Use of Isotache for Long Term Radial Consolidation Analysis

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ABSTRACT: An analytical solution for radial consolidation capturing strain rate dependency with the help of isotaches is presented. The strain rate dependency of pre-consolidation pressure is obtained based on the Constant Rate of Strain (CRS) and Long-Term Consolidation (LTC) tests. The behavior of embankment in terms of the settlement and excess pore water pressure is determined using the equivalent pre-consolidation pressure from the reference isotache using the log $(\frac{\sigma_p{'}}{\sigma_{p0}{'}})$ - log strain rate $(\dot{\varepsilon_v})$ domain. The long-term behavior is calculated using the change in the secondary compression coefficient $(\frac{C_{\alpha e}}{C_c})$. This model is further validated using various case histories in Australia and Southeast Asia.

RÉSUMÉ : Une solution analytique pour la consolidation radiale capturant la dépendance du taux de déformation à l'aide d'isotaches est présentée. La dépendance du taux de déformation de la pression de préconsolidation est obtenue sur la base des tests à taux constant de déformation (CRS) et de consolidation à long terme (LTC). Le comportement du remblai en termes de tassement et de pression interstitielle excessive est déterminé en utilisant la pression de préconsolidation équivalente de l'isotache de référence en utilisant le log log $(\frac{\sigma_p{'}}{\sigma_{p0}{'}})$ - vitesse de déformation log $(\dot{\varepsilon_v})$ domaine. Le comportement à long terme est calculé en utilisant le changement du coefficient de compression secondaire $(\frac{C_{\alpha e}}{C_c})$.. Ce modèle est ensuite validé à l'aide de divers antécédents de cas en Australie et en Asie du Sud-Est.

KEYWORDS: Radial consolidation, isotaches, viscosity, and creep settlement.

1 INTRODUCTION.

Analytical solutions for radial consolidation induced by vertical drains have been developed to capture various aspects including the smear zone, stratified soils, and vacuum preloading (Barron, 1948; Tang and Onitsuka, 2001; Indraratna et al. 2005; Walker and Indraratna, 2009), however, limited efforts have been made to analytically incorporate the effect of soil viscosity influenced by the strain rate (Yang et al. 2016). Evaluating the time-dependent behavior of soft soils is crucial for predicting the correct settlement and excess pore water dissipation. There are studies considering the time-dependent behavior of soft soil using the Isotache concept (Leroueil et al., 1985; Yin et al., 1994; Kim and Leroueil, 2001; Watabe et al. 2012; Qu et al., 2010; Degago et al., 2011) in conjunction with the strain rate dependency of pre-consolidation pressure (Watabe and Leroueil, 2012; Watabe et al. 2012; Watabe et al. 2008; Tanaka, 2005). These isotaches are normally characterized by a reference compression line obtained from a constant rate of strain (CRS) test at a specific timeline and a series of long term consolidation (LTC) tests. Tsutsumiu and Tanaka (2011) introduced a special CRS test and carried out a testing program with a multiple strain rates applied at different stages. In this study, the isotache concept is incorporated into the radial consolidation model to predict settlement and the associated dissipation of excess pore water. The model is then validated using case studies.

2 ANALYTICAL MODEL FOR RADIAL CONSOLIDATION

Based on the distinct relationship between the strain rate and pre-consolidation pressure (Watabe and Leroueil 2015), a strain rate dependency relationship with pre-consolidation pressure can be established using LTC and CRS tests (Fig. 1). The upper bound of the isotache represents the compression curve under the laboratory environment with a higher strain rate whereas other isotache lines resemble compression curves at a lower strain rate.

Watabe and Leroueil (2015) proposed the relationship between pre-consolidation pressure and strain rate as:

$$\sigma_p{}' = f(\dot{\epsilon_v}) \quad (1)$$

In the above, the exponential relationship between the pre-consolidation pressure and the strain rate can be determined from:

$$\ln \sigma_p{}' = a_1 + a_2 \ln \dot{\epsilon_v} \quad (2)$$

$$\sigma_p{}' = \sigma_{pL}{}' + b_1 \exp(b_2 \ln \dot{\epsilon_v}) \quad (3)$$

$$\ln \frac{\sigma_p{}' - \sigma_{pL}{}'}{\sigma_{pL}{}'} = c_1 + c_2 \ln \dot{\epsilon_v} \quad (4)$$

