RESOLVING MAJOR DISCREPANCIES BETWEEN PREDICTED AND MONITORED SETTLEMENTS OF A HIGHWAY EMBANKMENT ON SOFT SOIL

David Watson

Geotechnical Engineer, WSP Pty Ltd, Level 27, 680 George Street, Sydney, NSW 2001; Australia, Ph (61)292725280; email: david.m.watson@wsp.com

ABSTRACT

This case study discusses settlement predictions and how they compare to monitored settlements at an embankment constructed over soft soil forming part of the Woolgoolga to Ballina Pacific Highway upgrade on the north coast of New South Wales. The embankment height ranged from 3.4 to 7.7 metres with wick drains installed across most its 410-metre length. Settlement predictions ranged from 125 to 300 millimetres with a predicted construction and waiting time of 1.5 years. Settlement plates, inclinometers, magnetic extensometers, hydrostatic profile gauges and vibrating wire piezometers were installed to monitor settlements, pore pressures and embankment stability. It was found that after 3 months of construction and 9 months of combined construction and consolidation monitoring the soft soil had reached 95 to 100% consolidation, however the monitored settlement had exceeded predictions by up to nearly 300% in some areas of the embankment while in other areas the monitored settlement was less than 20% of prediction. Site investigation data was scrutinised and back-analysis was performed to match the monitoring data in a one-dimensional consolidation model. Back analysed parameters were compared to design parameters and possible reasons for the discrepancies in settlements and consolidation rates were discussed.

1 INTRODUCTION

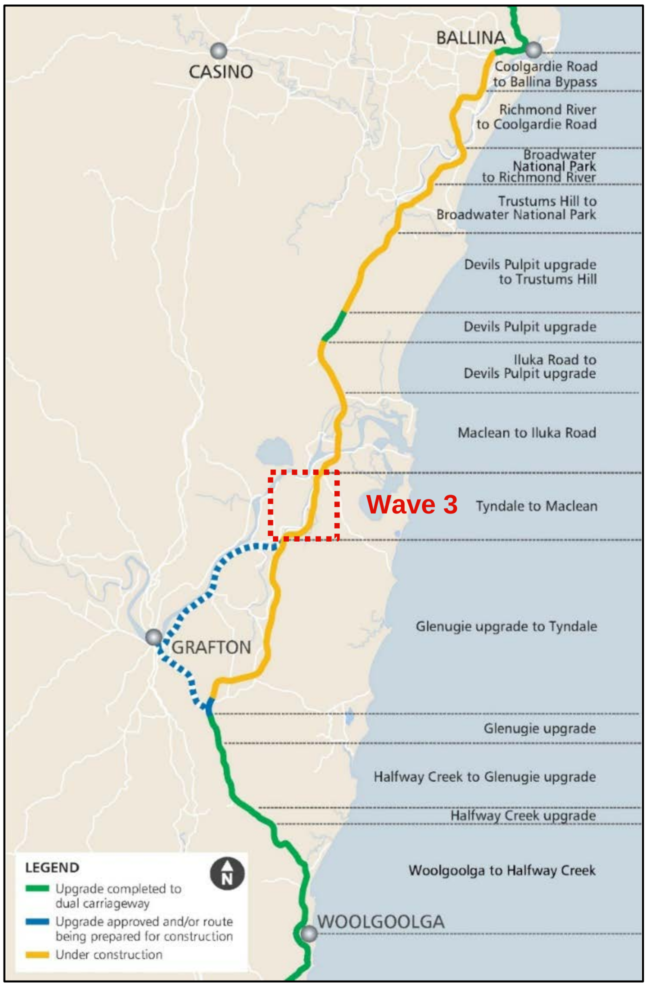
The 155 kilometres between Woolgoolga and Ballina is the last highway link between Hexham and the Queensland border to be upgraded to four lanes. Roads and Maritime Services (RMS) and Pacific Complete are working together to deliver the 126 kilometres between Glenugie and Ballina. The 24 kilometres between Woolgoolga and Glenugie was opened to traffic in 2018. The Woolgoolga and Ballina Pacific Highway upgrade (W2B) is jointly funded by the Australian and NSW governments. WSP is working in joint venture with Laing O'Rourke as Pacific Complete in the role of delivery partner for this project. This approach was used to oversee the construction of infrastructure for the London Olympics. It supports collaboration and innovation by bringing businesses, workers, consumers and suppliers together. It encourages the best ideas and solutions from the private sector while also drawing on Roads and Maritime's knowledge to ensure better engineering and design, customer outcomes and public value.

