Pile Embankment on Soft Clay: Comparison Between Model and Field Performance

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SYNOPSIS: The paper describes the performance of a full scale pile embankment on soft clay foundation which has been built and instrumented in Malaysia. Reference is made to a parallel design study carried out using centrifuge modelling technique.

INTRODUCTION

Quaternary erosion accentuated by climatic and sea level changes has produced widespread and thick alluvial deposits in the coastal areas and major river valleys in Malaysia. Roads often have to be founded on the soft deposits and raised on high embankments, giving rise to problem of instability and settlement.

One engineering solution that is often used, notably at bridge approaches, is to support the embankment fill on a grid of timber or concrete piles. For economic reasons the piles are spaced as widely as possible and capped individually, rather than by a continous slab. In theory the piles have two effects - they stiffen the soft subsoil and reduce stresses on the upper subsoil by transferring load to lower elevations. However the complexity of the soil structure interaction problem is such that no fully developed theoretical solution exist. Design methods are based on field experiences and on the results of low stress level model test on sand rather than clay.

The cost of the piled embankment construction is very high, and in Malaysia there has been a number of failures in recent years. This led to the construction of a trial embankment by the Malaysian Highway Authority between the period of 1987 - 1988. The main objectives of the trial were to verify the conventional design method, and to obtain an economical solution.

In this paper details are given of the construction and instrumentation techniques used for the trial, and results are presented in terms of displacements, stability of the foundation, and load sharing between the pile and ground. Reference is made to a parallel study carried out using centrifuge modelling technique.

FIELD STRUCTURE

The site chosen for the trial construction was at a lay-by on the Seremban - Air Hitam section of Malaysia's north-south expressway project. Detailed descriptions of the site, geology and subsoils are given in the investigation report prepared by LLM (1987). Within the area of the trial, the thickness of the strata were known to vary a little but the subsoil profile can be generalized. The upper 17 m consists of very soft to soft marine clay with natural water content of 50 - 120 %, w_L of 40 - 80 %, w_p of 20 - 40 %, overlain by a surface crust of about 1 m thick. Underlying this clay layer is a layer of peat of about 0.5 m, followed by some 2 m of sandy clay which is underlain in turn by a thick deposit of loose to dense, medium to coarse sand with SPT values ranging from 6 - 50. A summary of the geotechnical properties of the clay is given in Figure 1. The undrained strengths obtained from the vane tests showed an almost linear increase of strength below a surface crust with an average strength of 9 kPa at depth 1 m, increasing to 36 kPa at depth 17 m. The clays are fairly sensitive with sensitivity ratio in the range of 3 - 6. Results obtained from the oedometer tests indicate that the clays are slightly over consolidated but highly compressible, with c_v ranging from 1 - 10 m^2/yr.

