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Estimating pre-load induced settlement from laboratory test data and field monitoring records: A Case Study at Temple View Eastern Redevelopment Project, Hamilton

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ABSTRACT

Predicting consolidation settlements in soft soils based on laboratory test data is commonly undertaken by geotechnical engineers, whereas the use of full scale trial embankments to compare these predictions is not always possible at the design phase. At Temple View both laboratory and trial embankment settlement data was used to predict fill induced consolidation settlements and to design surcharge embankments over compressible peat deposits to mitigate post construction creep settlements. Laboratory data used for the settlement analysis comprised oedometer tests that were undertaken on representative peat samples from across the site. The laboratory data was used to compute primary and secondary settlements under various surcharge heights. In comparison, settlement monitoring data from the trial embankment was used to back calculate both time rate (C_v) and settlement magnitude (C_c) parameters using the Asaoka graphical method. The parameters back calculated from the trial embankment data resulted in surcharge embankment design heights of between 0.5m and 1.5m lower than those designed from the laboratory parameters. The comparison has shown that laboratory values can be far higher than what full scale field data suggests and that reliance on 1D laboratory consolidation testing alone may be over conservative.

Keywords: consolidation settlement, creep settlement, Asaoka method, surcharge embankment, settlement monitoring, oedometer testing

1 INTRODUCTION

The proposed development site comprises a large block of land located on the eastern side of Tuhikaramea Road in Temple View, situated approximately 6km south-west of the Hamilton CBD. The western portion of the site, adjacent to Tuhikaramea Road, comprises elevated rolling hill topography that slopes down on to a low-lying alluvial floodplain encompassing the central and eastern areas of the site.

It was proposed to undertake bulk earthworks to raise the level of the low-lying floodplain, suitable for residential building construction, with the majority of the fill material sourced from excavation of adjacent rolling hills. The proposed earthworks will support the construction of single and two storey residential buildings plus a retirement village complex together with other ancillary buildings, associated roading and infrastructure.

Published geological information, and the geotechnical investigation results split the site into three main landforms. Landform Zone 1 comprises the elevated western portion of the site, Landform Zone 2 comprises the low-lying floodplain across the eastern part of the site and Landform Zone 3 is situated between Zones 1 and 2 and has been subject to more significant modification as part of historical site development. For the purposes of this paper only Landform Zone 2 was considered, as this area comprised the most geologically recent peat swamps, which are susceptible to significant primary and secondary settlement.

The primary purpose of the settlement analyses across the lower-lying areas was to assess a suitable surcharge design height that would limit future creep settlements beneath the proposed residential building platforms to a nominal value of 50mm (total) over a 50 year design life of the dwellings with differential settlements of no more than 25mm. It was a requirement that a minimum of 1.0m of structural filling was to be placed over the peat prior to the placement of any surcharge.

2 GROUND CONDITIONS - PEAT DEPOSITS

Peat is a characteristic soil deposit of the Waikato Basin comprising a network of open hollow cellular fibres (high void ratio), has a high water content, and has a high permeability, which makes it susceptible to significant settlement when subject to loading or drainage (i.e. change in effective stress). The general thickness of the peat across the lower-lying regions of the site was between 2.0m and 6.0m observed in trial pits, boreholes and interpreted from Cone Penetration Test (CPT) plots. The peat thickened towards the north and east of the site within the lowest lying areas and thinned out in the west where it sharply transitioned at depth (refer to Figure 1 below) into older elevated volcanic air-fall deposits.

The peat was typically amorphous, but had fibrous zones that contained large branches and tree trunks (up to 1.0m long and approx. 500m wide) and frequent roots. The tree fragments were fresh to moderately decomposed. The larger branches and roots were observed to provide pathways for water to seep into the trial pits during the investigation. Vane shear strengths (S_u) within the peat ranged from 20kPa to 100kPa, but were typically between 25kPa and 60kPa. As zones of peat were relatively fibrous, these larger, less decomposed fragments may have provided higher shear vane readings in places. CPT traces within the peats typically recorded cone resistance (q_c) values from 0.1 MPa to 1 MPa, with friction ratio values frequently alternating between approximately 1% and 7%.