Of the 126km of highway being constructed between Glenugie and Ballina, 27 kilometres of the alignment is being constructed over soft ground. These soft ground areas are primarily located in the low-lying river basins of the Clarence and Richmond rivers. The soft soil areas were divided into four groups called Waves as shown in Figure 1, with the focus of this paper being an embankment located in Wave 3, located along the southern bank of the Clarence River between the towns of Tyndale and Maclean. Here, soft soil thicknesses are on average two to three metres but there are isolated paleochannels with soft soil depths of up to a maximum of twenty metres.

From a geotechnical perspective, the role of Pacific Complete has been to maintain and monitor the instrumentation throughout the project, carry out back analysis of soft soil monitoring data for hold point releases and provide technical advice to contractors and consultants. It should be noted that Pacific Complete are not the designers on this project, and so the focus of this paper is on the how the monitored settlements and back-analysed results compared to the predicted values rather than how those predictions were made.

Soft soil creates many design challenges due to its high compressibility, low strength and low permeability, leading to high settlements that take a long time to consolidate. The Pacific Highway guidelines specify strict residual and differential settlement criteria on all embankments in the project, limiting movement to as little as 50mm over 40 years at bridge abutments ranging up to 200mm for flexible pavements on general embankments. These strict long term settlement criteria can only be met by using ground improvement techniques such as preload, surcharge, wick drains and Concrete Injected Columns.

Figure 1: Map of the W2B project, with Wave 3 soft soil areas shown in red



Along the entire W2B alignment, over 2900 instruments have been installed, including over 900 settlement plates, 400 inclinometers, 300 vibrating wire piezometers and 90 extensometers. With such an extensive monitoring network Pacific Complete developed an innovative online monitoring dashboard that provides all the database management and graphical interpretation for the 1.9 million individual readings taken to date throughout the project. This data is also automatically checked against several alert and alarm levels for inclinometer movement to provide early warning for embankment instability and has allowed for safe construction and time saving interpretation of monitoring data (Zhang et al., 2016). Settlement plate readings were regularly checked on the dashboard against target fill thicknesses and predicted settlements to determine when a soft soil embankment had achieved a sufficient degree of consolidation (DOC) to have its consolidation hold point released. In some cases, the predicted settlement and the estimated consolidation time differed significantly from what was monitored.

One such case was in Wave 3 at an embankment designated Fill 4-5. The embankment height ranged from 3.4 to 7.7 metres with wick drains installed along most of its 410-metre length. Settlement predictions ranged from 125 to 300 millimetres with a predicted construction and waiting time of 1.5 years.

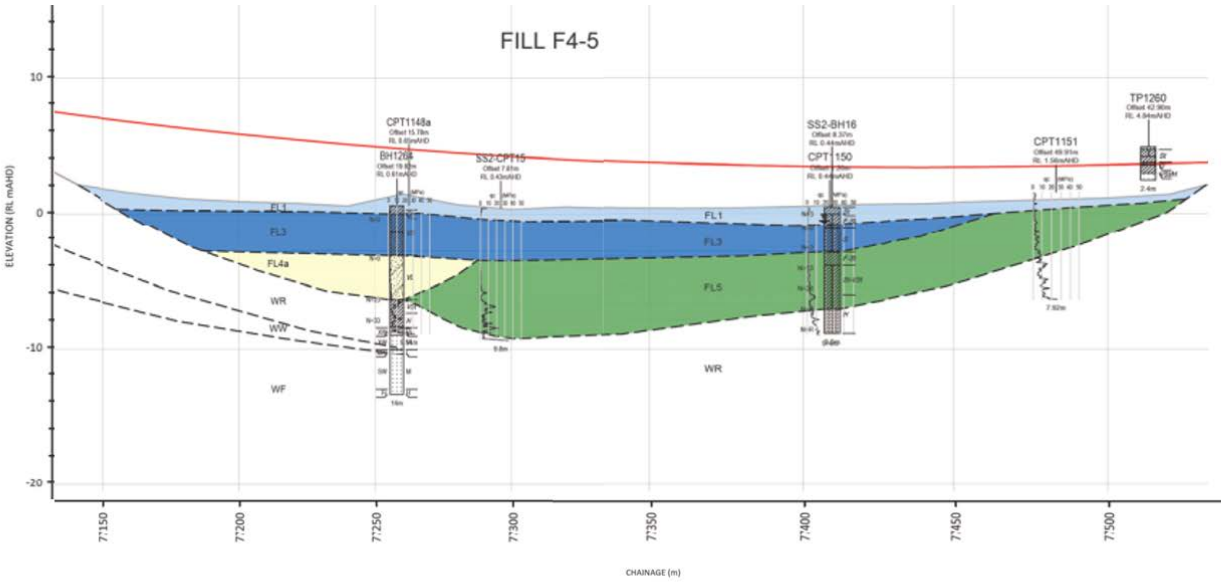
2 GROUND CONDITIONS

In general, the Wave 3 fill embankments lie within the Clarence River Basin, which forms part of the Mesozoic Clarence-Moreton Basin. This is characterised by fluvial, estuarine and marine Quaternary sediments deposited on the

Clarence River and Shark Creek floodplains overlying Walloon Coal Measures comprised of thinly bedded sandstone, carbonaceous siltstone/claystone and minor coal (Coffey, 2017).

