Fourth Australia - New Zealand
Conference on Geomechanics,
Perth, 14-18 May 1984

The Use of Settlement Monitoring for Roadworks Construction Control

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SUMMARY Consolidation of road embankments constructed on compressible foundations often will control aspects of the construction programming, to ensure that the safety and serviceability of associated structures is maintained, and to ensure completion of the facility within time constraints. The paper describes an approach to settlement management, which relies heavily on settlement monitoring and interpretation during construction. Included in this approach are the determination before construction of expected settlement behaviour and desired behaviour criteria, measurement and rapid interpretation of behaviour as it is occurring, and implementation, if necessary, of action during construction to ensure that the criteria are satisfied. The successful application of this "observation and action" approach, (the Observational Method) is described in the form of two case histories, in which vertical wick drainage and embankment surcharging have been employed as settlement control measures. In addition to forming an important link in the management cycle, settlement monitoring has provided valuable feedback to benefit future construction of a similar nature.

1. INTRODUCTION

The development of major highways in Queensland frequently leads to elevated roadways, on embankment or structure, being located in areas with weak, compressible foundation conditions. Construction in such locations, however, within the flood plains of present streams and buried estuarine deposits along the Queensland coastline, presents the geotechnical problems of stability, settlement, and soil interaction with structures.

The difficulties in reliably predicting the stability and settlement behaviour of soft soil foundations are well known in geotechnical engineering practice. In view of the serious implications of such behaviour with regard to design and construction of projects incorporating road embankments of soft foundations, the Queensland Main Roads Department (QMRD) has adopted procedures for managing the geotechnical problems, and these incorporate

- detailed site investigations, subject to practical time and resource constraints.
- behaviour analyses before and during construction, which provide relevant data for design and construction decisions.
- field monitoring of behaviour, to facilitate analyses and to compare with predicted behaviour.

The dual considerations of stability and settlement behaviour are usually present to varying degrees, with, in some cases, control of stability the major factor (Robertson & Reeves, 1980). This paper addresses cases in which management of settlement and settlement interaction with structural components have predominated.

2. THE DEMANDS FOR SETTLEMENT MANAGEMENT

The principal project management demands in consolidation-oriented problems arise from conflicts in time and deformation compatibilities. The time requirements of the consolidation process must be balanced against the time constraints of the construction programme. The impact of deformations on design requirements, construction sequence, and the safety and serviceability of the facility must be reliably assessed.

The procedures outlined above, and described in this paper, have evolved to provide the necessary information on which effective management of consolidation issues has been based. They include activities such as data collection, analysis and interpretation, and behaviour prediction, but equally important with these activities is the selection of realistic design criteria which define the framework of acceptable behaviour.

The types of consolidation-oriented problems which can arise, and the procedures used for predicting settlement behaviour, and for managing embankment settlement within the construction programming context, can best be exemplified by presenting some details of two case histories. While stability and settlement issues are often closely linked in these situations, in each of the case histories discussed, stability, while monitored, was not a controlling influence on the construction sequence. The procedures illustrated have been successfully used on several major projects in recent years, and are examples of application of the Observational Method, in which settlement monitoring plays an essential and integral part of project management.

3. STRATIGRAPHY

The two selected case histories are located in the Gold Coast region on south east Queensland, within the buried estuaries of drainage systems inland from the present coastal dune deposits. The soft alluvium stratigraphy is essentially similar in each case, and is summarised as typical soil profiles in Figure 1. These profiles are generally indicative of conditions in contemporaneous estuarine deposits through much of coastal Queensland, with the exception that a commonly observed near-surface stratum of soft silty clay was not present in the cases described.

Deep (6-30 m) estuarine sediments (of approximately Holocene age) are present, consisting predominantly of alternating deposits of clay/silty clay and fine to medium sandy materials. Clayey strata are usually greater than 2 m thick, and extend up to 10 m thick. However in the latter case they contain thin, often discontinuous sandy lenses. Sandy strata are found at the base of the alluvial sequence, and interspersed within it, in layers varying from less than 100 mm thick to 12 m thick. The clayey materials below 2-4 m below present ground surface are normally to very lightly overconsolidated, even when overlain by considerable thicknesses of sand.

4. SITE INVESTIGATION

In both cases, the site investigation has incorporated the following relevant field and laboratory activities.

