Slope Stability of Cuttings in Brown London Clay

Stabilité des Pentes de Voies en Tranchées au London Clay

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SYNOPSIS In this paper a summary is presented of research on first-time slides in cuttings in the brown London Clay: a classic example of the geological materials classified by Tergaghi (1936) as 'stiff fissured clays'. The two principal conclusions may be summarised as follows: (i) Failure generally occurs many years after excavation; and field evidence is now available to indicate that the main reason for this delay is a very slow rate of pore pressure equilibration, despite the fissured structure of the clay. (ii) Back analysis of typical long-term slips shows that the strength of the clay at failure corresponds rather closely to the 'fully softened' condition, or to the fissure strength; and that the field strength is greater than the residual but smaller than the peak strength, even as measured on large samples.

INTRODUCTION

Slope failures, or 'slips' as they are traditionally called, in London Clay cuttings present three inter-related problems (i) determining the shear strength at failure, (ii) deciding on the best means of measuring or predicting this strength in laboratory tests, and (iii) finding an explanation for the long delayed failures so characteristic of the slips.

Research into these problems has progressed from an initial stage in which the rate of softening of the clay was deduced, in terms of undrained strength, from the analyses of various case records; through a stage in which the analyses were carried out in terms of effective stress, but with an inadequate knowledge of the pore pressures; and finally to the current stage where the importance of the very slow rate of pore pressure equilibration, after excavation, has come to be recognised.

Further research is still required before the processes involved in first-time slides are fully understood, especially concerning the strength properties at failure.

BROWN LONDON CLAY

Where the London Clay extends up to ground level, or is covered only by a thin mantle of drift deposits, it is oxidised to a brown colour to depths of 5 to 15m. At greater depths the clay is blue-grey in colour. Small joints and fissures occur throughout the clay. The fissures are more numerous, and therefore of smaller size, in the upper zone. Studies by Skempton, Schuster & Petley (1969) show an average fissure size in the brown London Clay of around 4cm, ranging from a maximum of 10cm down to about 1cm. The fissures exhibit little preferred orientation in azimuth, but their dip angles tend to be either fairly steep or rather flat; not many dips are recorded around 45°. The joints are nearly always steeply dipping.

Throughout the London area, from which the case records have been obtained, the clay exhibits only minor variations in index properties. Thus a direct comparison can be made between any one site and the rest, and all the sites contribute towards establishing a unified interpretation of the phenomena.

Typical index properties of the brown London Clay, as quoted by Chandler & Skempton (1974), are given in Table I.

TABLE I

Typical properties of brown London Clay

water content	=	31
liquid limit	=	82
plastic limit	=	30
plasticity index	=	52
clay fraction	=	55 per cent
unit weight	=	18.8 kN/m^3

Below the depth of seasonal variation, 1.5 to 2m beneath ground surface, the undrained shear strengths increase from roughly 70 kN/m^2 to 160 kN/m^2 at a depth of 10m (Skempton 1959).

The London Clay is of Eocene age and, except for the lowest and highest parts of the stratum, it was deposited in a moderately deep marine environment. It has been over-consolidated by the erosion of at least 150m of sediments. In its natural state, away from excavations, the horizontal stresses exceed the vertical pressures, with values of K_o which have been estimated to be more than 2.0 in the top 15m (Skempton 1961). Subsequent to its deposition the clay, in the London area, has been subjected to gentle folding; bedding dips of more than 2° are rare. Stratification is usually not apparent and, when considered on the scale of a mass several cubic meters in size, the clay is a remarkably uniform material.