Fill 4-5 lies between two cut sections through to low-lying hills comprised of residual soil through overlying rock of the Walloon coal measures formation. In the trough between these two hills, which forms the base of the fill embankment, lie layered alluvial sediments overlying weather rock. The geotechnical model derived from the site investigation defined four distinct geotechnical soil units beneath the fill. Based on the design geotechnical model, a top layer of Holocene desiccated surface soil comprised of sandy clays and medium dense sands (designated FL1) with a thickness of 1- 3 metres overlies a layer of very soft to firm estuarine clay (FL3) with a thickness of 0 – 4 metres thick. This soft clay layer overlies Holocene estuarine/alluvial medium dense to dense sand and clayey sand (FL4a) in the southern end of the embankment and overlies stiff to hard Pleistocene clay (FL5) in the centre and northern sections of the fill. A geotechnical long section can be found in Figure 2.

Figure 2: Geotechnical long section (AECOM, 2015)



The site investigation for the design of Fill 4-5 comprised two boreholes, four Cone Penetrometer Tests (CPTs) and one test pit. Analysis of the data from this investigation was the basis for the soft ground design parameters for the four geotechnical units as presented in Table 1. Design parameters were derived using primarily CPT and dissipation testing supported by laboratory testing on samples from boreholes. Schmertmann and Casagrande methods were both used to approximate consolidation parameters.

Table 1: Design parameters for materials at Fill 4-5

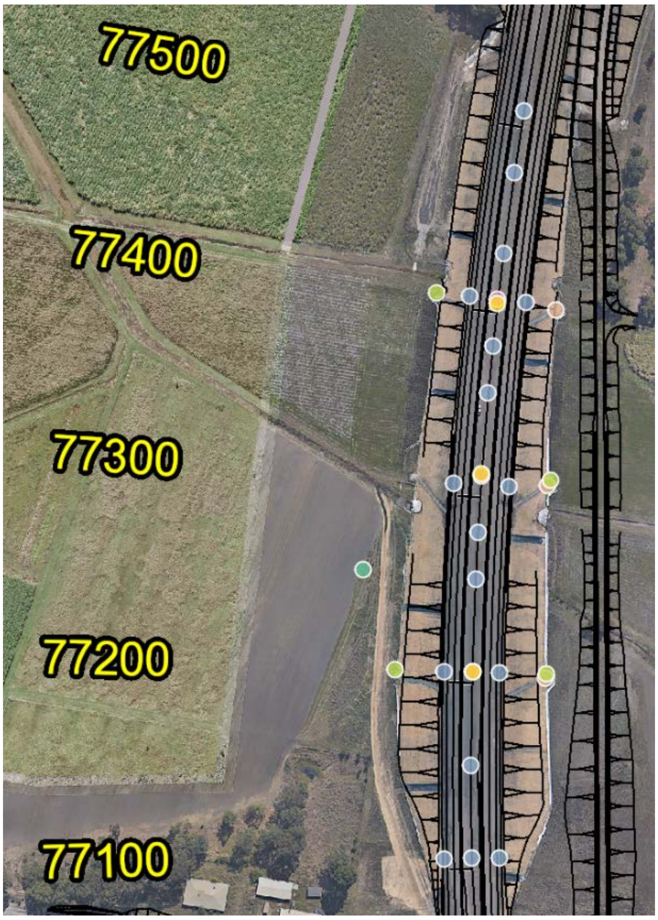
Geotechnical unit	General Description	Bulk unit weight γ (kN/m^3)	OCR	CR (Cc/1+e0)	RR (Cα/1+e0)	Cv (m^2/year)
FL1	Medium dense sand	17	45	0.15	0.03	25
FL3	Very soft to firm clay	16	2.2 – 6.4	0.15 – 0.25	0.03 – 0.04	3
FL4a	Medium dense to dense sand	18	7	0.15	0.01	50
FL5	Stiff to hard clay	17	11 – 24	0.21	0.03	25

OCR – Over-Consolidation Ratio, CR – Compression Ratio, RR – Recompression Ratio, Cv – Coefficient of vertical consolidation, assumed to be 0.5 * Ch (coefficient of horizontal consolidation)

3 CONSTRUCTION AND MONITORING

Filling began at Fill 4-5 on 4 November 2016 at a rate of up to one metre per week until 28 November 2017, when filling paused for two months. Filling continued in three one-month stages until full height was reached on 14 August 2017. Seventeen settlement plates, four inclinometers, three hydrostatic profile gauges (HPG), three magnetic extensometers and four vibrating wire piezometers (VWPs) were installed in the embankment to monitor its performance. A plan of the instrumentation installation at Fill 4-5 can be found in Figure 3.

