A CONTRIBUTION TO THE SETTLEMENT ANALYSIS OF FOUNDATIONS ON CLAY

by

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SYNOPSIS

The settlement of a foundation on a saturated clay is composed chiefly of the "immediate" settlement due to deformations taking place at constant volume, and the "consolidation" settlement due to volume reduction consequent upon the dissipation of pore pressures. This latter component is therefore dependent on the pore pressures set up by the foundation load, and these pore pressures are themselves dependent on the type of clay. An approximate theory is described which takes account of the type of clay; and comparisons with observed settlements in a number of practical cases show the theory to be an improvement on existing methods of calculation.

Le tassement d'une fondation sur une argile saturée se compose principalement du tassement "immédiat" dû aux déformations ayant lieu suivant une ampleur constante, et le tassement "de consolidation" dû à la diminution d'ampleur par suite de la dissipation de la pression interstitielle. Cette dernière composante dépend donc des pressions interstitielles provoquées par la charge de fondation, et ces pressions interstitielles dépendent elles-mêmes du type d'argile. On décrit une théorie approximative qui tient compte du type d'argile; et des comparaisons avec les tassements observés dans un nombre de cas pratiques démontrent cette théorie comme étant une amélioration des méthodes de calcul existantes.

INTRODUCTION

If we imagine a footing on a saturated clay to be loaded quite rapidly, then during the load application the clay will be deformed and pore pressures will be set up in the clay. Owing to the extremely low permeability of clays little if any water will be squeezed out of the clay during the load application, and the deformations therefore take place without change in volume. The deformations have both lateral and vertical components, and the vertical component constitutes what is known as the "immediate settlement".

In the course of time, however, some of the pore-water drains out of the clay, leading to a volume decrease, and the vertical component of this volume change is known as the "consolidation settlement".

Now the consolidation of a clay results from the dissipation of pore pressure, with an accompanying increase in effective pressures. But a given set of stresses will cause different pore pressures in different clays. Thus if we have two identical footings, carrying identical loads, and these footings rest on two clays with identical compressibilities, yet if the pore pressures set up in the two cases are different, the consolidation settlements will also be different. And this is true in spite of the fact that no difference would be seen in the results of the oedometer test.

This may seem paradoxical. But the explanation is that in the oedometer test no lateral strains are permitted and, under this special condition, the pore pressure set up in a saturated clay by an applied pressure is always equal precisely to that applied pressure irrespective of the type of clay; provided only that it is fully saturated.

It also follows that if, in practice, the conditions are such that no lateral strains can take place during the load application then, other things being equal, the pore pressures will be the same in all saturated clays, and the consolidation settlement (there will be no immediate settlement if there can be no lateral strains) will be directly proportional to the compressibility of the clays. The condition of no lateral strain is approximately true for at least two practical cases: (a) that of a thin layer of clay lying between beds of sand or between sand and rock, and (b) that of a loaded area of horizontal extent which is great compared with the thickness of the underlying clay, when the lateral strain will be negligible except near the edges of the loaded area. In cases such as these the consolidation settlement can be estimated with

reasonable accuracy by a direct application of the oedometer test results, and it is exactly for such cases that Terzaghi (1925) developed his one-dimensional theory of consolidation, the essential data for which are derived from the oedometer.

But in the more general case where lateral deformations can occur the pore pressures set up by the stresses depend upon the type of clay, as well as on the stresses themselves; and the consolidation settlements therefore also depend on the type of clay. Any method of calculating consolidation settlements which does not enable some allowance to be made for this effect is bound to be unsatisfactory in principle.

In 1939 the senior Author developed a theoretical approach to this problem but it was expressed in fundamental parameters which cannot readily be measured in ordinary laboratory tests. More recently the concept of pore pressure coefficients has been introduced which, together with simplifications in the analytical work, has made possible a solution yielding results of practical value.* Philosophically, the solution should be regarded as illustrative or semi-empirical.

