The Use of Trial Embankment Observations in the Construction Control of Roadway Embankments on Soft Soil

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SUMMARY Measurements from an instrumented trial embankment were used to develop calibrated stability and settlement analysis models, for the behaviour prediction of high roadway embankments on soft soil. The analysis models were based on the relatively unsophisticated classical theories. The effective stress stability model was used to prepare a Stability Monitoring Diagram, which, in conjunction with very simple pore pressure and settlement measurements, was used to control the rate of filling the roadway embankments. The settlement model provided realistic estimates of settlement behaviour, and showed that surcharging was required to bring consolidation times within the available construction period. The embankments were successfully constructed under stability control without need of stabilisation measures, and primary consolidation effectively completed within the available construction period.

1 INTRODUCTION

The approaches to a new high level bridge over the Pioneer River at Mackay, Queensland included embankments up to 7 m high located over tidal river mudflats. The embankment foundations included varying depths of very soft organic silty clay alluvium overlying loose to dense sandy alluvium and stiff silty clay above bedrock.

The initial site investigation incorporated total stress stability analyses using peak undrained shear strengths, which showed that extensive toe berms, at least 2 m high, would be required to maintain stability of the high embankments during construction. Settlement analyses indicated that settlements in excess of 1 m would occur over 2 to 12 years. In order to more reliably define the expected behaviour of the proposed embankments, an instrumented trial embankment was constructed with the following general aims.

(i) Determine reliable insitu strength and consolidation data for use in prediction analyses.

(ii) Assess changes in the state of stability during and after construction and, if necessary, devise corrective or control measures for use with the roadway embankments.

(iii) Assess the settlement behaviour with a view to the reliable prediction of settlement behaviour for the roadway embankments.

This paper describes the development of stability and settlement analysis models, based on the trial embankment observations, and the application of these to the construction control of the roadway embankments.

Stability monitoring of daily construction, using Stability Monitoring Diagrams, has been described by a number of authors (e.g. Margason and Symons, 1969; Cook and Ingold, 1974; Cole, 1974; Symons, 1976), in which the most popularly used method of control is based on effective stress stability analyses relying, as input data, on excess pore pressure observations. The relevance of this technique to the control of the roadway embankment construction was recognised. However, practical necessity dictated that the aims of reliable prediction and control would have to be achieved by largely relying on the commonly available, relatively unsophisticated computer-based settlement and stability analytical tools, and on simple, limited scope but reliable instrumentation for the roadway embankments. Consequently, reliance was placed on calibration of the settlement and stability analytical models for the local situation, based on back-analysis of the trial embankment observations.

2 SITE DESCRIPTION

The soft compressible surface alluvium consisted predominantly of silty clay, and varied in thickness from 2 m to 9 m across the site. A continuous silty sand layer, between 0.2 m and 1.8 m thick, existed over the site within the silty clay below a depth of 1.5 m to 3.0 m, and numerous other thin sandy lenses of unknown continuity were observed within the silty clay. A typical profile is shown in Figure 1 together with moisture content data and undrained shear strengths from field shear vane tests. The silty clay is highly sensitive (sensitivity range 4-34), with mean sensitivity of 15. The sensitivity is reflected in the apparent high state of liquidity shown by the moisture content and plasticity results. Extensive consolidated undrained triaxial testing was carried out from which representative (c', ϕ') values were selected (Figure 1) for effective stress stability analyses.

