

DESIGN, MONITORING AND ANALYSIS OF A TRIAL EMBANKMENT OVER SOFT ALLUVIAL DEPOSITS FOR THE TAURANGA NORTHERN LINK: A CASE STUDY

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ABSTRACT

The Tauranga Northern Link (TNL) is a designated 6.9km long, 4-lane road alignment for State Highway 2. The ground model is complex and across the valley floors up to 20m of geologically recent, soft alluvial sediments were encountered. Analysis of laboratory oedometer testing results was undertaken to determine suitable preliminary consolidation parameters for design. Primary and secondary settlement calculations were completed in accordance with Terzaghi 1-dimensional consolidation theory. Preliminary calculations indicated that without mitigation measures, post-construction consolidation and secondary creep settlements would exceed the NZTA Bridge Manual serviceability limit state performance criteria. To refine the design, a trial embankment was constructed adjacent to the Wairoa River where significant soft alluvial deposits were encountered, to enable back-calculation of actual consolidation parameters for the soil units using the Asaoka method and Terzaghi's theory of consolidation. Construction and ongoing monitoring results are discussed as a case study within this paper, along with comparison of the predicted versus actual design parameters and settlement results.

1 INTRODUCTION

The Tauranga Northern Link (TNL) is a New Zealand Transport Agency (NZTA) project that involves the design and construction of a 6.9km long, 4-lane road alignment for State Highway 2, along with an upgrade of the existing Takitimu Drive and addition of a new on-ramp at 15th Avenue. The ground conditions encountered across the project are complex and variable, with the road alignment crossing four main terrace features, elevated some 30 to 50 metres above intervening low-lying valley floors and separated by steeply inclined natural escarpments. To reach design subgrade levels, cut and fill depths of up to approximately 23m and 20m respectively are proposed.

A trial embankment was constructed across the valley floor adjacent to the Wairoa River following the completion of the specimen design stage, where up to 20m of geologically recent soft alluvial soils were encountered during investigations comprising interbedded layers of soft silts and loose sands. It was constructed to enable back-calculation of the consolidation parameters for these soft soils. To construct the trial embankment, borrow material was taken from a small knoll located across the valley floor and approximately 6,900m³ of fill was placed. This resulted in a trial embankment with a roughly rectangular form measuring 22m by 36m and 4m in height.

2 GROUND MODEL

2.1 GEOMORPHOLOGY

The general landform within the local Tauranga and Te Puna areas typically comprises a series of north north-east trending terraces, elevated at approximately RL35m to RL55m (Moturiki Datum) separated by deeply incised low-lying valleys at RL1m to RL3m.

The trial embankment is situated at Chainage 3750m along the TNL route within the 1500m wide Wairoa River valley and is located approximately 220m from the western bank of the Wairoa River. The average ground level beneath the trial embankment is approximately RL 1.5m.

2.2 PROJECT GEOLOGY

The Briggs et al (1996) geological report indicates that the low-lying Wairoa River valley floor is underlain by recently deposited alluvial and estuarine sediments. This was confirmed by the extensive field investigations completed during the specimen design stage. The recent alluvial deposits were split into two main categories for analyses as follows:

- Recent Alluvium – Swamp Deposits, consisting of soft organic and estuarine silts, clays and peat;
- Recent Alluvium – Sands, consisting of very loose to medium dense sands and gravels.

Beneath the valley floor, these deposits tend to be bedded and result in alternating layers of the material. This was evidenced beneath the trial embankment and this, along with accessibility, was the reason this location was chosen.

2.3 STRATIGRAPHIC UNITS

The main geological units encountered beneath the trial embankment footprint are shown in the geological section presented in Figure 1 (CMW Geosciences, GIR, 2017) and summarised in Table 1 below.