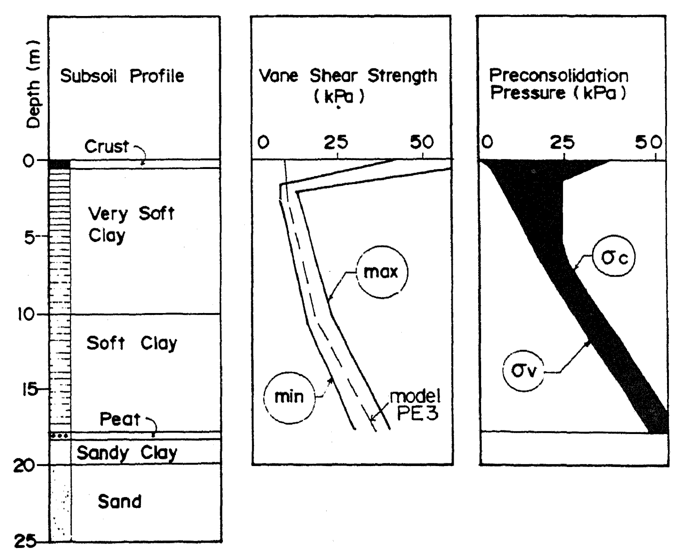

Fig. 1 : Subsoil Profile and Soil Properties for Trial Site

Figure 2 shows a cross section and instrumentation details of the piled trial embankment. Precast spun concrete piles (94 in total) of 400 mm in diameter, 70 mm wall thickness, were driven vertically to bear in the underlying sand layer at depth 21 m, and were designed to carry the full weight of the 6 m high embankment, which was assumed to be uniformly distributed based on empirical relationship between dimension of pile cap, a, height of fill above the pile head, H, and spacing of the pile, s, after the Swedish Road Board (1974). Each of these piles was provided with a separate precast concrete cap, 1.8 m x 1.8 m, and was installed in a grid spacing of 3.5 m x 3.5 m. No piles were installed under the embankment slope, rather a berm 3 m thick and 15 m wide with slope of 1 : 2 was used to maintain side stability. The fill was lateritic soil compacted to given $c' = 14$ kPa, $\phi' = 31^\circ$ and $\rho_b = 2.08$ Mg/m^3 .

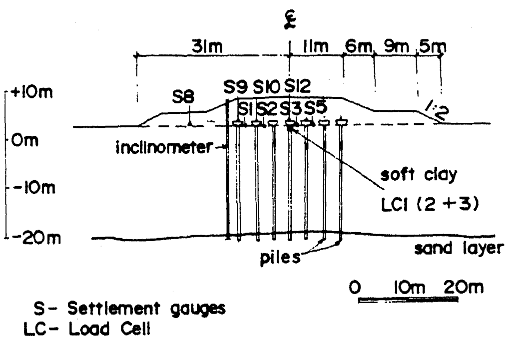

Fig. 2 : Cross Section of Piled Structure

CENTRIFUGE MODEL

A series of model test has been performed aboard the Manchester centrifuge, all nominally at 1 : 100 scale. The objectives were to simulate the essential features of the prototype structure and to provide a design study of the various parameter which influence the performance of such structure. Detailed descriptions of the test models, preparation and procedures have been described by Bujang et al. (1991).

Figure 3 shows a typical cross section of the model embankment. Troll clay with w_L = 60 %, w_p = 30 % and c_v = 1- 3 m^2 /yr that reasonably matched the field material was used to model the foundation. This was consolidated from slurry in a 1.0 x 1.0 m cell using the hydraulic gradient method (Zelikson, 1969), to a similar undrained strength profile as in the prototype (Figure 2). A very sandy clay mixed with kaolin to a sand : clay ratio of 4 : 1 by weight, and which yielded compacted values of $c' = 20$ kPa, $\phi' = 30^\circ$ and $\rho_b = 2.09$ Mg/m^3, was used to model the field fill.

Commercially available aluminium rods of 4.6 mm diameter, 210 mm in length were used to model the prototype concrete piles. They were rigidly connected to aluminium pile caps 18 x 18 x 5 mm.

Instrumentation installed included pile load cells, Druck pore pressure transducers, LVDTs and spaghetti displacement indicators. Continuous undrained strength profiles were obtained during the centrifuge runs by using a miniature cone penetrometer device.

The staged construction of the prototype was simulated by spinning the model at 65 g for 40 minutes and then at 100 g for a further 1 - 2 hours, i.e. using the gravity turne on technique. During centrifuging a steady head of water in the base sand layer was achieved by linking via external piping to the upper surface of the model foundation. Greased rubber sheets were used to minimize boundary friction. Climatic conditions around the model slope was simulated with a precipitation simulator system (Craig et al. 1991).

