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Progressive failure in clay embankments due to seasonal climate changes

Rupture progressive de talus d'argile due aux changements climatiques saisonniers

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ABSTRACT: Observations show that old railway embankments of dumped London Clay move as superficial pore pressures change seasonally. Slips coincide with long winter periods in which the surface pore pressure is zero and swelling is encouraged. Finite element analyses show that the seasonal cyclic stress changes cause outward movement, strain softening and, eventually, collapse through progressive failure. This mechanism depends on the number and severity of the shrink/swell cycles and the end of winter pore pressures. Movements prior to collapse are small and collapse is abrupt.

RÉSUMÉ: Des observations montrent que des mouvements se produisent dans d'anciens talus de voies ferrées construits avec de l'argile de Londres en relation avec un changement saisonnier des pressions interstitielles superficielles. Les glissements coïncident avec de longues périodes hivernales au cours desquelles la pression interstitielle de surface est nulle et le gonflement est favorisé. Des analyses par éléments finis montrent que le cycle saisonnier de changement de contraintes cause des mouvements de l'intérieur vers l'extérieur du talus, une relaxation des déformations, et au bout du compte un effondrement par rupture progressive. Ce mécanisme dépend du nombre et de la sévérité des cycles rétraction/gonflement et de la fin des pressions interstitielles hivernales. Les mouvements précédant l'effondrement sont faibles et ce dernier est brutal.

1 INTRODUCTION

There are many old railway embankments in the UK built between 70 and 160 years ago. In North London they are made of dumped London Clay fill, taken from cuttings and tunnels. Many are owned by London Underground Ltd (LUL). These embankments move and track maintenance has become expensive. Slips occur occasionally.

The embankments were end and side tipped from rail track, without compaction. The slopes formed are irregular, from as steep as 1:1.5 to as flat as 1:3.5. The fill probably spread variably during and after dumping. The embankments usually slipped when tipped to a height greater than about 5m. A sliding surface would have formed in the top of a London Clay foundation, in the base of the fill, or sometimes in soft alluvial clay occurring in the floor of small valleys. The maximum embankment height is around 10m. Most embankments are around 4 - 6 m high.

When slips occurred during tipping, embankments were often finished in steam locomotive ash, giving a steep upper slope, typically about 1: 1.5. Tipping of ash was continued until slipping and consolidation produced stability. Up to 1945 ash was used as ballast. Nearly all the embankments have a mantle of ash fill over the crest and the upper slopes, between 1m and 3m thick. Ash (from power stations) is still used to make up embankment slopes. Figure 1 shows a typical embankment cross section, 8m high.

Embankment slopes were colonised by rough grass and shrubs. More recently trees have been allowed to grow and many embankments are now covered by mature deciduous trees.

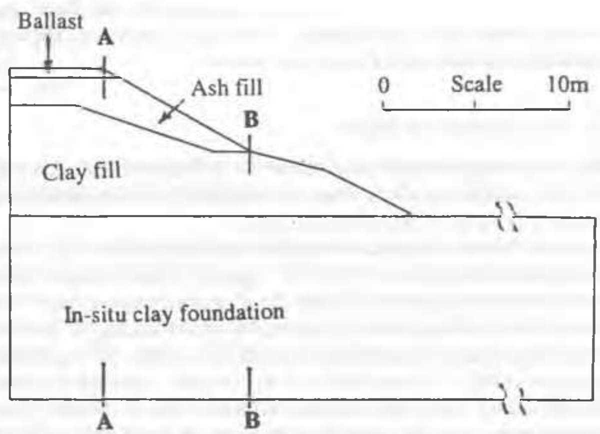

Figure 1. The embankment cross-section analyzed.

2 OBSERVATIONS OF MOVEMENT IN RESPONSE TO CLIMATE

An initial programme of measurement of embankment movement was started by the Imperial College soil mechanics group in 1992 (Standing et al. 2001). This showed that movement of the clay fill occurred in response to climate. The embankment slopes moved outwards in the summer and the crest settled. In winter the crest heaved, although there was no measurable outward movement. Lateral movement was around 5-10mm per year; vertical movement up to ± 30mm/year.

It was found that movement could be correlated with Soil Moisture Deficit (SMD). This is calculated from meteorological data by the Meteorological Office, primarily for use in agriculture. The SMD represents the amount of water which the ground can absorb before refusal is reached. It is calculated for different vegetation cover and for different soil types. Figure 2 shows a typical settlement/heave observation of LUL track on an embankment compared with SMD for grass. SMD for trees increases to a higher maximum than SMD for grass and often does not reach zero in winter. SMD for grass nearly always does. The few slips, which have occurred over the last decade, coincide with winters with long wet periods, as indicated by SMD.