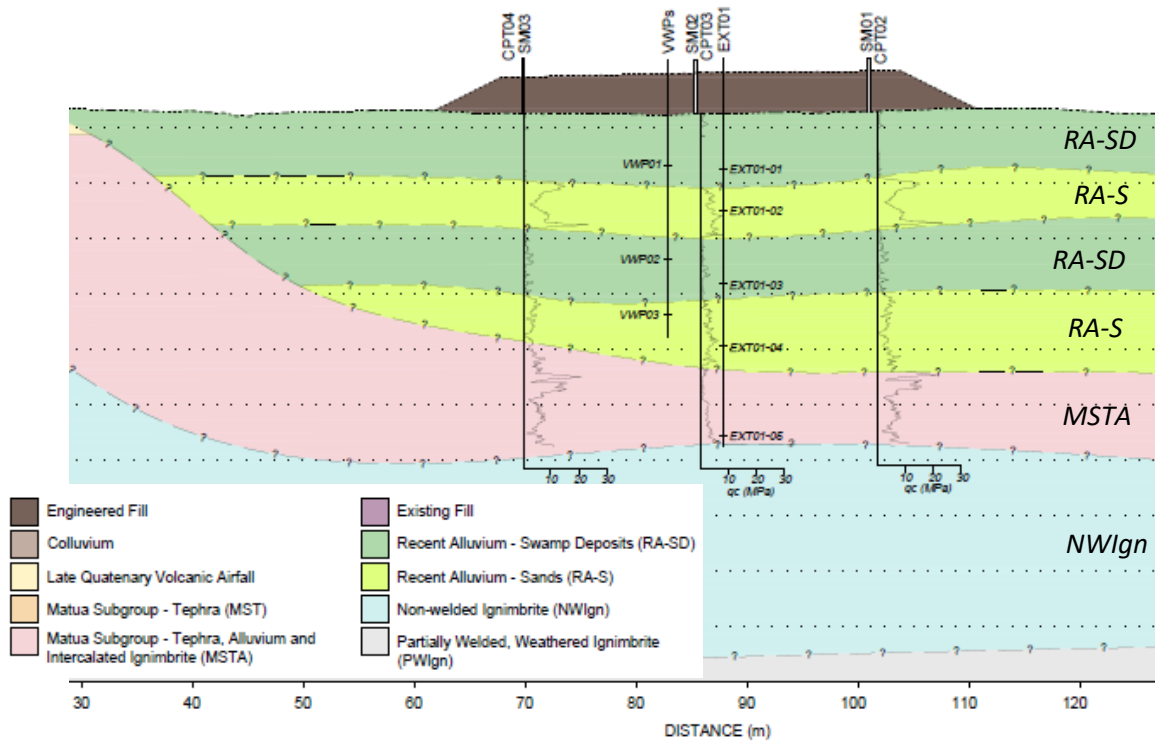


Figure 1: Geological cross section below trial embankment

Table 1: Summary of geological units below trial embankment

| Geological Units | | Approx. thickness of unit (m) | Description |
|-------------------|--|--------------------------------------|---|
| Unit | Name | | |
| Holocene alluvium | Recent Alluvium – Swamp Deposits (RA-SD) | Upper layer 5.5m Lower layer 6.5m | Interbedded estuarine and alluvial silts, clayey silts, sandy silts, organic silts and peat; generally brown, grey and blue-grey in colour. Typically very soft to soft cohesive material and very loose to loose granular materials. Variable organic content observed (none to highly). |
| | Recent Alluvium – Sands (RA-S) | Upper layer 4.5m Lower layer 6m | Sand, gravelly sand and silty sand layers. Pale grey to grey in colour. Typically loose to medium dense. |
| Matua Subgroup | Matua Subgroup – Tephra, Alluvium and Intercalated Ignimbrite (MSTa) | 7 | Interbedded clayey silts, silts, sandy silts, silty sands and sands. Typically firm to very stiff cohesive materials or loose to dense granular materials. |
| Ignimbrite | Non-welded Ignimbrite (NWIgn) | 17 | Highly to completely weathered ignimbrite. Grains typically consist of sands and silty sands. Variable strength, ranging from loose at top of geological unit to dense at greater depths. |
| | Partially welded, weathered Ignimbrite (PWIgn) | 10+ | Highly to completely weathered ignimbrite. Grains typically consist of silts, clayey silts, sandy silts and silty sands. Typically dense to very dense in granular materials or very stiff to hard in cohesive materials. |

The upper and lower RA-SD unit beneath the trial embankment were the target layers for settlement analysis. Interpretation of the five CPT results completed across the embankment footprint indicated that there was an average cone resistance (q_c) of 0.25MPa for the upper RA-SD unit, with a higher strength of 0.94MPa for the lower RA-SD unit.

The CPT traces indicate that the lower RA-SD unit has thin lenses of loose sand interbedded throughout the layer which could account for the difference in soft soil characteristics when compared to the upper RA-SD unit. It should be noted that although this is the case beneath the trial embankment footprint, it is not representative of the entire valley floor as other CPT traces reviewed during the specimen design process of the TNL project showed a softer lower RA-SD unit that more closely matched the upper RA-SD unit.

2.3 GROUNDWATER

Groundwater levels were measured within a selected number of the investigation boreholes completed during the specimen design stage of the TNL project and relevant piezometers were reviewed to give an average groundwater level of RL1.1m at the trial embankment location.

3 INSTRUMENTATION INSTALLATION

The instrumentation installation consisted of:

- Three settlement markers installed along the alignment to monitor ground settlements in response to the placement of fill;
- Two vertical inclinometers installed at the toe of the embankment to measure the deflection of soils below the embankment toe and to assess vertical vs horizontal deformations;
- One horizontal inclinometer installed across the centre of the embankment, perpendicular to the TNL alignment, to measure the cross-sectional differential settlements beneath the embankment;
- Three vibrating wire piezometers (VWPs) installed at various depths within a single 30m deep pre-drilled and cased borehole in the centre of the embankment. The VWPs were designed to be installed within the RA-SD and RA-S units;
- One extensometer, with five magnets, installed at the centre of the embankment using groutable anchors with spring legs. A low strength grout was used to back-fill the hole to allow settlement of the magnets with the RA-SD and RA-S units. It should be noted that the extensometer results have proved to be unsatisfactory and it is assumed that installation was not sufficiently implemented. Therefore, the extensometer results are not discussed further within this paper;
- 11 survey monitoring marks installed to allow monitoring of the sensitivity of ground movements offset from the embankment toe in response to the placement of fill.

