LONG-TERM FAILURE OF A BANK ON A SOLIFLUCTION SHEET

RUPTURE A LONG TERME D'UN REMBLAI SUR UNE COUCHE DE SOLIFLUCTION

ДЛИТЕЛЬНОЕ РАЗРУШЕНИЕ БЕРЕГА НА СЛОЕ, СКЛОННОМ К СОЛИФЛЮКЦИИ

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SYNOPSIS. A railway embankment in Northamptonshire,England,showed considerable instability some 70-80 years after construction. The bank, on an 8 1/2° slopes, was founded on a solifluction (fossil periglacial mudslide) sheet,dating back at least 10,000 years to the last glacial period. The shear surface at the base of the solifluction sheet proved to be the seat of the instability of the bank. The limited movements probably developed after the establishment of long-term pore water pressures, controlled by a perched water table in the ballast beneath the railway track. Stability analyses suggest that the bank was only unstable under the increment of undrained loading provided by passing trains,a conclusion consistent with the small movements of the bank. If the bank had not been previously stabilized by grouting, the progressive displacements would have led to complete failure.

INTRODUCTION

It is generaly assumed that the critical period for the stability of embankments constructed on weak foundations will be during or shortly after construction, and that as a result of the consolidation of the foundation material the factor of safety will increase with time. A few recent failures of railway embankments, constructed during the last century on areas that had previously been subject to landsliding or solifluction, suggest that this assumption may not always be correct. One of these failures is the subject of this paper; as part of the movement was within the bank, the strength of the embankment fill has to be considered as well as that of the pre-existing shear surface beneath the bank. A guide to the strength of the embankment fill is provided by a second failure at an adjacent site, where the foundation soil was not involved, and where the fill material appears to be very similar.

The piezometer observations at the two sites have some interesting implications for the long-term stability of embankment slopes.

THE EMBANKMENT FAILURE AT GRETTON, NORTHAMPTONSHIRE

The railway between Kettering and Nottingham in midland England runs northeast along the foot of the main Jurassic escarpment in north Northamptonshire, before crossing the Welland valley into Rutland. The escarpment, which has a vertical height of about 60m, is composed of the heavily overconsolidated Upper Lias Clay capped at its crest by the Inferior Oolite limestones and ironstones. Where the escarpment slope is steepest it is extensively landslipped, while on the more moderate slopes (8°-9° and flatter) there is usually little topographic expression of landsliding but it is known from subsurface exploration that spreads of fossil mudflows (or, more properly, mudslides), often termed 'solifluction' sheets, are likely to occur.

These solifluction sheets developed during the periglacial conditions that existed beyond the margins of the main ice sheet during the most recent (Weichselian) glaciation of England. The main engineering importance of these fossil solifluction deposits in areas of clay outcrop is that as a consequence of their downslope movement by landslide processes they are underlain by extensive, polished shear surfaces that run roughly parallel to the ground surface, and have a much lower strength than the clay around them (e.g. Skempton and Hutchinson, 1969).

The location of the two embankment sites in relation to the Lias clay outcrop and limits of the icesheet of the last glaciation are shown in Fig. 1.

Unstable section of embankment. Close to the village of Gretton (Grid Ref.SP899951) the railway crosses, on a low embankment, a section of the Lias clay escarpment where the natural slope is about $8\frac{1}{2}^{o}$. The embankment, which carries two tracks, was constructed about 1878, probably by end tipping to provide a bank under one track, then widened by side tipping to take the second track.

In 1961, following slow subsidence, perhaps over many years, of the track on the downslope side, an investigation was initiated by British Rail, who sunk borings and also installed alkathene tube slip indicators through the embankment into the *in situ* Lias clay beneath. Fig.2 is a cross-section of the bank at this point showing the borings and slip indicators.

The deflection of the slip indicators, detected by the jamming of a rod passed down the tube, showed the embankment to be moving on a shear surface parallel to and about 1.2m beneath the original ground surface. Some heave had occurred at the toe of the embankment. The slip indicator on the upslope side of the bank showed no movement confirming that the instability was limited to the downslope half of the bank.

The existence of a solifluction sheet, with its underlying shear surface, was confirmed by a trial pit excavated as part of a further investigation carried out as a research project during the period 1970-72.