The mineralogy of the clay fraction (particles smaller than two microns) may be summarised from the work of Burnett & Fookes (1974) approximately as follows:

illite	47 per cent
montmorillonite	35
kaolinite	15
chlorite	3

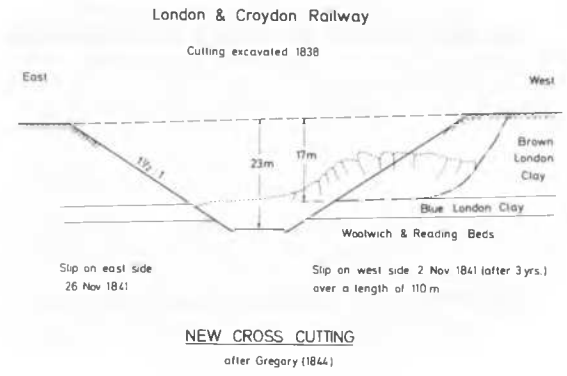
FIRST-TIME SLIDES

With a single exception, which has been included to point a contrast, all the slope failures mentioned here are 'first-time slides'. That is to say there has been no previous instability and the slip does not take place on a pre-existing shear surface.

EXAMPLES OF DELAYED FAILURES

(a) New Cross One of the earliest deep cuttings in London Clay was excavated in 1838 on the London & Croydon Railway. The line was opened in June 1839. On 2 November 1841, without warning, and in the course of four hours, nearly 40,000 cubic metres of clay slipped into the position shown in Fig. 1. Work was still in hand to clear the line when a similar large slip occurred in the opposite side of the cutting. The cutting was eventually stabilised by forming wide benches and making the slopes between the benches at 2:1, a total volume of 200,000 cubic metres of clay having been removed.

Gregory (1844) gives a good description of the slips and the geology of the cutting, and he records that the slip surface passed along the base of the brown clay. The fact that slips in cuttings do not penetrate any appreciable depth into the blue clay has been noted in several cases, for example at Northolt (Henkel 1957), and demonstrates that the



NEW CROSS CUTTING
after Gregory (1844)

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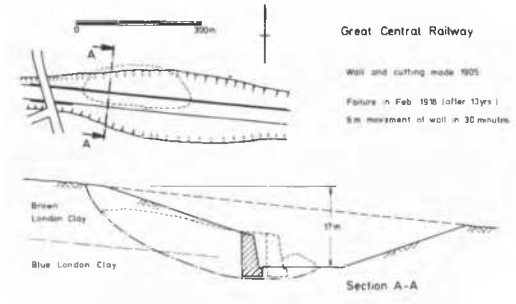
strength of the blue clay must be greater than that of the brown clay.

(b) Wembley Hill The cutting, with a retaining wall, shown in Fig. 2 was completed in 1905 at Wembley Hill on the Neasden-Northolt line of the Great Central Railway. Movement of the permanent way appeared in February 1918 and a few days later the wall slid forward 6m in less than half an hour (Anon, 1918). Owing to the presence of the wall the slip surface was forced into the blue clay. Piezometers were installed at this site in 1956, but it is now realised that the pore pressures measured then, 51 years after construction, are not relevant to an analysis of the failure in 1918.



WEMBLEY HILL CUTTING
after Skempton (1946)

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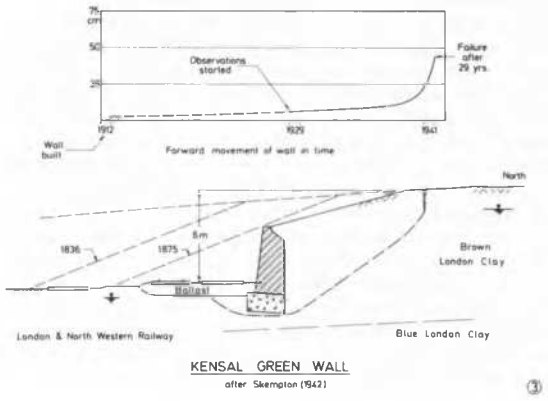
(c) Kensal Green This site lies to the east of Kensal Green station and tunnel. A cutting was first made in 1836 for the London & Birmingham Railway, renamed, after an amalgmation, the London & North Western Railway. The cutting was widened in 1875 and again in 1912, to accomodate the Euston to Watford electric lines, when the retaining wall shown in Fig. 3 was built. The wall extends west to the station, where it is higher, and a failure occurred there in 1927. After remedial measures had been carried out accurate surveys were made along the entire length of wall and repeated at regular intervals. They showed a gradually accelerating movement until, at