Construction of the trial embankment began in August 2017 and was completed in November 2017. Five CPT tests were completed across the trial embankment footprint prior to installation of the instrumentation. The layout of the instrumentation can be seen in Figure 2 below.

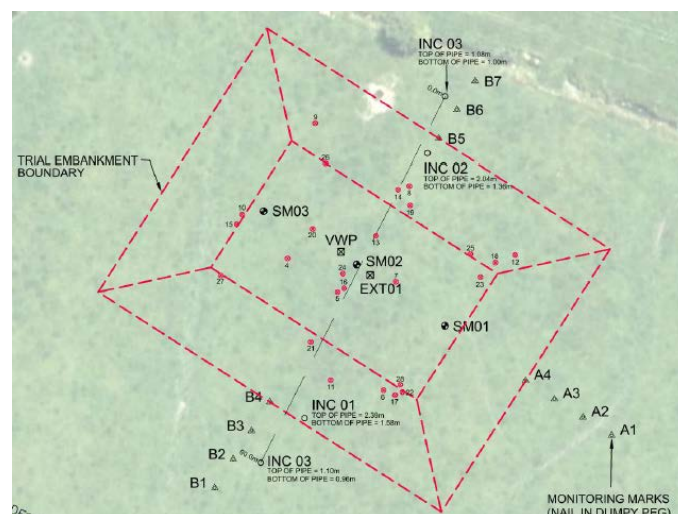


Figure 2: Instrumentation layout across trial embankment (provided by Bloxam Burnett & Olliver Ltd)

4 INITIAL GEOTECHNICAL PARAMETERS AND PREDICTIONS

4.1 LABORATORY TESTING

Laboratory testing completed during the specimen design process for the TNL project included 29 one-dimensional consolidation (oedometer) tests which were predominantly taken from within the compressible RA-SD unit. The oedometer testing was completed to obtain the necessary consolidation parameters for analysis. The tests were undertaken in an IANZ accredited laboratory in accordance with New Zealand Standard 4402: 1986, Test 7.1 and 2.1.

Oedometer tests simulate the one-dimensional consolidation that will be experienced by the in-situ soil after a surcharge load has been placed. Oedometer tests provide or allow the calculation of the coefficient of volume compressibility (m_v), the compression index (C_c), the recompression index (C_r), the coefficient of consolidation (C_v), the initial void ratio (e_0), the specific gravity (G_s), the initial water content (w_i), and the initial bulk density (ρ_b). A summary of the results of the lab testing and conservative parameters used in design are presented in Table 2 below and reference was also made to the in-situ CPT dissipation tests completed during the GIR stage.

Table 2: Summary of consolidation design parameters

| Parameter | Test Range | Design Value |
|---|--------------|--------------|
| Compression Index (C_c) | 0.05 to 1.02 | 0.60 |
| Recompression Index (C_r) | 0.02 to 0.12 | 0.08 |
| Initial void ratio (e_0) | 1.4 to 3.2 | 2.3 |
| Specific Gravity (G_s) | 2.3 to 2.72 | 2.6 |
| Secondary / Primary Compression Index ratio (C_d / C_c) | - | 0.05 |
| Coefficient of vertical consolidation ($C_v - \text{m}^2/\text{year}$) | 3.0 to 111 | 8.0 |
| Coefficient of horizontal consolidation ($Ch - \text{m}^2/\text{year}$) | 3.9 to 131 | 35 |

Results of the laboratory oedometer testing show that most of the RA-SD samples were near-normally consolidated as the over-consolidation ratio (OCR) was close to 1 for most samples. This means that for the proposed fill embankment pressures, the soils will consolidate following the virgin compression (C_c) line.

4.2 INITIAL SETTLEMENT PREDICTIONS

An assessment of static settlements was completed for a range of fill embankment heights constructed over the RA-SD unit within the Wairoa River valley.

Primary consolidation settlements were assessed in accordance with Terzaghi's 1-dimensional consolidation theory. Secondary creep settlements were estimated based on Mesri *et al.* (1994), which considered a design life of 100 years in accordance with the NZTA Bridge Manual version 3.2 (BM3.2) and a construction period of 18 months. It was assumed that local borrow materials would be used for embankment construction and therefore a unit weight of 16 kN/m³ was assumed for the fill surcharge.

The initial estimates for the settlement at the trial embankment are summarised in Table 3 below.