IMMEDIATE SETTLEMENTS

By definition, the immediate settlement takes place without dissipation of the pore pressures. Consequently it is possible to measure the relevant properties of the clay in an undrained triaxial test.

According to the theory of elasticity, the settlement of a loaded area is given by the classical expression:

$$\rho = q \cdot b \cdot \frac{1-\nu^2}{E} \cdot \mathrm{I}_\rho \tag{1}$$

where
q = net foundation pressure
b = breadth or diameter of the loaded area
ν = Poisson's ratio
E = Young's modulus
I_ρ = Influence value, depending on the shape of the loaded area and the depth of the clay bed.

For saturated clays there is no volume change so long as there is no dissipation of pore pressure. Consequently in the calculation of immediate settlements $\nu = 0{\cdot}50$. The value of E can be found from the stress-strain curve obtained in the undrained triaxial test, although experience has shown that E is sensitive to sampling disturbance, especially in normally consolidated clays, and a correction may often be necessary (for example, Peck and Uyanik, 1955; Simons, 1957)

CONSOLIDATION SETTLEMENTS

If $\Delta\sigma_1$ and $\Delta\sigma_3$ are the increases in the principal stresses at any point, caused by loading the footing, then the excess pore pressure set up in the clay at this point may be represented by the expression

$$u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \tag{2}$$

where A and B are the pore-pressure coefficients (Skempton, 1954).

For saturated clays $B = 1$. The value of A can be determined from pore pressure measurements in undrained triaxial tests (for details, see Bishop and Henkel, 1957). In general, the coefficient A is not a constant for a given clay, but depends on the magnitude of the applied stresses. Nevertheless for our present purpose a range of values can be quoted for various

* Preliminary statements outlining this solution, and giving equation (9) of the present Paper, were published by the Authors in 1956 (see References on p. 178)

types of clay, as in Table 1. The value of A depends primarily on the geological history of the clay.

Table 1

Typical values of the pore pressure coefficient A for the working range of stress below foundations

Type of clay	A
Very sensitive soft clays	>1
Normally-consolidated clays	$\frac{1}{2}$–1
Overconsolidated clays	$\frac{1}{4}$–$\frac{1}{2}$
Heavily overconsolidated sandy clays	0–$\frac{1}{4}$

For simplicity in the analysis only points on the axis of symmetry below a foundation will be considered. The directions of principal stress are then vertical and horizontal.

If at the point under consideration p_1 is the vertical effective stress before the foundation load is applied, then the vertical effective stress immediately after load application is :

$$\sigma_1' = p_1 + \Delta\sigma_1 - u$$

As the pore pressure gradually dissipates to zero during the consolidation process, Poisson's ratio (in terms of total stresses) decreases from 0·50 to some smaller value. But this has little effect on the vertical stresses and therefore, so long as the foundation pressures have not changed, the vertical effective stress when consolidation is completed is :

$$\sigma_1' = p_1 + \Delta\sigma_1$$

Hence the change in vertical effective stress during consolidation is equal to u, and this is wholly an increase above the original stress p_1.

If p_3 is the horizontal effective stress before the foundation load is applied, then immediately after load application the stress is :

$$\sigma_3' = p_3 + \Delta\sigma_3 - u$$

But u is greater than $\Delta\sigma_3$ (see equation 2). Hence the horizontal effective stress is reduced by the load application. During the earlier stage of consolidation, as the pore pressure dissipates, the clay is therefore subjected to a recompression in the horizontal direction, under a stress increase equal to $(u - \Delta\sigma_3)$. Owing to the comparatively low compressibility of clays in recompression the strains associated with this effective stress increase are small. However, in the later stage of consolidation, after the horizontal effective stress has regained its original value p_3, the clay will be subjected to a net increase in horizontal effective stress, from p_3 to $p_3 + (\Delta\sigma_3 - \delta\sigma_3)$, where $\delta\sigma_3$ is the decrease in horizontal total stress due to the decrease in Poisson's ratio as consolidation takes place.

