Investigations of Two Long-term Failures in London Clay Slopes at Wood Green and Northolt

Analyse de Deux Glissements à Long terme de Talus dans l'Argile de Londres à Wood Green et Northolt

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Summary

Long-term failures of a retaining wall and a cutting in London clay are described. Using measured ground water levels, effective stress methods of stability analysis are applied to the determination of the cohesion intercept c' operating at failure and these values are related to the values measured in the laboratory and implied by failure of natural slopes on a geological time scale. The way in which deep counterfort drains contribute to stability by the increases of effective pressure following the changed pattern of ground water flow is indicated.

Sommaire

On décrit des ruptures à long terme d'un mur de soutènement et d'une tranchée dans l'argile de Londres. Ensuite, en utilisant les niveaux phréatiques constatés, on détermine la cohésion c' qui agit à la rupture par une analyse de stabilité en contraintes intergranulaire. Les valeurs obtenues de cette façon sont comparées aux valeurs constatées au laboratoire et déduites par des ruptures de pentes naturelles à l'échelle géologique en fonction du temps. On indique la manière dont le drainage profond des contreforts contribue à la stabilité en augmentant la pression effective par un changement de l'écoulement de l'eau souterraine.

Introduction

The chief characteristic of stiff fissured clays is that they show a marked decrease in strength with time when the shearing stress applied to them is associated with reductions in lateral stress; as when cuttings or retaining walls are built. Many failures have taken place due to this decrease in strength with time, and the first explanation of how softening may occur was given by Terzaghi (1936).

The softening is usually confined to a relatively small volume of the clay and in some cases may be limited to the vicinity of the surfaces where sliding occurs. Undisturbed samples taken in these clays are unlikely to pick up these softened layers and in many cases, where failures have occurred, the measured undrained strength has been 3 to 4 times greater than that required for equilibrium (Skempton, 1948).

It has recently been shown (Bishop and Henkel, 1953) that the large undrained shear strengths measured in heavily overconsolidated clays, of which the stiff fissured clays are a special type, are due very largely to the negative changes in pore pressures which accompany the application of shearing stresses. In long-term stability problems any negative excess pore pressures set up in the construction period are able to dissipate and the pore pressures at any point in the ground will ultimately be determined by the prevailing ground water conditions. In these circumstances, where the pore pressures can readily be measured, effective stress methods of stability analysis offer great advantages over the $\phi = 0$ method which has commonly been used in these clays.

Effective stress methods have, however, been applied to a number of failures of natural slopes in fissured materials where the time between the downcutting, which produced the slope, and failure can be considered to be on a geological time scale (Suklje, 1953; Henkel and Skempton, 1955; Nonveiller and Suklje, 1955). Although the effective stress envelope determined from drained triaxial or direct shear tests for these heavily overconsolidated fissured clays has a cohesion intercept c', as well as an angle of shearing resistance ϕ', it was found that on the geological time scale the clay behaved as though $c' = 0$. The precise way in which the reduction in c' takes place is not fully understood, but it is thought that the softening of the clay in open fissures together with the dilatancy accompanying local shear deformations, which may occur in the vicinity of open fissures, play a major part.

Engineers, however, are often more concerned with time scales of up to 100 years, and research into the application of effective stress methods in this time range is being carried out at the Imperial College. Although the work is not yet complete, it is hoped that the description of two failures in London clay and their analyses by effective stress methods will be of interest to engineers concerned with the stability of walls and slopes in fissured clays.

The London Clay

The London clay is a fissured clay of marine origin laid down in Eocene times and it has been overconsolidated by some 500 to 700 ft. of deposits, subsequently removed by erosion.

In spite of its wide distribution its properties have been found to be remarkably uniform. The Liquid Limit varies only between 70 and 90, the Plastic Limit between 24 and 32, but is typically about 28, and the natural water content is usually slightly above the Plastic Limit. A number of drained triaxial tests, as well as direct shear tests, have been carried out on undisturbed samples from many sites and it has been found that the strength envelope in terms of effective stresses may be represented by the average values of $c' = 250$ lb./sq. ft. and $\phi' = 20$ degrees.

