Consolidation and creep settlement of Ballina soft soils

H.D. Subasinghe
Advisian (formerly SMEC), Sydney, Australia

M.A. Khan & M.P. Dineshharan
SMEC, Melbourne, Australia

ABSTRACT: As part of the Pacific Highway upgrade (Broadwater to Pimlico), SMEC was engaged by Pacific Complete to carry out detailed design for Portion D, starting at Richmond River and going as far as Ballina Bypass. This paper discusses ground improvement design over thick, variable soft soil deposits around Randle's Creek / Coolgardie Road to enable construction of reinforced concrete box culvert (RCBCs) and bridge approaches. Particular emphasis is placed on ground characterization, identification of geological risk, interpretation of consolidation parameters of soft soils and a comparison between predicted settlement vs. field measurements. Due to the presence of thick soft soils layers, a combination of Surcharge-Wick Drains (SWD) and Concrete Injected Columns (CICs) were employed to reduce the post construction secondary settlement of the embankments/approaches and box culvert structures. Measured field settlements were compared against the predicted settlement; to confirm release of preloads and to validate that ongoing long-term settlements are within design requirements.

1 INTRODUCTION

SMEC Australia Pty Ltd was engaged by Roads and Maritime (represented by Pacific Complete, PC) to provide detailed design services for Portion D – Broadwater to Pimlico (B2P) of the Pacific Highway that commences on the north bank of Richmond River and stretches approximately 17.7 km north to the Ballina Bypass. The project comprises construction of around 17.7 kms of dual carriageway to Class M motorway standard, 14 bridge structures and several drainage and fauna connectivity structures. The following two distinct soft soil areas were identified along the alignment by PC:

Randle's Creek Bridge/Approaches
Whytes Lane overpass to Ballina Bypass

Initial ground information comprising boreholes, laboratory testing data, test pits, DMTs and CPTs were provided by PC. This paper discusses the soft soil treatment design at Randle's Creek bridge location and the additional soft soils identified in vicinity (at Coolgardie Road). Treatment of Whytes Lane overpass to Ballina Bypass has not been discussed in this paper. Figure 1 shows the location of Randle's creek bridge and Coolgardie Road.

State-of-the-art design approaches and analysis techniques were used to analyze and design the soft soil treatment. Following the discussion on design, performance of the treatment carried out till date and its comparison with the behavior predicted in design has been discussed.

2 REGIONAL GEOLOGY AND SUBSURFACE CONDITIONS

About 1 km south of Coolgardie road, the highway alignment continues across mixed floodplain and gently undulating areas of colluvium located on either side of the proposed Coolgardie Interchange.

North of Coolgardie road, the road alignment crosses the floodplain of the Richmond River, underlain by Quaternary age alluvial and estuarine sediments which reflect multiple changes in sea level over the last 150,000 years. These alluvial sediments are underlain by sedimentary rocks of the Bundamba Group and Neranleigh Fernvale Beds.

3 IDENTIFICATION OF SOFT SOILS AND GEOTECHNICAL RISKS

Initial geotechnical investigations by PC were limited due to the environmentally sensitive nature of the bridge site. Additional geotechnical investigation were requested by SMEC including a CPT probe for a single cell RCBC and multicell RCP culverts in vicinity of the bridge on the mainline Pacific Highway.

The investigation results were then studied in combination with the ground topography and hydrology. A review of the contour map of the area indicated a trend where areas present under a level of 4.0m AHD indicated presence of soft soils. The Creek flew in two distinct channels and both presented potential for soft soils. A second stage of geotechnical investigations was then planned and implemented at Coolgardie Rd.

This investigation showed confirmed presence of soft soils of up to 11.0m depth at Coolgardie road. This posed significant challenges as the Coolgardie Road embankment is up to 8.0m high and has two RCBCs, located over the soft ground. There were clearly significant risks of the embankments and RCBC undergoing large deformations during and post construction.

This highlighted the need to have sufficient geotechnical investigations and to look at the geotechnical data in combination with the hydrological and topographic conditions of the area.

