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The behaviour of a reinforced trial embankment on soft shallow foundations

R.H.Bassett & K.C.Yeo
King's College, University of London, UK

ABSTRACT: The paper describes the behaviour of a reinforced earth embankment constructed across soft alluvial deposits. The Project involved an instrumented trial embankment - which included monitoring the load in the reinforcement. The key data is presented, the findings derived from the data and the insight gained are discussed.

1 INTRODUCTION

Stanstead Abbotts lies 20 miles to the north of London. It is bisected by the River Lea running south and the A414 running east-west. The A414 is a primary route within Hertfordshire providing an important link between the nearby towns and an alternative circumferential route round north London. The development of these towns has resulted in a heavy rise of road traffic. The A414 has a poor alignment and a number of secondary and local roads join the route at junctions with poor visibility, further hazards included a gated rail crossing, two pedestrian crossings and a 1:10 hill. The number of accidents caused the Council concern and local opinion was sought to choose an acceptable route for a Bypass. The route favoured contained a number of major civil engineering problems; the one which concerns this paper was a 1km length of embankment built across a 4.5m thickness of soft clay/peat/soft clay terminating at the river viaduct. The initial proposal had been for a viaduct crossing the complete valley but this was rejected on grounds of high cost, for similar reasons replacement of the soft materials beneath the embankment was also rejected. An acceptable economic solution required the embankment to be constructed on top of the marshland, thus stability and settlement together with timing became the critical issues. Calculations for settlement showed (i) that the magnitude of the primary settlement would be between 1.2m-1.5m and (ii) that a significant secondary settlement would need to be eliminated to avoid maintenance problems. Natural dissipation of pore water pressure would be too slow due to the sealing effect of the upper and lower clay layers and lateral migration in the peat would give rise to instability at the embankment toe. It was therefore decided that (i) wickdrains should penetrate the two clay layers to increase the vertical permeability and (ii) the fill material plus a surcharge load should be placed as quickly as possible to maximise the pore pressure gradient. These two features would enhance pore water pressure dissipation and induce rapid settlement.

The penalty was the short term instability of the side slopes and a layer of Tensar SR2 reinforcement was therefore included to safeguard against premature failure. As efficient action of these features was critical to both the economic success and the timing of the project, it was decided to construct a fully instrumented trial embankment to provide forewarning of any problems. Fifteen load cells and two pairs of Bison strain measuring devices were installed in the SR2 grid, with funding provided by Netlon and the SERC. The load cells were installed in sets of 3 (Fig.1a) at five locations (Fig.1b). The strain devices enabled direct correlation with the load measurements. The soils data and the instrumentation locations were used as the basis for an international symposium to which eleven predictions and the actual results were presented in September 1986 at King's College London.

The highlights of the measured data are summarised in the following sections.

2 SUMMARY OF THE MEASURED DATA

2.1 Pore Pressure

The main thrust of the field instrumentation was to measure the pore water pressure, its distribution and its decay together with the resulting settlement. Key instrumentation included inclinometers, hydraulic and pneumatic piezometers and a horizontal profile gauge. The data obtained from the piezometers (Fig.2a) showed that the build-up and the dissipation of the pore pressure during the 14 day loading period was very sensitive to the actual construction process (Fig.2b) marked falls being noted during weekends and overnight, this was attributed to the wickdrain system.

2.2 Settlement

At the end of construction the settlement was 600mm, thereafter the shape of the settlement pattern remained consistent, the maximum increasing to 1.2m over 18 months (Fig.3).

2.3 Inclinometers

The inclinometers were located at mid slope (see Fig.1b) and near to the crest of the embankment both showed similar deformation patterns, developing a maximum lateral displacement within the peat layer (Fig.4a). Detailed study of the slope changes indicated that the maximum shear strain occurred at the interface between the peat and the lower clay (Fig.4b). The shear gave an angle change of 11 degrees at this interface and 5 degrees at the embankment/upper clay interface.

2.4 Horizontal Profile Gauge

The horizontal profile gauge showed (Fig.5) that the displacement at mid-slope for the 1 on 1.5 side and 1 on 3 side were 290mm and 150mm respectively. Replotting this information as a direct strain showed zero strain positions towards the two toe areas. A compression zone occurred in the area between chainage 25 & 35m but no instrumentation was placed in the corresponding location beyond the steeper slope. A maximum tensile strain of 3% developed below the crest. The pattern of strain distribution remained consistent thereafter (Fig.6).

