DESIGN AND CONSTRUCTION OF A RESILIENT MOTORWAY ON DIFFICULT GROUND

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ABSTRACT

Resilience is the ability of assets, networks and systems to anticipate, absorb, adapt to and / or rapidly recover from a disruptive event. Where motorways cross floodplains, a major flood poses the greatest risk of a disruptive event. Deep deposits of soft, compressible materials are often encountered in floodplains making construction of resilient infrastructure difficult and potentially expensive.

In order to optimise the balance between time, cost, resilience and risk the Roads and Maritime Authority of New South Wales (RMS), in conjunction with the Ballina Bypass Alliance, developed a low embankment strategy to minimise the whole of life cost of the Ballina Bypass motorway while allowing the motorway to operate during a 1 in 20 year flood. The low embankments traversed very poor ground conditions and the geotechnical challenge was to estimate performance of the embankments at the design stage, monitor the actual performance of the embankments during construction and to take actions to achieve the strategic goals if required.

This paper presents the low embankment strategy, the associated pavement strategy and discusses the geotechnical elements pivotal to its success.

1. INTRODUCTION

Resilience is the ability of assets, networks and systems to anticipate, absorb, adapt to and / or rapidly recover from a disruptive event. Where motorways cross floodplains, a major flood poses the greatest risk of a disruptive event and historically the Pacific Highway has been closed for periods of time.

Deep deposits of soft, compressible materials are often encountered in floodplains. Embankments constructed on these materials will settle and potentially be unstable. Long term deformations will occur causing embankments to potentially settle below flood level and/or affect pavement performance if action is not taken to improve the ground.

There are several ways of achieving a suitably resilient motorway.

- To eliminate settlement as a constraint a bridge could be built across the floodplain;
- To reduce settlement ground treatment could be provided across the floodplain; or
- Embankments could be built directly on the ground and a whole of life construction and maintenance strategy developed to manage the impacts of settlement during construction and operation.

Deciding which option to adopt required optimisation of conflicting cost, time, performance and risk constraints. For example, a highly resilient solution is to build a bridge across the floodplain but the expense would mean that construction of other sections of the highway could not be afforded. Given that the fundamental driver of the Pacific Highway upgrade is to improve safety, building kilometres of road outweighed flood proofing the Ballina Bypass. The optimum balance was determined to be adopting a low capital cost solution by constructing directly on the soft ground and implementing a whole of life maintenance and intervention strategy to manage the settlements.

The Ballina Bypass was a motorway with a 6km length traversing the floodplain associated with the Richmond River. The location of the motorway is shown in Figure 1 by the dashed line traversing the Holocene sediments.

The Roads and Maritime Authority of New South Wales (RMS), in conjunction with the Ballina Bypass Alliance (BBA), developed a “low embankment strategy” (LES) to minimise the whole of life cost of the Ballina Bypass motorway while allowing the motorway to operate during a 1 in 20 year flood.

The low embankment strategy had the following elements:

- The formation was designed such that the 1 in 20 year flood level was below the pavement level immediately on opening of the Ballina Bypass;

- A post construction settlement of 200mm was allowed to occur before intervening to top up the embankments. After 200mm settlement, the outside lanes would be below water during a 1 in 20 year flood but the inner two lanes would be free of water and trafficable;
- The embankment formation was to be placed shortly after construction commenced and allowed to settle for the maximum time available prior to pavement construction. A minimum settlement time of 17 to 20 months was specified; and
- Monitor the performance of the embankments during the consolidation period and take any actions required to achieve long term whole of life goals.

A pavement strategy was developed in conjunction with the low embankment strategy to allow the pavements to function when subjected to residual settlements.

