Fourth Rankine Lecture

LONG-TERM STABILITY OF CLAY SLOPES

by

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I wish to thank the British Geotechnical Society for inviting me to give the fourth Rankine Lecture, and to thank Dr Cooling for his very kind remarks.

In preparing the Lecture I have received a great deal of help from various people, especially Mr John Hutchinson, Dr Laurits Bjerrum and Mr Kenneth Early, while most of the field work which I shall describe was carried out jointly with Dr David Henkel. The long-term stability of clay slopes has indeed been one of the preoccupations of Dr Henkel and myself for the past 12 years; and I would like to take this opportunity of paying tribute to the value of the contributions which he made in this particular subject, and generally to soil mechanics and engineering geology, during the time he spent in Great Britain. I am also deeply grateful to my wife for her interest, criticism, and suggestions which have been most valuable.

INTRODUCTION

When a cutting is made in clay the pore pressure at any point depends partly on the position of the ground-water level, but also, to an important extent, on the response of the clay to the changes in stress which have taken place during excavation. In the course of time, however, the pore pressures through the clay gradually adjust themselves until finally they are everywhere in hydrostatic equilibrium with the ground water. This is the stage known as the "long-term" condition, in contrast with the "short-term" or "end-of-construction" condition.

All natural slopes exist in the long-term condition, while in cuttings this stage is not reached until several months or years after excavation; the time required depending chiefly upon the permeability of the clay.

I shall be concerned only with the long-term condition. Thus in all the field cases considered, whether in cuttings or natural slopes, the pore-pressure u at any point on a potential or real slip surface can be determined from the ground-water flow net (Fig. 1). From statical considerations it is also possible to calculate with reasonable accuracy the pressure σ and shear stress τ at any point on the slip surface.

Turning now to the resistance offered by the clay to sliding along this surface, it has long been known that the shear strength of a clay may be represented by the Coulomb-Terzaghi equation:

$$s = \bar{c}' + (\sigma - u) \tan \phi'$$

where $\bar{c}'$ is the "cohesion intercept" and ϕ' is the "angle of shearing resistance"; both parameters being expressed in terms of the effective stress $\sigma' = (\sigma - u)$ acting normal to the slip surface.

In a stable slope only a part of the total available shear resistance along a potential slip surface will be mobilised to balance the total shear force. Thus, in general, we may write:

$$\sum \tau = \sum \frac{\bar{c}'}{F} + \sum (\sigma - u) \frac{\tan \phi'}{F}$$

where F is defined as the "factor of safety". For stable slopes F is greater than 1.0. But when a failure occurs the sum of all the shear stresses along the slip surface exactly equals the available strength of the clay, and $F = 1.0$.

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There are several quite reliable methods for calculating F for a given slope, once the pore pressures are known and values of $\bar{c}'$ and $\bar{\phi}'$ have been obtained.* Moreover it is a relatively simple matter to carry out shear tests on undisturbed samples of clay. But, as we shall see, from the analysis of actual slips in clays, the values of the shear strength parameters as determined by conventional tests do not necessarily bear any relation to the values which must have been operative in the clay at the time of failure.

This conclusion, which has now been established beyond the slightest doubt, is obviously one of immense practical significance. We must therefore endeavour to understand why, in certain cases, there is a wide discrepancy between our ordinary laboratory test results and the actual field values of shear strength. For such cases we must also try to develop modified laboratory procedures which give results of sufficient reliability to be used with confidence in engineering design. These two objectives form the substance of my Lecture.

τ = shear stress

σ = total pressure

u = pore pressure

$\sigma' = \sigma - u$ = effective pressure

s = shear strength of clay

$= \bar{c}' + \sigma' \tan \bar{\phi}'$

In a stable slope

$$\sum \tau = \sum \frac{\bar{c}'}{F} + \sum (\sigma - u) \frac{\tan \bar{\phi}'}{F} \quad F = \text{factor of safety}$$

When a slip occurs $F = 1.0$ and $\sum \tau = \sum s$

LONG - TERM STABILITY OF CLAY SLOPES

Fig. 1

PEAK AND RESIDUAL STRENGTHS

Frequent reference will be made to "over-consolidated" clay and it is well to be clear as to the meaning of this term. In Fig. 2, point (a) represents a clay immediately after deposition, for example on the bed of an estuary. The deposition of more clay will cause an increase in effective pressure and a decrease in water content. At a stage represented by point (b) the clay is "normally-consolidated", in the sense that it has not been subjected to a pressure greater than the present overburden. The shear strength of normally-consolidated clay is proportional to the effective pressure, and the graph expressing the relation between strength and pressure is therefore a straight line passing through the origin of axes.

Many post-glacial clays are normally-consolidated; having been deposited during the eustatic rise of sea level consequent upon the melting of the ice sheets in Late Pleistocene times. But the great majority of clays are much older, and during their geological history have been subjected to very considerable pressure, corresponding to depths of overburden of

* In the cases considered in this Lecture where the slip surface has a section approximating to a circular arc the well known method of analysis due to Dr A. W. Bishop (1954) has been used.

several hundred, or even several thousand feet of sediments, which have subsequently been removed by erosion. The clay is then left in an "over-consolidated" state represented in Fig. 2 by point (d).

The removal of pressure is accompanied by an increase in water content, but this increase is far less than the decrease in water content during consolidation. Thus although the clay at point (d) is under the same effective pressure as the clay at point (b), the water content of the over-consolidated clay is considerably smaller. The particles are therefore in a denser state of packing and, not surprisingly, the shear strength is greater than that of the normally-consolidated clay.

Fig. 2

Examples of over-consolidated clays include the Lias, the Gault and the London Clay. These, and many other clays in this category, are fissured and jointed. They occasionally contain slickensides as well; probably as a result of tectonic movements or unequal expansion during the erosion cycle. Boulder clays are also over-consolidated, by the weight of ice rather than the weight of sediments, but they often show very little in the way of structural discontinuities; being intact, non-fissured materials without joints or slickensides. Most normally-consolidated clays are also free from such imperfections, though fissures can some times be seen in clays of this type.

In this Lecture I shall be concerned mainly with over-consolidated clays, since they give rise to much greater problems than the normally-consolidated clays, and they are more widespread.*

Let us, therefore, examine the shear strength characteristics of an over-consolidated clay, as shown in Fig. 3. This figure typifies the results obtained by carrying out slow drained tests in a shear box apparatus, in which the clay is subjected to displacements amounting to several inches.

