THE USE OF EARLY-WORKS EMBANKMENTS IN SOFT SOIL AREAS TO OPTIMISE DETAILED DESIGN: GATEWAY MOTORWAY CASE STUDY

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ABSTRACT

This paper presents a case study on the use of early-works preload[1] embankments in soft soils areas to provide information to optimise detailed design. The Gateway Upgrade North (GUN) project involved the widening of the existing Gateway motorway from four to six lanes with some areas of re-alignment. Early-works for the motorway upgrade involved construction of sections of embankment located in areas of soft soils. From a geotechnical perspective, the early-works were essentially instrumented trial embankments constructed 9 to 12 months ahead of the main package and therefore provided an opportunity to observe embankment and wick drain performance and back-analyse soft soil consolidation parameters used for the detailed design for the final motorway construction.

Data from settlement plates, vibrating wire piezometers and inclinometers was used in conjunction with site investigation and laboratory data to assess consolidation parameters of highly compressible Holocene-age alluvial clays. Asaoka's method and Terzaghi's theory of one-dimensional consolidation were used in the back analysis of primary consolidation parameters. Secondary settlement was also observed allowing back analysis of secondary compression parameters. Using consolidation parameters derived from the back analysis, design parameters were allocated to relevant geological units which were then applied in settlement modelling for critical sections in the detailed design.

Assessment of the early-works embankment monitoring data enabled a more robust prediction of embankment behaviour during and post-construction. This resulted in a more cost-effective and optimised embankment design with higher confidence in predicted post-construction settlements.

1 INTRODUCTION

The Gateway Motorway between Nudgee and Deagon was originally constructed in the 1980s as a four-lane vehicular motorway which provided a critical transport corridor servicing the Brisbane Airport, Port of Brisbane and the Australia TradeCoast precincts. From August 2014, Queensland Motorways Pty Ltd (QM) oversaw the procurement of the major construction contract for the Gateway Upgrade North (GUN) on behalf of the State Government to relieve congestion and demands from the growing South East Queensland population and continued expansion of the Australia TradeCoast. The upgrade project included 11.3 km of motorway widening, reconfiguration, modifications to on-ramps and off-ramps, construction of off-road cycle/ pedestrian facilities and integration of Intelligent Transport Systems.

Two early-works embankments were constructed in 2015 in areas of soft soils:

1. Preload Area A at Nudgee Golf Course – approximately 1.1 km of motorway realignment.
2. Preload Area B north of Nundah Creek – approximately 850 m of motorway widening.

Using an observational approach, the performance of these early-works embankments was monitored and used to reduce the geotechnical uncertainty for the remainder of the motorway upgrade works. A set of parameters were selected to fit the monitoring data.

The observational approach with instrumented trial embankments constructed to full scale have been carried out for other motorway and civil infrastructure projects overseas (Poulos et al., 1989; Balasubramanium et al., 1995) and in Australia (Kelly 2008; Ameratunga et al. 2010). This case study details the methodology and findings of this approach carried out for the GUN project.

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[1] The term preloading is used in this report to refer to both 'preloading' and 'surcharging'. The former is the application of a temporary load, usually via fill, equivalent to the future fill plus in-service load, for inducing a substantial fraction of the expected settlement prior to construction. The latter refers to applying extra load to enhance preloading.

2 GEOLOGICAL SETTING AND STRATIGRAPHY

Reference to the published geological mapping (1:100,000 series Geological Map for Brisbane, by Queensland Department of Employment Development and Innovation, 2011) indicates that the majority of the motorway corridor traverses either Holocene or Pleistocene estuarine deposits overlying a deeply-weathered residual soil profile originating from the Tertiary-age Petrie Formation (i.e. sandstone, siltstone, mudstone and basalt). Some uncontrolled fill is known to be present in places.

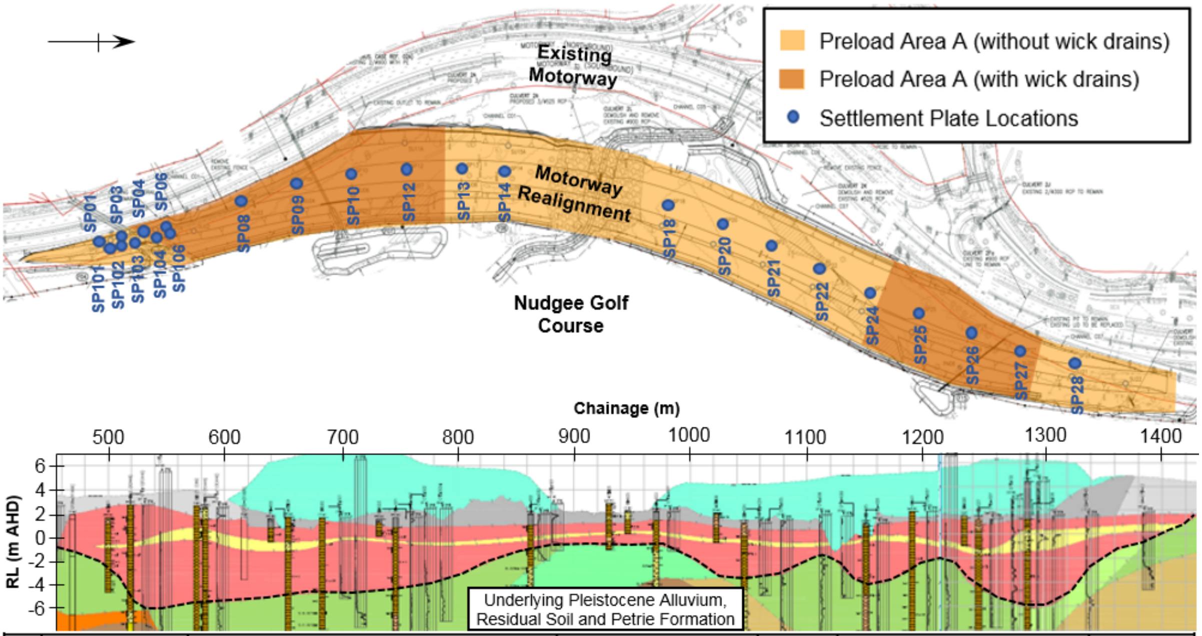
The Holocene deposits are recent alluvial deposits that are mainly mapped as undifferentiated coastal plains of mud, sand and gravels, underlain by tidal flats comprising sand and mud. Holocene deposits typically comprise very loose to loose clayey sands and/or very soft to firm and highly compressible clays.