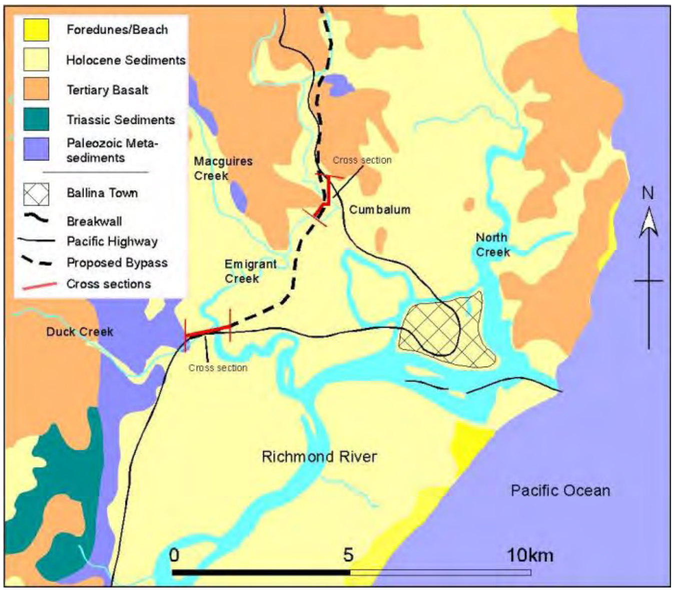

Figure 1: Location of the Ballina Bypass.

The pavement strategy aimed to construct heavy duty pavements using a staged process. On road opening, the pavement would be made from a granular chert material with a seal and an asphalt wearing course. Maintenance allowances and asphalt layers were developed such that a heavy duty deep lift asphalt pavement would be constructed over the design life of the pavement. The pavement strategy is summarised in Table 1.

Table 1: Construction of Staged Pavements

Year	**Staged Pavement: Required Maintenance / Pavement Improvement Interventions**
Construction	300mm chert pavement with spay seal wearing surface
0-1	Repair any initial defects and settlement (Allow 0.25% surface area)
1	Apply 40mm thick asphalt wearing surfacing
2 to 10	Asphaltic Concrete (AC) correction 10% (allowance)
8 to 10[#1]	100mm overlay (Chert or Asphalt)
30	Replace wearing surface including settlement correction (100mm) plus 1% heavy patching
40	Salvage

#1 Overlay thickness will vary from location to location

RMS had extensive experience in the Ballina region with the construction and maintenance of chert pavements. Chert pavements are essentially an unbound granular pavement, with a specific local grading. RMS experience was that chert pavements are a cost efficient and well performed pavement where they:

- are produced from high quality materials;
- are constructed with personnel experienced in chert pavement construction; and
- where particular attention is paid to edge detailing and seal widths.

The construction sequence generally comprised:

- a geotextile separation layer with provision for a bi-axial grid where required;
- a bridging layer of a graded rocky fill;
- an upper zone and select material zone (SMZ) of high quality earthfill material
- full width construction with chert "daylighting" on low side and sealed on high side

RMS requirements stipulated a pavement life for the BBA main line carriageways of 40 years. Due to the potential settlements in the low embankment areas it was considered that a pavement structure which would not require significant rehabilitation to address structural (trafficking) distress within the design period was not appropriate, as extensive planned intervention/maintenance/rehabilitation was expected to be required within the first 10 years of service to address settlement and functional performance issues.

There was considerable uncertainty in both the overall and differential settlement that would occur; hence provision of overlays was expected to be required for shape correction and/or to maintain specified minimum flood levels.

Granular pavements were the preferred option as they are flexible structures which accommodate more differential settlement than the other types of pavements. Full depth or deep lift asphalt solutions can be successfully employed under these conditions. However, they have a higher initial cost than a chert pavement but this was not realised in whole of life terms as overlays are still required and overlays result in under-utilisation of expenditure on the deeper pavement layers.

Given the low embankment areas were susceptible to flooding, water ingress was expected to affect the performance of the pavement materials. As such, it was essential that the materials providing the majority of the structural strength within the pavement comprise materials that are not significantly influenced by their moisture condition.

