# Piled Bridge Abutments on Soft Clay – Experimental Data and Simple Design Methods

D.P. STEWART
B.E., Grad.I.E.Aust.
The University of Western Australia
R.J. JEWELL
B.E., M.Eng.Sc., M.I.E.Aust.
The University of Western Australia
M.F. RANDOLPH
M.A., Ph.D., M.I.E.Aust.
The University of Western Australia

SUMMARY Construction of bridge approach embankments on soft clay can result in the development of significant lateral soil movements, which impose lateral loads on the abutment piles. Estimation of these loads is highly uncertain at present. Results from two centrifuge model tests are presented and compared with predictions from several current simple design methods. Some of the factors influencing the loads induced in piles adjacent to embankments are discussed.

## 1. INTRODUCTION

Construction of embankments on soft clay results in the development of significant time dependent embankment settlements and movements within the soft clay. Where a bridge approach embankment is founded on soft clay, construction of the bridge itself is sometimes commenced before full settlement of the embankment has occurred. Piles supporting bridge abutments adjacent to such embankments may therefore experience significant lateral loads from horizontal soil movements, Fig. 1. These lateral loads induce bending moments and deflections in the piles which may lead to structural distress or failure of the piles or bridge structure (Leussink and Wenz, 1969; Heyman and Boersma, 1961).

Comparisons of loads predicted by various theoretical approaches with those measured in a limited number of full scale field trials have generally shown poor correlation. Hence estimation of the bending moments and deflections which will be induced in the abutment piles is uncertain. In practice, a conservative design incorporating hollow caissons to shield piles from lateral soil displacements is often adopted, or bridge construction is delayed until most of the settlement of the approach embankment has occurred.

A series of model tests have been performed on the geotechnical centrifuge at the University of Western Australia with a view to obtaining data from which a better theoretical approach may be derived. Results from two of these tests will be presented briefly and compared with several current simple design techniques.

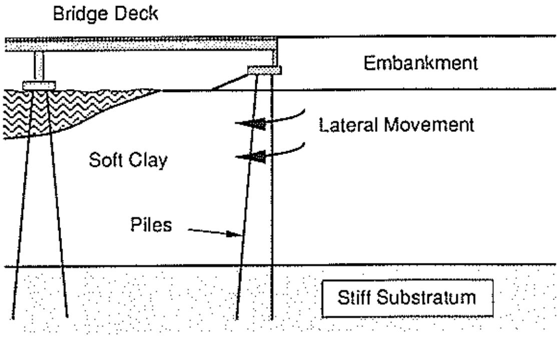



Fig. 1 Piled bridge abutment subjected to horizontal soil movements

## 2. CENTRIFUGE MODELLING

Geotechnical centrifuge modelling is a relatively new technique in the Australasian region, beginning with installation of a centrifuge at the University of Western Australia in 1989 (Fahey et al, 1990 and Randolph et al, 1991). The technique is well established internationally and has been used extensively for research, and as an aid to design, over about the last 20 years (Schofield, 1980).

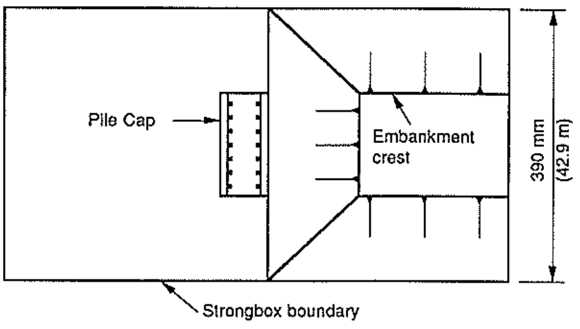
The aim of a centrifuge test is to obtain similitude of stress and strain in model and prototype. This is achieved by accelerating a scale model, where all linear dimensions are reduced by a factor, n, to an acceleration of n gravities (g), effectively increasing the self weight by a factor n. Centrifuge models have major advantages over full scale field trials, being cheaper, well controlled, more easily and accurately instrumented and will have more uniform soil conditions in areal extent.

