Prediction and Monitoring of Embankment Performance

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Summary The road embankments adjacent to Valery Road Bridge on Raleigh Deviation, NSW, Australia have been constructed on deep soft clays. As a result, large settlement of the embankment has taken place since completion of construction. Extensive geotechnical investigations and field monitoring have been carried out in this area to provide information on subsurface conditions and an understanding of the ground behaviour. A coupled numerical method in conjunction with the Modified Cam Clay model has been used to predict ground deformation and pore water pressure response under the embankment loading. Creep settlement has also been included in the prediction. The geotechnical model has initially been calibrated against the monitoring results and then used for forecasting the long-term performance of the embankment.

1. INTRODUCTION

Road embankments constructed on soft foundations are often accompanied by large ground deformation, which, if exceeding the allowable tolerance, could render the road unserviceable. In such a situation, it is necessary to incorporate measures to accelerate the settlement process and minimise the post-construction long-term settlement. Early settlement is commonly generated by the construction of a preload embankment. The installation of wick drains allows quicker dissipation of the excess pore water pressure generated in the foundation by the embankment loading.

Sections of the Raleigh Deviation, a newly constructed section of the Pacific Highway about 8 km long, traverse flood plains of deep alluvial soils. The newly constructed road embankments over these areas have experienced large settlements during the course of construction. To control the expected large settlements the areas of concern have been preloaded, wick drains installed and the embankment performance monitored by instrumentation.

To investigate the ground response to the embankment loading, a trial embankment, about 100 m to the north of Valery Road Bridge, as shown in Figure 1, was constructed at the early stage of the project. Extensive instrumentation has been implemented in this area and the performance of the embankment has been closely monitored.

To accurately predict the magnitude and rate of the embankment settlement, a coupled numerical method has been adopted to calculate soil deformation in response to pore water pressure dissipation. The well-established Modified Cam Clay model has been used to simulate the elasto-plastic behaviour of the clays. Creep settlement has also been taken into consideration in the prediction of long-term settlement.

The field instrumentation records have been used to calibrate the geotechnical model interpreted from the available geotechnical data. The calibrated geotechnical model has then been used for the prediction of embankment performance, and provided a basis for future road maintenance strategy.

The paper summarises the background information on the Raleigh Deviation, the geological and geotechnical conditions of the site, the results of geotechnical investigations and field monitoring, the arrangement of preloading and wick drains, the method of numerical modelling, and the results of settlement prediction.

2. GEOTECHNICAL INVESTIGATIONS

The geotechnical investigations carried out near Valery Road Bridge and the trial embankment, as shown in Figure 1, include the following:

- two boreholes,
- eight electric piezocone tests with pore pressure measurements (CPTU), and
- one test pit.

The drilling indicates that the alluvial deposits extend to a depth of about 27 m, with a 5 m thick surface layer of silty and sandy clays, underlain by a 15 m thick layer of soft clay, then a 7 m thick layer of sandy clay, clayey sand and clay. Sandy gravel occurs at a depth of about 27 m with phyllite below about 29m.

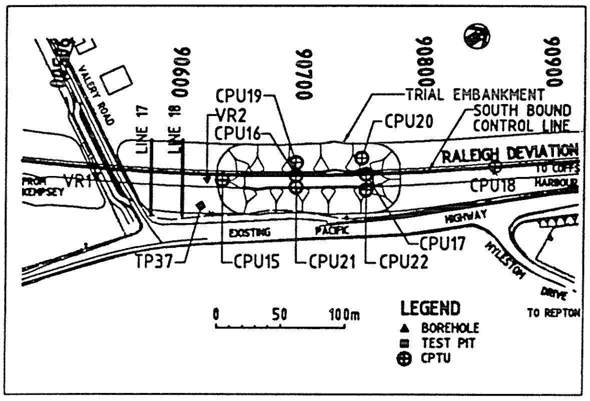

Figure 1. Site plan.

The foundations of the trial embankment are similar to the Valery Road Bridge and the results of the geotechnical investigations for the trial embankment have been used in the assessment of the northern abutment of Valery Road Bridge.