Table 3: Summary of initial settlement predictions at Chainage 3750 (trial embankment location)

| Chainage (m) | Soft Soil Thickness (m) | Proposed Road Embankment Height (m) | Construction Settlement ($t_{90} - \text{mm}$) | Post Construction Settlement (mm) |
|---|-------------------------|-------------------------------------|--|-----------------------------------|
| 3750 | 10.5 | 4.83 | 1020 | 290 |
| Note: Post construction settlements made up of secondary creep + remaining 10% consolidation. | | | | |

The time taken to reach 90% consolidation (t_{90}) was evaluated for a range of soft soil thicknesses and relevant thicknesses are summarised in Table 4 below.

Table 4: Estimated time to reach 90% consolidation (t_{90})

| RA-SD Layer Thickness (m) | Maximum drainage path for two-way drainage (m) | Time to Reach t_{90} | |
|---------------------------|--|------------------------|--------|
| | | Years | Months |
| 4.0 | 2.0 | 0.4 | 5 |
| 6.0 | 3.0 | 1.0 | 12 |

5 MONITORING RESULTS AND DISCUSSION

5.1 VERTICAL SETTLEMENT MAGNITUDE AND RATE

5.1.1 Settlement Marker Plates

The three settlement markers (SMs) were monitored regularly during the construction phase for the trial embankment and then monthly following the full height of the embankment being reached. Surveys were undertaken by Bloxam Burnett & Olliver Ltd (BBO) using GPS, with 50 observations averaged per mark and the first set of readings taken on 21 August 2018. Outputs of the settlement plots can be seen below in Figure 3.

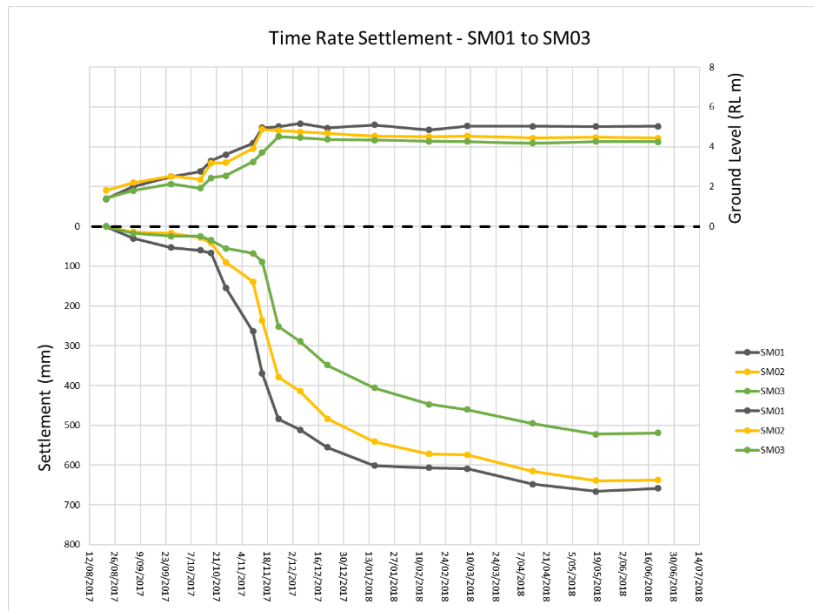


Figure 3: Settlement Marker Plots showing ground level (RL m) and settlement (mm) versus time (weeks)

The settlement plates show that approximately 520mm to 680mm of settlement has occurred beneath the trial embankment. SM01 shows the most amount of settlement, with SM03 showing the least, which correlates well with the finished contour as the trial embankment is slightly ramped up from the north-east to the south-west. All three settlement plots show a sharp drop in the settlement curve once approximately 2m of fill has been placed. This indicates that there is a fairly stiff soil response to the initial 2m of fill placement, which could indicate that the soils are actually lightly over-consolidated.

Back calculation of the pre-consolidation pressures from the laboratory oedometer tests resulted in an average of 50.8kPa. The three settlement markers started to drop off significantly once a loading pressure of approximately 38.5kPa was reached which is fairly close to the back calculated-laboratory pre-consolidation pressure. It can therefore be assumed that any loads above the pre-consolidation pressure of approximately 40kPa to 50kPa would follow the C_c compression line, whereas any below this would follow the C_r line.

5.1.2 Horizontal inclinometer

The horizontal inclinometer was monitored regularly throughout the construction period and monthly following the end of construction. However, the horizontal inclinometer did not have the zero-reading taken until approximately 1.4m of

fill had already been placed and therefore the initial settlement was not captured. Based on the other instrumentation installed, approximately 50mm of settlement would likely already have occurred at this time and so this should be accounted for when reading off the horizontal inclinometer plot. Outputs of the horizontal inclinometer readings can be seen in Figure 4 below.