As the clay is strained, so it builds up an increasing resistance. But, under a given effective pressure, there is a definite limit to the resistance the clay can offer, and this is the

* Over-consolidated clays of high sensitivity, such as the Leda clay in eastern Canada, represent a special class the properties of which may differ appreciably from those of the typical over-consolidated clays considered here.

“peak strength” s_f . In ordinary practice the test is stopped shortly after the peak strength has been clearly defined, and s_f is referred to simply as the “shear strength” of the clay (under the given effective pressure) without further qualification except, of course, the statement that the test has been carried out under drained conditions, i.e. without the development of pore pressures.

If, however, the test is continued, then we find that as the displacement increases so the resistance or strength of the clay decreases. But this process, which may be called “strain-softening”, is not without limit, for ultimately a certain “residual strength” s_r is reached which the clay maintains even when subjected to large displacements.* In the comparatively small number of clays so far investigated from this point of view, the strength falls to the residual value after displacements past the peak of the order of 1 in. or 2 in. But there is field evidence that the strength is not less than this laboratory value (or only slightly less) even at displacements of many feet.

A second shear box test could now be carried out on another specimen of the same clay, but under a different effective pressure. The results would be of the same type as those

SHEAR CHARACTERISTICS OF OVER-CONSOLIDATED CLAY

Fig. 3

already described. And if we suppose that three such tests be made, the peak and residual strengths, when plotted against effective pressure, would show a relationship approximately in accordance with the Coulomb-Terzaghi law. The peak strengths can therefore be expressed by the equation:

$$s_f = c' + \sigma' \cdot \tan \phi'$$

and the residual strengths by the equation:

$$s_r = c'_r + \sigma' \cdot \tan \phi'_r$$

The test results at present available show almost invariably that c'_r is very small, and probably not significantly different from zero. Thus I shall assume throughout the rest of the Lecture that for the residual strength we may write:

$$s_r = \sigma' \cdot \tan \phi'_r$$

* The first determination of the residual strength of an undisturbed clay (from the Weser-Elbe canal) was published by Dr B. Tiedemann in 1937. He carried out drained tests in a ring shear apparatus. Tests of the same type were made by Dr M. J. Hvorslev (1937) on two clays (Wiener Tegel and Little Belt Clay) consolidated from the slurry condition. Professor R. Haefeli introduced the term “residual” in 1938 (in German) and in 1950 (in English). Credit is due to him for insisting on the practical importance of residual strength; at a period when scarcely anyone was interested in this aspect of the shear properties of clays. Tests leading to an approximate determination of the residual strengths of some compacted soils have been reported by Mr J. MacNeil Turnbull (1952).

In other words, in moving from the peak to the residual, the cohesion intercept c' disappears completely. During the same process the angle of shearing resistance also decreases; in some clays by only 1° or 2° , but in others by as much as 10° .

It is notable that during the shearing process over-consolidated clays tend to expand, especially after passing the peak. Part of the drop in strength from the peak value is therefore due to an increasing water content. Of comparable importance, however, is the development of thin bands or domains in which the flaky clay particles are orientated in the direction of shear. The shear strength of a mass of such particles in random orientation must be greater than when the particles are lying parallel to each other. And while it is probable that the formation of orientation domains begins at relatively small strains (Goldstein *et al.*, 1961), there is decisive evidence for the presence of continuous bands of almost perfectly orientated particles in clays subjected to large strains; both in the laboratory (Astbury, 1960) and in the field (as will be described later in this Lecture).

Irrespective of the physical explanation of the drop in strength after passing the peak, the existence of this decrease in strength (especially in over-consolidated clays) must be accepted as a fact which has been fully established. Thus, if for any reason a clay is forced to pass the peak at some particular point within its mass, the strength at that point will decrease. This action will throw additional stress on to the clay at some other point, causing the peak to be passed at that point also. In this way a progressive failure can be initiated and, in the limit, the strength along the entire length of a slip surface will fall to the residual value. Obviously, in any given case, a slip may occur before the residual strength is attained throughout the clay, but once a progressive failure has started the average strength of the clay will decrease inexorably towards the limiting residual value.

Now it is well-known that the strength of solid materials is greatly diminished by the presence of microscopic cracks, holes and other imperfections. In the simplest terms these act as stress concentrators, and they are responsible for fracture taking place at an average stress which is far less than the ideal strength of the material.* And there would seem to be no reason to suppose that the macroscopic fissures, joints and slickensides, present in so many clays, do not also act as stress concentrators in a roughly analogous manner; quite apart from the fact that they act as discontinuous planes of weakness—for it is unlikely that the strength on a fissure or joint can be appreciably higher than the residual value.

Hence we may logically expect that a fissured or jointed clay would not be able to develop its peak strength along the full length of a slip surface. Not only will the fissures and joints reduce the average strength of the clay mass, but they can cause the peak to be crossed, as a result of local over-stressing, and a progressive decrease in strength will follow.

Fissures and other physical discontinuities may not be the only explanation of landslips in clays taking place at strengths well below the conventional peak value. The peak strength as measured in laboratory tests lasting, at the most, a few weeks might be appreciably greater than the strength which could be developed in some clays when the stresses are applied over periods of years, decades or centuries. In other words the effects of shear creep have to be considered; though at present very little quantitative information is available to assess these effects so far as the peak strength is concerned.

Seasonal variations of water content and temperature will also reduce the strength of a clay, but their influence is restricted to a rather shallow depth. Thus, although perhaps of great significance in geomorphological processes, seasonal variations are unlikely to prove important in the relatively deep-seated shear failures of the type considered in this Lecture, except for the evident fact that a landslide would tend to occur at a time when the ground water level was exceptionally high. The pore pressures would then also be exceptionally high, and the effective pressure (and hence the strength) correspondingly low. Nevertheless it is with the

* An excellent survey of the physical processes involved in the fracture of solids will be found in the 1963 Bakerian Lecture by Professor A. H. Cottrell.

deletion

shear-strength parameters of the clay that we are concerned; not with the effects of such phenomena as pore pressures, earthquakes, lateral pressure release, and so on.

After these generalities let us examine some actual tests. Fig. 4 shows the results of tests carried out in the laboratory of Soil Mechanics Ltd and also at Imperial College on samples taken from clay involved in a landslide at Walton's Wood, Staffordshire. The clay forms the weathered zone, up to 35 ft thick, of a Carboniferous mudstone and it blankets the sides of a valley eroded during a retreat phase of the Last Glaciation in that area.