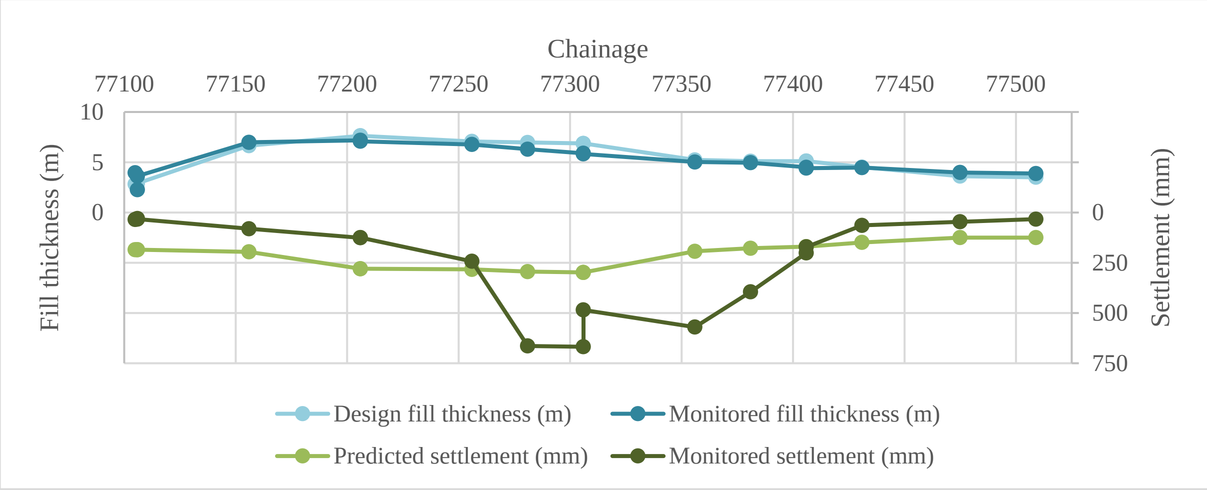
Figure 3: Plan of geotechnical instrumentation installed in Wave 3 Fill 4-5



Blue - settlement plate, Yellow/Dark green – VWP, Purple – extensometer (installed at the same locations as yellow VWPs), Light green – inclinometer, Brown - HPG

All seventeen settlement plates were monitored weekly throughout the filling and waiting periods. Monitoring data was uploaded to the online dashboard by the surveyor, which allowed for close monitoring by project engineers and designers. Monitored values were continuously compared to design predictions of settlement. It was found that at the centre of the embankment the maximum recorded settlement reached 668mm, compared to a prediction of 298mm, while at the southern edge of the embankment, maximum monitored settlements were as little as 30mm compared to a predicted settlement of 185mm. A long section of settlement and fill thickness is displayed in Figure 4. The design construction and waiting period was set to be 1.5 years; however, it was found that settlement had apparently ceased within three months of the full fill thickness being reached, or a total of 9 months after start of filling. The hold point was released in late October 2018, six months earlier than prediction.

Figure 4: Long section of settlement and fill thickness at Wave 3 Fill 4-5



Note: points of apparent discontinuity in settlement is caused by multiple settlement plates located at the same chainage, with different offsets from the control line.

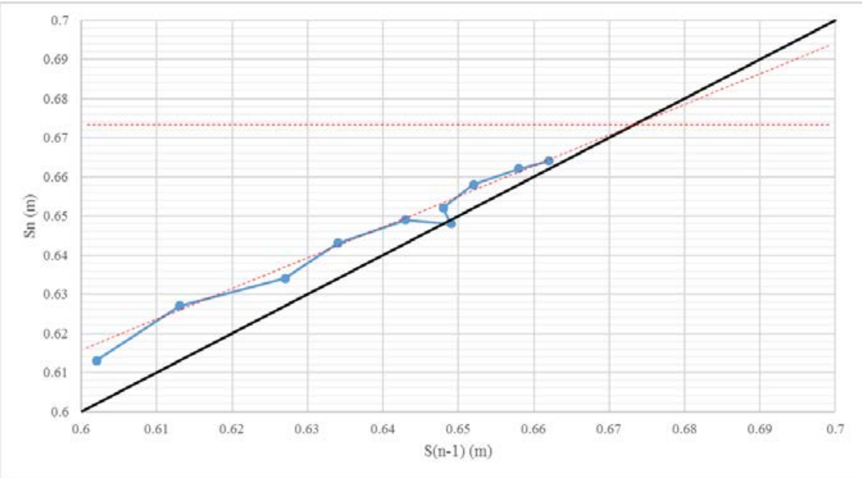
4 BACK ANALYSIS

Once the embankment had reached full design height and the rate of settlement had reduced to a level that appeared to be close enough to completion to warrant releasing the consolidation hold point, back-analysis was performed on five critical settlement plates to ensure that the Pacific Complete residual and differential settlement criteria were met.

