APPROACH FOR ASSESSING TIME OF PRELOAD AND SURCHARGE REMOVAL OF EMBANKMENTS ON SOFT SOILS

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ABSTRACT

The process of preload release involves a review of the instrumentation and monitoring data. Back analysis is carried out to match field measurements with numerical predictions by adjusting relevant geotechnical model, parameters and construction sequence. Once a match is achieved, the calibrated geotechnical model is used for the prediction of long-term settlement. The removal of preload and surcharge fill is only allowed via the release of a Hold Point, when the predicted long term settlement satisfies the design criteria. This paper provides technical advice and guidance to undertake geotechnical review of preload performance as part of the Hold Point release process.

1 INTRODUCTION

Preload and surcharge with or without prefabricated vertical drains is a commonly adopted soft ground treatment method. In order to confirm that the soft ground has been suitably treated, a process of preload and surcharge release is in place during construction. The removal of preload and surcharge is only allowed via the release of a Hold Point by a nominated verified, certifying that the validated predicted total and differential post construction settlements meet the design criteria. The process of preload performance assessment involves three main steps that can be outlined in Fig. 1.

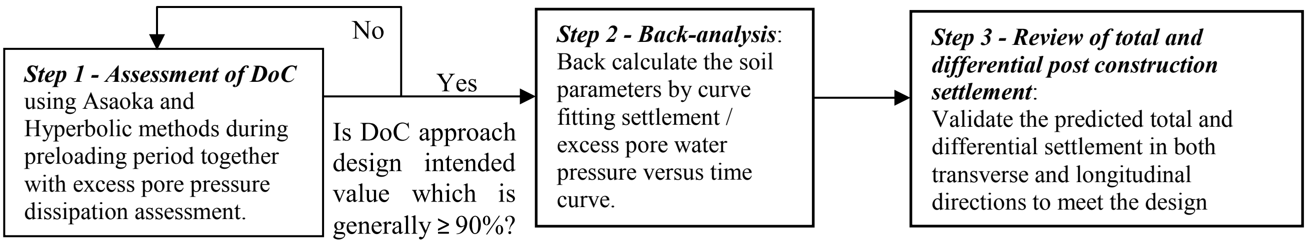

Figure 1: Three main steps involved in the preload performance assessment

By way of showing some examples cited from two soft ground projects, namely, Harwood Bridge project and Warrell Creek to Nambucca Heads (WC2NH) project, this paper provides technical advice and guidance to geotechnical practitioner on the three aspects of Hold Point release process outlined in Fig.1.

2 ASSESSMENT OF DEGREE OF CONSOLIDATION

The instrumentation data should be closely monitored during the preloading period. The objective is to assess the DoC prior to carrying out more rigorous back-analysis of the soil properties. Two observational methods, namely, Asaoka (1978) method and Hyperbolic method proposed by Tan *et al*. (1991) can be used to assess DoC, which are then compared with other DoC values assessed based on pore water pressure measured from the vibrating wire piezometers (VWP).

Asaoka (1978) method is a commonly used graphical method to estimate final total primary consolidation settlement from the settlement data obtained during a certain time period after fill placement. The measured time-settlement curve is plotted to an arithmetic scale, and the total primary consolidation settlement is given where the straight line fitted through the points during the primary consolidation stage plotted as (s_{t-1}, s_t) intersects the 45° line ($s_{t-1} = s_t$). The disadvantage with this method is that it is strongly affected by the choice of time interval Δt used in constructing the Asaoka diagram. Asaoka (1978) states that the accuracy of the graphical method depends on the magnitude of Δt, with larger Δt values giving greater accuracy. The use of a small Δt should be avoided. Fig. 2c shows an Asaoka plot constructed using $\Delta t = 5$ days, with the initial settlement data taken as the settlement at $t_0 = 105$ days (i.e. Point A in Fig. 2a), which was the beginning of the full load. Due to the relatively small Δt adopted, the plotted data are almost sub-parallel to the 45° line. The assessment of the total primary settlement has become very sensitivity to the subjectivity in fitting a line through these points, leading to the uncertainty of the estimated DoC value ranging from 90% to 98%. Edil *et al*. (1991) proposed a parameter j_{95} and recommended that Δt to be chosen to give a j_{95} value between 10 and 30. The j_{95} is essentially the number of samples to reach a 95% DoC and is defined as:

$$j_{95} = \frac{ln(0.05s_p/(s_p - s_0))}{ln\beta_1} \quad (1)$$

where s_0 is the settlement at the end of the field construction period after which the added embankment load is constant. s_p is defined as $s_p = \beta_o / (1 - \beta_1)$. β_o is the intercept and β_1 is the slope of the straight line in the s_t vs. s_{t-1} plot. The use of Δt = 5 days in Fig.2c gives a j_{95} value of about 60, which exceeds the recommended range suggested by Edil *et al*. (1991). By adopting Δt =20 days, the corresponding j_{95} = 20 is within the recommended range of 10 – 30, and the Asaoka plot shown in Fig. 2d exhibits a more oblique trend to the 45° line than that of using a smaller Δt value. The assessed DoC = 91% based on Δt =20 days is therefore considered to be a more reliable prediction than that using Δt =5 days.