FIG.1 LOCATION OF SITES DESCRIBED IN PAPER

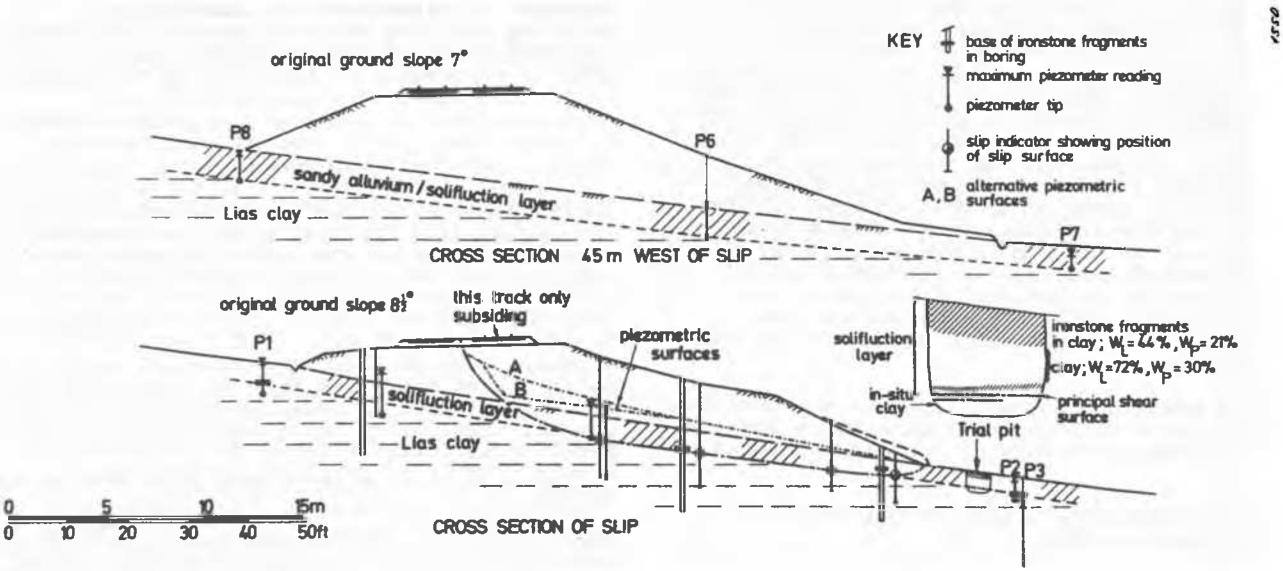

FIG.2 EMBANKMENT FOUNDATION FAILURE, GRETTON, NORTHAMPTONSHIRE

This trial pit, dug in ground undisturbed by the embankment movements (see Fig.2) showed that the solifluction layer consisted of reworked Lias clay with bands of ironstone-rich clay above a polished shear surface. Water seeped into the trial pit along these layers. These ironstone fragments are derived from the 'cap-rock' at the crest of the escarpment, about 100m upslope from the embankment. The depth to which ironstone fragments were noted, both in the original borings and in those carried out more recently for piezometer installation, also provides evidence for the thickness of the solifluction layer, which as seen in Fig.2, agrees well with the depth of the slip surface beneath the embankment. There can be no doubt that the bank instability is due to the reactivation of movement on the shear surface beneath the solifluction layer.

As a result of the 1961 investigations the embankment was stabilized by grouting, using the methods described by Toms and Bartlett (1962).

The 1970-72 investigation was largely an attempt to assess the likely piezometric levels on the slip surface within and under the bank to enable an effective stress stability analysis to be carried out. This proved to be difficult due to the presence of grout in the bank, which prevented boring. Casagrande type piezometers (P1, P2 and P3) were, however, installed late in 1970 in the natural ground either side of the bank. In addition, the two slip indicator tubes installed either side of the track in the top of the bank remained from the 1961 investigation. It has been found at other similar sites that these tubes, where not sealed by grouting, show water levels that compare closely with adjacent Casagrande piezometers. These two tubes therefore provide piezometric data for points beneath the bank. The downslope tube gives data related to two levels, since after the piezometric level relative to the base of the tube had been established, the bottom of the tube was blocked with cement grout and the tube cut at the point of contraflexure just above the slip surface. The piezometric levels relative to these depths are shown in Fig.2, and clearly indicate a downward component of flow, as do piezometers P2 and P3 at the toe of the bank. All the piezometric levels shown, both at Gretton and at the site at Seaton described subsequently, are the maxima recorded in the period December 1970 to April 1972; in almost every case these maxima occurred in February - March 1972.