A typical stress-displacement curve for an undisturbed sample of the clay is shown. In this particular test the effective pressure was held constant at 22.2 lb/sq. in. The peak strength was found to be 10.8 lb/sq. in. and the residual strength 5.1 lb/sq. in. These strengths are plotted in the upper diagram with an open circle and a cross respectively. Similar tests on other specimens, at various effective pressures, show that the peak strength can be represented by the parameters $c' = 320$ lb/sq. ft and $\phi' = 21^\circ$, while the residual strengths fall along a line passing through the origin ($c'_r = 0$) with a slope $\phi'_r = 13^\circ$.

Fig. 4

These tests were carried out in a set of Casagrande shear boxes with specimens 2.36 in. square in plan and 1 in. thick. After completing the first traverse, with a displacement of about 0.3 in., the box was pushed back to its original position and then sheared again: the process being repeated until the strength of the clay had dropped to a steady (residual) value. Movement in both directions was slow, so that there were presumably no excess pore pressures in the clay, and the whole test took 6 days to complete.*

This technique is admittedly not perfect. After each reversal a small peak is often observed, as indicated by the dotted line in Fig. 4, and there is some danger of slurring the clay on the slip surface. Ideally the displacement should be applied continuously in one

* A different type of reversal shear test, carried out rather quickly, in which the vertical load is varied to maintain a constant volume in the clay specimen during shear, has been described by Professor H. Borowicka (1963).

Fig. 5. Slickensides formed in large strain test on specimen of undisturbed clay from Walton's Wood

Fig. 20. Slickensides formed in large strain test on specimen of London Clay from Hendon motorway

direction, and, as an approach towards this ideal, Dr Alan Bishop is now designing a shear box with a length of 7 in. We are also hoping to build a large ring shear apparatus. Meanwhile, the reversal technique appears to provide a simple and reasonably satisfactory method of measuring the residual strengths of undisturbed clay samples, using equipment already available in every laboratory.*

At Walton's Wood several samples were also obtained which included the actual slip plane. Triaxial test specimens were prepared from these samples in such a way that the slip plane was inclined at about 50° to the horizontal, and the specimens were then subjected to slow drained tests. Failure took place, as expected, along the pre-existing slip surface, and the results are shown in Fig. 4 by solid black circles.† It will be seen that the strengths developed on the slip plane correspond very closely to the residual strengths obtained by subjecting specimens of originally undisturbed, unsheared clay to large displacements.

Moreover, by the time the strength of these latter specimens had fallen to the residual value, extremely well-defined slickensides had developed (Fig. 5).

Using resin and wax impregnation techniques, Mr Early and Mr W. F. Nenninger, of Soil Mechanics Ltd, have succeeded in making several thin sections of the Walton's Wood clay, showing the natural slip plane. This was seen, in fact, to consist of a continuous band within which the clay particles were so strongly orientated, in the direction of shearing, that they formed a domain exhibiting sharp extinction when viewed between crossed nicols. The domain comprising the main slip "surface" had a thickness of the order 20μ . Associated with it were several secondary slip domains, within a matrix of clay showing moderate orientation, not necessarily parallel to the slip surface, and having a thickness up to about 1 in. This is the softened zone often observed in the immediate vicinity of the slip plane. On either side of this zone the clay was found to exhibit scarcely any orientation.

Domains of strongly orientated particles have also been observed when quite soft remoulded clay is subjected to large strains (Astbury 1960). Moreover, tests by Mr Derek Petley, at Imperial College, have shown that the residual angle of shearing resistance of the Walton's Wood clay when normally-consolidated from a slurry, prepared at the liquid limit, is compatible with the value of $\phi_r' = 13^\circ$ as measured in large strain tests on the undisturbed over-consolidated clay,‡ and in tests on the natural slip plane.

It is therefore possible to suggest, with perhaps a slight degree of over-simplification, that the residual strength of a clay, under any given effective pressure, is the same whether the clay has been normally- or over-consolidated; as indicated in Fig. 6.§ If this idea is correct then the angle ϕ_r' should be a constant for any particular clay whatever its consolidation history; depending only on the nature of the particles.

Some further support for this view is provided by the data in Fig. 7. Here the residual

* It is possible that the residual strength can also be measured by cutting a plane in the clay with a fine-wire saw and testing in the triaxial apparatus with this plane inclined at approximately $(45^\circ + \phi_r'/2)$ to the horizontal. Preliminary tests, using very slow rates of strain, suggest that this technique is promising and deserves intensive study.

† If the vertical and horizontal effective stresses acting on the triaxial specimens at failure are σ_1' and σ_3' , and if α is the inclination of the slip plane to the horizontal, then

$$\begin{aligned} s_r &= \frac{1}{2}(\sigma_1' - \sigma_3') \cdot \cos(2\alpha - 90^\circ) \\ \sigma' &= \frac{1}{2}(\sigma_1' + \sigma_3') - \frac{1}{2}(\sigma_1' - \sigma_3') \cdot \sin(2\alpha - 90^\circ) \end{aligned}$$

Four of the ten points plotted in Fig. 4 as solid black circles are the results of tests on specimens set up in the shear box with the slip plane coincident with the plane of separation of the top and bottom halves of the box.

‡ A similar result was noted by Tiedemann (1937) who found that the residual strength of the Weser-Elbe clay, when normally-consolidated from a slurry, differed little from the residual strength of the undisturbed (over-consolidated) samples.

§ Thus, although it can be said that an over-consolidated clay at large strains behaves, in effect, as a normally-consolidated material with zero cohesion intercept, this statement is generally true only if we make the comparison with a normally-consolidated clay which has itself been brought into its residual condition.

angles of shearing resistance of a number of normally- and over-consolidated clays are plotted against the clay fraction (percentage of particles, by weight, smaller than 2μ). No marked difference is found between the two conditions of clay, while all the points indicate a most definite tendency for ϕ_r' to decrease with increasing content of clay particles. Indeed the results suggest that if the soil consisted entirely of clay particles the angle of residual shearing resistance would be directly comparable to the angle of friction ϕ_μ as measured by Horn and Deere (1962) for various layer-lattice minerals including biotite, talc, and chlorite, the physical properties of which are possibly not very different from those of such clay minerals as illite and kaolinite.*

By contrast, the quartz grains in a sand or silt, being roughly spherical in shape, cannot orientate themselves. Thus the residual angle ϕ_r' for sands and silts always remains at a value given approximately by the relationship (Caquot, 1934) for particles in random orientation, shearing at constant volume:

$$\tan \phi_r' = \frac{\pi}{2} \tan \phi_\mu$$

Evidently, in the general case, where the soil consists of particles of clay and silt (and sand), the silt will tend to increase ϕ_r' above the value of ϕ_μ for the clay particles; by inhibiting, to some extent, the full orientation of the clay particles and, at least when there is an appreciable content of silt, by contributing some measure of its own higher angle of shearing resistance.