- static friction cone penetrometer sounding, to provide stratigraphical data (specifically identification of compressible clays and sandy drainage layers, and to provide undrained strength data).
- undisturbed sampling, using standard class thin wall tube sampling 50 and 100 mm diameter, and limited thin wall piston sampling.
- laboratory oedometer testing, vertical and radial drainage.
- laboratory strength testing, mainly UU and CU triaxial testing.

Consolidation test data has been treated statistically assuming log-normal distributions for the m_v and C_v parameters, to yield mean properties and 90 or 95% confidence intervals on the population mean.

5. SETTLEMENT ANALYSIS

Two general approaches to settlement analysis, viz. pre-construction, and pre-completion analysis, have been used in the QMRD.

5.1 Pre-construction Analysis

The general aim in this analysis is to produce predictions of settlement behaviour for design and project planning purposes, accommodating whatever project attributes and constraints that can be defined often well in advance of actual construction.

It is normally a conventional theoretical analysis of a suitably quantified geomechanical model, utilizing elastic theory, and one dimensional and radial drainage consolidation theory. In all cases, simplifying assumptions must be made in the model, regarding the degree and nature of drainage, the statistical selection rules for critical parameters, and the likely loading history. Normally, in view of the assumptions made and required accuracy, there is little justification for two- or three-dimensional analysis complexities.

5.2 Pre-completion Analysis

The general aim of this analysis is the interpretation of actual settlement behaviour before completion of consolidation, to predict future outcomes such as the following.

- the magnitude and time of final settlement
- the time beyond which a realistic in-service settlement behaviour criterion will be satisfied.
- the need for additional consolidation acceleration, such as surcharging or vertical drains.
- the time for removal of surcharge
- the effects of rebound after overconsolidation.

Two different alternative types of pre-completion analysis are employed. Back-analysis of measured settlement data is carried out, using consolidation theory within an adopted geomechanical model. Normally this is an iterative process, and attempts to calibrate the model (by adjusting soil parameters and drainage conditions), so that the model will adequately predict future behaviour. However, for the calibration to be reliable, this approach should be used in cases where the consolidation process being back-analysed is approaching completion (i.e. U=80-100%),

As an alternative to the back-analysis approach, a graphical interpretation of measured settlement/ time data has also been used. The Asaoka graphical method (Asaoka & Suzuki, 1979, Magnan & Deroy, 1980, McAnally, 1982) has provided good approximations of behaviour in most cases, and within its recognised limitations, has contributed some powerful advantages, which are illustrated in the presented case histories.

In this method, a series of settlement values is taken at a fixed time interval, from the observed data for a monitoring point, and each value is plotted as an ordinate against the preceding value as an abscissa. The result is a linear trend.

A principal attraction is that, independently of any original settlement datum, and of any assumed geomechanical model, the method has been found to quickly and simply provide future predictions of settlement and time directly from settlement/time observations. In addition, for certain limited consolidation model types, the method has enabled back-calculation of soil properties. Theoretically, the method should provide reasonable estimates of future behaviour when consolidation is 30-60% complete. It may therefore be applied at an earlier stage than the back analysis method.

The back-analysis approach has the advantage of providing a greater understanding of the theoretical implications in the process being modelled, providing valuable feedback for application in the preconstruction analysis of future projects. However back-analysis has a greater chance of yielding misleading predictions than the Asaoka method, and is considerably more complex and time consuming to perform. In order for the results of a pre-completion analysis to influence construction management, they must be obtained quickly.