Method of Approach

If the values $c' = 250$ lb./sq. ft. and $\phi' = 20$ degrees are used in the analyses of wall and slope failures, factors of safety slightly greater than one are obtained, while if the geological time scale value of $c' = 0$ is used the factors of safety are slightly less than one. In order to find the value of c' that can be considered acting at failure the analyses have been made in such a way that, assuming ϕ' to be fully mobilized, the value of c' required for stability has been found.

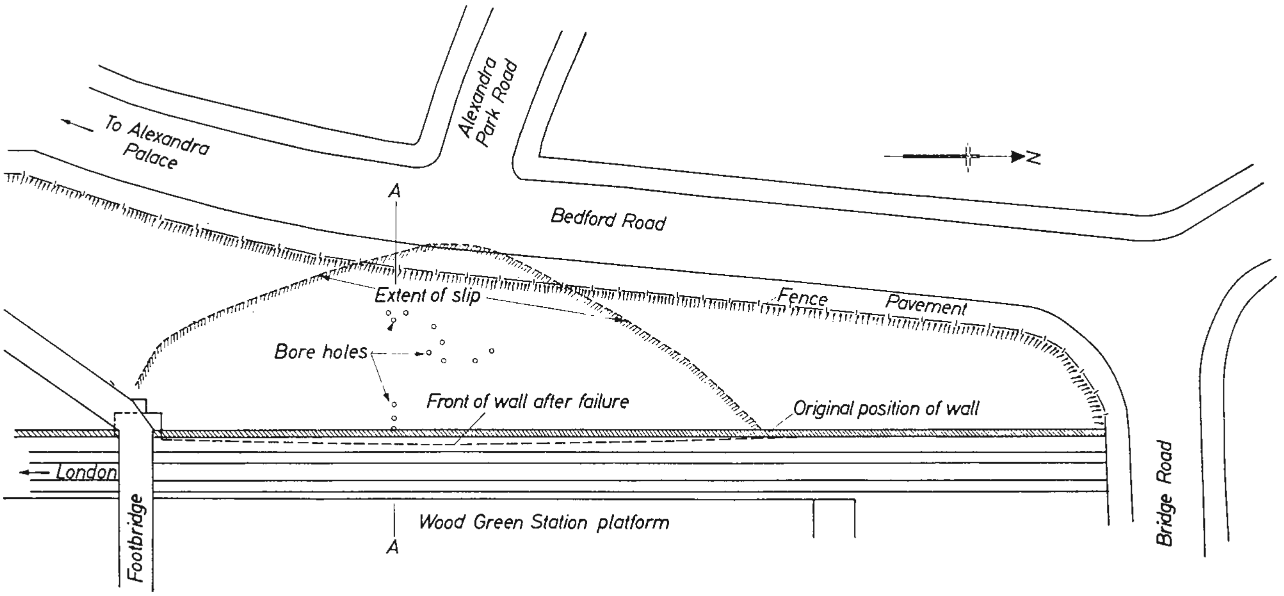

Fig. 1 Plan—Wood Green
Plan—Wood Green

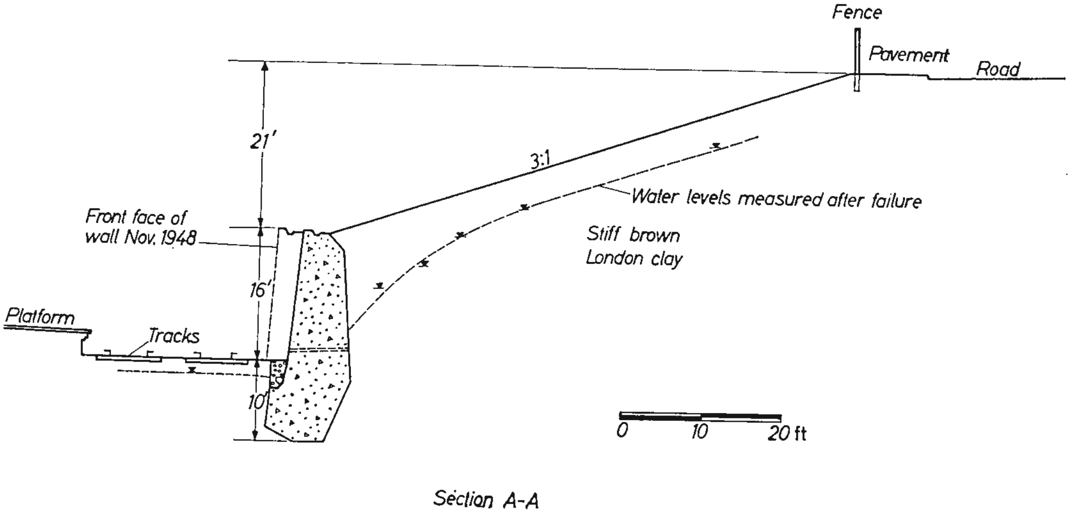

Fig. 2 Section through Wood Green retaining wall
Coupe du mur de soutènement à Wood Green

Failure of a Retaining Wall at Wood Green Station

Wood Green Station is built in a cutting in the London clay on the north eastern slopes of the hill upon which Alexandra Palace is sited. The retaining wall on the west side against the slope was built in 1893 and supports a clay face 16 ft. high with the ground surface rising a further 21 ft. at a slope of 3:1 until Bedford Road is reached. The plan, Fig. 1, and the section Fig. 2, show the original position of the wall and the ground profile at the time of construction.

In November 1948 the wall began to show signs of movement and some heaving of the track nearest to the wall occurred. Cracks opened up in the pavement of Bedford Road and by the time remedial measures had been completed the top of the wall had moved forward by 3 ft. The position of the wall at this stage is shown in plan and section by the dotted lines and the area of clay involved in the movement is indicated by the hatched line shown in the plan.

Borings* shown in the plan were put down in 1952 and 1953, and in a number of the holes 2 in. pipes were installed for the measurement of ground water levels. From the measurements made in these holes during the winter and spring seasons the ground water surface has been plotted on the section.

The clay behind the wall was found to consist entirely of brown London clay and, with two exceptions, undrained triaxial tests gave strengths ranging between 1500 and 3000 lb./sq. ft. The clay had an average water content of 31 per cent and the average Liquid and Plastic Limits were 78 and 30. Two samples, however, were found to be very wet and soft and had water contents of 38 per cent and a shear strength of about 800 lb./sq. ft.

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* These borings, and the subsequent tests, were made by the Civil Engineering Staff of the Eastern Region, British Railways, and by Messrs Le Grande, Sutcliffe and Gell.