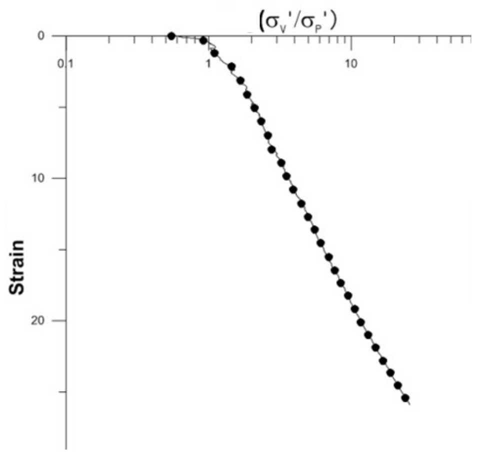
$$\dot{\epsilon_v} = \ln\left(\frac{\sigma_p{}' - \sigma_{pL}{}'}{\sigma_{pL}{}'}\right)^{\frac{1}{c_2}} \quad exp^{-\frac{c_1}{c_2}} \quad (5)$$

$$\dot{\epsilon_v} = c_3\left(\frac{\sigma_p{}' - \sigma_{pL}{}'}{\sigma_{pL}{}'}\right)^{c_4} \quad (6)$$

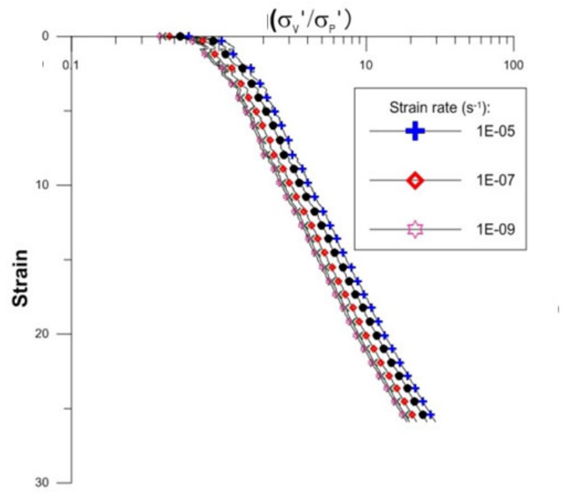
where
$\sigma_p{}'$ = preconsolidation pressure at a given strain rate,
$\sigma_{pL}{}'$ = lower limit ofpreconsolidation pressure
$\sigma_{p0}{}'$ = preconsolidation pressure when $\dot{\epsilon_v} = 1 \times 10^{-7} s^{-1}$,
$\dot{\epsilon_v}$ = axial strain rate

c_1, c_2, c_3 & c_4 are four constants used in the model

The lower limit of pre consolidation pressure $(\sigma_{pL}{}')$ can be calculated based on Eq. (6). The concept of the above-mentioned procedure in Fig. 1. This analysis was also applicable for clay samples obtained from Ballina, Australia.



(a)



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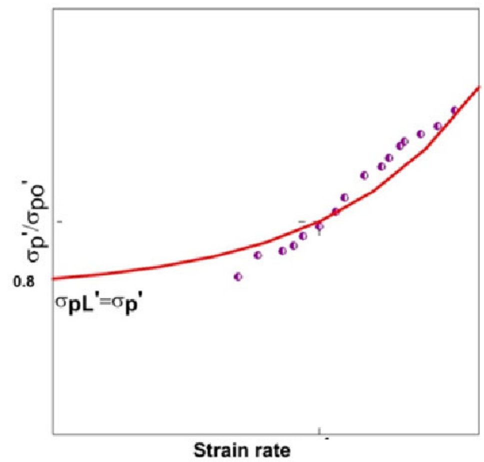

(c)

Figure 1. Strain rate-preconsolidation pressure relationship (a) reference time line; (b) series of isotaches for Ballina clay; and (c) strain rate – normalised preconsolidation pressure (Modified after Baral et al. 2019).