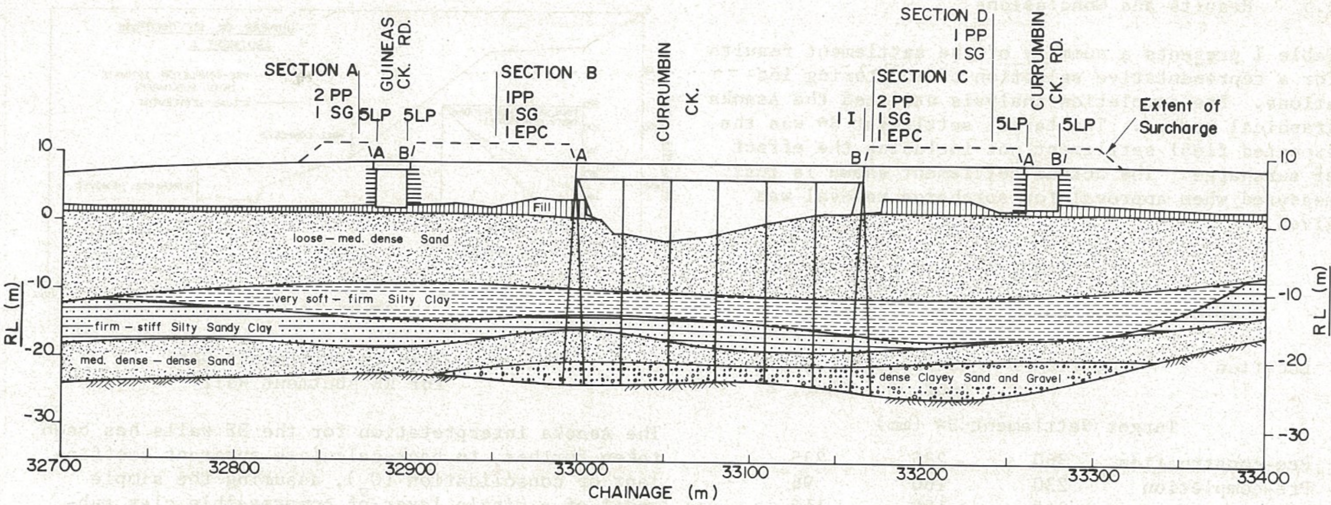
6. CASE HISTORY A - CURRUMBIN CREEK VICINITY

6.1 Site Description

At this site within Gold Coast City in south east Queensland, the Pacific Highway was to be constructed as an elevated crossing of Currumbin Creek, together with grade-separated crossings of major feeder roads serving the Currumbin Creek valley on each side of the waterway. The longitudinal profile of embankments and structures, together with subsurface details, is shown in Figure 1(a). A particular feature of the site is the nature of the alluvial stratigraphy, in which the compressible clay is particularly deep within the sequence.

6.2 The Design Problem

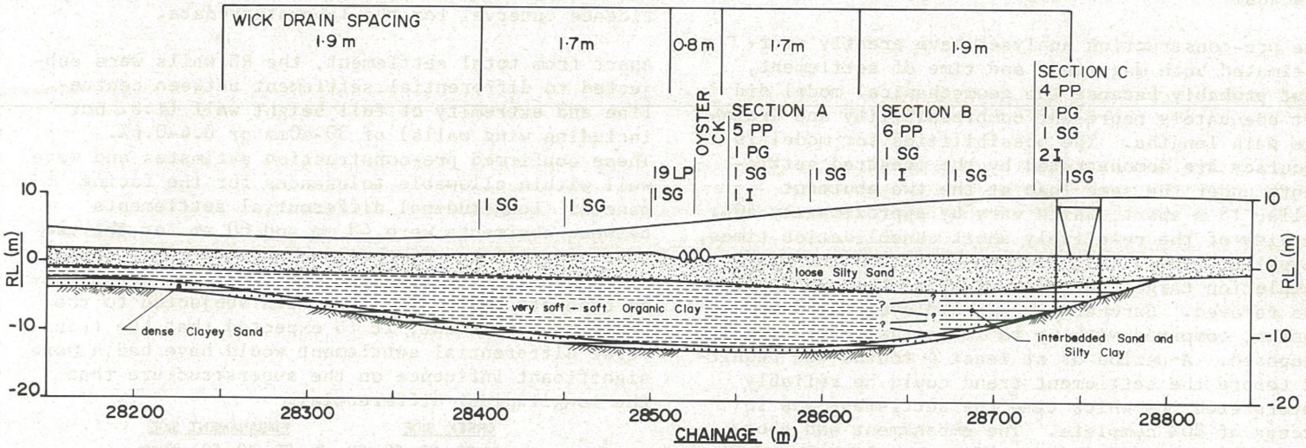
The major geotechnical problems at the site involved



(a) CASE HISTORY A

INSTRUMENTATION LEGEND

PP	Pneumatic Piezometer	PG	Horizontal Profile Settlement Gauge	EPC	Pneumatic Earth Pressure Cell
SG	Water-overflow Settlement Gauge	LP	Survey Level Point	I	Vertical Inclinometer Tube



(b) CASE HISTORY B

Figure 1 Longitudinal Sections

resolving conflicting embankment and bridge construction requirements within a limited construction period. The substructure design for the abutments of the creek bridge, with long, slender driven concrete piles raked under the approach embankment (approximately 7 m high), dictated that all consolidation under the embankment must be complete before abutment pile driving. As the flanking road overpass structures were to be simply-supported single span structures, it was decided to support the superstructures of both overpasses on top of reinforced earth (RE) abutment walls. This provided a particularly economical structural solution, but again dictated that the abutment consolidation should be essentially complete before placing the superstructures.