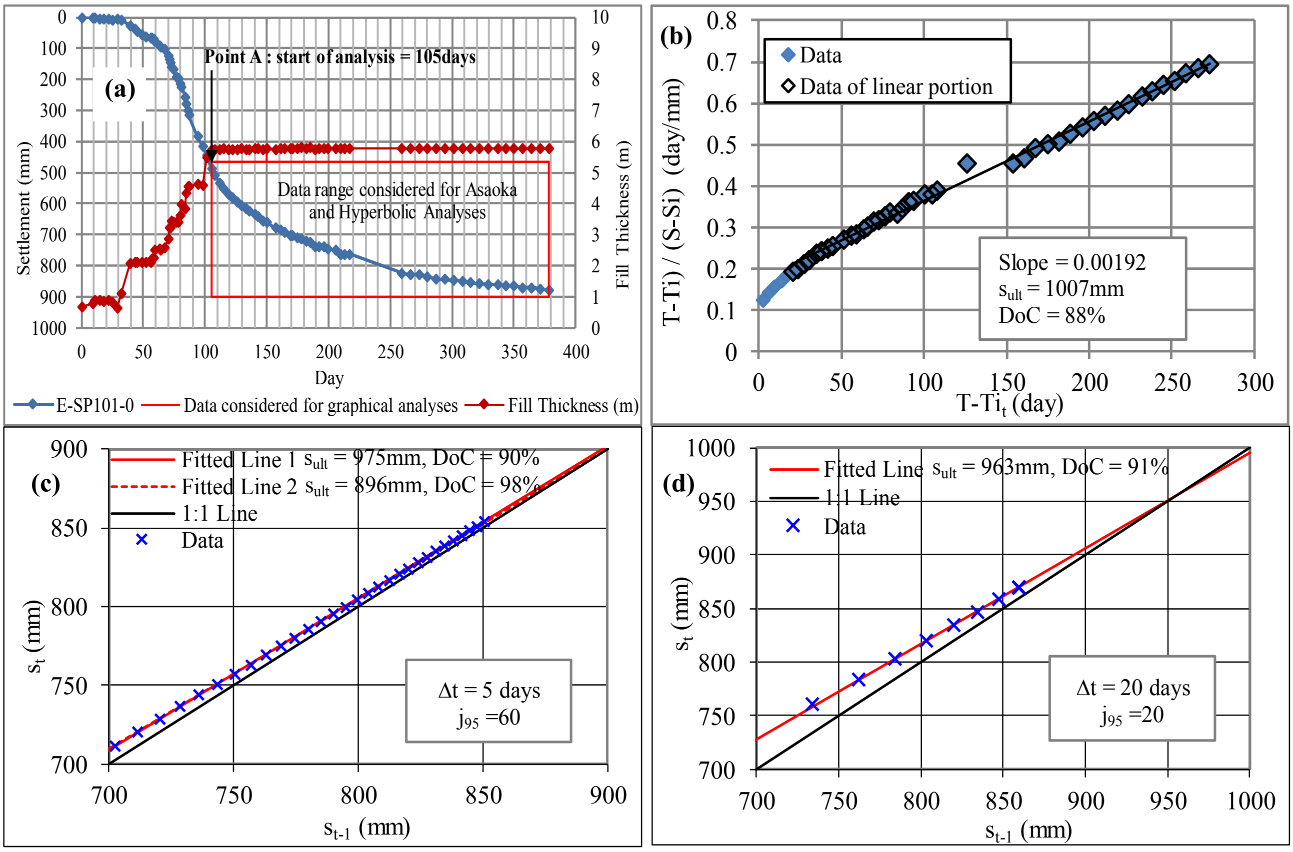

Figure 2 – (a) SP101 data, (b) Hyperbolic analysis, (c) Asaoka analysis (Δt = 5 days), (d) Asaoka analysis (Δt = 20 days)

Tan *et al*. (1991) proposed a hyperbolic relationship between monitored settlement, *s*, and consolidation time, *t*, given by $t/s = \alpha + \beta t$. Hence the ultimate settlement s_{ult} is defined as:

$$s_{ult} = \lim_{t\to\infty} s = \frac{1}{\beta} \quad (2)$$

where α and β are the intercept and slope of the initial linear line. The above equation literally means that when time tends to infinity, the inverse of the slope of the graph at linear segment will give the ultimate settlement. The main attraction of the hyperbolic method is its simplicity without the need to consider Δt value. Unlike the Asaoka's method that is based on a governing partial differential equation that considers only the primary consolidation, the hyperbolic method is based on actual data, which intrinsically includes creep settlement. As a result, the ultimate settlements obtained from the hyperbolic method are usually slightly larger than those obtained from Asaoka's method. The settlement plate SP101 data from Harwood Bridge project was assessed in Fig. 2b using hyperbolic method. The estimated ultimate settlement is compared with those assessed using Asaoka (1978) method as show in Table 1 (also include other settlement plate data in the preloading area). It can be seen that the settlements obtained from hyperbolic method are slightly larger than those obtained from the Asaoka's method, irrespective to the time interval adjustment made in the latter methods.

The assessment of DoC should also consider the excess pore water pressure measured from the vibrating wire piezometers (VWP). In the interpretation of excess pore water pressure, it is important to correct the measured total pore water pressure for the increase in pressure head due to the settlement of piezometer under load. The settlement of the piezometer with time can be estimated from the registered settlement of the magnet in the nearby extensometer. By way of an example, Fig. 3 shows a higher excess pore water pressure inferred from a piezometer at RL-4.32 m in E-VP101 (from Harwood Bridge project) than if it was correctly interpreted with settlement correction. The corrected excess pore pressures of all piezometers fat E-VP101 are summarised in Table 2. Also refer to the nearby ACPT010 shown in Fig. 5a and the nearby boreholes, the soil thicknesses of the upper and lower clay layers, separated by a drainage layer, are 6 m and 4 m, respectively. By considering two-way drainage for each of the clay layers, the corresponding drainage distances H are 3 m and 2 m. The 'z' in Table 2 refers to the depth of the piezometer relative to the drainage boundaries. Uz (i.e. DoC at depth z) were then calculated based on the total applied fill load of 108kPa. By plotting Uz vs z/H for the upper clay layer (values calculated in Table 2) on the chart in Fig. 5b, the time factor T for the upper clay layer can be assessed to be about 0.8. The assessed average DoC from the chart given in Fig. 5c is about 88% based on T=0.8. This is consistent with graphical analysis results. In summary, the Asaoka method, the hyperbolic method and the measured excess pore pressure from the VWP indicated that the DoC at the northern abutment at the end of the preloading period was about 90 ± 2%.

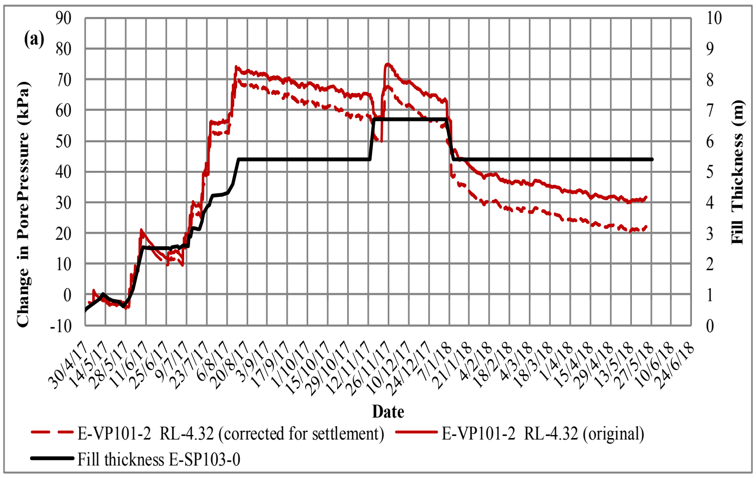

Figure 3: Excess pore pressure – time curve for piezometer at RL-4.32m in E-VP101 (Interpretation with and without correction for settlement)

Table 1: Summary of predicted DoC values

Settlement plate ID	Predicted total settlement (mm)		Assessed DoC (%)	
	Asaoka	*Hyperbolic*	*Asaoka*	*Hyperbolic*
E-SP101	963	1007	91	88
E-SP102	950	995	93	90
E-SP103	997	1060	91	89
E-EM101	1000	1065	92	89