## 3. LAYOUT OF CENTRIFUGE MODELS

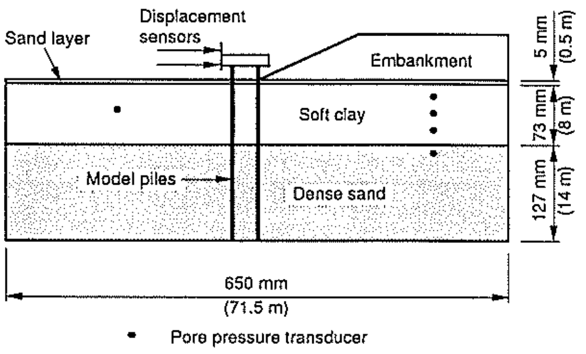
A number of models of piled bridge abutments on soft ground have been tested recently on the UWA centrifuge. Initially, tests were designed to replicate conditions at a proposed bridge site in Western Australia, although later tests were altered to examine a wider range of pile configurations and soil profiles. Model preparation and testing techniques for this project have been presented elsewhere (Stewart et al, 1991) and so will not be described in detail here.

The results of two tests will be presented; an 8 m thick layer of soft clay being modelled in one test, and an 18 m thick layer in the other. The configuration of the tests and layout of instrumentation is indicated on Fig. 2. For the shallow clay layer test, the undrained shear strength of the soft clay varied approximately linearly from about 13 kPa at the top to about 16 kPa at the base of the layer. In the deep layer test, the undrained shear strength varied similarly from about 13 kPa at the top to about 20 kPa at the base. The upper sections of each model had identical stress histories. In each test the soft clay layer was underlain by a very dense fine sand stratum.

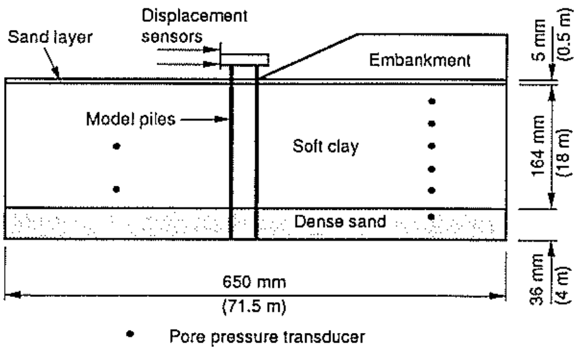
Miniature model piles of brass were constructed to replicate the steel H piles (310 UC 158 sections) proposed for the prototype structure. The scaling ratio (and hence gravity level) of the models was chosen as 110 to match approximately the bending stiffness of the model and prototype piles. Each pile was strain gauged at ten levels to measure bending moment. Four instrumented piles were included in a group of 14 (two rows of seven piles) and were connected by a rigid pile cap. Two of the instrumented piles were positioned in the front row and two in the rear row.




(a) Plan view




(b) Side view - shallow clay layer test




(c) Side view - deep clay layer test

Fig. 2 Centrifuge model layout

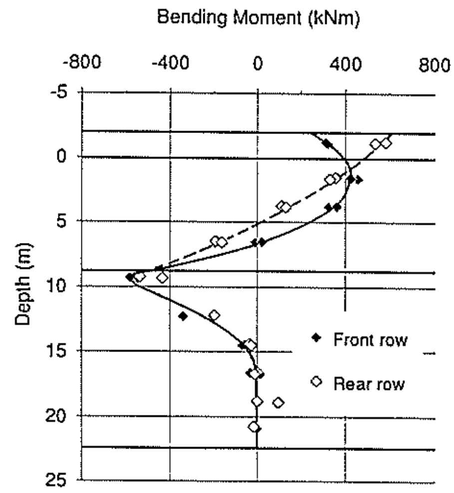
The piles were installed before the tests commenced, by pushing them fully into the sand stratum at the base of the models. This effectively fixed the piles at the base of the soft clay.

In both tests, a sand embankment was constructed in six lifts adjacent to the pile group. The embankment was built in-flight in the centrifuge by dropping sand from a hopper located directly above the model. Embankments were three-dimensional in shape, having both front and side slopes at approximately 2H : 1V, Fig. 2. The embankments were constructed to a maximum prototype height of 8 m. The dry density of the sand, as poured, was 17.5 kN/m$^3$.

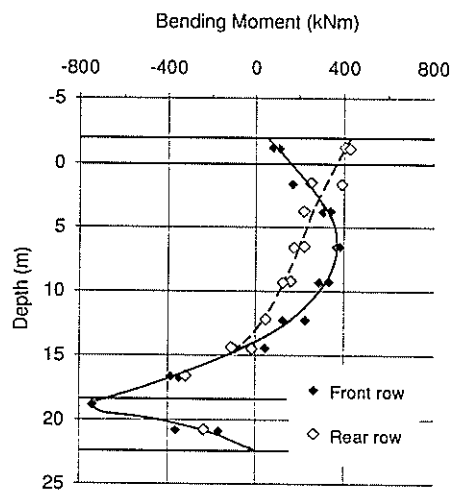
The embankments were constructed relatively quickly during the first four stages, so that the foundation behaviour was essentially undrained. The final two stages were then added after longer time delays. Pore pressure transducers located beneath the embankments allowed pore pressure dissipation between stages to be assessed. The construction histories for the two tests were designed such that the same degree of consolidation would be achieved under each embankment at corresponding heights.