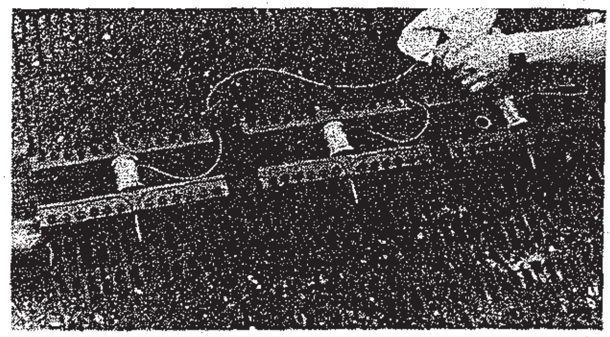

Fig.1(a) Load Cell Installation

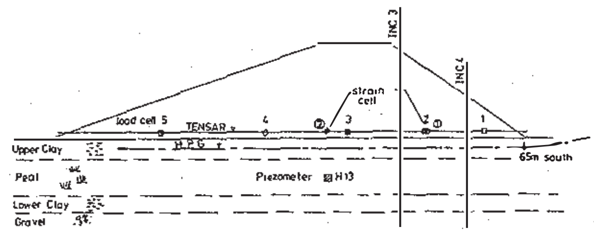

Fig.1(b) Section of Trial Embankment

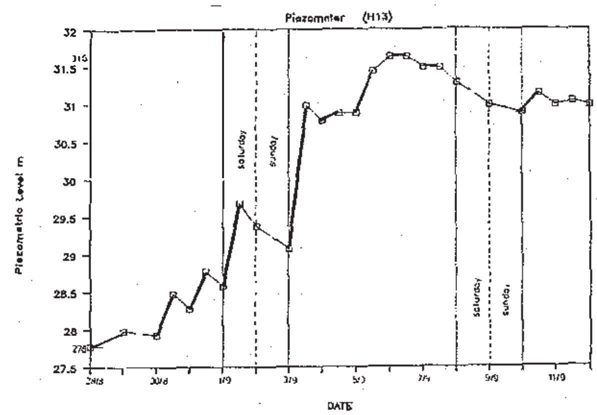

Fig.2(a) Typical Piezometer Response

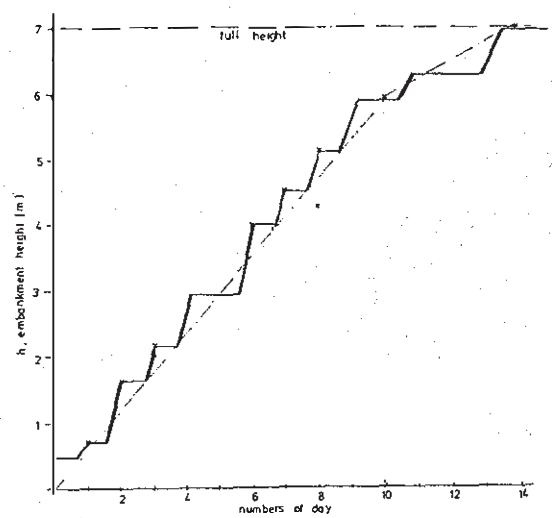

Fig.2(b) Details of the Construction Process

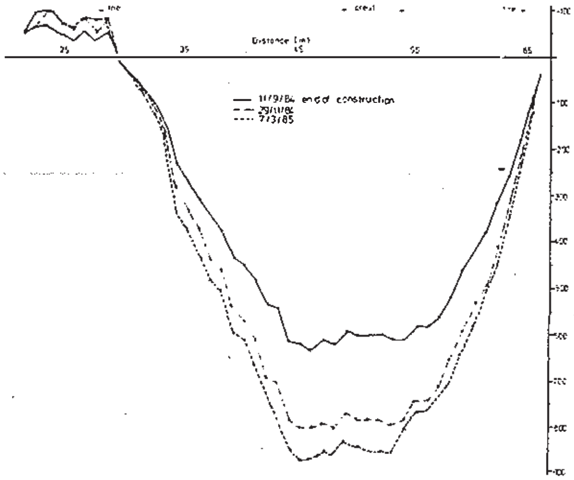

Fig.3 Settlement Profile

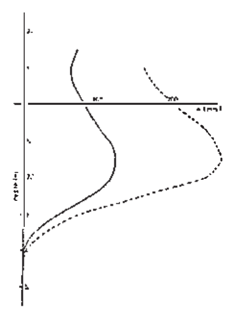

Fig.4(a)