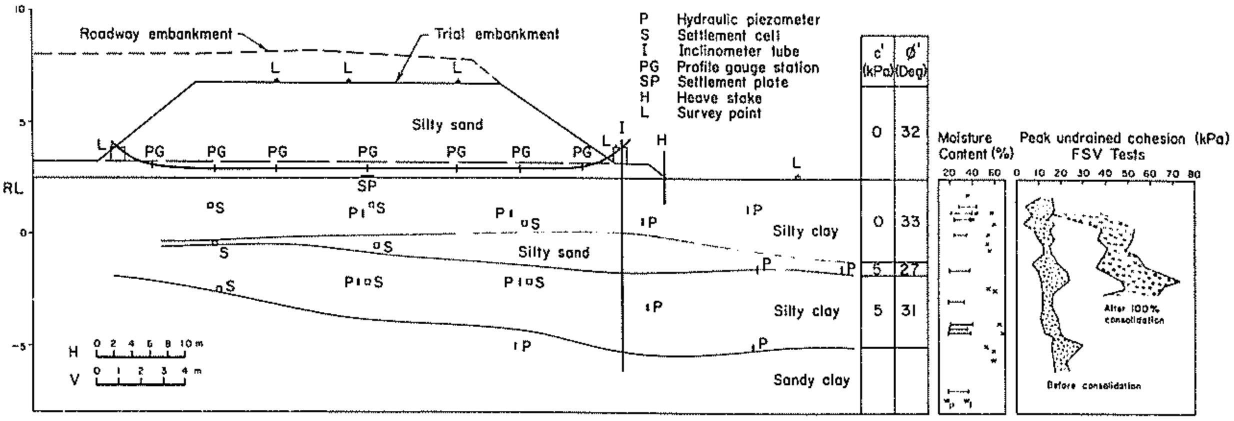


Figure 1 Typical subsurface details and trial embankment instrumentation

Insitu undrained shear strengths were remeasured by field shear vane after completion of consolidation of the trial embankment. Strength increases of 3 to 5 times were recorded, and the sensitivity reduced to a mean of 8, and range of 2 to 16.

Laboratory consolidation tests were performed on 23 specimens (76 mm diameter) of silty clay, and the consolidation parameters (Coefficient of volume change m_v, and Coefficient of consolidation C_v) were statistically reduced, assuming log normal distributions (Lumb, 1968) at constant effective stress. The laboratory mean relationships with effective stress, and confidence limits are shown later in Figure 6, compared with insitu relationships derived from back-analysis of the trial embankment settlement.

3 EMBANKMENT DETAILS

The roadway embankments included approximately 700 m of 2 and 4 lane embankment over the soft soil foundation, incorporating a large intersection area, and with heights of 5-7 m. They were constructed between October, 1978 and June, 1979, using the stability and settlement control methods described in this paper.

The trial embankment consisted of a rectangular embankment, 100 m by 60 m at the base, with maximum 1 on 3 batters, which was located wholly within the limits of the final roadway embankments. The trial and final embankments had a common batter under which the instrumentation for stability and settlement monitoring was concentrated on three cross-sections. The instrumentation included hydraulic and pneumatic piezometers, horizontal profile settlement gauges under the embankment at ground surface, pneumatic settlement cells within the silty clay, vertical borehole inclinometers, settlement plates, heave stakes and survey stations. The instruments were selected with a view to portability of readout equipment and minimising obstructions to construction activities, and were mainly commercially available items from the United Kingdom. Monitoring positions for one cross-section are shown in Figure 1. The instrumentation and monitoring for the trial construction constituted a net additional cost to the project of approximately A$51,000.

The trial embankment was constructed during August-November, 1977. After raising to an initial 4.1 m height in 4 weeks, construction was halted because of instability indications, discussed later. A further 0.4 m was added after a 6 week break to allow some dissipation of pore pressure. Complete dissipation of excess pore pressures had occurred by November, 1978, i.e. approximately 12 months after the end of construction.

