Determination of age of Tauranga/Maketu Basin peat based on apparent pre-consolidation pressure due to soil creep

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ABSTRACT

The Tauranga Eastern Link project is a 22km length of Road of National Significance from Te Maunga to Paengaroa. The project included over 13km of embankments constructed on very soft, highly compressible peats and estuarine silt alluvium within the Tauranga/Maketu Basin with up to 4.5m recorded settlement. The embankments were surcharged to reduce the residual long-term settlement to an acceptable level. It is one of the most extensive use of surcharged embankments over peats for motorway projects in New Zealand. The Tauranga/Maketu Basin peats geologically are normally consolidated, however soil behaviour shows that they are slightly over-consolidated. This over-consolidation is most likely due to secondary compression and aging effects. This paper presents the design procedures for determining the depositional age based on embankment settlement data. Effects of depositional rate, stress history and ground water level fluctuation are considered.

Keywords: Peat, creep, secondary consolidation, delayed compression, pre-consolidation pressure, over-consolidation ratio.

1 INTRODUCTION

The Tauranga Eastern Link (TEL) design-construct project is a 22km length of Road of National Significance from Te Maunga to Paengaroa which was awarded to the Fulton Hogan HEB Construction Alliance by New Zealand Transport Agency.

The site is situated in between Te Maunga to Paengaroa and bypasses Te Puke town centre to the northeast. The motorway is designed to carry 4 lanes of traffic at a design speed of 110km/hr. The site is underlain by an alluvial deposit including up to 9m of highly compressible peats and 15m of very soft estuarine silts.

Fulton Hogan HEB Construction Alliance, on the advice of Gaia Engineers Ltd (formerly Peters and Cheung Ltd) acting as specialist geotechnical consultant to the lead consultant, URS, made the decision to preload and surcharge the alluvium to reduce residual settlements to an acceptable level. Six trial embankments were constructed and monitored to establish settlement design parameters. Based on the information gained from the trial embankments, ground improvement including surcharging up to 5m thick was employed. A typical cross section of the TEL embankment is shown in Figure 1.

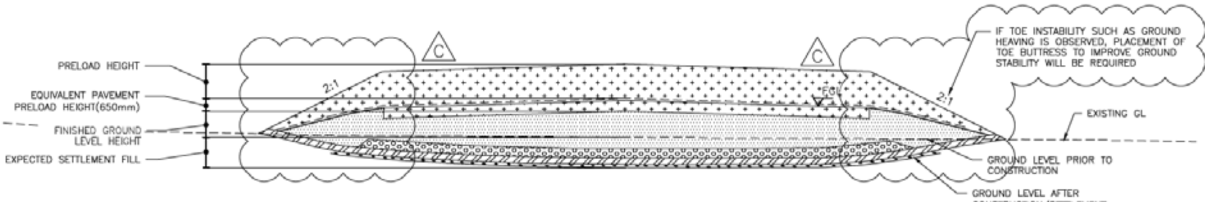

Figure 1. Typical Cross Section of Surcharge Embankment at TEL

Surcharging was found to be effective and economical for the ground conditions along the proposed motorway. 13km of the 22km motorway was preloaded and surcharged with surcharge periods generally around 9 months. Preload settlement up to 4.5m was recorded but generally smaller settlements between 1m to 2m were recorded.

During the construction of the embankments, an apparent over-consolidation of both peats and silts was observed. This paper presents the settlement back-analysis for the embankments specifically focused on the apparent over-consolidation ratio of the highly compressible, very soft alluvial peats.

2 GEOLOGICAL SETTING

With reference to the published geology it can be seen that the wider Maketu Basin consists of a Pleistocene Age, predominantly alluvial or estuarine basin which has in-filled during a period of rapid tectonic subsidence. The basin infill consists of a series of reworked terrestrial/volcanogenic and estuarine deposits, non-welded distal ignimbrites and airfall Tephra. During the late Pleistocene and Holocene, coastal and alluvial sedimentation has resulted in a series of low terraces, coastal and sand dune complexes and associated back-dune swampy environments.