Numerical investigations of an approximate nature show that as a consequence of the foregoing effects, the lateral strains during consolidation are so small that they may be neglected without involving an error of more than roughly 20% in the value of the vertical consolidation movements. And this result holds good no matter how important the lateral strains may have been during the " immediate " stage, when the clay was deforming under undrained conditions.

Now, in the oedometer test the vertical compression of the clay is measured under the condition of no lateral strain and if a vertical compression ρ is caused by consolidation, in this test, under an increase in effective pressure $\Delta\sigma'$, then :

$$\rho = m_v . \Delta\sigma' . h$$

where h is the thickness of the sample and m_v is defined as the coefficient of compressibility in the oedometer test.*

Hence, since the consolidation of an element of clay, beneath a foundation, takes place without appreciable lateral strain, the vertical compression of an element during consolidation can be expressed approximately by the analogous equation :

$$d\rho_c = m_v . u . dz$$

where dz is the thickness of the element. And the consolidation settlement ρ_c of the centre of a foundation, resting on a bed of clay of thickness Z is therefore :

$$\rho_c = \int_0^z m_v . u . dz \qquad (3)$$

But, from equation (2), with $B = 1$,

$$u = \Delta\sigma_1\left[A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1 - A)\right]$$

Hence :

$$\rho_c = \int_0^z m_v . \Delta\sigma_1\left[A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1 - A)\right] dz \qquad (4)$$

This expression completes the first part of our investigation, for the " immediate " and " consolidation " settlements are given in equations (1) and (4) in terms of soil properties which can be measured in ordinary laboratory tests.†

CONSOLIDATION SETTLEMENTS : A FURTHER SIMPLIFICATION

It is possible, however, to reduce equation (4) to a more practical form.

In the case of one-dimensional consolidation, in which the lateral strains are zero throughout the loading period as well as subsequently, the settlement is :

$$\rho_{oed} = \int_0^z m_v . \Delta\sigma_1 . dz \qquad (5)$$

This settlement is given the suffix *oed* to indicate that it is the value obtained by a straightforward application of the oedometer test results. It can be readily computed as a matter of routine, and we may repeat that for thin layers of clay, or for loaded areas that are wide as compared with the thickness of the underlying clay, the settlement given by equation (5) is a reasonable approximation to the total settlement which, in these cases, is composed almost entirely of consolidation.‡

But a comparison of equations (4) and (5) will show that there is a broad similarity between the " oedometer " settlement and the approximate " consolidation " settlement for the general case of a footing on a deep bed of clay. And it seems reasonable to postulate that the two can be related by a factor μ as shown in the following equation :

$$\rho_c = \mu . \rho_{oed} \qquad (6)$$

If the value of μ can be found in any particular case without resource to elaborate testing or

* For corrections to be applied to the oedometer test results to allow for sample disturbance, see Terzaghi and Peck (1948).

† In general there will also be a " secondary consolidation ", additional to the " immediate " and the " consolidation " settlements. But in most clays the secondary consolidation is small and, moreover, it in no way affects the problem considered here.

‡ Even where the foundation bears directly on a thick bed of clay the final settlement is often given with tolerable accuracy by ρ_{oed}. This is known as the " conventional " method of calculating final settlements. It has been discussed in detail, with references to practical examples, by MacDonald and Skempton (1955), and by Skempton, Peck, and MacDonald (1955).

computations, then the consolidation settlement can be derived quite simply from the standard calculation of ρ_{oed}.

From equations (4) and (5) it follows that :

$$\mu = \frac{\int_0^z m_v . \Delta\sigma_1 \left[A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1 - A) \right] . dz}{\int_0^z m_v . \Delta\sigma_1 . dz} \quad . \quad . \quad . \quad (7)$$

And by assuming constant values of m_v and A with depth, it is possible to express μ by the simple equation :

$$\mu = A + \alpha(1 - A) \quad . \quad . \quad . \quad . \quad (8)$$

where

$$\alpha = \frac{\int_0^z \Delta\sigma_3 . dz}{\int_0^z \Delta\sigma_1 . dz}$$

The coefficient α depends only on the geometry of the problem, since Poisson's ratio is 0·5 for all saturated clays during load application ; and this coefficient has been computed for circular and strip footings, with various ratios of the thickness of clay Z to the breadth of footing b. The results are given in Table 2.