4 TRIAL EMBANKMENT STABILITY

The trial embankment instrumentation was designed to allow stability monitoring of construction on a daily basis, as well as to provide data for the detailed stability analysis of the roadway embankments. As the embankment was to be

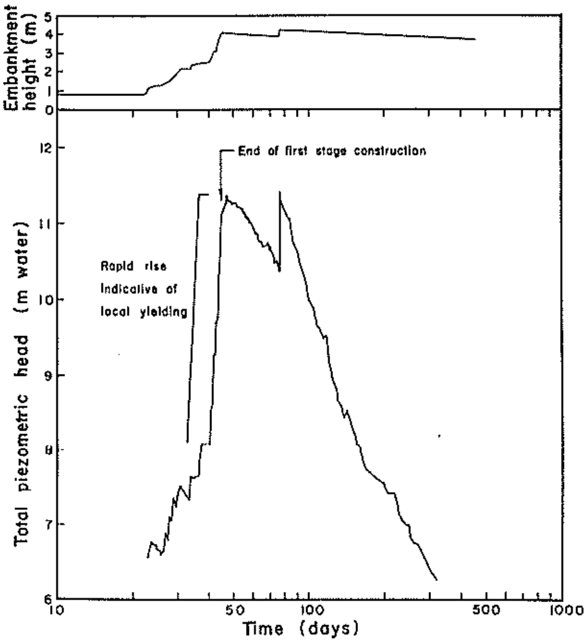


Figure 2 Example trial embankment piezometer response

incorporated within the final works, prevention of failure was of paramount importance. However, no specific failure criteria were defined for the trial. Reliance was placed on daily computer reduction, plotting and interpretation of a large volume of data covering pore pressures, inclinometer deflections, and vertical and horizontal surface survey in the vicinity of the toe, to detect significant trends.

An example of piezometric response under the centre of the embankment, related to loading history, is shown in Figure 2. All piezometers under the embankment showed a marked increase in rate of pore pressure rise above an embankment height of 2.5 m, indicative of local yielding occurring within the silty clay. Pore pressures continued to rise during overnight breaks, and after construction was halted. The change in rate is also apparent in Figure 3, relating excess pore pressure to embankment height for piezometers under the centre of the embankment, and under the batter. Pore pressure changes within the pre-failure "elastic" region tend to be masked by concurrent dissipation. Within the plastic region, pore pressure changes are related to the major principal stress (Hoeg et al, 1969; D'Appolonia et al, 1971; Symons, 1976) by

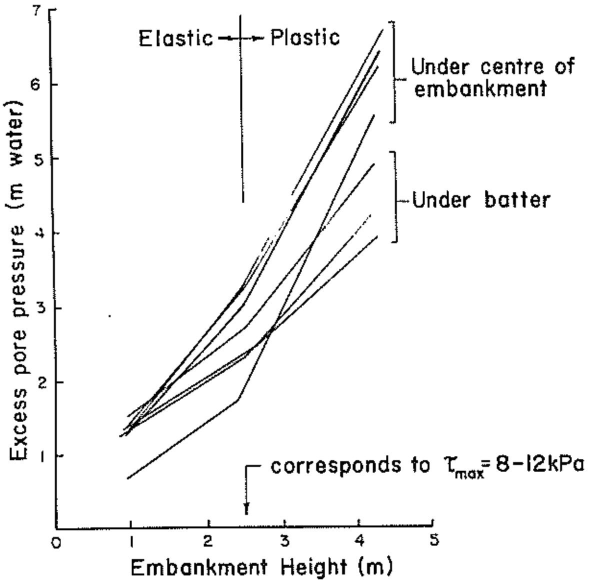


Figure 3 Excess pore pressure response with increasing embankment height

$$\Delta u = \Delta\sigma_1 \qquad (1)$$

The observed relationships are, under the centre of the embankment

$$\Delta u = (0.8 \text{ to } 1.0) \quad \Delta\sigma_1 \qquad (2)$$

and under the batter (where the effects of dissipation are greater)

$$\Delta u = (0.6 \text{ to } 0.9) \quad \Delta\sigma_1 \qquad (3)$$

Under the 2.5 m high embankment, the maximum shear stresses lie approximately in the range 8 to 12 kPa, which is within the observed range of insitu undrained shear strengths.