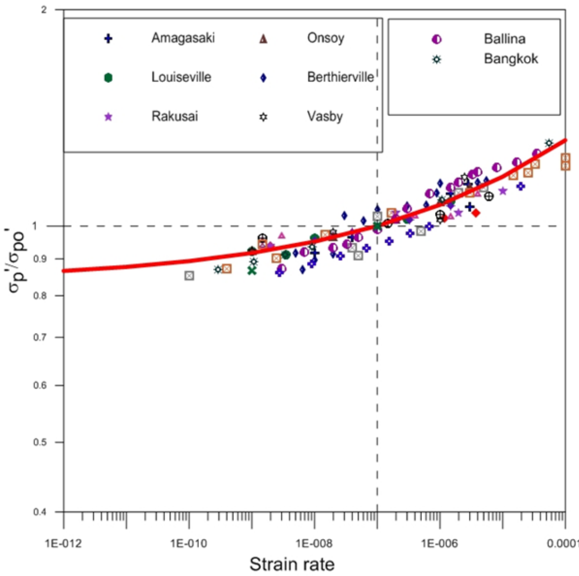

Figure 2. $(\sigma_p{}'/\sigma_{p0}{}')$ – log strain rate $(\dot{\varepsilon_v})$ curve for other clays (Modified after Baral et al. 2019).

The normalised relationship between the (σ_p'/σ_{p0}')- strain rate $(\dot{\varepsilon_v})$ for an array of clays has already been examined by Watabe et al. (2008, 2012) and Leroueil (1988). The relationship also captures other clays from Ballina, Bangkok (Rujikiatkamjorn et al., 2008), Malaysia (Indraratna and Redana, 2000) and Singapore (Cao et al., 2001) (Fig. 3). The ratio $(\sigma_{pL}'/\sigma_{p0}')$ for Ballina clay is 0.86 and (σ_p'/σ_{p0}') approaches unity at $\dot{\varepsilon} = 1 \times 10^{-07} s^{-1}$; for Ballina clay the value of c_1 and c_2 are 0.887 and 0.158, respectively (Fig. 2).

Figure 3 shows a soil with a pre-consolidation pressure (σ_p') of soil that is loaded from an initial effective stress of σ'_{v0} to the final effective stress σ'_{vf} and therefore the corresponding strain rate for the sample is $\dot{\varepsilon_1}$. For a given preloading $(\Delta\sigma_t)$, the change in effective stress can be calculated by:

$$\Delta\sigma_t = \Delta\sigma_c + \Delta\sigma_d \quad (7)$$

The first term $\Delta\sigma_c$ is an increase in effective stress due to the dissipation of excess pore-water whereas the second term $(\Delta\sigma_d)$ is an increase in effective stress due to delayed consolidation caused by the viscosity of clay. To estimate $\Delta\sigma_d$, the strain rate dependency of preconsolidation pressure can be employed using the following relationship:

$$\Delta\sigma_d = \sigma_{p0}' - \sigma_{p(\dot{\varepsilon})}' \quad (8)$$

where σ_{p0}' corresponds to the laboratory pre-consolidation pressure which corresponds to a strain rate of $1 \times 10^{-07} s^{-1}$, and $\sigma_{p(\dot{\varepsilon})}'$ is the preconsolidation pressure at a given strain rate $(\dot{\varepsilon})$. It should be noted that the strain rate can influence the location of $\sigma_{p(\dot{\varepsilon})}'$in isotaches. The strain rate can be estimated using the following formulation:

$$\dot{\varepsilon} = \frac{\varepsilon_{U100}}{\Delta t_{U100}} \quad (9)$$

where ε_{U100} *and* Δt_{U100} are the strain and time at the degree of consolidation based on pore pressure equal to 100% (*U100*) respectively; these parameters can be calculated based on the formulations proposed earlier by Indraratna et al. (2005), so once the strain rate $(\dot{\varepsilon})$ is known, $\sigma_{p(\dot{\varepsilon})}'$ can be calculated using Equation (6).

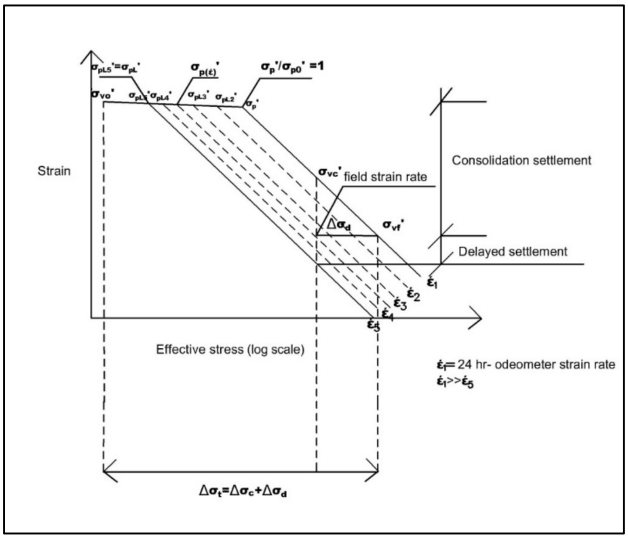

Figure 3. Conceptual model considering isotaches (Baral et al. 2019 with permission from ASCE)

3 APPLICATION TO CASE HISTORIES

The application of the proposed model is validated, for selected case studies at the Pacific Highway, Ballina, Australia, the Second Bangkok International Airport in Thailand, and Muar clay in Malaysia. The settlement and excess pore water pressure predicted at the centrelines of the embankments were used to compare with the field data.