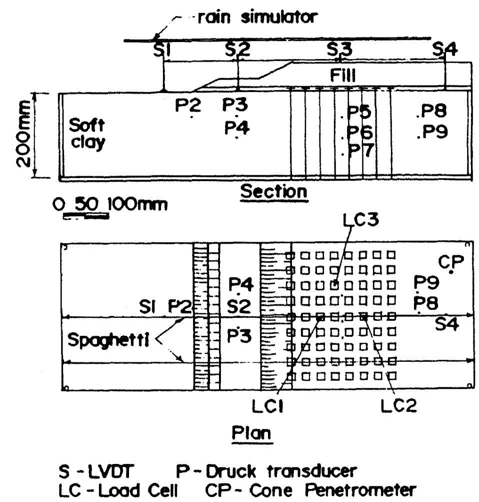

Fig. 3 : Typical Cross Section of Model, PE3

PROTOTYPE AND MODEL PERFORMANCE

The prototype embankment was instrumented as shown in Figure 2. Pile installation began in July 1987 after placement of a 0.6 m working platform of sand, and construction of the compacted lateritic embankment took place between April and October 1988 with a construction pause period between May and September as indicated in Figure 4. Sixty four days after start of filling, plate S8, located at the centre of the unsupported berm, indicated a settlement of 500 mm, 2 - 9 times greater than settlement of the original ground surface in between the pile caps. Berm thickness was the full design height 3 m and the main section fill was 4.3 m. This differential settlement resulted in the formation of a tension crack at the junction between the unsupported and supported section of the structure. Secondary surface cracks were also seen running parallel to the lines of piles along the embankment axis. During the following pause period the berm continued to settle from 500 mm to 650 mm. Addition of more filling between September and October to a final level of 6.4 m triggered eventual collapse of the unsupported berm, Figure 4. The high settlement of gauge S9 which was located on the foremost row of pile, and of S10 resulted from the piles being pushed laterally during collapse of the berm and also from a highly stressed location at the edge of the ground. No piezometers were installed the trial embankment but measurements from the nearby site showed no significant dissipation indicating that consolidation effects were minimal.