A third and final stage of geotechnical investigations was carried out and investigations indicated presence of three distinct soft soil layers, identified as follows:

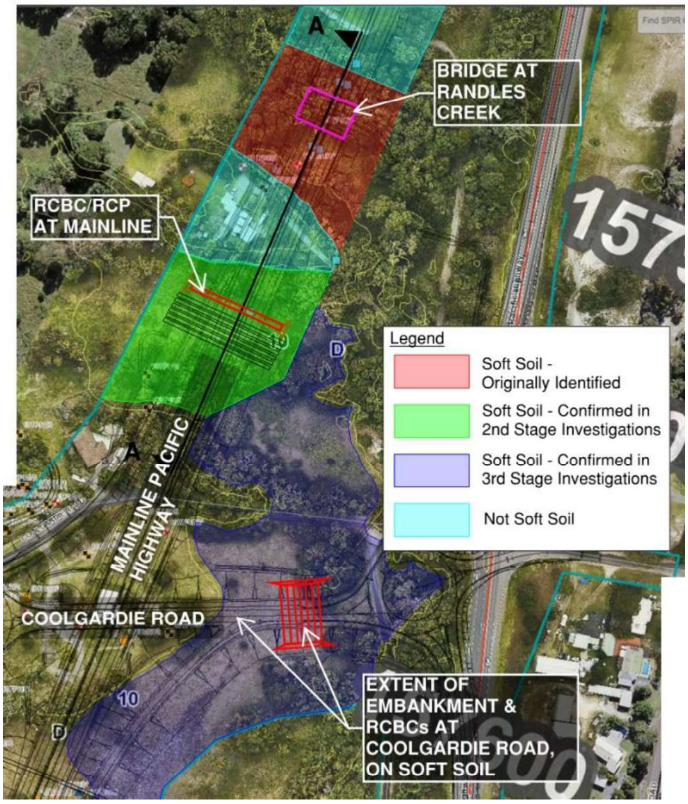

Figure 1. Project Soft Soil Extents

Holocene Alluvium / Crust (Unit 2c): Soft, medium to low plasticity, grey to brown Clay
Holocene Estuarine Clay (Unit 2e): Very soft to soft, medium to high plasticity, dark grey to black
Holocene Estuarine Silty Clay with layers of loose Sand

Soft soil layers were followed by a thin layer of indurated Sand, followed by thick layers of Pleistocene Estuarine Clay, generally present in a Stiff consistency. Figure 2 indicates the Geological Profile along Coolgardie road alignment, interpreted after completion of Stage-3 investigations. A suite of laboratory testing was also carried out on undisturbed samples extracted from borehole.

4 INTERPRETATION OF PARAMETERS FOR GEOTECHNICAL DESIGN

The geotechnical design parameters were adopted based on the interpretation of previous geotechnical investigations and the additional in situ and laboratory test results planned by SMEC. The groundwater table has been assumed to be at 3.0 mAHD.

4.1 *Unit weight (γ), undrained shear strength (Su) and over-consolidation ratio (OCR)*

Unit weights of 15.5 kN/m^3 to 15.2 kN/m^3 were used for Units 2c and 2e. Where silty Sand/clayey Sand layers were encountered within these units, unit weights were increased to 17.5 kN/m^3 and 16.0 kN/m^3 respectively.

Cone penetration testing (CPTs), Seismic Dilatometer Tests (DMTs) and Vane Shear Tests (VSTs) were analysed for estimation of the undrained shear strength of soft soil units. The results of the VSTs and CPTs were used to calibrate the site specific N_{kt} and a value of 15 was used with equation 1:

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}} \tag{1}$$

Where q_t is the cone tip resistance, corrected for pore-pressure; and σ_v, the total overburden pressure.