This study is concerned with the upper 20m of relatively recent, very loose sands, firm alluvial silt, very soft estuarine silts and surficial highly compressible fibrous peat. Ground water level across the site is generally about 1.0m to 1.5m below the ground surface in summer, lifting to within 0.5m of the surface in winter.

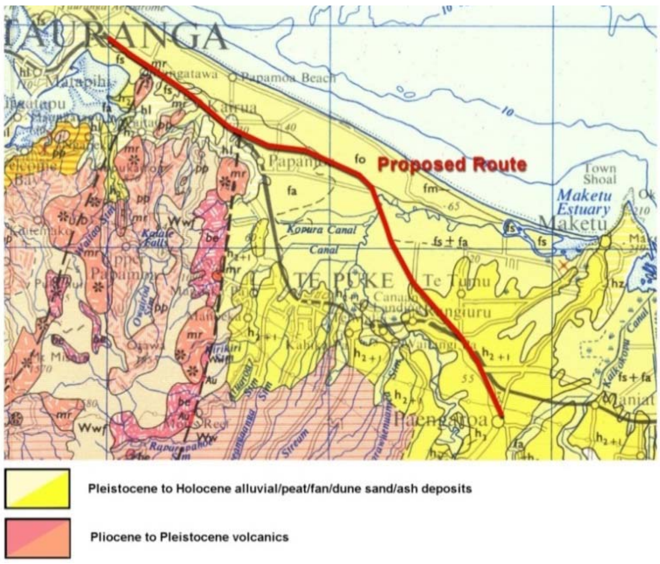

Figure 2. Published geological map of the Tauranga to Te Puke region (modified from NZGS Sheet 5 Rotorua Geological Map of New Zealand 1:250,000)

3 DESIGN PHILOSOPHY

Back-analysis was undertaken with a one-dimensional analysis of the centre-line conditions. Given the relatively large embankment platform compared to the shallow depth of compressible deposits, a one-dimensional elastic visco-plastic model was considered appropriate.

The classical expression for strain due to an increase in effective stress for normally consolidated soils is given by:

$$\frac{\Delta H}{H_0} = c_{c\varepsilon} \log \frac{\sigma' + \Delta\sigma}{\sigma'} \qquad (1)$$

where $c_{c\varepsilon} = \frac{c_c}{1+e_0}$; c_c, compression index ; e_0, initial void ratio ; and H_0, initial soil layer thickness.

The expression for strain due to delayed compression at a constant effective stress is:

$$\frac{\Delta H}{H_0} = c_{\alpha\varepsilon} \log \frac{t+\Delta t}{t} \qquad (2)$$

where $c_{\alpha\varepsilon} = \frac{c_\alpha}{1+e_0}$, where c_α represents the secondary compression index.

All consolidation and creep settlement has been normalised to compressional strain based on the initial soil thickness of the layer.

In practice, it is often difficult to obtain undisturbed sampling to determine the void ratio of soils, especially peats. As such it was found determining settlements based on strains is much more effective as the initial layer thickness can easily be determined through CPT soundings and the change in settlement can be easily measured by settlement plate monitoring.

3.1 Compression Index of Large Strain Soils

Compression index is the slope of the strain versus logarithmic effective stress plot. As demonstrated by Bjerrum (1967), if different effective stress points of a soil are allowed to creep for the same set time, they will each undergo the same amount and rate of additional strain, irrespective of their effective stress states. This additional strain at constant effective stress is referred to as delayed compression.

Typically in determining the compression index, the strain to determine this slope is selected at the end of primary consolidation, or when the average degree of consolidation reaches 90%. Contrary to Terzaghi's theory of 1-D consolidation, for peats, there is a marked reduction in permeability as the soil compresses, therefore the time to reach end of primary consolidation increases significantly as the soil undergoes strain. Therefore if two identical peat samples are subjected to different loads, the sample subjected to the lesser load will reach end of primary consolidation before the sample subjected to the greater load.