Table 2

Values of α in the equation $\mu = A + \alpha(1 - A)$

Z/b	Circular footing α	Strip footing α
0	1·00	1·00
0·25	0·67	0·74
0·5	0·50	0·53
1	0·38	0·37
2	0·30	0·26
4	0·28	0·20
10	0·26	0·14
∞	0·25	0

Values of μ have been plotted against the pore-pressure coefficient A in Fig. 1 with the ratio Z/b as an independent parameter. Reference to this graph will show that for normally-consolidated clays the factor μ is typically rather less than 1, while for overconsolidated clays μ is in the region of $\frac{1}{2}$. In the more extreme cases of heavily overconsolidated sandy clays μ can be as low as $\frac{1}{4}$, and in very sensitive clays the consolidation settlement can even exceed the oedometer settlement, that is, μ is greater than 1·0.

The pore-pressure coefficient, which itself depends greatly on the geological history of a clay, is thus seen to be a vital factor in settlement analysis.

PRACTICAL PROCEDURE

It must be remembered that this analysis has been derived in terms of the consolidation settlement of the centre of a foundation. But it does not seem unreasonable to assume that the results apply, broadly, to other points. If this be granted, then the procedure to be used in practice may be summarized as follows :

The net final settlement is :

$$\rho_{final} = \rho_i + \rho_c$$

and, in some clays, the " secondary " settlement must also be considered.

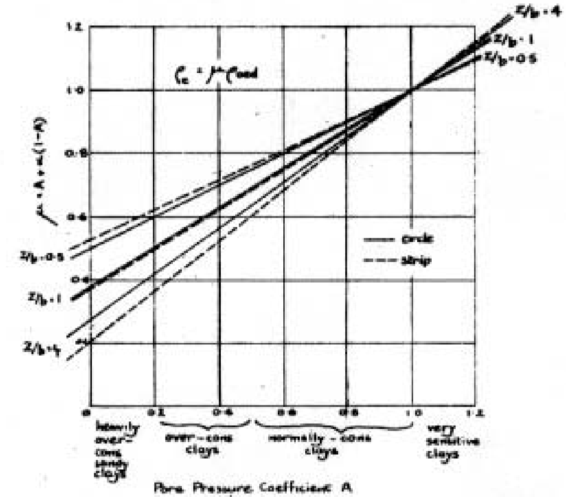

Fig. 1. Values of the factor μ

The net " immediate " settlement ρ_i can be calculated for any point in a foundation, on saturated clay, from the equation :

$$\rho_i = q . b . \frac{3}{4E} . I_p$$

where I_p is the appropriate influence value, as given by Steinbrenner (1934), q is the net pressure, b is the width of the foundation, and E is determined from undrained compression tests with a correction for sampling disturbance if necessary.

The net " consolidation " settlement ρ_c can be calculated from the equation :

$$\rho_c = \mu . \rho_{oed}$$

where

$$\rho_{oed} = \int_0^z m_v . \Delta\sigma_1 . dz$$

and

$$\mu = A + \alpha(1 - A)$$

Values of $\Delta\sigma_1$ have been tabulated by Jurgenson (1934), Newmark (1935), and others. Oedometer tests, again with corrections if necessary, give m_v ; and A can be found from undrained triaxial tests with pore pressure measurements. Values of α are given in Table 2. For many purposes, however, a knowledge of the geological history of the clay together with Fig. 1, is sufficient to enable an approximate value of μ to be chosen.