3. EMBANKMENT CONSTRUCTION

The road embankments near Valery Road Bridge have been constructed to a height of about 5.5 m with a width of 55 m over a period of about 4 months and then left unpaved to allow for settlement to occur before placement of the road pavement. The geometry and the construction sequence of the embankment are shown in Figure 2.

Wick drains were installed to a depth between 25 m and 35 m beneath the embankment on a 1.5 m grid spacing within 30 m of the bridge and at a 3.0 m grid spacing between 30 m and 50 m from the bridge.

4. INSTRUMENTATION AND MONITORING

Field instrumentation has been installed at 18 locations throughout the project site identified as Line 1 to Line 18. The instruments used include:

- hydro-static profile gauges (HPG) installed at the base of the embankments to measure the settlement profiles of the natural ground surface across the embankment,
- piezometers installed at different depths in the foundation to measure pore water pressures in response to embankment construction, and
- inclinometers installed at the toe of the embankment to measure lateral soil movements as a result of embankment settlement.

The instrumentation of the northern abutment of Valery Road Bridge consists of Line 17 close to the bridge and Line 18 about 30 m to the north of the bridge, as shown in Figure 1. The instruments installed on these two lines include:

Line 17:

- one HPG at the base of embankment,
- three piezometers at depths of 4 m, 13 m and 21 m below natural ground surface at the centre of the embankment, and
- two inclinometers at each end of the line below the toe of the embankment.

Line 18:

- one HPG at the base of embankment, and
- three piezometers at depths of 4 m, 13 m and 18 m below natural ground surface at the centre of the embankment.

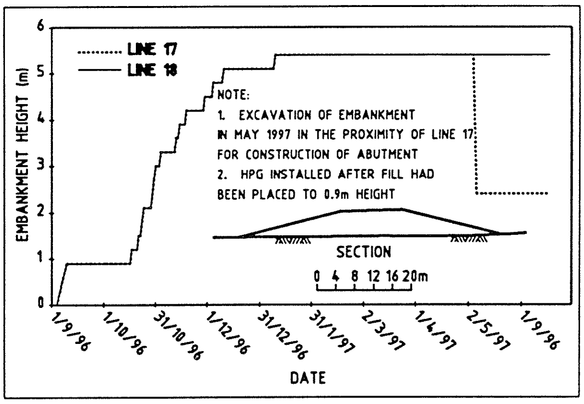

Figure 2. Embankment section and construction sequence.

5. GEOTECHNICAL MODEL

In the trial embankment area the undrained shear strength s_u and the over consolidation ratio (OCR) have been interpreted from the CPTU results. Figure 3 shows the plots of the interpreted parameters.

The presence of silty and sandy materials within 4 m depth has produced a large variation in s_u, as shown in Figure 3. Consistent results have been recorded between 4 m and 15 m with an average s_u of about 25 kPa. Below 15 m, the strength increases with depth. The scattered results at depth indicate the presence of silty and sandy bands.

The OCR value, as shown in Figure 3, decreases with depth and becomes unity below about 14 m. This indicates that the soil is over-consolidated at shallow depths and becomes normally consolidated below 14 m.

The horizontal coefficient of consolidation c_h has been derived from the CPTU dissipation test results with the interpreted values summarised in Figure 3. Higher values of c_h occur near the surface with the presence of silty and sandy materials resulting in faster dissipation of pore water pressures.

The Cam Clay parameters λ, κ and M can be derived from the compression and re-compression indices C_c and C_r and effective friction angle ϕ. Schmertmann (1978) suggested a $C_c/(1+e_o)$ value (where e_o is the initial void ratio) of 0.4 for a clay with s_u/σ'_{vo} (undrained shear strength/effective overburden stress) ranging from 0.1 to 0.25 and an OCR of approximately 1, similar to the clay of the project site. Experience obtained from a similar site near the project suggests that the $C_c/(1+e_o)$ value of soft clays ranges between 0.3 and 0.4. A $C_c/(1+e_o)$ value of up to 0.4 has been assumed in this study. The C_r/C_c ratio has been assumed to be 0.1 (Holtz and Kovacs, 1981). An effective friction angle ϕ between 30° and 33° has been considered reasonable for the type of clays at the site.