Table 2 –Predicted Uz of piezometers at E-VP101

Layer	RL (mAHD)	z (m)	z/H	Excess pore pressure (kPa)	Uz(1)
Upper Clay: Drainage distance H = 3m					
Upper Clay	-1.02	0	0	0	100
	-4.32	3.3	1.1	20	81
Sand Layer					
Sand	-4.32	–	–	0	100
Upper Clay: Drainage distance H = 2m					
Lower Clay	-14.82	2.14	1.07	0	100

(1) Uz calculated based on a total applied load of 108kPa

3 BACK ANALYSIS PROCESS

Back analysis is usually carried out towards the end of the preload waiting period or when the assessed DoC values (based on Asaoka, Hyperbolic and VWPs) have reached the intended DoC as per design, which is in general 90% or greater. The objective of carrying out the back analysis is to verify/adjust the consolidation and compressibility parameters by curve-fitting the recorded settlement-time curves and the excess pore pressure measurements. This is followed by the review of the long term creep settlement predictions based on the actual final effective stress after the preloading and surcharging in order to validate the predicted total and differential settlement in both transverse and longitudinal directions to meet the design criteria. The discussion of post surcharge creep settlement is provided in Section 5. This section focuses on the back-analysis of engineering parameters. The analysis aspects involved in the back-analysis are provided in Section 4. In the curve fitting process as part of the back-analysis, the adjusted engineering properties shall always be reviewed by comparing with laboratory and in-situ test data, as well as published correlations. A methodological approach is outlined herein in order to provide guidance for undertaking the back-analysis.

Step 1 – Review the soil stratigraphy and total soil depth – In the back-analysis of the settlement plate and extensometer data, the soil layer thicknesses of the adopted geotechnical model shall be accurately defined based on the nearest CPTs, even though they may be different to those of the original design model. The soil model adopted in the detailed design is for an area whereby the soil depth could be the deepest or the average within the area. There is no guarantee that the soil layer thicknesses or total soil depth used in the design are representative of those at the monitoring locations. For the case of Harwood Bridge project, the soil stratigraphy has been reviewed during the back-analysis of settlement plate E-SP103 based on the nearby ACPT010. It can be seen that the upper and the lower clay thicknesses are 5.5 m and 3.6 m, respectively (see Fig. 5a), which are greater than the corresponding thicknesses of 4.5 m and 3 m adopted in the design.

Step 2 – Adjust the over consolidation ratio (OCR) and review of ground water level – Over consolidation ratio (OCR) is one of the most influential parameter affecting the predicted total settlement. During the detailed design stage of Harwood

Bridge project, the OCR values of the clay layers were assessed by using a numerical "aging" technique in conjunction with the sophisticated Soft Soil Creep Model embedded in the commercially available FEA software program PLAXIS 2D. For the case of WC2NH project, the design OCR values were derived based on SHANSEP equation (Eq. 3) and using the SHANSEP parameters of m = 0.8 and S = 0.22. In both design cases, it was found that using the design OCR values underestimate the measured total settlements. During the construction stages of both projects, the OCR values were correlated with the S_u inferred from the nearest CPT via SHANSEP while adopting m = 0.95 and S = 0.22 (see Fig. 5b and 5c). This set of SHANSEP parameters has been used in many soft soil projects along the east coast of Australia with success. It has also been used by the Authors to predict the behavior of a well-known trial embankment in Ballina (see Chan *et al.* 2018). The back-analysed OCR values via SHANSEP approach is strongly influenced by the ground water level since the vertical effective stress σ'_v is used in Eq. 3. It is therefore imperative to review the ground water table via monitoring data as part of the back-analysis process.

$$S_u/\sigma'_v = S \times OCR^m \tag{3}$$

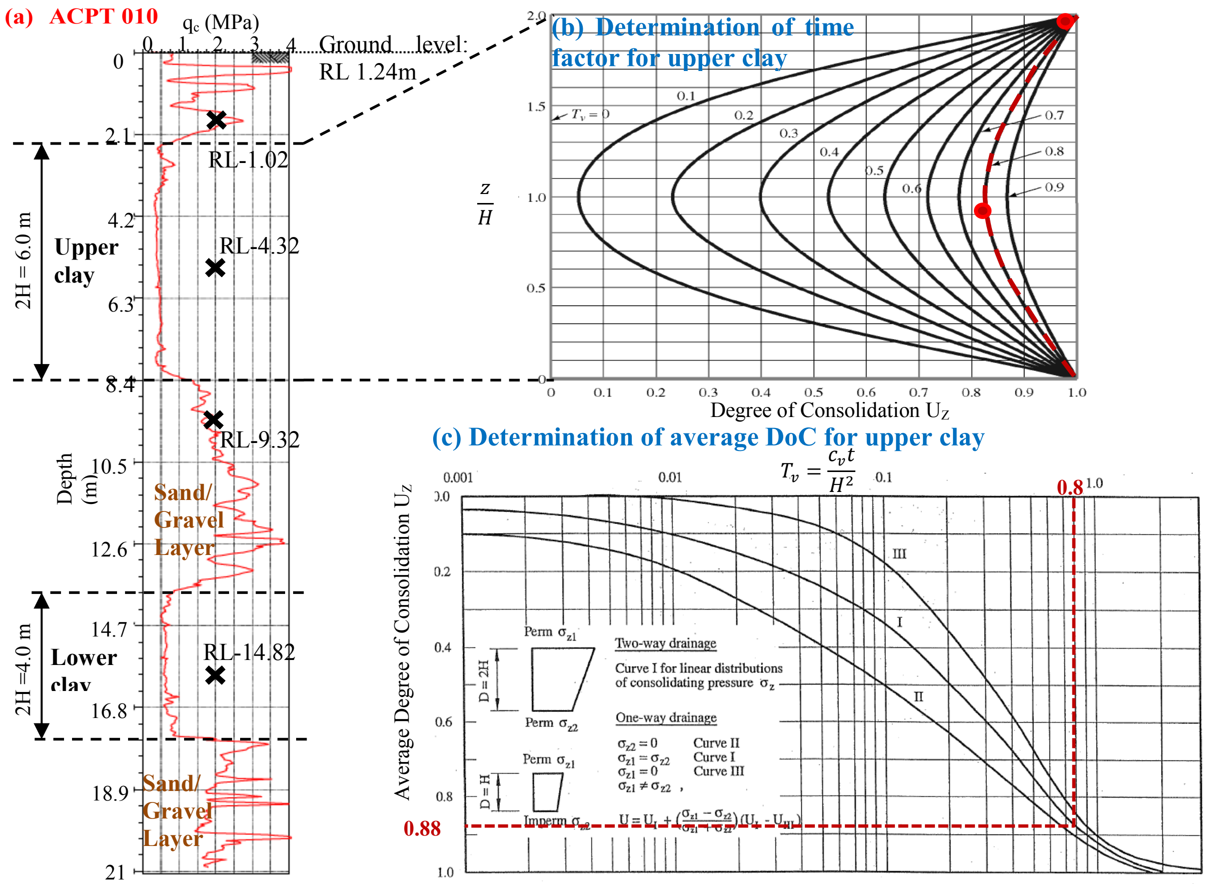

Figure 4: (a) ACPT010, (b) Time factor for upper clay layer, (c) Average DoC for upper clay