3.1 *Pacific Highway, Ballina, Australia*

This site at bridge transition zone formed a trial embankment constructed on uniform layers of soft to incompressible estuarine and alluvial clays above residual soil and bedrock. The soft clay below the embankment was 25 m thick, and the clay's basic properties (unit weight, compressibility and the soil profile) are presented in Figure 4 (Indraratna et al., 2012). The groundwater table was at 0.2 m below the ground surface, and the soil layers composed of 10 m thick, soft clay with undrained shear strength of 5-10 kPa, above medium silty clay to 25 m deep with maximum undrained shear strength of 48 kPa.

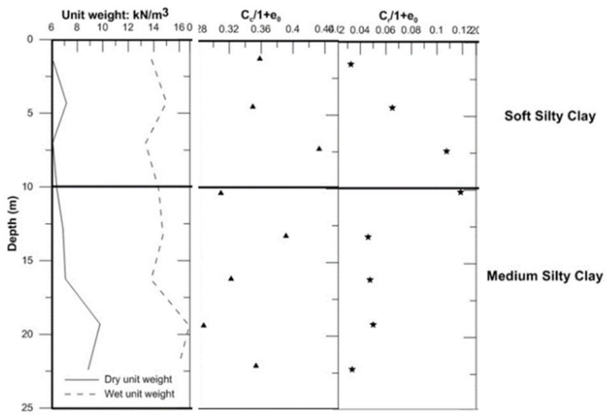

Figure 4. Basic soil properties of Ballina bypass

Kelly and Wong (2009) showed that a total surcharge of up to 11.2m, was chosen to limit long term settlement to be within 50 mm assuming fill density of 20 kN/m^3. To speed up consolidation, circular PVDs (34 mm diameter) were installed 1m apart in a square pattern. The first section was consolidated with fill surcharge only. The second section was consolidated with vacuum and surcharge preloading, which can significantly reduce the construction time. The surface settlement obtained from settlement plate (SP1) and excess pore pressure obtained from the piezometer (P1) at the depth of 3.3 m at the embankment centreline was used for comparison. The embankment was built to a height of 2 m within 5 months, and then to 4 m high after 8 months. Table 1 summaries the drain properties with the diameter of the smeared zone and the ratio of pre-consolidation pressure $(\sigma_{pL}'/\sigma_{p0}')$ with two other parameters (c_1 & c_2), as defined in Eq (4).

Table 2: Drain properties and other strain rate dependency parameters

Parameter	Value
d_w	51.5 mm
d_s	287 mm
d_e	1356 mm
S	5.57
N	26.33
k_h/k_s	3
M	6.620
$\frac{\sigma_{pL}'}{\sigma_{p0}'}$	0.86
c_1	0.887
c_2	0.158

The plots for time-settlement, and excess pore water pressure dissipation in the field with predicted results using the proposed model, Yin and Graham (1989), as well as the numerical analysis using converted permeability by Indraratna and Redana (2000) are presented in Fig 5. An average field strain rate of 8.54×10^{-11}

s^{-1} is obtained by Equation (9), and it is significantly lower than the laboratory one (1×10^{-07} s^{-1}). The settlement prediction obtained from the current model always yields higher settlements than two other curves as it accounts for non-linear creep. Nonetheless, the settlement predicted from all three models still provides an acceptable agreement with the field data. The measured excess pore water pressure before 3 months is unavailable (Fig. 5c), but the dissipation curve based on the current model is in closer agreement with the field data, in contrast to those determined by Yin and Graham (1989) and Indraratna and Redana (2000). The current model confirms the delay of excess pore water pressure dissipation, and the value of EPWP remains at 32 kPa even after 14 months, while it decreases to 20 and 12 kPa based on the models proposed by Yin and Graham (1989) and Indraratna and Redana (2000).