Stable section of embankment. Three further piezometers (P6, P7 and P8) were installed in an apparently stable section of the bank 45m west of the unstable area in order to investigate the reasons for the stability at this point and to obtain further piezometric data at the level of the base of the solifluction layer. This section of the bank is also shown in Fig.2. The embankment at this point crosses a slight hollow and consequently the ground slope is less than at the unstable section, 7° compared to $8\frac{1}{2}^{\circ}$. The borings in which the piezometers were installed showed that the shallow layer of surface material on the natural slope differed from the solifluction deposits at the unstable section. In particular sand layers were encountered in each boring, probably alluvial material deposited in the hollow. A trial pit was also excavated at this section which confirmed the alluvial material encountered in the borings; as would be expected for alluvium, no shear surface was located at the base of this material.

Piezometer P6 under the embankment and piezometers P7 and P8 at the toes of the side slopes showed similar piezometric levels and fluctuated in a similar manner, evidence that here the sandy alluvium is acting as a drainage layer, preventing the buildup of pore-pressures beneath the bank.

		w_L %	w_P %	w %	c_u kN/sq.m. (lb/sq.ft)	γ kN/cu.m. (lb/cu.ft.)
GRETTON	(a) unstable bank	-	-	32	38.5 (800)	18.4 (117)
	(b) stable bank	63	29	29	-	18.5 (118)
	(c) solifluction layer	72	30	32	38.5 (800)	18.4 (117)
SEATON	(a) embankment	59	29	30	-	18.5 (118)

TABLE I. AVERAGE INDEX PROPERTIES OF THE LIAS CLAY AT THE TWO SITES

THE EMBANKMENT FAILURE AT SEATON, RUTLAND

Two miles (3km) north east of Gretton a slip occurred in the 12m high embankment just north of the viaduct over the Welland valley, near the village of Seaton, Rutland (Grid Ref. SP914982). This slip, which took place entirely within the bank, is relevant to the Gretton failure for two reasons. Firstly it provides a measure of the strength of the fill material, which at both sites, is Lias clay of similar index properties (Table I). Secondly, it provides piezometer observations which enable an upper limit to be set to the likely piezometric levels in the Gretton embankment.

The slip occurred at the end of January 1961 after slight settlements of the one track involved in the failure had continued at an accelerating rate over many years. Aerial photographs taken in 1947, fourteen years prior to the slip, show the fence line at the toe of the bank to be slightly displaced at the point where failure eventually occurred. The final rapid movement to the 'after-slip' profile shown in Fig.3 took place in a period of less than an hour, and at least twelve hours after the last train passed.

The position of the slip surface was located after the failure, with alkathene tube slip indicators. No further investigation was undertaken until 1971 when piezometers P1 to P5 were installed at the depth of the observed slip surface, but either side of the site of the landslip.

Over the 1971-72 winter these piezometers indicated high water pressures towards the crest of the bank with considerably lower pressures near the foot of the slope. Such a water pressure distribution is almost certainly the result of the downward flow of water ponded in the ballast beneath the rail way track. As the crest of the embankment