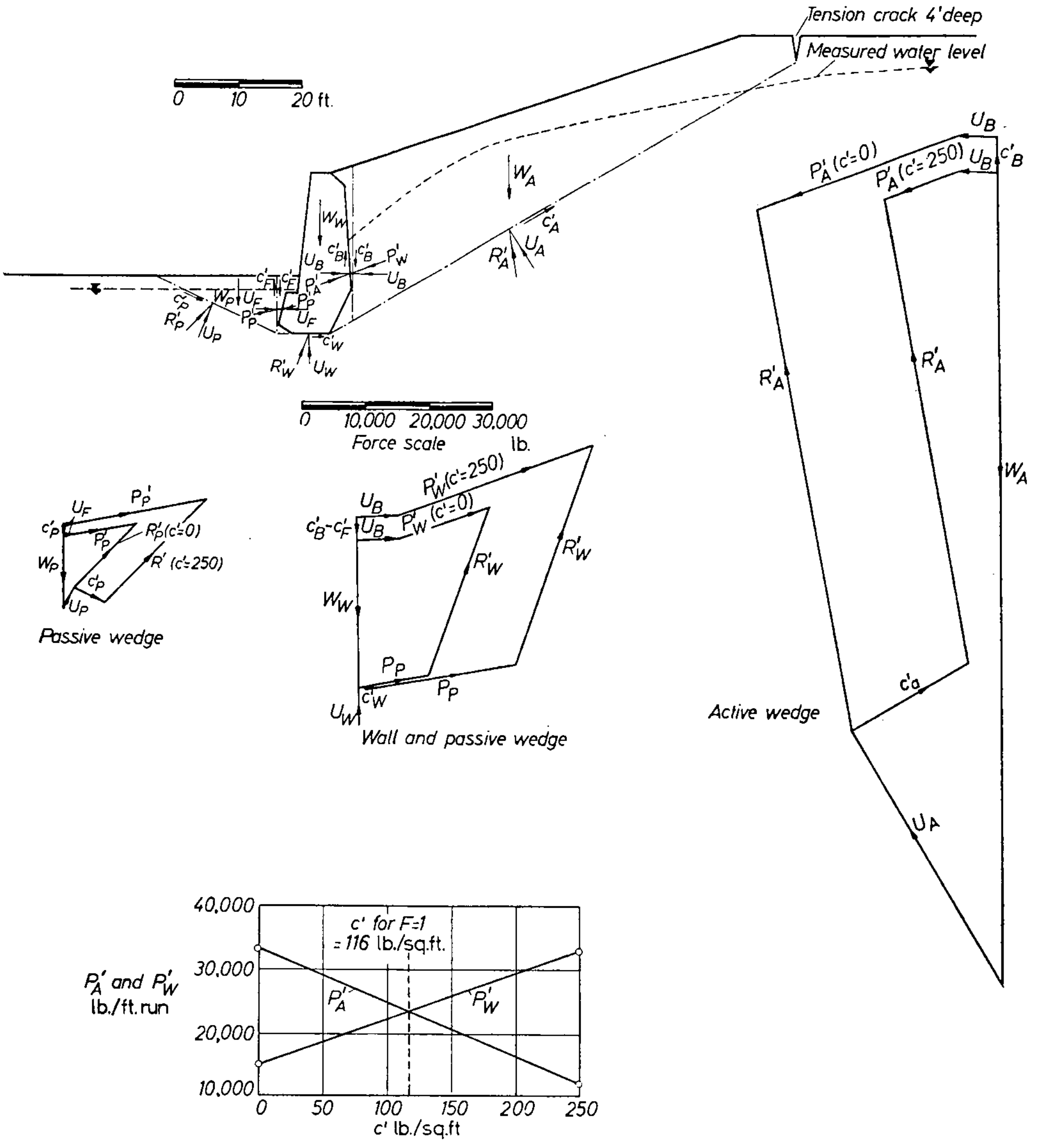


Fig. 3 Analysis of Wood Green failure
Analyse de la rupture de Wood Green

Analysis at Wood Green

The wall at Wood Green is of a rather curious shape and to simplify the analysis it has been assumed that the effective stress component of the active pressure is inclined at ϕ' to the horizontal, and the water pressure component is horizontal.

The procedure adopted in the analyses of retaining walls, illustrated by Fig. 3, is to determine by the wedge method the maximum active force, P'_a, which can be exerted on the back of the wall assuming the full mobilization of friction and cohesion on the assumed virtual back of the wall. The calculation is carried out for $c' = 0$ and $c' = 250$ lb./sq. ft. The minimum wall resistance for the two conditions $c' = 0$ and $c' = 250$ lb./sq. ft. is also found by combining the passive resistance of the wedge of clay at the toe of the wall with the resistance to sliding of the wall itself. These steps are shown in the force diagrams. For equilibrium the active force and the wall resistance must balance, and by plotting wall resistance and active force against c', the value of c' for equilibrium was found to be 116 lb./sq. ft.

Failure of a Cutting at Northolt

The cutting in which the failure occurred is immediately on the London side of Northolt Station on a line operated jointly by the Western Region of British Railways and the London Transport Executive. Originally the cutting, constructed in 1903, carried only the two Western Region lines but in 1936 it was widened on the south side to take the two electric tracks of the Central Line (L.T.E.). The cutting has a maximum depth of 33 ft. and was made with 2½:1 slopes rising from a small toe wall about 3 ft. high. Shallow surface drains some 4 ft. deep were installed at 33 ft. centres along the slope.