The Pleistocene deposits represent an older, overconsolidated alluvium mapped as clay, silt, sand and gravel; flood plain alluvium is mapped on high terraces. The top of the Pleistocene represents a former land surface subject to dramatic erosion and stream down-cutting when sea levels were lower. This former land surface is now buried beneath the compressible Holocene alluvium laid down during the most recent period of sea level rise, producing several infilled paleochannels. Of critical importance from an engineering perspective is the depth of the Pleistocene surface (i.e. the base of the soft, highly compressible Holocene alluvium).

During the early stages of the project, the alignment was divided into five geo-sections based on geological conditions. The compressible Holocene deposits within each geo-section are generally continuous, of a similar geological origin and have comparable geotechnical properties (taking into account natural variability). Preload Area A was located within Geo-section 1 and Preload Area B was located within Geo-section 3.

2.1 GEO-SECTION 1 (COMPRISING PRELOAD AREA A)

Geo-section 1 covers approximately 1.6 km of the motorway alignment and is located at the fringes of the Brisbane River delta. Based on the published geological mapping, the section comprises two different Holocene units: undifferentiated coastal plains (Qhc); and sandy / gravelly beach ridges (Qhcb). Preload Area A covers a significant portion of this section in an area comprising the section’s deepest Holocene deposits. Geotechnical investigation data indicates that the Holocene alluvium extends up to depths of 8 m below natural ground level with inferred stratigraphy illustrated on Figure 1.