Stability analyses based on peak undrained shear strengths indicated on allowable initial height of 4.4 m for a factor of safety of 1.5. The first stage construction was halted at 4.1 m height, as a result of the observed pore pressure behaviour. At this stage, effective stress analyses using the measured excess pore pressures gave safety factors in the range 1.1 to 1.4. Shallow seated failure circles, within 2.5 m below ground surface, were indicated. This was supported by horizontal deformation measurements, showing maximum deformation occurring approximately at ground surface, and major shear deformation within a zone to 2.5 to 3.0 m below ground surface.

Other methods of instability detection, by interpretation of deformation measurements, were attempted and shown to be inconclusive. Interpretation of inclinometer data by the method of Wilkes (1974) showed that rates of change of maximum horizontal deformation with increasing embankment height appeared to decrease (rather than an expected increase) for the higher fill heights. A non-uniform rate of filling has affected the interpretation method, as there was evidence of a time lag between placing a fill layer, and detecting consequent horizontal deformation. Surface survey measurements in the vicinity of the toe were also inconclusive indicators, as any significant movements were completely masked by tidal movements.

5 ROADWAY EMBANKMENT STABILITY CONTROL

5.1 Monitoring Details

Roadway embankment construction above 3.5 m height was controlled using a stability monitoring diagram (SMD), prepared using effective stress analysis (with circular failure arcs) of stability models based on the trial embankment strength and excess pore pressure observations. Monitoring was performed at 10 locations under the embankment batters. At each location, measurements consisted of embankment height, pore

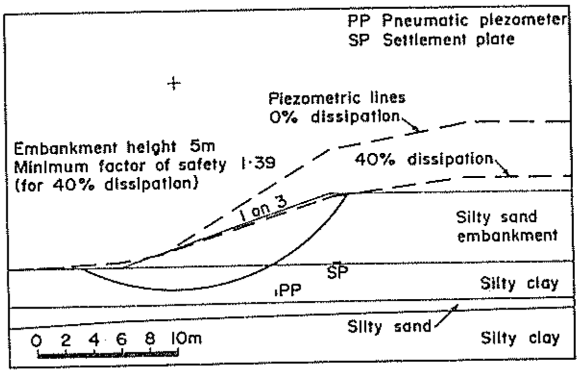


Figure 4 Typical monitoring section used for SMD preparation

pressure (using only 1 suitably located pneumatic piezometer), and settlement of ground surface above the piezometer. A typical arrangement is shown in Figure 4. The shallow nature of predicted failures, and the uniformity in depth of the upper silty clay layer, allowed the use of one SMD, shown in Figure 5, to cover all positions.

5.2 Construction of Stability Monitoring Diagram

The SMD relates embankment height, measured excess pore pressure at a point, and factor of safety. The essential points in derivation of the relationships for different embankment heights are as follows.

(a) The theoretical lateral distribution of undrained excess pore pressure was obtained for the mid-depth of the upper silty clay layer, using the classical Skempton theory with pore pressure parameters A = 0.7, B = 1.0, obtained from laboratory tests. This tended to over estimate the observed lateral distribution under the batters, a feature noted by Symons (1976). The observed undrained excess pore pressures were approximately constant with depth under the trial embankment.

(b) A "zero dissipation" piezometric line was obtained by subjectively adjusting the theoretical distribution line to account for lower excess pressure under the batter region.

(c) Piezometric lines representing other degrees of dissipation were obtained largely by proportion, but modified to account for observed lower rates of dissipation under the centre of the bank.

(d) Minimum factors of safety for combinations of embankment height and degree of dissipation were obtained and used to construct Diagram 2 of the SMD.

(e) The measured excess pore pressure at the piezometer was assumed to be equal to the single ordinate of the excess head piezometric line above the piezometer tip. The measured excess pore pressure was related to degree of dissipation and embankment height by constructing Diagram 1 of the SMD.

An allowable minimum factor of safety of 1.5 from the SMD was adopted for the following reasons.