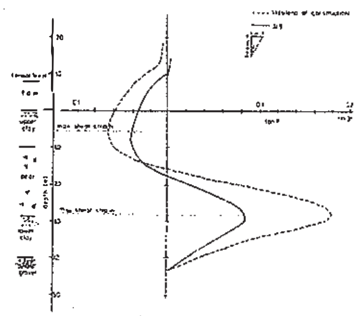

Fig.4(b)

Inclinometer Data

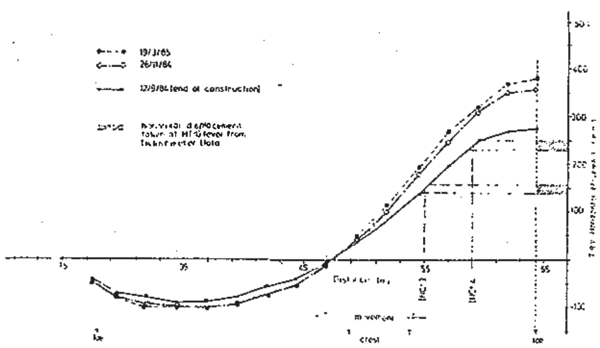

Fig.5 Horizontal Displacements

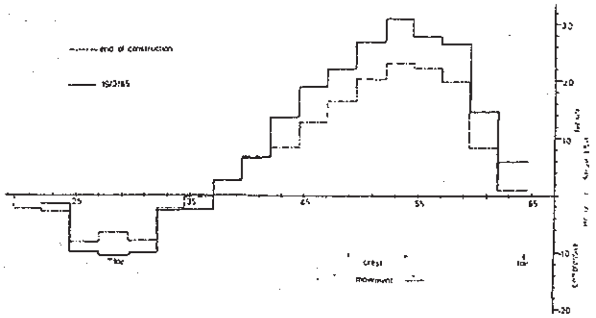

Fig.6 Horizontal Strains

2.5 Load and Strain in the Reinforcement

Data obtained from the reinforcement load cells is shown in Fig.7. The loads developed rapidly during construction but surprisingly continued to increase with time under the subsequent steady state condition. This disagreed with the currently held belief that the maximum should occur after construction and that thereafter stress relaxation would result in slowly falling load. Strain measurement Fig.8, increased from 2% to 3%, i.e. (50%) while the load in the Tensar at the same position increased by 10%.

3 COMPARISON OF LOAD AND STRAIN IN THE REINFORCEMENT AND IN THE SOIL

Correlation between the load and the strain data was assessed by the application of isochronous curves based on Yeo (1985) and Bassett & Yeo, (1988). Figure 9 shows the correlation between the directly measured strains and the strains interpreted from the load at the same point. The correspondence was good. A second comparison is the strain in the foundation soil and that developed by the