Early in 1955 substantial movements occurred, a series of tension cracks opened up near the top of the slope and the clay piled up on the small toe wall. The extent of the slip and a typical cross-section are shown in Figs. 4 and 5. Casagrande type piezometers were installed in December 1955 and the measured water levels are shown on the section.

Very soft clay with a water content of 44 per cent was encountered on the slip plane, and a laboratory vane test on a remoulded specimen of this clay gave a shear strength of 270 lb./sq. ft. Undrained triaxial tests on samples of clay taken above and below the slip plane, where the water content was about 30 per cent, gave shear strengths of 2100 lb./sq. ft. The Liquid and Plastic Limits of the clay were 78 and 28 respectively.

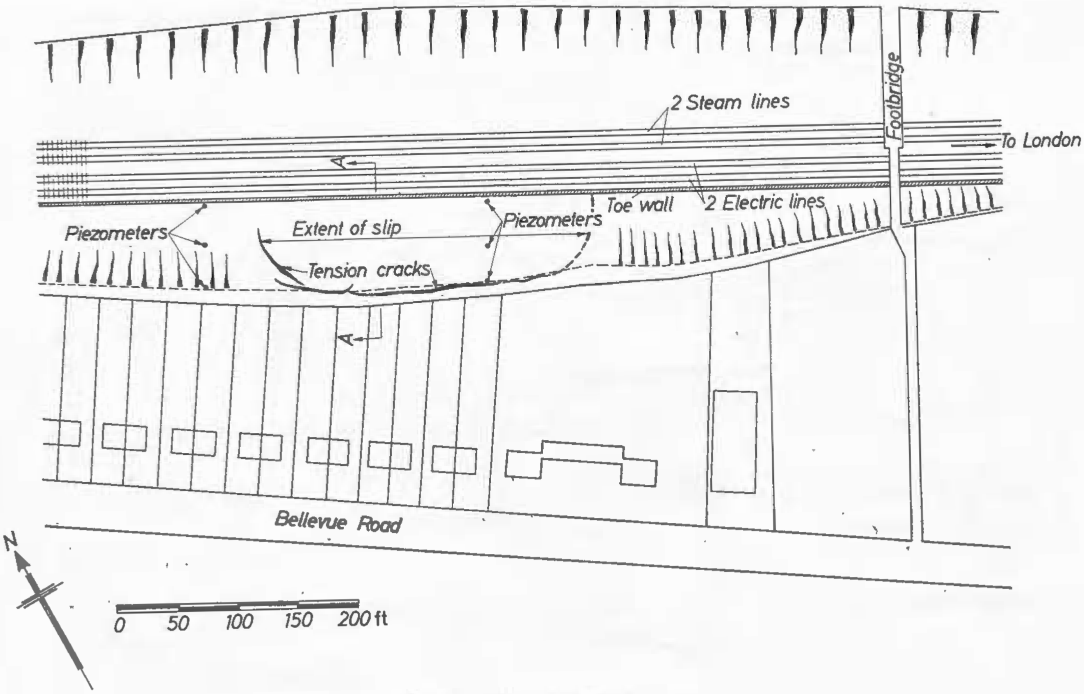
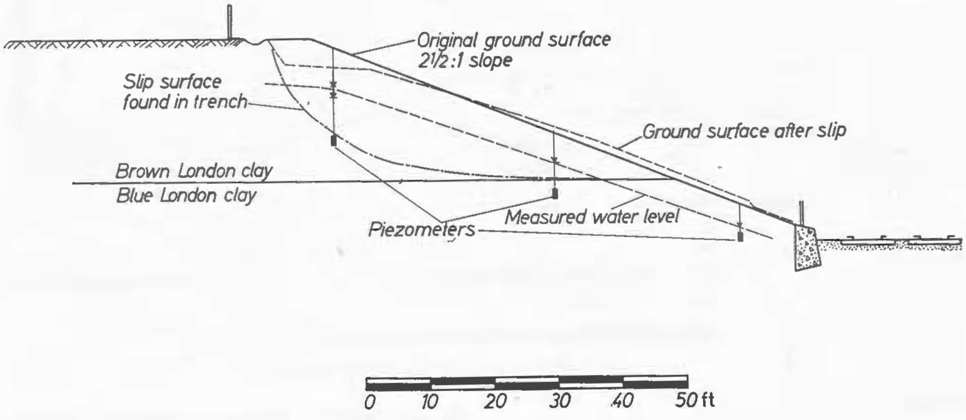