Pre-consolidation pressures were calculated using the following correlation:

$$p_c' = 0.3(q_t - \sigma_v) \tag{2}$$

The OCRs were in turn calculated using the relationship:

$$OCR = \frac{p_c'}{\sigma_v'} \tag{3}$$

4.2 *Coefficient of vertical and horizontal consolidation (cv and ch) and permeability*

The coefficients of vertical and horizontal consolidation as well as permeability in the vertical and horizontal direction were interpreted from the results of dissipation tests carried out at and in vicinity of Coolgardie road. The resulting values adopted at detailed design for normally consolidated soils are:

Unit 2c/Unit 2e: c_h = 6 m^2/year, c_v = 3 m^2/year; k_h = 5.0 x 10^{-9} m/sec, k_v = 2.5 x 10^{-9} m/sec

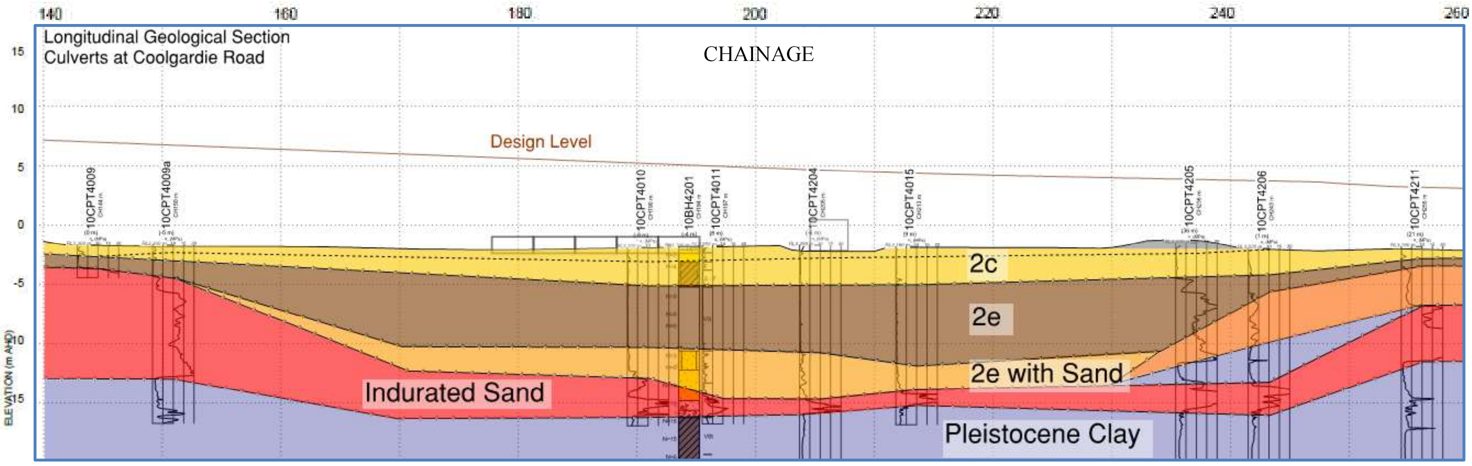

Figure 2. Geological long section at Coolgardie Road

Where silty/clayey Sand layers are encountered interbedded within the 2c or 2e layers, an average horizontal permeability of 1 x 10-7 m/sec was adopted. The use of empirical correlations referencing (NAVFAC DM7.1, 1982) with liquid limit (using a value of 60) results in a c_v value of around 3 m^2/year, confirming the design assumptions. The presence of silty Sand layers was accounted for in determination of the drainage path for consolidation and for this assessment a drainage path of 3m was deemed appropriate. The values for c_h/c_v and k_h/k_v were reviewed based on experience with soft soils in the project area and a ratio of 2.0 was adopted for design.

