

WOOLGOOLGA TO BALLINA PACIFIC HIGHWAY UPGRADE – EMBANKMENT PERFORMANCE MONITORING AND BACK-ANALYSIS

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ABSTRACT

The section of the Pacific Highway Upgrade project between Woolgoolga and Ballina (W2B) traverses areas having significant depths of soft soils. The presence of these soft soils causes significant risks in relation to timely project completion and potential cost blow out. To reduce these risks, soft ground treatment using wick drains and preloading was used in the form of Early Works to de-risk the project and to reduce the post construction settlement.

During the embankment construction and preloading period, instruments were installed to monitor the embankment performance. Monitoring was carried out using an innovative web-based system to enable continuous and real time monitoring of embankment performance, technical review and modifications of construction sequence as required.

Monitoring data were back analysed periodically to assess the embankment performance. Through this exercise, additional filling and stability measures were implemented over a number of embankment sections, where the monitored embankment settlements exceeded the design prediction. In addition to the innovative web-based monitoring system, advanced numerical modelling was used to predict future creep settlement using the back-analysed results. It was found that advanced numerical modelling of creep settlement must carefully consider the two-dimensional stress distribution beneath the embankment including toe berms in order to model accurately the stress history and over-consolidation effects on creep after surcharge removal. When the two-dimensional stress distribution is properly taken into account, the results of forward creep prediction matches well with one-dimensional analytical methods such as that proposed by Wong (2010).

1. INTRODUCTION

The section of the Pacific Highway Upgrade project between Woolgoolga and Ballina (W2B) traverses areas having significant depths of soft soils. The presence of these soft soils causes significant risks in relation to timely project completion and potential cost blow out. Early Works involved soft ground treatment and embankment preloading being undertaken ahead of the main upgrade construction works to reduce the cost of ground treatment and to provide certainty of delivery of the highway upgrade within a given time during the main contract. The Early Works also have other benefits such as improving long-term pavement performance and community benefits via improved safety and reduced maintenance.

Over the approximately 155 km project length from Woolgooga to Ballina, there are a total of eleven highway sections underlain by soft alluvium, with lengths ranging from about 0.5 km to 4 km. Three significant sections with deep soft soil are around Maclean, south and north of the Clarence River, and south of Ballina. At these areas, soft soil thickness ranges from 5 m up to approximately 26 m depth and extends over kilometer distance especially around Maclean and south of Clarence River areas.

To the south of Clarence River, the highway upgrade involves widening of the existing highway embankment toward north to the new bridge crossing Clarence River and construction of a new major interchange at Yamba Road.

This paper presents the process of the soft ground treatment design, instrumentation design, fill construction, monitoring and back analyses for an embankment section south of Clarence River Bridge. Lessons were learnt through the review and back analysis of the instrumentation. The advanced numerical modelling adopted to predict future creep settlement and comparison with one-dimensional analytical methods is also described.

2. PROJECT INFORMATION

Figure 1 shows the extent of areas underlain by soft alluvium along the highway upgrade alignment to the north and south of Clarence River. Figure 2 shows the proposed highway upgrade intersection to the south of Clarence River. Over this area, access ramps and new highway widening embankments up to 8.5 m high are to be constructed adjacent to the existing highway.