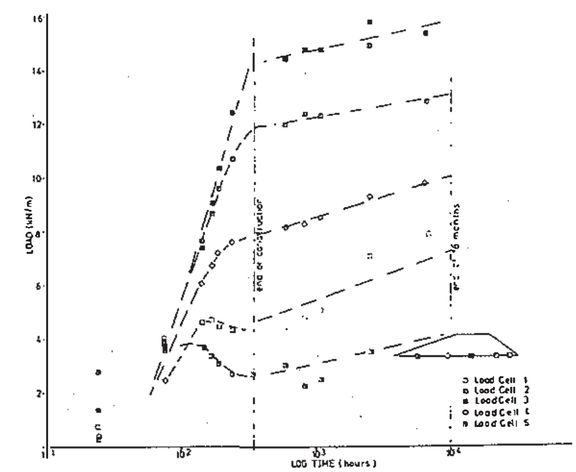

Fig.7 Loads Measured in the Tensa Reinforcement

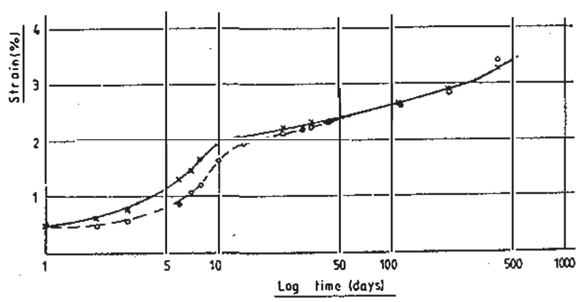

Fig.8 Strains Measured in the Tensar Reinforcement

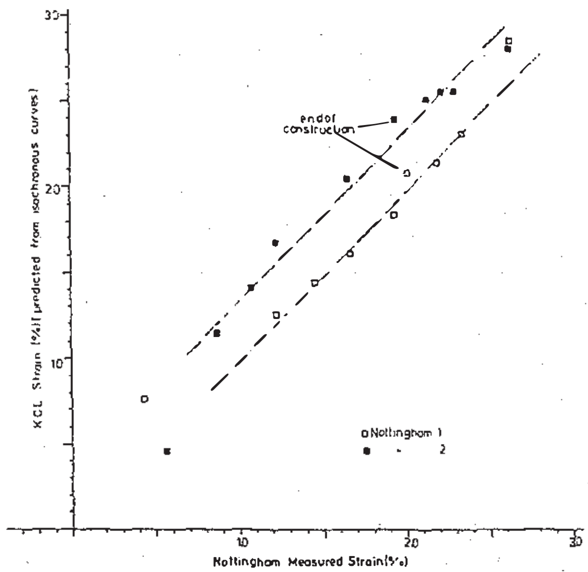

Fig.9 Comparison of Strains Derived from Loads and Directly Measured

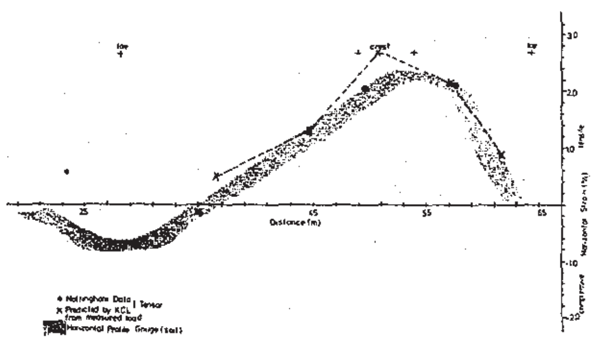

Fig.10 Comparison of Foundation Strain and Reinforcement Strain

Tensar layer. The Tensar strains and those from the horizontal profile gauge are compared in Fig.10. The close correlation implies little or no slip between the Tensar reinforced base layer and the underlying foundation soil.

4 ANALYSES

The analytical approaches adopted by predictors at the symposium divided into two categories (i) conventional, classical solutions and (ii) numerical models.

The conventional predictions divided into (a) those based on stress field solutions and (b) those based on variations of the slip circle analysis. For the particular geometry the stress field solutions generated acceptable answers but for the slip circle methods the shallow depth resulted in large radius circles which gave poor estimates of the forces and poor estimates of the performance. It was concluded that the slip circle analysis was not satisfactory for the case of a wide load on a shallow foundation.

The various numerical predictions were based on finite element methods, some incorporating simple elastic formulations while others used highly developed elastic/plastic criteria. The predicted values showed (i) that the accuracy of the prediction depended on the interpretation of suitable parameters based on the soil data rather than on the sophistication of the computing model and (ii) that the least satisfactory aspect of the numerical methods was the representation of the drainage behaviour. It was shown that the fill properties have a marked effect on the behaviour. The conclusion reached was that conventional site investigation does not yield suitable parameters for incorporation in the rapidly developing field of numerical analysis.