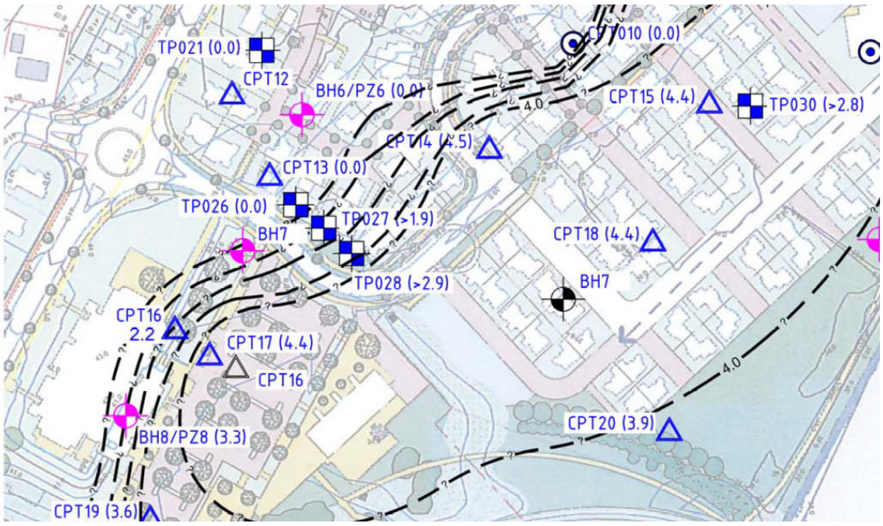


Figure 1. Depth Contours of Compressible Soils

Across the lower-lying portions of the site a non-saturated peat / fill crust (approx. 0.5m thick) was generally present above the peat. Ground-water was consistently observed at the base of this crust.

3 SOIL SAMPLING

Obtaining 'undisturbed' samples from peat soils can be challenging due to the common fibrous inclusions and as there is a high risk of soil degradation, loss of structure and moisture content during sampling and transportation of the sample from site to the laboratory. This can significantly affect the results of consolidation testing and therefore the sampling method was carefully considered. For this reason a series of push tubes (50mm in diameter) were used to recover the soil samples using an attachment on a rotary core drilling rig. The samplers were pushed into the ground without the use of drilling fluid and the retrieved core was then sealed at each end using melted wax. The samples were then sent to the laboratory at the earliest opportunity (within 2 to 3 days) to reduce the risk of further degradation within the sample. This sampling technique was favoured due to its cost-effectiveness and simplicity to carry-out and is generally undertaken as standard practice for retrieving one dimensional consolidation samples.

The locations of the tests targeted the thicker peat soils where the deposits were generally in excess of 4m thick. The spread of test locations aimed to pick up the variability in the peat soils across the site. The depths of the soil samples were also varied to account for the range of in-situ vertical stresses that the soils were currently subject to. The sample depth range for the peat soil was from approximately 1.5m to 5.0m below ground level.

4 GEOTECHNICAL PARAMETERS

4.1 Laboratory Testing

One-dimensional consolidation tests were undertaken on the peat samples received from the field to obtain the necessary soil parameters for estimating primary consolidation and secondary creep settlements. The oedometer testing was undertaken in accordance with the New Zealand Standard, NZS 4402:1986: Methods for testing soils for civil engineering purposes, Test 7.1. The average for the laboratory testing results used for corresponding settlement parameters were, initial moisture content 471.7%, Bulk Density 11.1 kN/m^3, initial void ratio (eo) 11.4, Compression Index (C_c) 4.84, Coefficient of consolidation (C_v) 5.5 m^2/yr. C_v and C_c values are reported for pressure increments, which represent an increase in additional design load with respect to the original pre-consolidation pressure, typically between 25 kPa to 50kPa.

The C_c values were estimated from the virgin compression slope of the e vs log σ'v plots and the C_v values were calculated using both the square root and logarithm of time fitting methods.