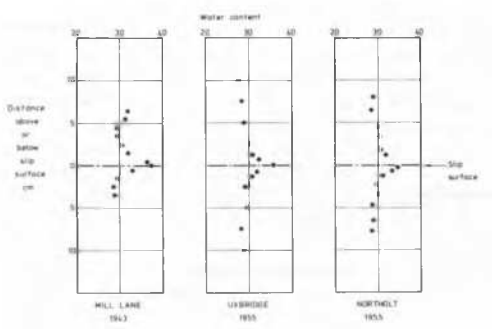
the section in Fig. 3, a tension crack appeared in the clay 6m behind the wall and the wall itself had slid forward by 30cm. This was in April 1941. Eight months later the movement had increased to about 40cm, the wall cracked, and remedial works were started.



KENSAL GREEN WALL
after Skempton (1942)

During these works, in January 1942, the writer investigated the clay. He noted the fissured structure and found slip surfaces behind and in front of the wall. It was also noted that the clay on the slip surface and adjacent to some fissures was much softer than in the main body of the stratum (Skempton 1942).

Next year, with his colleage W.H. Ward, the writer examined another retaining wall failure in London Clay, at Mill Lane, and detailed water content determinations were made across the slip surface. The results are plotted in Fig. 4 together with similar observations made by D.J. Henkel in 1955 at Uxbridge and Northolt.

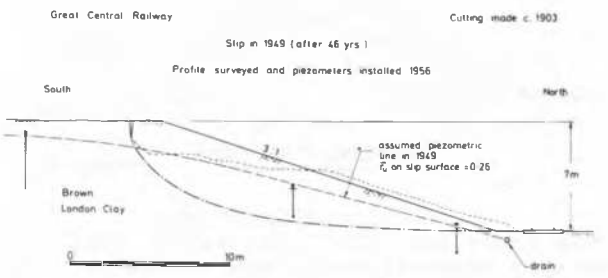


Distribution of water content near slip surface
after Skempton (1964)

The Kensal Green investigations established that there must be a reduction in strength of clay with time, and it was an easy matter to calculate the undrained strength at failure (16 kN/m^2) using the Ø = 0 analysis. In the same way the strengths at Mill Lane and several other sites were determined and, by plotting these strengths against the age of the cutting, a rough time-scale for the softening or strength reduction could be derived (Skempton 1948). The explanation of the softening process was assumed to be as given by Terzaghi in 1936; namely the infiltration of ground water into fissures opened in consequence of lateral movements following stress release during excavation.

(d) Sudbury Hill In 1953 the writer decided to examine the problem of slope stability in stiff fissured clays in terms of effective stress. Work began in the autumn of 1954 with D.J. Henkel and the able assistance of F.A. DeLory. New sites were found, piezometers were installed and laboratory tests carried out to determine the effective stress shear-strength parameters. By 1956 it was clear that the cohesion intercept c' at the time of failure was very considerably smaller than the value in laboratory tests. Moreover the data could be interpreted as showing a decrease in c' with time, approaching c' = 0 after several decades.

One of the newly discovered sites was a cutting at Sudbury Hill on the same line as Wembley Hill. The cutting dates from about 1903 and a slip occurred on the south side in 1949 (Fig. 5). As will be shown later, stability analyses of the first-time slide and also of the post-slip movements yield valuable results; but for the present it is sufficient to note that we had a 'long-term' case record of a slip in which pore pressures were measured only a few years after the event, and which showed that c' was very small.



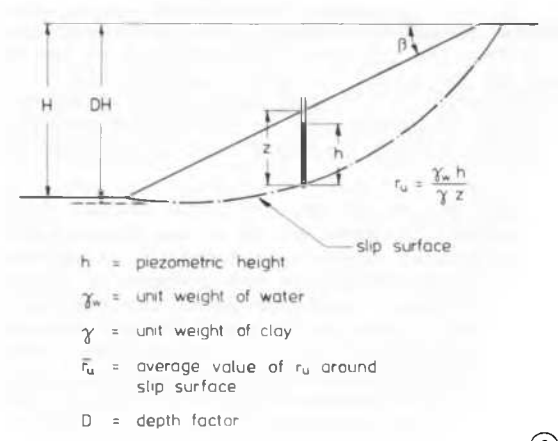
SUDBURY HILL CUTTING Section 1
after Delory (1957)

It is perhaps worth mentioning that the germ of the idea that clays may loose their 'cohesion' can be found in Rankine (1862) and several other works on civil engineering in the 19th century; while experience had shown that London Clay cuttings were generally not stable at slopes steeper than 3:1 (Baker 1881).