There is much debate over exactly when delayed compression commences, however it is generally agreed that this it is independent of permeability and that a significant portion of the delayed compression strain occurs concurrently with consolidation settlement due to dissipation of excess pore water pressure. Therefore if two identical peat samples are subjected to different loads, the sample subjected to the lesser load will reach end of primary consolidation before the sample subjected to the greater load. However, delayed compression will have commenced at the same time and rate for both samples. This means that for the sample subjected to the greater load, a longer time will have elapsed to reach end of primary and therefore it will have also undergone more delayed compression within the primary consolidation period.

In other words, for peat, the normally consolidated line is not parallel to the creep isotaches for large strain soils because of the inclusion of delayed compression during the primary consolidation period. This phenomenon is demonstrated in Figure 3.

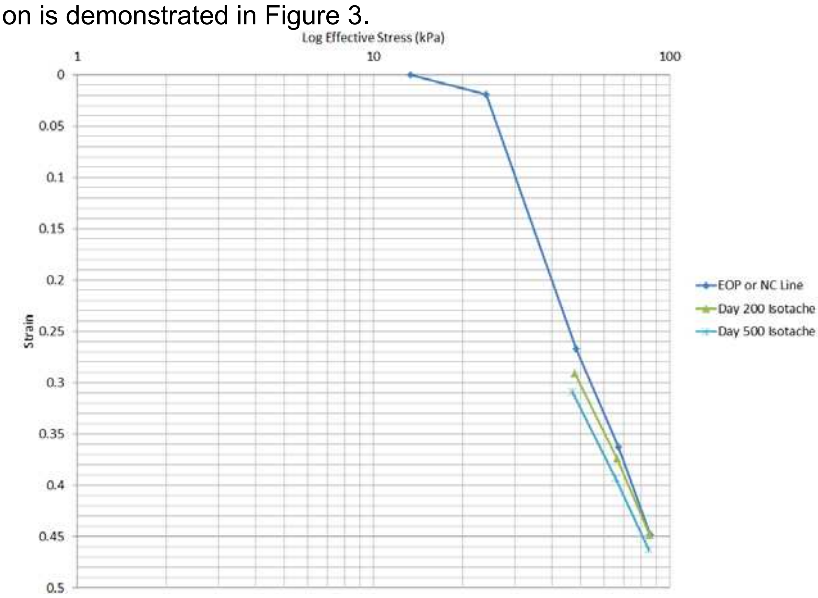

Figure 3. Site Office Trial Embankment Compression Index of Peat

Therefore in determination of the compression index, the strain selected should be at a consistent time which is after the excess pore water pressure has dissipated for the sample which had the greatest time to reach end of primary.

Another consideration which should be given for large strain soils is whether to use the initial soil layer thickness or the corrected soil layer thickness after compression has taken place. The peats measured at this site show better correlation using the initial soil layer thickness for fitting the compression index.

3.2 Secondary Compression Index of Large Strain Soils

The secondary compression index is the slope of the straight line portion of the strain versus logarithmic time plot. For practical purposes this is generally taken after 90% of the average degree of consolidation is reached. In creep oedometer tests, load is applied instantaneously and the logarithmic time is plotted from this moment. For physical embankments, often a significant amount of time has elapsed before the full embankment load is applied. In determining the secondary compression index, it is important for consistency to restart the time to the moment when the embankment was completed. If not corrected, then the slope of the data will be flattened for about 2 logarithmic time cycles until the data will eventually asymptote to the true secondary compression index.

As discussed for determining the compression index for large strain soils, the same consideration should be given whether to use the initial soil layer thickness or the corrected soil layer thickness after compression has taken place. The peats measured at this site show better correlation using the initial soil layer thickness for fitting the secondary compression index.

