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Embankments with base reinforcement on soft clay

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ABSTRACT: The behaviour of reinforced embankments on soft clay has been investigated by centrifuge model testing. A parametric study was conducted in which the type of the reinforcement and the depth of the clay foundation were varied. The effect of wick drains in the clay foundation on the behaviour of reinforced embankments was also studied. Direct measurement of tensions induced in the reinforcement was carried out using load cells constructed on the reinforcement. The displacement of the clay foundation is strongly influenced by its depth. The smaller the depth, the smaller the displacements. The reinforcement placed directly on top of the clay seems to attract less tension than that buried in the middle of a sand layer. Presence of wick drains in the clay hastens its consolidation considerably. It also appears to reduce the lateral displacement of the clay foundation.

1 INTRODUCTION

The design of embankments founded on soft clay is primarily influenced by the potential instability during or immediately after the construction of the embankment. One of the solutions to this short-term instability problem is to provide a geosynthetic reinforcement at the interface between the embankment and the soft clay foundation. Although this technique is widely used now-a-days and data is available from detailed finite element analyses and instrumented field trials, the behaviour of reinforced embankments on soft clay is far from clear. This paper describes an investigation into the behaviour of such embankments using centrifuge model testing.

2 CENTRIFUGE TESTS

Centrifuge model testing, because of its ability to reproduce the same stress levels in a small-scale model as in a full-scale prototype, is a powerful tool in exploring soil-structure interaction problems. In the present study, six 1:40 scale centrifuge model tests were performed using the Cambridge University 10m balanced beam centrifuge. Figure 1 shows the details of a typical centrifuge model. Table 1 gives a brief description of each of the six centrifuge tests. Due to the inherent symmetry about the centre line, only one half of the structure was modelled. Such an arrangement helped in constructing a reasonable sized model within a relatively small strongbox. A special clamp was built to anchor the model reinforcement to the right side of the liner. This clamp, while preventing the horizontal movement of the reinforcement, allowed for its vertical movement following the settlement of the clay foundation. A smooth nylon sheet was glued to the inside vertical surfaces of the liner in order to reduce the friction between the liner and the soil. The speswhite kaolin clay was consolidated to a maximum vertical pressure of 100kPa in a consolidometer. Two days before the day of centrifuge test, it was unloaded and removed from the consolidometer and trimmed to the dimensions of the model. The clay block and the liner were then placed in a strongbox. A matrix of small black plastic markers was installed on the front surface of the clay block. These markers were used for the measurement of clay displacements from the photographs taken in-flight through the front perspex window.

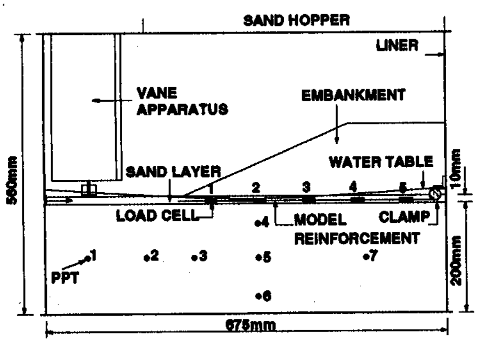

Figure 1 A typical centrifuge model

Table 1 Description of the centrifuge tests

Test code	Depth of clay	Type of reinforcement
JSS7	200mm	Geotextile
JSS8	200mm	Unreinforced
JSS9	200mm	Geogrid
JSS11	100mm	Geogrid
JSS12	100mm	Geotextile
JSS14	200mm	Geotextile*

* Wick drains installed in the clay.

Two types of model reinforcements, geotextile and geogrid, were used in the present study. The model geotextile was supplied by Akzo Industrial Corporation b.v., Netherlands and the model geogrid was supplied by Netlon Corporation, U.K.. The process of scaling down the prototype reinforcement to obtain the model reinforcement was discussed in detail by Springman et al. (1992). Figure 2 shows the load-extension curves (supplied by the manufacturers) for the two model reinforcements. Table 2 gives the properties of the prototypes represented by these model reinforcements at 40g. Figure 3 shows the position of load cells constructed on the model reinforcement for measuring tension induced in it. A typical load cell consists of approximately 12.5mm wide and 0.7-1.5mm thick strip of epoxy resin cast across the entire width of the reinforcement. The epoxy strip is reinforced with a combination of insulated copper wires and carbon fibre strips.