6.3 Design Criteria

At the time of pre-construction analysis, criteria for settlement behaviour were selected as follows.

(i) Settlement of the proposed embankments should occur within a consolidation period not exceeding 18 months. Subsequent programming changes reduced this period to approximately 9-12 months.

(ii) In-service settlement at structure abutments should not exceed 50 mm.

6.4 The Design Solution

Pre-construction analysis indicated total (i.e. immediate + consolidation) settlements generally in the range 300-400 mm under embankment areas, to occur generally over 3-5 years. Lesser total settlements of 200-250 mm (resulting from the load discontinuity at the vertical wall) were expected at the RE abutment walls. Consolidation acceleration, by means of an additional 3 m of embankment surcharge applied for up to 18 months, was adopted on the basis of the pre-construction assessment.

Instrumentation for settlement monitoring, consisting primarily of pneumatic piezometers and water-overflow settlement gauges, was installed at four centreline locations shown in Figure 1(a). The instrumentation was directed solely towards settlement monitoring as there was no significant likelihood of instability. Vertical inclinometer tubing was also installed at Section C (Abutment B of the creek structure), to observe lateral movements in the vicinity of where the abutment piles would be driven.

6.5 Results and Conclusions

Table I presents a summary of the settlement results for a representative selection of monitoring locations. Pre-completion analysis utilized the Asaoka graphical method. The target settlement S* was the expected final settlement not including the effect of surcharge. The actual settlement shown is that measured when approval for surcharge removal was given.

TABLE I

SUMMARY OF SETTLEMENT - CASE HISTORY A

Location	Section D	Guineas Ck. Rd. Abut. A	Abut. B
Target Settlement S* (mm)			
Pre-construction	360	235	235
Pre-completion	230	160	96
Actual	245	180	116
Time to achieve S* (days)			
Pre-construction	1400	1300	1300
Pre-completion	150	93	102
Actual	155	96	98

The pre-construction analyses have greatly overestimated both magnitude and time of settlement, most probably because the geomechanical model did not adequately represent compressibility and drainage path lengths. The possibilities for model inaccuracy are demonstrated by the measured settlements under the same load at the two abutment walls, 18 m apart, which vary by approximately 40%. In view of the relatively short consolidation times, actual settlement was allowed to exceed the pre-completion target settlement before the surcharge was removed. Surcharging was employed for 5-6 months, compared with up to 18 months originally proposed. A period of at least 4 months was required before the settlement trend could be reliably interpreted, by which time the settlement was in excess of 80% complete. The embankment and abutment areas therefore appeared to be slightly overconsolidated by the surcharge, a desirable condition to minimise the possibility of further long term movements. The estimated rebound after removal of the surcharge was approximately 5 mm, which could not be detected conclusively by the measurement techniques employed.

The observations have shown that surcharging has had a favourable effect on settlement behaviour throughout the length treated. However, at the discontinuities in surcharge at each abutment, the required effect has not been fully achieved, because of the geometric limitations on extent of surcharge. In 12 months after removal of surcharge, 10-30 mm of additional settlement has been recorded at locations where abutment piles have been installed, and at RE abutment walls.

An example of the Asaoka graphical interpretation for a RE abutment wall is given in Figure 2. As the method is applicable to any type of consolidation model, with any type of imposed stress distribution, it is well suited to interpretation of the settlement of a flexible retaining wall carrying a battered surcharge. The discrepancy between pre-construction estimates and actual behaviour would have made a back-analysis approach difficult and time-consuming, whereas the Asaoka method allowed a timely decision on surcharge removal, to the overall benefit of the project.