PORE PRESSURES

The most convenient parameter for characterising the piezometric conditions in a slope is the average pore pressure ratio $\bar{r}_u$ introduced by A.W. Bishop in 1960 and

defined in Fig. 6.



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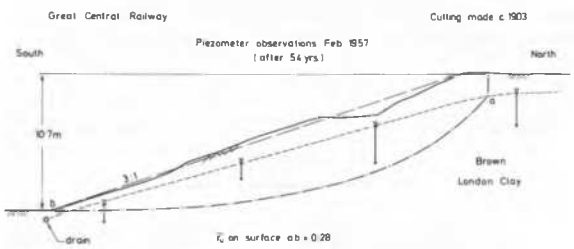
For the south side of Sudbury Hill cutting (Fig. 5) the value of $\bar{r}_u$ along the slip surface is about 0.26. Piezometers were also installed on the north side (Fig. 7) and they gave an average value on a typical surface of 0.28, despite the fact that some trench drains had been placed in this slope two years before the piezometers; though the piezometers were of course located mid-way between the drains.



SUDBURY HILL CUTTING Section II

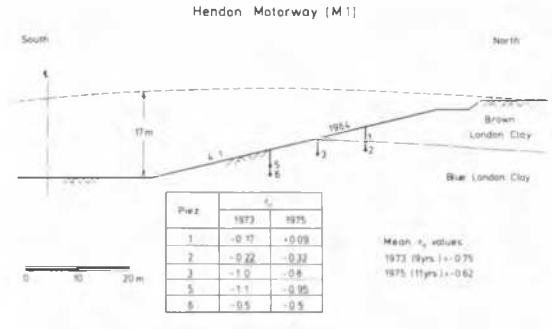
after Skempton & Henkel (1960)

From these and a few other rather fragmentary observations it was decided by 1970, when P.M. James completed the next research thesis on London Clay slopes, that the long term value of $\bar{r}_u$ could be taken as lying between 0.25 and 0.35, and that no important errors would be involved in taking 0.3 as a typical figure for back analysis (or design) in the absence of reliable piezometric data at any given site.

Under the direction of N.R. Morgenstern and the writer, James had discovered some more sites and analysed these slides, as well as most of the earlier ones, using the method of Morgenstern & Price (1965) for non-circular slip surfaces. The results, more numerous and precise than those previously available, appeared to confirm the interpretation regarding the decrease in c' with time. They also led to a conclusion of major importance: namely that the lower bound of all the strengths calculated from back-analysis of first-time slides lay well above the residual strength (Skempton 1970).

However, by 1973 a radical change had occurred in our concept of the physical process responsible for delayed failures. Already it was known that the rate of pore pressure equilibration in clay fill embankments could be extremely slow. Observations made in 1971 under the direction of P.R. Vaughan had shown, for example, that negative pore pressures were still existing in a 16m high dam, built in 1963, with clay having average liquid and plastic limits of 47 and 22 respectively, even in the upstream shoulder six years after impounding (Walbancke 1973). This result brought forcibly to mind the fact that no field studies were available on the rate of equilibration in London Clay cuttings; the tacit assumption had been that, owing to the fissured structure and the possibility (suggested by Terzaghi) of the fissures opening up during excavation, the in-situ permeability would be relatively high.

It was then decided to install piezometers in a London Clay cutting which had been excavated in recent times. The only one we knew was on the Hendon Motorway (M1) at Edgwarebury. In 1972 the Science Research Council made a grant for the work to be done, and Miss Walbancke obtained the first reliable readings in the early months of 1973. These showed negative pore pressures at each of the five piezometers (Fig. 8) although the cutting had been completed nine years earlier in 1964 (Vaughan & Walbancke 1973). The piezometers were of the twin-tube hydraulic type developed by A.W. Bishop, with high air-entry ceramic filters (Bishop et al 1960).