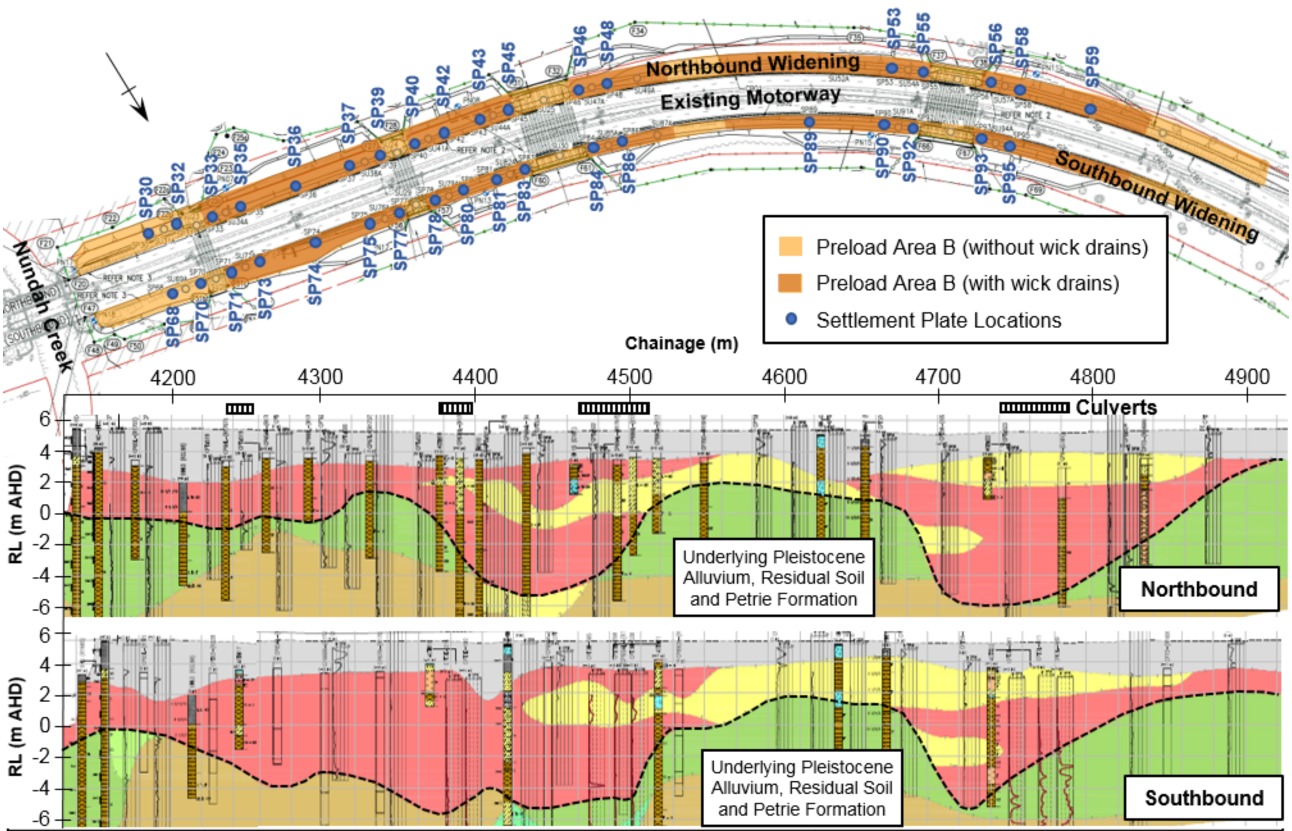


UNIT	GEOLOGICAL ORIGIN	GEOLOGICAL DESCRIPTION	MATERIAL TYPE	SUB LAYER	CONSISTENCEY/DENSITY/INFERRED STRENGTH	LONGSECTION HATCH COLOUR
PF	PRELOAD FILL	PRELOAD SURCHARGE FILL	–	–	–	
UF	UNCONTROLLED FILL	–	–	–	–	
F	EMBANKMENT FILL	ENGINEERED ROAD EMBANKMENT	"FINE" GRAINED/COHESIVE	1	VARIABLE	
			"COARSE" GRAINED/COHESIVE	2	VARIABLE	
Qh	HOLOCENE ALLUVIUM	NORMALLY CONSOLIDATED DEPOSITS FROM ACTIVE STREAM CHANNELS AND LOWER TERRACES, TIDAL FLATS, UNDIFFERENTIATED COASTAL PLAINS, BEACH RIDGES	"FINE" GRAINED/COHESIVE	1A	VERY SOFT OR SOFT	
				1B	FIRM	
			"COARSE" GRAINED/COHESIVE	2A	VERY LOOSE OR LOOSE	
				2B	MEDIUM DENSE OR DENSE	

Figure 1: Preload Area A Inferred Stratigraphy

2.2 GEO-SECTION 3 (COMPRISING PRELOAD AREA B)

Geo-section 3 covers approximately 2.5 km of the motorway alignment and contains Holocene deposits around the approaches to Nundah Creek. Preload Area B covers a portion of this section north of Nundah Creek in an area comprising the section's deepest Holocene deposits. Geotechnical investigation data indicates that the compressible Holocene alluvium extends further west than mapped extents and extends up to depths of 8 m below natural ground level with inferred stratigraphy illustrated on Figure 2.



UNIT	GEOLOGICAL ORIGIN	GEOLOGICAL DESCRIPTION	MATERIAL TYPE	SUB LAYER	CONSISTENCEY/DENSITY/INFERRED STRENGTH	LONGSECTION HATCH COLOUR
UF	UNCONTROLLED FILL	–	–	–	–	
F	EMBANKMENT FILL	ENGINEERED ROAD EMBANKMENT	"FINE" GRAINED/COHESIVE	1	VARIABLE	
			"COARSE" GRAINED/COHESIVE	2	VARIABLE	
Qh	HOLOCENE ALLUVIUM	NORMALLY CONSOLIDATED DEPOSITS FROM ACTIVE STREAM CHANNELS AND LOWER TERRACES, TIDAL FLATS, UNDIFFERENTIATED COASTAL PLAINS, BEACH RIDGES	"FINE" GRAINED/COHESIVE	1A	VERY SOFT OR SOFT	
				1B	FIRM	
			"COARSE" GRAINED/COHESIVE	2A	VERY LOOSE OR LOOSE	
				2B	MEDIUM DENSE OR DENSE	

Figure 2: Preload Area B Inferred Stratigraphy

3 EARLY-WORKS GROUND TREATMENT AND MONITORING

The early-works embankments were designed based on a nine-month preload period for embankment consolidation, using preliminary site investigation data, proposed design road finish levels for the motorway upgrade and preliminary modelling of settlement and embankment stability. Areas of preload (up to 1.0 m above proposed design level for Preload Area A and up to 2.0 m above proposed design level for Preload Area B) were included and wick drains were installed in several sections to shorten the drainage paths within the compressible deposits and, therefore, accelerate the consolidation process under the preload. Construction of the early-works embankments was completed in August 2015.

The early-works preload embankments contained the following instrumentation from which monitoring data was obtained:

- Settlement Plates (25 for Preload area A and 36 for Preload area B);
- Surface Displacement Markers (22 for Preload area A and 30 for Preload area B);
- Inclinometers (3 for Preload area A and 7 for Preload Area B);
- Vibrating Wire Piezometers (paired with each inclinometer).

4 BACK ANALYSIS OF EARLY-WORKS PRELOAD AREAS

During the detailed design phase of the project all settlement plate data were reviewed. Back analysis was carried out using the results of settlement plate monitoring data from the early-works, to refine the assessment of the values of various parameters controlling embankment settlement. Key areas selected for back analysis were based on an assessment of the settlement plate data, ground conditions (i.e. thickness of soft and firm clay) and the amount of preloading which had been applied. Generally, the selected settlement plates exhibited the largest observed settlements within the respective preload areas. Consideration was also given to providing coverage and spread across the extents of each preload area, whilst ensuring that the areas of deepest soft clay were also included in the assessments. In total, 18 of 25 settlement plates in Preload Area A and 19 of 36 settlement plates in Preload Area B were back-analysed. At the time of back analysis, the early-works preload had been in place for approximately 10 months.

Symbols and units used for various geotechnical parameters are indicated in Table 1.