The creep factor $C_{\alpha\varepsilon} = C_\alpha/1+e_o$ (where C_α is the secondary compression index), based on Mesri (1973), can be related to the moisture content w_c of the soil. The clays of the site have been found to have an average w_c of about 50%, giving $C_{\alpha\varepsilon}$ ranging between 0.002 and 0.015. Typically C_α = $0.05C_c$ (Holtz and Kovacs, 1981). For $C_c/(1+e_o)$ = 0.15 to 0.4 as shown in Table 1, $C_{\alpha\varepsilon}$ is calculated to be between 0.0075 and 0.02. In this study the creep factor $C_{\alpha\varepsilon}$ has been assumed to be 0.01.

The bulk density γ_t, moisture content w_c and void ratio e_o of the soil obtained from the laboratory testing are given in Figure 4.

The soil stratigraphy and geotechnical parameters adopted for the analysis are given in Table 1.

6. METHODOLOGY

A numerical method for the analysis of consolidation in a saturated soil (Small *et al.*, 1976, Hsi, 1992, and Hsi and Small, 1992a and 1992b) has been adopted in this study. The method calculates the deformation of soil and the dissipation of excess pore water pressure simultaneously during the course of consolidation. This method is capable of solving the following problems that are relevant to this study:

- consolidation settlement with pore pressure dissipation,
- two-dimensional effects in the transverse direction of the road embankment,
- lateral deflection of soil due to embankment loading,
- multiple soil layer system,
- staged embankment construction sequence, and
- wick drain effects with lateral pore pressure dissipation.

A finite element program COFEA (COupled Finite Element Analysis) implementing the above mentioned numerical method has been used for the analysis. This program was coded in Fortran 77 language and uses 8-noded isoparametric elements. The program is capable of performing plane strain and axi-symmetric analysis and is also capable of simulating the elastic or elasto-plastic (Modified Cam Clay) behaviour of the soil.

7. FINITE ELEMENT SIMULATION

As shown in Figure 2, during bridge abutment construction in May 1997 the embankment was partly excavated near Line 17. It is difficult to quantify the impact of this excavation to Line 17 and it has not been included in the numerical analysis. It is also noted that the HPG's were installed after the fill had been placed to a height of 0.9 m; any ground settlement before the HGP installation was therefore not recorded. The finite element simulation has taken into account this delayed installation of the HPG's.

A symmetric finite element mesh has been adopted for the analysis that consists of 308 isoparametric elements and 997 nodes.

The modelling of wick drains in the two-dimensional plane strain analysis requires some approximation and simplification. Equivalent horizontal permeability k_h of the soil has been assessed taking into account the actual wick drain spacing, the finite element mesh size and the smear effects. A wick drain spacing of 1.5 m is considered for Line 17 and 3 m for Line 18. Smearing of wick drains often occurs due to the installation process that generally results in a reduction in soil permeability and a delay in soil consolidation. Based on the back analysis, a smear factor of 2.5 was assumed.

Baligh and Levadoux (1980) suggested that k_h/k_v (where k_v is the vertical permeability) ranges from 1.2 to 10 depending on the condition of soil layering. As the clay at the site is reasonably uniform with little evidence of layering, the k_v values were assumed based on a k_h/k_v ratio of 1.2. In this study, the k_v value has little effect on the process of consolidation due to the presence of the wick drains.

Table 1. Geotechnical parameters assumed for analysis.