3.3 OCR Due to Aging

The process of over-consolidation leading to delayed compression was first formulated by Bjerrum (1967) and subsequently developed by Yin and Graham (1989) and Nash and Ryde (2001). Their idea's express delayed compression settlement as a series of parallel lines on a strain vs. log σ' plot and the equation is given in the form:

$$OCR^{1-\frac{c_s}{c_c}} = \left(\frac{\mathrm{t}+\Delta\mathrm{t}}{\mathrm{t}}\right)^{\left(\frac{c_\alpha}{c_c}\right)} \qquad (3)$$

The over-consolidation ratio due to delayed compression stems from two main concepts, 'apparent time' and 'equivalent stress'.

To illustrate these concepts the following example is given. An embankment is constructed over a soil which increases the effective stress state from σ_0' to σ_1'. This embankment, left for a time Δt, undergoes further strain in the form of delayed compression which commences after the creep reference time for the soil.

$$\varepsilon = \mathrm{c}_{\mathrm{c}\varepsilon} \,.\log\left(\frac{\sigma_1'}{\sigma_0'}\right) + \mathrm{c}_{\alpha\varepsilon} \,.\log\left(\frac{\mathrm{t}_{\mathrm{reference}}+\Delta\mathrm{t}}{\mathrm{t}_{\mathrm{reference}}}\right) \qquad (4)$$

The strain from the delayed compression portion is equal to the consolidation strain that would occur if an equivalent stress was placed in addition to σ_1'. The soil becomes over-consolidated with time by the same amount as if it had experienced an equivalent stress.

$$\varepsilon_{\text{delayed compression}} = \mathrm{c}_{\alpha\varepsilon} \,.\, \log\left(\frac{\mathrm{t}_{\mathrm{reference}}+\Delta\mathrm{t}}{\mathrm{t}_{\mathrm{reference}}}\right) = \mathrm{c}_{\mathrm{c}\varepsilon} \,.\, \log\left(\frac{\sigma_1'+\Delta\sigma_{\mathrm{equivalent}}}{\sigma_1'}\right) \qquad (5)$$

Conversely, suppose an embankment with surcharge is constructed over the same soil. The embankment is left for a time t_1 and the soil effective stress increases from σ_0' to σ_2' resulting in strain.

$$\varepsilon = \mathrm{c}_{\mathrm{c}\varepsilon} \,.\log\left(\frac{\sigma_2'}{\sigma_0'}\right) \qquad (6)$$

The surcharge portion, Δσ is then released so the soil now has an effective stress of σ_1'.

$$\sigma_1' = \sigma_2' - \Delta\sigma \qquad (7)$$

The strain from the additional surcharge component is equal to the strain if the embankment was allowed to undergo delayed compression for an apparent time. The rate of delayed compression after surcharge removal is the same as if the embankment was allowed to undergo delayed compression for an apparent time.

$$\varepsilon_{\text{surcharge}} = c_{c\varepsilon} \,.\, \log\left(\frac{\sigma_1' + \Delta\sigma}{\sigma_1'}\right) = c_{\alpha\varepsilon} \,.\, \log\left(\frac{t_{\text{apparent}}}{t_1}\right) + c_{r\varepsilon} \,.\, \log\left(\frac{\sigma_1' + \Delta\sigma}{\sigma_1'}\right) \tag{8}$$

This apparent time can be expressed in the form:

$$t_{\text{apparent}} = t_1 \,.\, \left(\frac{\sigma_2'}{\sigma_1'}\right)^{\left(\frac{c_{c\varepsilon} - c_{r\varepsilon}}{c_{\alpha\varepsilon}}\right)} \tag{9}$$

It is this relationship between strain, stress and time which makes it possible to determine the age of a soil based on the apparent over-consolidation ratio as given in Equation 3.

4 SELECTION OF SETTLEMENT DESIGN PARAMETERS

Design parameters were based on laboratory tests and field trial embankments. Due to difficulty in sampling peat, emphasis was placed on the back-analysis of the trial embankment settlement data. Five trial embankments were constructed in advance of the main motorway embankment construction and monitored for 1 year. These trial embankments were instrumented with settlement plates, anchor extensometers and piezometers. The surficial peat settlement was differentiated from the lower estuarine silt and sand settlement. A cross section of the instrumentation used to monitor the Site Office Trial Embankment is shown in Figure 4. This paper focuses on the data recorded and back-analysed from the Site Office Trial Embankment.