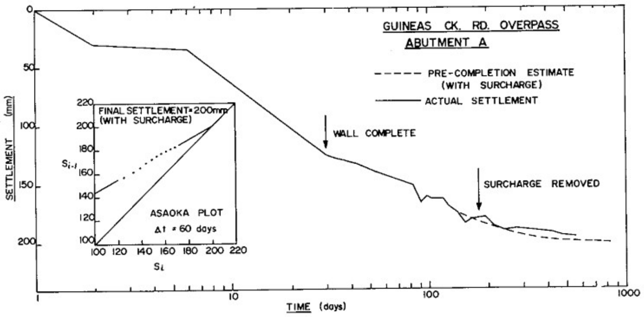


Figure 2 Time vs Settlement Data for RE Abutment Wall

The Asaoka interpretation for the RE walls has been taken further, to back-calculate apparent coefficient of consolidation (C_v), assuming the simple model of a single layer of compressible clay subjected to two-way vertical drainage (Magnan & Deroy, 1980). The calculations yielded field C_v values 10-50 times higher than laboratory C_v values at similar effective stress levels. The field C_v values approximate the upper limit of the 90% confidence interval for the laboratory data.

Apart from total settlement, the RE walls were subjected to differential settlement between centreline and extremity of full height wall (i.e. not including wing walls) of 30-60mm or 0.4-0.6%. These confirmed pre-construction estimates and were well within allowable tolerances for the facing panels. Longitudinal differential settlements between abutments were 40 mm and 60 mm for the two road overpasses. If the superstructures of these overpasses, being transversely tensioned prestressed concrete deck units, had been subjected to the abutment settlement, it is expected that the transverse differential settlement would have had a more significant influence on the superstructure than the longitudinal differential.

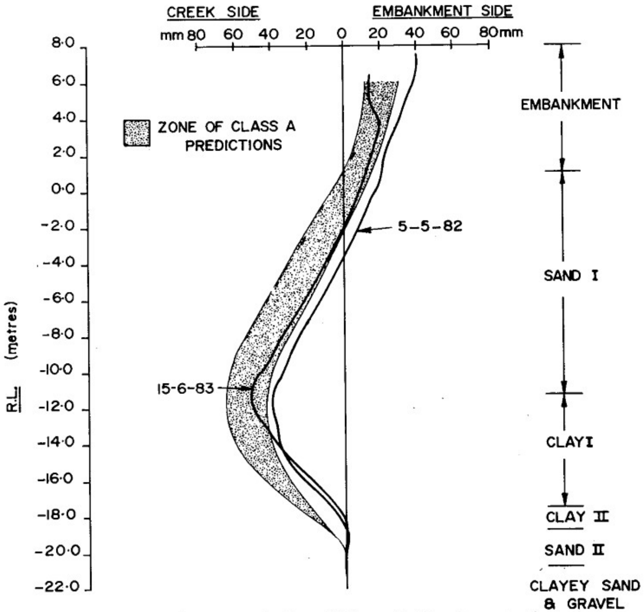


Figure 3 Depth Profile of Horizontal Displacement at Currumbin Creek Bridge, Abutment B.

The nature of loading and deformation of the raked pile group at the creek bridge abutments was investigated by two-dimensional finite element consolidation analysis during the design phase. While the analysis model was limited by insufficient soil property data, and by the assumption of complete adhesion of soil to the piles, the results showed that if the pile group had been subjected to the full embankment settlement, then pile integrity would have been destroyed. The measured profile of lateral displacements at Abutment B of the creek bridge is shown in Figure 3. This confirms completely the form of the Class A predicted displacements throughout the length of the pile. Throughout the consolidation history, the maximum horizontal displacement in Clay 1 remained at approximately 20% of the vertical consolidation.

7. CASE HISTORY B - OYSTER CREEK VICINITY

7.1 Site Description

This site is also situated on the Pacific Highway in the West Burleigh area of the Gold Coast, which formed a similar, and adjacent, estuarine depositional environment to that of Case A. The highway at this location is on embankment 1-9 m high, forming the approaches to grade separation and creek structures. Subsurface materials (Fig. 1(b)) include a loose, fine silty sand 3-5 m thick overlying very soft to soft organic clay to a maximum depth of 15-16 m. Thin sandy interbeds were identified irregularly and infrequently within the clay, such that continuity for drainage could not be assumed.

7.2 The Design Problem

The geotechnical problems identified at this site mainly involved the effects of differential settlements.

- Large differences in expected settlement along the alignment would arise from variation in subsurface conditions and embankment load.
- Transverse differential settlement would affect the serviceability and integrity of cross-drainage structures.
- Post-construction differential settlement at the overpass abutments should be avoided.

7.3 Design Criteria

Realistic design criteria, based on job programming requirements, were set initially as follows.

(i) a maximum available consolidation period before pavement finishing of two years.