**SIMPLIFIED RELATION BETWEEN
NORMALLY- AND OVER-CONSOLIDATED CLAY**

Fig. 6

RESIDUAL FACTOR

When confronted with the problem as to whether the peak or residual strength should be used in calculating the stability of a slope in over-consolidated clay, the designer is facing the most important question, numerically speaking, with which he will be concerned. To illustrate this point I have collected together in Fig. 8 the results of tests on four quite typical over-consolidated clays, the index properties of which are summarised in Table I.

The boulder clay from Selset has been consolidated under a moderate thickness of ice, but it is very sandy with a clay fraction of only 17%. Consequently the cohesion intercept of the peak strength is not large and there is a drop of only 2° between the peak and residual angles of shearing resistance. Even so, in the stability analysis of a slope in this clay there is, as we

* These elegant tests by Dr H. M. Horn and Professor D. U. Deere were carried out by measuring the friction developed between the cleavage surfaces of two specimens of a layer-lattice mineral. The angle of friction obtained in such tests is therefore also the angle of shearing resistance of an assemblage of the particles in perfect orientation. The values of ϕ_μ shown in Fig. 7 are for the minerals in a saturated condition.

Table 1
Index Properties of Clays in Fig. 8
 (average values)

	LL	PL	Clay fraction
Selset	26	13	17
Jari	70	27	47
London Clay	82	29	55
Walton's Wood	53	28	69

shall see, a difference of more than 30% in the factor of safety as calculated on peak or on residual strengths.

The clay from Jari, in the foothills of the Himalaya, belongs to the Upper Siwalik series and has been consolidated under at least 2,000 ft of sediments. This clay has been tested in connexion with the design of a dam forming part of the great Mangla project in West Pakistan. The samples showed no marked evidence of weathering and, as will be seen, the peak cohesion intercept is high. Indeed, the clay is very strong in its undisturbed state. Yet the residual strengths show no cohesion intercept and an angle of shearing resistance of 18°. At an

**DECREASE IN ϕ_r WITH
 INCREASING CLAY FRACTION**

Fig. 7

effective pressure of 1000 lb/sq. ft, for example, the peak strength is 3.6 times greater than the residual, and the practical significance of deciding which strength to use in design therefore requires no further emphasis.

The London Clay, of Eocene age, has been consolidated under a thickness of sediments, subsequently removed by erosion, varying from 500 ft in the eastern parts of Essex up to 1,000 ft in the region west of London. The test results shown in Fig. 8 apply to the zone of weathered clay, extending typically to a depth of 40 ft below the surface. Several case records

**PEAK AND RESIDUAL STRENGTHS
OVER-CONSOLIDATED CLAYS**

Fig. 8

of slips in this zone will be given later in the Lecture, and for these the factor of safety based on peak strengths is about 2.7 times greater than the figure based on residual strengths.

Finally, data are given in Fig. 8 for the Walton's Wood clay. Here it may be mentioned that the peak factor of safety, as calculated on the actual slip surface of this landslide, is about three times higher than the result obtained by using residual strengths.

The question as to whether peak or residuals should be used in design cannot therefore be dismissed as a matter of merely academic interest. And in the remainder of the Lecture I shall consider as briefly as possible those case records at present available which can help to throw some light on this problem.

My procedure will be, in the first place, to describe the slip and then present the results of a stability analysis of the slip. These results will be given in terms of the average effective

DEFINITION OF RESIDUAL FACTOR

Fig. 9

stress $\bar{\sigma}'$ and the average shear stress $\bar{\tau}$ acting on the slip surface. In Fig. 9 these two values give the point shown by a solid black circle, and this point remains constant (after the "long-term" condition has been reached) except for the relatively small seasonal variations in pore pressure. The actual value of $\bar{\sigma}'$ is, however, calculated using the best estimate of pore pressures existing when the slip took place.

In the second place I shall give the results of laboratory determinations of the peak and residual strengths. With reference to Fig. 9 it is then at once possible to read off the peak strength s_f and the residual strength s_r corresponding to the average effective pressure $\bar{\sigma}'$.

Now, as in each case a slip has occurred, the factor of safety must be equal to 1.0 and the actual average shear strength of the clay at the time of the slip $\bar{s}$ must be equal to the average shear stress $\bar{\tau}$. Thus, by comparing the value of $\bar{s}$ with the values of s_f and s_r , we can see at a glance whether the strength of the clay involved in the slip was at the peak, or at the residual or, perhaps, at some intermediate value. Indeed, if the concept of progressive failure is valid for fissured clays, we may expect to find that the average strength of these clays decreases with time; and the slip occurs when the average strength has reached a value equal to the average shear stress which, as previously mentioned, has been acting continuously since the slope came into being.

In order to have a convenient quantitative expression for the amount by which the average strength has fallen I shall introduce the "residual factor", defined by the expression:

$$R = \frac{s_f - \bar{s}}{s_f - s_r}$$

or

$$\bar{s} = R s_r + (1 - R) s_f$$

Thus, in physical terms, R is the proportion of the total slip surface in the clay along which its strength has fallen to the residual value.

Clearly, if no reduction in strength has occurred and the whole of the clay is at the peak strength, then $R = 0$. Conversely, if the average strength has reached the residual value, $R = 1.0$. For the point shown in Fig. 9 the average strength has fallen rather more than halfway between the two limits and R is about 0.6; or about 60 per cent of the slip surface passes through clay having the residual strength.

The object in examining the case records is, therefore, to determine the value of R and relate this as far as possible to the type of clay.

CASE RECORDS

*Jackfield**

In 1952 a landslide occurred at the village of Jackfield, Shropshire, on the River Severn 1½ miles downstream of Iron Bridge; destroying several houses and causing major dislocations in a railway and road (Fig. 10). In this locality the Severn flows through a V-shaped valley (the so-called Iron Bridge gorge) which has been eroded largely, if not entirely, since the retreat of the main ice sheet of the Last Glaciation. Erosion is indeed still continuing, and the sides of the valley are covered by a mosaic of landslides, of varying ages.