4.1 BACK ANALYSIS FOR DEGREE OF CONSOLIDATION

Both the Asaoka method (Asaoka, 1978) and the inflection point method (Robinson, 1997) were used to estimate the total primary settlement under the surcharge load based on site settlement data. As the filling rate was uneven during construction, implementing the inflection point method was considered unreliable. Despite this, the results from the inflection method were within +/- 5% of the Asaoka method results. It was found that after 12 months from the start of filling, the embankment had reached 98 – 100% degree of consolidation (DOC) by the Asaoka method, which was higher than the design DOC of 76 - 90%. An example Asaoka plot can be found in Figure 6, the angled red broken line indicates the B line of best fit, while the horizontal broken line indicates the intersection of the black 1:1 line with the B line, and shows a predicted 100% DOC settlement of 673mm.

Figure 5: Asaoka plot of example critical settlement plate



Note: Sn – Settlement in metres at current time-step n (e.g. 7 days). S(n-1): settlement in metres at time the previous time step (n-1).

4.2 BACK ANALYSIS FOR COMPRESSION RATIO

Magnetic extensometer data was used to back-analyse Compression Ratio (CR) values that vary with depth. This was done by isolating the settlement observed between each of the extensometer magnets. The initial distance between the magnets was determined and one dimensional consolidation theory was used to back calculate CR for each individual layer. This analysis was only performed on soft soil layers and resulted in CR values of 0.05 – 0.193. These values for CR were used to guide the back-analysis for Total Residual Settlement (TRS).

4.3 BACK ANALYSIS FOR TOTAL RESIDUAL SETTLEMENT

The geotechnical model and geotechnical design parameters for the TRS back-analysis were initially based on the design model for Wave 3 and the available geotechnical investigation data. Back-analysis for TRS was firstly carried out by estimating settlements based on these initial parameters. Where the prediction was either higher or lower than the monitored settlements, the fill thickness, OCR and consolidation parameters (CR) were adjusted until the estimated primary settlement matched the monitoring records or slightly higher.

The back analysed values found for CR in section 4.2 were found to be underestimated where the monitored settlement exceeded the modelled predictions, and so the upper value (CR = 0.193) was used as a starting point for refining the consolidation parameters. Where the predicted settlement was greater than monitored, the back-analysed values for CR were used. With no reliable data to the contrary, the soft soil depth was assumed to be the same as the design model, and so OCR and CR was adjusted from the predicted values so that the estimated settlement matched the monitored settlement when adjusted for DOC. Once the parameters were adjusted in the model the resulting TRS was checked against the design performance criteria.

The estimated TRS is comprised of two components:

- Residual primary settlement (any remaining primary settlement to achieve 100% degree of consolidation under the final embankment load, without any surcharge).
- Creep settlement

When sufficient surcharge is applied and the monitored settlement under the surcharge load is greater than that caused by the final embankment load it is reasonable to assume that there will be no residual primary settlement in the pavement. The results of back-analysis accounting for the surcharge applied are presented in Table 2 while the results of the back-analysis without surcharge are presented in Table 3.

Table 2: Result of back-analysis (surcharge)

Chainage	Critical Settlement Plate ID	Available Surcharge[1] (m)	Monitored Settlement (mm)	Back Analysed Settlement (mm)	DOC in Back Analysis (%)	Back Analysed TRS[2] (mm)	TRS criteria[2] (mm)
77256	A-SP182-0	2.2	243	244	99	4 (0 + 4)	100
77281	A-SP183-0	2.2	664	672	99	6 (0 + 6)	
77356	A-SP186-0	1.1	570	580	98	22 (0 + 22)	
77406	A-SP188-0	0.5	201	209	96	11 (0 + 11)	
77475	A-SP194-0	0.6	46	46	100	0 (0 + 0)	

1. Available surcharge = Current fill height – finished design surface level
2. Total Residual Settlement over 40 years. The value shown in brackets is the breakdown of residual primary and creep settlements.

Table 3: Result of back-analysis (no surcharge)

Chainage	Critical Settlement Plate ID	Monitored Settlement (mm)	Back Analysed Settlement[1] (mm)	DOC in Back Analysis (%)
77256	A-SP182-0	243	157	100
77281	A-SP183-0	664	527	
77356	A-SP186-0	570	486	
77406	A-SP188-0	201	180	
77475	A-SP194-0	46	43	

1. Predicted settlement at 100% primary consolidation without surcharge using the same design parameters as in Table 2.

Table 3 indicates that at all critical locations the monitored settlement under surcharge load exceeds 100% primary settlement under final embankment load (no surcharge), therefore at these locations there are no residual primary settlements expected.