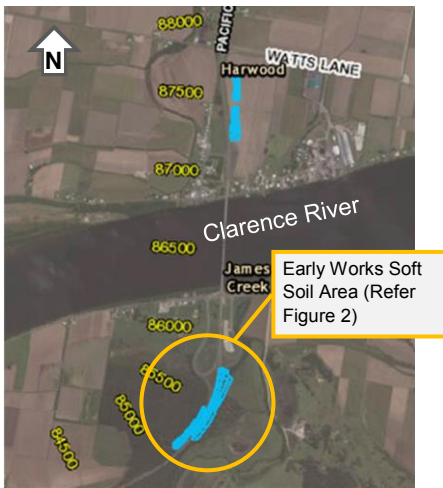


Figure 1: Location of soft soil areas south of Clarence River

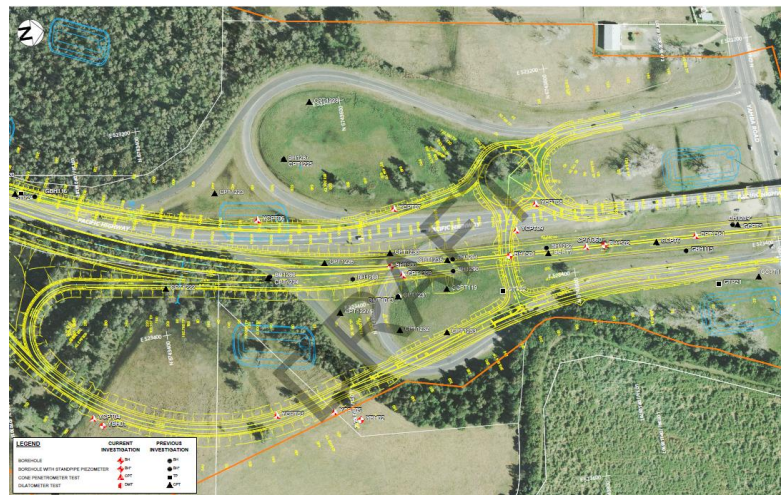


Figure 2: Proposed highway upgrade at south of Clarence River

3. GEOLOGICAL SETTING

The soft soil area to the south of the Clarence River is a large freshwater swamp associated with James Creek, which is part of the river floodplain. It has a natural ground surface elevation generally at approximately RL 0.0 m AHD and is inundated by the 1 in 20 year flood. Highway embankments over this area are underlain by Quaternary sediments, which are inferred to have been deposited as fluvial, deltaic, estuarine and marine sediments in approximately the last 200,000 years (Late Tertiary to Holocene). The sediment stratigraphy comprises recent, Holocene floodplain sediments (mud, silt, sands and local gravels) overlying extensive Holocene estuarine-deltaic sands and clays. These Holocene sediments overlie Pleistocene estuarine and lacustrine sediments (mottled clays) and basal fluvial gravels. The basal gravels are assessed to be pre-Pleistocene and directly overlie bedrock.

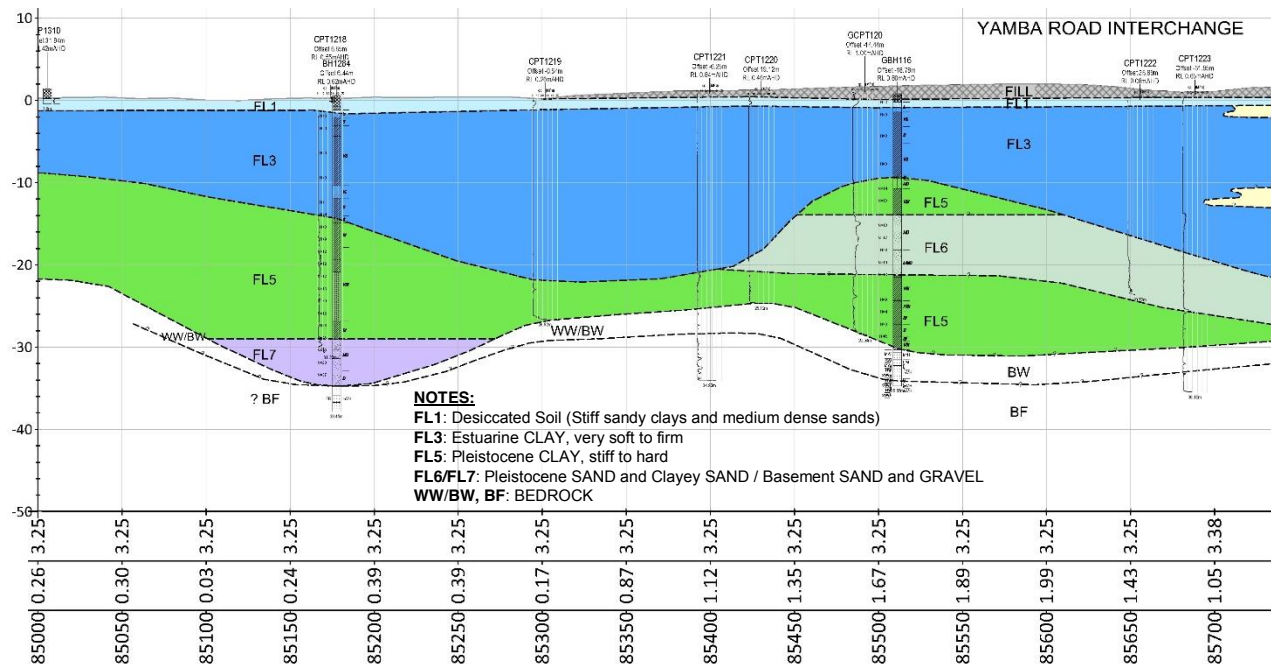


Figure 3: Geotechnical Long Section South of Clarence River (between CH 85000 and CH 85800 along the Pacific Highway Alignment)

Figure 3 shows a typical geotechnical long section along the highway main alignment (northbound lane control line) between CH 85000 and CH 85800. Soft Soil Unit FL3 is estuarine silts and clay, dark grey in color, very soft to firm, with shells and minor organics. This soft soil unit has the layer base level varying from RL -9 m AHD at CH 85000 to below RL -21 m AHD between approximately CH 85300 and CH 85400. It should be noted that this figure only shows the inferred soft soil thickness along the main alignment control line. Soft soil thickness and depth also vary transversely across the highway.