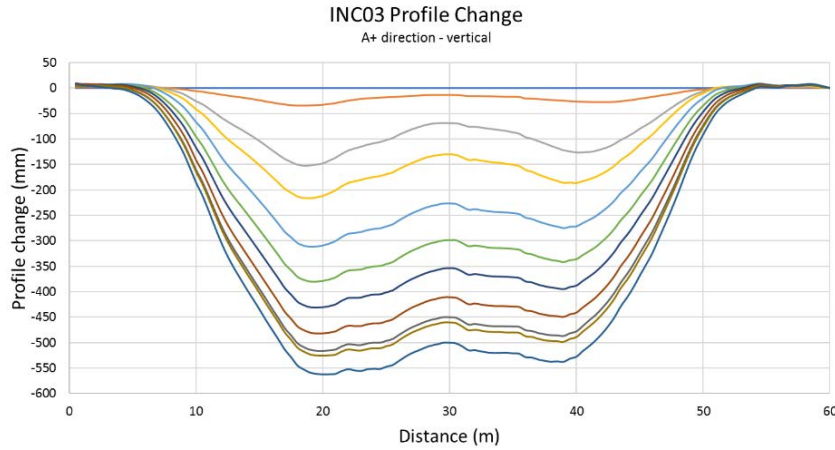


Figure 4: Horizontal inclinometer plots showing distance (m) versus movement (mm)

The horizontal inclinometer shows approximately 500mm to 600mm of settlement occurring across the centre of the embankment cross section. Within 7m of either end of the horizontal inclinometer plot minor bulging can be seen beyond the toe of the embankment.

An interesting feature to note is that during construction, two peaks were observed at approximately 18m length and 38m length. This trend continued during the monitoring process and has slightly levelled out following the completion of the embankment. The likely reason for these peaks relates to the fact that the trial embankment shoulders were constantly lifted during construction whilst the centre of the embankment was where the haul road was located. The centre of the embankment was only topped up to final fill height at the very end of the construction period. It is interesting to note the sensitivity of the soils to this differential loading and should be considered during the final construction stage.

5.2 VERTICAL SETTLEMENT RATE AND DISTRIBUTION

Vibrating wire piezometers were installed to measure the pore pressures beneath the trial embankment and to quantify the vertical settlement rate and distribution. The vibrating wire piezometers (VWPs) have been monitored continuously since their installation, before the placement of fill across the embankment footprint. Outputs of the VWPs can be seen in Figure 5 below showing the total fill placed (RL m), the pore water pressures (kPa) and the excess pore water pressures (kPa) generated.

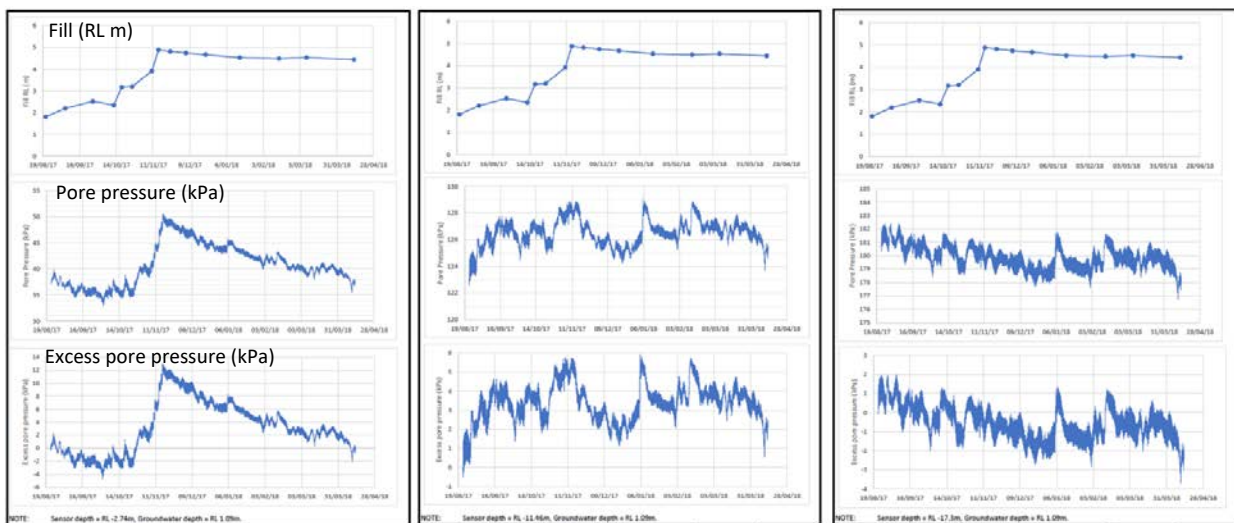


Figure 5: VWP Plots (located from left to right in: upper RA-SD unit, lower RA-SD unit, lower RA-S unit)

The first VWP plot, located within the upper RA-SD unit and denoted VWP01, shows a significant reaction to the placement of the fill, with an approximate 50kPa increase in pore pressure during October and November 2017 when the final fill height of the trial embankment was reached. This was calculated by adding the sum of all the peaks in the VWP plot during this time. This corresponds to the 53kPa load applied at SM02, located adjacent to the VWPs, once the full embankment height was reached.