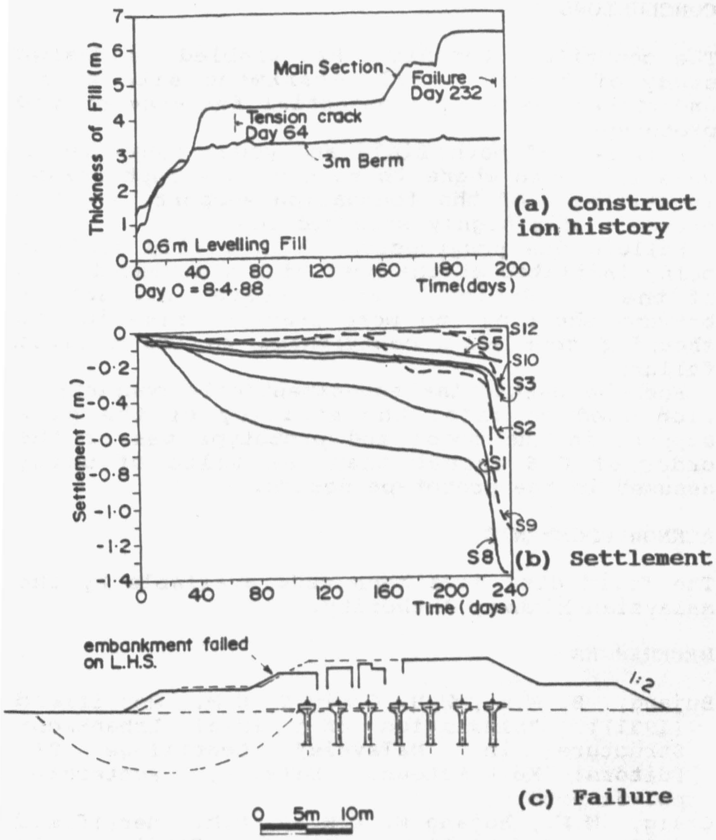

Fig. 4 : Field Results

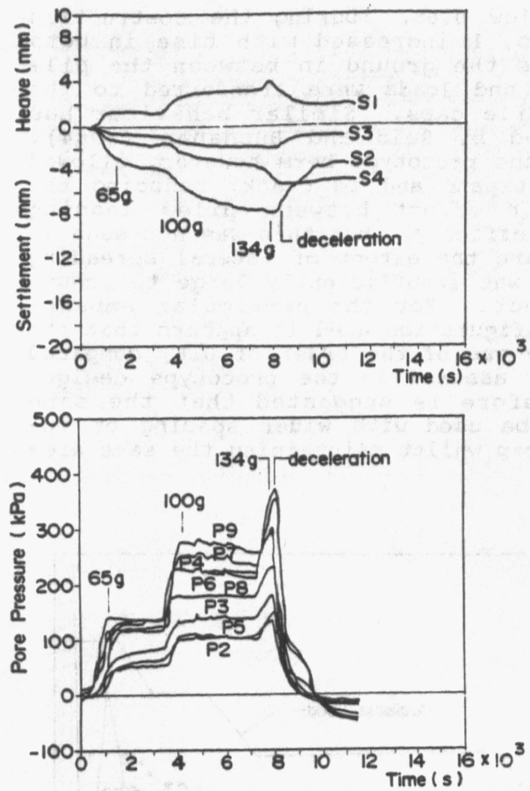

Fig. 5 : Settlements and Pore Pressure, PE3

Figure 5 shows the settlement and pore pressure record of the centrifuge model (PE3) which closest simulate the prototype. Whilst neither the construction stress paths nor the materials used were exactly identical to that of the prototype, qualitative observations were similar - the unsupported berm continued to settle whilst the continued to heave, even when the acceleration level was held constant for 1 hour at 100 g (1.1 years at field scale). Pore pressure transducers, P2 and P3, located 49 mm (4.9 m) and 48 mm (4.8 m) beneath embankment toe and berm showed high initial (end of construction) pore pressures followed by a small but continued rise in pore pressure with average increase in pore pressure ratio ($\Delta u / \Delta \sigma_v$) from 0.95 to 1.09 the test duration, indicating zones of high shear strain or local failure progression to collapse, whilst pore pressure dissipation (albeit slowly) was observed that deeper transducer P4, located 98 mm (9.8 m) below the model berm. These high initial pore pressures beneath the embankment toe and berm may have lowered the effective stress of the soil sufficiently to permit more shear strain to develop. In turn these shear strains resulted in further generation of excess pore water pressure in the shearing zone.

Tension cracks about 1 mm (100 mm) wide by 10 - 20 mm (1 - 2 m) deep were observed at the junction between the unsupported and supported sections of the model embankment with secondary parallel cracks further back, and as in the prototype these cracks occurred along lines of pile. The spaghetti tell tales exposed at the end of the test run, Figure 6, indicated

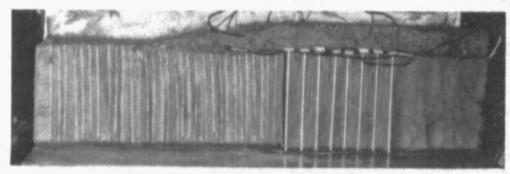

Fig. 6 :Section through Model PE3, after Testing

no major rupture but a shear zone which originated at the base of the major cracks, extended over a depth of 60 mm (6 m) beneath centre of the berm and outcropped a distance of 100 mm (10 m) forward of the toe. This broad, non circular shear zone reflects the graded profile, low absolute strengths and high plasticity of clay as well as substantial width of the berm. Although the test did not show a total collapse as in the prototype, there is sufficient evidence to indicate that essential conditions which pertained in the field had been reproduced. Both model and prototype showed that instability of the berm, but not the piled section was caused by an abrupt change in the embankment support stiffness, indicating that the decision to use the berm rather than to extend piles over a wider area under the embankment slope was a false economy.