3 EMBANKMENT STABILITY AND ANALYSIS

3.1 *Purpose of the analyses*

The clay fill of the LUL embankments is strain softening and brittle in shear. Thus such embankments might collapse by progressive failure, at an average operational strength less than peak strength. Previous numerical studies at Imperial College had

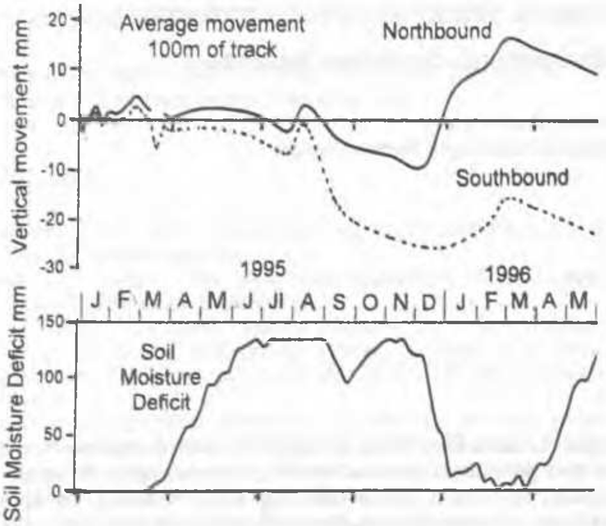

Figure 2. Settlement/heave of rails on embankment at Cannon Park compared with Soil Moisture Deficit (grass) for London Clay.

shown that progressive failure could be significant (Potts, Dounias & Vaughan, 1990; Potts, Kovacevic & Vaughan, 1997). It was decided to analyse a typical section using the same procedures.

Embankment construction was simulated in 10 horizontal layers at zero pore pressure, followed by desiccation simulating the growth of vegetation. K_0 was equivalent to normal consolidation. Swelling was then simulated to pore pressure equilibrium or collapse, whichever occurred first. Cycles of shrinkage and swelling were also investigated. The cyclic analyses showed continuing and non-recoverable movement.

3.2 *The assumptions made*

There were rigid boundaries below the 10 deep foundation and 40m beyond the toe of the slope. A London Clay foundation was assumed, typical of most embankments.

A non-linear elasto-plastic model was used for the clay, with the properties shown on Table 1. Figure 3 shows stress strain curves predicted from the model for a drained triaxial test. The same stiffness was adopted in unloading and reloading of the fill. These properties are similar to those used by Potts, Kovacevic & Vaughan (1997). Consolidation was coupled. Permeability was assumed to be a function of mean effective stress. Swelling and consolidation were simulated by changing the hydraulic surface boundary condition. Each stage was allowed to reach equilibrium. This caused pore pressure changes to penetrate more deeply than occurs in the field. Changes over a shorter period could have been simulated but the field pore pressure changes are controlled by root growth and cracking during shrinkage and by rapid inflow of water into cracks when swelling starts, more than by conventional swelling and consolidation. Simulation of field conditions would have required pore pressures to be specified to match those measured and there was little information from which to do this. Since the analysis was diagnostic, not predictive, the simpler option of modelling shrinkage and swelling to equilibrium was adopted.

4 THE RESULTS OBTAINED

4.1 *First analyses - construction, shrinkage and first swelling*

Run 1 with swelling to a surface suction of 10kPa did not predict collapse during the first swelling. However, collapse was quite near. A small tension zone was predicted at the top of the clay fill after shrinkage, but this was not thought likely to have much effect on the stresses after swelling.

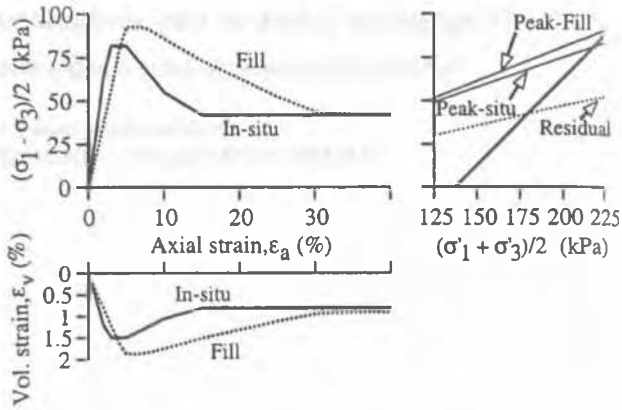

Figure 3. Drained triaxial test results predicted by the model.

Table 1. Properties assumed for materials modelled.