4.2.1 *Compression ratio (CR), recompression ratio (CRR) and creep coefficient ($c_{\alpha e}$)*

The CR and CRR values were interpreted from the void ratio/moisture content values measured for Unconsolidated Drained samples extracted at Coolgardie road. Oedometer test results were also analysed for determination of the CR and CRR values. The outcome was plotted with elevation, from where the design values were adopted as follows:

Unit 2c (Clay): CR = 0.25 and CRR = 0.030
Unit 2e (Clay): CR = 0.35 and CRR = 0.045

Where layers of loose Sand were indicated within Unit 2e, the compressibility was reduced to account for the Sand layers. The following compressibility values were interpreted for Unit 2e, interbedded with layers of Sand:

Unit 2e (with Sand): CR = 0.175 and CRR = 0.022

The creep coefficient $c_{\alpha e}$ ($c_\alpha/(1+e_o)$) for normally consolidated soils was adopted as 4 per cent of the compression ratio based on experience with soft soils within the vicinity of Coolgardie Road. The following equation (Wong 2007) was adopted to calculate creep coefficient for over consolidated soils:

$$\frac{C_{\alpha(oc)}}{C_{\alpha(nc)}} = \frac{(1-m)}{e^{(OCR-1)n}} + m \qquad (4)$$

The model constant values of m and n were adopted as 0.1 and 6, respectively.

4.2.2 *Strength Gain on Consolidation*

The strength gain in soils on consolidation was also considered for the assessment of stability during surcharge treatment, as well as in service. The following correlation was used to estimate the gain in undrained shear strength:

$$S_{u(NC)} = 0.23\sigma'_v \qquad (5)$$

To achieve the required Factor of Safety under short and long-term scenarios, four high strength geofabrics (HSG) were incorporated in the design. Assessment of embankment stability was conducted using commercial software Slope/W during detailed design.

5 PROBLEM DEFENITION AND DESIGN REQUIREMENTS

5.1 *Embankment stability*

Project specific requirements from the client required the embankment stability to conform to the following conditions;

Long term slope stability, FoS >1.50
Short Term Slope stability, FoS >1.25

5.2 *Settlement criteria*

The settlement criteria of the embankments were prescribed for a 40 year pavement design life as per project specifications. All pavements were designed as flexible pavements and targeted to achieve Total Residual settlement ≤ 200mm, change in grade ≤ 0.5% at any direction and at approach embankments to culverts and bridges, the settlement was required to be ≤ 50mm.

6 GROUND IMPROVEMENT DESIGN

The following ground improvement options were considered due to their suitability:

6.1 *Preload only*

Preload only option was used for sections with thin layers of soft soils. No additional surcharge was placed at these areas. Preload periods were released once the post construction projected settlement was within allowable limits.

6.2 *Surcharge-wick drains (SWD)*

The SWD solution incorporated two components; installation of Prefabricated Vertical Drains and placement of additional surcharge above design embankment levels.

This technique was used where the soft soils were thicker and embankment heights were relatively high. The SWD was designed to achieve 90% primary consolidation of the total surcharge within 6 months after construction of fill. A wick drain spacing of 1.5m centers in a triangular grid was adopted in design. Additional surcharge up to 3m were also placed at Coolgardie Road section. At some locations, the increased embankment height (up to total height of 11.5m with surcharge) lead to need for additional stability berms.

6.3 *Concrete injected columns (CICs)*

Hard inclusions in the form of CICs were used underneath RCBC culverts overlying thick soft soils and at the approaches to minimize the change in grade in pavements. CICs with a 450mm diameter, constructed using 40MPa concrete at a grid spacing of 1.8m to 2.5m were constructed.

6.4 *2D finite element modelling*

All ground improvement zones were modelled in Geotechnical 2D Finite Element software Plaxis V.AE; to estimate two-dimensional behavior of the embankment. 2D-Plane-Strain models were developed using 6-node Linear Strain Triangular elements for separate cross-sectional and long sectional models.

An overview of ground improvement zones is shown in Figure 3 below.

Figure 3. Ground Improvement at Coolgardie Road

7 INSTRUMENTATION, MONITORING AND FIELD PERFORMANCE

A comprehensive instrumentation design was implemented at both Coolgardie mainline and local roads to monitor consolidation and creep settlement as well as embankment stability. This included settlement plates, magnetic extensometers, vibrating wire piezometers and inclinometers. Instrumentation plans for Coolgardie road is presented in Figure 4.

