DISCUSSION

Site characterisation for the Ballina field testing facility

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Contribution by G. Mesri and T. Kane

The comprehensive laboratory and in situ measurements reported by Kelly *et al.* (2017) and also by Pineda *et al.* (2016) on geotechnical characteristics of Ballina soft clay in New South Wales, Australia, deserve further interpretation. The stratigraphy is uniform between reduced levels of –2 m to –10 m; therefore, this interpretation is limited to Ballina soft clay at mid-depth of –6 m. At this depth, plasticity index, I_p , is near 70%; clay size fraction, CF, is near 70%; and in situ effective vertical stress is 42 kPa. Ballina soft clay was deposited about 8000 years before present (BP) in saline water in an estuarine environment.

Preconsolidation pressure

Before introduction of the constant rate of strain consolidation test (CRS) most data on preconsolidation pressure of soft clay deposits had originated from incremental loading (IL) consolidation tests, and corresponded to an end-of-primary (EOP) e –log σ'_v relation of 20 mm thick specimens. Therefore, an expression was developed for strain rate imposed on the CRS test that would lead to the EOP e –log σ'_v relationship (Mesri & Feng, 1992; Mesri *et al.*, 1994a)

$$\dot{\epsilon}_p = \frac{k_{v0}}{2C_c/C_k H^2} \frac{\sigma'_p}{\gamma_w} \frac{C_a}{C_c} \quad (2)$$

where k_{v0} is the initial permeability in vertical direction; C_c/C_k has typical values of $\frac{1}{2}$ to 2 (Mesri & Rokhsar, 1974); H is maximum drainage distance; σ'_p is preconsolidation pressure; γ_w is unit weight of water; and, for soft clays, $C_a/C_c = 0.04 \pm 0.01$ (Terzaghi *et al.*, 1996). An imposed vertical strain rate equal to $\dot{\epsilon}_p$ leads to excess pore water pressures near zero at the bottom (impermeable boundary) of a CRS specimen. For soft clay deposits, the most typical value of $\dot{\epsilon}_p$ is equal to $2.5 \times 10^{-7} \text{ s}^{-1}$ (Mesri *et al.*, 1994a).

An imposed vertical strain rate equal to $\dot{\epsilon}_p$ results in a rather long duration of the CRS test, and very small, near-zero excess pore water pressures do not allow the coefficient of permeability to be computed using the CRS porewater pressure measurements. Therefore, Mesri & Feng (1992) proposed for the CRS test an imposed vertical strain rate

$$\dot{\epsilon}_1 = 10\dot{\epsilon}_p \quad (3)$$

which produces u'_b/σ_v values in the range of 3–5% (where u'_b is the excess pore water pressure at the bottom of the specimen and σ_v is the imposed total vertical stress), and allows the Darcy flow equation to be used for reliable calculation of the coefficient of permeability (Mesri *et al.*, 1994a, 1994b). However, the resulting preconsolidation pressure must be corrected to obtain EOP σ'_p (Mesri & Choi, 1979, 1984; Mesri, 1987; Terzaghi *et al.*, 1996).

$$[\sigma'_p]_{\dot{\epsilon}_p} = [\sigma'_p]_{\dot{\epsilon}_1} \left(\frac{\dot{\epsilon}_p}{\dot{\epsilon}_1} \right)^{C_a/C_c} \quad (4)$$

For Ballina clay, the value of σ'_p/σ'_{v0} is near 1.77 from the CRS oedometer data in Fig. 4(b). However, the imposed strain rate was $3.3 \times 10^{-6} \text{ s}^{-1}$ for the CRS tests on Ballina soft clay, compared to $\dot{\epsilon}_p = 3.4 \times 10^{-7} \text{ s}^{-1}$ from equation (2) for $k_{v0} = 10^{-9} \text{ m/s}$, $C_k/C_c = 1$, $C_a/C_c = 0.04$ and $\sigma'_p = 68 \text{ kPa}$. Therefore, using equation (4) together with $C_a/C_c = 0.04$, leads to $\sigma'_p/\sigma'_{v0} = 1.61$, and for $\sigma'_{v0} = 42 \text{ kPa}$, to $\sigma'_p = 68 \text{ kPa}$.