The net settlement at any time t after the load application is :

$$\rho_t = \rho_i + U . \mu . \rho_{oed} \quad . \quad . \quad . \quad . \quad (9)$$

where U is the degree of consolidation as evaluated from the theory of consolidation.

Finally, it may be noted that if the loaded area is very wide compared with the thickness of clay, or if the clay exists as a layer between beds of sand, the influence value I_p tends to zero and the factor μ tends to unity. Hence, in these cases :

$$\rho_t = U . \rho_{oed} \quad . \quad . \quad . \quad . \quad (10)$$

This equation expresses Terzaghi's theory of one-dimensional consolidation, which applied strictly to the cases mentioned above. Equation (10) is also the basis of the " conventional " method of settlement analysis for footings on clay.

EXAMPLES OF APPLICATION

The settlements of an oil tank 144 ft in diameter, on 90 ft of normally consolidated silty clay, have been published by Cooling and Gibson (1955) together with the calculated settlements. The observed immediate settlement at the centre of the tank was just over 2 in. and the consolidation settlement, although not quite complete at the time of publication, could be extrapolated to a final value of about 19 in.

The calculated immediate settlement was 3 in., and the oedometer settlement was 18·5 in. The clay had a moderate sensitivity and A was about 0·65.* With $Z/b = 0{\cdot}63$, $\alpha = 0{\cdot}46$ and the value of μ is 0·8. Consequently the calculated consolidation settlement is equal to $0{\cdot}8 \times 18{\cdot}5 = 15{\cdot}0$ in. The comparison between observed and calculated settlements are set out in Table 3 and the agreement is reasonably satisfactory.

Table 3

Oil tank, Isle of Grain. Comparison of calculated and observed settlements at centre of tank

	Calculated (in.)	Observed (in.)
Immediate	3	2
Consolidation	15	19
Final	18	21

Settlement records have also been published, with the appropriate soil properties, for three buildings founded on the normally-consolidated Chicago clay (Skempton, Peck, and MacDonald, 1955). In those cases it is not possible to deduce the observed immediate settlement with precision, but the final settlements are known with some accuracy.

The clay has a depth of roughly 50 ft and the buildings a width of the order of 100 ft. The ratio Z/b is thus about 0·5 and hence α is also about 0·5. The A value for Chicago clay is probably in the region 0·7 to 0·9, and consequently μ lies between 0·85 and 0·95. We will take $\mu = 0{\cdot}9$, and the calculated consolidation settlements are then as given in Table 4. The immediate settlements are those quoted in the above-mentioned Paper.

Table 4

Calculated and observed final settlements of three buildings on Chicago Clay

(Settlements in in.)

	ρ_i	$\rho_c = 0{\cdot}9\,\rho_{oed}$	ρ_{final} calc.	ρ_{final} observed
Masonic Temple	3	8	11	10
Monadnock Block ..	6	14·5	20·5	22
Auditorium tower ..	6·5	19·5	26	24

The calculated final settlements are seen to be in good agreement with the observations. And, although the rate of settlement is not our present concern, it is interesting to note that

* This value of A has been derived from a preliminary analysis of pore pressure observations at three different depths in the clay beneath the tank (private communication from Dr R. E. Gibson).