DEPTH (m)	SOIL TYPE	γ_t (kN/m^3)	k_h (m/day)	k_v (m/day)	ϕ (degree)	e_o	$C_c/(1+e_o)$	$C_r/(1+e_o)$	λ	κ	M	OCR
0 - 2	clay	18	1.0E-04	8.33E-05	30	1.5	0.4	0.04	0.434	0.0434	1.2	10
2 - 5	silty/sandy clay	18	1.0E-03	8.33E-04	30	1	0.35	0.035	0.304	0.0304	1.2	4
5 - 10	clay	17	1.0E-04	8.33E-05	30	1.5	0.4	0.04	0.434	0.0434	1.2	2
10 - 15	clay	17	1.0E-04	8.33E-05	30	1.5	0.4	0.04	0.434	0.0434	1.2	1.2
15 - 20	clay	17	1.0E-04	8.33E-05	30	1.5	0.2	0.02	0.217	0.0217	1.2	1
20 - 24	silty/sandy clay	18	1.5E-04	1.25E-04	33	1	0.15	0.015	0.130	0.0130	1.3	1
24 - 27	clay	19	1.0E-04	8.33E-05	33	1.5	0.15	0.015	0.163	0.0163	1.3	1
>27	sandy gravel	-	-	-		-	-	-	-	-	-	-

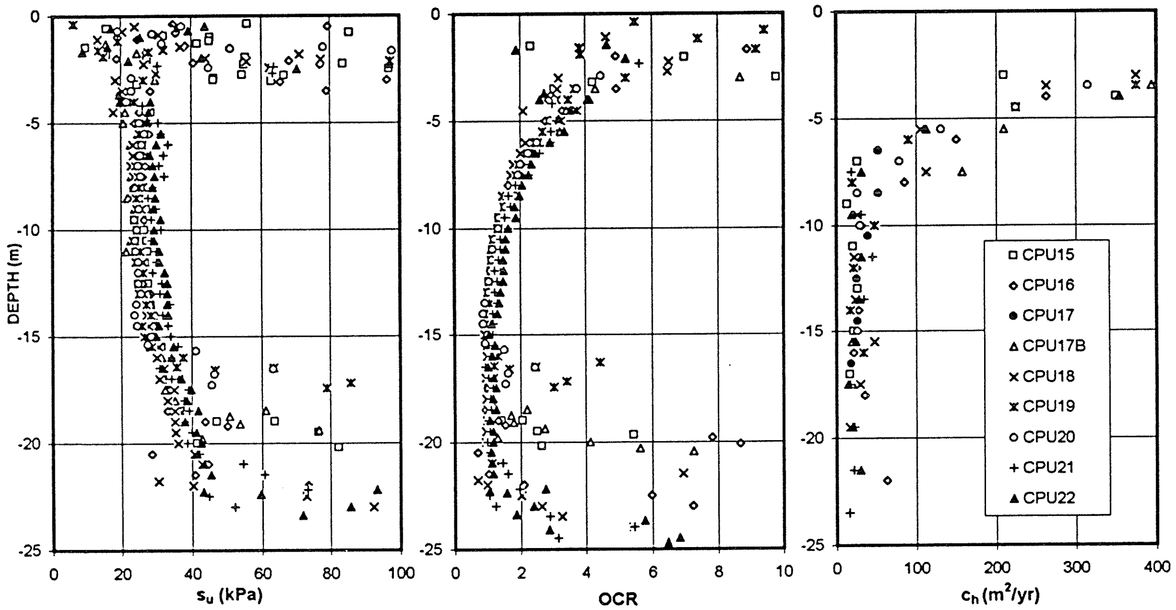

Figure 3. Interpreted results of piezocone testing.