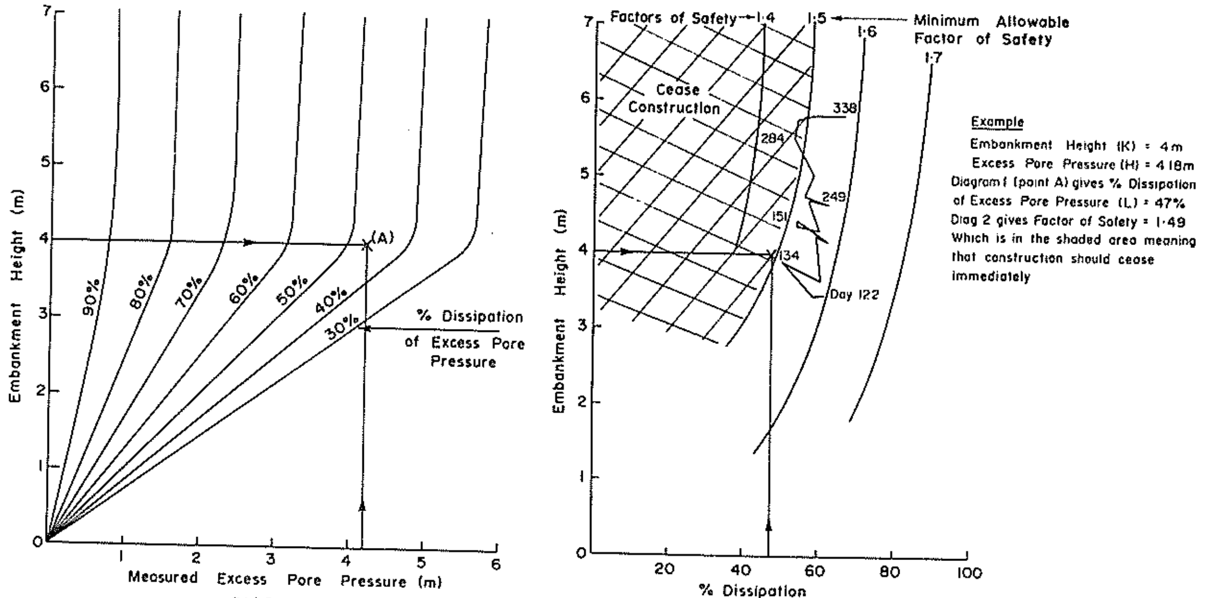
(i) Factors of safety in the range 1.1 to 1.4 were obtained for the trial embankment, using the same calibrated stability model. Allowing the same degree of local yielding over the whole site was considered to be too high a risk.

(ii) Limited analyses with non-circular failure surfaces indicated factors of safety approximately 0.2 lower than those in the circular analysis. Shallow non-circular failures were considered more likely, but more difficult to analyse for the SMD development.

(iii) The silty clay was highly sensitive.

5.3 Observed Stability Behaviour

Factors of safety at all positions were in the range 1.5 to 2.0 throughout construction and no failures occurred. At most locations dissipation was relatively rapid, and only at one location did the stability control impede construction progress. At this location the SMD factor of safety was



DIAG 1

DIAG 2

Example

Embankment Height (K) = 4m
Excess Pore Pressure (H) = 4 18m
Diagram 1 (point A) gives % Dissipation of Excess Pore Pressure (L) = 47%
Diag 2 gives Factor of Safety = 1·49
Which is in the shaded area meaning that construction should cease immediately

Figure 5 Stability Monitoring Diagram

consistently between 1.5 and 1.6 through most of the loading history (see Figure 5) and dissipation was slow. Detailed check calculations at Day 284, incorporating additional data from a second piezometer close to the monitored cross-section, showed a minimum factor of safety in the range of 1.47 to 1.74, depending on the assumed pore pressure distribution within a feasible range. The corresponding SMD result was 1.48 and was therefore conservative.

Major factors influencing the success of the adopted control method are the assumptions concerning excess head piezometric lines, with respect to lateral and depth pore pressure distributions, and the ability of a single piezometer to reliably characterise the whole excess pore pressure distribution. Local drainage effects at the piezometer tip would also have an important influence on results. For this reason, locations of piezometer tips free of the influence of sandy lenses were selected by use of static friction cone penetrometer soundings at each piezometer position.