Piez	r_u 1973	r_u 1975
1	-0.17	+0.09
2	-0.22	-0.32
3	-1.0	-0.8
5	-1.1	-0.95
6	-0.5	-0.5

Mean r_u values:
1973 (8yrs.) = -0.75
1975 (11yrs.) = -0.62

EDGWAREBURY CUTTING

after Walbancke & Vaughan (1973) and Walbancke (1976)

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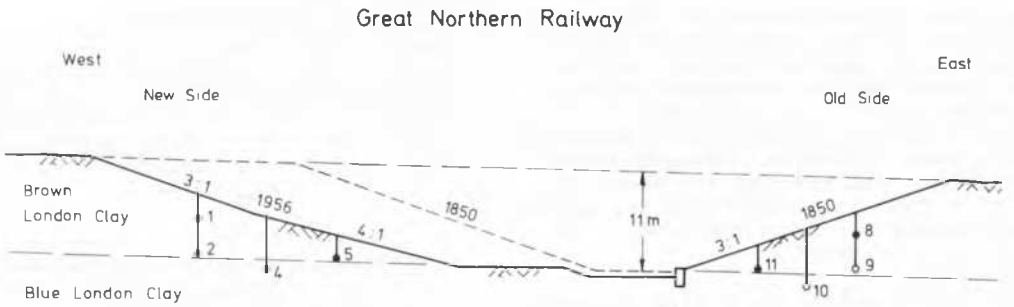
It has of course long been known that the removal of load by excavating a cutting would cause an immediate reduction in pore pressure. What caused surprise at Edgwarebury was that the pore pressures were still negative after 9 years, and calculations indicated a coefficient of swelling not dissimilar in magnitude from values measured in the laboratory on small undisturbed samples. Clearly in this case the fissures had little effect

on in-situ permeability of the clay mass after excavation.



Piezometer readings 1975

New Side (19 yrs.)	Piez	r_u	Piez	r_u	Old Side (125 yrs.)
mean r_u = 0.15	1	0.06	8	0.31	mean r_u = 0.32
	2	0.18	9	0.34	
	5	0.21	11	0.31	
	4	0.09	10	0.32	

POTTERS BAR CUTTING

after Walbancke (1976)

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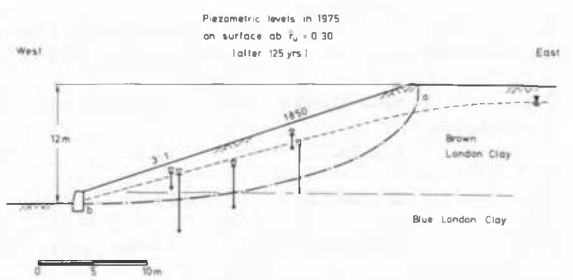
Unfortunately the Edgwarebury cutting is predominately in blue London Clay. It therefore became a matter of great interest to see whether the same conclusion applied in the brown clay. After an intensive search an ideal site was discovered at Potters Bar. A cutting here, on the main line from King's Cross to York, had been made in 1850, to a depth of 11m, entirely in the brown clay. But so recently as 1956 the cutting was widened on the west side, leaving the old east side unaltered except for deepening by 1 metre and the construction of a small toe wall (Fig. 9). Piezometers were installed in 1974, and the observations for 1975 are summarised in Fig. 9. There can be no doubt that in the east side, after 125 years, the pore water pressures have reached a state of equilibrium; and it is interesting to note that the average value of $\bar{r}_u$ along a representative (imaginary) slip surface is 0.30 (Fig. 10). In striking contrast the pore pressures of the west side, after 19 years, are only about one-half of the equilibrium values, although there is no essential difference between the two sides other than age.

Taking all the pore pressure evidence into account it seems that equilibration is indeed a slow process; at least 40 to 50 years, for practical purposes, being required for its completion in typical cases (Fig. 11). If this is correct, the conclusion is inescapable that we have here the principal reason for delayed failures in London Clay cuttings.