After the final fill height is reached, VWP01 shows a gradual pore pressure decay trend with minor fluctuations and peaks in the data corresponding to pore pressure changes of approximately 1kPa. It took approximately 140 days (or 4.7 months) for the excess pore water pressures to dissipate back to pre-fill levels within this upper RA-SD unit.

The other two VWP plots, located within the lower RA-SD unit and the lower RA-S unit, show very little response to the placement of the fill. VWP03 (right plot) located within the RA-S unit shows evidence of the fluctuation of groundwater levels between the winter and summer months, as indicated by the slight downward trend. This indicates that most of the effects of the embankment fill being placed are felt within the upper RA-SD unit and that is therefore where all the settlement is occurring.

5.1 HORIZONTAL DEFORMATION

Vertical inclinometers were installed to monitor the horizontal deformation of the soils at the toe of the trial embankment. The vertical inclinometers were monitored regularly throughout the construction period and monthly following the end of the construction, with zero-readings taken for the vertical inclinometers before any fill was placed across the embankment footprint. Outputs of the inclinometer plots can be seen below in Figure 6.

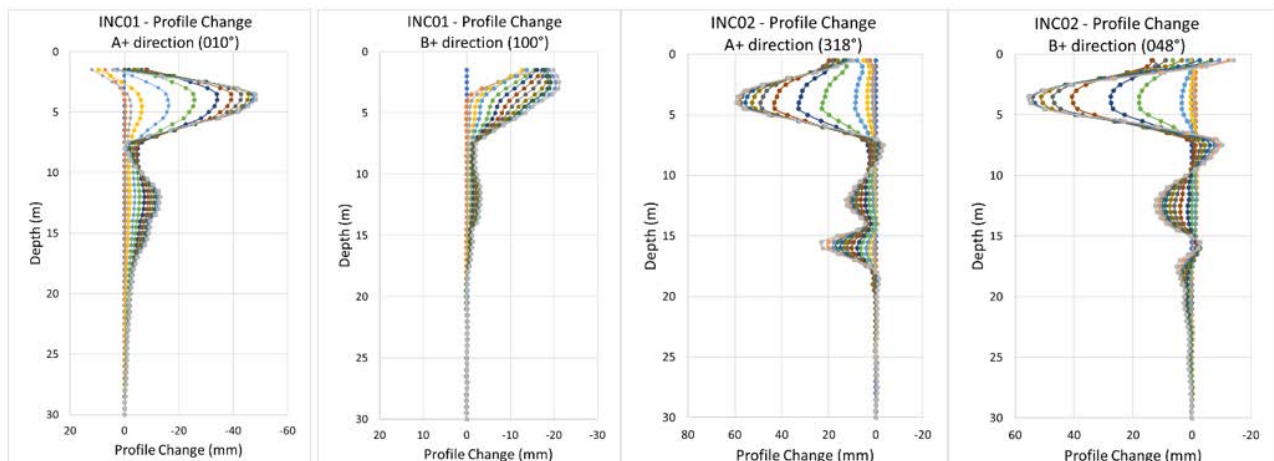


Figure 6: Inclinometer Plots showing depth (m) versus deflection (mm)

The net vector magnitude of the deflections for the inclinometers was calculated from the observed movement in both the A and B directions. This resulted in approximately 50mm to 80mm deflection within the upper 7m of the existing soil profile, which corresponds to the upper RA-SD unit. Both traces also show smaller peaks with up to 20mm of deflection between 10m and 18m below the existing ground surface which corresponds to the lower RA-SD unit. The ratio of the vertical settlement to the horizontal deformation is approximately 7.5 (vertical to horizontal) in the upper RA-SD unit.

6 ANALYSES

6.1 TOTAL SETTLEMENT PREDICTIONS

Total primary settlement predictions were completed for the three settlement markers using the Asaoka graphical method (Asaoka, 1978). This involves initially plotting the time (days) against the total settlement (mm) occurring. A curve fitting exercise was then completed to obtain a smooth curve that accounted for the construction period. This was completed using the method described by Craig (2004) where the critical time (t_c) is described as the effective time taken for the construction period. Corrected curves were completed for the three settlement markers and an example of this is shown in Figure 7 below.