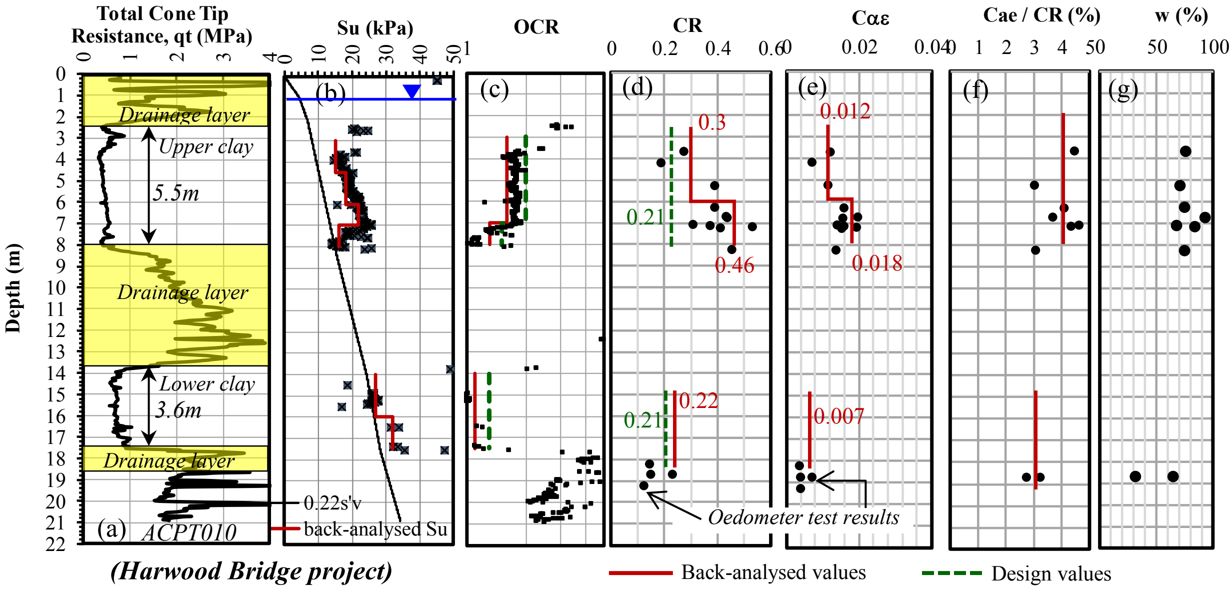

Figure 5: (a) ACPT010 q_t, (b) CPT inferred S_u, (c) CPT inferred OCR, (d) CR, (e) $c_{\alpha\varepsilon}$, (f) $c_{\alpha\varepsilon}$ /CR, (g) moisture content with depth

Step 3 – Review the compression Ratio (CR) and recompression Ratio (RR) – While OCR and water level are the major parameters influencing the predicted total settlement. The adopted compression ratio (CR = $c_c/(1+e_0)$) and recompression ratio (RR = $c_r/(1+e_0)$) should also be reviewed (or adjusted if required) for their consistency with oedometer test data as part of the back-analysis process. It is not unusual to adopt a higher compressibility values than the laboratory results that are derived from disturbed soil samples. Take Harwood Bridge project for example, the adopted CR value in the design was 0.21, which lay close to the lower bound of the oedometer test results (Fig. 5d). For back-analysis, the CR value was increased to become more aligned with the upper bound of the test results. Moreover, the upper clay layer was divided into two sub-layers, with the lower layer having a higher back-analysed CR value. This was consistent with the high compressibility identified by the oedometer test data, as well as the high water content exhibited within this sub-layer (Fig. 5g). The RR value may need to be adjusted in order to better fit the proportion of settlement occurred before and after OCR =1. RR can be correlated with CR. Typical RR/CR ratio is about 0.14 and with a likely range of 0.1 to 0.2.

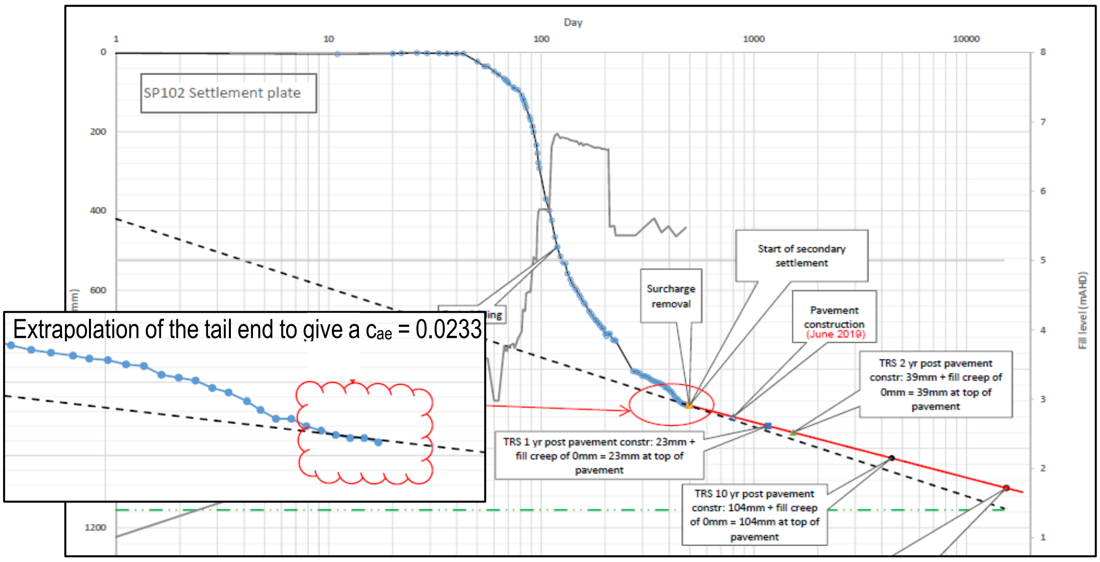

Figure 6: Extrapolation of the tail end of SP102 data (on Harwood Bridge project) to assess $c_{\alpha\varepsilon}$ (not recommended)

Step 4 – Review the normally consolidated creep strain rate $c_{\alpha\varepsilon\,(NC)}$ – Unless otherwise suggested by laboratory test data, the normally consolidated creep strain rate $c_{\alpha\varepsilon\,(NC)}$ can be taken as 4% of the back-calculated CR in accordance with the

c_α/cc law of compressibility for inorganic clay proposed by Mesri and Godlwski (1977). Figs. 5d, e and f show that the back-analysed $c_{\alpha\varepsilon\,(NC)}$ values based on the correlation are consistent with the oedometer test results on Harwood Bridge project. Fig. 6 shows a case where the tail end of the settlement vs. time curve was interpolate to derive a straight line. This linear line shall not be misrepresented as the pure creep settlement behavior since it may comprise a large proportion of residual primary settlement. It is the authors' opinion that using the observed settlement rate at the tail end to assess the creep rate is likely to result in excessive post construction settlement prediction. The $c_{\alpha\varepsilon\,(NC)}$ should more appropriately be assessed from the back-analysed CR value, and compared with the laboratory test data.