POTTERS BAR CUTTING . Old Side

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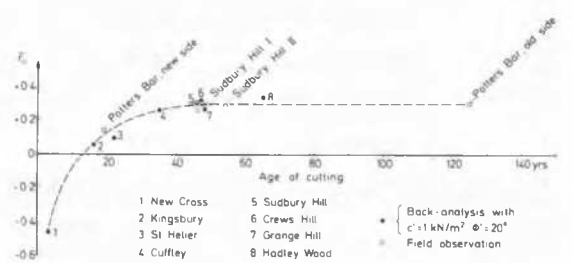

Variation of $\bar{r}_u$ with time : cuttings in Brown London Clay

depth more than 6 metres

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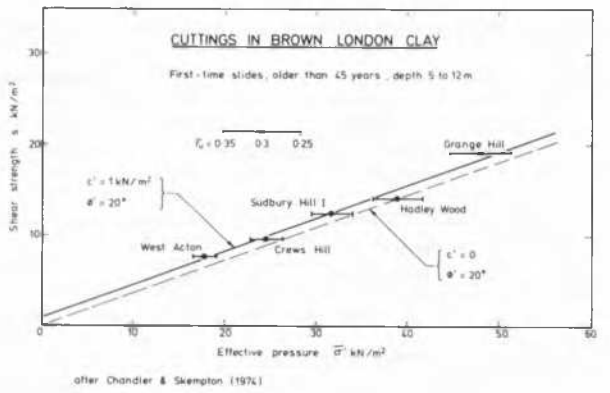
SHEAR STRENGTH

(a) Back analysis, first-time slides From the available case records five have been chosen as being 'long term' (more than 45 years to failure) and as covering a reasonably wide range of depth (and therefore of effective pressure). The basic data are set out in Table II. Each slip is analysed taking a factor of safety = 1.0, and assuming $\bar{r}_u$ = 0.30, to find the average shear strength and average normal effective pressure along the slip surface. The results are shown by solid points in Fig. 12. Effective pressures are also calculated for $\bar{r}_u$ = 0.25 and 0.35, as indicated in this graph.



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The best fit to the points is a line defined by the parameters

$c' = 1\ kN/m^2 \qquad \phi' = 20^o$

and a lower limit is given by

$c' = 0 \qquad \phi' = 20^o$

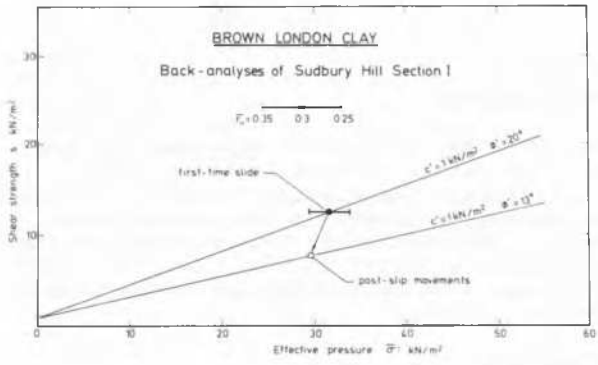
TABLE II

First-time slides in brown London Clay

	Site	Date of cutting	Date of slip	Time to failure years	Height m	Slope
Inter-mediate	New Cross	1838	1841	3	17.0	1½:1
	Kingsbury	1931	1947	16	6.0	2 1/4:1
	St Helier	1930	1952	22	7.0	2:1
	Cuffley	1918	1953	35	7.2	2 3/4:1
Long-term	Sudbury Hill	1903	1949	46	7.0	3:1
	Crews Hill	1901	1956	47	6.2	3 1/3:1
	Grange Hill	1902	1950	48	12.2	3 1/4:1
	West Acton	1916	1966	50	4.9	3:1
	Hadley Wood widened	1850 1916	1947	c.65	10.4	3 2/3:1

(b) Back analysis, post-slip movements
After the slip in 1949 at Sudbury Hill (Fig.5) no remedial works were undertaken; the toe of the slip was merely trimmed back. Further small movements occurred in succeeding winters, and were similarly treated. Piezometer levels during these post-slip movements are known and it is therefore possible to calculate with some accuracy the residual shear strength and average effective normal pressure. The results are compared in Fig. 13 with those obtained from back-analysis of the first time slide.