Maintenance would be triggered by any of the following factors:

- **Safety and function:** The rideability of the road would be affected by shape loss and surfacing defects such as potholes and water ponding arising from differential settlement and flood inundation. The first intervention was considered likely to be triggered by ride comfort;
- **Flood immunity level:** Level restoration to maintain the minimum flood immunity levels. It was considered unlikely that the formation and/or pavement would settle uniformly, and therefore maintenance for shape correction would be required prior to addressing extreme settlement.
- **Pavement distress:** Stripping and flushing were considered likely to trigger resealing activities of granular pavements with sprayed seal surfacings during service. In this type of pavement, surfacing failureswere expected and could be exacerbated by saturation of the underlying granular layers in the case that a flood event occurred. Pavement distress may also occur at isolated locations where extreme differential settlement was concentrated. This was considered most likely to happen at the transitions between ground treatment areas and untreated areas.

It was considered likely that maintenance activities during service, to address both settlement and functional issues, would be undertaken with overlays, or in minor works, with asphalt regulation. When maintenance was undertaken to address functional deficiencies or minor settlement, thicker overlays was expected for the granular pavements. Similar overlay thicknesses were expected when addressing formation settlement.

2. GEOTECHNICAL INPUT

The geotechnical challenge was to estimate performance of the embankments at the design stage, monitor the actual performance of the embankments during construction and to take actions to achieve the strategic goals if required.

2.1 GEOTECHNICAL MODEL

The ground at the southern end of the Ballina Bypass is comprised of a thin layer of over-consolidated crust overlying deep soft to firm Holocene clay over Pleistocene stiff clay. The soft Holocene clay thickness varies from 5m to 28m and has low shear strength, high plasticity and high compressibility. The soft soil is near normally consolidated and is described as dark grey alluvial clay, containing shells and traces of silts. A unit of loose silty sand was occasionally encountered as discontinuous layer or lens within the clays. The land was used for agriculture and mostly sugar cane.

Site investigations included 100MPa piezocone penetration tests (PCPT), dissipation tests, boreholes, hand vane, and down-hole shear vane tests. Laboratory oedometer tests were performed to assess the consolidation parameters.

Over-consolidation ratio (OCR) values were obtained from one-dimensional consolidation tests and from correlations with undrained shear strengths inferred from piezocone test data. Unfortunately many of the oedometer tests showed signs of significant disturbance and interpreting preconsolidation pressures from these tests was difficult. Therefore, the adopted OCR design values were made consistent with the undrained shear strength profile using Equation 1 (Ladd, 1991).

$$s_u = s(OCR)^m \sigma'_v \qquad (1)$$

In Equation 1, s_u is the undrained shear strength, σ‟$_v$ is the vertical effective stress, „m‟ was assumed to by 0.8, „s‟ = (s_u/σ'_v) for normally consolidated soils and a value of 0.22 was adopted.

The undrained shear strength of the clays was obtained from hand vane, down-hole shear vane and correlations with piezocone tests using an empirical N_{kt} factor of 15.

Rates of horizontal consolidation were obtained from insitu piezocone dissipation tests. The piezocone data for the project were interpreted using the method described by Teh and Houlsby (1991). Rigidity Indices of 20 and 40 were used in the interpretation. The rigidity indices are low and represent soil stiffnesses at strains expected during consolidation. Rates of vertical consolidation, c_v, were obtained by dividing the rate of horizontal consolidation by a factor of 2. They can also be obtained from available one-dimensional laboratory consolidation tests. However, these values are typically much lower than those recorded in the field and were not adopted for design. Field values are considered to reflect the presence of anisotropic fabric in the soil that might not be captured by small scale laboratory tests. In general, the interpreted c_v values were generally in the order of 1m^2/yr to 2m^2/yr in the Holocene clays. The interpretation of the c_v values was affected by an initial increase in water pressure with time prior to dissipation. This is considered to be a result of desaturation of the filter element as it is pushed through the crust or insufficient de-airing prior to testing.