Table 1: Parameter Symbols and Units

Parameter	**Symbol**	**Units**
Bulk Unit Weight	γ_b	kN/m^3
Initial Void Ratio	e_0	-
Initial Vertical Effective Stress	σ_v'	kPa
Compression Index	C_c	-
Compression Ratio	$CR = C_c / (1+e_0)$	-
Re-compression Index	C_r	-
Recompression Ratio	C_c / C_r	-
Secondary Compression Index	C_α	%
Coefficient of Secondary Compression	$C_{\alpha\varepsilon} = C_\alpha / (1+e_o)$	%
Coefficient of Consolidation (Vertical)	C_v	m^2/year
Over-consolidation Ratio	OCR	-
Pre-Overburden Pressure	POP	kPa
Settlement During Construction	SDC	mm
Post Construction Settlement within 40 years of pavement construction	PCS	mm

4.1 ONE DIMENSIONAL SETTLEMENT MODELLING

Golder's in-house computer program PCON was used for the back analysis. PCON is a spreadsheet program used to calculate primary consolidation and secondary compression in response to sequential addition and/or removal of surface loads or change in groundwater level.

Primary consolidation is computed using a sub-layer formulation of Terzaghi's one dimensional consolidation theory and the idealised one-dimensional vertical strain vs. log σ_v' curve in the classical way; using the Cc/(1+e0) slope for the virgin portion and the Cr/(1+e0) slope for the unload-reload portion (taken as linear with log σ_v'). Initial over-consolidation is taken into account, as is any induced over-consolidation of sub-layers occurring during unloading. Normally consolidated (virgin) secondary compression is computed using the log-time rule from a start time of when 90% of primary consolidation is complete. Secondary compression for over-consolidated conditions is computed by a log-time rule time base transformation based on the current over-consolidation ratio.

Wick drains are taken into account by adopting an equivalent vertical C_v based on the wick drain dimensions, layout and spacing, assumed ratio of horizontal to vertical permeability for the native soil, and assumed smear zone characteristics.

For each back-analysed settlement plate for which the underlying ground conditions could be accurately deduced, the following input data was entered into PCON:

- recorded surface level history (fill history);
- soil stratification indicated by the available geotechnical investigation data;
- groundwater level indicated by the available geotechnical investigation data;
- assumed parameters for each soil type, including an estimate of the initial over-consolidation ratio interpreted from Cone Penetration Test with pore pressure (CPTu) data.

The PCON program was then used to estimate settlement vs. time until the end of the monitoring period. The measured settlements vs. time were plotted in the same graph for comparison. Some adjustments were then made to the consolidation parameters for the soils to curve-fit estimated and measured settlements to date. Adjustments to the consolidation parameters were such that the back-analysed parameters are generally consistent over the respective preload areas, as would be expected for soils of the same geological deposit.

4.2 ASAOKA METHOD

The observational method proposed by Asaoka (1978) was initially considered as part of the process to assess performance of the preload embankments. However, due to limitations with this approach as outlined below, it was only used as a method to correlate final primary consolidation with PCON models. Two main limitations of the use of the Asaoka method for this assessment are as follows and supported by Brand & Brenner (1981):

1. Forward predictions of settlement using the Asaoka method are valid only at the current preload stress levels, which in many cases do not correspond to the final design stress level.
2. The Asaoka method was developed based on a homogeneous soil layer with moderate compressibility. When there is more than a single consolidating layer (as is the case at some locations on this site), the Asaoka plot may be misleading as a layer may consolidate faster than the others because of variations in compressibility and drainage paths.

Figure 3 illustrates an example of the prediction of end of primary consolidation based on settlement plate back analysis using PCON and an Asaoka plot for the same settlement plate data within Preload Area A. The predicted end of primary consolidation settlements are comparable between methods.