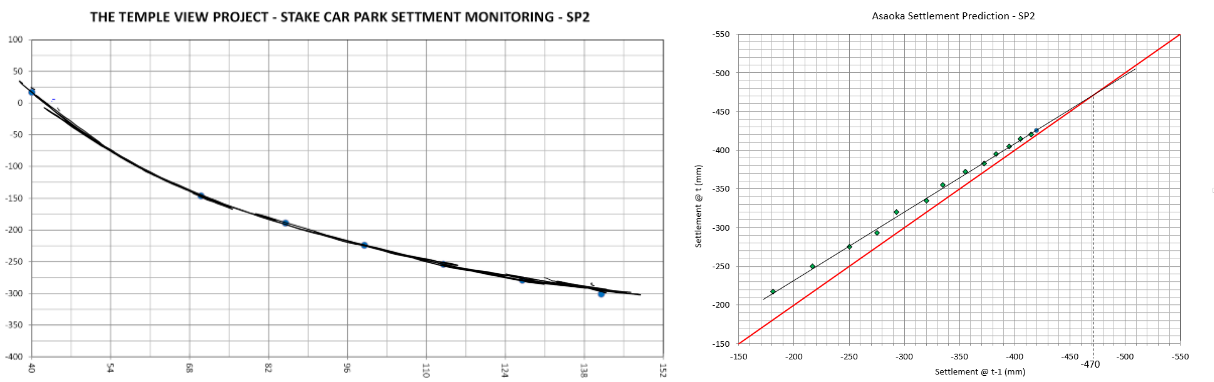
In general the laboratory results provide extremely high moisture contents and very low bulk densities, typical of juvenile peat deposits. The initial void ratio results are considered extremely high on some of the samples, but this is considered to fall within the typical range of values for peat soils.

4.2 Settlement Monitoring

A fill embankment was constructed across the low-lying peat-land immediately to the south of the current study area as part of the preparatory works for a previous stage of the development. A series of settlement monitoring pins were installed beneath the embankment to monitor the settlement. The observed settlement monitoring data therefore provided the opportunity to compare and contrast the laboratory derived settlement parameters. The trial fill embankment was approximately 1.8m high and comprised site-won fill with a bulk unit weight of approximately 16 kN/m^3. The ground beneath the preload consisted of a 0.5m over-consolidated crust underlain by 4.0m of peat. Groundwater was encountered at the base of the over-consolidated crust.

4.2.1 Predicting Total Settlements

In order to predict total primary consolidation settlements an analysis of four relevant settlement monitoring pins was undertaken by applying the Asaoka graphical Method (Asaoka, 1978). A time rate of settlement plot was developed for each monitoring point and a trend-line was drawn from when the full surcharge placement had begun. The settlement at a series of chosen t (mm) and t-1 (mm) values were then selected from the established trend-line and plotted against a 45^o regression line on a separate chart. Where the two lines converged a reading of total predicted settlements was taken. The Asaoka plots predicted total settlements to be between 470mm and 650mm with primary consolidation reaching between 84% (T_{84}) and 92% (T_{92}) for the four reviewed settlement monitoring pins. An example of the evaluated settlement monitoring data for one of the pins has been summarised in Figure 2(a) and Figure 2(b) below.



(a) *(b)*

Figure 2. (a) Time (Days) vs Settlement (mm) trendline; and (b) Asaoka Plot

4.2.2 Back-calculating Parameters

The following well known Terzaghi One Dimensional relationship was re-arranged to back-calculate representative C_c values based on the total primary consolidation settlements predicted from the Asaoka plots.

$$Sp = \frac{Cc}{1+e_0} H \log \frac{\sigma_v' + \Delta\sigma_v'}{\sigma_v'} \tag{1}$$

Where: Sp = primary consolidation settlement; C_c = compression index; e_0 = initial void ratio (from laboratory data); H = depth of compressible soil; σ_v' = initial vertical effective stress; $\Delta\sigma_v$' = change in vertical effective stress.

The back-calculated C_c values ranged from 3.25 to 3.78 with an average value of 3.53. These values were approximately 32% smaller than those derived from the laboratory test results.