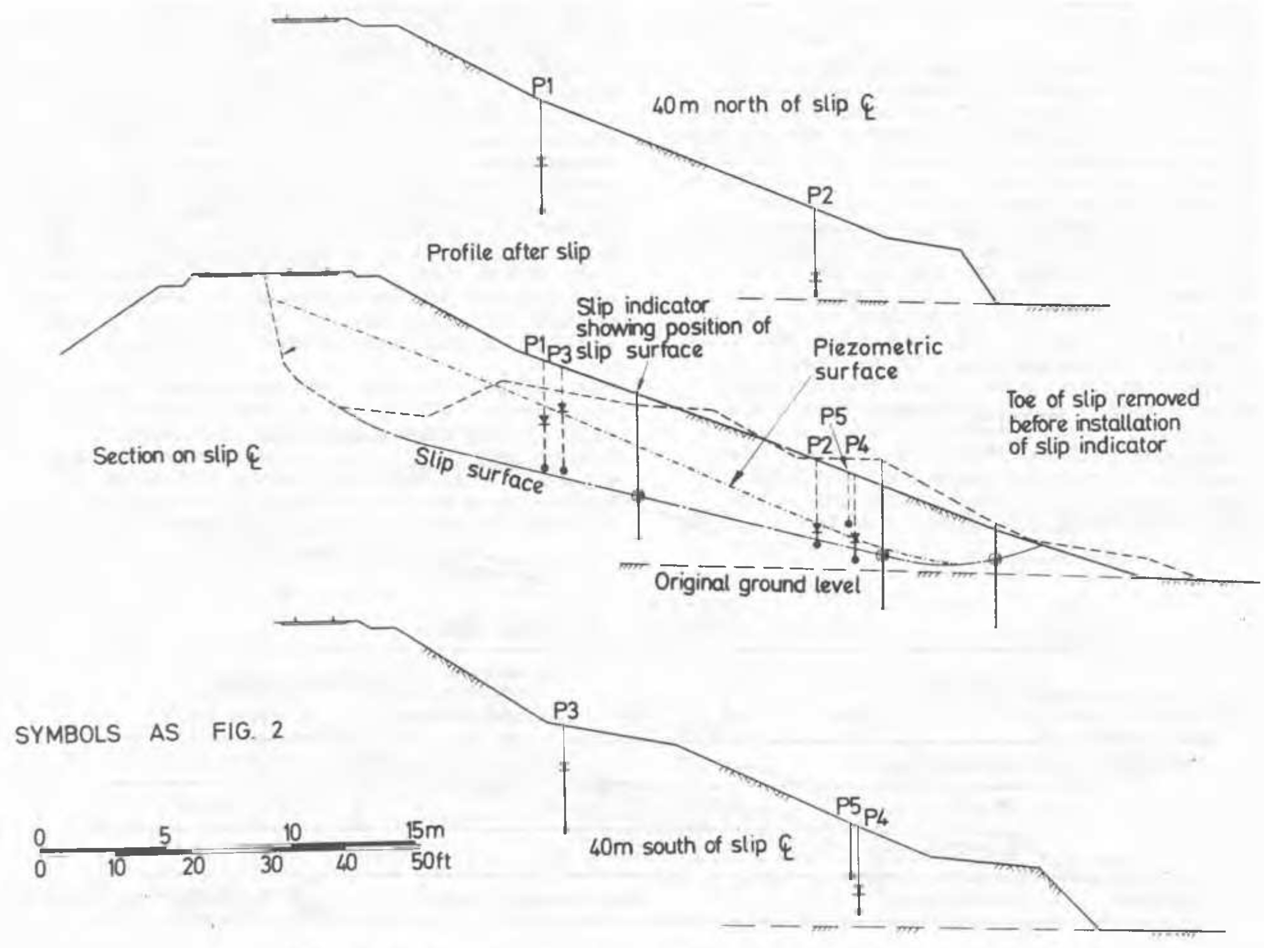

FIG.3 SLIP IN RAILWAY EMBANKMENT, SEATON, RUTLAND

is level at this point this water must be the result of infiltration of rainfall, perhaps concentrated locally where the ballast is thickest. (Piezometer P5 was dry throughout the period of observations.)

STABILITY ANALYSES

Seaton embankment. The piezometric surface used in the stability analysis of this slip was that determined from the piezometers located at the depth of the slip surface adjacent to the site of the slip. These piezometers and the corresponding maximum piezometric levels are projected onto the cross-section of the slip in Fig.3. The stability analysis, in common with all those considered in this paper, was carried out using the method of Morgenstern and Price (1965). The result of this analysis suggests that at failure the average strength along the slip surface was 14.8kN/sq.m (310 lb/sq.ft), which corresponds to effective stress parameters $c' = 0$, $\phi' = 21^\circ$. A study of a number of long term failures of Upper Lias Clay cutting slopes shows that for the undisturbed clay the lowest average effective stress parameters operative at failure are $c' = 0$, $\phi' = 23^\circ$. It is probable that the fill material had similar effective stress parameters at failure, since higher than usual pore pressures were no doubt the final trigger mechanism causing failure. There is no evidence that the 1971-2 pore pressures used in the analysis were other than usual for the winter period, and were probably lower than those operative in the slip area in January 1961. An average rise of the piezometric surface of 0.4m above the 1971-72 values would lead to failure with $c' = 0$, $\phi' = 23^\circ$.

No account has been taken of passing trains in the analysis, as the rapid movements of the final 'failure' occurred without the rear of the slide mass being subject to a train load. The fact that the slide mass encompassed one of the tracks does however suggest that passing trains may have played a part in some progressive failure mechanism. Alternatively the position of the rear of the slip may result from weaknesses 'built-in' at this point by the side tipping method of construction.