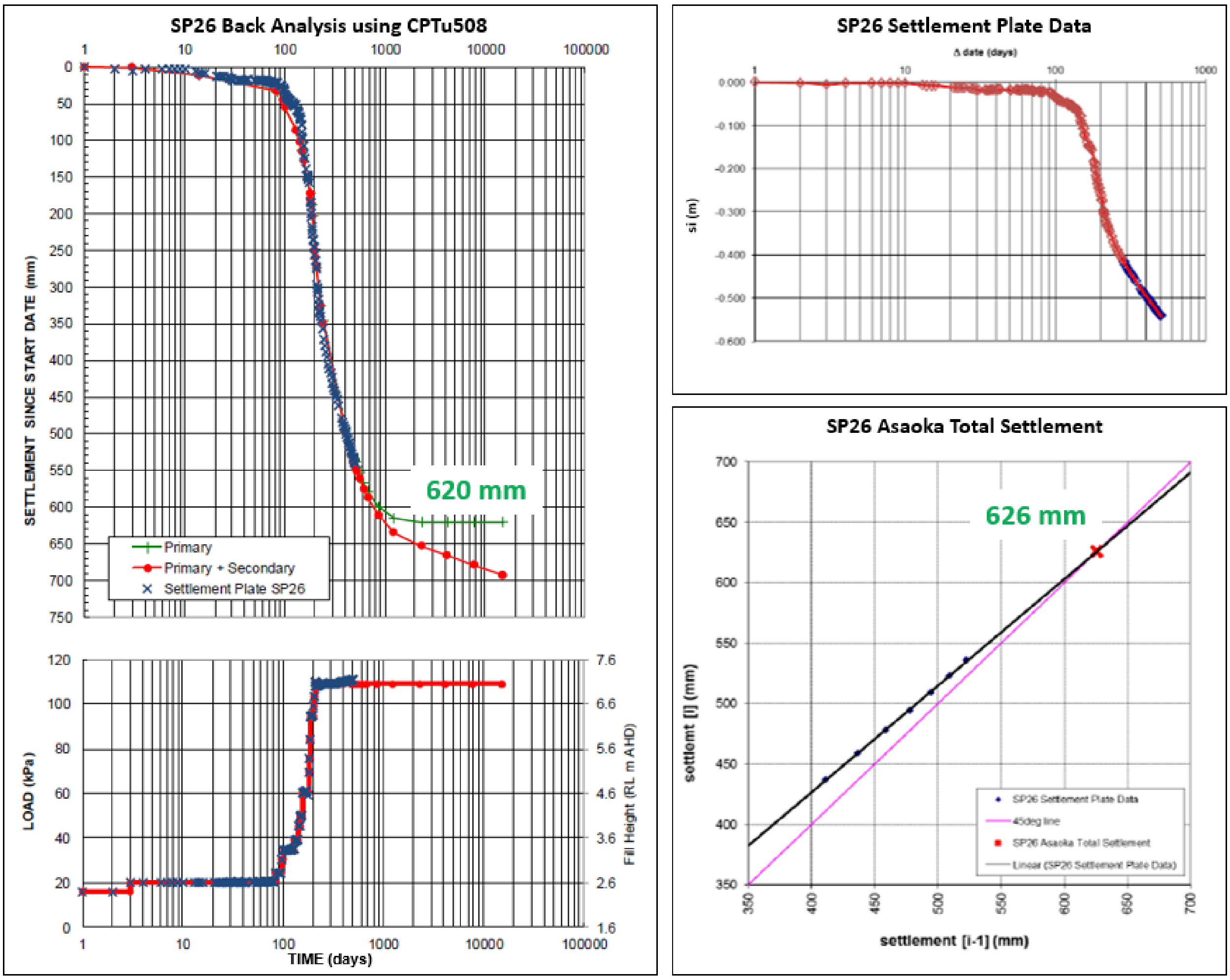


Figure 3: Settlement Plate Back analysis and Asaoka Method Predicting End of Primary Settlement

4.3 OBSERVATIONS OF SECONDARY COMPRESSION

In several cases, secondary settlement was observed in settlement plate data allowing secondary settlement parameters to be back-analysed. The relationship between back-analysed values of $C_{\alpha\varepsilon}$ and CR were within the ranges published by Mesri & Godlewski (1977) for the relevant materials. Figure 4 illustrates examples of the observed secondary compression data from settlement plates within Preload Area A and B.

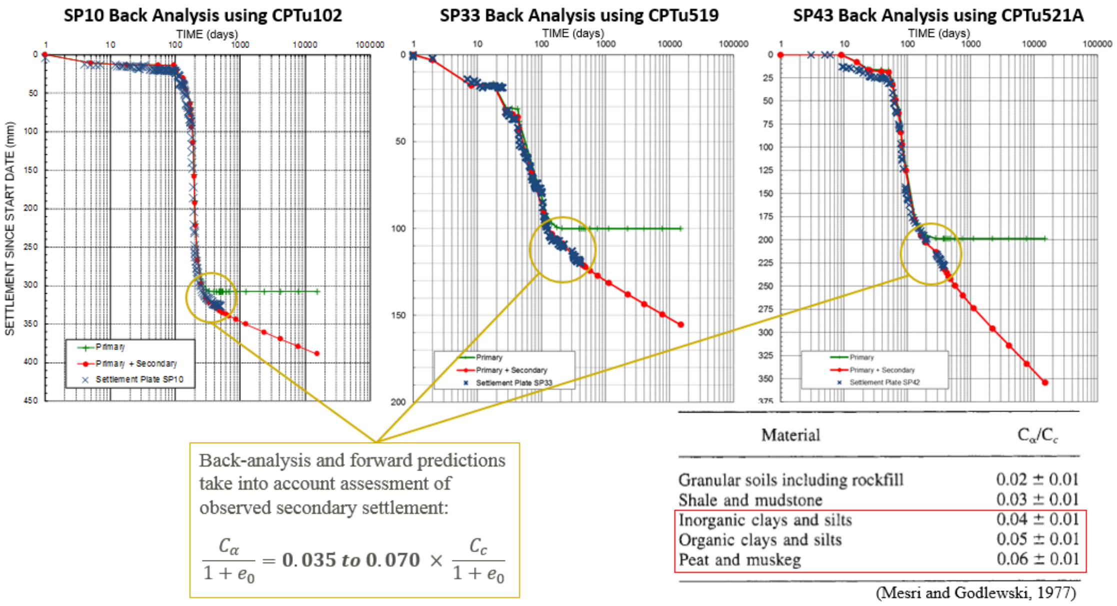


Figure 4: Observed Secondary Settlement in Settlement Plate Data

Where secondary settlement was observed, Asaoka plots were "best fitted" by two straight lines (1) and (2) shown on Figure 5 using an example from Preload Area B. Line (1) corresponds to the primary consolidation curve and line (2) corresponds to the secondary compression curve. This trend is consistent with Magnan & Deroy (1980).

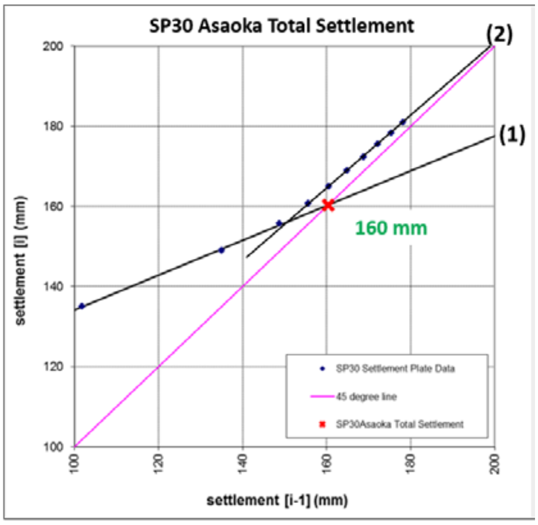


Figure 5: Trend of Secondary Compression in Asaoka Plots

4.4 BACK ANALYSIS RESULTS AND PARAMETER REFINEMENT

Table 2 summarises the range of back analysed consolidation parameters for the very soft to soft clay (Qh1A) and firm clay (Qh1B) units for each preload area and compares these to the parameters established for the early-works design and tender design phases. Consolidation parameters used for the early-works design and tender design were assessed by a combination of insitu testing including: CPTu testing and field dissipation testing; laboratory testing including oedometer testing; and, empirical correlations with index properties from historical information for the site. Final design parameters were assessed with more consideration given to the back analysed parameters from observations of actual ground performance in these areas.