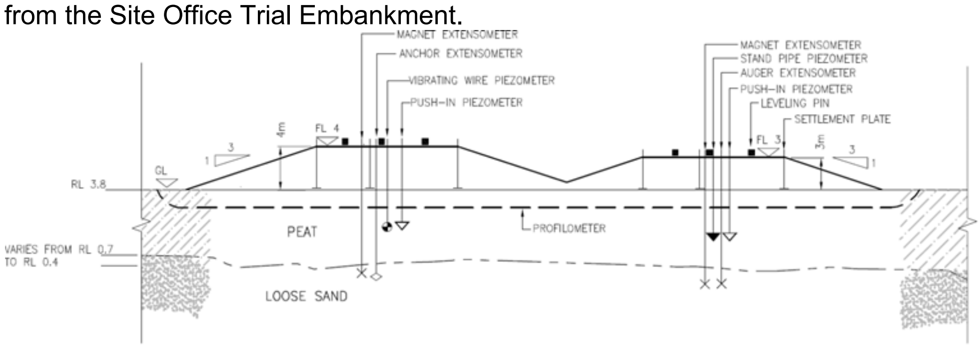

Figure 4. Site Office Trial Embankment Instrumentation Cross Section

5 SITE OFFICE TRIAL EMBANKMENT

The site office trial embankment comprised the following key features:

- Three 13m by 17m cells construction with 2m, 3m and 4m thick of material γ_{bulk} = 21.3kN/m3;
- Surficial fibrous peat 2.7m thick beneath a 0.3m thick stiff topsoil crust;
- Loose alluvial sand underlain by deep, slightly over-consolidated estuarine silts;

5.1 Settlement Monitoring Data

In the peat, primary consolidation settlements of 0.71m, 0.98m and 1.20m were recorded respectively for the 2m, 3m and 4m cells.

In the peat, creep settlements of 0.060m, 0.065m and 0.060m were recorded respectively for the 2m, 3m and 4m cells between the period 200 days until 500 days after construction.

5.2 Parameter Back-Analyses

A compression index, $c_{c\varepsilon}$ of 0.59 to 0.63 was determined from the straight line portion on the strain versus log effective stress plot.

To determine the $c_{c\varepsilon}$, strain for each cell was select 200 days after the embankment was completed to ensure the majority of excess pore water pressure had dissipated. This was also cross-checked with the strain after 500 days which gave a parallel line indicating its correctness. In addition, the preconsolidation point has been included which indicates a recompression index, $c_{r\varepsilon}$ of 0.07.

To determine the secondary compression index, $c_{\alpha\varepsilon}$ the difference in strain from day 200 until day 500 was measured. This recorded the same difference in strains for each cell. A $c_{\alpha\varepsilon}$ of 0.05 was selected based on Figure 5.

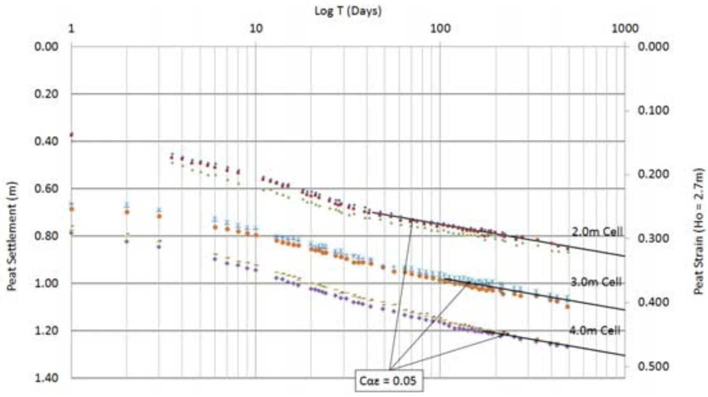

Figure 5. Site Office Trial Embankment Secondary Compression Index of Peat

5.3 Determination of OCR

From the excess pore water pressure and settlement monitoring data, a significant change was only noted once the embankment fill exceeded 0.5m thick, or when the effective stress at the centre of the peat exceeded 22.7kPa. This equates to an OCR of 1.8 based on initial effective stress of 12.5kPa.