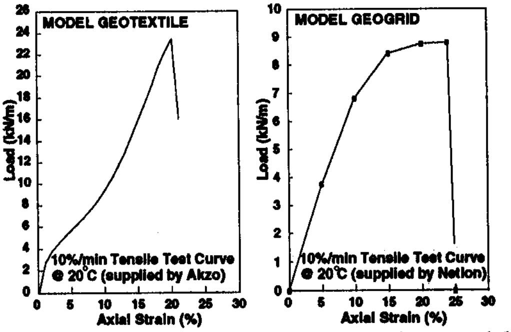

Figure 2 Load-extension curves for model reinforcements

Table 2 Tensile strengths of the reinforcements @ 10% axial strain.

Type	Model	Prototype
Geotextile	9.5 kN/m	380 kN/m
Geogrid	6.8 kN/m	272 kN/m

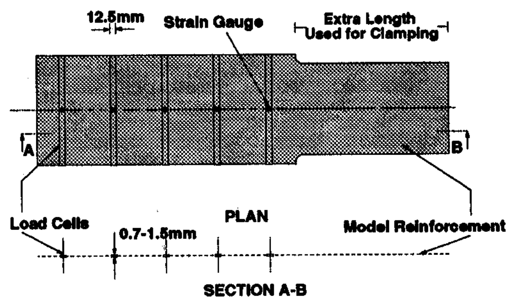

Figure 3 Position of load cells on model reinforcement.

The method of construction of load cells was different for each of the two model reinforcements. Figure 4 shows the details of the load cells. The procedure for the calibration of these load cells was described in detail by Springman et al. (1992) and hence is not reported here. Figure 5 shows the response of the load cells during the calibration. The load cells constructed on model geogrid were an order of magnitude stiffer than those constructed on model geotextile which meant that their outputs were noisier than those from the geotextile load cells.

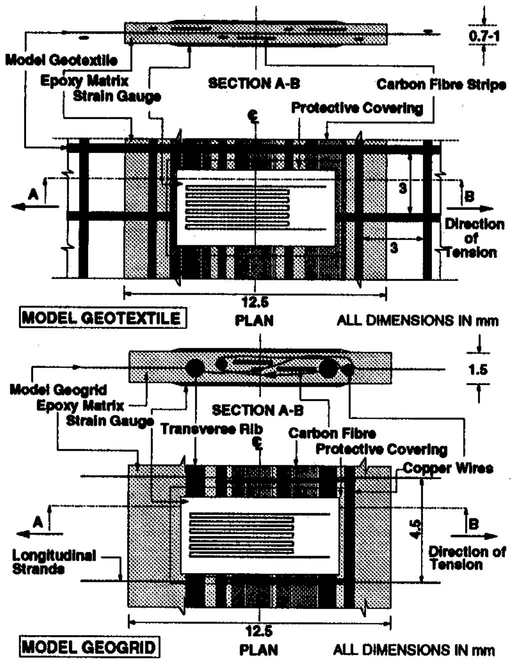

Figure 4 Details of reinforcement load cells

The calibration curves for both types of load cell are presented in Figure 6. They were fairly linear for the geotextile load cells but were non-linear for the geogrid load cells. The calibration procedure was repeated several times before and after centrifuge model testing and the calibration curves were found to be repeatable. The sheet of reinforcement with the calibrated load cells was attached to the clamp and placed directly on top of the clay for all the tests except test JSS11 for which the model reinforcement was buried at the centre of a 10mm thick sand layer.

After the clay foundation reached pore pressure equilibrium in the centrifuge, vane shear tests were carried out at different depths in order to measure the consistency of the model. Figure 7 shows the undrained shear strength profile of the clay measured by the in-flight vane shear tests. An embankment was then placed in-flight in 20 stages (15 sec time interval between successive stages) by pouring sand from a hopper mounted on the top of the strongbox.