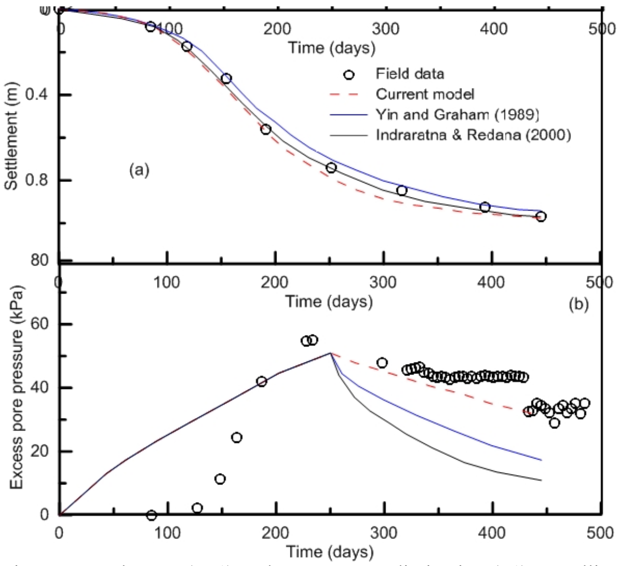

Figure 5. Settlement (SP1) and excess PWP dissipation (P1) at Ballina Bypass project (modified after Baral, 2019)

3.2 *Second Bangkok International Airport (SBIA), Thailand*

The Suwanabhuni Airport or the Second Bangkok international airport was built on marshy land with marine deposits. Ground level was almost 1m above the mean sea level. The site geology included approximately 1.5 m thick weathered clay followed by 12m of soft clay above 8 m of stiff clay followed by a uniform dense sand layer.

Synthetic vertical drains were anchored to a depth of 12m to increase the undrained shear strength and minimise long term settlement. PVDs (94 mm by 3 mm) with core channels (made up of Polyolefin) were pushed in using a 125 mm × 45 mm rectangular mandrel. The drain spacing was 1.2 m installed in a square pattern. The surcharge (4.2m high) was completed in four different stages within 9 months (see Figure 6a).

The field time settlement and excess pore pressure are plotted together with the predictions shown in Fig. 6. It is noted here that, as with previous case histories, the settlement predictions from three models are in general in a good agreement with the field measurement. The residual excess power water pressure to be dissipated at the end of 7 months are 4.40, 2.60 & 1.30 kPa for the current model, Yin and Graham (1989) and Indraratna and Redana (2000), respectively. The predicted excess pore water pressure from the proposed solutions agrees well with field observations, unlike those by Yin and Graham (1989) and Indraratna and Redana (2000).

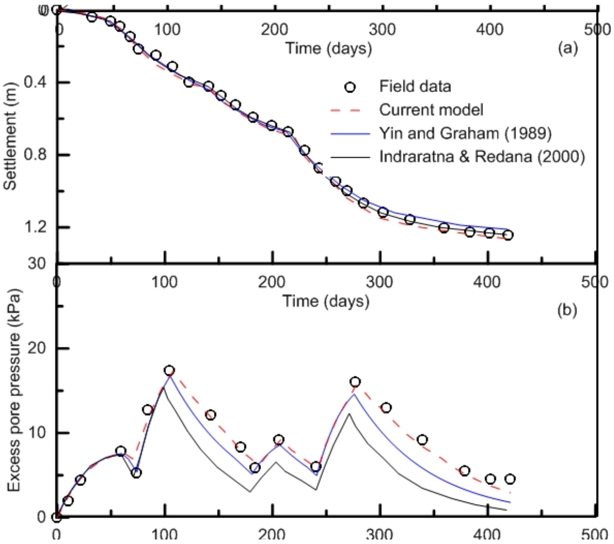

Figure 6. Comparison of surface settlement and excess PWP dissipation at 2 m deep in SBIA embankment (a) settlements; and (b) excess pore water pressure (Modified after Baral 2019)

3.3 *Muar clay, Malaysia*

The Malaysian Highway Authority (MHA) adopted various soil improvement techniques at Muar plain along the North South Expressway, to study their efficiency (Ratnayake, 1991). The methods included electro osmosis, chemical injection, micro piles, sand compaction piles, surcharge preloading, vacuum preloading, and sand drain installation and preloading. These were trialled on Muar clay at 14 different sections above marine and deltaic deposit with thick and compressible soft soil. In this section, the site with fill surcharge preloading was adopted for comparison.