While the control method has been successful in this case, universal application must be considered with caution, because of the abovementioned factors.

6 TRIAL EMBANKMENT SETTLEMENT BACK-ANALYSIS

6.1 The Settlement Analysis Model

All settlement analyses were to be carried out using a computer program based on the Terzaghi one-dimensional consolidation theory, in which variation in the consolidation parameters (m_v and C_v) with changes in effective stress are taken into account. Thus voids ratio vs effective stress p' (and therefore m_v vs p') and C_v vs p' relationships form part of the input data, together with soil profile and loading details. The aim of the trial embankment back-analysis was, therefore, to obtain "insitu" consolidation parameter relationships which, when used with the one-dimensional consolidation program, constituted a calibrated settlement model. This could then be applied to all parts of the roadway embankments, with differing soil depth profiles and differing embankment loading magnitudes and histories.

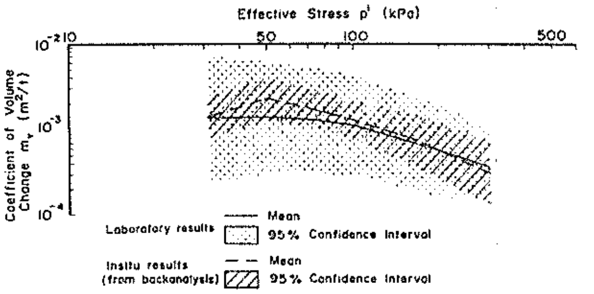
6.2 Consolidation Parameter Modelling

Back analysis of the trial embankment settlement behaviour was carried out using the measured time/settlement data for five profile gauge positions located within the embankment shoulders on one cross section, shown in Figure 1. Using the measured data, it was possible to model soil consolidation characteristics at each position (using the known soil profile and load history at the position) to give similar time/settlement behaviour. The aim was to obtain statistical mean consolidation parameter relationships representative of the five PG positions. These are here referred to as the insitu consolidation parameters, to be distinguished from the laboratory consolidation parameters.

The m_v vs p' relationship determines the predicted magnitude of settlement, while the C_v vs p' relationship determines the predicted time performance. In the back analysis for each PG position, assumed relationships were adjusted, firstly to achieve equality between predicted and measured final settlement, and secondly to achieve, as closely as possible, agreement throughout the time/settlement history. Equality in the final settlement was considered reasonable as primary consolidation was effectively completed (i.e. better than 95% dissipation of excess pore pressure). No allowance was made for possible secondary consolidation occurring in conjunction with primary consolidation.

It was found that, while different m_v vs p' relationships were obtained for each PG position, the same C_v vs p' relationship suitably predicted the time performance at all positions. The five m_v vs p' relationships were statistically reduced, using the log-normal distribution model, to yield an insitu mean relationship, which is shown in Figure 6, together with the 95% confidence interval. The adopted insitu C_v vs p' relationship is also shown. These relationships are compared with laboratory determined data.

(a) m_v vs p' relationship



(b) C_v vs p' relationship

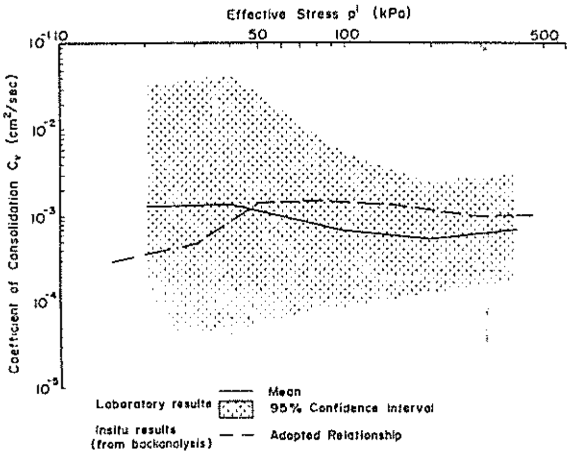


Figure 6 Consolidation parameter relationships

6.3 Comparison of Laboratory and Insitu Data

The confidence intervals obtained for the laboratory data are wide, particularly so in the case of the C_v relationship. This largely unavoidable factor is one of the principal sources of inaccuracy in predictions of settlement time and magnitude, when based on laboratory data.