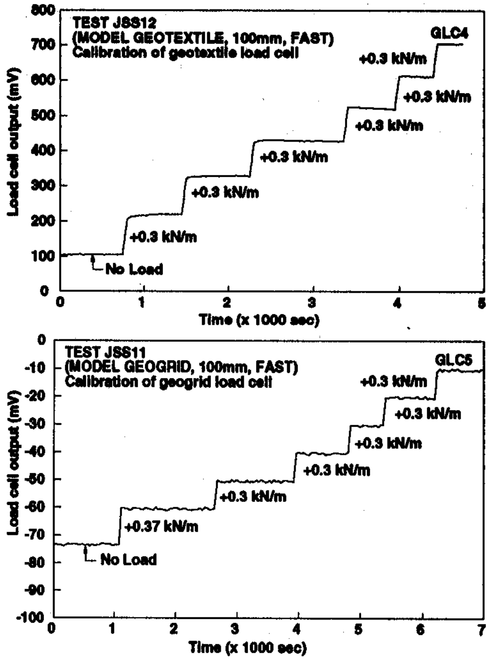

Figure 5 Response of a typical load cell during calibration.

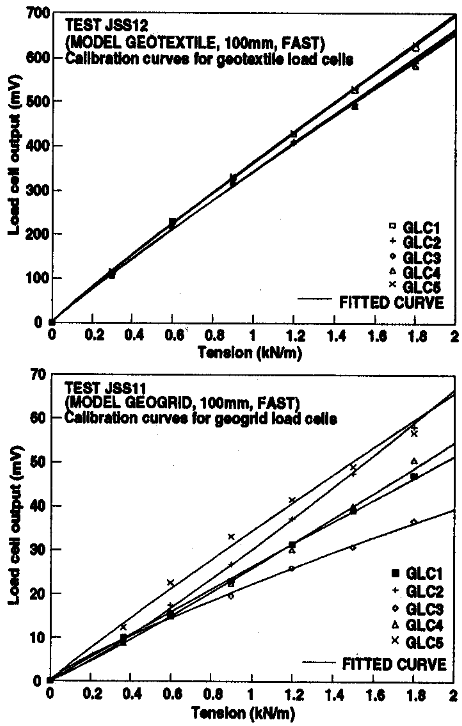

Figure 6 Calibration curves for the load cells

The rapid construction of the embankment caused significant deformation of the clay foundation. The clay foundation for test JSS8 (unreinforced) failed when about 85% of the embankment was constructed (Figure 8). The undrained shear strength obtained from back analysing the centrifuge test was 13.8 kN/m^2. In the analysis, the friction between the sides of the liner and the soil was neglected.

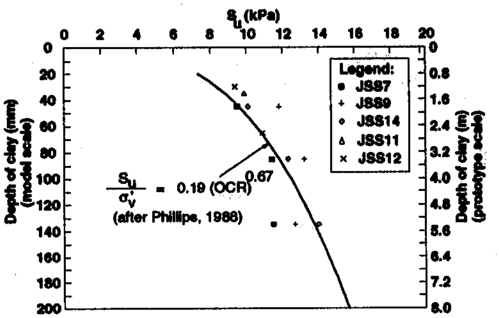

Figure 7 Undrained shear strength profile of clay

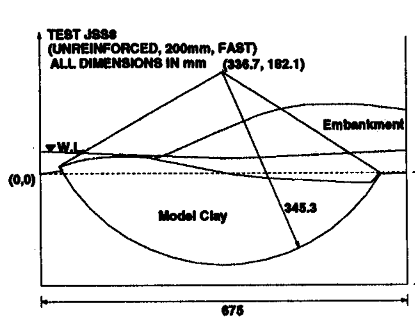

Figure 8 Failure of unreinforced clay foundation

Pore pressures in the clay and the tension in the reinforcement both increased as a result of the embankment construction. Figure 9 shows the pore pressures in the clay during embankment construction for tests JSS7 and JSS11. Faster dissipation of excess pore pressures between the successive stages occurred in the case of test JSS11 because of the shallower depth of clay. Figures 10 and 11 show the tension induced in the model reinforcement during the embankment construction for tests JSS7 and JSS11. The tension profile for both the tests reached a plateau beyond the crest of the embankment towards its centre line but for test JSS11, the tension was much less beneath the slope of the embankment as compared to that for test JSS7.

The consolidation of the clay foundation was nearly complete after 6 hours for test JSS7 and 1.5 hours for test JSS11. There was a small reduction in the reinforcement tension during consolidation for both the tests. Hence, the critical phase for the structure is the undrained loading due to rapid construction of the embankment. Figure 12 shows the displacement of the clay foundation just after the embankment construction for tests JSS7 and JSS11. The displacement increases towards the surface unless the clay foundation fails.