From Fig. 13 it is seen that the strength mobilised in a first-time slide is significantly greater than the residual strength; a conclusion which has been emphasised already (Skempton 1970) but is not widely appreciated.



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(c) Laboratory tests The peak strength parameters of brown London Clay as measured in 6cm shear box tests or 38mm diameter triaxial tests, are:

$$c' = 14 \text{ kN/m}^2 \qquad \phi' = 20^\circ$$

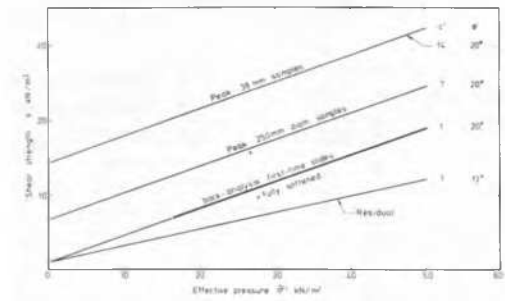
Samples of this size are too small to include a representative assemblage of fissures, so tests have been made on the largest triaxial samples (250mm diameter) which can be handled at all conveniently in the laboratory (Sandroni 1977). The resulting parameters

$$c' = 7 \text{ kN/m}^2 \qquad \phi' = 20^\circ$$

certainly show a marked reduction from those obtained in the usual small sized samples; but even so the strength is around 30 to 70 per cent in excess of the field values obtained from back analysis (Fig. 14).

Laboratory tests to measure the residual strength on natural slip surfaces in the brown London Clay (Skempton & Petley 1967) give the parameters

$$c' = 1.4 \text{ kN/m}^2 \qquad \phi' = 13^\circ$$



BROWN LONDON CLAY summary of shear strengths

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These are in good agreement with the analysis of post-slip movements at Sudbury Hill (Fig.13) but of no relevance to first-time slides, while ring shear tests give even lower results (Bishop et al 1971).

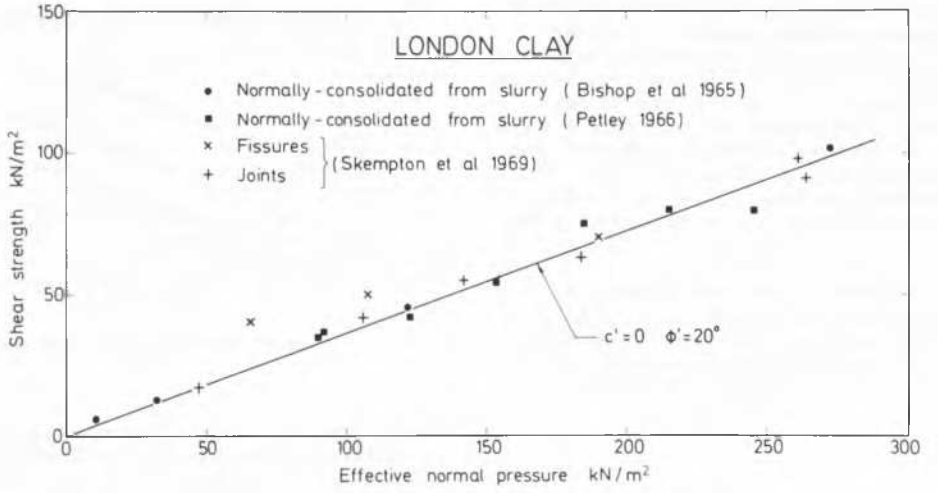
We therefore return to the conclusion (Henkel 1956, Scholfield & Wroth 1968, Skempton 1970) that the most appropriate laboratory parameters are those for the 'fully softened' or 'critical state' condition, which can be determined by measuring the strengths of remoulded, normally consolidated clay. For London Clay (Gibson 1953, Bishop et al 1965, Petley1966) these parameters are approximately

$$c' = 0 \qquad \phi' = 20^\circ \quad \text{(Fig. 15)}$$

However, a few tests have been carried out to measure the strength of joints and fissures in London Clay (Skempton et al 1969) and the parameters representing the lower limit of the results are also approximately $c' = 0$, $\phi' = 20^\circ$. (Fig. 15).