4. SOFT GROUND TREATMENT DESIGN

Soft ground treatment design and construction was carried out in advance of the main upgrade works. The design works were completed at the end of 2014 and construction of the early works commenced in Sept 2015 for target completion of the highway upgrade in 2019.

Soft ground treatment design was carried out using analytical and finite difference methods using the software CAOS (Consolidation Analysis of Soils) developed by Professor Harry Poulos with modification to include post surcharge creep reduction using the method described in Wong (2010). The design considers the following embankment preloading aspects:

- 2-dimensional stress distribution with depth under a fill embankment,
- ground consolidation with vertical drains including smear effect,
- large embankment deformation and settlement,
- time dependent settlement (creep settlement) and creep reduction due to surcharge removal and over consolidation effect, and
- Embankment construction staging.

5. INSTRUMENTATION AND MONITORING

Due to the complexity of fill placement and significant variation in the soft soil thickness, approximately 100 settlement plates, 26 inclinometers, 24 multi depth piezometers, and 13 extensometers were installed and monitored over this embankment area from CH 83400 to CH 85800.

Pacific Complete (PC), a Joint Venture of Lang O'Rourke and WSP, is the "Delivery Partner" engaged by RMS to manage delivery of the upgrade. PC developed an in-house online monitoring system (Zhang et al., 2016) because of the scale of this project with extensive soft ground treatment works. The system development was funded by Road and Maritime Services (RMS) and operated by PC.

PC set up the format and calculation process for each instrument type and the link between different instrument types, e.g. inclinometer and settlement plate, piezometer and settlement plate. Review levels were set up by the PC team to control short term slope instability and hold-point release.

PC developed the system using a data application for data uploading/editing/downloading, a Microsoft SQL Server database for all calculations and the Qlik Sense dashboard with the QlikMap GIS extension for visualisation. A screenshot of the Geotech Dashboard is shown in Figure 4 overleaf.

The tables, charts, graphs and maps used are interactively linked to the underlying data, so that selecting a project area, period, instrument type or instrument, will automatically filter the data being displayed on every control. The same session being open on multiple pages, screens or even computers and mobile devices will maintain a link to this dynamic filtering.

Features of this online monitoring system included:

- Automatic uploading using standard format for all instruments, either via email or a third-party website (e.g. VW piezometers owned by the suppliers).
- Automatic Alert System: an email was automatically sent to all relevant parties when the predetermined Review Levels were exceeded.
- A quick comprehensive review of all relevant instruments linked with a GIS Map to enable quick decision making, which is critical for short term slope stability.
- Open/flexible for further improvement.
- Automatic email reminders of late reading of any instruments.

Short term slope stability was monitored and delivered by PC adopting different trigger levels which addressed the rate of lateral displacement as shown in Table 1. With this online monitoring system, PC managed to build all the embankments safely without slip failure and 75% of all the preloading fills has been released for pavement construction on the whole project.

Table 1: Trigger levels for embankments with and without wick drains (Zhang et al., 2016)

Parameter	Wick drains		No wick drains	
	AL	WSL	AL	WSL
h_m/S (=DRc)	0.25	0.35	0.5	0.7
$\delta h_m/\delta S$ (=DRi)	0.5	0.7	N.A.	N.A.
δ_{hm}/δ_t (mm/day)	10	15	10	15
Action	Increase measurement frequency	Stop filling	Increase measurement frequency	Stop filling

Notes (for Table 1): h_m = max cumulative horizontal displacement from an inclinometer; S = cumulative settlement from adjacent settlement plate; δ = incremental; t = time in days; δ_{hm}/δ_t = daily lateral displacement; AL = Alert Level; WSL = Work Suspension Level; DRi only applies for high risk areas (Yamba and Maclean Interchanges)