It is possible that previous landslides may have taken place along at least a part of the present slip surface, but the slope must have been more or less stable for a long time before 1950, when warnings of instability were observed in the form of a broken water main serving cottages near the river bank. Towards the end of 1951 further movement was noted, and by February 1952 the road was becoming dangerous. During the next month or two the landslide developed alarmingly. Six houses were completely broken up, gas mains had to be relaid above ground, the railway could be maintained only by daily adjustments to the track and a

* The Jackfield landslide has been published previously (Henkel and Skempton, 1954), but I have recently made a critical re-examination of all the original data, with results that differ in some respects from those given 10 years ago.

JACKFIELD LANDSLIDE 1952

Fig. 10

minor road along the river had to be closed to traffic. By this time the maximum downhill displacement totalled 60 ft.

In the autumn of 1952 an investigation was undertaken by Dr Henkel and myself. Movements, which had become almost stationary during the summer, were again taking place and we were able to observe the exact depth of the slip surface in four boreholes (Fig. 11).

JACKFIELD LANDSLIDE 1952

Fig. 11

The strata, consisting of very stiff clays and mudstone, alternating with marl-breccia and occasional coal seams, dip gently in a south-easterly direction with the strike running roughly parallel to the section of the landslide.* The slide, however, was confined wholly within the zone of weathered, fissured clay extending to a depth of 20 ft to 25 ft below the surface.† The

JACKFIELD Shear Tests

Fig. 12

slip surface ran parallel to the slope (which is inclined at 10°), at an average depth of 18 ft. The length of the sliding mass, measured up the slope, amounted to about 550 ft and in the winter 1952-53 ground-water level reached the surface at a number of points, although on average it was located at a depth of 2 ft.

Analysis of the forces acting on the slip surface shows $\bar{\sigma}' = 1,300$ lb/sq. ft and $\bar{\tau} = 400$ lb/sq. ft.

Drained shear tests on samples taken from depths between 15 ft and 19 ft, but not in the immediate vicinity of the slip plane, showed peak strength parameters of $c' = 220$ lb/sq. ft and $\phi' = 25^\circ$ (Fig. 12).‡ When these tests were made we had no clear idea of the significance of residual strengths. Fortunately, however, in most cases we did at least continue the observations throughout the full travel of the shear box, and it is possible from the results to make an

* These strata comprise the Coalport Beds of the Upper Carboniferous.

† The weathered clay, though quite firm and still retaining the characteristics of an over-consolidated clay is nevertheless far less strong than the hard, almost rocklike, unweathered strata. This explains why the slip was relatively shallow, instead of extending to the considerable depths which would be expected in a homogeneous cohesive material.

‡ As in most slides in over-consolidated clays, we found at Jackfield a zone of softened clay, not more than about 2 in. thick, containing the slip plane. Shear tests on specimens cut from the outer edge of this zone showed $c' = 150$ lb/sq. ft. and $\phi' = 21^\circ$. The value of ϕ'_r on the slip plane itself was not determined experimentally.

approximate estimate of the residual angle of shearing resistance. As will be seen from Fig. 12, this estimate gives the value $\phi_r' = 19^\circ$. Very probably a more correct value would be rather less than 19° .

Referring now to Fig. 13, the peak and residual strengths corresponding to the average effective pressure of 1,300 lb/sq. ft acting on the slip surface, are 825 and 425 lb/sq. ft respectively. But, as previously mentioned, the average shear stress (and hence the average shear strength) along the slip surface at the time of failure was 400 lb/sq. ft.

It is therefore clear that when the landslide took place the strength of the clay was closely equal to its residual value. In fact, taking $\phi_r' = 19^\circ$, we arrive at the conclusion that the residual factor $R = 1.12$ but, when the approximate nature of ϕ_r' is taken into account, I doubt if the value of R is significantly different from 1.0.

Expressing the results in another way, had the peak strength been used in a stability analysis of the Jackfield slope, the calculated factor of safety would have been 2.06 (an error of more

Stability analysis shows $R = 1.12$, with $\bar{c} = 0$ $\bar{\phi}' = 17^\circ$

Calculated factors of safety :-

(i) on peak strength $F = 2.06$

(ii) on residual strength $F = 1.11$

JACKFIELD SUMMARY OF DATA

Fig. 13

than 100%, since the true factor of safety was 1.0). On the other hand, using even the rather crude value of $\phi_r' = 19^\circ$, the calculated factor of safety based on residual strength would differ by only 11% from the correct result.

It may be added that the values of $\bar{\sigma}'$ and $\bar{\tau}$ calculated on the actual slip surface correspond to an average angle of shearing resistance of 17° (with $\bar{c}' = 0$). This figure, which I obtained from an approximate analysis, has been confirmed by Mr Morgenstern using a more refined method of calculation.

Selset

During an early visit to the site proposed for Selset dam in north Yorkshire, near Middleton-in-Teesdale, Dr Bishop and I found an interesting landslide which had taken place not long before in a boulder clay slope of the River Lune valley. During the years 1955-60, while work was proceeding on the dam, a short distance upstream, various opportunities were taken of investigating the slide and testing samples of the clay. The results have been published

(Skempton and Brown, 1961) and it is necessary to add very little here in order to bring this case record into the framework of our present enquiry.

The river, when in flood, is eroding the valley side and in this way is still widening its alluvial plain (Fig. 14). Nevertheless the process is not very rapid (on an historical, rather than a geological time scale) and the Ordnance Survey map of 1856 shows the river to have been at that time in just about its present position, relative to the toe of the slope. Thus there can be no doubt that we are dealing with a "long-term" condition.

The boulder clay was remarkably uniform, without fissures or joints, and it showed little if any signs of weathering except in the very shallow zone of seasonal variations.

The exact location of the slip surface was not determined, although the tension crack near the top of the slope was clearly visible, and various possible slip circles were therefore analysed. Representative values, sufficiently accurate for our purpose, of the average effective stress $\bar{\sigma}'$ and the average shear stress $\bar{\tau}$ are 760 lb/sq. ft and 640 lb/sq. ft. respectively.

Fig. 14

Triaxial and shear box tests on eight samples of the boulder clay gave a consistent set of results, showing that the peak strength could be expressed by the parameters $c' = 180$ lb/sq. ft and $\phi' = 32^\circ$. The tests were not carried to strains which, in the light of subsequent experience, we know to be necessary if the residual strength is to be accurately defined, but even so it is possible to estimate the parameters $c_r' = 0$ and $\phi_r' = 30^\circ$ with reasonable certainty (Fig. 15).