The detailed design back analyses include the following features for wick drains:

- Wick drains at 1.5 m triangular centres, installed to the base of the compressible Holocene deposits;
- A value of 3 for the ratio of the smear radius to the wick drain radius;
- A value ranging from 2 to 5 for the ratio of insitu permeability to smeared soil permeability;
- A ratio of 2 for the horizontal to vertical rate of consolidation.

Table 2: Summary of Back Analysis Results and Parameter Refinement

Preload Area	Unit	Project Stage		γ_b	CR	C_c/ C_r	C_v [1]	$C_{\alpha\varepsilon}$	OCR[2]	POP[3]
A	Qh1A	Early-works Design		16.5	0.30	5	3	1.2	1 - 5	N/A
		Tender Design		16.5	0.28	6	3	0.98	3	75
		Detailed Design Back analysis	Min	16.5	0.25	7	3 (0.8 - 1)	0.88	1	N/A
			Max	16.5	0.30	7	4	1.05	2.7	N/A
	Qh1B	Early-works Design		16.5	0.30	5	3	1.2	N/A	N/A
		Tender Design		18	0.23	6	5	0.805	4	100
		Detailed Design Back analysis	Min	18	0.23	7	5	0.81	N/A	50
			Max	18	0.25	7	5	0.88	N/A	100
B	Qh1A	Early-works Design		16.5 - 17.5	0.20 - 0.30	5	3 - 20	0.8 - 1.2	1.2 - 2	N/A
		Tender Design		16.5	0.23	6	5	0.805	2.5	75
		Detailed Design Back analysis	Min	16.5	0.18	7	12 (7)	0.75	1.5	N/A
			Max	16.5	0.23	7	15	0.92 (0.99 - 1.76)	4.2	N/A
	Qh1B	Early-works Design		16.5 - 17.5	0.20 - 0.30	5	20	0.8 - 1.2	2 - 10	N/A
		Tender Design		18	0.18	6	8 (15)	0.63	3	100
		Detailed Design Back analysis	Min	18	0.15	7	12 (7)	0.53	1.9	N/A
			Max	18	0.18	7	15	1.12	7.5	N/A

Notes: 1. Values in brackets ( ) represent parameters which are applied to localised areas or materials only.
2. OCR applied to a specific area is based on locally specific values correlated from nearby CPT data.
3. POP applies to the upper Qh1A / Qh1B layer, OCR applies to the lower Qh1A / Qh1B layer.

In most cases, the parameters used for the initial design phases were generally within the ranges determined from the back analysis; however, a few exceptions were determined which had significant impact on the predicted settlements for the detailed design phase. These exceptions were:

1. A slight reduction in CR for Qh1A and Qh1B units in Preload Area B.
2. A slight increase in the ratio of C_c / C_r for Qh1A and Qh1B units in both preload areas.
3. A slight increase in C_v for Qh1A unit Preload Area A, with a localised area of significantly reduced C_v for Qh1A unit.
4. A significant increase in C_v for Qh1A and Qh1B units for Preload Area B.
5. A localised area of significantly increased $C_{\alpha\varepsilon}$ for Qh1A unit for Preload Area B.

Figure 6 shows adopted parameters along with the results of laboratory and insitu testing, as well as empirical relations.