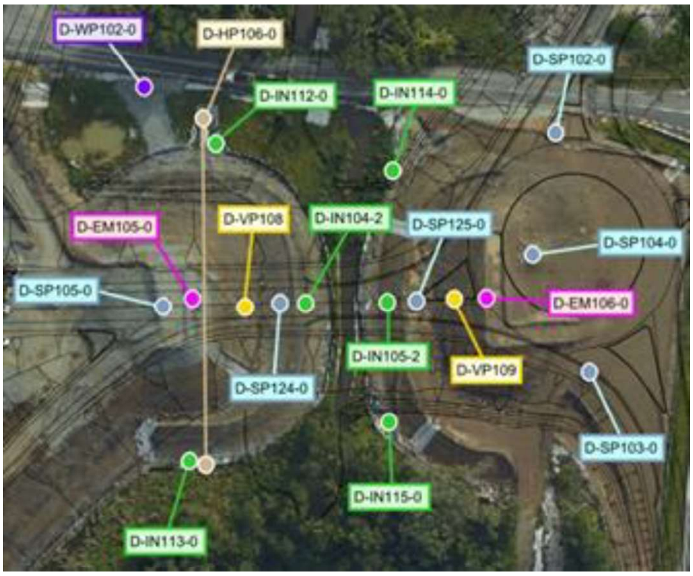

Figure 4. Instrumentation plan at Coolgardie Road

8 BACK ANALYSIS

A back analysis was completed based on field instrumentation data for Coolgardie. The Asaoka method (Asaoka, 1978) was used calculate the current degree of consolidation and estimate the remaining predicted primary and recompression settlement. The DOC vs. time plots from the Asaoka method were then used to back analyse the adopted coefficient of consolidation.

8.1 *Assessment of total settlement and degree of consolidation*

The Asaoka method was used to assess the current degree of consolidation from settlement plate data. VWP data was also used to estimate the current degree of consolidation. A summary of the current degree of consolidation for both Coolgardie and Mainline culvert embankments is presented in Table 1.

In general, both the settlement plate and VWP data suggest that the degree of consolidation at the time of this assessment lies between 95-100% at Coolgardie road and around 90 – 95% at the mainline culvert location.

The time-degree of consolidation curve from measurements were then compared with theoretical curves derived from 'Time rate of consolidation for

gradual load application' (NAVFAC, 1986 pg 7.1-232).
Coefficients of vertical and horizontal consolidation were then adjusted until a reasonable fit between measurement and theory is obtained.

Table 1. Summary of current degree of consolidation

Location	Instrument	Current DOC
		%
Coolgardie Local Road	SP-104	> 95
	SP-105	> 94
	SP-124	> 90
	SP-125	> 94
	VP-108	>97
	VP-109	>94
Coolgardie Main-line Culverts	SP-100	>94
	SP-112	>93
	SP-122	>93
	SP-130	>91
	SP-133	>92
	VP-106	>90
	VP-107	>91

Plots of the degree of consolidation assessed using the Asoka method from settlement plate data in comparison with theory for Coolgardie Road and the Mainline preload are presented below in Figure 6.

The mainline culvert preload settled more or less as predicted during detailed design indicating that the interpreted ground profile and parameters were reasonable. However, like Coolgardie local road, the paleochannel tapers off into stiff ground more rapidly than what was assumed in detailed design. The rate of consolidation of the soft ground at the mainline is closer to what was assumed in design indicating the clay at this location has less sand lenses. Back analysed c_v and c_h are around 5 and 10m^2/year, respectively at this location.