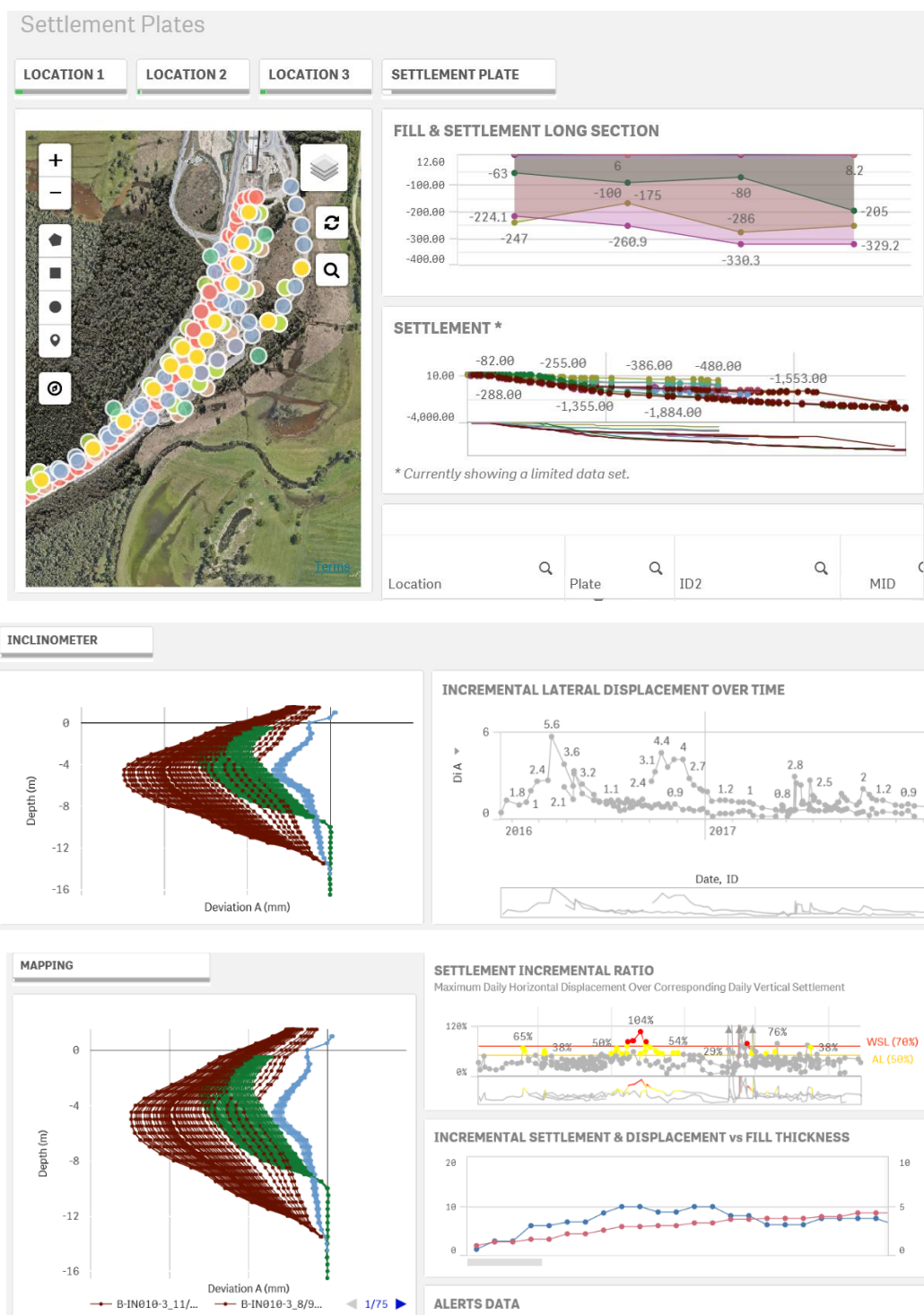


Figure 4: Web based monitoring dashboard of settlement plates and inclinometers

6. SETTLEMENT BACK ANALYSIS

6.1 TYPICAL EMBANKMENT SECTION

Back analysis results for a typical embankment section at approximate chainage CH 85290 are presented in this paper. At this location the following instruments were installed and monitored (1) Extensometer EM05007 with magnets at different depths; (2) Piezometers VP05014 at 6m, 12m, and 20m depths; (3) Inclinometer IN05015. In addition, there are

two settlement plates SP05057 and SP05058 at CH 85240 and CH 85330 respectively. The schematic of the embankment section is shown in Figure 5. A stability berm of 2 m height by 10 m width was used together with high strength reinforcement as stability measures for embankment construction.