5 POST CONSTRUCTION DATA

The trial section (Fig.1b) was of restricted width and was designed with one slope at 1 on 1.5 to test the reinforcement principles. In practice the rapid drainage resulted in foundation strengthening even during the construction period. This was an excellent result but as far as research was concerned it limited the significance considerably as the direct necessity for the reinforcement was removed. However, in view of the large deformations it is believed to have been beneficial in that the Tensar acted as a strain distributor and prevented the development of a single shear rupture as observed by Davies (1981) in centrifuge tests of unreinforced embankments on soft ground.

6 BEHAVIOUR OF THE MAIN EMBANKMENT

After 12 months consolidation the main embankment was completed to the north side of the trial, Fig.11. The newly placed material represents an embankment shaped load in its own right as shown by the shaded area. 800mm of vertical settlements occurred between construction in April '85 and May '87 when the surcharge was removed. Removal of surcharge was followed by 50mm of swell back. Fig.12 shows the horizontal tube gauge data for the same period. It can be seen that the whole trial embankment

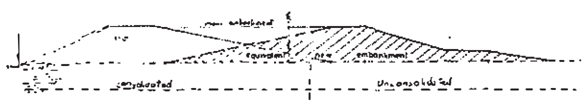

Fig.11 Dimensions of Additional Main Embankment

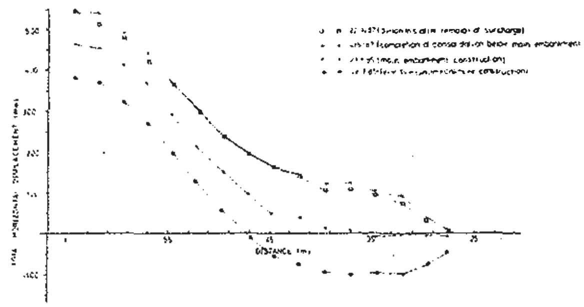

Fig.12 Additional Horizontal Movement

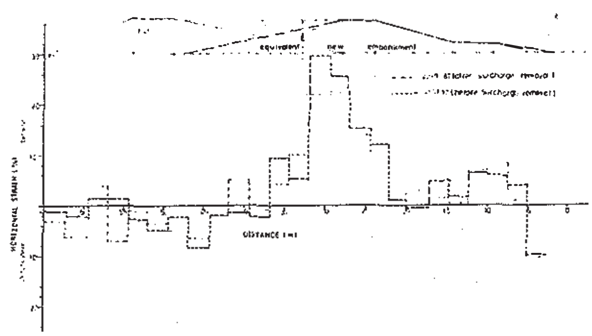

Fig.13 Horizontal Strain in the Foundations

from the 65m reference has been bodily displaced southward by some 150 to 170mm. Fig.13 shows the horizontal strain developed. As with the trial section the maximum tensile strain approached 3%, with small compression zones outside the toe areas. It was surprising that the strain fell to near zero between 12m to 22m, yet outside this zone 1% was developed. There is no obvious reason for this behaviour other than the fact that the 10m strip had earlier been used as a haul road. Fig.14 showed the loads measured in the Tensar SR2 reinforcement (see Fig.1b) over this same period. Cell groups 2, 3 and 4 showed falls in load when the main embankment was constructed, consistent with the compression observed in Fig.13. However, during the subsequent consolidation period they return to a steady rise with log time. Load cell group 5 (near the crest of the "new material") showed a marked rise in measured load. Removal of the surcharge seems to have produced a small fall in load throughout the five sets.

