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Hexham embankment, case study, wick drains, predictions

Remblai d'Hexham, analyse d'un cas, drains verticaux, prédictions

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SYNOPSIS This paper presents a case study of the behaviour of the southern approach embankment to a major new bridge at Hexham, N.S.W., Australia. The embankment traverses a swamp underlain by soft estuarine clay deposits up to 16 metres deep. Topics covered include site investigations, in situ and laboratory testing, vertical wick drain design and settlement predictions. A comparison is made between predicted and measured embankment behaviour. In conclusion, comments are offered on the efficiency of the vertical drain system and the suitability of the various prediction methods adopted.

INTRODUCTION

A new 15-span-578 metre long bridge, designed by the N.S.W. Department of Main Roads, is nearing completion across the Hunter River at Hexham, just north of Newcastle, on the mideast coast. Located at the intersection of the heavily trafficked Pacific and New England Highways, the $14 million bridge will carry northbound Pacific Highway Traffic on its scheduled completion in late 1985. This Paper presents a brief case study of the geotechnical aspects and behaviour of the southern approach embankment (Fig. 1).

SITE INVESTIGATION AND IN SITU TESTING

Site investigation was carried out using seven 'N' size boreholes in conjunction with the quasi-static cone penetration test (Dutch Cone), the Vane Shear Test (VST) and the Standard Penetration Test. Drilling revealed 0.5 - 1 m of slag fill overlying 11 - 16 m of high plasticity, very soft silty clay with occasional bands of shells. The moisture content, varying from 40 to 80% is close to the liquid limit. Below the clay is a 4 - 8 m layer of dense, fine-medium sand which in turn overlays a very stiff slightly over-consolidated silty clay varying in thickness from 2 - 10 m. Fresh bedrock occurs around 5 m deeper at RL. 70 - 75 m. Longitudinal and transverse soil sections with embankment plan are presented in Fig. 1.

A comparison of the VST results with the Dutch Cone logs yields a computed mean cone factor of 15.9 which is consistent with published data (Schmertmann, 1975).

FIELD INSTRUMENTATION

The field instrumentation (Fig. 1) consists of:

(i) 15 Petur pneumatic piezometers - 12 x R103 and 3 x R102 Wellpoint units.

(ii) Three Sinco 'Sondex' downhole settlement monitors. This system uses a 65 mm corrugated plastic pipe with wire rings and an induction probe. Accuracy ± 5 mm.

(iii) Two inclinometer monitors using a standard Sinco grooved 70 mm O.D. casing installed in N-size boreholes.

(iv) Surface survey pegs, used as a simple check on the other systems.

LABORATORY TESTING

The laboratory programme comprised Moisture Content, Dry Unit Weight, Atterberg Limits, Specific Gravity, Oedometer Settlement, CU and UU Triaxial tests. The latter were found to be suspiciously low due to sample disturbance of the very soft material and were rejected in favour of the corrected VST values (Bjerrum, 1972). A summary of the most important results for settlement predictions is presented in Fig. 2.

SETTLEMENT PREDICTIONS

Two different levels of predictions were made on this Project, Type A and Type B/B1 (Lambe, 1973). These were principally concerned with the absolute values of total final settlement (S_{TF}) and the time (t_{90}) taken to reach 90% degree of consolidation (U_{90}).

Type A Predictions

In the early planning stage of the bridge earthworks, urgent estimates were required of S_{TF} and t_{90}. This was to enable basic decisions to be reached on blanket drain