For cone penetration tests, according to Mesri (2001)

$$\sigma'_p = 0.28(q_t - \sigma_v) \quad (5)$$

From the cone penetration tests on Ballina clay, at –6 m, $q_t = 327 \text{ kPa}$ and $\sigma_v = 100 \text{ kPa}$ lead to $\sigma'_p = 64 \text{ kPa}$.

For a geologically normally consolidated clay, the value of σ'_p/σ'_{v0} resulting from secondary compression and thixotropic ageing alone is (Mesri & Choi, 1979, 1985; Mesri, 1993)

$$\frac{\sigma'_p}{\sigma'_{v0}} = \left(\frac{t}{t_p} \right)^{[(C_a/C_c)/1 - (C_r/C_c)] + \beta} \quad (6)$$

Using $t = 8000$ years, $t_p = 10$ years, $C_a/C_c = 0.04$, $C_r/C_c = 0.1$, and thixotropic hardening parameter $\beta = 0.02$, it is possible to calculate $\sigma'_p/\sigma'_{v0} = 1.53$. The difference between 1.61 and 1.53 has probably resulted from groundwater fluctuations.

Undrained shear strength

For Ballina clay, the undrained shear strength was estimated from field vane tests (FV), triaxial compression tests (TC) and triaxial extension tests (TE). At –6 m depth, the value of s_{u0} (FV) is 20.0 kPa from Fig. 15. Thus s_{u0} (FV)/ $\sigma'_p = 0.294$. However, from the Bjerrum–Mesri s_{u0} (FV)/ σ'_p against I_p relationship, for $I_p = 70\%$, s_{u0} (FV)/ $\sigma'_p = 0.32$ (Terzaghi *et al.*, 1996, fig. 20.20). For Ballina clay at –6 m depth s_{u0} (TC)/ σ'_p is 0.300, and s_{u0} (TE)/ σ'_p is 0.263.

The undrained shear strength mobilised in failure of embankments, footings and excavations is

- (a) Bjerrum (1972, 1973) with Bjerrum correction $\mu_B = 0.70$ for $I_p = 70\%$ (Terzaghi *et al.*, 1996, fig. 20.21).

$$\begin{aligned} \frac{s_{u0}(\text{mob})}{\sigma'_p} &= \mu_B \frac{s_{u0}(\text{FV})}{\sigma'_p} \\ &= 0.21 \\ s_{u0}(\text{mob}) &= 0.21 \times 68 = 14.3 \text{ kPa} \end{aligned} \quad (7)$$

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- (b) Mesri (1975), for inorganic soft clays

$$\frac{s_{u0}(\text{mob})}{\sigma'_p} = 0.22 \quad (8)$$

$$s_{u0}(\text{mob}) = 0.22 \times 68 = 15.0 \text{ kPa}$$

- (c) Mesri (2001), mobilised undrained shear strength from cone penetration test

$$s_{u0}(\text{mob}) = \frac{q_t - \sigma_v}{N_k(\text{mob})} \quad (9)$$

where q_t is the corrected cone tip resistance, σ_v is the total vertical stress and $N_k(\text{mob}) = 16$.

$$s_{u0}(\text{mob}) = \frac{327 - 100}{16} = 14.2 \text{ kPa}$$

- (d) Mesri (1989) and Mesri & Huvaj (2007) mobilised undrained shear strength from laboratory tests. In the absence of $s_{u0}(\text{DSS})/\sigma'_p$

$$\frac{s_{u0}(\text{mob})}{\sigma'_p} = \frac{1}{2} \left[\frac{s_{u0}(\text{TC})}{\sigma'_p} + \frac{s_{u0}(\text{TE})}{\sigma'_p} \right] \mu_t \quad (10)$$

where μ_t is a time to failure correction factor as a function of plasticity index, I_p , equal to 0.83 for $I_p = 70\%$ (Terzaghi *et al.*, 1996, fig. 20.24).