Comparison of the laboratory and insitu m_v vs p' relationships in Figure 6 shows that the insitu mean values are slightly higher than the corresponding laboratory mean values, and that the 95% confidence interval is considerably narrower for the insitu results.

Both features of the insitu property result from the elimination of the sample disturbance factor present in the laboratory results, and the effect of inbuilt "vertical averaging" present in the insitu results, which are obtained from a surface settlement measurement over a considerable depth of compressible soil whose compressibility varies from point to point with depth.

The adopted insitu C_v vs p' relationship shown in Figure 6 is considerably higher than the laboratory mean relationship over the effective stress range (i.e. greater than 50 kPa) applicable to most of the loaded foundation. The laboratory C_v results are very sensitive to sample disturbance (causing reduction in apparent permeability) and are usually not representative of the apparent macro-permeability of the non-homogeneous soil mass. For this reason, settlement predictions based on laboratory C_v values commonly seriously over-estimate consolidation times.

Use of the insitu mean parameter relationships in an after-the-event prediction (Type C1, Lambe, 1973) of the behaviour of the PG positions results in predictive errors in final settlement, when compared with measurements, in the range 3 to 10% (mean 7%) for points between the embankment shoulders. The error becomes considerably larger for points under the batters, where the measured settlement was considerably less than the prediction. This is indicative of the limitations in the use of the one-dimensional consolidation theory under the batter regions of an embankment. However, generally, the settlements of consequence under a road embankment are those under the pavement, except, perhaps, when considering the deformed profile of a cross drainage pipe.

7 ROADWAY EMBANKMENT SETTLEMENT CONTROL

Settlement control was necessary to the extent that all significant primary consolidation was required to be complete by the end of 1979, so that paving and other finishing works could proceed early in 1980. There was thus a total available consolidation period, including a 6 month construction period, of 15 months. For comparison, the trial embankment primary consolidation was complete in 15 months, including a 3 month construction period.

Settlement predictions, using the calibrated analysis model, were made for all proposed roadway embankment monitoring positions, established for stability control. The insitu mean consolidation parameter relationships were used in the models. The calculations established that embankment surcharges of 1.0 m and 1.5 m (for the worst case) were required to achieve the timing objectives.

The measured behaviour has shown that the predictions have tended to overestimate both settlement rate and magnitude - i.e. the actual settlements have been smaller, and have occurred more slowly. An example of measured settlement/time curves, compared with the predictions, is shown in Figure 7. Settlement is not complete at the end of the measurements shown. In spite of the slower rate, the timing and settlement objectives will have been satisfied, and in some parts of the embankments, an earlier start on the finishing works has been possible. The surcharge requirement has been proven.

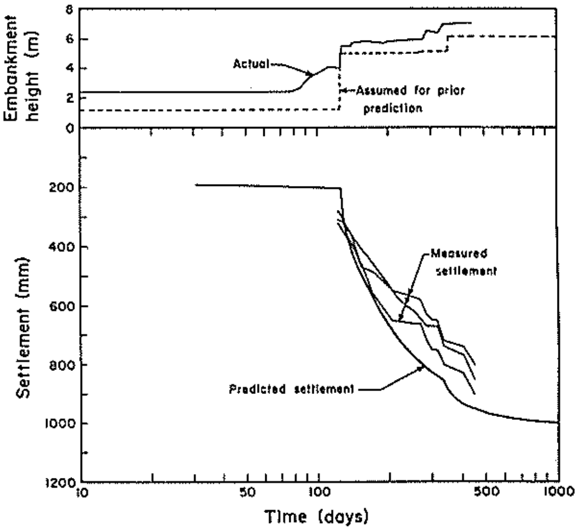


Figure 7 Example settlement behaviour of roadway embankment

Major sources of error having an influence on the accuracy of the settlement predictions should be noted. These have firstly influenced the predictive processes in the trial behaviour back-analysis, and some then have had a further, potentially greater influence on the roadway embankment predictions.