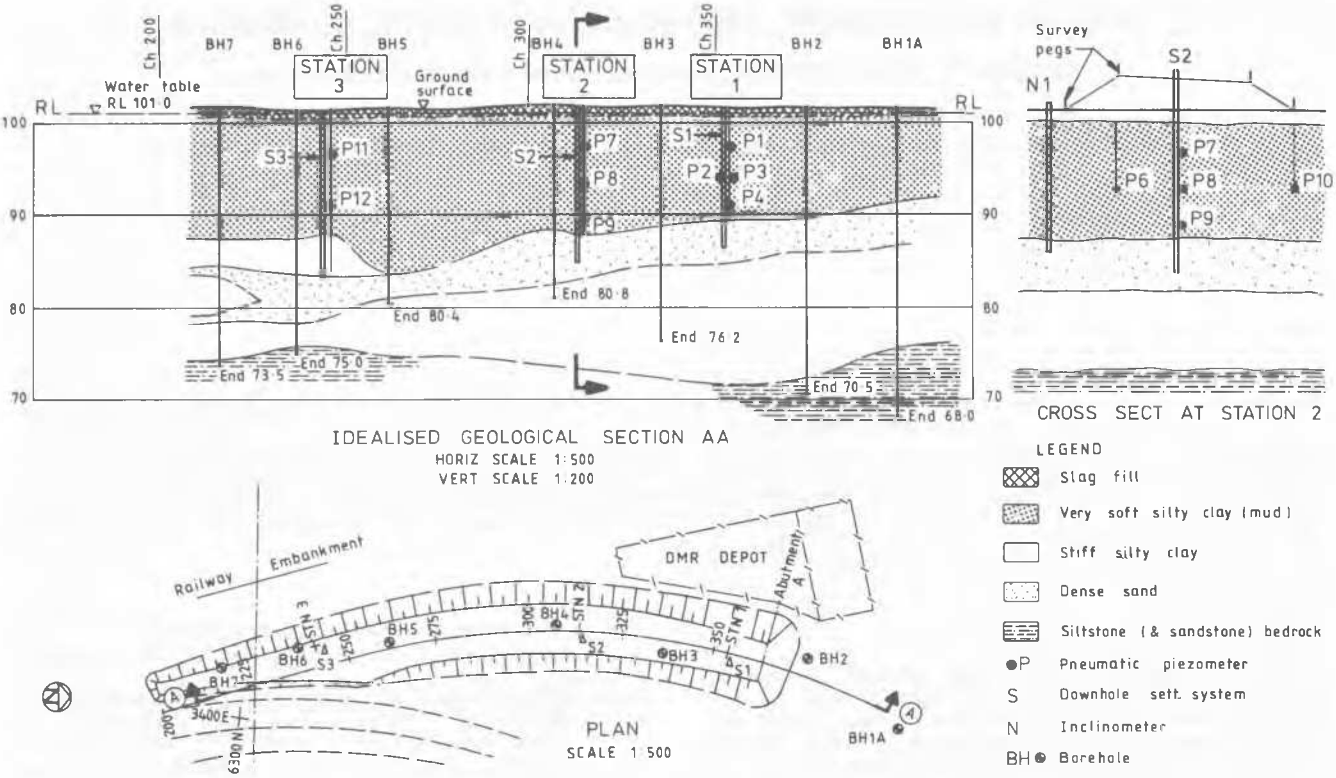

Fig. 1 Longitudinal and Transverse Sections. Embankment Plan and Instrumentation.

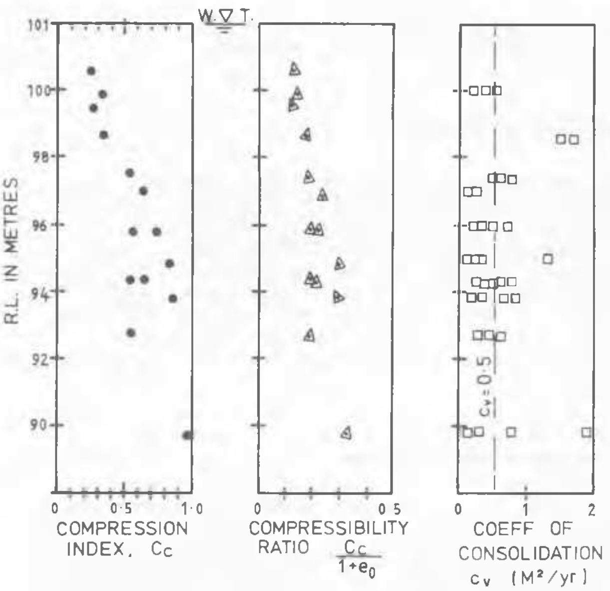

Fig. 2 Results from oedometer tests on the very soft silty clay.

thickness under the embankment and the type of surcharge or other mechanism needed to accelerate consolidation.