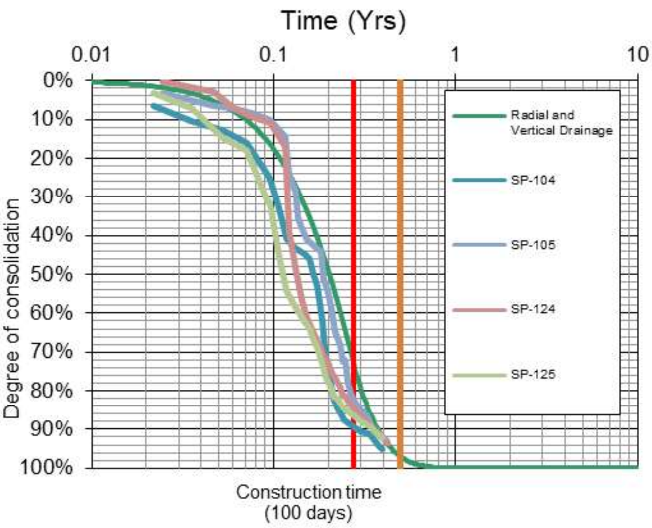

Figure 6. DOC vs. Time at Coolgardie Rd

8.2 *Finite element analysis for Coolgardie Road preload*

8.2.1 *Total settlement and back analysis of soft soil thickness using PLAXIS 2D*

A 2D finite element analysis was performed in PLAXIS to re-evaluate the thicknesses of soft clay at Coolgardie road based on the embankment performance as indicated by the instrumentation data. Iterations were performed at the settlement plate/extensometer locations until a reasonable match between the finite element model and the field data was achieved.

A comparison of the original and re-analysed geological profile and total settlement contours are presented below in Figures 7 and 8 respectively.

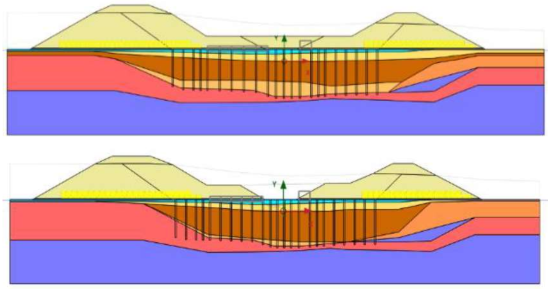

Figure 7. Comparison of original and back analysed geological profile

The instrument data and 2D finite element back analysis indicated that the upper 2e thickness around the berm locations of the preload (within vicinity of the culverts) is thicker than what was assumed in design and tapers off into stiff ground under the full surcharge height.

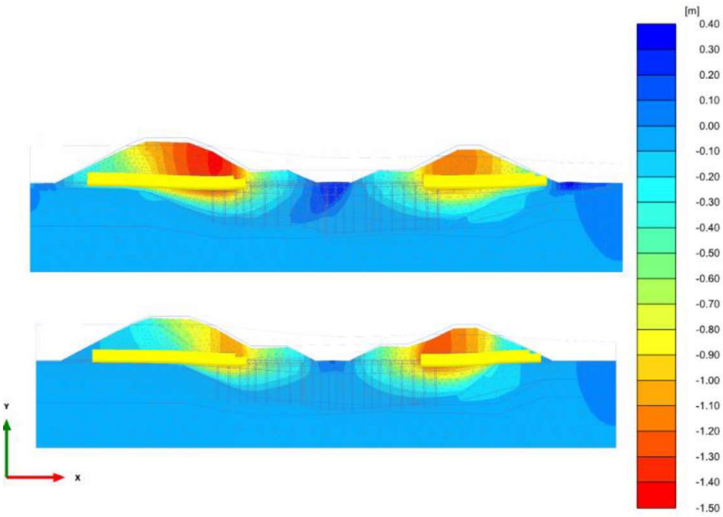

Figure 8. Total settlement contours, original design vs. back analysed ground model at 90% DOC

A back analysis of geotechnical design parameters was also conducted using an axisymmetric model in Plaxis 2D for SP-124 to attain a full time settlement history at the thickest soft soil area within the Coolgardie Road area. The purpose of this was to re-assess the permeability of the soft clay at this location. The back analysis was conducted using the following approach:

Keep deformation and stress state parameters (Unit weight, CR, CRR and OCR) consistent with what was assumed during detailed design

Adjust k_v and k_h until a match with SP-124 instrument data was attained, employing a permeability change index of $c_k = 0.5e_0$ for unit 2c and 2e layers.