## 4. CENTRIFUGE TEST RESULTS

Some representative centrifuge test results are presented here in prototype units. The model bending moments have been scaled up by a factor of $n^3$ ($110^3$), and lengths and displacements by a factor of n (110). Full explanation of the scaling laws has been given by Schofield (1980).




(a) Shallow clay layer test




(b) Deep clay layer test

Fig. 3 Centrifuge model test results - bending moment distributions at embankment heights of 8 m.

4.1 Bending Moment Profiles

Bending moment profiles from comparable stages of the two tests are plotted on Fig. 3. In this Figure, the embankment is located on the left hand side of the piles. The general shape of the profiles are seen to be similar. For both tests the maximum bending moments were recorded at the top of the sand substratum for the front row piles. However, for the rear row piles, the maximum moment occurs at pile cap level for the shallow layer test and at the top of the sand substratum for the deep layer test.

4.2 Maximum Bending Moment

The maximum recorded bending moment for the piles in both tests is plotted against the applied embankment loading on Fig. 4. The maximum moments recorded in both tests are similar, although the rear row piles in the shallow layer test experienced significantly higher moments. The bending moments recorded during the tests were not sufficient to cause plastic deformation of the piles.

The data may be interpreted simply as a bilinear response, as shown. Inspection of the Figure suggests that for any set of data, the two lines will meet at a value of the embankment loading of about 3 to 3.5 times the undrained shear strength, $s_u$ (taking 13 to 15 kPa as representative values). This load corresponds to a factor of safety for the embankment of between about 1.5 and 1.7 (5.14 / 3.5 and 5.14 / 3).

This observation compares well with the initiation of plastic deformation in the soil beneath a strip footing, which may be calculated from elasticity to occur at an applied load of about $3s_u$ (Poulos and Davis, 1980). The precise loading pressure which will initiate plastic deformation will depend on the initial stress state, as discussed by D'Appolonia et al (1971).

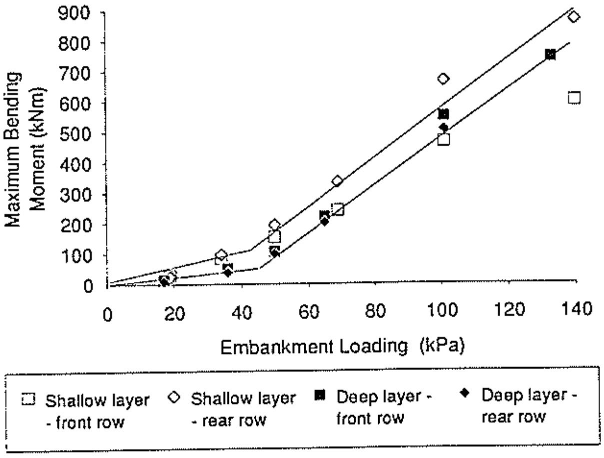



Fig. 4 Centrifuge model test results - maximum bending moments

5. CURRENT DESIGN TECHNIQUES

5.1 Pressure Based Methods

Several design techniques have been proposed based on relatively simple assumptions regarding the pressure exerted on the piles by the soft clay. These methods have all been developed from back analysis of field trials or model tests where the depth of clay was relatively small (less than 8 m) and the piles were generally relatively stiff in bending.

DeBeer and Wallays (1972) presented an empirical design method which takes into account the geometry of the embankment and the distance of the piles from the toe, Fig. 5(a). The method assumes that a uniform pressure acts on the piles over the full depth of the soft stratum. This pressure is used along with very simple assumptions regarding pile head fixity and support from stronger soil layers to estimate the maximum pile bending moment. The variation of bending moment along the pile can not be calculated.

Comparison of this simple design method with some field studies showed relatively good prediction of the maximum bending moment where the factor of safety against failure of the embankment was greater than about 1.6. This led DeBeer and Wallays to develop a second method for cases where the factor of safety is less than 1.6. In this case the most critical slip circle is drawn and the full limiting pressure ($10.5s_u$) is assumed to act on the pile above the point where the slip circle crosses the pile, Fig. 5(b).