Corresponding to the average effective pressure $\bar{\sigma}' = 760$ lb/sq. ft, the peak strength is therefore $s_f = 660$ lb/sq. ft and the residual strength $s_r = 440$ lb/sq. ft (Fig. 16). But the average shear stress, and hence the average shear strength, at the time of the slip, is 640 lb/sq. ft. This is scarcely less than the peak strength; and the residual factor $R = 0.08$.

Thus, if the slip had been analysed using $c' = 180$ lb/sq. ft and $\phi' = 32^\circ$, the calculated factor of safety would have been very close to the correct value of 1.0. By contrast, using $c_r' = 0$ and $\phi_r' = 30^\circ$ the calculated factor of safety is 0.69, a result which is about 30% in error.

SELSETSHEAR TESTS

Fig. 15

Stability analysis shows $R = 0.08$, with $\bar{c}' = 160$ $\bar{\phi}' = 32^\circ$

calculated factors of safety:-

(i) on peak strength $F = 1.03$

(ii) on residual strength $F = 0.69$

SELSETSUMMARY OF DATA

Fig. 16

It is notable that the peak strength appears to have been almost fully mobilized simultaneously along the entire length of the slip surface. Even in this intact, non-fissured clay one might perhaps expect to find a rather more pronounced indication of progressive failure than that corresponding to a residual factor of only 8%. Nevertheless the fact remains that an analysis based on peak strengths leads to a result which is quite satisfactory from a practical point of view.*

London Clay

Data are available for a number of slips which have occurred in the weathered zone of the London Clay. Fissures and joints and occasional slickensides can be seen throughout the full depth of the London Clay, but such features are far more conspicuous in the weathered zone, which is typically about 30 ft to 40 ft deep (Fig. 17). An obvious sign of weathering is the brown colour of the clay, in contrast to the "blue" (actually a slaty grey) colour of the unweathered material. A more sensitive indication of weathering, however, is the presence of thin films of minerals such as goethite and limonite in some of the fissures and joints. These alteration products are frequently found at depths well below the base of the "brown" clay.

The peak strength parameters of the weathered clay naturally vary to some extent from site to site, but representative values can be taken as $c' = 320$ lb/sq. ft. and $\phi' = 20^\circ$ (Fig. 18).† So far as residual strengths are concerned, Mr William Watson of Soil Mechanics Ltd has carried out a shear test on a sample including the existing slip plane, taken from the site of a landslide at Dedham, Essex; and Mr Petley at Imperial College has measured the residual strength of three specimens of undisturbed clay, using the reversal shear box technique. The results indicate that ϕ_r' is about 16° (Fig. 18).

One of the large-strain reversal tests is plotted in Fig. 19, and the slickensides in the specimen at the completion of the test are shown in Fig. 20, facing p. 83. Examination under the microscope of a thin section prepared by Mr Early from a specimen of clay containing the Dedham slip plane showed a domain of strongly orientated particles, similar to that observed in the Walton's Wood clay, with a thickness of approximately 10μ .

A zone of softened clay extending for roughly an inch on either side of the slip plane is characteristic of London Clay, and three representative examples are shown in Fig. 21. The water contents immediately adjacent to the slip plane will be seen to be about 35, as compared with water contents of around 30 in the unsoftened clay.

Turning now to the case records of slips in the London Clay, the first refers to the failure of a retaining wall at Kensal Green, which took place in 1941 (Fig. 22).‡ The wall has a considerable length. It was built in 1912, and at its highest section, about 250 ft from the

* A similar conclusion can be derived from the investigations of a slip which occurred on 6 October 1954 in a deep cutting at Lodalen, near Oslo (Sevaldson, 1956). Here the clay, of marine origin, was lightly over-consolidated and non-fissured with $c' = 210$ lb/sq. ft and $\phi' = 27^\circ$; $w = 29$, LL = 35, PL = 19 and clay fraction = 46%.

† The case records of Selset and Lodalen may further be interpreted as indicating that shear creep has little effect on peak strength in the clays at these sites.

‡ Tests on block samples taken from a deep shaft at Ashford Common, near Staines, show that the values of c' and ϕ' in the unweathered London Clay are considerably greater than those in the weathered zone. The tests have been carried out at Imperial College and at the Building Research Station, and will be published in a paper to *Géotechnique* by Dr A. W. Bishop, Mr D. L. Webb, and Mr P. I. Lewin.

Peak strength parameters as quoted for the London Clay and other fissured clays (unweathered or weathered) are measured on comparatively small specimens and relate essentially to the intact material. Tests on large specimens show lower strengths due to the inclusion of fissures. The residual strength, however, should be independent of sample size.

‡ A cutting at Kensal Green was excavated in 1835 for the London-Birmingham Railway. Widening took place in 1875 and again in 1912, when the present wall was built. It seems unlikely that the 1875 cutting had any significant effect on the clay involved in the 1941 failure. As a member of the Building Research Station staff, under Dr Cooling, I had the good fortune to be given the job of investigating this failure.

500'-1000' removed by erosion
since Eocene times

TYPICAL PROFILE OF LONDON CLAY

Fig. 17

WEATHERED LONDON CLAY

SHEAR TESTS

Fig. 18

HENDON

LARGE STRAIN TEST

Fig. 19

1941 slip, a failure occurred in 1929. After repairs had been carried out observation points were established at various places along the whole length of the wall. As it happens, one of these survey sections was located only a few feet from the position of maximum movement in 1941. Therefore we are fortunate enough to have a continuous record of displacements over a period of 12 years preceding failure (Fig. 22). For several years after observations started, when the wall had already been in existence for 17 years, the rate of movement was small (about $\frac{1}{4}$ in./year). But gradually the rate increased, and finally the movement amounted to 18 in.

It would be difficult to imagine a better demonstration of progressive failure. Looking at the time-displacement curve in Fig. 22 one can sense that the average strength of the clay is decreasing and approaching the existing shear stress, with correspondingly larger and larger movements as more of the clay is brought to its residual strength.