4.2.3 Back-calculating Coefficient of Consolidation (C_v) Values

The following Terzaghi rate of settlement equation was then re-arranged and used to estimate the C_v values from the settlement monitoring data:

$$Tv = \frac{Cv\, t}{H^2{}_{dr}} \tag{2}$$

Where: *Tv* = Time Factor; C_v = Coefficient of Consolidation (m^2/yr); t = time (mins); H_{dr} = Height of drainage path (m).

C_v values were between 32.8 m^2/yr and 40.5 m^2/yr with an average value of 36.5 m^2/yr. Corroboration of the C_v values was undertaken by assessing the coefficient of horizontal consolidation (C_h) derived from a series of dissipation tests undertaken within the peat. The ratio of C_h to C_v values averaged between 2.4 and 4.0 based on T (theoretical time factor) values equal to 3 and 5, where:

$$C_h = \frac{R^2\, T}{t} \tag{3}$$

Where: C_h = coefficient of horizontal consolidation, R = average radius of CPT cone filter sensing pore pressures, T = theoretical time factor for given tip geometry and porous element location, t = time to reach given value of Δu(t) / Δu where Δu is change in pore pressure.

The estimated C_h / C_v values were considered to fall within the expected range for peat soils and provided verification that the back-calculated C_v values from the settlement monitoring data were reasonable.

4.3 Comparison of Laboratory and Settlement Monitoring Parameters

A summary of the back-calculated C_c and C_v values using the different methods is provided in Table 1.

Table 1. Comparison of Derived Settlement Parameters

	Laboratory C_c	**Settlement Monitoring C_c**	**Laboratory C_v (m^2/yr)**	**Settlement Monitoring C_v (m^2/yr)**
	9.01	3.65	0.89	33.5
	5.2	3.45	0.53	39.3
	9.5	3.78	3	40.5
	4.46	3.25	26	32.8
	7.28	-	0.55	-
	1.19	-	1.9	-
	0.82	-	-	-
	1.25	-	-	-
Average	**4.84**	**3.53**	**5.48**	**36.53**
Range	**8.68**	**0.53**	**25.47**	**7.70**

Table 1 highlights the variability in both C_c and C_v values estimated from the laboratory results. The variability in the C_c values along with the high moisture contents and initial void ratios observed within the peat materials indicated that the laboratory results may provide too large a range to adequately predict total settlement within the peats, with average C_c values considered to be very high. The available settlement monitoring derived results then provided validation of the laboratory data, which was undertaken as a comparison prior to the design of surcharge heights. The settlement monitoring parameters had a much smaller range of results and as such were considered to more accurately represent the peat mass as a whole.

5 ESTIMATING POST SURCHARGE CREEP SETTLEMENTS

5.1 Mesri Cα/Cc Relationship

There is a well-documented relationship between Compression Index (C_c) values and the Secondary Compression Index Value (C_α) for all soil types. This relationship is also known as the Law of Compressibility and simply states that the trend of primary compression and re-compression observed within a soil is closely linked to the behaviour of a soil undergoing secondary compression. Published values from Mesri et al (1997) of C_α/C_c for peat soils are typically between 0.06 +/- 0.01. A conservative value of 0.07 was used for back analysing $C\alpha$ for this project.

5.2 Surcharge to reduce secondary compression

Creep settlement in peat comprises a large component of its total settlement due to several reasons. Its hollow cellular structure and high initial water contents, results in large initial void ratio, which has a high potential to undergo significant settlements over time. The porous nature of the peat also provide a relative short duration of primary consolidation, with primary consolidation typically completed in field trials typically between 2 weeks to 2 months. The breakdown of the organic cell structures through biodegradation of the plant cell walls further compounds the secondary compression.