Nine CPTs and four hand augers with vane shear testing were carried out across the site and used to determine the undrained shear strength of the peats as shown in Figure 6a. For a normally consolidated soil, its undrained shear strength increases proportionally with increasing effective stress at depth. This fundamental concept is discussed in detail by Bjerrum (1967):

$$\frac{S_u}{\sigma'} \cong 0.22\ \mathrm{OCR} \tag{10}$$

Based on this undrained shear strength to effective stress relationship, an OCR in the order of 1.8 to 2.2 was determined for the peats as shown in Figure 6b. This matched very well with the OCR determined through settlement and excess pore water pressure measurements.

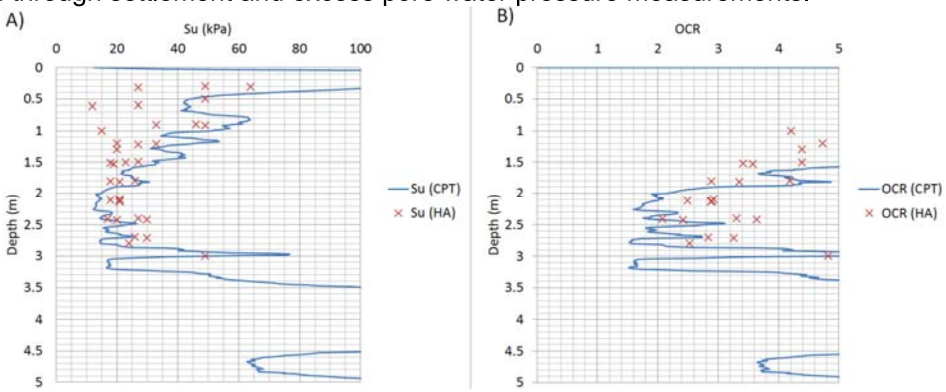

Figure 6. Site Office Trial Embankment a) Undrained Shear Strength of Peats and b) Inferred Over-Consolidation Ratio of Peats.

5.4 Settlement Monitoring Curve Fit and Reference Time

In order to curve fit the monitored settlement presented in Figure 7 in keeping with the terminology used by Bjerrum (1967), we have added the recompression settlement, instant compression settlement and delayed compression settlement.

Recompression settlement of 50mm was measured from the initial stage of loading which made up the initial settlement portion up until the preconsolidation pressure.

Instant compression settlement (similar to primary consolidation) is purely a function of effective stress and the compression index. Continuous pore water pressure measurements enabled the effective stress to be known at each moment in time. This eliminated the uncertainty of the c_v changing with strain and a soil not conforming with Terzaghi's degree of consolidation assumptions.

Delayed compression is based on the secondary compression index, logarithmic time and the reference time. The only unknown was the reference time and this was adjusted to curve fit the data. Based on a reference time between 30 to 100 days, a reasonable curve fit, shown in Figure 7, was established.