(ii) a maximum tolerable in-service settlement of 100 mm.

(iii) at the overpass abutment at one end of the section, all settlement to be complete before abutment construction, and within 1 year of embankment construction.

(iv) at a major cross-drainage location, the target settlement to occur within 1 year of embankment construction.

7.4 The Design Solution

Pre-construction settlement analyses indicated expected total settlements of 100-1100 mm, to occur over periods between 2-50 years, depending on location. Consolidation acceleration or control was clearly warranted, and of the available options, vertical drainage was selected on grounds of economics and practicality. A wick drainage system emerged as the successful method from contract tendering. Approximately 125,000 lin. m of wick drains were installed at 1.7 and 1.9 m triangular spacings, and 0.8 m spacings in the cross-drainage area.

The drain spacings were determined assuming the following.

- Isotropic permeability conditions prevailed (a conservative condition from laboratory testing);
- Horizontal sand layers did not contribute to drainage;
- Smear effects were ignored in the light of other uncertainties.

In view of the possibilities for better drainage than assumed (i.e. higher horizontal permeability in the clay, and interconnection of discontinuous sandy layers by vertical drains), the design consolidation period, (which determined the drain spacing) before invoking the in-service settlement criterion was relaxed from 2 years to 5 years, to limit the cost of what was then an untried treatment method by QMRD.

Embankment surcharging was not included within the design, but was considered as a possible supplementary treatment if further acceleration of settlement was required to achieve the job programme target.

Embankment behaviour was monitored at locations indicated in Figure 1(b), by means of settlement measurement by water-overflow gauge at a number of locations, and more intensive instrumentation concentrated on three cross-sections. Stability was expected to be marginal in the higher embankment areas, and was monitored on these cross-sections by pneumatic piezometer networks, and by inclinometer tubing at the embankment toe. Settlement profiles across the embankment width were measured on these sections by a settlement profile gauge, backed up by water-overflow gauges.

7.5 Results and Conclusions

The settlement monitoring, together with pre-completion analysis using the Asaoka graphical method showed, (before completion of consolidation), that the vertical drainage treatment was successful in meeting the initial design criteria. Two years after embankment completion, settlement was essentially complete in 57% of the overall 700 m of embankment subject to settlement, while in a central 300 m long section between 60-100 mm settlement was still to occur, over the following 1-3 years. The pavement was constructed with final grade levels set up to 80 mm high over this section.

Some interesting comparisons can be drawn between pre-construction estimates, and reliable pre-completion estimates for settlement magnitudes. The actual settlement measured at Ch. 28592 (in the area of maximum settlement) is shown in Figure 4, compared with the pre-construction estimate, and the pre-completion estimate, prepared 1 year before the preparation of subgrade for paving. The Asaoka plot, from which the pre-completion future prediction of settlement/time behaviour was derived, is also shown. Reasonably reliable estimates of final settlement could have been obtained from the Asaoka trend line when settlement was 60-70% complete. The pre-completion estimate was actually made when settlement was 67% complete.

Over the central 300 m section of the project, where the organic clay is thickest (6-10m) and where there was the least evidence of sandy interbeds within the

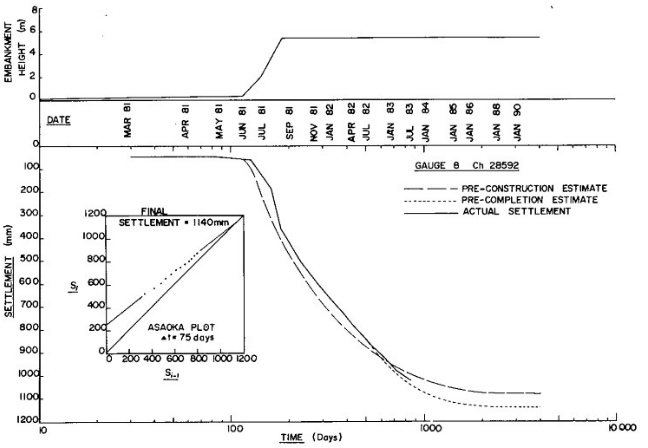


Figure 4 Time vs Settlement Data for Settlement Gauge 8

clay, a very high level (i.e. 0.99) correlation was achieved between pre-construction and pre-completion estimates of settlement. Using the pre-completion estimates (the best available indicator to date of actual final settlement) to normalise the pre-construction estimates a mean normalised settlement prediction of 0.95 was obtained, with CV=7%. Actual settlements of between 400 mm and 1100 mm occurred within this section. However, towards the upchainage end of the project, where significant occurrences of mixed sandy and clayey materials were observed in the site investigation pre-construction analysis over-estimated settlement by between 50 and 400%.