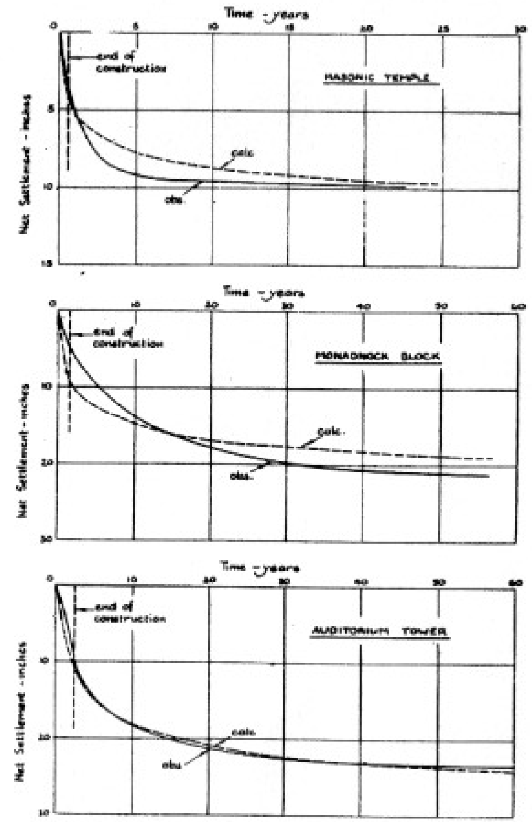

Fig. 2. Buildings on Chicago Clay

the time-settlement curves, for which U in equation (9) is calculated from the theory of one-dimensional consolidation, are also in tolerable agreement with the observations (see Fig. 2).

Three structures in London provide good examples of settlements on overconsolidated clay (Skempton, Peck, and MacDonald, 1955). For the London Clay, A is about $\frac{1}{3}$ in the stress range under foundations.* The clay is moderately deep, compared with the foundation width, and thus the factor μ has a value of about $\frac{1}{2}$. The calculated and observed settlements† are given in Table 5. Here again there is satisfactory agreement. However, the calculated time-settlement curves are widely different, in the early stages of consolidation, from the observations (Fig. 3); and further work is obviously required in this aspect of the problem.

* At failure, A has a negative value in London Clay.

† The final net settlement of Waterloo Bridge has been given as 3·4 in. (Cooling and Gibson, 1955). But this is derived by subtracting the average heave from the total settlement. It seems more in keeping with the usual definition of net settlement to subtract the settlement when the excavation load has been replaced. In this way the figure of 3·7 in Table 5 is obtained.

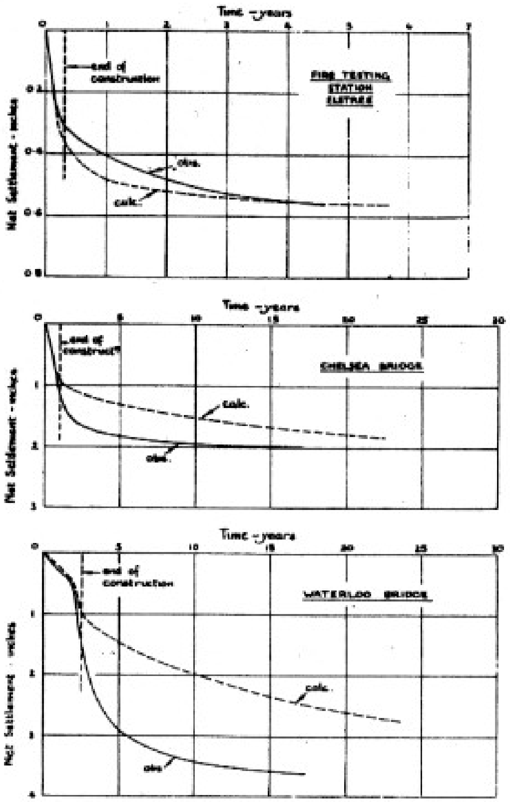

Fig. 3. Structures on London Clay

Another example of settlements on a heavily overconsolidated clay is provided by the Peterborough grain silo (Cooling and Gibson, 1955). The wings of the silo have a width of $b = 35$ ft and are founded on Oxford Clay with a thickness of about 0·9 b. Thus α is approximately 0·4. The A value may well be rather less than for London Clay, possibly about 0·25

Table 5

Calculated and observed final settlements of three structures on London Clay

(Settlements in in.)

