic waves is very great, for with such instantaneous micro-stresses the soil has the properties of a medium with a modulus of elasticity many times higher than in slow static tests.

P. E. RAES (Belgium)

Monsieur le Président, Messieurs: Ce matin mon collègue, E. de Beer, a présenté quelques observations au sujet de ma contribution à notre congrès. Cette contribution est fort modeste; elle consiste à dire que, depuis que nous disposons d'excellentes formules de fondation, découvertes par de brillants mathématiciens dont il est inutile que je vous rappelle les noms, il est superflu de se servir encore de formules simplifiées sujettes à de nombreuses erreurs. Parmi ces formules simplifiées j'ai cité celle d'Andersen qui a pu rendre certains services en son temps et qui est basée sur une ligne de glissement composée de deux arcs de cercles.

Quand on se donne la peine d'examiner de quelle façon cette formule a été établie, on est un peu étonné de constater que, pendant dix ans au moins, personne n'a remarqué qu'elle était non seulement entachée d'erreurs d'hypothèse, mais aussi d'erreurs de calcul et de statique élémentaire.

Ce matin, E. de Beer, que je n'ai pas cité dans mon article, a défendu cette formule dont il n'est pas l'auteur, mais qu'il a introduite dans le diagramme qu'il a construit patiemment avec le talent qu'on lui reconnaît et la grande puissance de travail qui le caractérise. Il était arrivé à diviser le sol en deux secteurs. L'un d'eux était réservé à la formule de Buisman, qui suppose que la ligne de glissement est une spirale logarithmique prolongée par deux droites, et un autre à la formule d'Andersen, adoptée tout simplement parce qu'elle donnait des résultats situés 'on the safe side' par rapport à la première expression.

Plus tard, constatant que les courbes de Buisman et d'Andersen se coupaient dans le diagramme suivant un angle, il intercala un troisième secteur qui reflète la formule de Mizuno. Finalement, le sol devrait donc se comporter dans un secteur suivant Buisman, dans un autre suivant Mizuno et dans un troisième suivant Andersen.

Evidemment, comme E. de Beer est un excellent expérimentateur, il a comparé son diagramme aux résultats des essais connus, et il a attribué à la formule d'Andersen le secteur où, par hasard, elle se comportait assez bien, ce qui ne signifie naturellement pas que cette formule soit exacte.

Notre collègue a prétendu ce matin que l'exemple traité dans mon exposé est inadmissible parce que les terres y sont supposées non-pesantes et, d'autre part, qu'il n'est pas exact que la formule d'Andersen donne toujours des résultats trop faibles. Puisqu'on a bien voulu attirer votre attention sur mon article que beaucoup d'entre vous n'ont certainement pas remarqué, je vous recommande de le lire, et vous verrez que je n'ai rien affirmé de semblable. J'ai simplement écrit que, en raisonnant comme Andersen, on pouvait obtenir un résultat beaucoup trop faible, comme le prouve l'exemple en question. Mon but était de démontrer que les formules basées sur des lignes de glissement arbitraires sont nécessairement fausses et qu'il est inutile d'en faire une sorte de sandwich compliqué alors que nous possédons des formules mathématiques remarquables.

Ce que E. de Beer n'a pas dit, c'est que la plus grosse erreur, qu'il n'a pas relevée et que je suis bien forcée de souligner maintenant, se trouve dans l'équation de moment écrite par Andersen. En effet, si on se réfère à ma Fig. 1, que E. de Beer a eu l'amabilité de reproduire au verso du tableau, on constate que, pour éliminer les tensions normales agissant sur l'arc AE , le point C a été choisi comme centre des moments, tandis que, pour éliminer celles agissant sur ED' , le centre est situé en C' . Je n'ai jamais vu ailleurs une équation d'équilibre où les moments des diverses forces sont pris autour de centres différents, sauf si ces centres sont situés sur une parallèle à la résultante de translation. Or, quand on calcule les réactions R et R' qui résultent des contraintes (inexactes) supposées par Andersen, on constate qu'elles sont fortement inclinées dans le même sens sur la direction CC' . Si l'on prend C pour centre du moment de R , le moment de R' doit donc être pris également autour de C et, en le prenant autour de C' , on commet une erreur qui n'est autre que le moment du vecteur R' appliqué en C , autour de C' , et qui devient énorme quand la profondeur CC' augmente. Il en résulte entre autres que, pour une largeur B nulle, c'est-à-dire pour un massif sans épaisseur (et donc fort économique) on obtient une force portante aussi grande que l'on veut, à condition de descendre la fondation à une profondeur suffisante. On peut difficilement prétendre qu'il s'agit là d'un résultat trop faible.

Si on cherche, par contre, la largeur B du massif en fonction de la profondeur D , pour une charge de rupture unitaire constante, on trouve une courbe qui présente un maximum de D , de sorte que, pour une profondeur donnée, on obtient deux valeurs de B différentes. En conclusion, je dirai que la formule d'Andersen ne mérite vraiment pas d'être défendue.

J. A. J. SALAS (Spain)

We have heard today about two different classes of heaving, that caused by frost and that caused by the swelling of clay. Although different in origin they are similar in their effects on structures, and, as A. R. Jumikis has pointed out, in this session, the important thing for the engineer is, in general, not the total heaving but the differential one.

A very interesting discussion has been contributed to this session by the South African delegates. I would like to know if it is possible to predict with the new method of calculation that J. E. Jennings has announced the probable magnitude of the possible differential heaving. Without this estimate the results of the calculations will have a very limited practical value. Furthermore, even if we know the probable magnitude of the differential heaving, its influence on the structure remains to be estimated. This is a very difficult problem as has been demonstrated by the numerous contributions on this particular point; it is not a problem of elasticity but of plasticity, involving the creep of the building materials.

If there is no more conclusive data about this question of the effect of the differential heaving on buildings, I think that a more sound approach to the problem is that which I have proposed in the paper which I have presented to the conference. I must add that the formula developed in my paper was inspired by the South African formula.

The Chairman

I now call on the Assistant Reporter to sum up the discussion that has taken place.

Assistant Reporter

I have only a few remarks to make. The discussion this afternoon has covered a very wide range of problems so that I think you will agree with me that it would be quite impossible for me to give a general summary. I am very much afraid that the only contribution I could make to such a summary would consist of a reference to the General Report of this and the following conference.

The conference adjourned till Thursday, 15 August, at 10 a.m.

E. DE BEER and J. WALLAYS (Belgium)

In his reply, P. E. Raes drew attention to two imperfections of the method of Andersen which he had not mentioned in his paper. First, he showed that the equation of the moment equilibrium has been written by taking two different poles. When foundations very near to the surface, or on the surface, are considered, which are the only cases of practical interest, this imperfection decreases rapidly or even disappears. Furthermore, it can be shown that the error arising from considering two poles gives for the bearing capacity a calculated value Q_c which is greater than the exact value Q_e . Thus this imperfection points again to the fact that the method of Andersen will tend to give too high values, and supports the reasoning indicated in the previous discussion.

Secondly, it is stated that for $B=0$, the bearing capacity becomes infinite; but when B is zero, the depth factor

$$\frac{p_0}{\gamma B} = \frac{\gamma D}{\gamma B} = \frac{D}{B}$$

becomes infinite. We have already shown that in this case the method of Andersen gives $N_q = \eta$, and this has no longer any practical meaning.

The two new objections which have been put forward do not improve the reasoning made in our previous discussion which indicates why in the range of the low values of $(p_0/\gamma B)$ the method of Andersen gives results which do not differ much from those found many years later by Meyerhof and Caquot-Kérisel.