The application of a surcharge (i.e. increase in effective stress) will increase both the rate of primary consolidation and secondary compression. Subsequent permanent removal of part of this load will cause the underlying soil to go through a phase of primary rebound before then continuing to undergo a slightly lesser rate of secondary compression, denoted C'_α. Post-surcharge creep settlements can then be analysed by estimating the secant secondary compression index C''_α for a given time from primary rebound. Based on the following Mesri relationship the creep settlements post surcharge removal can be estimated by Equation 4.

$$S = \frac{C''\alpha}{1+e_0} L_o \log\frac{t}{t_l} \tag{4}$$

Where C''_α = Post-surcharge Secant Secondary Compression index, L_o = preconstruction thickness of compressible layer with initial void ratio e_0, t = time, t_l post surcharge time at which secondary compression appears.

The largest reduction in creep settlement can be achieved by ensuring the preload effective stresses are significantly higher than the post-surcharge final effective stresses. The time related values in equation 4 above are related to the ratio of preload effective stress to final effective stress (Rs), i.e. the magnitude of over-consolidation induced preload governs creep.

6 COMPARISON OF LABORATORY AND SETTLEMENT MONITORING DESIGN HEIGHTS

The use of equation 4 above has been used to determine a suitable pre-load height, which limits post-surcharge creep settlements to within a nominal 50mm total settlement and 25mm differential over a 50 year design period. Creep settlement of this magnitude are not considered likely to affect the proposed residential dwellings constructed on top of the minimum 1.0m of structural filling.

An iterative design process was implemented to determine the preload design. Varying surcharge heights were selected relative to the pre-filling and post surcharge fill conditions until the long-term creep settlements were reduced to less than 50mm over 50 years.

Surcharge heights were also varied to cover the range of peat thicknesses (2m, 4m, and 6m) to sufficiently reduce the differential creep settlements for different areas. Using the back-calculated parameters surcharge heights of 1.0m, 1.5m and 2.0m were estimated for the respective peat thicknesses. The preload design heights derived from laboratory test data were 0.5m, 1.0mm and 1.5m higher than those predicted using the back-calculated values. Durations for the preloads were also given for the various peat heights of between 3 months and 9 months.

7 CONCLUSIONS

The resulting preload design heights for the laboratory parameters are between 50% and 75% greater than those predicted from the back calculated parameters. Significantly higher construction costs would have been incurred if the surcharge design heights were solely based on laboratory derived parameters.

It is likely that a combination of factors has contributed to the difference between the laboratory and settlement monitoring derived parameters:

- Firstly, the soils samples within the laboratory environment may have undergone an increased rate of decomposition due to the exposure of the sample to oxygen, therefore a higher rate of settlement may be predicted, as suggested by Van der Heijden et al. (1994).

- Secondly, the difference in values may be due to highly variable components within the peat. As seen within the site investigation the peat comprised both fibrous plant / wood materials that were suspended within an amorphous matrix of more compressible organic material. Both of these components have very different properties in terms of long-term settlements. The push-tube soil samples retrieved from the field were relatively narrow (50mm in diameter) and did not allow for large inclusions of organics. It is likely that the samples primarily comprised amorphous peat, and therefore in general provided Cc and Cv values that were much higher than those seen within the back-calculated parameters. In contrast the settlement monitoring data derived parameters as a result of a widespread load which affects a much larger area and may therefore present more realistic values. The difference in settlement parameters could therefore be explained by the presence of less decomposed wood fragments / fibrous materials generally not accounted for within the laboratory samples.

- Thirdly, it is reasonable to also conclude that the number of samples may have not adequately picked up the variability within the peat soil. A small sample size will not take into account discontinuities, lenses, roots, or fissures of a macro scale and therefore the permeability provided by the laboratory is typically considered to provide a lower bound estimate. Further, the laboratory testing only considers vertical permeability and in reality the horizontal permeability will govern as is typically at a faster rate due to the fabric of the soil contributing to secondary compression.

- Fourthly, the lab samples were not undertaken in the same location as the trial embankment and therefore ground conditions beneath the trial embankment may differ slightly from those obtained within the test samples.

A larger laboratory test data set may have provided a more representative average, which would have more accurately reflected the back-calculated values. Future laboratory testing may be better to target more frequent tests within a single profile to obtain a more reliable average of the peat matrix throughout its vertical extent. It is recommended that a trial embankment is undertaken where practical to validate laboratory testing.

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