Fig. 4 Plan—Northolt
Plan—Northolt



Section A-A

Fig. 5 Section through Northolt cutting
Coupe de la tranchée de Northolt

Analysis at Northolt

An analysis has been carried out with a circular arc as a failure surface and using the procedure outlined by Bishop (1955) which takes into account the effects of internal forces. The small retaining wall at the foot of the slope appeared to be unaffected by the movements, and circles passing through the upper point of this wall have been considered.

In carrying out the analysis ϕ' was considered to be fully mobilized and the value of c' required for equilibrium has been found. The circle which gives the highest value of c' is the most probable failure circle and this maximum value was found to be 88 lb./sq. ft.

Significance of Results

The values of c' obtained from analyses of failures at a number of sites have been plotted in Fig. 6 against the time which has elapsed between the dates of construction and failure. Starting from the laboratory value of $c' = 250$ lb./sq. ft. at zero time an approximate curve has been drawn indicating the average rate of decrease in c' with time. It is unlikely however that time is the only element controlling the decrease in c' and it is probable that the height of the wall or slope and the relative ground water levels also play a part.

To put the variation of c' in its perspective it is useful to examine the values for factors of safety obtained at various sites for the two limiting conditions of $c' = 250$ lb./sq. ft. and $c' = 0$. The results of these calculations for the two examples described in the paper together with data from a wall at Uxbridge* are given in the table below.

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* This failure was described by Watson (1956) and details of the stability calculations given by the author in discussion.

	c' $F = 1$	F $c' = 250$	F $c' = 0$
Wood Green	116	1·32	0·78
Northolt	88	1·35	0·81
Uxbridge	138	1·18	0·82

This comparatively small range of factor of safety for the limiting values of c' shows the advantage of the effective stress methods over the $\phi = 0$ method. All the quantities necessary for the analysis can be measured directly and the chief uncertainty at any site lies in the estimation of c' within a fairly small range.

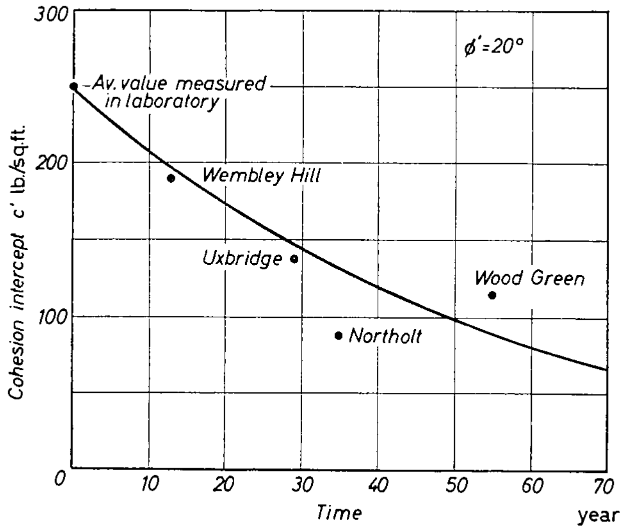

Fig. 6 Relation between cohesion intercept and time between construction and failure

Relation entre la cohésion et le temps écoulé entre la construction et la rupture

While no method can be expected to show precisely when failure is to be expected, a knowledge of ground water conditions and the use of the approximate curve of c' against time will enable a reasonable idea of the factor of safety at any site to be obtained. A further important point is that the method enables the effect of various remedial measures, particularly that of drainage, to be assessed in quantitative terms.