Structure	ρ_i	$\rho_{cons} = \frac{1}{2}\rho_{oed}$	ρ_{final} calc.	ρ_{final} observed
Fire Testing Station, Elstree	0·25	0·35	0·6	0·7
Chelsea Bridge	0·8	1·65	2·45	2·1
Waterloo Bridge	0·9	2·6	3·5	3·7

The factor μ is therefore about 0·55. Cooling and Gibson give the calculated immediate and oedometer settlements as 0·35 in. and 1·0 in. respectively. Hence on the basis of the present Paper the calculated consolidation settlement is 0·55 in. These calculated settlements are compared with the observations in Table 6, from which the agreement is seen to be excellent.

Table 6

Comparison of calculated and observed settlements, Peterborough grain silo

	Calculated (in.)	Observed (in.)
Immediate	0·35	0·25
Consolidation	0·55	0·65
Final	0·9	0·9

SUMMARY

A method of calculating the net settlements of foundations on clay is given, which takes into account the pore pressures set up in the clay when the foundation load is applied. The final settlement is expressed by the equation :

$$\rho_{final} = \rho_i + \mu . \rho_{oed}$$

where ρ_i is the immediate settlement, ρ_{oed} the settlement calculated in the usual manner from oedometer test results and μ is a factor depending chiefly on the pore-pressure coefficient A.

The observed final settlements for eight structures are given in Table 7 where the values calculated from the above equation are also given, in column (a). The comparison between calculated and observed settlements is satisfactory in all cases.

The conventional method of estimating settlement is expressed by the equation :

$$\rho_{final} = \rho_{oed}$$

The settlements obtained in this way are given in Table 7, column (b). The method is in error in three of the eight cases ; and it tends to underestimate the settlements on normally-

Table 7

Calculations of net final settlement, by three methods

Structure	Net observed settlement (in.)	Net calculated settlement (in.) (a) $\rho_i + \mu . \rho_{oed}$	$\frac{\text{calc.}}{\text{obs.}}$	(b) ρ_{oed}	$\frac{\text{calc.}}{\text{obs.}}$	(c) $\rho_i + \rho_{oed}$	$\frac{\text{calc.}}{\text{obs.}}$
Normally-consolidated clays :							
Oil tank, Isle of Grain . .	21	18	0·86	18·5	0·88	21·5	1·02
Masonic Temple, Chicago . .	10	11	1·10	9	0·90	12	1·20
Monadnock Block, Chicago . .	22	20·5	0·93	16	0·73	22	1·00
Auditorium tower, Chicago . .	24	26	1·08	22	0·92	28·5	1·18
			0·99		0·81		1·10
Overconsolidated clays :							
Fire Testing Station, Elstree	0·7	0·6	0·86	0·65	0·93	0·9	1·28
Chelsea Bridge, London . .	2·1	2·45	1·17	3·3	1·57	4·1	1·95
Waterloo Bridge, London . .	3·7	3·5	0·95	5·2	1·40	6·1	1·65
Grain silo, Peterborough . .	0·9	0·9	1·00	1·0	1·11	1·35	1·50
			0·99		1·25		1·59

consolidated clays, and overestimate the settlements on the overconsolidated clays. Nevertheless the conventional method is very simple, and is not without use as providing a basis for the approximate estimation of settlements in a wide range of clays (MacDonald and Skempton, 1955).

In some publications it has been suggested that the settlement can be calculated from the expression :

$$\rho_{final} = \rho_i + \rho_{oed}$$

Reference to column (c) in Table 7 will show, however, that this method leads to severe errors in overconsolidated clays, although it is fairly satisfactory for normally-consolidated clays (see also, Simons, 1957).

The new method of settlement calculation can therefore be regarded as an improvement on existing methods. It is, however, semi-empirical in nature and there is ample room for a more exact theoretical analysis.

ACKNOWLEDGEMENTS

The Authors are grateful to their colleagues at Imperial College, University of London, and the Norwegian Geotechnical Institute, for many helpful discussions and for assistance in the numerical investigations.

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THE STABILIZATION WITH CEMENT OF WEATHERED AND SULPHATE-BEARING CLAYS*

by

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SYNOPSIS

This Paper describes the results of an investigation undertaken to study the influence of the chemical properties of a clay on its strength when stabilized with cement. The samples of clay examined were taken from a profile through clay at Stewartby, Bedfordshire. It was found that the profile consisted of 3–4 ft of boulder clay overlying two lower layers, both of Oxford Clay.