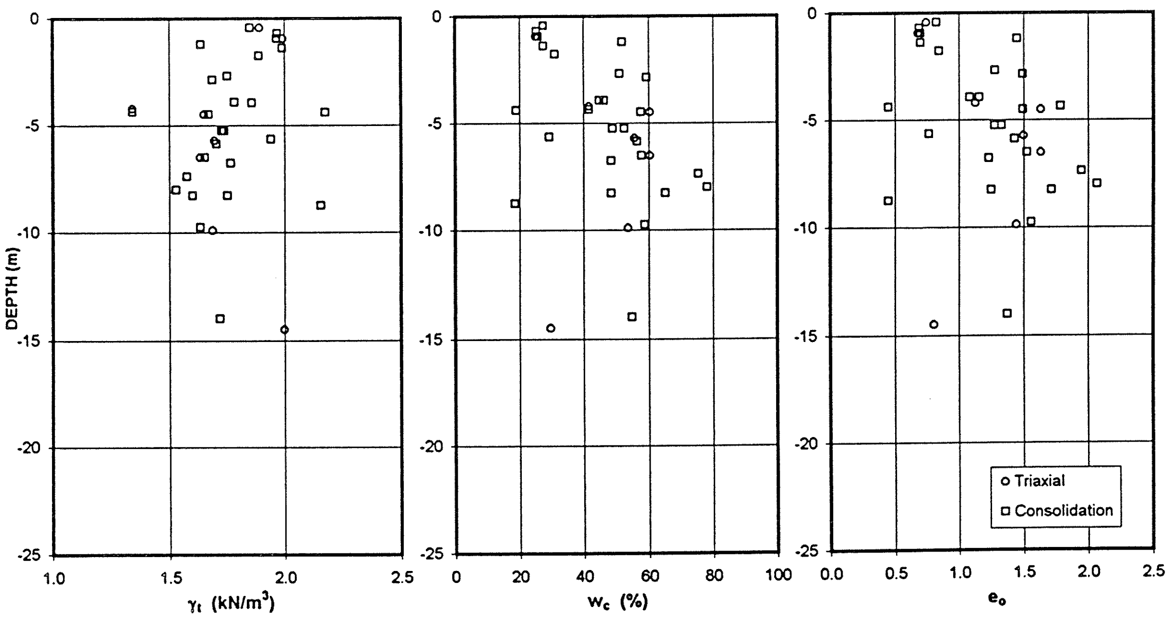

Figure 4. Physical soil properties.

The bottom sand and gravel layer is not modelled in the finite element analysis, ie, the layer is fixed from movement in the vertical direction. The Standard Penetration Test results in refusal in this layer indicating that the material is very dense and its contribution to settlement would be negligible. The base sand and gravel layer is assumed to be permeable in the analysis with the total water head in this layer fixed at the level of the water table.

The numerical analyses provide solutions of primary consolidation settlement in the ground as a result of pore water pressure dissipation. Creep settlement is added to the primary consolidation settlement to give the total settlement. Due to the nature of the soft clays, it is assumed in the analysis that the creep settlement started from the time when the major filling activity commenced (approximately in mid-October 1996).

8. PREDICTIONS

The predicted and measured settlements for Lines 17 and 18 below the centre of the embankment are shown in Figure 5. Reasonable agreement between the measurements and the predictions has been found for Line 18. It is noted that in May 1997 the measured settlements along Line 17 deviated from the predictions. This is apparently related to the excavation for the bridge abutment adjacent to Line 17. The predicted trend of settlements appears to be of the correct order. The predicted and measured settlement profiles across the base of the embankment are shown in Figure 6.

surrounding wick drains resulting in quicker dissipation of excess pore pressure. In the numerical modelling the piezometer is assumed to be at equal distance to the surrounding wick drains. The response of the pore pressures at different depths in the soil has been well predicted and indicates that the settlement rate of each soil layer has been reasonably predicted.

The predicted and measured lateral soil deformations beneath the toe of the embankment are shown in Figure 8. The predicted lateral deflections have been found to be generally greater than the measurements. Poulos (1972) noted that, due to the nonhomogeneous and anisotropic nature of the soil, lateral soil deflections are difficult to predict even though it has been possible to obtain very good predictions of vertical movements and pore pressures.

9. SUMMARY AND CONCLUSIONS

Some road embankments at Raleigh Deviation, NSW, Australia have experienced large settlements since construction. Extensive geotechnical investigations and instrumentation monitoring have been undertaken to provide crucial information in the prediction of the long-term performance of the embankment and the determination of future maintenance strategies.