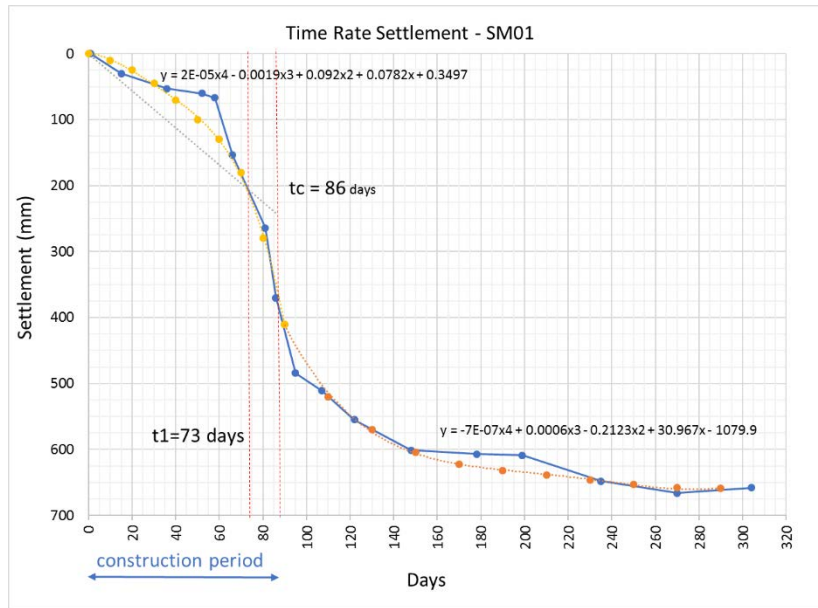


Figure 7: Curve fitting of the settlement plots showing actual and smoothed settlement (mm) versus time (days)

The Asaoka method then involves taking the fitted curve settlement values at time = P_t (mm) and time = $P_t - 1$ (mm) and a line of best fit is plotted through the data set. Where the line of best fit intercepts a 45-degree regression line, is the maximum settlement that is predicted to occur as $P_t = P_t - 1$. The data sets are taken from the smoothed curve of settlement after t_c has been reached.

The percentage of consolidation that has been reached at any given time can be calculated by dividing the current settlement by the total predicted settlement.

The two peaks observed within the horizontal inclinometer (INC03) plots were also taken and plots of settlement versus time were created to complete a further design check on the monitoring results. The Asaoka method was then used to determine the percentage of consolidation for these two points along the inclinometer plots.

A summary of the Asaoka method results for the settlement markers and the horizontal inclinometer are presented in Table 5 below.

Table 5: Asaoka graphical method summary

| SM no. | RA-SD thickness (upper + lower) (m) | Embankment height (m) | Actual total settlement (mm) | Percentage of consolidation (%) | Time to reach t_{90} (months) |
|--------------|-------------------------------------|-----------------------|------------------------------|---------------------------------|---------------------------------|
| SM01 | 6 + 5.5 | 3.95 | 658 | 97 | 3.8 |
| SM02 | 5 + 5.5 | 3.31 | 637 | 98 | 4.3 |
| SM03 | 5.5 + 8.5 | 3.36 | 519 | 95 | 4.7 |
| INC03 at 20m | 5 + 5.5* | 3.31* | 612 | 98 | 3.4 |
| INC03 at 38m | 5 + 5.5* | 3.31* | 587 | 99 | 3.1 |

*Assumed values based on nearby settlement marker (SM02)

From the table above, the horizontal inclinometer Asaoka method results are seen to provide similar total settlements, consolidation percentages, and t_{90} values to the settlement marker results. This corroborates the settlement marker results and shows a good correlation between the two different instrumentation monitoring results. They can therefore both be considered reliable outputs.

The time to reach t_{90} is approximately 3.9 months, which is relatively close to the time taken for the excess pore pressures to dissipate beneath the embankment. This is considerably less than the t_{90} value initially predicted, which for a 6m thick RA-SD unit was approximately 12 months. This indicates the effectiveness and importance of trial embankments as a geotechnical tool for back-calculating consolidation parameters, as a laboratory sample is only a small representative of the overall geological unit and will not be representative of the overall soft soil and consolidation behaviour in the field.

6.2 BACK CALCULATION OF PARAMETERS

6.2.1 C_c value

Where all other parameters are known, following the measurement of ground settlements from a trial embankment, the C_c value can be back calculated by using the equation proposed by Terzaghi, Peck and Mesri (1996).

The back calculated C_c values are summarised in Table 6 below and are compared to the laboratory measured values. It should be noted that when back calculating the C_c values, the e_0 value was kept the same as that assumed in the preliminary design predictions.

Table 6: Summary of back-calculated C_c values and comparison to laboratory values

| SM no. | RA-SD thickness (upper + lower) (m) | Embankment height (m) | Laboratory C_c value | Back calculated C_c value |
|--------|-------------------------------------|-----------------------|------------------------|-----------------------------|
| SM01 | 6 + 5.5 | 3.95 | 0.60 | 0.52 |
| SM02 | 5 + 5.5 | 3.31 | 0.60 | 0.8 |
| SM03 | 5.5 + 8.5 | 3.36 | 0.60 | 0.56 |

Based on the results above it can be seen that the back calculated C_c values have an average value of 0.62 but are generally lower than the laboratory C_c value used in the preliminary design predictions. The exception to this is SM02, where the C_c value is actually greater than the 0.6 used in preliminary predictions.

6.2.2 C_v value

The rate of consolidation settlement is dictated by the rate at which excess pore water pressures can dissipate from within the RA-SD unit, which is in turn dictated by its permeability and thickness. Beneath the trial embankment, and elsewhere within the Wairoa River valley, two RA-SD layers are generally present but are separated by an intermediate sand layer (RA-S unit). This predominantly sandy material above and below the compressible soils provides two-way drainage.