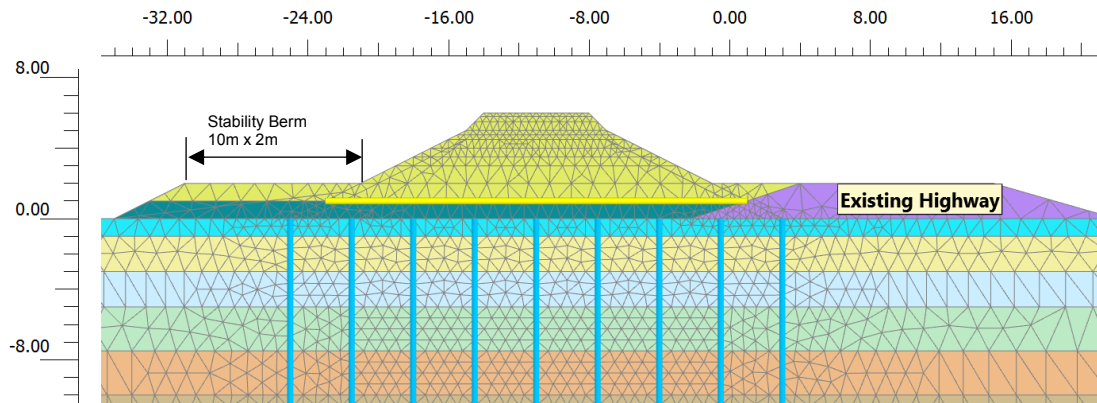


Figure 5: Geometry of Embankment Section at CH 85290

6.2 GEOTECHNICAL MODEL AND PARAMETERS

The back analysis involves settlement curve matching technique using Coffey in-house settlement software (CAOS) and PLAXIS 2D for settlement and consolidation. The primary parameters that were assessed in the settlement back analysis of the embankment performance are the compression and recompression ratios, pre-consolidation pressure, creep strain rate and the coefficient of consolidation. These parameters vary with depth so there are innumerable possible combinations of soil parameters that could match the measured ground settlement under the fill embankment. Finding the best fit parameters that matched excess pore pressure and settlement profiles with depth and embankment construction against time was extremely difficult and time consuming, and the resulting solution is by no means unique. The back analysed geotechnical model and soil parameters for ground under the embankment at CH 85290 are summarised in Figure 6.

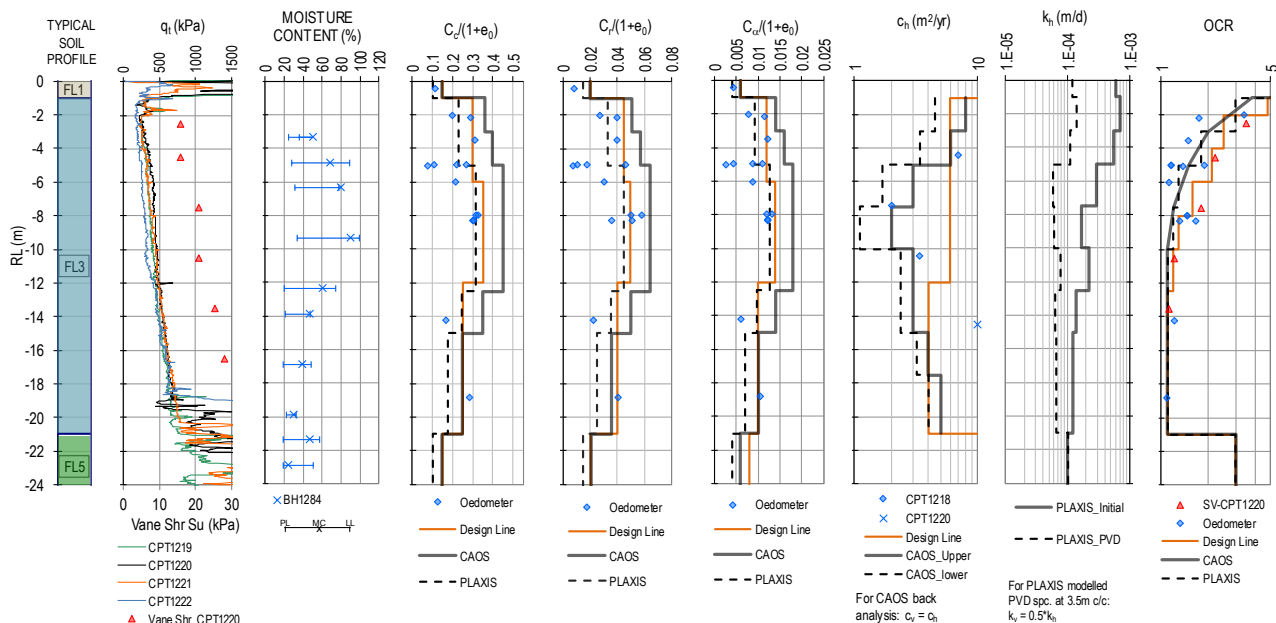


Figure 6: Back analysis parameters comparison with the interpreted design soil parameters.

The upper and lower values of the horizontal consolidation coefficient (c_h) are provided in Figure 6 for soft soil Unit FL3. These values are for soil behavior under the normal consolidated stress range. The adopted consolidation coefficient corresponding with the over consolidated stress range is 10 times the upper value of consolidation coefficient for the normal consolidated stress range. For soil Unit FL3 layer above 10 m depth, the lower value of consolidation coefficient is applied when the vertical effective stress increases more than approximately 20kPa beyond the pre-consolidation pressure. For soil Unit FL3 layer below 10 m depth, the lower value of consolidation coefficient is applied when the vertical effective stress increases more than approximately 10kPa beyond the pre-consolidation pressure. Vertical

consolidation coefficient has been assumed to be similar to the horizontal coefficient of consolidation (c_h), although the original design was based on $c_v = 0.5c_h$.