Figure 7 shows the pile load measurement as a function of time for both prototype and model. Figure 8 shows the variation in efficiacy, E (defined as ratio of average load carried by the piles to total load above the pile heads) with H/s from the prototype with value of E from the model superimposed. The maximum values in both cases are below 0.65. During the costruction pause periods, E increased with time in both structures as the ground in between the pile caps settled and loads were tranferred to the top of the pile caps. Similar behaviour has been reported by Reid and Buchanan (1984). Collapse of the prototype berm however, allowed the fill to expand and to crack, reducing the load transfer effect between piles leading reduction in efficacy, but this was not seen in the model where the extent of lateral spreading and cracking was insufficiently large to induce such an effect. For the particular embankment/pile configuration used it appears that the pile efficacy was of the order of 0.6, compared with the 1.0 assumed in the prototype design. It may therefore be suggested that the same piles could be used with wider spacing or enlarged pile cap whilst maintaining the same area ratio.

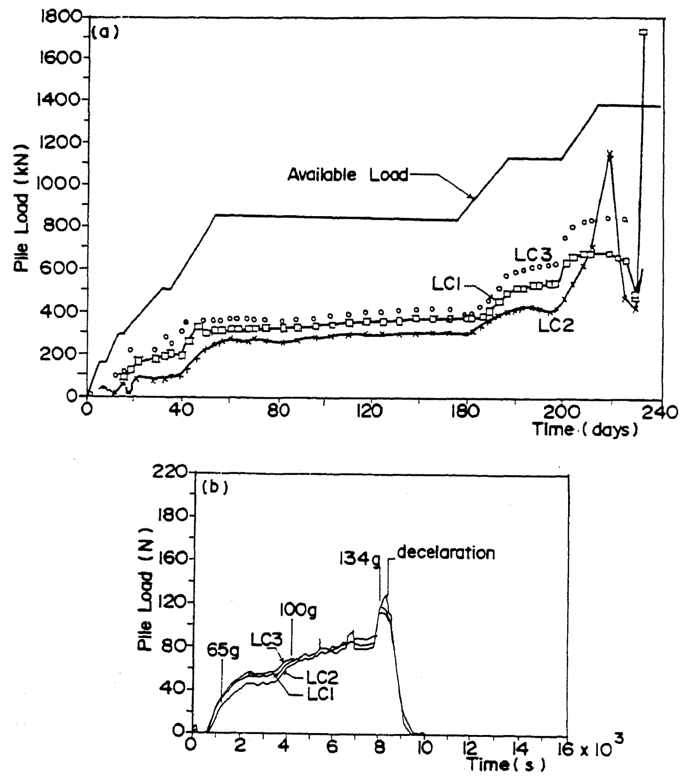

Fig. 7 : Pile Loads, (a) Prototype (b) Model PE3

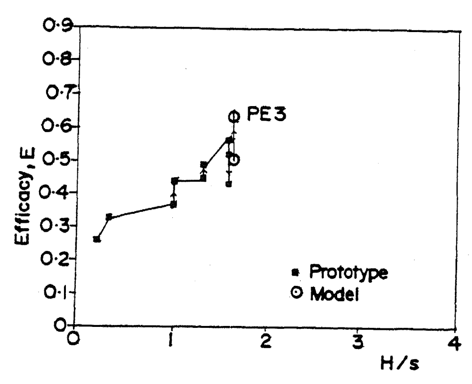

Fig. 8 : Pile Support Efficacy

CONCLUSIONS

The centrifuge technique had enabled a design study of a controlled embankment site to be undertaken, retaining essential features of the prototype.

Failures of both field and model embankments were initiated where there was an abrupt change in stiffness of the foundation support, i.e the presence of a highly stressed zone.

Failure was progressive, with a slip surface being initiated at the most highly stressed zone of the foundation. Progressive interaction between shearing and pore pressure rise in the shearing zone also contributed to the delayed failure.

For the particular embankment/pile configuration used on site, the efficacy of the pile support in the model and prototype was of the order of 0.6 rather than the value of unity assumed in the prototype design.

ACKNOWLEDGEMENTS

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