Three cells of 1800 mm diameter pipes to carry the flow of Oyster Creek under the embankment were installed in a hogged profile, with a maximum elevation above design pipe grade of 600 mm (Figure 5). The adopted profile was reduced from the initially recommended profile for the following reasons.

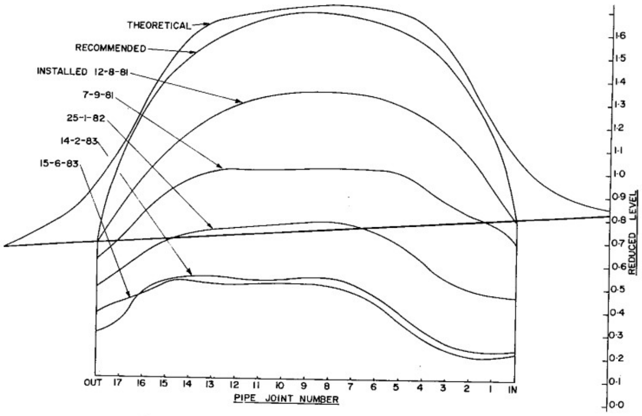


Figure 5 Longitudinal Profiles of 1800 mm diameter Pipe, showing Settlement after Installation

- The need for the pipes to operate during the 1981-82 wet season.

- The acceptability of a possible final 300 mm sag in the pipe profile.

- Reduction of stresses in the pipes.

Several observations can be made from a study of the actual settlement vehaviour of the pipes. The pre-completion estimate of final settlement under centreline is 930mm, which agrees well with the original design estimate of 900 mm. The theoretical settlement profile, with settlement at pipe inlet and outlet approximately one-third of centreline settlement, is shown to be more accurate than the recommended profile. Both ends of the culvert have settled more than originally envisaged, and a hog of approximately 300 mm still remains. The pipe inlet has settled 600 mm below design grade.

The vertical wick drains were placed at a closer spacing under the culvert area, to achieve the target settlement within 12 months. While the drainage system has performed well in meeting the design criterion in the general embankment area, it has not been able to satisfy the more demanding criterion in this instance, and target settlement was not achieved until 20-22 months after embankment construction. This can most probably be ascribed to the unaccounted effects of disturbance caused by a closed mandrel 150 x 75 mm being driven at 800 mm centres.

8. PREDICTIONS AND ACTUAL BEHAVIOUR

In each of the case histories, comparisons have been made between pre-construction settlement estimates, pre-completion estimates, and actual behaviour. The illustrated difficulties in achieving reliability in pre-construction settlement analyses are common experiences in geotechnical engineering practice. A study of these cases, and other projects not described in this paper, has yielded some general conclusions to assist in formulation of more reliable predictions in the future.

In most cases, the settlement magnitude of clay sequences has been reliably predicted using mean compressibility properties from careful laboratory testing. Significant, even major errors have arisen when large proportions of sandy materials are present, particularly when interbedded within the clay strata. Theoretical distinction between immediate and consolidation settlement may be one source of such errors.

As is well known, errors in settlement time prediction result from imperfect definition of drainage and permeability within the geomechanical model. However, the pre-construction estimates of vertical drain system performance have proven very reliable, provided proper attention is paid to soil properties (including anisotropy), and to the effects of smear caused by the installation process.

Back-analysis techniques, using both iteration on the theory-based geomechanical model, and the Asaoka interpretation, have provided measures of apparent field consolidation properties, for comparison with laboratory properties. For both normally- and over-consolidated clays the insitu m_v property has been reasonably estimated by the laboratory mean property. The apparent insitu C_v property for relatively simple consolidation models without vertical drains can be reasonably approximated, in the case of normally consolidated clays, by the upper limit of the 90-95% confidence interval of the laboratory mean property. The field C_v property for over-consolidated clays appears to fall somewhat variably within the confidence interval.

9. SETTLEMENT MANAGEMENT

The general approach to settlement management is summarized as follows.

(i) Obtain sufficiently reliable pre-construction estimates to enable satisfactory stability and settlement behaviour criteria, and design and job programming requirements to be determined.