Step 5 – Refine the shape of the fitted curve – The shape of the fitted curve is largely influenced by (i) drainage boundary of the clay layers, (ii) load history and (iii) adopted coefficient of consolidation values c_v. With regard to the drainage boundary, it is imperative to review the sand-clay boundaries based on the area specific CPTs such as that outlined in Fig.5a. The applied fill load in the back-analysis should also follow closely the loading history. An example of which is shown in Fig. 10. In relation to c_v, the back analysis should consider firstly the normally consolidated $c_{v(NC)}$ by comparing the back-analysed values with piezocone measurements as well as published correlation, followed by using appropriate over-consolidated $c_{v(OC)}$ values. Fig.7c presents c_v values assessed from piezocones on WC2NH project. The c_v values were assessed based on $c_v = c_h/2$ (based on Beales and O'Kelly, 2008 and our experience in the region), where c_h is the coefficient in the horizontal direction inferred from the dissipation test results. The c_v values in Fig.7 are plotted side by side with the OCR profiles inferred from the corresponding piezocones. If the inferred c_v within the same soil unit (Unit 2a) are plotted against the corresponding OCR on a log-log plot as shown in Fig. 8b, a prominent linear trend can be seen and the intersection of the median line of this trend line with OCR = 1 gives an estimated $c_{v(NC)}$ of about 12 m^2/yr. NAVFAC (1971) developed correlations of c_v of remoulded, normally consolidated and over-consolidated clays with their liquid limits (LL) as shown in Fig.8a. The Unit 2a on WC2NH project has a LL range of 35% – 50%. By using a mean LL of 42%, the correlated $c_{v(NC)}$ is about 8 m^2/yr, which is consistent with that inferred from field measurements in Fig.8b. During the design stage, a conservative design trend line of c_v vs. OCR was adopted (on the safe side of the median line) and the adopted design $c_{v(NC)}$ and $c_{v(OC)}$ values were 7 m^2/yr and 20 m^2/yr, respectively. During construction, the back-analysed $c_{v(NC)}$ and $c_{v(OC)}$ were 15 m^2/yr and 30 m^2/yr. These back-analysed values, while greater than the design values, are more aligned with the median trend line as. The linear c_v vs. OCR trend shown in Fig. 8c is material type specific. Poon and Chan (2015) have shown that similar linear trends can be created for other soil types on the double log space. But the slope and y-intercept (i.e. $c_{v(NC)}$ at OCR = 1) would be different and are dependent on LL.

For ground treatment using prefabricated vertical drains, the smear zone characteristic of the drains such as the smear radius ratio and the permeability ratio should also be considered. For design and back-analysis purposes, the smear radius ratio of 8 -11 and permeability ratio of 2 based on the research works by Indraratna et al. (2015) may be applied.

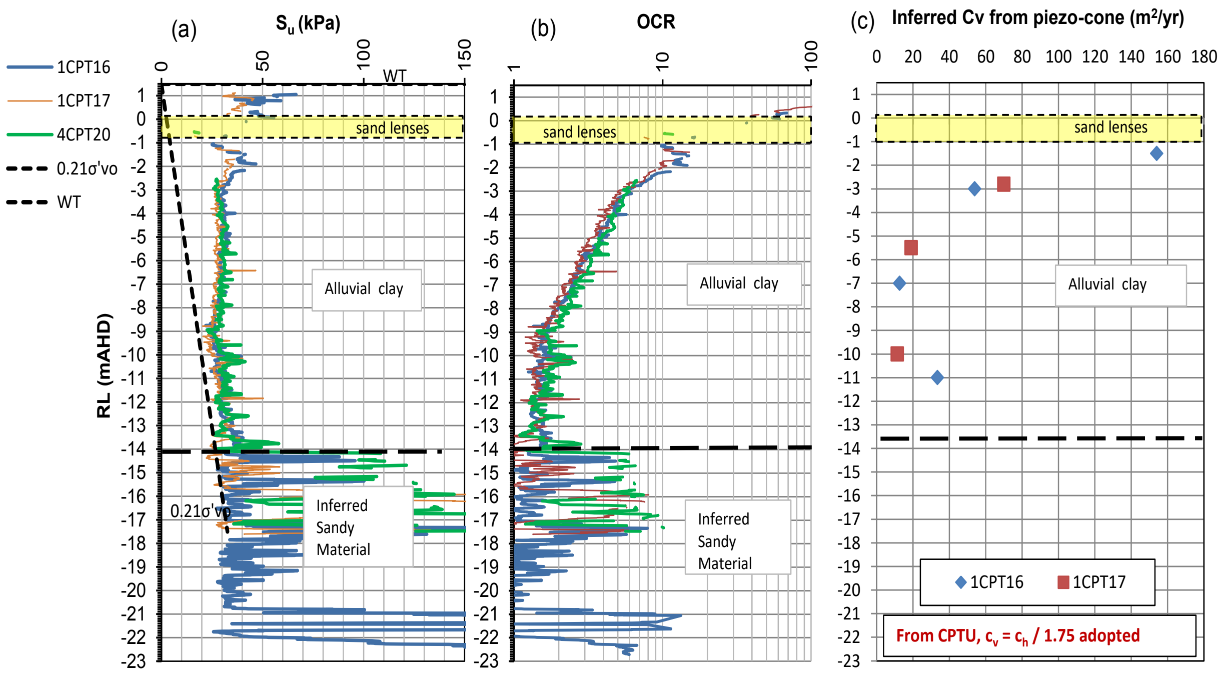

Figure 7: Typical profiles of (a) S_u, (b) OCR and (c) c_v vs. RL inferred from piezocones on WC2NH project