The initial ground level under this embankment section was at an approximately reduced level RL 0.0 m AHD. The initial groundwater level has been assumed at the initial ground surface in this back analysis. The groundwater level fluctuation with time has not been considered in the back analyses. Bulk unit weight of embankment fill has been assumed as 20 kN/m³.

6.3 BACK ANALYSED SETTLEMENT RESULTS

In the back analysis using CAOS, Boussinesq equations were used to estimate the embankment induced vertical stress, which is calibrated with the induced stress from a PLAXIS 2D finite element analysis for this embankment. It was observed that the analytical method provides the embankment induced stress distribution within approximately 5% - 15% of that obtained from the PLAXIS 2D analysis.

Prefabricated Vertical Drains (PVD) were installed at 1.2 m center to centre in a triangular pattern. PVD were embedded approximately 3 m in very stiff clay layer (soil Unit FL5) below the base of the soft soil unit FL3. In the back analyses using CAOS software, PVD has been modelled as an equivalent radius of 0.026 m for the band type PVD. The smear zone has been assumed to extend to a radius of 0.13 m. The ratio of permeabilities of the undisturbed to remoulded soil is 5. In PLAXIS 2D, the equivalent 2D vertical spacing was assumed as 3.5 m to account for the plane strain (2D) modelling effect. Smear effect was accounted for in PLAXIS 2D by reducing the soil permeability within the vertical drain zone. In the PLAXIS 2D analysis, vertical permeability of half the horizontal permeability for the PVD zone has been adopted to minimize the impact of vertical consolidation due to the used of relatively large equivalent 2D PVD spacing of 3.5 m, especially for the soil layers within the top 5 m depth.

Figure 7 shows the back analysed settlement results at different depths for extensometer EM05007 at CH 85290 with forward prediction of settlements. The initial design embankment fill thickness was 5 m with the corresponding design settlement of 1.5 m at the end of 1.5 year preloading period. The back analyses indicate the actual ground settlement would have been approximately 1.8 m under a 5 m design height embankment at this location. Additional fill was recommended to compensate for the settlement and to speed up the preloading process.