(ii) Monitor stability and settlement behaviour at critical locations with suitable instrumentation.

(iii) Provide pre-completion estimates and updates, within the time demands of the construction programme.

(iv) If necessary, take action within the construction context to ensure that the behaviour criteria are satisfied.

The successful implementation of this approach has depended upon adopting realistic criteria for acceptable settlement behaviour during and after construction implementing effective monitoring systems, and the availability of effective pre-completion analysis techniques. Criteria, and monitoring systems, are examined further in subsequent sections. The QMRD experiences have exposed the strengths and weaknesses of the two analysis methods used. On balance, the Asaoka method offers clear advantages in terms of ease of application, timeliness in producing a result, and its relatively simple data demands.

10. SETTLEMENT BEHAVIOUR CRITERIA

The adoption of realistic criteria of acceptable behaviour greatly simplifies the settlement management task in construction. The type of criterion to be considered will depend on particular project attributes, such as embankment, structural components and cross-drainage, as discussed below.

- The serviceability of road embankments not adjacent to fixed structure abutments is determined by differential settlement effects, rather than total settlement. However, differential settlement, in terms of a gradient, cannot practically be defined as a criterion, nor monitored through the full extent of a project. The setting of a realistic in-service settlement criterion throughout an embankment section subject to settlement will automatically tend to control differential settlements between different points within the embankment. For example, in Case B, the 100 mm in-service criterion has resulted in a maximum future change in level between any two points on the embankment of 60-100 mm, while the differential movement between any two points, say 50 m apart, should be less than 30 mm, over the life of the road. This compares with differential movements of up to 1 m, which would have occurred over the road life if settlement control had not been implemented.

- When structural elements are involved, criteria must necessarily be tighter. Reinforced earth walls must have both a longitudinal differential settlement limitation, to prevent interaction between facing panels, and a transverse limitation, to prevent adverse effects on mobilisation of stresses in strips. With RE abutment walls, the additional transverse and longitudinal differential effects on the superstructure must be considered. In Case A, maximum differential settlement of 2% was set in each direction, and actual distortion of 0.4-0.6% was achieved.

- Allowable post-installation settlement around abutment piles must be carefully considered in each case. Overconsolidation by means of surcharge before pile driving has been favoured in the cases described. This has the effect of reducing future movements (e.g. secondary consolidation) at the expense of the prospect of minor rebound after piles are installed.

- Where pipe culverts are to be subjected to settlement, flexible pipe joints are necessary, and ample capacity within the allowable joint rotation should be allowed in assessing the capability of particular pipe sizes to accommodate the expected deformation. Expected settlement should be accommodated for the full length of pipe, from inlet to outlet.

11. PRACTICAL INSTRUMENTATION

Successful monitoring systems depend on attention to practical details through all phases of installation, operation and maintenance, result processing and interpretation. Such details are beyond the scope of this paper.

The QMRD experience has been that for stability monitoring, pneumatic piezometer grids and inclinometer tubes, together with frequent embankment height data provide the most useful data. Piezometric measurements have been found useful for indicating undrained pore pressure increase, but less suitable for interpretation of consolidation progress. Water-overflow settlement gauges have been found the most suitable for settlement measurement, giving the advantages of simplicity and reliability of operation, and limited hindrance to plant operations while placing embankment. No monitoring activity will be successful without comprehensive survey control throughout the measurement period. This aspect, more than others, tends to require the continuing close co-operation of job management throughout construction.

12. CONCLUSION

The experiences of the QMRD show that management of settlement behaviour of road embankments on compressible foundations can benefit from the application of the Observational Method. Prior predictions of settlement behaviour, even when based on detailed site investigations, often have an unknown degree of reliability. However, confidence in a future outcome is improved by measuring settlement behaviour during construction, and rapidly interpreting the observations to obtain a pre-completion prediction of future behaviour. The reliability of the latter predictions benefits construction management and programming. Measurement of the behaviour of full scale construction also provides a valuable feedback, to the benefit of future projects of a similar nature.

As part of settlement management to meet design and construction programming criteria, two methods of settlement control have been used. Embankment surcharging has been effective in reducing consolidation times, and in achieving slight overconsolidation, to control future movements, and vertical wick drain systems have been shown to be highly effective in ensuring that construction criteria are satisfied.

13. ACKNOWLEDGEMENT

The permission of the Commissioner for Main Roads, Queensland to publish this paper is gratefully acknowledged.

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