Property	Clay foundation	Clay fill
Unit weight	18.8 kN/m^3	18.1kN/m^3
Peak Strength	c'_P = 7kPa ϕ'_P = 20°	c'_P = 3.4kPa ϕ'_P = 22.9°
Residual strength	c'_R = 2kPa ϕ'_R = 13°	c'_R = 2kPa, ϕ'_R = 13°
Plastic strain at peak	0 - 5 %	0 - 5 %
Plastic strain at residual	20 %	50 %
Poisson's Ratio	0.2	0.3 (0.2 unloading)
Young's Modulus	2500(p'-100)/100 min 4000 *^	1500(p'-100)/100 min 2000 *^
Angle of dilation	0	0
Permeability, k	$2.10^{-10}.e^{-0.035p'}$ *	$10^{-9}.e^{-0.030p'}$ *

* p' is the mean effective stress;
^ During unload/reload: E = 5000.(p'-100)/100

Run 2 was the same as Run 1 except that zero pore pressure was assumed over the surface of the embankment during swelling. Collapse occurred well before swelling reached equilibrium. Pore pressures at collapse (Figure 4) are similar to those observed in the field below grass (Vaughan, 1994). Those at the end of Run 1 are shown for comparison. At depth they are slightly higher than in Run 2. They are lower in the fill. On the rupture surface the average pore pressure ratio was $r_u = u/(\gamma.h)$ = 0.260 at equilibrium It was 0.306 at collapse in Run 2, compared with 0.350 at equilibrium.. The collapse showed little progressive failure, the operational strength was 4% of the way from peak to residual. The failure was quite abrupt, and the slip passed through the fill only.

Three other runs showed that increased suction at the end of the shrinkage stage, reduced stiffness of the fill (x 0.5) and increased stiffness of the foundation (x 4) had little effect.

4.2 *Supplementary analyses – the effect shrink /swell cycles*

These analyses were continued from the end of construction, initially with 10kPa surface suction as in Run 1. Collapse occurred during swelling as a fifteenth cycle was attempted. The pore pressures after swelling were virtually the same as those during Run 1. Figure 5 shows the vectors of displacement for the shrinkage and swelling stages of cycle 14, the last before collapse. Vertical movement of the embankment crest relative to the foundation is 40mm. Figure 6 shows the net vectors of displacement for the whole cycle. The movements now define the slip,

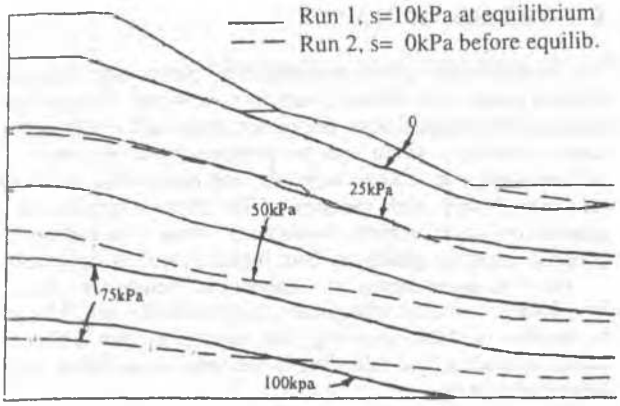

Figure 4. Pore pressure contours at the end of Run 1 (no collapse at equilibrium, u = 10kPa) and Run 2 (collapse before equilibrium, u = 0).