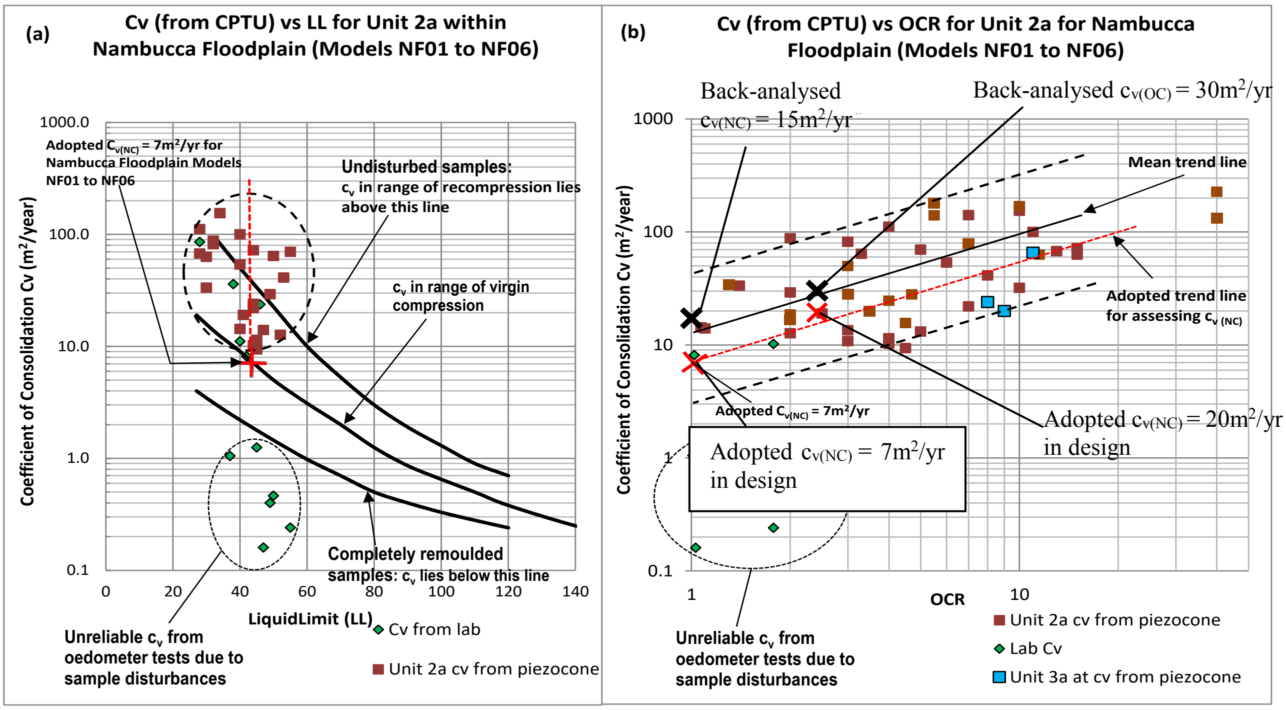

Figure 8: (a) c_v vs. LL, (b) c_v vs. OCR for Unit 2a from WC2NH project

4 ANALYTICAL TECHNIQUES FOR BACK-ANALYSIS

For back-analysis purposes, it is the Authors' preference to deploy simple 1D consolidation model in which creep settlement is assumed to commence at the end of the primary consolidation (considered to be at 90% DoC). Notwithstanding this, a more sophisticated Soft Soil Creep (SSC) model that is embedded in the commercially available software program PLAXIS 2D has gained much popularity in recent years. This model utilizes a form of time-dependent concepts of Modified Cam-Clay and visco-plasticity, thus taking into account of the simultaneous nature of primary consolidation and creep. This section provides some remarks on the use of SSC for back-analysis.

Permeability – In the FEA model, c_v is not a direct input, but is a composite parameter that depends on both the coefficient of permeability, k, and the coefficient of volume compressibility of the soil, m_v. The m_v value is calculated within the FEA program based on the input parameters of CR and RR, whereas k is related to the void ratio, e, in accordance with the permeability function: $e = e_0 + c_k \log (k/k_0)$, where e_0 is the initial void ratio, k_0 is the initial permeability value and c_k is the permeability index. k_0 and c_k are the input parameters to FEA to define the permeability function. For clays, k_0 is typically about 1e-4 m/day. k_0 can be also be converted from c_v using the following equation:

$$k_0 = k_v = 0.434 \times CR \times c_v \times \gamma_w / \sigma_v' \qquad (4)$$

where γ_w is the water unit weight. The permeability index c_k is an influential parameter that affects most of the post peak excess pore pressure dissipation rate. The smaller the adopted c_k value, the slower is the rate of excess pore pressure dissipation, and the flatter is the slope of the predicted excess pore pressure vs. log time curve after fill placement as demonstrated by an example presented in Figure 9. Tavenas *et al.* (1983) proposed a correlation of $c_k = 0.5e_0$, but can be as low as $c_k = 0.25e_0$ as pointed out by Chan *et al.* 2018.

Buoyancy effect – Fig. 10 shows a fitted settlement-time curve for a settlement plate on WC2NH project using 1D consolidation analysis. Using the same set of back-analysed parameters from 1D analysis, it can be seen that the prediction given by the 2D FEA with SSC model overestimated the measured settlement. Noted that the FEA undertaken was a small strain analysis applying a constant load with time after fill placement. However, the embankment has settled a maximum of 1.1m during the preloading period. Part of the fill material that was originally above the ground water table (GWT) has settled below the GWT (GWT near ground surface). This entailed a load reduction of about 10kPa ≈ (1.1-0.1)×10kN/m^3. To investigate the buoyancy effect, a large strain 2D FEA with SSC model was conducted with updating mesh and pore water pressure. As can be seen in Fig. 10, the buoyancy effect has compensated the creep settlement accumulated during primary consolidation, leading to relatively reasonable estimate of the settlement compared to the measurements. The settlement prediction shown in Fig. 10 has demonstrated an important point that while the simple 1D analysis gives a similar predicted settlement-time curve compared to that given by the sophisticated large strain SSC FEA, its theoretical considerations were far less rigorous than the latter. Simple soil model could be used more consistently with simple numerical method (e.g. updating mesh or buoyancy effect may not be necessary), whereas complex soil model is best to be used in conjunction with sophisticated computational approaches (e.g. large strain analysis with updating mesh and pore water pressure). Mixing complex soil models with simplified computational approaches, or vice versa, may not give desirable outcomes.

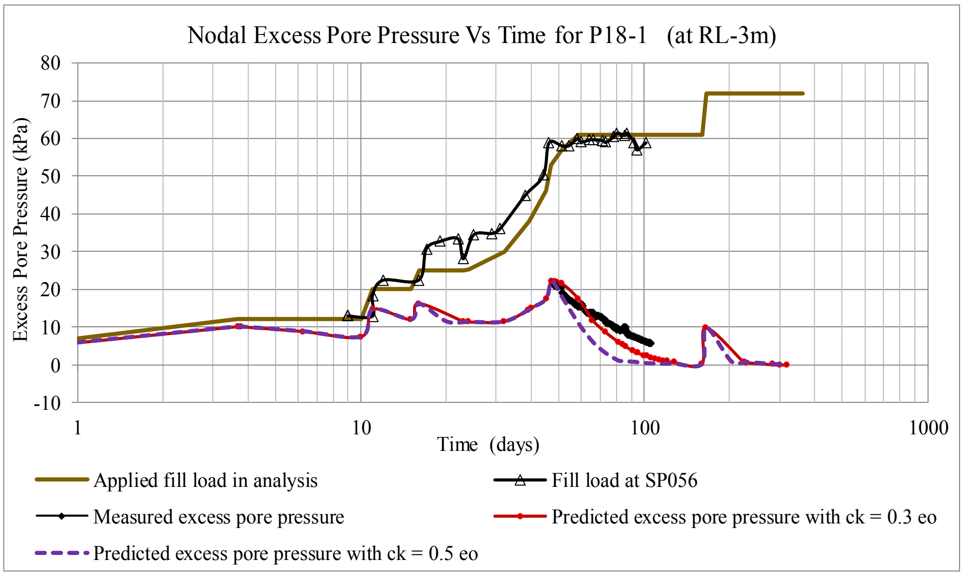

Figure 9 – Comparison of large strain SSC using c_k = 0.3 e_0 and c_k = 0.5 e_0 with measured excess pore pressure from WC2NH project