Differential settlement and grade change have been checked using the back analysed TRS estimates presented in Table 4 and the and compared to the Pacific Highway Design Criteria at critical analysis points along the embankment. The predicted differential settlements and changes in grade were found to be far less than the maximum allowable values.

Table 4: Differential settlement checks

Chainage	Critical Settlement Plate ID	Chainage (m)	Back Analysed TRS (mm)	Differential Settlement[1] (%)	Change in grade[2] (%)	Differential Settlement Criteria (%)
77256	A-SP182-0	77256	4	-	-	0.3
77281	A-SP183-0	77281	6	<0.01	0.01	
77356	A-SP186-0	77356	21	0.02	-0.04	
77406	A-SP188-0	77406	11	-0.02	-0.02	
77475	A-SP194-0	77475	0	-0.04	-	

1. Differential settlement between adjacent analysed settlement plates
2. Absolute change in grade between each differential settlement value

4.4 BACK-ANALYSIS FOR COEFFICIENT OF VERTICAL CONSOLIDATION

Back-analysis to estimate the coefficient of vertical consolidation (Cv) was undertaken using one-dimensional consolidation theory and by Asaoka's method at each critical location. The estimated degree of consolidation from the analysis by Asaoka method, consolidation time and drainage conditions were input into the 1D and Asaoka formulae to back calculate the Cv values. Single drainage was assumed for all locations due to the presence of wick drains, and Cv was assumed to be 0.5*Ch, an adopted ratio developed using data from extensive testing throughout the project. Consolidation time was calculated to be half the time from start of filling to full height + time since full height was reached. Cv was not back-calculated for A-SP194-0 as the magnitude of settlement was so low, the soil is not considered to be soft soil and is apparently elastic in behaviour. Back calculated values of Cv compared to design values are given in Table 5. In this case study, the ground consolidated at a faster rate than was predicted, it is expected that the true Cv of the soft soil is significantly higher than the design value of $3m^2$/day. As such it is expected that Cv calculated by 1D consolidation theory is more accurate than by Asaoka method because the results more closely mirror what was expected from the monitored settlement rate.

Table 5: Back calculated values of Cv

Chainage	Critical Settlement Plate ID	Cv (m^2/day) (by 1D Consolidation theory)	Cv (m^2/day) (by Asaoka)	Design Cv (m^2/day)
77256	A-SP182-0	4.5	5.0	3
77281	A-SP183-0	5.5	3.8	
77356	A-SP186-0	5.5	3.1	
77406	A-SP188-0	6.0	2.9	
77475	A-SP194-0	NA	NA	

5 COMPARISON WITH PREDICTIONS

To assess the performance and accuracy of the embankment predictions, the parameters assumed in the design were compared to the back-analysed consolidation parameters. In the centre of the embankment, it was found that the back-analysed OCR was typically lower and the CR typically the same or higher than the design, indicating that the soil is more compressible than assumed. On the northern and southern ends of the embankment, the CR was lower than design and OCR higher or equal to design, indicating that the soil is less compressible than assumed. Design and back-analysed consolidation parameters are compared in Table 6.

The performance of the model can be broken up into two sections: the areas where the monitored settlement exceeded the design indicating that the soil was softer than predicted (Chainages 77260 – 77406) and the areas where the monitored settlement was less than the design indicating that the soil was stiffer than predicted (Chainages 77100 – 77260 and 77406 – 77510). At chainage 77406, A-SP188-0 had only slight adjustments made to the CR, with the design and back-analysed soft soil parameters being nearly identical, which indicates that the design values were very close to the monitored values.

Table 6: Design and back-analysed soft soil thickness and design parameters

Critical Settlement Plate ID	Chainage (m)	Design soft soil thickness[1]	Thickness used in back analysis[1]	Design CR[1]	Back Analysed CR[1]	Design OCR[1]	Back Analysed OCR[1]
A-SP182-0	77256	4.0	4.0	0.15 - 0.25	0.05 - 0.19	2.2 - 6.4	3.4 - 9.8
A-SP183-0	77281	4.0	4.0	0.15 - 0.25	0.15 - 0.25	2.2 - 6.4	1.5 - 4.2
A-SP186-0	77356	3.0	3.0	0.15 - 0.25	0.23 - 0.38	2.6 - 6.4	1.6 - 3.8
A-SP188-0	77406	3.0	3.0	0.15 - 0.25	0.14 - 0.23	2.6 - 6.4	2.6 - 6.4
A-SP194-0	77475	2.0	2.0	0.15 – 0.25	0.05 – 0.09	3.3 – 6.4	4.0 – 7.7

1. For the FL3 (soft soil) layers only

5.1 AREAS WITH SOIL STIFFER THAN PREDICTED

The northern and southern edges of the embankment had monitored settlements that were much less than predicted, with the monitored settlements less than 20% of the prediction at the southern edge of the embankment (Chainage 77106). When analysing the critical settlement plates in these areas (A-SP182-0 and A-SP194-0), the parameters of all layers, both soft soil, and non-soft soil, in the model were stiffened to match the monitored settlement, indicating an absence of true soft soil in these areas.