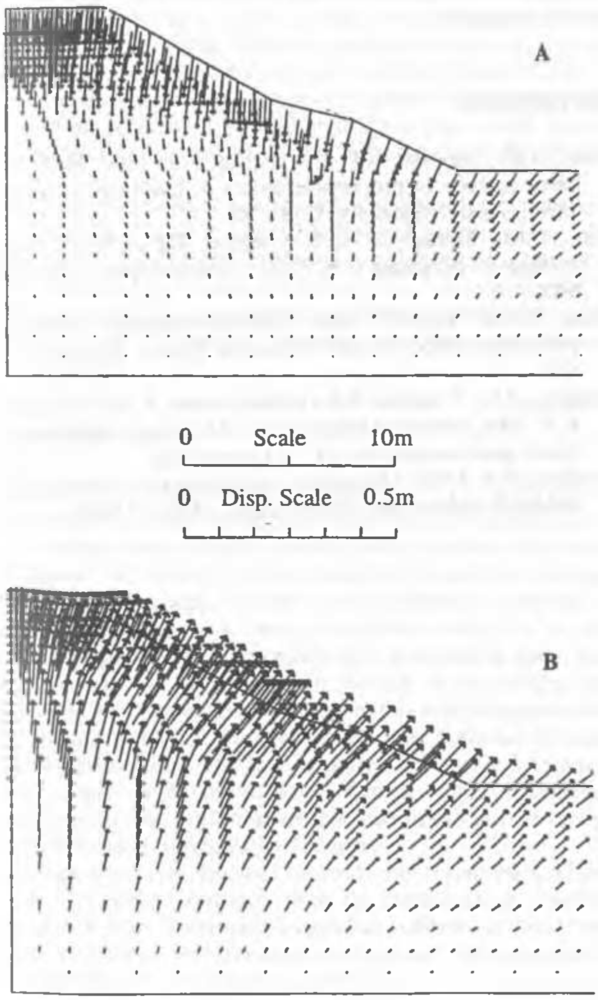

Figure 5. Displacement vectors for the 14th cycle: A - shrinkage, B swelling.

which is predicted during the next cycle. Lateral movements are about 2 mm at the crest and 5 mm at mid-slope.

Figure 7 shows the horizontal displacement with increasing shrink/swell cycles. It increases slightly as collapse is approached, but it is small (average 2 mm/year compared with 2-5 mm/year measured in the field). There is no useful warning of collapse, particularly as in the field the stress changes and movements would vary from cycle to cycle. The amount of progressive failure is significant. The average operational strength at collapse has dropped 20% of the way from peak to residual. Figure 8 shows the horizontal displacements on the two vertical profiles indicated on Figure 1. Profile B nearer the toe develops a discontinuous rupture zone of the minimum thickness allowed by the mesh (half the thickness of an element). The reduction of strength with cycling allows collapse to occur at lower pore pressures.

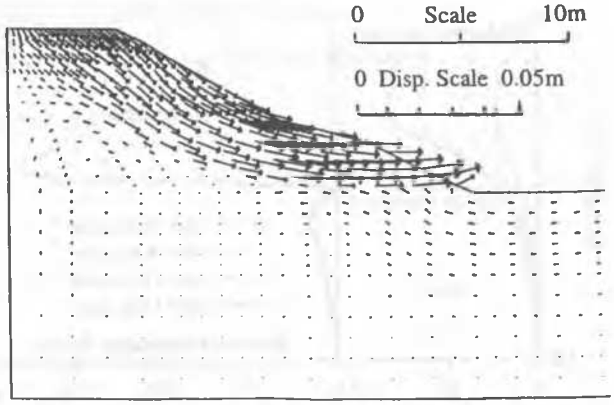

Figure 6. Displacement vectors for the whole of the 14th shrink/swell cycle.

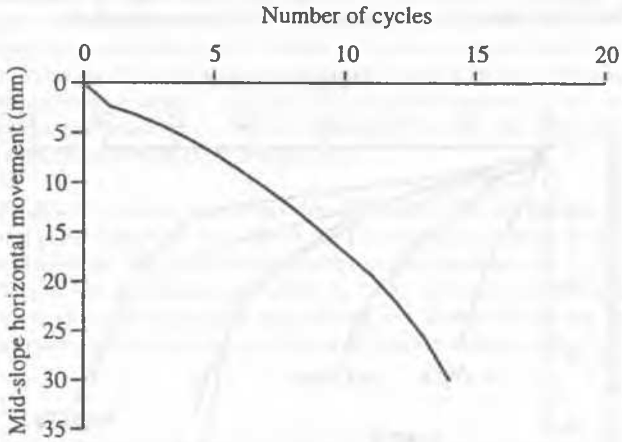

Figure 7. Horizontal displacement of the slope at mid-height with shrink/swell cycles - 10kPa boundary suction during swelling.

To examine the relationship between pore pressure after swelling and the number of cycles required to cause collapse three other similar analyses were run to collapse, with surface boundary pore pressures of -5kPa, -15kPa and -20kPa. Two analyses were made at each pressure, with stiff and soft (x 0.5) fill properties. Figure 9 shows the displacement of the slope with the various surface suctions modelled, as the number of cycles increases. Collapse always occurred along the same rupture surface. Stiffness of the fill has little influence. Displacement is due primarily to plastic yield rather than to elastic deformation. The number of cycles to cause collapse increases approximately linearly as the pore pressure is reduced.