(i) Soil profiles are based on interpolations between borehole and penetrometer data, so that the adoption of layer arrangements, thicknesses and most particularly, drainage characteristics usually required important simplifying assumptions.

(ii) The consolidation parameters, which exhibit a proven natural variability, are approximated by the insitu mean parameters in the analysis. The true variability is offset somewhat by the "vertical averaging" which results from the back-analysis process.

(iii) The actual, irregular loading history is not entirely consistent with the prior analysis assumptions of a series of instantaneously applied load increments at different times. This does not greatly affect the final settlement outcome, but is a partial explanation for discrepancies between prediction and measurement in the early consolidation stages.

(iv) The analysis method assumes one-dimensional vertical compression and drainage only, whereas these phenomena exhibit significant two-dimensional components under the outer regions of embankment.

(v) The stress and pore pressure distribution from the embankment and within the deforming foundation can only be approximated by the elastic stress distribution necessarily assumed in the analyses.

8 CONCLUSIONS

A large-scale instrumented trial construction project has been used to facilitate reliable prediction of stability and settlement behaviour of high road embankments built on a soft soil foundation. The data obtained from the trial has been used in conjunction with the classical methods for analysis of stability and settlement to produce, by back-analysis, calibrated stability and settlement models. Reliable behaviour predictions for the roadway embankments were obtained by the use of these models, giving effective means of control of both stability and settlement during the construction period.

A condition of local contained yield in the very soft, highly sensitive silty clay was detected under the trial embankment at a height approximately 0.6 of the final height at which construction was terminated on stability grounds. This condition was readily identified on graphical plots of excess pore pressure against embankment height, for virtually all piezometers located under the embankment. Attempts at alternative instability detection methods, based on inclinometer and surface survey measurements, gave inconclusive results.

The first onset of local yield was not taken as sufficient loss of stability to halt construction. The trial construction was terminated when effective stress stability analyses, incorporating the measured excess pore pressures, yielded minimum factors of safety in the range 1.1 to 1.4.

Stability control was applied to the roadway embankments using a Stability Monitoring Diagram (developed using the calibrated stability model) and very simple instrumentation installed at 10 monitoring sections within the embankments. At each section, the pore pressure measured by only one piezometer was used together with settlement and embankment height measurements, to establish the degree of dissipation, and predict factor of safety using a simple rapid pro forma and graphical procedure on a daily basis. The minimum allowable factor of safety below which construction should be terminated was set at 1.5.

The success of the control method depended on analysis assumptions of the excess pore pressure distribution, and on the ability of the piezometer to adequately characterise the whole pore pressure distribution under the embankment. All sections of the embankments were successfully constructed using this method, without the need of "safe" but expensive toe berms, which were considered to be necessary in the early stages of the project investigation. The method of control, therefore, was apparently successful in this case, with an important benefit being the timely availability to construction personnel of a reliable result. However, the simplicity of the method can not be considered universally applicable at this stage.

The settlement model showed that for the roadway embankment some control of settlement rate was necessary to achieve the project timing constraints. Embankment surcharges of 1 m and 1.5 m height were employed. The observed behaviour has shown no major unexpected departure from prediction at all monitoring positions. However, some tendency is evident that smaller settlements are occurring slightly slower than the predictions indicated. The method of settlement control (through more reliable prediction) has successfully met the timing requirements, to a satisfactory level of accuracy.

The benefits to the construction project, in terms of achieving full embankment heights without stabilisation, and in terms of satisfying critical path project timing, have fully justified the net cost of the trial.

9 ACKNOWLEDGEMENT

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