The back analysed permeability (with comparison to the detailed design assumptions) is summarised in Table 3 below:

Table 3. Geotechnical model at SP-124 and back analysed permeability

Unit	Vertical permeability, k_v (m/s)	
	Design	Back analysis
2c - crust	2.50E-09	9.50E-09
2c	2.50E-09	9.50E-09
2e	2.50E-09	9.50E-09
2e (with sand)	1.00E-09	1.00E-07
Indurated sand	1.00E-06	1.00E-07
Pleistocene clay	5.00E-09	5.00E-08

Time-settlement curves for SP-124 and the PLAXIS axisymmetric model were plotted together for comparison in 9 below. This demonstrates that a good curve fit was achieved using the permeability values above and by keeping original deformation and stress state parameters the same.

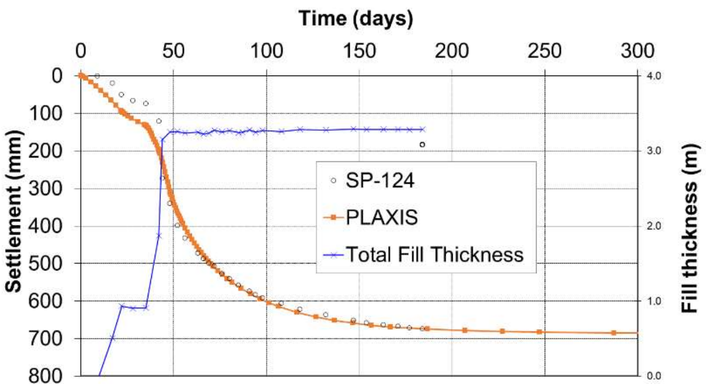

Figure 9. Back analysed time settlement response at SP-124

9 CONCLUSION

In general, the instrument data suggests that the performance of the embankment at the mainline culvert is more or less as predicted during detailed design. The soft clay at this location contains less sandy lenses and this also implies the interpreted geotechnical parameters for 2c and 2e are quite reasonable. It was found that the total predicted settlements lie within the actual observed values.

At both locations, a current degree of consolidation of 90-100% was assessed using different methods around 5 months since start of fill placement.

The rate of consolidation and thickness of soft clay at Coolgardie Road were the key aspects of the geological uncertainty of the extent of the paleochannel at Coolgardie Road and this is evident in the settlement plate data which shows a large variance in total settlement and rate of consolidation.

10 ACKNOWLEDGEMENTS

The authors acknowledge the contributions of the all parties involved in this technical paper. Any opinions, findings, conclusions and recommendations in this paper are those of the authors only, and do not necessarily reflect the views of NSW Roads and Maritime Services. In particular, the authors would like to acknowledge Pacific Complete for their support and Richard Kelly for his guidance throughout the design process.

REFERENCES

Asaoka, A. (1978). Observational procedure of settlement prediction. Soils and Foundations, Journal of the Soils and Foundations Engineering, 18 (4), 87-101.

Bjerrum, L. (1972). Embankments on Soft Ground. State of the Art Report, presented at the June 11-14, 1972, ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, held at Lafayette, Indiana, Vol 2, pp 1-54.

Chai, J., Shen, S., Miura, N., & Bergado, D. (2001). Simple method of modelling PVD improved subsoil. J. Geotech Eng. ASCE, 127(11), 965-972.

NAVFAC. (September 1986). DM7.01 - Soil Mechanics.

Pineda, J., Suwal, L., Kelly, R., Bates, L., & Sloan, S. (July 2016). Characterisation of Ballina clay. Géotechnique, Volume 66 Issue 7, 556-577.

Robertson, P. (2010). Soil Behaviour Type from CPT: an update. Proceedings of the 2nd International Symposium on Cone Penetration Testing, CPT'10. Huntington Beach, CA.

Wong, P. (October 2007). Preload design to reduce post construction creep settlement. Soft Soils and Ground Improvement Workshop, Proceedings of the 10th Australia and New Zealand Conference on Geomechanics, Brisbane, Australia, 23-31.