5.2 AREAS WITH SOIL SOFTER THAN PREDICTED

The central areas of the embankment had monitored settlements that were in places greater than predicted, with the monitored settlements being as high as 300% of the prediction (at A-SP186-0 – Chainage 77356). The back-analysed CR at A-SP186-0 was almost doubled compared to the design while the OCR was reduced by as much as 50%.

Here, it has been assumed that the reason for higher than expected settlement is due to the encountered soft soils being significantly softer than in the original design, however it may be possible that there is a deeper paleochannel present at these chainages due to limited site investigation data. A review of the installation logs of nearby VWPs, inclinometers and extensometers revealed a variety of soft soil thicknesses ranging from 1.8 metres to over 5.7 metres at the same chainage. As the instrumentation logs are not as accurate as conventional boreholes, and given that broad range of values given, this data is considered to be unreliable. It is unlikely that variation in soft soil thickness between Chainages 77250 and 77410 is responsible for the discrepancies in settlements, as the monitoring results from extensometer A-EM007-0 indicated that the soft soil thickness was around 3 – 4m, which is similar to design.

5.3 DISCUSSION OF BACK-ANALYSIS

The site investigation for the soft soil design consisted of four Cone Penetration Tests (CPTs), two boreholes and one test pit as shown in Figure 2. The two most northern tests; CPT1151 and TP1260 both show a lack of soft soil which is consistent with the back-analysis. In the southern portion of the embankment, the southern-most investigation location is BH1264 which indicates a soft soil thickness of approximately 4 metres at Chainage 77250, and this was assumed to have continued to the southern extent of the fill. Without investigation data south of this, the true soft soil thickness could not have been known. Back-analysis indicates that the true soft soil thickness south of chainage 77250 is effectively zero, with back-analysed consolidation parameters more consistent with stiff soil, indicating that the soft soil paleochannel was isolated to the centre of the fill between chainages 77250 and 77410.

At the two locations where there are both CPTs and Boreholes (Chainages 77250 and 77410, see back-analysis for A-SP182-0 and A-SP188-0) the predicted settlement and monitored settlements were reasonably close, which indicates the chosen consolidation parameters were appropriate at these investigation locations. However, settlement plates further from these CPT locations show a divergence in monitored settlement from the design that increases in distance from the investigation locations.

Back-analysis from A-SP183-0, near to SS2-CPT15 shows that predicted settlement was 294mm compared to a monitored settlement of 664mm. Here, the design OCR was generally estimated to be between 2.5 – 6.4 around this chainage, which is considered high for soft soil in this region. Back analysis revealed that in this case a more conservative approach to estimating OCR in this location would have reduced the difference between monitored and design settlements. Typical OCR values for soft clay throughout the southern Clarence floodplain have been found through extensive monitoring to be closer to 1.5 – 4.0, which is just over half of the design OCR. Had these values been adopted, some settlements would be even more over-estimated, but would have prevented some of the underestimation of settlements, leading to a design with less risk.

Due to the extensive and near-real time monitoring of the embankment, the site team could closely monitor the performance of the embankment and use early intervention to adjust the design as necessary to meet the Pacific Highway design criteria.

Predicted settlements were reasonably close to monitored settlements in areas close to CPT locations, but significantly different in all other areas, indicating that there was insufficient site investigation data to adequately characterise the soils for this fill. CPT and borehole spacing for Wave 3 is generally at 100 metres intervals, which is consistent with the investigation at Fill 4-5. However, in hindsight, it would have been prudent in the investigation stage to position one borehole or CPT at a more central point in the floodplain between the two hills, (around Chainage 77325) and then position the other investigation locations 100 metres before and after this point. Or, that an additional second phase of investigation could have targeted areas with a higher risk of potential variations in soft soil thickness. Both of these options would have given a clearer picture of variability of the soft soil.

In addition to the differences in consolidation parameters, there was also a difference in the back-analysed and predicted Cv values. Cv had been set to 3m^2/day across most of Wave 3, and at Fill 4-5, led to an overestimation of the consolidation time. The borehole log for BH1264 in the southern portion of the fill indicates that the soft soil is comprised of a combination of sandy clay and silty clay, indicating a higher Cv may be more appropriate. The adopted back-analysed Cv by 1D consolidation method was up to 6.0m^2/day; while the back-analysed values by Asaoka method gave values much closer to design. The adoption of the 1D method in back analysis demonstrates the benefit of using multiple methods, where available, to approximate field parameters and using engineering judgement to determine the most realistic parameters.