LONDON CLAY

Distribution of water content near slip plane

Fig. 21

An analysis of the forces acting on the slip surface shows that $\bar{\sigma}' = 800$ lb/sq. ft. The average peak and residual strengths are therefore 610 lb/sq. ft and 230 lb/sq. ft respectively (Fig. 23). But the average shear stress along the slip surface is 380 lb/sq. ft, and this is also the shear strength at the time of failure. Thus the residual factor can be immediately calculated, since:

$$R = \frac{610 - 380}{610 - 230} = 0.61$$

Hence, after a period of 29 years, the average strength has fallen about 60% of the way from the peak to the residual. Or, in other words, the strength has fallen to the residual value along 60% of the slip surface, whilst remaining at the peak value over the other 40%.

Looking at the problem from another point of view, if the peak strengths had been used throughout, the calculated factor of safety would have been 1.6; while, using the residual strengths, the figure would have been 0.6. The correct factor of safety is 1.0, and the strength parameters required to give this result in a stability analysis are $\bar{c}' = 125$ lb/sq. ft and $\phi' = 17\frac{1}{2}^\circ$, corresponding to $R = 0.61$.

Very similar data have been derived from the analysis of a slip in a cutting at Northolt

KENSAL GREEN, WALL FAILURE 1941

Fig. 22

+90ft O.D. —

(Fig. 24).* Here the excavation was made in 1936, and the failure occurred 19 years later in 1955. A site investigation was carried out shortly after the slip had taken place. The investigation included the installation of piezometers, taking samples for test and surveying considerable portions of the slip surface exposed in trenches which were later cut in the slope for counterfort drains.

KENSAL GREEN**SUMMARY OF DATA**

Fig. 23

* The investigations at Northolt, as also at Sudbury Hill and various natural slopes in London Clay, described later in this Lecture, were carried out by Dr Henkel and myself with the able assistance of Dr F. A. DeLory. Papers dealing with Northolt (Henkel, 1957) and with the natural slopes (Skempton and DeLory, 1957) have been published. In recent months I have re-examined the data and made new stability calculations with the help of Mr Noel Simons.

Fig. 24

The analysis, made on a circular arc approximating to the observed slip surface, shows that $\bar{\sigma}' = 750 \text{ lb/sq. ft.}$ The corresponding average values of peak and residual strength are 595 lb/sq. ft. and 215 lb/sq. ft. respectively. The average shear stress, and hence the average shear strength acting along the slip surface at the time of failure, is 380 lb/sq. ft. Consequently $R = 0.56$, and the strength parameters consistent with a factor of safety of 1.0 are $\bar{c}' = 140 \text{ lb/sq. ft.}$ and $\phi' = 18^\circ$.

The third slip in weathered London Clay which I shall consider took place in 1949, in a cutting excavated at Sudbury Hill in 1900 (Fig. 25). Apart from trimming the toe of the slip where it impinged on the railway, no remedial measures were undertaken. The profile of the slope after the slip is shown by the dashed line in Fig. 25, and it will be seen that there has been a displacement of several feet along the slip surface.

Fig. 25

Fig. 26

Between 1949 and 1956, when this slip was investigated, small intermittent movements had occurred. Therefore the slope was standing in a condition of limiting equilibrium, and an analysis of the forces should lead to an estimate of the residual strength.

Three piezometers were installed, and the water levels plotted in Fig. 25 are the highest recorded during a period of 20 months. Using these water levels and the slip circle as shown, the average effective pressure is found to be $\bar{\sigma}' = 650$ lb/sq. ft with a corresponding shear stress of $\bar{\tau} = 170$ lb/sq. ft. Thus, if we take $c'_r = 0$, the residual angle of shearing resistance can at once be calculated, since in this case:

$$\tan \phi'_r = \bar{\tau} / \bar{\sigma}'$$

WEATHERED LONDON CLAY

SUMMARY OF DATA

Fig. 27

and we find that $\phi_r' = 15^\circ$. This value is in reasonable agreement with the test results previously mentioned.*

The conditions obtaining just before the slip can also be investigated, assuming that the piezometer levels observed in 1956–57 are sufficiently representative of those in 1949. The results of this analysis may be summarised by saying that $R = 0.80$, with $\bar{c}' = 60$ lb/sq. ft and $\phi' = 17^\circ$.

These figures must be regarded as approximate, since they depend to some extent on rather uncertain knowledge of piezometric levels. Nevertheless, when considered in relation to the results from Kensal Green and Northolt, they show an unmistakable tendency for the strength to decrease with time; as would be expected in a material susceptible to progressive failure.†

In natural slopes the time scale is so great that we may anticipate that the strength of a fissured clay has fallen to the residual value. Observations of natural slopes in London Clay, where the water-table is close to the surface, have shown that an inclination of 10° marks approximately the division between stable and unstable conditions (Fig. 26). Values of $\bar{\sigma}'$ and $\bar{\tau}$ have been calculated for slip surfaces at depths between 5 ft and 8 ft, and for water levels between 6 in. and 30 in. below the surface.‡ The results are plotted in Fig. 26, from which it will be seen that a limiting slope of 10° is compatible with a residual angle of shearing resistance equal to about 16° .

All the foregoing data on London Clay are assembled in Fig. 27, and the pattern which emerges is so consistent that I feel encouraged to think we are working along the right lines in this problem.

SUMMARY

Residual factors for the various slips which have been considered in this Lecture are summarised in Table 2. The Selset boulder clay, without fissures or joints, develops its peak strength along very nearly the entire length of the slip surface. The weathered London Clay is heavily fissured and jointed. In such a material there will be some decrease in the shear strength parameters, below the peak values, even during the process of excavation; and there is definite evidence from the investigations at Northolt and Kensal Green that in slips which have taken place after 20 or 30 years the average strength of the clay has fallen about 60% of the way from the peak to the residual. At Sudbury Hill, where the slip occurred after nearly 50 years, the factor has increased to 80%, while in natural slopes in the London Clay the strength appears to be close to the residual. The Jackfield landslide, which also occurred in a natural slope in weathered, fissured clay, similarly shows a strength approximately equal to the residual value.

It would therefore seem that the presence of fissures and joints can indeed lead to progressive failure in a clay slope and, in the limit, this process can continue until the residual strength is reached. But in clays which are not fissured or jointed the decrease in strength from

* The rather close correlations between laboratory and field values of ϕ_r' as found at Sudbury Hill and Jackfield (and also in the investigations now being made at Walton's Wood) suggest that the residual strength of clays is virtually time-independent for failure times ranging from about one week, as in the laboratory tests, to periods of several years.