Tschebotarioff (1973) presented design recommendations similar to the first DeBeer and Wallays method. He originally recommended that a triangular shaped lateral pressure be calculated as shown in Fig. 5(c). A revision was made to this recommendation in the light of field measurements detailed by Nicu et al (1971). In this Figure $\sigma_z$ is the increase in vertical stress at the mid depth of the soft layer at the location of the piles. Tschebotarioff suggested that the lateral pressures could be ignored where the embankment loading was less than $3s_u$ (corresponding roughly to a factor of safety of 1.7).

Springman (1989) developed a relatively simple design method based on the results of some centrifuge model tests. The method predicts a parabolic shape lateral pressure acting on the piles, the magnitude of which is calculated from a complex expression approximately taking into account the average differential pile-soil movement. The method is based on a simple triangular shaped soil deformation mechanism (Bolton et al, 1991). An interactive spreadsheet is under development to assist with this analysis for single piles (Randolph and Springman, 1991). Pile groups may be analysed by applying this lateral pressure to individual piles, and then calculating the head fixing moments to yield equal deflection and zero rotation at pile cap level (Springman, 1991). Higher lateral pressures which result from the increased lateral stiffness of the pile group (compared with a single pile), are not considered.

5.2 Displacement Based Methods

Poulos (1973) presented a boundary element analysis of a single pile embedded in an elastic soil, where the soil was undergoing lateral movement. The method requires the input of a lateral soil displacement profile, and an optional limiting pressure which can act on the pile. The method accounts for continuity of the soil mass and allows varying soil properties with depth to be specified approximately. Charts have been presented showing calculated results for single piles, although pile groups could be analysed by modifying the computer program. Comparisons with some field data proved to be inconsistent, although good agreement was shown in some cases.

Bourges et al (1980) adopted a similar approach to Poulos, where a lateral soil movement profile must be specified, except the soil was represented by a series of non-linear springs (p-y curves) and soil continuity was not accounted for. The computer program allows single piles to be analysed, although may be modified to analyse pile groups.

The disadvantage of these design methods is that the results are sensitive to the lateral soil movement profile which is input. Lateral movements are difficult to predict accurately (Poulos, 1972) although recent research has shown relatively good correlations between field data and numerical analysis using complex soil models.