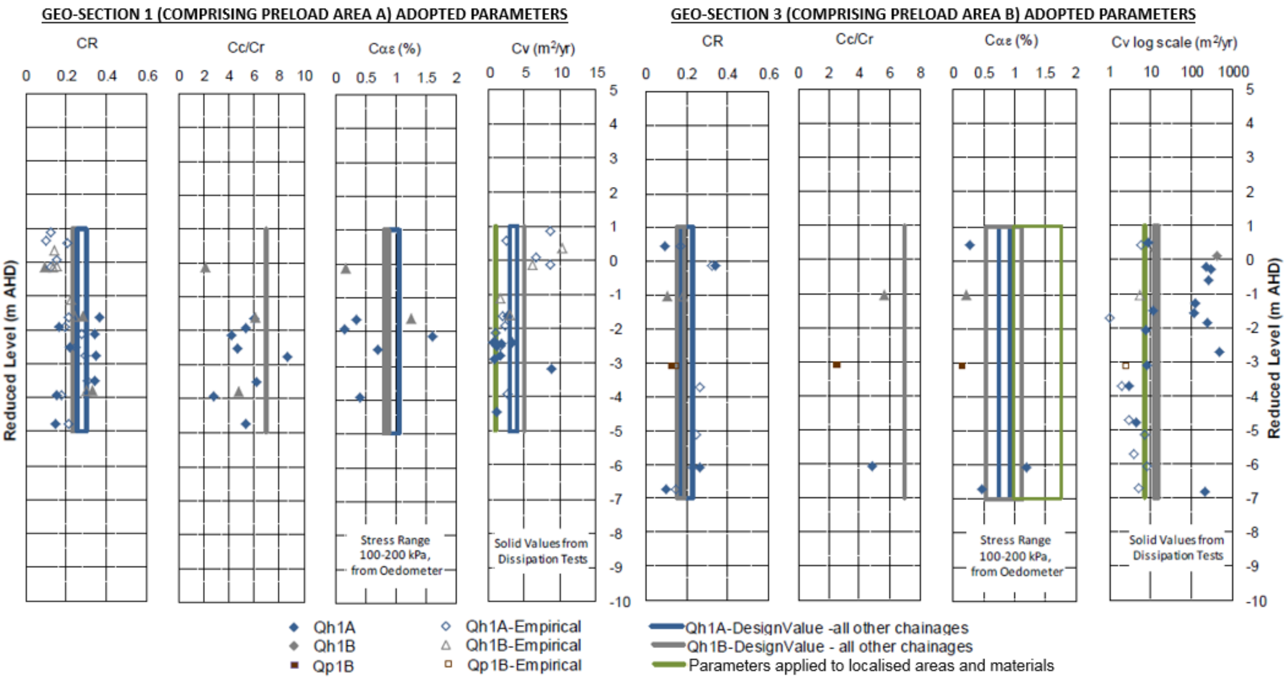


Figure 6: Adopted Parameters, Laboratory and Insitu Testing and Empirical Relations

5 OPTIMISATION OF DETAILED DESIGN BASED ON EARLY-WORKS

Early-works embankments were designed based on the settlement criteria for general embankment areas (maximum post construction settlement (PCS) of 200 mm) and did not consider the more stringent settlement criteria for structure zones (maximum PCS of 50 mm), nor the criteria for differential settlement (maximum differential settlement at any time limited to 0.5% change of grade over any 5 m length of pavement). As such, during detailed design, some further ground treatment was required within both preload areas as well as areas of new embankment construction.

Within the early-works preload areas, consolidation parameters derived from the nearest settlement plate were adopted for the settlement analysis and design of any further ground treatment. Outside of early-works preload areas, but within the relevant geo-sections, moderately conservative parameters from the refined back analysed range were adopted. Over-consolidation ratio and/ or pre-overburden pressure applied to a specific area of interest was based on locally specific values using nearby CPTu data. Whilst the back analysis of the consolidation parameters for the preload areas specifically relate to the geo-sections in which they are located, parameters used for other areas of compressible clays within the project were modified based on comparison of index properties (moisture content and Atterberg limits) of the soft soil deposits in the two preload areas.

In-service settlement within 40 years of pavement construction were forward predicted for the final design load using PCON. Where required, ground treatment comprising preloading for periods up to 6 months were applied to limit predicted settlement to the relevant design criteria. In some areas, post-construction settlement was predicted using the method developed by Ladd (1989). Figure 7 illustrates an example of forward predictions of settlement approaching a piled culvert structure in Preload Area A which uses the back analysed parameters from the early-works settlement plate monitoring data. In this instance, no further ground treatment was required for predicted post-construction settlement (PCS) to be within the design criteria limits.