† It should be noted that in some landslips the kinetic energy of the movement, after the initial failure, may be sufficient to cause appreciable "over-riding". In such cases the sliding mass will come to rest in a position corresponding to a strength lower than the true residual value.

‡ As previously mentioned, the existence of fissures implies that the average strength is less than the peak even if no progressive failure has occurred. It is, for example, quite possible that the London Clay might have a value of R in the region of 0.25 immediately after excavation, simply by virtue of the fissures and joints. With time, progressive failure leads to the higher values of R as found at Kensal Green, Northolt and Sudbury Hill, and in the natural slopes.

§ Details of the field observations and the method of stability analysis will be found in a paper by Skempton and DeLory (1957). In that Paper, ground-water level was taken as coincident with the surface. I now realize that this is too great an over-simplification.

Table 2
Values of R

Condition of clay	Stratigraphy	Location	Natural slope N cutting C (time to failure)	Residual factor: R
No fissures or joints Unweathered	Boulder clay	Selset	N	0.08
Fissures and joints Weathered	London Clay	Northolt Kensal Green Sudbury Hill	C (19 yrs) C (29 yrs) C (49 yrs)	0.56 0.61 0.80
		Sudbury Hill 10° slopes	C (after slip) N	1.04 0.92-1.06
	Coalport Beds	Jackfield	N	1.12
Movement on existing slip surface in <i>any</i> type of clay				<i>c.</i> 1.0

the peak value is small, or even negligible. Compacted clay fills, as used in embankments and earth dams, may well fall into this latter category.*

Finally, if a failure has already occurred, any subsequent movements on the existing slip surface will be controlled by the residual strength, no matter what type of clay is involved. It is probable that this is the main reason for the exceptionally low angles of shearing resistance (compared with conventional peak values) deduced from landslides such as those, for example, in the Bearpaw clay-shale in south Saskatchewan (Ringheim, 1964); and the analysis of the slope at Sudbury Hill, after the initial slip, provides supporting evidence for this interpretation. It is also probable that in shear zones, caused by tectonic movements, the strength will be at the residual value.

The case records from which these conclusions have been derived are still few in number. But they are perhaps sufficient to justify the belief that a more logical basis than existed hitherto is now available for analysing the long-term stability of clay slopes.

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* The three conclusions stated in this paragraph were first put forward in a discussion at the 5th International Conference on Soil Mechanics (Skempton, 1961).

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The Chairman invited the Society's distinguished visitor, Dr Laurits Bjerrum, Director of the Norwegian Geotechnical Institute, to propose the vote of thanks.

Dr Bjerrum said that the Rankine Lecture had proved to be not only an important yearly event for the Society in an international sense, but each of the printed papers formed a milestone which measured the progress of the science. Considering the Rankine Lecture from this viewpoint, Professor Skempton deserved to be complimented upon selecting a topic which concerned one of the most fundamental and central problems in soil mechanics.

Everybody would have been impressed by Professor Skempton's extremely clear and fascinating presentation of his subject. His unique ability to give a difficult problem an intelligible presentation might, however, convey to a listener who was not familiar with the subject an insufficient impression of the complication of the subject and of the amount of work which had been put into the Lecture.

Having had the good fortune to be able to follow very closely Professor Skempton's work on shear strength and stability of slopes, he could assure the audience that the Lecture they had just listened to was the result of many years' study, during which time many case records had been investigated. The work had been very much intensified during the past year.

It might be appropriate to mention that the number of case records shown during the Lecture was only a small fraction of those which formed the basis for the conclusions drawn in it. Each of them had been subjected to a most detailed investigation. As an illustration, during the period when Professor Skempton was working on the Norwegian case records—which had not been presented that evening—a selected group of engineers were busy answering the problems raised in Professor Skempton's letters and working to give an account of even the most minute details, including, for instance, the foraminifera which became buried in the clay during its deposition.

The origins of rational methods for analysing slopes dated back about 50 years. It was, however, when Terzaghi presented his classical concept of effective stresses and their importance in shear strength that an understanding of the subject was first reached. In the past 25 years important developments in the application of these principles had been experienced, and it was no exaggeration to say that the progress in this field was marked by the papers published by Professor Skempton and his co-Authors.

A detached outside observer who had tried to follow the development of the various

approaches to the problems of computing the stability of slopes might have been left with a somewhat confusing impression. The starting point was the $\phi = 0$ analysis which, due to its simplicity, was so readily developed, adopted and used. Its validity proved to be limited, however, and in most instances it was necessary to work in terms of effective stresses. This was a great step forward.

It proved, nevertheless, that for some clays a progressive softening was taking place and that complete reliance could not be placed upon the values that were determined in the standard tests. The cohesion intercept c' had in certain conditions to be reduced, and it had now also been learned that the ϕ' value might also be less than was found by conventional testing.

Dr Bjerrum had mentioned these features of the development as they provided a very welcome opportunity to emphasize one of Professor Skempton's greatest contributions, in bringing everybody into the picture by formulating simple, intelligible principles and drawing up limitations of their applicability.

It would be the wish not only of everyone present that night but of every engineer who had read any of Professor Skempton's papers, that their deeply felt gratitude should be expressed to the Lecturer for having guided them through the jungle of apparently unrelated observations with which they were so frequently faced in soil mechanics. His ability to bring order into the various matters was based upon a number of unusual gifts. Imagination and intuition were needed to see possible correlations and to formulate alternative working hypotheses. Courage was required for rejecting irrelevant considerations while accepting those which were believed to be essential. The first and foremost requirement, however, was a thorough and intimate familiarity with the fundamentals of the problems to ensure that the conclusions were deeply rooted in sound basic principles. Persons gifted with these abilities were, not surprisingly, very scarce. It was extremely fortunate that in the present period of very rapid development, those interested in the subject had such a gifted person as Professor Skempton in their midst.

It might not be fully appreciated among British engineers what a great impact Professor Skempton's work had had upon the science on a world scale. The present gathering was an occasion when this should be emphasized. Dr Bjerrum pointed out that he was properly qualified to make that statement since he had received so much stimulus and inspiration from Professor Skempton. In addition, he was sufficiently familiar with the conditions in Europe and in the United States to know the influence which Professor Skempton had had upon the development of the science all over the world.

It was, therefore, with a combination of warm personal feelings that he concluded his vote of thanks by expressing gratitude to Professor Skempton for his significant and interesting lecture, not only on behalf of all those present that evening, but also on behalf of future readers of the paper.

The vote of thanks was accorded unanimously by acclamation.