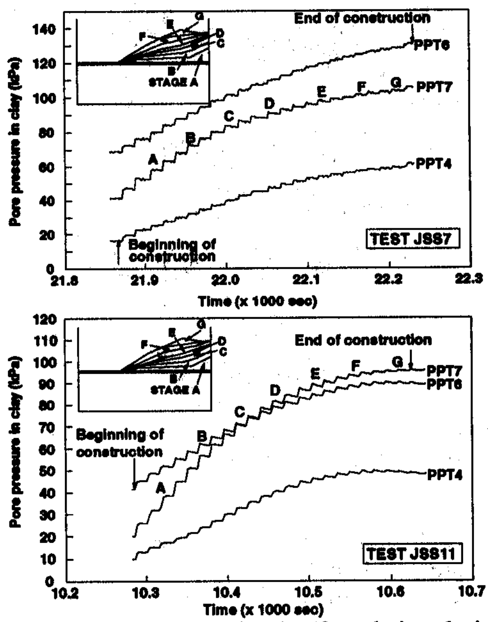

Figure 9 Pore pressure in clay foundation during embankment construction

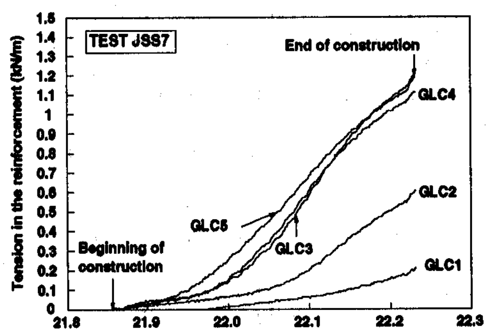

Figure 10 Tension induced in the reinforcement during embankment construction (Test JSS7)

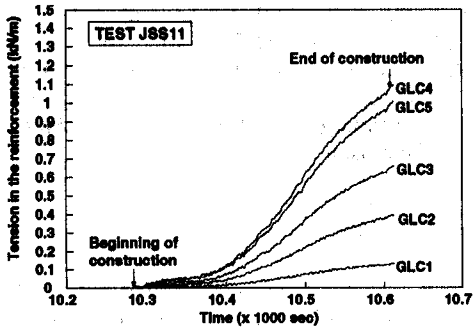

Figure 11 Tension induced in the reinforcement during embankment construction (Test JSS11)

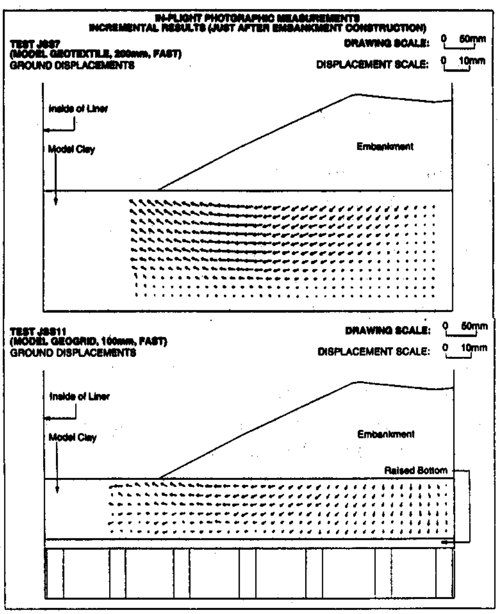

Figure 12 Displacement of clay foundation just after the embankment construction

3 WICK DRAINS IN MODEL CLAY

For natural clay deposits, the horizontal permeability is generally much greater than the vertical permeability. Also, in most situations, the horizontal extent of such deposits is much greater than their depth. Therefore, the consolidation of such deposits can be hastened by installing vertical drains: wicks, made up of geocomposites, are typically used. It has become a common practice to use base reinforcement of the embankment in conjunction with the installation of wick drains as a possible solution to the problem of short-term instability. Test JSS14 was carried out in order to investigate the influence of wick drains on the consolidation of clay and on the soil-reinforcement interaction. A twisted multifilament polyester string, approximately 1.5mm in diameter, was used as model wick drain. The wick drains were installed @ 32mm c/c in a square layout and they extended through the entire depth of the whole clay foundation (200mm). The installation was carried out after the clay block was trimmed to the required depth. It was done with the help of a stainless steel hypodermic needle (2.94mm outer diameter and 2.2mm inner diameter) which acted as a plunger. The installation process has been described in detail by Sharma (1994). The water table achieved in-flight was about 35mm higher than the desired water table (about 5mm above the top of clay foundation) due to an unavoidable technical snag. Otherwise, the test arrangement was similar to that for test JSS7.