The behaviour is controlled by progressive failure. Figure 10 shows how the average stresses on the rupture surface just before collapse change with decreasing pore pressure. The normal effective stress and the peak and residual strengths increase as the pore pressure decreases. The average shear stress (fixed by equilibrium) remains constant. The amount of progressive failure is indicated by the ratio (difference from peak strength to average stress)/(difference from peak to residual strength). This increases from a little less than 10% to 50% as the pore pressure drops and the number of cycles to collapse increases. Despite this the displacement at collapse is small (around 35mm) and similar in all four analyses (Figure 9). An embankment must be stable if the shear strength required is less than residual. However, the analy-

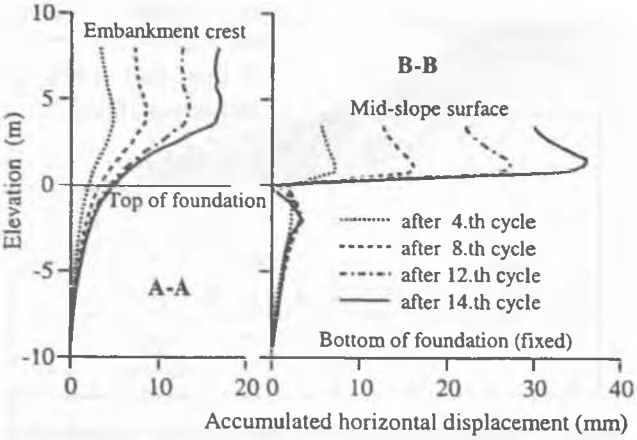

Figure 8. Horizontal displacements during cycling on the vertical planes below the crest (A-A) and below the toe of the upper slope (B-B).

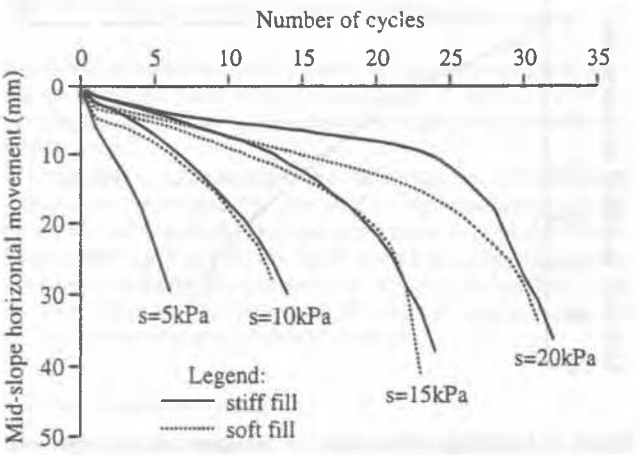

Figure 9. Variation of horizontal displacement at mid-slope with cycles – the influence of pore pressure at the end of swell and of fill stiffness.

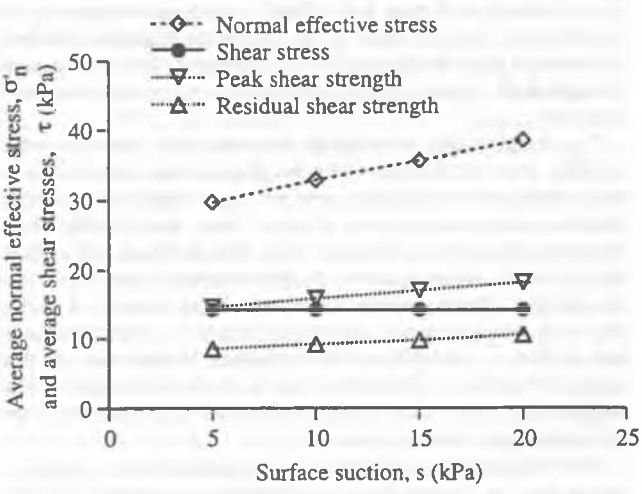

Figure 10. Normal and shear stresses and strengths on the rupture surface - progressive failure v. boundary pore pressure and number of cycles to failure.

ses indicate that collapse may occur after many cycles if the operational strength required is close to residual. If it is less than residual then cyclic movement can continue without collapse.

5 CONCLUSIONS

The analyses show that embankments of plastic clay may suffer delayed progressive failure if subject to seasonal effective stress changes. Movements occur during shrinkage and swelling which cause cumulative strain and the progressive development of a rupture surface at reduced strength. The movements are small, they would vary with the size of the stress changes and are masked by larger vertical movements. Thus it is unlikely that collapse could be predicted from measurement of deformation.

There is no evidence of 'shakedown' behaviour (Shiau & Wu, 2000), in which cycling leads to elastic behaviour. This may be because of strain-softening. The assumption that stiffness in elastic unloading and reloading is the same could allow shakedown but does not.

The actual field behaviour of these embankments is more complex than is modelled here and, as yet, both material properties and stress changes are uncertain. Thus the analyses should be regarded as diagnostic so far as mechanisms are concerned and not predictive.

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