6 CONCLUSION

Back-analyses on the measured settlement of a pre-loaded embankment over soft soil were conducted to assess the performance of the embankment and the underlying soft soil and to verify the design predictions. The monitored settlement was found to exceed predictions by approximately 300% at the centre of the embankment, as well as settle more rapidly than predicted.

Soft soil design is inherently difficult, and achieving realistic predictions of settlement under a surcharge load is a persistent challenge in embankment design. Case studies such as this highlight the importance of monitoring, back-analysis, checking design assumptions against reality and using the lessons learned for future projects. Back-analysis revealed that the OCR was overestimated in the centre of the embankment while CR was also underestimated in the centre of the embankment and over estimated at the edges. The reasons for this discrepancy can be at least partially attributed to a high variability in soft soil thickness and compressibility, which was not identified due to limited site investigation data available. Cv was also underestimated, likely due to the higher sand and silt content in the soft alluvial clay layers. Refined soft soil parameter were presented which can be used to more accurately predict settlements in similar conditions on future projects. This case study also demonstrates the benefits of using multiple methods to determine soil parameters, and the use of engineering judgement to determine which method is most appropriate on a case by case basis.

It would be recommended that additional CPT, geophysics and/or borehole data be collected in areas with a risk of encountering highly variable subsurface conditions such as regions known for soft soil paleochannels in order to develop more accurate settlement predictions. However, as has been demonstrated in this project, a rigorous and detailed instrumentation and monitoring program can be used to readily adjust design predictions as construction continues. Geotechnical site investigations are limited by their nature, and the cost of additional monitoring with reasonable contingencies in place may be lower than additional investigation boreholes. Engineering judgement should be used to balance the financial and geotechnical risks associated with additional investigations in order to get more accurate settlement predictions versus the accepting a less accurate prediction, and having a more extensive monitoring program with flexibility available in the program to adapt to changes in design.

An extensive and agile monitoring network like that used in the Woolgoolga to Ballina Pacific Highway upgrade project allowed for thorough and detailed back-analysis, while also minimising consolidation waiting times due ease of access to interpreted data. It can be used to rapidly adapt to adjust design predictions on the fly and respond rapidly to areas of embankment which are not behaving as predicted.

7 ACKNOWLEDGEMENTS

The author would like to acknowledge the support of the Roads and Maritime Services and the Pacific Complete delivery partner in the preparation of this paper, as well as Henry Zhang and Robert Kingsland, for their review and advice. The views presented in this paper are those of the author and do not necessarily reflect those of the supporting organisations.

8 REFERENCES

AECOM, (2015), “Early Works Detailed Geotechnical Design for Wave 3 Soft Soil Sites SS02 to SS04”, Report to Roads and Maritime Services NSW, Reference: W2BEW-REP-SS00-GT-121A-FD_05, 04/12/2015, AECOM Australia Pty Ltd, Level 21, 420 George Street, Sydney NSW Australia

Alonso, E. E., Gens, A. & Lloret, A. 2000. “Precompression design for secondary settlement reduction.” Géotechnique, Vol 50, No. 6, 645-656.

Asaoka, A. (1978). “Observational procedure of settlement prediction,” Soils and Foundations, 18, No. 4, pp. 87-101.

Coffey, (2017), “Pacific Highway Upgrade – Woolgoolga to Ballina Additional Geotechnical investigation, Portion A”, report to Roads and Maritime Services NSW, Reference: GEC-W2B-COF-GEC-00579-GEOTLCOV24443AL-AA, 17/01/2017, Coffey, Level 19, Tower B, 799 Pacific Highway, Chatswood NSW Australia.

Geological Survey of New South Wales, (1972), “1:250 000 Maclean Geological Series Sheet SH56-7”, Geological Survey of New South Wales, Sydney

Holtz, R. D., et al. (1991), “Prefabricated Vertical Drains: Design and Performance, CIRIA ground engineering report: ground improvement’. Oxford: Butterworth – Heinemann Ltd.

Ladd, C. C. & DeGroot, D. J. (2003), "Recommended Practice for Soft Ground Site Characterization. 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering MIT Cambridge, MA USA 22-25 June, 2003

Roads and Maritime Services (2015), “Upgrading the Pacific Highway – Design Guidelines, Rev 4

Robinson, R. G., (1997), “Consolidation analysis by an inflection point method”, Géotechnique Vol 47, No. 1, 199-200

Zhang, H., Gombault, N. & Choi, B. (2016) "An Innovative Geotechnical Monitoring System for Soft Ground Treatment on W2B Pacific Highway Upgrade Project", AGS Symposium 2016.