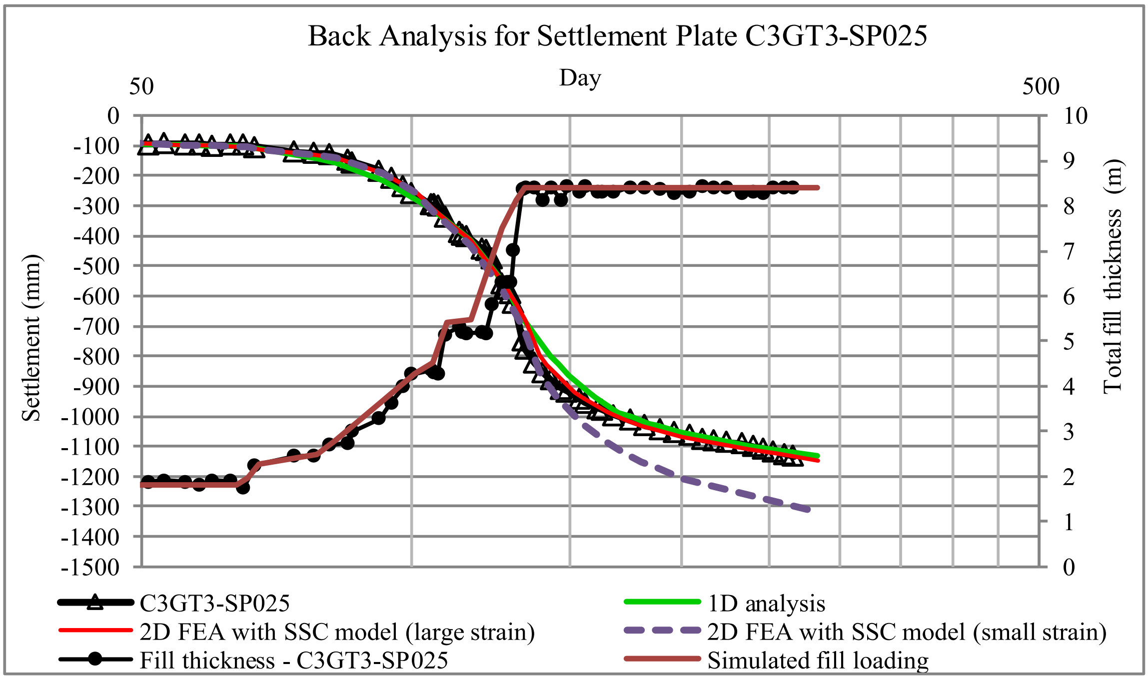

Figure 10: Numerical analysis results with measured settlement plate data (from WC2NH project)

5 POST SURCHARGE CREEP SETTLEMENT

When an over-consolidated state is induced in soil by removal of a surcharge, the creep settlement is expected to be less than if the soil remains in a normally consolidated state. The methods that are commonly used for the prediction of post surcharge creep settlement are: (i) Ladd (1989) methd, (ii) Mesri (1994) method, (iii) Yuan *et al.* (2015) method, and (iv) FEA using SSC model embedded in PLAXIS. This section provides discussion of these methods.

Based on the data presented by Ladd (1989) together with test data from recent soft ground projects in Australia including Gateway Upgrade and Ballina Bypass, Wong (2007) adopted an empirical exponential relationship for varying $c_{\alpha\varepsilon(OC)}$ to $c_{\alpha\varepsilon(NC)}$ ratio with OCR (see Fig.11) as given by:

$$\frac{C_{\alpha\varepsilon(OC)}}{C_{\alpha\varepsilon(NC)}} = \frac{(1-m)}{e^{(OCR-1)n}} + m \qquad (5)$$

In Eq. 5, the constant *m* represents the minimum value of $c_{\alpha\varepsilon(OC)} / c_{\alpha\varepsilon(NC)}$. The magnitude of *n* controls the rate of reduction of $c_{\alpha\varepsilon(OC)} / c_{\alpha\varepsilon(NC)}$ with OCR. In the absence of site specific test data, the *n* value of 6 may be adopted for organic clays for preliminary assessment purposes. Also shown in Fig. 11 is a new $c_{\alpha\varepsilon(OC)} / c_{\alpha\varepsilon(NC)}$ vs. OCR function proposed by Yuan *et al*. (2015) based on separate creep test data:

$$\frac{C_{\alpha\varepsilon(OC)}}{C_{\alpha\varepsilon(NC)}} = \frac{2}{(OCR^{7.2}+1)} \qquad (6)$$

The Mesri (1994) method prescribes an over consolidated secant creep index to determine settlements after creep has recommenced. Fig.12 compares PLAXIS SSC model results and the Mesri expression for the ratio of start time of creep to rebound time. In the PLAXIS analysis rebound time (t_{pr}) is taken as 0.1 days. The start time of creep (t_l) is assessed visually where the rate of settlement is seen to increase on the settlement curve. For a given OCR, Mesri (1994) suggests that the time prior to the commencement of creep will be shorter than predicted by PLAXIS. Fig. 13 compares the secant creep index relationships at various OCR values. The rates of creep computed by PLAXIS are greater than given by Mesri (1994) for a given ratio of t/t_l. It is interesting to note that PLAXIS suggests that the secant creep strains for OCRs in the range of 1 to 1.2 will be similar, although this is very sensitive to the value of t_{pr} selected in the assessment. PLAXIS appears to broadly give results closer to Mesri (1994) in modelling post unloading creep than Ladd (1989) or Yuan *et al.* (2015) since these methods prescribes a constant creep index for all time. PLAXIS has the advantage over Mesri (1994) that it calculates creep settlements when the OCR falls below 1.2. When surcharging is used to consolidate deep soft clay layers, the lower clays may have OCR values less than 1.2 after unloading.