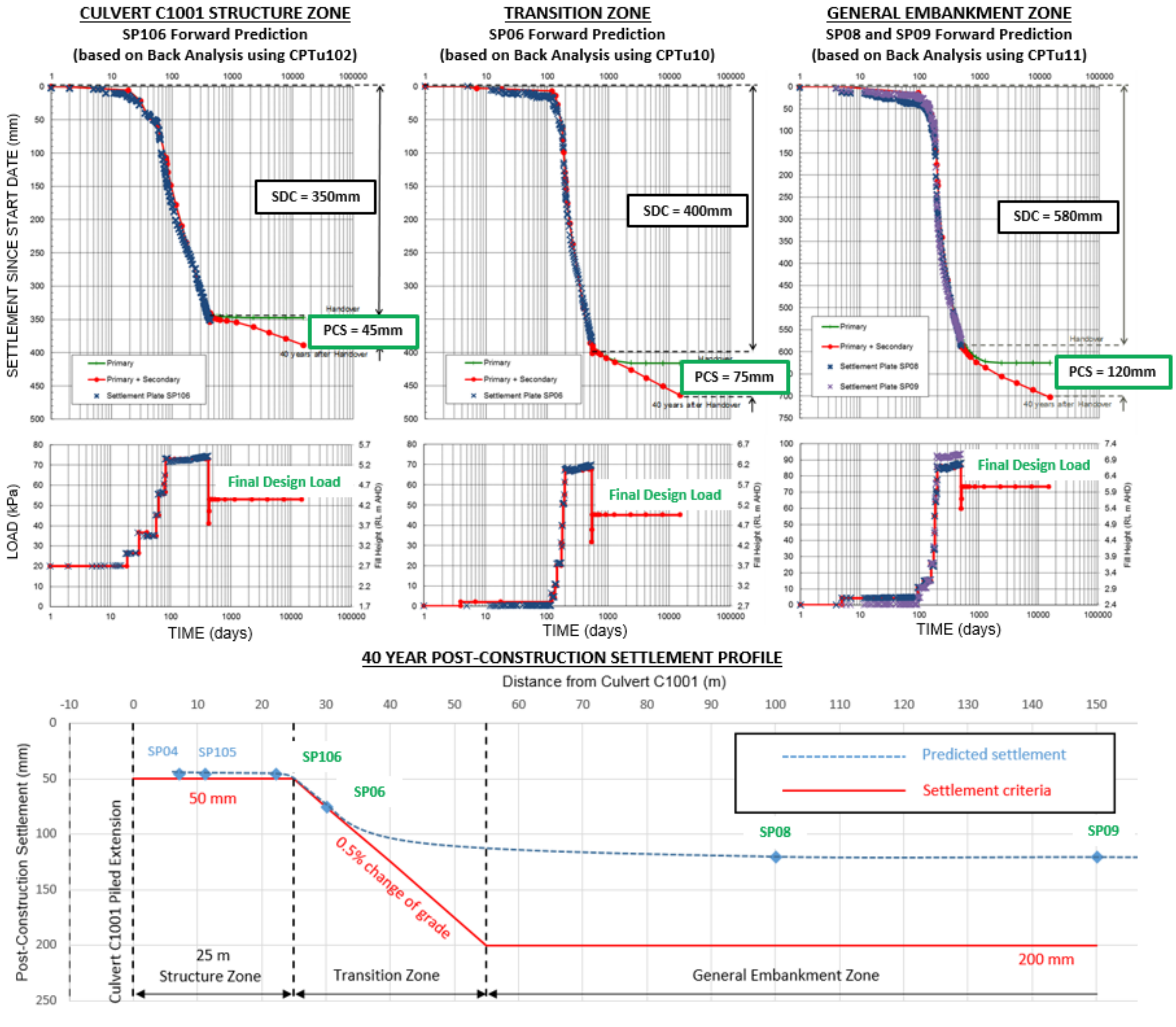


Figure 7: Forward Predictions of Settlement for Detailed Design Based on Back Analysed Parameters

Areas of further preloading as recommended during the detailed design phase were instrumented through construction phase to monitor that actual settlements were within predicted values. Figure 8 illustrates an example of settlement monitoring of an additional preload recommended for the structural zone of a piled culvert within Preload Area B.

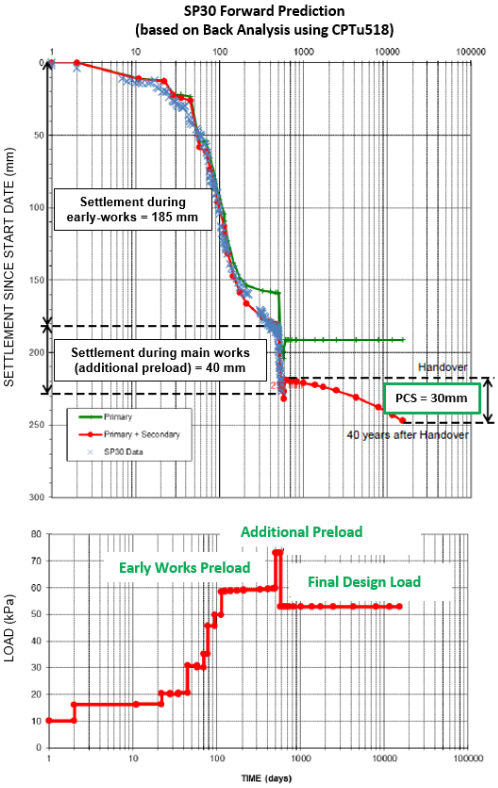


Figure 8: Monitoring of Additional Preloading through Construction Phase

Several benefits for the project resulted from the parameter refinement during the detailed design phase. For areas where the design C_v was improved, preloading was more effective with the time available and the design team was able to justify omission of “structure zone” and “transition zone” rigid inclusions adjacent to piled culvert extensions proposed in the tender design. Instead, the detailed design adopted high-level transition mattresses comprising high strength geogrid to smooth out minor differential settlements.

6 DESIGN CHALLENGES

Several challenges were encountered for the embankment design of the motorway upgrade. Firstly, as the existing motorway needed to stay operational during the construction, planned temporary traffic switches affected the construction staging and available time for construction of preloads. Timings for such ground treatment were considered during the detailed design and alternative methods of ground improvement including remove and replace were considered for areas where time did not permit preloading. In other cases, preloading under temporary traffic loading was incorporated into the ground treatment design.

Another challenge facing the embankment design was the interaction between the original embankment and the new embankment widening. As the original embankment had been in place since the 1980s, underlying Holocene clay materials had consolidated under the load of the embankment and the in-situ properties had changed compared to the properties of the adjacent virgin ground. Published information by Litwinowicz & Smith (1988) on observed settlement of the existing embankment at the Nundah Creek western approach was used to validate the back-analysed and adopted settlement parameters. Differential settlement between the footprint of the old embankment and the newly widened section were considered in the design. For critical cross-sections, complex two-dimensional modelling using Plaxis and Settle 3D software was undertaken.

The embankment widening also involved the widening of many large, multi-cell culvert structures. In the early-works preload areas, the culvert widenings were piled structures with invert levels matching the existing, non-piled structures. Differential settlements in both the transverse and longitudinal sections were considered during the design. A geogrid transition mattress incorporating multiple layers of high strength, low strain structural geogrid reinforcement was designed and constructed perpendicular and parallel to the road alignment in areas of piled culvert extensions to comply with the maximum differential settlement criteria.

7 CONCLUSIONS

The settlement behaviour of embankments in areas of soft soils are difficult to predict as it depends on many factors including the thickness of clay deposits, the rate of consolidation, the compressibility and the stress history of the material. Assessment of in-situ, laboratory testing and empirical relations aid in developing an understanding of these parameters; however, this can result in a wide range of possible parameters and can lead to very conservative or unrealistic assumptions used for detailed design. By observing real settlement behaviour of soft soil areas, design parameters can be refined to enable more accurate estimations of future settlement and allow a more cost-effective embankment design. Back analysis of settlement data also assists in identifying localised problem areas which allows an opportunity for further ground treatment to be recommended during the detailed design phase.

This is demonstrated in a case study for the Gateway Upgrade North project where one-dimensional consolidation analyses, along with Asaoka's method have been performed to back analyse soft clay parameters beneath early-works preload embankments.

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