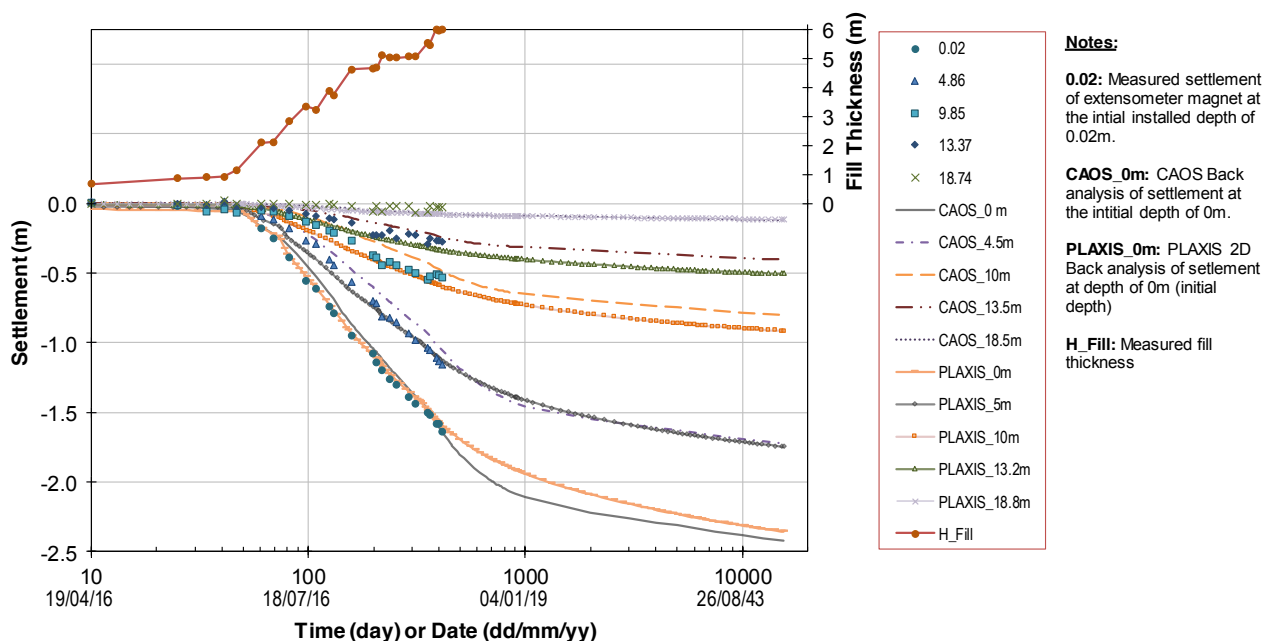


Figure 7: Back analysis of Extensometer EM05007 at CH 85290

As can be seen from the settlement curves at ground surface and at 10 m depth in Figure 7, the majority of the ground settlement (up to approximately 75% of the total ground surface settlement) occurred within 10 m depth. The remaining approximately 25% of ground settlement was due to the compression of the soil layer from 10 m to 19 m depth. Relatively minor settlement occurred in soil layers below 19 m depth. This is partly because the average embankment induced vertical stress within 10 m depth is higher than those in the underlying soil layers. The average embankment induced vertical stress within 10 m depth is approximately 1.5 and 2 times higher than the average induced vertical stresses in soil layer from 10 m to 20 m and soil layer at below 20 m depth respectively. In addition, it appears from the back analysis

soil parameters and the investigation data that the soil layers within 10 m depth are softer than the soil layers below 10 m depth.

There have been no sensible extensometer reading available at this location beyond May 2017 as extensometer EM05007 was broken due to settlement. Further embankment settlement assessment has relied on settlement plate SP05058 installed at CH 85330. Embankment geometry (width, batter slope, wick drains, and stability berm) and ground condition at CH 85330 are generally similar to that at CH 85290, except that fill thickness at this settlement plate was at maximum 5.8 m. The monitored settlement of plate SP05058 and the corresponding back analysed settlements using CAOS and PLAXIS 2D are shown in **Figure 8** Error! Reference source not found..

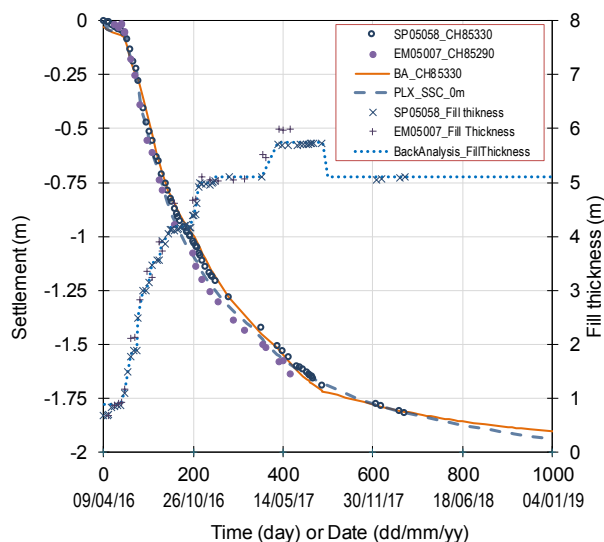
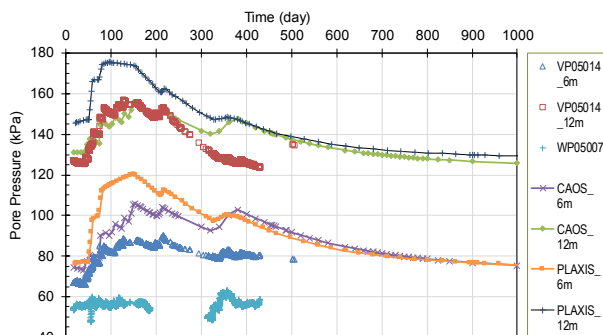


Figure 8: Back analysis of settlement plate SP05058 at CH 85330

The reduction in fill thickness shown in **Error! Reference source not found.** at 490 days was due to excavation for temporary traffic diversion but not at the completion of the preloading period. Approximately 0.7 m thick fill was excavated from the main embankment while an additional 2 m thick fill was stockpiled on the stability berm (over the whole berm width) for this embankment area. Embankment was restored to 6 m thick during June 2018 for further preloading, which has not been captured in the forward settlement prediction shown in Figures 7 and 8.

Figure 9 shows the measured pore pressure in the piezometers (VP05014) installed at 6 m and 12 m depth (installed depths below the initial ground surface) at CH 85290 and the back analysed pore pressures from CAOS and PLAXIS 2D. Groundwater table fluctuation with time was measured using a standpipe piezometer (WP05007) installed outside the embankment and stability berm footprint at CH 85180, i.e. approximately 110 m from the back analysed embankment section. The measured pore pressure in standpipe piezometer WP05007 at approximately CH 85180 are also included in Figure 9. The standpipe piezometer WP05007 was installed to 5.2 m depth below the initial ground surface level. The measured pore pressure fluctuation of up to approximately 14 kPa with time in the standpipe piezometer indicates the groundwater level fluctuation could be up to 1.4 m.