The results of the investigations and field monitoring in the vicinity of the Valery Road Bridge northern abutment have been presented in this paper. Deep alluvial soft clays have contributed to significant ground settlement. Measured settlement beneath the centre of the embankment over a period of one and a

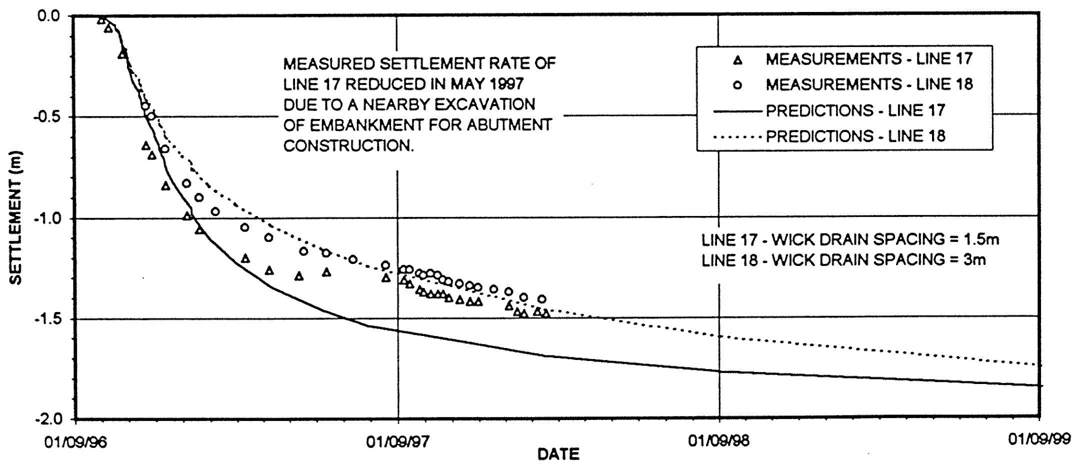

Figure 5. Predicted and measured settlements below centre of embankment.

The pore water pressure measurements and predictions are shown in Figure 7. The predicted pore pressures are in general slightly higher than the measurements. This could be due to the fact that the piezometer may be close to one of the

half years has been about 1.5 m. This settlement is considered to be a combination of the primary consolidation and creep settlement.

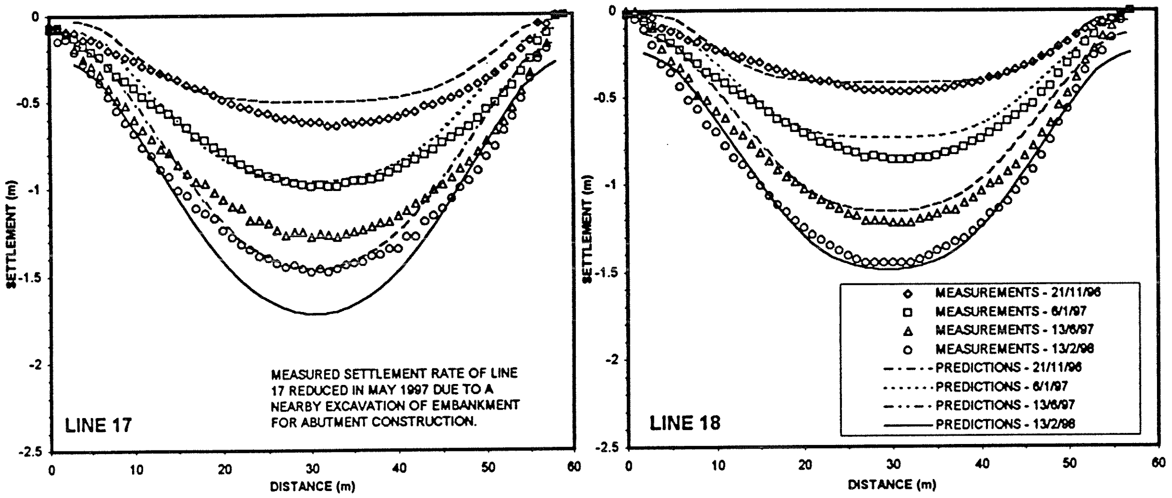

Figure 6. Predicted and measured settlement profiles at base of embankment.