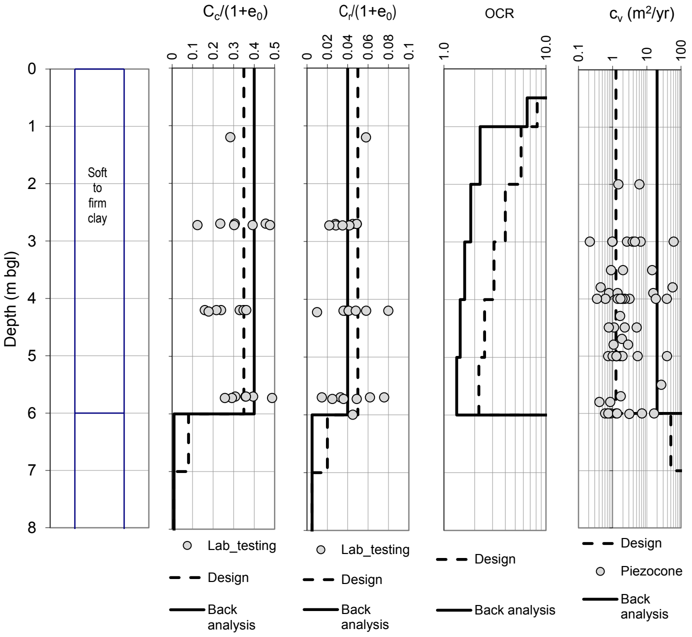

Figure 2: Geotechnical model.

The geotechnical model is summarised in Figure 2 and adopted design profiles are shown by the dashed lines. The design lines were believed to represent a cautious set of parameters. Upper quartile values for the compression parameters were adopted and lower quartile values for the rate of consolidation were adopted.

2.2 DESIGN STAGE SETTLEMENT PREDICTIONS

Settlements were predicted using one-dimensional consolidation theory, a bi-linear void ratio-log effective stress model and a finite difference numerical procedure for the adopted construction staging and geotechnical model.

Predicted settlements were expected to vary from construction performance and therefore an estimate of potential variability was evaluated using Duncan"s (2000) method. Uncertainties of the soil models are associated with evaluating soil and fill properties, with assessed depth of the soft clay deposit and with the unknown presence of sand/silt lenses or layers within clay unit between site investigation locations. The assessment indicated that the settlement may vary by as much as 50% from the predicted values where soft soil thicknesses are greater than 15m. The settlement may vary by 20 to 30% where the depth of the soft soil is less than 15m. The assessed rate of settlement may vary up to 50%.

The predicted settlement is shown in Figure 3 along with upper and lower bounds.