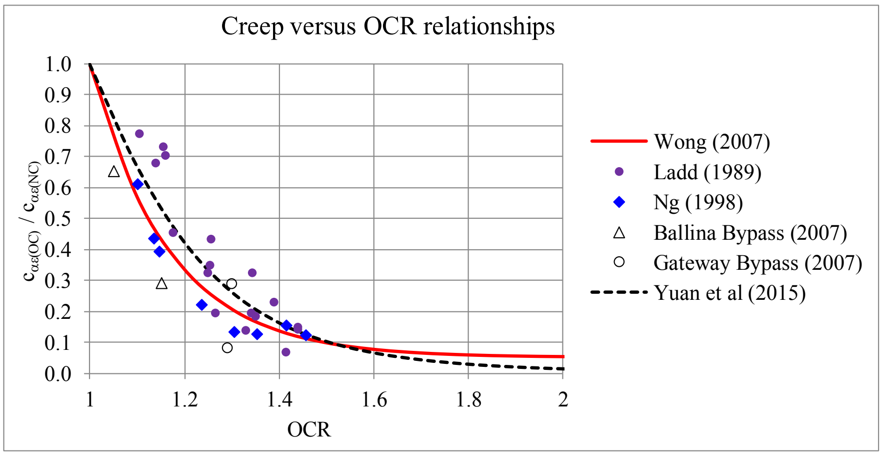

Figure 11: Creep versus OCR relationships

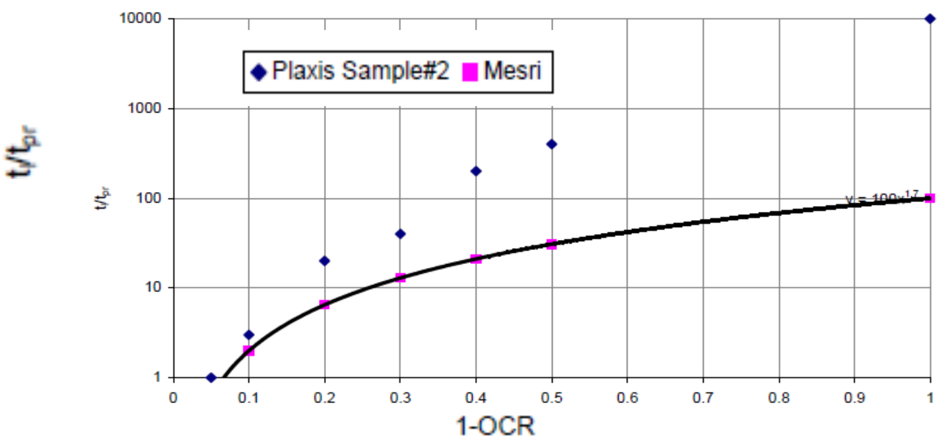

Figure 12: Start time of creep – PLAXIS SSC model compare with Mesri (1994)

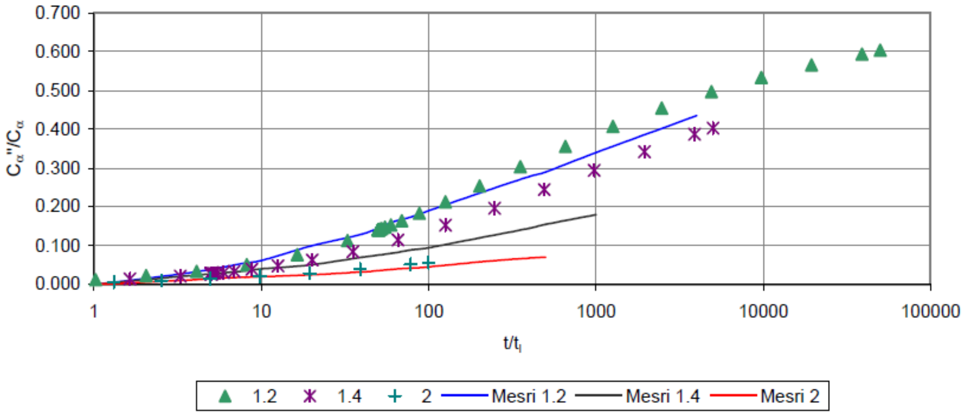

Figure 13: Secant creep index – PLAXIS SSC model compare with Mesri (1994)

6 CONCLUSIONS

The following points can be made in relation to the performance assessment undertaken as part of the preload and surcharge release process:

- The use of hyperbolic method to assess DoC is attractive due to its simplicity without the need to consider Δt. The ultimate settlements predicted by hyperbolic method are usually slightly larger than those from Asaoka's method. This is because the hyperbolic method is based on actual data, which intrinsically includes creep settlement, whereas Asaoka's method is based on a governing partial differential equation that considers the primary consolidation only.
- The assessment of DoC should also consider the excess pore water pressure measured from the VWP. In the interpretation of VWP data, it is important to correct the measured total pore water pressure for the increase in pressure head due to the settlement of piezometer under load.
- Back analysis is usually undertaken at the end of the preloading period or when the assessed DoC values have reached 90% or greater. The main factors affecting the predicted total settlement in the back-analysis are (i) soil stratigraphy and soil depth, (ii) adopted OCR values and (iii) ground water level. These elements should be reviewed based on the nearest CPTs to the monitoring location as opposed to relying on the original design model.
- For the assessment of field DoC using Asaoka's method, the use of small time interval Δt should be avoided. Edil *et al*. (1991) proposed a parameter j_{95} and recommended that Δt to be chosen to give a j_{95} value between 10 and 30.
- SHANSEP equation with parameters m = 0.95 and S = 0.22 has been used by the Authors on numerous soft ground designs and back-analyses with success.
- The $c_{\alpha\varepsilon\,(NC)}$ is best to be correlated with the back-analysed CR value c_α/cc law of Mesri. Using the observed settlement rate at the tail end to assess the creep rate is likely to result in excessive post construction settlement prediction.
- c_v is usually used as input 1D consolidation analysis. The back-analysed c_v values should be compared with the field piezocone measurements as well as published correlation, For 2D FEA, the input permeability k is linked to the void ratio e via the permeability index c_k, which has a great influence on the post peak excess pore pressure dissipation rate.
- For design and back-analysis purposes, the smear radius ratio of 8-11 and permeability ratio of 2 based on the research works by Indraratna *et al*. (2015) may be applied.
- It is the Authors' preference to carry out simple 1D consolidation analysis for back-analysis purposes. Notwithstanding this, sophisticated soil model such as PLAXIS SSC model has gained much popularity in recent years. Simple soil model could be used more consistently with simple numerical method whereas complex soil model is best to be used in conjunction with sophisticated computational approaches (e.g. large strain analysis with updating mesh and pore water pressure). Mixing complex soil models with simplified computational approaches, or vice versa, may not give desirable outcomes.
- For the prediction of post surcharge creep settlement, the PLAXIS SSC model appears to broadly give results closer to Mesri (1994) in modelling post unloading creep than Ladd (1989) or Yuan (2015) since these methods prescribes a constant creep index for all time. The PLAXIS SSC model has the advantage over Mesri (1994) that it calculates creep settlements when the OCR falls below 1.2.

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