The soil layers consisted of 2 m weathered crust above 15m very soft silty clay followed by 1m organic clay. It was found that medium dense to dense clayey silty sand located below the organic clay and extended up to 6 m from the organic clay layer. The weathered crust was slightly over-consolidated compared to beneath normally consolidated soft clay. The unit weight was fairly uniform (16 kN/m^3) except for the surface weathered clay which was 18kN/m^3. The undrained shear strength of the soil at 3m deep was about 8-10 kPa, and varied linearly with depth. Field and laboratory testing was also conducted to acquire the soil parameters required for the analysis.

An embankment (4.74m high) was built on the soft soil in two consecutive stages. In the first stage it was filled to a height of 2.57m, and then reached a height of 4.74 m. Vertical drains (equivalent drain radius=0.035 m) were then pushed in a triangular pattern at a spacing of 1.3m to 18 m deep. Instrumentation including settlement plates, piezometers and inclinometers was installed to monitor the settlement, pore water pressure and lateral displacement.

The strain rate during consolidation is determined as 6.78×10^{-11} s^{-1}. This has resulted in a ratio of field pre-consolidation pressure / laboratory pre-consolidation pressure of 0.79. These parameters were adopted to calculate the current model. Figure 7 depicts the predicted time settlement curve, and excess pore pressure dissipation pattern based on the current isotache model compared to the earlier models by Yin and Graham (1989) and Indraratna and Redana (2000). The settlement calculated by the current model agrees well with the field data although the disparity between the three models is minimal, except at time more than 300 days. The dissipation of excess pore water pressure is quite slow and the residual EPWP after 300 days is about 60kPa. When the proposed model adopted a pre-consolidation pressure ratio of 0.79, the predicted EPWP dissipation curve shows a much closer comparison with field data in relation to those by Yin and Graham (1989) and Indraratna and Redana (2000). The pore pressure dissipation rate is highest for Indraratna and Redana (2000) followed by Yin and Graham (1989). The current model predicts the gentlest dissipation rate with 55 kPa of undissipated EPWP after 300 days, compared to the measured value of 65 kPa.

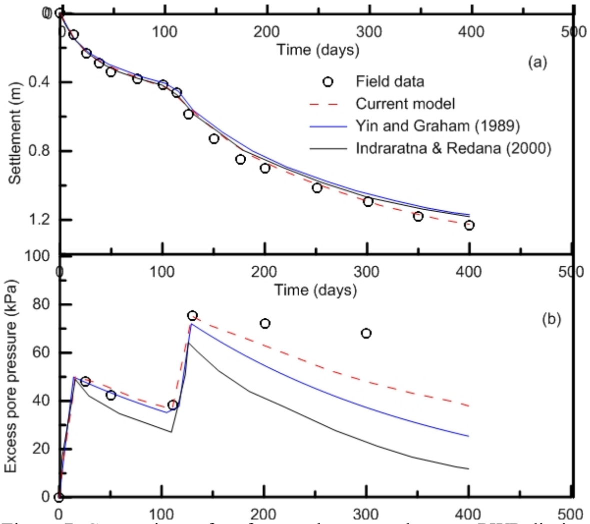

Figure 7. Comparison of surface settlement and excess PWP dissipation at 3 m deep in Muar embankment (a) settlements; and (b) excess pore water pressure (After Baral et al. 2019)

4 CONCLUSIONS

In this study, a strain rate dependency of pre-consolidation pressure incorporating a radial consolidation model incorporating was introduced to capture the in-situ strain rate. The constant rate of strain (CRS) plus long term consolidation (LTC) laboratory testing was required to obtain essential soil properties to establish the pre-consolidation pressure corresponding to the strain rate in the field. The increase in effective stress due to delayed consolidation can be determined with the associated excess pore water pressure dissipation rate. Moreover, the coefficients of secondary compression along with the strain rate can be employed to determine additional settlement.

The proposed method was benchmarked with three different case histories (Ballina bypass, Australia; Suvarnabhumi Airport, Bangkok and Muar clay, Malaysia). The predicted results of the proposed model and existing models show acceptable agreement with the field measurements. This demonstrates a further improvement in prediction using the current isotache model based on the conversion of laboratory pre-consolidation pressure into field pre-consolidation pressure.

The excess pore water pressures at the Ballina site began to dissipate after 3 months, even though settlement started earlier. While this could be caused by some instrumentation error, but with the Muar clay, the dissipation is very gentle, however, the proposed model improves the prediction in comparison to existing models.

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