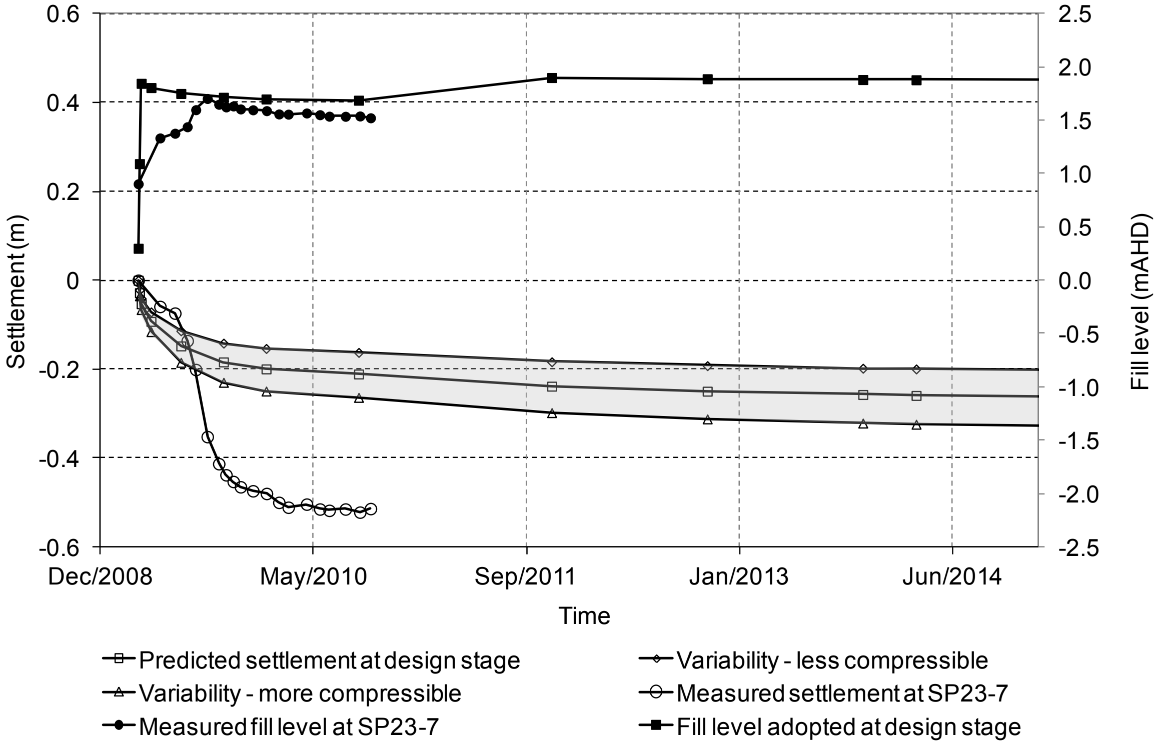

Figure 3: Comparison of predicted and measured settlements.

An intensive instrumentation and monitoring programme was implemented. Settlement plates were installed at 50m intervals staggered on the northbound and southbound lanes. The Observational Method (Peck, 1969) was implemented in the expectation that performance would not match expectation and additional actions might be required to achieve the desired outcomes.

2.3 PERFORMANCE OF THE LOW EMBANKMENTS

Predicted settlement at one particular location is compared with measured settlement in Figure 3. The monitoring data showed that actual settlement of the embankments was substantially greater than estimated at design phase and was greater than the upper bound predicted values up to May 2010. A fast rate of settlement was observed during the early stages of construction. This trend was observed along the entire length of the low embankments to a greater or lesser degree.

The magnitude of the settlement meant that the fill had settled significantly below design level. Additional fill would have to be placed in order to bring the embankment back to design level, but the additional fill would also trigger more settlement during construction and potentially more settlement than could be accommodated by the whole of life pavement strategy.

A remedial strategy was required in order to ensure that the post construction performance met the required maintenance activities and pavement interventions. The first step in this process was to perform back analysis to understand why the soil behaved differently to expectations and to refine the geotechnical model.

2.4 BACK ANALYSIS

Back analysis was performed by varying the compression, OCR and rate of consolidation parameters. These analyses were performed in 2010 and 2011 during construction.

Primary consolidation was observed to be completed when plotting measured settlement data against the logarithm of time. As a result 100% primary consolidation could be approximated with a high level of confidence. For a particular location, various combinations of compressibility parameters were adopted and the settlement analyses were carried out to fit 100% settlement where it was measured. The results of the analyses are shown in Table 1. Values of the empirical parameter „k" in Table 2 relate to Equation 2 and were varied in the analysis to give different OCR profiles. The values adopted for the back analysis of the low embankments are highlighted in bold.

$$OCR = \frac{k\left(q_t - \sigma_{v0}\right)}{\sigma'_{v0}} \qquad (2)$$

In Equation 2, q_t is the net cone resistance, σ_{v0} is the total initial stress, σ'_{v0} is the initial effective stress and „k" is an empirical fitting parameter. In Table 2, CRR is the recompression ratio and CR is the compression ratio.

This analysis was performed on a location that had settled 0.83m. A wide range of parameters can be selected to fit the measured settlement. The question is whether or not the parameters are credible. The parameter set adopted was selected because it provided an upper bound to predicted settlement at locations where the soft soils were deep and therefore was considered to be a cautious estimate. Of the adopted parameters, the k value is considered to lie at the low end of the range. The CR values are higher than measured in oedometer tests performed for the Ballina Bypass but are generally consistent with tests on high quality samples currently being performed at the University of Newcastle. At the time of back analysis, this process of back analysis was performed for areas with a range of soft soil depths (not presented in this paper) and the adopted parameters were considered to more adequately represent the range of behaviour than the other parameter sets presented in Table 2.