Limited field and laboratory data was available at this stage and monitoring had not commenced, so certain general assumptions had to be made for key soil parameters.

The determination of S_{TF} followed the conventional One-Dimensional Method in which σ'_{vm} was estimated from the VST undrained strengths, c_u as follows:

$$\text{since} \qquad \sigma'_{vm} = OCR\ \sigma'_{vo} \qquad (1)$$

$$\text{and} \qquad \frac{(c_u/\sigma'_{vo})\,O/C}{(c_u/\sigma'_{vo})\,N/C} = OCR^{0.8} \qquad (2)$$

(Ladd et al., 1977) and for Hexham clay,

$$\text{the estimated} \quad (c_u/\sigma'_{vo})\ N/C = 0.28 \qquad (3)$$

combining equation (1), (2) and (3) gives

$$\sigma'_{vm} = 4.91\ \frac{[c_u]^{1.25}}{[\sigma'_{vo}]^{0.25}} \qquad (4)$$

Consolidation time was estimated using the standard 1-D approach.

Type B Predictions

As monitoring of pore pressures and settlement progressed and the results of additional laboratory and Dutch Cone testing became available, more reliable Type B and B1 predictions were possible for S_{TF} and t_{90}. For example, the important σ'_{vm} distribution was replotted using a combination of data gathered from oedometer results, pore pressure response during construction and Dutch Cone traces. This is shown in Fig. 3.(Stone, 1984).

Additionally, back-analysis produced more accurate assessments of C_c and c_h, coefficient of consolidation horizontally and demonstrated that the profile behaved as two layers - an uppermost stratum of moderate permeability above RL. 97 overlying a less permeable stratum. A strain profile with depth could be plotted from the 'Sondex' monitoring. Back-analysis of the upper rings, using the σ'_{vm} values as per Fig. 3 resulted in good estimates for C_c. The predicted final primary strain distribution was computed using an embankment loading on an elastic foundation, the σ'_{vm} profile and the back-analysed C_c values.

The Asaoka (1978) settlement prediction method was also used at various stages during the monitoring programme with mixed results.

Only the techniques for settlement predictions have been discussed in this section. To avoid repetition the actual predicted figures are tabulated alongside the measured values in the section after the next one.

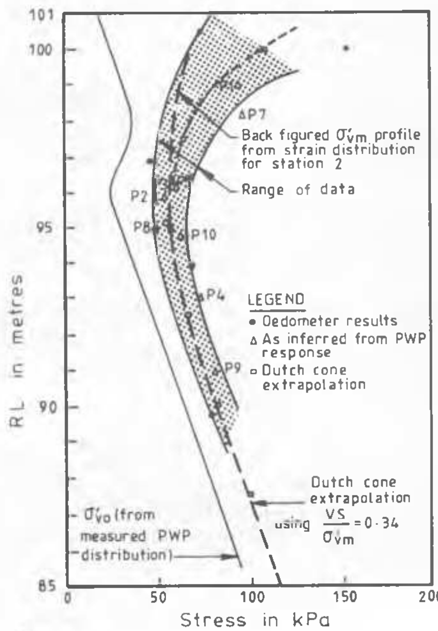

Fig. 3 In situ Stress Profile Summary

VERTICAL WICK DRAIN DESIGN

An estimated 740 mm total consolidation settlement taking 15 years to occur was unacceptable for project purposes. Large surcharges were investigated but found unworkable because of the low c_u values of around 20 kPa which would lead to shear failures. In order to achieve a U90 settlement under the scheduled construction time of 15 months, the decision was made to employ a vertical plastic wick drainage system. This system had proven very successful in a previous Departmental Project where similar soil conditions prevailed (Coleman, 1981). The wick drain was designed based on the principles discussed by Barron, 1948.