Gretton embankment. The uncertainties concerning the strength of the fill and the relevant piezometric levels in the unstable portion of the Gretton embankment mean that a definitive measure of the strength on the solifluction shear surface cannot be obtained. Realistic assumptions concerning these variables, however, lead to conclusions concerning the embankment instability that are consistent, both with the observed small deformations of the bank, and with the reactivation of a small portion of a similar Lias clay solifluction sheet near Uppingham, Rutland (Chandler, 1970).

On the basis of the evidence of the Seaton failure the embankment fill is taken to have effective stress parameters $c' = 0$, $\phi' = 23^\circ$. It seems unlikely that the embankment failed during construction, as the side-slope of the bank, apart from the irregularities presumably resulting from the recent movements, has a similar overall inclination to that of the adjacent stable section.

If, however, complete failure of the bank had occurred previously, so that a shear surface of similar strength to that beneath the solifluction sheet existed within the bank, then the values of ϕ' for the solifluction sheet discussed subsequently will be too low by about 2°.

The other major uncertainty concerns the piezometric levels related to the slip surface, particularly where this lies in the bank. The records from Seaton show that high piezometric levels can be expected within the bank; at Gretton, however, where the bank is much lower, the magnitude of the pore pressures will depend on how effective the ironstone fragments in the solifluction layer are as a drainage blanket. That water seeped into the trial pit along these layers suggests that they will be at least partially effective. Two piezometric surfaces have been chosen for analysis (Fig.2), both consistent with the records of piezometers P1 and P2 either side of the bank: surface A, taken from a flow net drawn assuming that the ironstone stratum is ineffective as a drainage layer, but that there is flow into the Lias at depth; and surface B, consistent with the slip indicator 'piezometer', which suggests that the ironstone stratum is partially effective as a drainage layer.

Stability analyses carried out making the assumptions discussed above, and considering only the self-weight of the bank, show that the strength on the solifluction portion of the slip surface for a factor of safety $F = 1$ is $\phi' = 14.0^\circ$ and 13.1° (both with $c' = 0$) for piezometric surfaces A and B respectively. The limited movements of the embankment, however, suggest that a true failure did not occur, but that there were small displacements when passing trains added a disturbing force. Accordingly analyses were carried out adding the weight of a passing train as a disturbing force, and assuming that this additional (undrained) loading caused no change in strength from that expected under the self-weight of the bank only. This assumption is equivalent to taking pore-pressure parameter $A = 0$ for the loading increment applied by the passing train. Skempton (1969) quotes results of undrained triaxial compression tests on specimens containing natural slip surfaces which showed that $A=0$ throughout the test, supporting this assumption, at least for the solifluction portion of the slip surface. Only small errors are introduced if A differs from zero. If it is taken that the undrained loading increment provided by the

train causes failure, then the corresponding change in F as the train passes is from about 1.1 to 1.0. As this final loading increment is quite small, and as it is over this increment only that undrained con- ditions apply, the strength at failure for values of A ≠ 0 will not differ much from that when A = 0. For example, the extreme value A = 1 leads only to a 10% reduction in strength.

The train load used in the analysis is based on the loaded weight of mineral trucks, as the line is used extensively for the transport of iron ore. Assuming the weight of a truck to be uniformly distributed by the track, the disturbing force due to the train load is then 100kN per metre of track (7000 lb/ft). With this additional undrained loading the value of Ø' required on the solifluction shear surface for F = 1 becomes 18.9° and 18.2° for piezometric levels A and B respectively.

In order to assess the significance of the results of the various analyses it is necessary to have an estimate of the actual strength mobilized on the solifluction shear surface. This is provided by the stability analysis of a shallow slab slide near Uppingham, Rutland, which on investigation proved to be the consequence of the reactivation of a solifluction sheet com**posed of Upper Lias Clay in a small area where a high ground water level developed (Chandler, 1970). This case record provides a value of Ø' for the solifluction shear surface under an effective normal stress of 12 kN/sq.m. (1.7 lb/sq.in) of between 18.2° and 17.2° depending on whether the ground water table is taken at ground** surface (as in the original paper) or at a depth of 0.15m, which is the lowest probable depth for the groundwater table at the time of failure. The embankment at Gretton, however, provides a higher effective normal stress to the solifluction slip surface, so that the corresponding likely value of Ø' will probably be rather lower, as demonstrated by Bishop *et al.*, (1971) for the residual strength measured in the ring shear apparatus. A possible field failure line for Lias clay solifluction shear surfaces has therefore been constructed with a relation between Ø' and effective normal stress similar to that obtained for ring shear tests but passing through the range of values obtained for the Uppingham landslide. This field failure curve, shown in Fig.4, indicates rather higher values of Ø' than obtained in the ring shear apparatus, which range between 13° and 11° at an effective normal stress of 20kN/sq.m (3 lb/sq.in), to about 9° at an effective normal stress of 65kN/sq.m (9 lb/sq.in).