The principal factors affecting the strength appeared to be the clay, sulphate, and organic contents of the samples. The boulder clay, having a low clay and organic content and no sulphate, could be satisfactorily stabilized with 10% of ordinary Portland cement. Neither of the other layers could be stabilized, however, owing to their high clay and sulphate contents, although the lower layer, which contained a large amount of organic matter and less sulphate, was rather better, possibly because of the flocculating effect of the organic matter on the clay.

Because of the high sulphate contents of the samples, further tests were made to find the effect of sulphates on cement-stabilized clay. These tests showed that the effect of sulphates only became apparent when the clay-cement was immersed in water, when 1·0% of calcium sulphate or 0·75% of magnesium sulphate (expressed as SO_3) incorporated in the clay led to complete disintegration of the clay-cement mixture. When sulphate-free clay-cement mixtures were immersed in aqueous sulphate solutions as little as 0·2% sulphate (as SO_3) in the surrounding solution caused disintegration. The substitution of sulphate-resistant cement for ordinary Portland cement in these experiments did not effect much improvement.

It is concluded that until further knowledge has been obtained about the damage caused by sulphates, soils containing them should not be used for soil-cement stabilization.

Cet article décrit les résultats d'une enquête entreprise pour étudier l'influence des propriétés chimiques de l'argile sur sa propre résistance une fois stabilisé avec du ciment. Les échantillons d'argile examinés furent prélevés d'une coupe dans de l'argile à Stewartby, comté de Bedford. On découvrit que la coupe était constituée de 3 à 4 pieds d'argile de moraine à blocs recouvrant deux couches inférieures, toutes deux d'argile d'Oxford.

Les facteurs principaux affectant la résistance semblèrent être l'argile, le sulfate et les contenus organiques des échantillons. L'argile de moraine à blocs ayant une faible teneur en argile et en contenus organiques, et pas de sulfate, put être stabilisée de manière satisfaisante avec 10% de ciment Portland ordinaire. Pourtant aucune des autres couches ne put être stabilisée, du fait de leur haute teneur en argile et en sulfate, quoique la couche inférieure, qui contenait une grande quantité de matière organique et moins de sulfate, fût bien meilleure, peut-être à cause de l'effet de floconnement sur l'argile de la matière organique. A cause de la haute teneur en sulfate des échantillons, on procéda à d'autres essais afin de découvrir quel effet les sulfates ont sur l'argile stabilisée par le ciment. Ces essais montrèrent que les effets des sulfates ne devinrent apparents que lorsque le mélange argile-ciment fut immersé dans l'eau, et que 1·0% de sulfate de calcium ou 0·75% de sulfate de magnésium (sous forme SO_3) incorporé dans l'argile amena la désintégration complète du mélange argile-ciment. Lorsque des mélanges argile-ciment non sulfatés furent immersés dans des solutions aqueuses de sulfate, pas plus de 0·2% de sulfate (sous forme SO_3) ne fut nécessaire dans la solution environnante pour causer la désintégration. Dans ces expériences le remplacement du ciment Portland ordinaire par du ciment résistant à l'effet du sulfate n'apporta guère d'amélioration.

On conclut que, jusqu'à ce que de plus amples connaissances sur les détériorations causées par les sulfates soient obtenues, les sols les contenant ne devraient pas être employés pour la stabilisation sol-ciment.

INTRODUCTION

Granular and light clay soils can be successfully stabilized with Portland cement for use as road bases, but few heavy clays have been used in this way, chiefly because of the practical difficulties encountered in mixing cement with them. The development in recent years of improved mixing machinery suggests that this limitation may soon be overcome, and it will then be necessary to know whether there are in clays, as in some granular soils, any chemical constituents which affect the hardening of cement.