As expected, the clay reached pore pressure equilibrium much more quickly in test JSS14 (less than two hours as compared to about six hours for test JSS7). The dissipation of excess pore water pressures during embankment construction was also faster as seen from figure 13. The tension induced in the reinforcement during embankment construction (figure 14) was significantly higher than that recorded for test JSS7. Figure 15 shows the displacement of the clay foundation just after the construction of the embankment. The displacements were much smaller than those recorded for test JSS7 (figure 12).

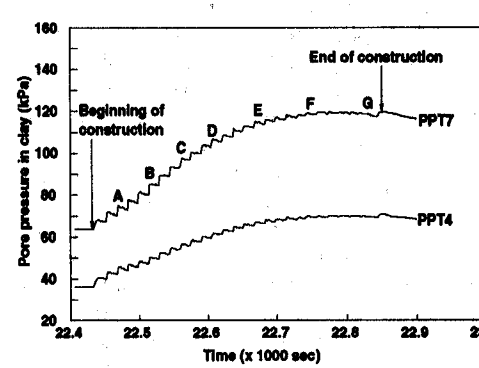

Figure 13 Pore pressure in model clay during embankment construction (test JSS14)

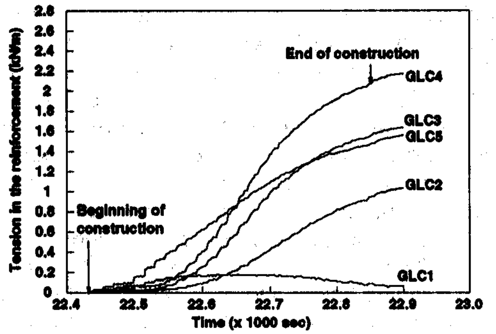

Figure 14 Tension induced in the reinforcement during embankment construction (test JSS14)

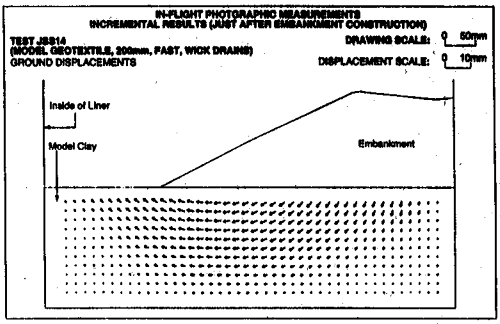

Figure 15 Displacement of clay foundation just after the embankment construction (test JSS14)

4 CONCLUSIONS

The behaviour of commonly used prototype reinforcements (geotextiles and geogrids) has been replicated reasonably well by the model reinforcements. Load cells on these model reinforcements have worked quite satisfactorily as evident from their calibration results. The undrained displacement of a clay foundation seems to be strongly influenced by its depth: the smaller the depth, the smaller the displacements. Reinforcement placed directly on top of the clay seems to attract less tension than that buried in a layer of sand. This may be due to the increase in the adherence at the bottom soil-reinforcement interface. The presence of wick drains in a clay foundation reduces its consolidation time considerably. It also seems to reduce the displacement of clay foundation. However, the presence of a higher water table may also have been influential in reducing the displacement. The tensions in the reinforcement were significantly higher for test JSS14 as compared to test JSS7. Increase in the adherence at the clay-reinforcement interface due to the presence of the ends of the wick drains, protruding out of the clay foundation, may be the reason.

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REFERENCES

Phillips, R. 1988. Centrifuge lateral pile tests in clay: Task 2 & 3. A report to Exxon Production Research Corp. by Lynxvale Ltd., Cambridge, U.K..

Sharma, J. 1994. Behaviour of reinforced embankments on soft clay. Ph.D. thesis, Cambridge University (in process).

Springman, S.M., Bolton, M.D., Sharma, J. and Balachandran, S. 1992. Modelling and instrumentation of a geotextile in the geotechnical centrifuge. *Proc. Int. Symp. on Earth Reinforcement Practice, Kyushu:* 167-172. Rotterdam: Balkema.