Also shown in Fig.4 are the results of the stability analyses carried out for the Gretton embankment. When these are compared with the suggested field failure curve it is seen that with only the self-

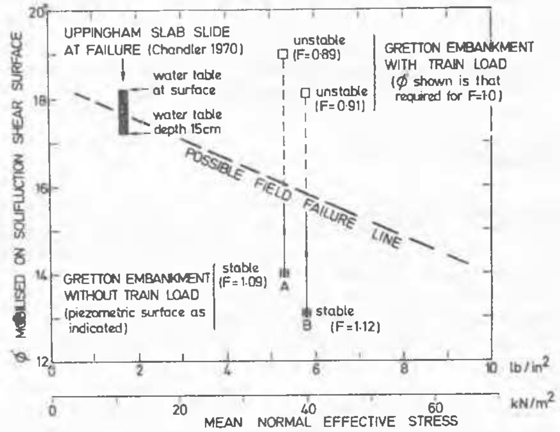

FIG.4 SUMMARY OF STABILITY ANALYSES, GRETTON EMBANKMENT

weight of the fill providing the disturbing force the bank was probably stable, the mobilized Ø' was between 13° and 14°, and **F ≈1.1; with the passing of a train the bank became unstable, with a factor of safety perhaps as low as 0.9.**

The limited movements of the embankment arise from the limited period of loading **during the passing of a train, and the associated high rate of shearing. In the latter context it may be noted that tests on pre-existing shear surfaces reported by Skempton and Hutchinson (1969) show that an increase** in the rate of shear of from 1cm/year to 100cm/day might increase the strength by be**tween 3 and 10 per cent.**

A stability analysis of the stable portion of the bank, based on the observed piezometric levels beneath the bank and assuming a perched water table at the crest of the bank, shows that if a solifluction shear surface were present the bank here too must have become unstable as trains passed.

CONCLUSIONS

The instability, 80 years after its construction, of an embankment founded on an 8½° slope in Northamptonshire, England, resulted from a combination of three factors:

1) The presence, under the embankment, of a 'solifluction' shear surface that was formed by mudslides that developed under periglacial conditions at least 10,000 years ago during or at the end of the last glaciation of Britain.

2) The development, at the top of the embankment, of a perched water table. Under the influence of this 'wet' upper boundary the strength of the fill decreased and the

pore-water pressures on the underlying slip surface increased as long-term seepage conditions gradually developed.

3) The short-term undrained loading provided by mineral trains passing along the embankment, which was stable under self-weight loading. As a result of the short-term loading the observed movements were not large, nor did a rapid failure occur.

The condition of marginal stability of the embankment may have existed for many years. The relatively high permeability of the poorly compacted fill and low height of the bank would have led to the relatively rapid establishment of long-term conditions. It may be, however, that movements became greater about 1961 as the mineral traffic on this line increased. If, as seems probable, the bank did not fail during construction, then the progressive movements within the bank due to passing trains would have led to a gradual drop in strength along this portion of the slip surface, which, had the bank not been stabilised by grouting, may well have led to a complete failure under the self-weight of the bank alone. Before a complete 'self-weight' failure could occur, however, the strength of the fill would have had to fall to a magnitude corresponding to Ø' between 19° and 16° (with c'= 0), depending on the piezometric level (A and B respectively) in the bank. With the latter value of Ø' there would probably not have been rapid movement following failure under the self-weight of the fill alone, as a further drop in strength below 16 as a result of continuing movement would not appear likely.

Low ground water levels provide the probable reason why failure did not occur during construction; the piezometers at the toes of the embankment dried out completely during the autumn of 1971. Stability during construction would have been assured if such conditions had prevailed at that time.

ACKNOWLEDGEMENTS

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