Notes:
 VP05014_6m: measured pore pressure at 6 m. Measurements started from 26 April 2016.
 WP05007 is standpipe piezometer at CH85180 installed to 5.2 m depth away from embankment footprint.
 CAOS_6m: CAOS Back Analysis of maximum pore pressure at 6 m depth;
 PLAXIS_6m: PLAXIS 2D Back Analysis of maximum pore pressure at 6 m depth

Figure 9: Measured pore pressures at CH 85290

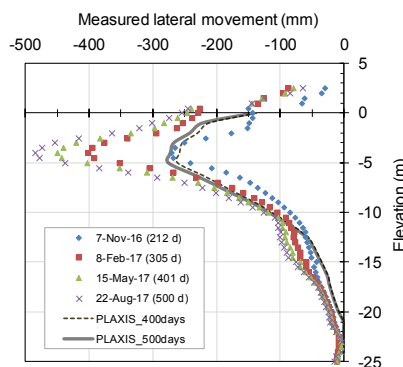


Figure 10: Lateral ground movement at CH 85290

Measured lateral ground movements versus depth at the toe of embankment fill at CH 85290 are presented in Figure 10. The measured lateral movements extended beyond 20 m depth below the initial ground surface level. Relatively large lateral movements occurred above 10 m depth. The maximum lateral movements happened at approximately 4 m to 5 m depth below the initial ground level. This indicates the soil layer within 0 m and 10 m depth could be significantly softer than the soft soil layer below 10 m depth. Overall, the measured maximum horizontal ground movement is 25% to 28% of the maximum vertical ground settlement at different times during the embankment construction and preloading period. It can be seen from Figure 10 that the lateral movement (negative implies outwards from the embankment toe) was significantly under-estimated by the numerical analyses. The authors believe that this under-estimation is caused by stress rotation effects on soil stiffness that have not been accounted for in the back-analyses.

6.4 BACK ANALYSIS PARAMETERS VS DESIGN SOIL PARAMETERS

Figure 6 shows the comparison between the back analysed compression ratios, over consolidated ratio (OCR), consolidation coefficient (c_h), and the interpreted design soil parameters for soft soil Unit FL3. The back analysed OCR and c_h are generally within the interpreted design parameters. Using CAOS, the back analysed compression ratios are approximately 30% higher than the interpreted design parameters from oedometer tests. However, using the Soft Soil Creep model in PLAXIS 2D, the back analysed compression ratios are within the oedometer test results.

It should be noted that there are more than one combination of back analysed soil parameters that could give reasonable matching with the actual settlement curves. A slightly lower OCR from 4 m to 8 m depth may also lower the compression ratios or lower consolidation ratio or permeability would result in higher compression ratios. It is possible that probabilistic and Bayesian updating methods such as those described by Huang et al (2010) and Kelly and Huang (2015) may provide better matching with quantifiable reliability of back-analyses using monitoring results.

7. CONCLUSION AND LESSONS LEARNT

Advanced instruments have been installed for embankment performance monitoring. Apart from surface settlement plates and inclinometer, extensometers and piezometers have been used for a more accurate back analysis that leads to more confidence in forwarded prediction of creep settlement.

Delicate instrument such as extensometer and piezometer are prone to being damaged during operation, when settlement or preloading time exceeds the design values respectively. They must be installed to accommodate contingency measures such as additional filling or can be operate at a significantly longer preloading time (long battery life).

The web-based monitoring system provides a tool to assist with early identifying problem area for review, assessment, and contingency measure action. Additional fill was identified for some area to compensate for embankment settlement in excess of the design and to accelerate the consolidation process.

Both conventional and advance soil models were utilized to assess and back analyse the monitored embankment performance. CAOS and PLAXIS 2D require different stiffness parameter input due to differences in the calculation approaches, particularly in the treatment of creep commencement time. However, they provide reasonably similar embankment settlement and long-term creep settlement under an embankment provided that comparable stiffness parameters have been input.

The use of CAOS for back analysis is fast and efficient to obtain a preliminary set of parameters that provide confident back analysis and prediction of embankment settlement. PLAXIS 2D with Soft Soil Creep model can be used subsequently to fine tune the back analysis and prediction to consider a more reasonable embankment filling and excavation stages. The use of PLAXIS 2D has a drawback to model consolidation with PVD and smear zone effect compared to using CAOS. Trial and error calibration of soil parameters is required to obtain the equivalent soil permeability for PVD zone in PLAXIS 2D. Future research of quantifying the reliability of back-analysed parameters and forward projection of settlement behavior may include probabilistic methods such as Bayesian updating of monitoring data.

8. ACKNOWLEDGEMENTS

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