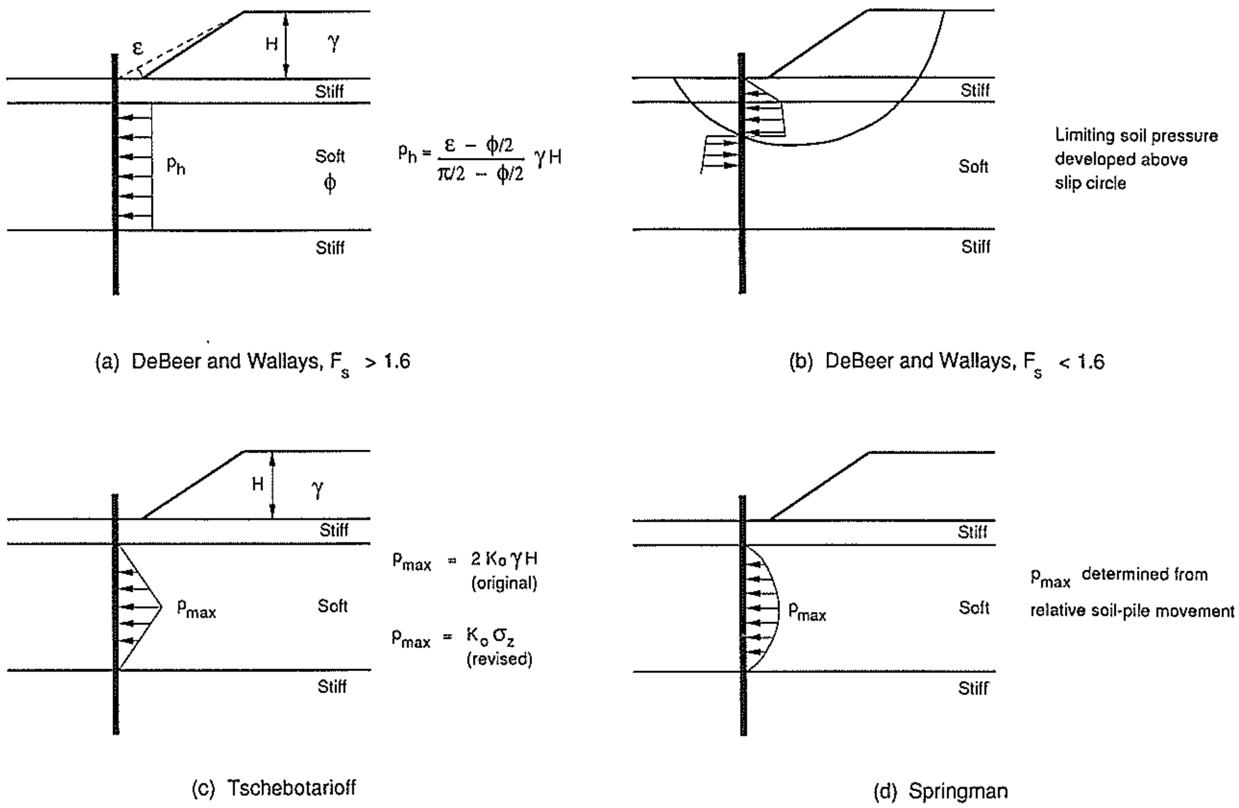



Fig. 5 Pressure based design methods

### 5.3 Finite Element Methods

Finite element analysis in several different forms has been conducted by various researchers. Randolph (1981) performed a site specific plane-strain analysis, where the piles were replaced by an equivalent sheet-pile wall with flexibility equal to the average of the piles and soil it replaced. The soft stratum was represented by the modified Cam Clay model, and the embankment by a pressure loading at the surface. The analysis allows pile groups to be analysed directly.

Naylor (1982) continued analyses of this form, but modelled the sheet-pile wall with link elements, thus allowing relative displacement of the soil and the wall and approximating more closely the true three dimensional behaviour around the piles. The soft stratum and embankment were represented by linear elastic models. The conclusions arising from this study were that link elements were not required in cases where the piles were quite flexible, or the soft layer was deep. A similar approach was adopted by Rowe and Poulos (1979) for the analysis of piles stabilising a slope, although an elastic-plastic soil model was used.

Carter (1982) performed a finite element analysis of a single pile at the centre of an axisymmetric mesh. An unsymmetric pressure loading was applied to the surface of the mesh, and a linear elastic soil model was used. Typical results were presented in chart form, showing the variation in bending moment with depth for various cases. The technique allows good representation of the three dimensional nature of the problem, although pile groups can not be analysed directly.

## 6. COMPARISON OF TEST RESULTS WITH CURRENT SIMPLE DESIGN TECHNIQUES

The centrifuge results presented earlier will be compared briefly with predictions from the three simple design methods outlined in section 5.1. The results of these analyses are shown on Fig. 6 as plots of maximum bending moment versus embankment loading. In all cases the embankment has been modelled as a plane-strain loading, which is not exactly equivalent to the centrifuge test conditions, but will provide an upper bound for the calculations.

### 6.1 DeBeer and Wallays

The results of the DeBeer and Wallays analysis are shown on Fig. 6 (a) as two linear sections, controlled by the factor of safety of the embankment. On Fig. 6 (b) only one section is shown, as the first method gives a higher moment than the second method. This analysis appears to compare reasonably well for the shallow layer test, but gives a gross over-estimate for the deep layer case. The predictions are not extended beyond an embankment loading of 70 kPa as this is the failure load predicted on the basis of plane-strain conditions and a totally undrained foundation response.

### 6.2 Tschebotarioff

Predictions from Tschebotarioff's original and revised design recommendations show inconsistent correlation with the centrifuge data. The original method compares relatively well with the shallow layer results, while the revised method compares well with the deep layer results.