Table 2: Comparison of parameter combinations

k	**CRR**	**CR**	**Settlement (m)**	**Decision**
0.15	0.123	0.25	0.83	k implies OCR < 1 for soils greater than 6m depth
0.15	0.04	0.297	0.829	k implies OCR < 1 for soils greater than 6m depth
0.2	0.123	0.3	0.829	
0.2	0.081	0.35	0.828	
0.25	0.135	0.35	0.831	
0.2	**0.04**	**0.4**	**0.83**	**adopted values**
0.25	0.111	0.4	0.83	
0.3	0.15	0.4	0.832	
0.2	0.0001	0.45	0.834	CRR Not Credible
0.25	0.087	0.45	0.829	
0.3	0.135	0.45	0.828	
0.3	0.122	0.5	0.831	CR considered upper bound

This process of back-analysis relies on engineering judgement and to a large extent is an exercise in curve fitting. As such, the parameters cannot be considered absolutely representative of the soil. However, the adopted process does maintain observed trends with depth in OCR values and results in CR and CRR values consistent with subsequent high quality testing. Therefore, the adopted parameters can be considered credible within the confines of a bi-linear one-dimensional model.

A comparison between the measured and back-figured settlements is shown in Figure 4. The back-figured soil parameters are shown in Figure 2. In order to fit the measured settlement, CR values were revised from 0.35 to 0.4, CRR values were reduced to 0.04 from 0.05, the OCR profile was reduced by between one-third and one-half and the rate of vertical consolidation was increased to 20m^2/yr.

The two most significant differences in the soil parameters are the values adopted for the OCR profile and the rate of vertical consolidation.

Determination of the OCR profile at the design stage was complicated by a lack of confidence in OCR values determined from laboratory testing and lack of sensitivity in the 100MPa piezocone. The back-figured OCR profile corresponds to an „s‟ value in Equation 1 of 0.3. Although this value appears to be high, Ladd (1991) reports that „s‟ can be in the order of 0.3 for high plasticity clays rather than the „default‟ value of 0.22 adopted in design.

Across the area traversed by low embankments, the back analysed coefficient of vertical consolidation ranged from 6 to 125m^2/year for the top few meters of soil. For deeper soft soil, the coefficient of vertical consolidation was smaller and ranged from 8 to 15m^2/year. However, where wick drains were used, the back-figured coefficients of consolidation were about 2m^2/year which is consistent with values adopted from the piezocone dissipation tests. The rate of settlement is a function of the rate of consolidation and also the length of the drainage path. As back-figured rates of consolidation in areas where wick drains were used were similar to values inferred from PCPT tests, it suggests that the difference between design and back-figured values is due to lengths of drainage path being shorter than assumed. The PCPT tests show that clay is uniform with depth and that there are no thin sand lenses present. One possible cause of a short drainage path is the presence of plant roots in the upper 6m of the soil profile (Brian Chandler, AECOM Victoria, personal communication). Sugar cane roots were detected in U75 samples taken in the upper 6m of the clay profile and it is possible that the roots acted as naturally occurring vertical drains. It is also possible that the limitations of the dissipation tests and adopted rigidity index resulted in underestimates of the coefficient of consolidation. Generally speaking, adopting a lower bound for the coefficient of consolidation in design is conservative as it suggests settlement will take longer than it actual, however in the case of the low embankment strategy it was unconservative because more materials were required to construct pavement and potentially more settlement related maintenance could have been required post construction. The compression and recompression ratios were slightly modified to fit the data.