(d) Conclusions It appears that the displacements preceding a first-time slide are sufficient to cause some progressive failure, reducing the strength towards the fully softened or the lower limit of fissure strength; but the displacements are not so large as to reduce the strength to the residual value. Field evidence in support of these conclusions is provided (i) by the observation at the Uxbridge retaining wall failure (Watson 1956) that a continuous, single slip surface had not yet been developed with a total movement of about 40cms, and (ii) by the fact that after the slip at Sudbury Hill, when the strength had fallen to the residual, the displacements were about 1.5 to 2m.

Some explanation is also required for the progressive failure mechanism which takes the clay past its peak strength, and this can probably be attributed in part to the presence of local stress concentrations at the fissures.



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RATE OF EQUILIBRATION

If the assumption is made that the shear strength parameters c' = 1 kN/m^2 and $\phi' = 20^o$ apply at failure in all cases, it is possible to evaluate the average pore pressure ratio $\bar{r}_u$ for each of the first-time slides for which adequate data are available (Table II). The results, when plotted against time to failure (Fig. 11), show a consistent trend from a strongly negative value at New Cross (3 years), though small positive values at St Helier and Cuffley (around 20 years) up to the full equilibrium value of $\bar{r}_u = 0.30$ after about 50 years. It is of particular interest that the point representing the observed pore pressures in the west side of Potters Bar cutting ($\bar{r}_u = 0.14$ on a typical slip surface) lies practically on the line deduced from back analysis of the slope failures, and thus provides an independent check on the validity of this method of calculating the pore pressures.

It is of course apparent that the rate of change of pore pressure will depend to some extent on the dimensions of the cutting. For this reason the slides used in deriving Fig. 11 have been selected to exclude cuttings of unusually shallow depth; in these it would be expected that equilibration is achieved on a shorter time scale, but the number of such cases is too small for a graph to be drawn.

Slips in the zone of seasonal variation have also been excluded. They take place after exceptionally heavy rainfall, especially following a prolonged dry season.

NATURAL SLOPES

Finally it may be pointed out that the stability of natural slopes in brown London Clay is a different and distinct problem, in which the residual strength is the controlling factor. Information on this subject is given by Skempton & Delory (1957), Skempton (1964), Hutchinson (1967 and 1974) and Hutchinson & Gostelow (1976).

CONCLUSIONS

(i) The shear strength parameters of the brown London Clay relevant to first-time slides are:

$$c' = 1 \text{ kN/m}^2 \quad \text{and} \quad \phi' = 20^o$$

(ii) The peak strength, even as measured on large samples, is considerably higher; so some progressive failure mechanism appears to be involved.

(iii) The in-situ strength is given approximately by the 'fully softened' value and also by the lower limit of strength measured on structural discontinuities (joints and fissures).

(iv) The residual strength is much smaller than this and corresponds to the strength mobilised <u>after</u> a slip has occurred, with large displacements of the order 1 or 2m.

(v) It is a characteristic feature of first-time slides in London Clay that they generally occur many years after a cutting has been excavated.

(vi) The principal reason for this delay is the very slow rate of pore pressure equilibration; a process which in typical cuttings is not completed, for practical purposes, until 40 or 50 years after excavation.

Acknowledgements

The first stage of this research was carried out during the years 1942-46 at the Building Research Station under the direction of Dr L.F. Cooling. In the second stage 1954-1970, the writer acknowledges the work by his former colleagues and students at Imperial College, particularly Dr F.A. DeLory, Dr D.J. Henkel, Professor N.R. Morgenstern and Dr P.M. James. The third stage of research, from 1972, has been carried out principally by Dr P.R. Vaughan and Dr Jane Walbancke. Dr R.J. Chandler has also contributed much by discussion and by his parallel work on the Lias Clay.

Co-operation from British Rail throughout the entire period is gratefully acknowledged and especially the help of Mr D.J. Ayres.

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