Due, however, to the additional softening accompanying the large shear strains at failure, the c' implied by failure will be further reduced and remedial measures should be designed using $c' = 0$.

Effect of Counterfort Drains in Stabilizing Cuttings and Walls

Deep counterfort drains have frequently been used to stabilize railway and road cuttings in clay. These have been designed on the principle that they should be carried down below the level of any slip plane discovered in excavations or in borings and the spacing between the drains depends on experience.

In addition to the transference of shearing stresses to a greater depth in the clay and their buttressing effect, the counterforts will materially increase the effective stresses in the clay by altering the ground water flow and decreasing the pore pressures. This process is illustrated by the study of an idealized two-dimensional case of drains 12 ft. deep, 4 ft. wide, spaced at 25 ft. centres. In Fig. 7 a pair of these drains is shown and under severe rainfall conditions it is considered that the water level can rise to the ground surface. Without drains, the pore pressure on a possible failure plane, say 10 ft. down, would correspond to a 10 ft. head of water. Due to the presence of the drains, however, the flow pattern under the same severe conditions would be altered, and the average pore pressure on the slip plane would be reduced to about 3 ft. of water. This would represent an increase in effective stress and shearing resistance of about 70 per cent. Under more typical conditions, where the ground water would not rise to the surface without drains, smaller increases will occur.

In order to solve the actual three-dimensional problem, information is required on how much of the water in the slope is derived from rainfall on the slope and how much flows in from the ground behind.

As water has to be squeezed out of the clay under the increased effective stresses some delay will occur between the time of installation and the time when consolidation is complete. Few data are available about the rate at which consolidation takes place in slopes of fissured clay, but evidence from piezometers installed at Northolt, and also at a site on Sydenham Hill, indicates that the pore pressure drops very much more rapidly than might be expected from a calculation in which the coefficient of consolidation, determined in the usual way from oedometer tests, is used.

Conclusions

Analyses of the failures in the London clay at Wood Green and Northolt show the advantage of using effective stress methods in reducing uncertainty in calculating the factor of safety, and under the limiting condition of $c' = 0$ the method underestimates the factor of safety by only 20 per cent. While the values of c' implied by the failures lie reasonably spaced between the laboratory value and the geological time scale value, a complete check on the reliability of the method requires that the presence of the stable walls and slopes should be adequately explained, and it is hoped to extend the work to cover these cases.

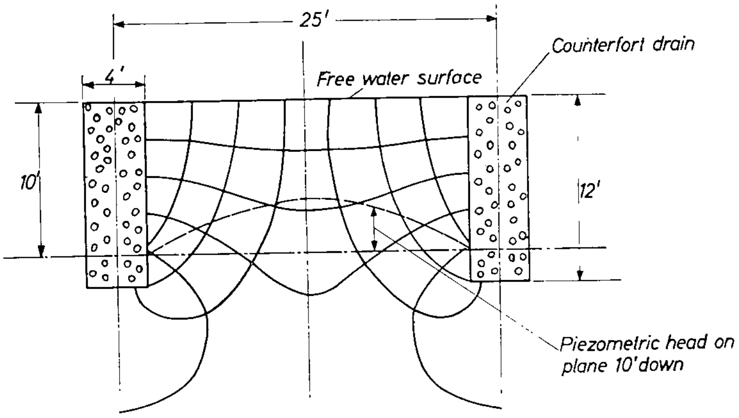

Fig. 7 Flow net showing influence of deep counterfort drains

Réseau d'écoulement montrant l'effet des draines contrefort profonds.

The author is grateful to the Chief Civil Engineer, British Railways Eastern Region, for his permission to use the data on the Wood Green wall and the Chief Engineer of the London Transport Executive for allowing field measurements to be made and details of the Northolt slip to be published. The research work has been carried out in the Civil Engineering Department of the Imperial College of Science and the author would like to express his thanks to Professor A. W. Skempton for helpful discussions and to Dr F. A. DeLory for carrying out many of the calculations and field measurements.

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