Rates of settlement are notoriously difficult to estimate in alluvial soils, such as the RA-SD unit, due to the depositional environment where there is inherent material composition variability, both laterally and with depth. The time rate of settlement is dependent on the C_v value which was able to be back calculated by rearranging Terzaghi's settlement time rate equation (Terzaghi, Peck and Mesri, 1996).

A summary of the back calculated C_v values is shown below in Table 7.

Table 7: Summary of back-calculated C_v values and comparison to laboratory values

| SM no. | RA-SD thickness (upper + lower) (m) | Embankment height (m) | Design C_v value ($m^2/year$) | Back calculated C_v value ($m^2/year$) |
|--------|-------------------------------------|-----------------------|-----------------------------------|--|
| SM01 | 6 + 5.5 | 3.95 | 8 | 23.5 |
| SM02 | 5 + 5.5 | 3.31 | 8 | 14.8 |
| SM03 | 5.5 + 8.5 | 3.36 | 8 | 16.4 |

From the results presented above, the back calculated C_v value is approximately two to three times greater than what was assumed during the preliminary design assumptions and therefore dramatically reduces the time taken to reach 90% consolidation. This accounts for the large difference between predicted and measured t_{90} values discussed in Section 6.1.

The reason for the small C_v values measured during the laboratory tests is because they only represent a very small volume of the soil and therefore do not take into consideration discontinuities within the overall in-situ soil unit, such as thin sand lenses. They also tend to be over-conservative, which is demonstrated with the difference in the C_v values shown above.

7 CONCLUSIONS

Based on the observations and results of the trial embankment construction and monitoring, the following conclusions can be made:

- Based on CPT results, the lower RA-SD layer beneath the trial embankment footprint does not appear to have the same soft soil characteristics as the upper RA-SD layer.
- Settlement beneath the embankment can be significantly affected by how the fill is placed during construction.
- Vibrating wire piezometer plots within the upper and lower RA-SD units indicated that nearly all the settlement was occurring within the upper soft soil layer and showed that it took approximately 4.7 months for the excess pore pressures to dissipate back to original levels beneath the trial embankment.
- The Asaoka graphical method showed that it took approximately 3.9 months to reach t_{90} which is considerably less than the 12 months initially predicted for a 6m thick unit based on laboratory oedometer testing results. The time to reach 90% consolidation was similar to the time taken for the excess pore pressures to fully dissipate.
- Back calculated C_c values, using the Asaoka graphical method, were approximately 80% to 90% of the laboratory obtained C_c values.
- Back calculated C_v values were two to three times greater than laboratory measured C_v values. This significant increase in the C_v value explains the large difference in t_{90} values.
- Care should be taken when reviewing these results as they do not represent the soil conditions across the entire Wairoa Valley. Although the upper RA-SD unit was found to account for nearly all the settlement observed at this location, it is unlikely to be the case for other locations along the TNL alignment within the Wairoa River valley.

8 ACKNOWLEDGEMENTS

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9 REFERENCES

- Asaoka A. (1978). Observational procedure of settlement prediction. *Soils and Foundations* Volume 18, Issue 4, pages 87–101
- Briggs, R.M.; Hall, G.J.; Harmsworth, G.R.; Hollis, A.G.; Houghton, B.F.; Hughes, G.R.; Morgan, M.D.; Whitbread-Edwards, A.R. (1996). *Geology of the Tauranga Area*, Occasional Report No 22, Department of Earth Sciences, University of Waikato, Hamilton, New Zealand.
- Craig, R.F. (2004). *Craig's Soil Mechanics*. 7th Edition. Spon Press, New York. Chapter 7, pages 227-276.
- CMW Geosciences (2017). *Geotechnical Factual Report*, Tauranga Northern Link, State Highway 2, Tauranga. Prepared for Bloxam Burnett & Olliver Ltd.
- CMW Geosciences (2017). *Geotechnical Interpretive Report*, Tauranga Northern Link, State Highway 2, Tauranga. Prepared for Bloxam Burnett & Olliver Ltd.
- Mesri, G.; Kwan Lo, D.O. and Feng, T.W. (1994). Settlement of embankment on soft clay, *ASCE Geotechnical Special Publication* Volume 40, Issue 1, pages 8-56.
- Mesri G, Ajlouni N (2007) Engineering Properties of Fibrous Peats, *Journal of Geotechnical and Geoenvironmental Engineering*, Volume 133, Issue 7. ASCE
- New Zealand Standard 4402: 1986, Methods of Testing Soils for Civil Engineering Purposes.
- New Zealand Transport Agency Bridge Manual, 3rd Edition, incorporating amendments to May 2016.
- Terzaghi, K.; Peck, R.B.; Mesri, G. (1996). *Soil Mechanics in Engineering Practice*. 3rd Edition. Wiley, NYC. Chapters 16.6 – 16.8, pages 106-113.