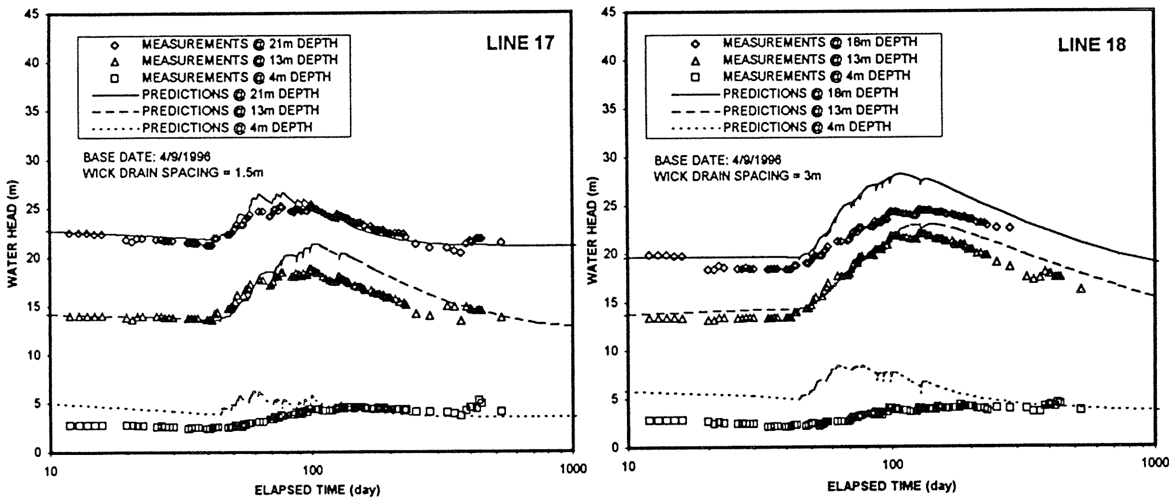

Figure 7. Predicted and measured pore pressures below centre of embankment.

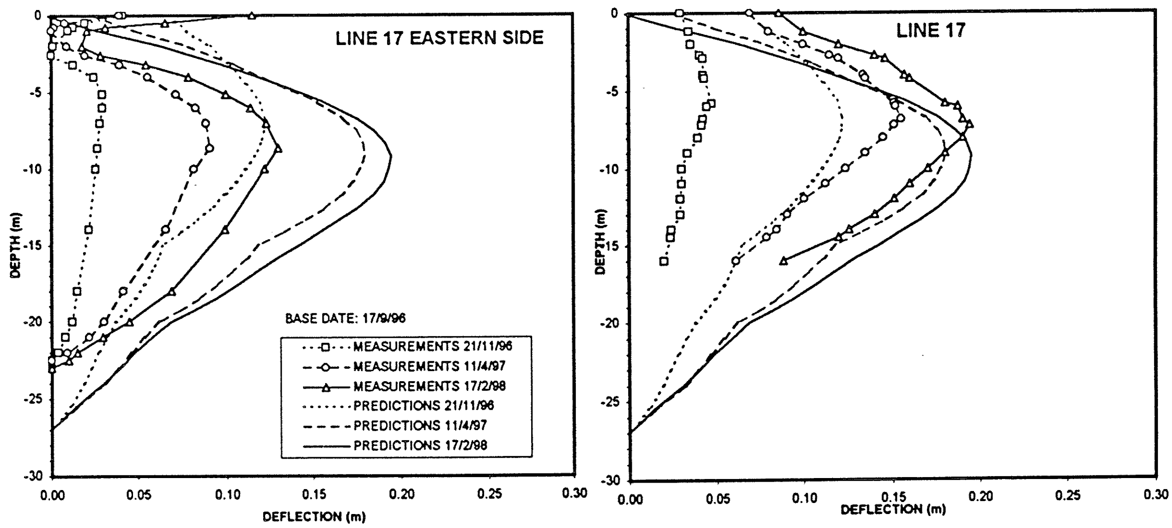

Figure 8. Predicted and measured lateral deflections below toe of embankment.

A coupled numerical method has been adopted in this study for the prediction of ground settlement in response to pore water pressure dissipation. The Modified Cam Clay model has been used to simulate the behaviour of normally consolidated to lightly over-consolidated clays.

The geotechnical model derived from the field investigations and laboratory test results has been calibrated based on the field monitoring results. Good agreement between the measurements and the predictions has been achieved giving a reasonable level of confidence on the forecast of the long-term performance of the embankment.

10. ACKNOWLEGMENTS

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