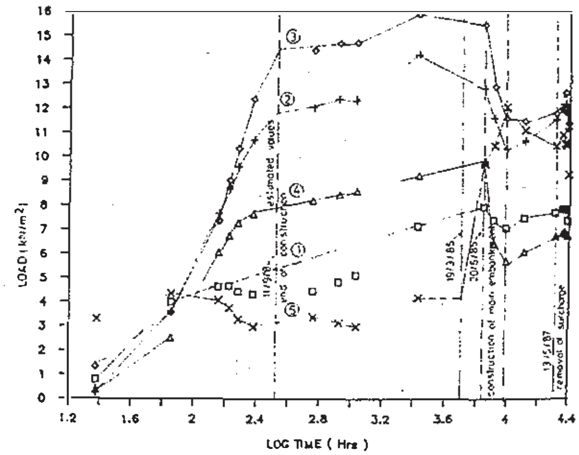

Fig.14 Loads in the Reinforcement During the Main Embankment Work

7 BEHAVIOUR OF THE STANDARD EMBANKMENT SECTION

Other sections of the main embankment, where the whole 60m wide section was constructed on a virgin foundation, were also instrumented for settlement and lateral spread but not for reinforcement loads. In Figs.15 and 16 the overall settlement profile at 26.1.87 and the horizontal strain distribution have been shown for chainage 1890. The settlement profile is nearly uniform at 1.1 to 1.2m between the crest points 24m and 44m respectively, the settlement below the slopes and the berms being pro-rata. The horizontal strain developed is uniform at 2.2% from the berm edge at 49m to the equivalent at 14m. A sharp drop occurred at the inner edge of the berm with a small rise to 1% below the berm shoulders. Assuming correspondence between strain in the horizontal profile gauge and in the Tensar SR2, this would indicate a near uniform load in the reinforcement between 49m and 14m of 14kN/m².

8 DISCUSSION

The most unexpected feature was the increase in the Tensar load during the consolidation periods from 14kN/m to 16kN/m while the strain increased from 2% to 3%. It is thought to be associated with the dissipation of suction within the cohesive fill, although at 3% strain recompacted London clay should be tension cracked and hence imposed no lateral forces on the system. This level of strain and loading was low compared with the recommended design values of 25kN/m to 30kN/m and 5% strain. The long term

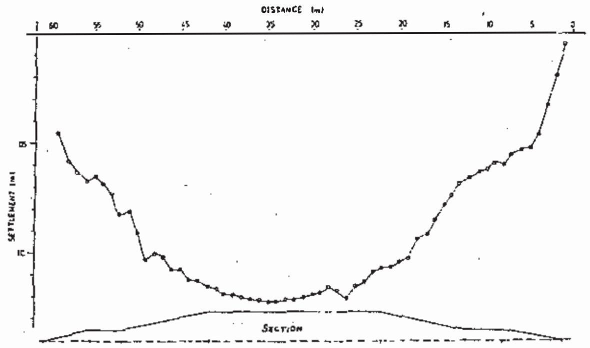

Fig.15 Settlement Under the Main Embankment

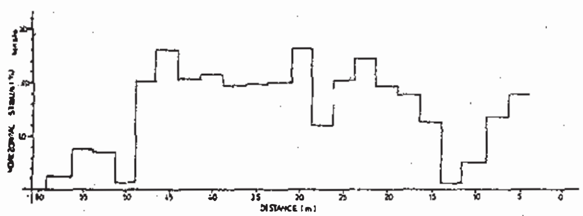

Fig.16 Horizontal Strain Measured Under the Main Embankment

creep strains and the loads can be correlated using laboratory isochronous curves. Tensar SR2 was strong enough and stiff enough to produce an ideally rough footing situation and the geometry allowed the fill material to interlock within the mesh and thus provided a non-slip relationship with the granular material. The behaviour of reinforced earth was a "self stressing" exercise and the forces that developed were dominated by two features (a) the properties of the fill, in particular cohesion and (b) the rate of dissipation of pore pressures in the foundation materials. In the prototype analysis of this project and in all the prediction papers an engineering judgement was applied to the effectiveness of the wickdrains the influence was invariably underestimated. Similarly poor fill properties were assumed. Both are conservative assumptions common to engineering practice but both have significant influence and cause considerable discrepancies between the analytical prediction and the actual performance. This typifies the differences been research for knowledge and the safe design of engineering works. In theory reinforcement was not required due to the rapid consolidation achieved by the wick drain system however the Tensar mesh contributed in the manner of distribution steel in that it maintained the integrity of the embankment against serious cracking during construction. Some base friction must have developed as there was a 5 degree inward rotation of the principle strain direction. Rapid drainage plus base reinforcement enabled the fill to be placed quickly which gave the shortest use of the expensive hired plant and overall saved money.

9 CONCLUSIONS

The final construction cost for the project using the reinforced earth solution was £1.11m which compared favourably with £9.9m for a viaduct alternative. The project was an example of how engineering design can provide an acceptable economic solution which not only provides the required facility but minimises detrimental effects to the overall environment.

10 ACKNOWLEDGEMENTS

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