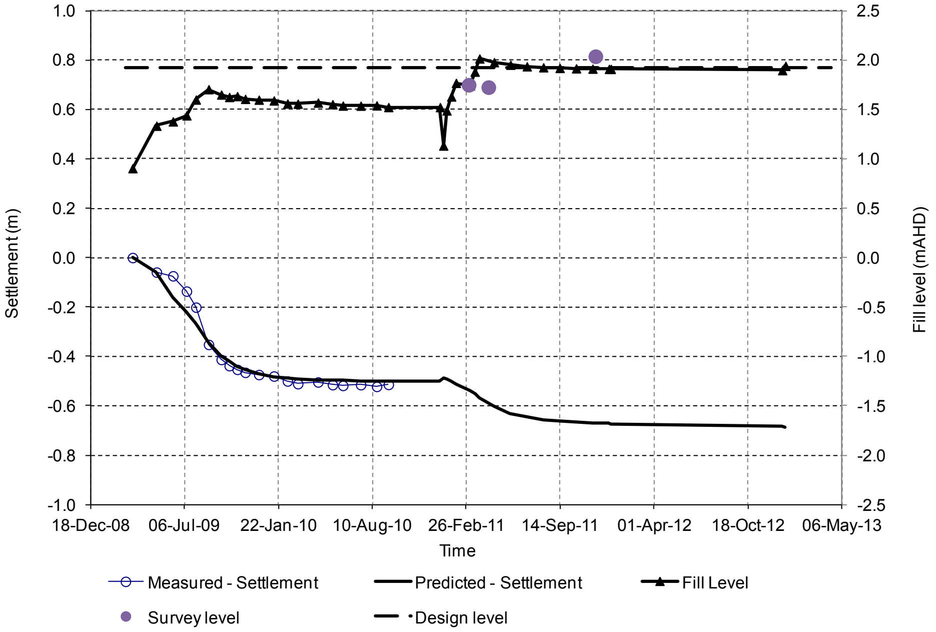

Figure 4: Example of a back analysis - settlement time curve

2.5 REVISED CONSTRUCTION STRATEGY

The fact that there was more settlement and settlement occurred at a faster rate than anticipated meant that additional fill would be required to construct embankments to the design level at road opening and the post construction settlement would be greater than assumed at in the pavement strategy. More interventions would be required at earlier intervals and greater frequencies than the design settlement estimates suggested.

Construction of the low embankments was revised to achieve the long term embankment settlement requirements. The following modifications were made:

- At the southern end of the alignment where the soft soils were deepest, the design vertical alignment was generally higher than required for flood immunity. The vertical alignment was lowered and a lower bound alignment specified that satisfied flood immunity and minimum road design requirements while allowing settlement to occur over time without going below the lower bound alignment;
- Surcharge was placed in some localised areas; and
- Based on the revised geotechnical model, the embankments were constructed to a targeted level above the design level at all locations, and the embankments were allowed to settle such that design level was achieved at the date of road opening.

In addition, it became evident that settlement would occur between placement of each layer of chert pavement. This had the potential to result in more pavement material being placed than provided in the cost estimate. Detailed numerical analyses were performed after the back analyses had been completed to assess the minimum thickness of fill required to construct the pavement to the required level. The predicted settlements are shown in Figure 4. Although these analyses were performed, construction actually occurred at different times to those used in the analyses. Top of fill levels measured during construction are compared with predictions in Figure 4. Measured settlement in February 2011 compare well with predicted settlement. The final pavement level was achieved in November 2011 compared with February in the analysis and this meant that the pavement level at road opening was higher than assessed.

3. CONCLUSIONS

The following conclusions can be drawn from the design, construction and back-analysis of the low embankments:

- Compression parameters and rates of consolidation inferred from conventional site investigation data may not accurately represent actual soil parameters. The highest quality of data is required when embankments are constructed over soft clays;
- Rates of consolidation and overconsolidation ratios were most difficult to estimate;
- Rates of settlement may be high in the upper 6m of the clay deposit due to the presence of roots;
- The Observational Method coupled with detailed back analysis is a vital component of constructing low embankments cost effectively and to achieve design requirements; and
- Low embankments should be constructed by filling sufficiently above design level such that the weight of fill removed when pavements are constructed is greater than the weight of the pavements. This method will prevent additional settlement being triggered by placement of extra fill later in the preloading period. The challenge is to accurately assess how much settlement will occur during the preloading period.

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