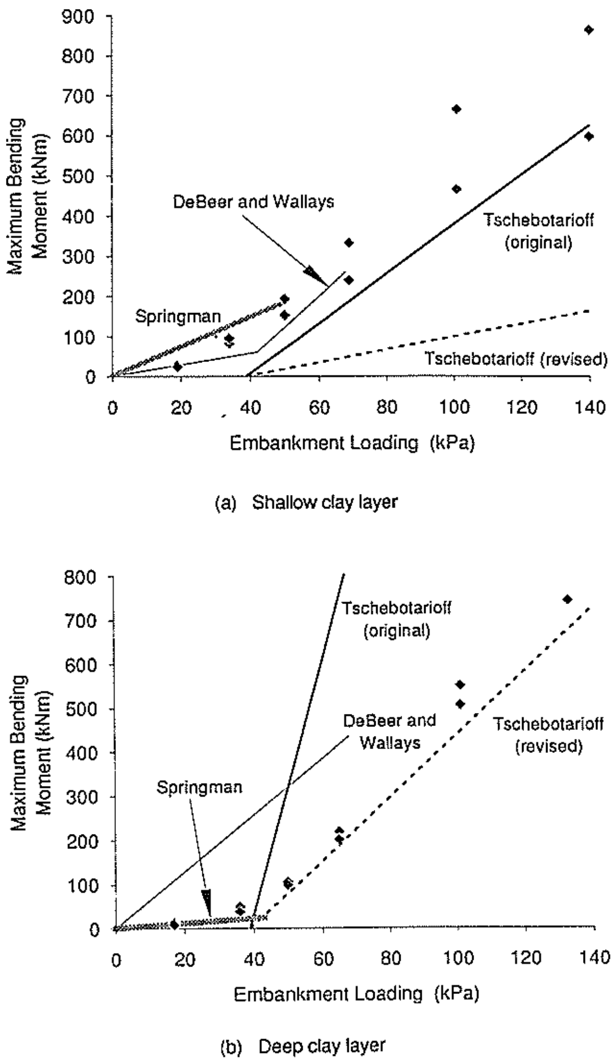



Fig. 6 Comparison of centrifuge data and simple design methods

## 6.3 Springman

Springman's method of analysis appears to predict the initial response of both tests relatively well, although it is sensitive to the elastic modulus chosen for the soft layer. The analysis is continued up until the so-called psuedo-elastic limit proposed by Springman at a factor of safety against bearing capacity failure of 1.5. Springman does not recommend design beyond this point as higher moments are expected to develop.

The predicted bending moment distributions along the piles have not been presented. However, the shape of the predicted distributions are not entirely dissimilar from those observed in the centrifuge data. The location of the maximum bending moment along the piles was generally correct, except for the rear row piles of the shallow layer test.

## 7. DISCUSSION

The simple design methods which are currently available have been shown to be generally inconsistent. The DeBeer and Wallays method is cumbersome to apply and, like Tschebotarioff's analyses, is sensitive to the simple assumptions of pile fixity. Neither of these techniques accounts for the flexibility of the piles or allows the bending moment distribution to be determined.

Springman's design method appears to reproduce the initial elastic response observed in the centrifuge data, but provides no estimate of the increased moments once plastic deformation is initiated under the embankment loading. Embankments are generally designed to factors of safety during construction of about 1.3. With the use of stabilising berms this could correspond to a maximum embankment loading well in excess of $4s_u$. Since plastic deformation has been observed to commence at an embankment load of about $3s_u$, plastic deformation is likely to be of considerable influence in the behaviour of most embankments on soft clay.

It should also be noted that Springman's design method may be unsuitable for pile groups containing a large number of very flexible piles. Individual pile flexibility is accounted for in the lateral pressure calculation, however if the individual piles form a group that has relatively high lateral stiffness, the loads may be significantly underestimated.

While the problem is obviously very complex - depending upon interaction of the embankment with the soft stratum, which in turn interacts with the pile group - a relatively simple approach to design is preferable to enable initial sizing and feasibility to be examined quickly. Complex soil conditions and pile configurations could then be accounted for by a more sophisticated analysis if necessary. The methods outlined in sections 5.2 and 5.3 may be suitable for these analyses, although more work is required to confirm their suitability.

This paper has considered the behaviour of the soft stratum to be undrained, with the effects of pore pressure dissipation and embankment construction history ignored. However, the test data suggest that bending moments develop rapidly after embankment construction, and then alter little during ongoing consolidation. The effects of embankment construction rate, and the timing of pile installation during the construction period, are likely to be of greater influence and are currently under assessment.

## 8. CONCLUSIONS

The simple design methods which are currently available to predict bending moments in piles adjacent to embankments on soft ground have been shown to be generally inconsistent, although some aspects of the observed behaviour are accounted for.

The influence of plastic deformation within the soft stratum on the magnitude of bending moments induced in the piles has been discussed. It is concluded that the ratio of load level to undrained shear strength of the soft stratum is an essential factor, that must be taken into account when assessing the loads applied to piles by lateral soil movements beneath embankments.

A new simple design method, which will take into account the observed behaviour of model and field tests, is currently under development. It is envisaged that this method will be suitable for the majority of design situations. However, for complex problems a more sophisticated approach may be necessary, perhaps based on finite element analysis.

## ACKNOWLEDGEMENTS

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