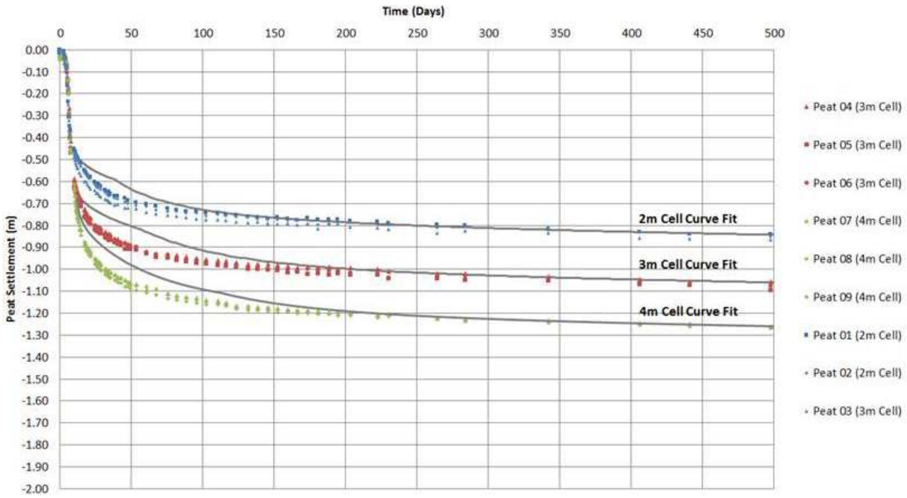

Figure 7. Site Office Trial Embankment Settlement Curve Fit

Although the curve fit was reasonable, it was quite sensitive to the parameters selected. In determining the settlement predictions for the 13km of Tauranga Eastern Link embankments over soft ground, an excellent curve fit was established using a simplified approach which ignored the pre-consolidation pressure and linked the reference time to degree of consolidation. This indicates there is further research to be carried out on the most appropriate settlement prediction method for large scale projects involving embankments on soft ground.

5.5 Age Determined By OCR

Based on the reference time established through curve fitting and OCR, the age of a deposit is able to be determined. The age of the soil is given by:

$$\text{Age} = t_{\text{reference}} \,.\, \text{OCR}^{\left(\frac{c_{c\varepsilon} - c_{cr}}{c_{\alpha\varepsilon}}\right)} \qquad (11)$$

An age of between 1600 years to 3100 years was calculated for the peat based on its back-analysed soil parameters.

In cases where the ground water table used in back-analyses is abnormally low, this would alter the results as follows:

A) The soil would have a higher initial effective stress than measured;
B) The apparent over-consolidation ratio would be lower;
C) The reference time, $c_{\alpha\varepsilon}$, $c_{r\varepsilon}$ and $c_{c\varepsilon}$ parameters would remain the same;
D) The back-analysed age of the soil would be younger.

In this instance, the back-analysed age, would represent then duration that the soil has been exposed to the higher initial effective stress.

For deeper, older deposits, the back-analysed age, would represent the time of the most recent significant increase in effective stress; often due to a subsequent deposition.

6 RADIO-CARBON DATED VERIFICATION

Remnant wood fragments from peat deposits were radio-carbon dated at the University of Waikato. Results of the radio-carbon dating are as follows:

- Peat sampled 1.0m deep at the Site Office Trial Embankment site: 3300 years old;
- Peat sampled 0.6m deep adjacent the Kaituna River Trial Embankment site: 2000 years old;

- Peat sampled 1.2m deep adjacent the Parton Road Trial Embankment site: 3400 years old.
- Peat sampled from a spoil stockpile of 3m deep excavation near the Domain Road Trial Embankment site: 1600 years old.

These measured ages agree well with the predicted age based on the over-consolidation ratio which demonstrates that Bjerrum's theory (1967) is valid. However, there is a high degree of sensitivity in the result, which indicates there is further research to be carried out on the theoretical reference time soil parameter.

7 DISCUSSION & CONCLUSIONS

Key findings from the project are discussed below:
- Superimposed instant compression and delayed compression settlement provide a reasonable curve fit where settlement parameters are known.
- In practice, settlement normalised to strain and initial soil layer thickness rather than the initial void ratio are more easily measured.
- Aged normally consolidated deposits display an apparent over-consolidation ratio.
- The apparent over-consolidation ratio can be calculated from the undrained strength and effective stress relationship.
- In highly compressible soils, c_v changes significantly with strain therefore the calculated c_c should be based on strains measured at consistent times which are beyond the end of primary for each stress-strain point.
- Reference time can be back-calculated where the effective stress with time is monitored and other settlement parameters are known.
- There is further research to be carried out on the reference time to enable better curve fitting of monitored data.
- The age of the most recent significant soil deposit, or increase in effective stress, can be calculated based on the over-consolidation ratio of the soil.

8 ACKNOWLEDGEMENTS

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