An assumption was made for c_h as 8 times the lab. c_v value of ½ m /yr to yield 4 m /yr. Using Australian manufactured 'NYLEX FLO-DRAIN' on a 1.8 x 2.3 rectangular grid pattern, the calculated t_{90} time was 1.1 years.

COMPARISON BETWEEN PREDICTED AND MEASURED SETTLEMENTS

Total Final Settlement

Table I below summarises the predicted total final settlements by various methods, as discussed previously, together with the actual measured values at 350 days.

TABLE I

Predicted and Measured Settlements

Prediction		Settlement (mm)	
Method	Type	Predicted	Measured
* 1-Dimensional	A	610	615
'Sondex' strain distribution	B	640	
Asaoka	B1	600	
** 1-Dimensional	A	740	Not yet determined
'Sondex' strain distribution	B	840	
***1-Dimensional	A	540	532
'Sondex' distribution	B	530	
Asaoka	B1	550	

*Station 1, 4.1 m fill, 1 m below design RL.
**Station 1, 5.1 m fill, @ design RL.
***Station 2, 3.5 m fill, @ design RL.

While the predicted values appear generally quite close to the measured ones, it is noted that the final settlements are as yet unknown, as monitoring on the Project is still proceeding. Because of unexplained residual pore pressures of 20 - 30 kPa above hydrostatic, a subject beyond the scope of this paper, it is not possible to state accurately the present degree of consolidation. However, it is likely that the U90 state is close to achievement.

Rate of Settlement

Typical behaviour is presented in Fig. 4 (Station 1). From these curves, it is noted that the predicted 350 day settlement of 533 mm (corresponding to an STF of 610 mm) represents 87% of the measured value of 615 mm at the same date.

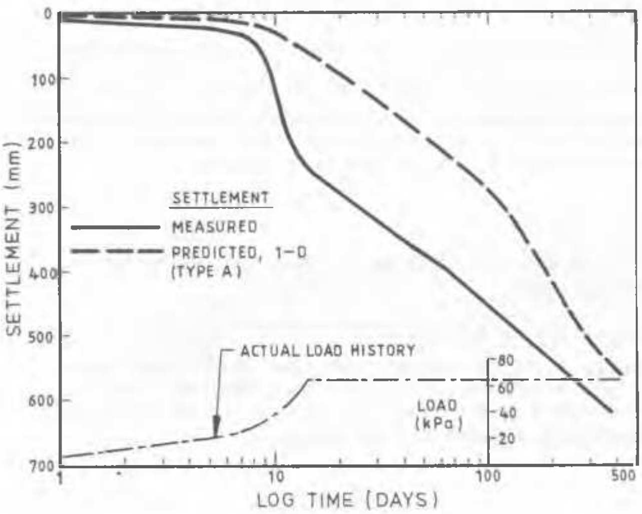

Fig. 4 Time-Settlement at Station 1 Measured and Predicted.

CONCLUSIONS

The wick drain system appears to be functioning satisfactorily. This is concluded from the fairly uniform strain profile with depth in the very soft clay. The difficult nature of predicting c_h with any certainty, short of actually building a trial embankment containing wick drains, was evident throughout the Project. This option was, unfortunately, not available due to time, space and other constraints. The wick spacing could have been closer, say at 1.5 m centres, i.e. a more conservative estimate made of field c_h. However, in the circumstances, the engineering judgement exercised initially in estimating c_h was reasonably adequate.

The predicted values for total final settlement are very similar by all three prediction methods and very close to the actual measured settlement to date. However, as noted, the degree of consolidation is not known exactly and it is possible that all methods have under-estimated the actual final settlement. The time-settlement comparison, although marginal during the first few months, is improving markedly with time as the two curves converge.

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