Note that shear strength from laboratory tests from Jamiolkowski *et al.* (1985) and Ladd (1991) have been incorrectly reproduced in Fig. 21.

$$\frac{s_{u0}(\text{mob})}{\sigma'_p} = \frac{1}{2} (0.300 + 0.263) 0.83$$

$$= 0.23$$

$$s_{u0}(\text{mob}) = 0.23 \times 68 = 15.6 \text{ kPa}$$

Therefore, for Ballina soft clay at -6 m depth, the most typical value of $s_{u0}(\text{mob})/\sigma'_p = 0.22$, and $s_{u0}(\text{mob}) = 15 \text{ kPa}$.

Coefficient of earth pressure at rest

According to Fig. 11(d), $K_0 = 0.560$, and according to Fig. 13, $\phi'_{cv} = 34^\circ$, where ϕ'_{cv} is the constant volume friction angle (Terzaghi *et al.*, 1996).

The Schmidt (1967) equation for $K_0 = \sigma'_{h0}/\sigma'_{v0}$ is (Mesri & Hayat, 1993)

$$K_0 = K_{0p} \left(\frac{\sigma'_p}{\sigma'_{v0}} \right)^{\sin \phi'_{cv}} \quad (11)$$

where K_{0p} is the coefficient of lateral pressure in the compression range beyond σ'_p , and according to Jaky (1948)

$$K_{0p} = 1 - \sin \phi'_{cv} \quad (12)$$

thus $K_{0p} = 0.4408$ and $K_0 = 0.575$.

The Mesri & Hayat (1993) equation for K_0 of soft clay and silt deposits is

$$K_0 = \frac{1}{2} K_{0p} \left(\frac{\sigma'_p}{\sigma'_{v0}} + 1 \right) \quad (13)$$

using $K_{0p} = 0.4408$ and $\sigma'_p/\sigma'_{v0} = 1.61$, $K_0 = 0.575$.

Thus the predictions of $K_0 = \sigma'_{h0}/\sigma'_{v0}$ by Schmidt (1967) and Mesri & Hayat (1993) empirical equations are in agreement with values reported in Fig. 11(d).

Compressibility

The compressibility of soft clay deposits is completely defined by the EOP e - $\log \sigma'_v$ relationship of each sublayer,

and C_d/C_c . For strongly non-linear EOP e - $\log \sigma'_v$ relations in the compression range beyond σ'_p , such as those for Ballina clay in figs 6 and 7 of Pineda *et al.* (2016), the values of tangent compression index, C_c , such as those in Fig. 9 of the paper under discussion cannot be readily incorporated into a simple settlement equation. The strongly non-linear compression curve is best described in terms of values of secant compression index, C'_c , as a function of $\log \sigma'_v/\sigma'_p$ (Mesri & Choi, 1985, Terzaghi *et al.*, 1996: figs 16.16 and 16.7, equation (16.9); Mesri & Funk, 2015: figs 7 and 8).

The values of C_d/C_c for Ballina soft clay are within the range of 0.04 ± 0.01 for soft clay and silt deposits (Terzaghi *et al.*, 1996, table 16.1). Because, in the recompression range C_d increases with time, frequently, an incorrect value of C_c is paired with C_d , thus resulting in high values of C_d/C_c as in fig. 12 of Pineda *et al.* (2016).

Coefficients of permeability and consolidation

According to Fig. 18(b) a typical value of k_v is 10^{-9} m/s for Ballina soft clay, which is also the most common k_{v0} for soft clay deposits. An empirical equation for k_{v0} (Mesri *et al.*, 1994b), in terms of e_0/CF and activity, $A_c = I_p/\text{CF}$, for $e_0 = 3.0$, $\text{CF} = 70\%$ and $A_c = 1.0$, predicts $k_{v0} = 1.4 \times 10^{-9} \text{ m/s}$. However, for Ballina soft clay, the value of C_k is near 1.0, based on data in fig. 11(b) of Pineda *et al.* (2016), whereas for most soft clays C_k is close to $e_0/2$, which for Ballina soft clay is 1.5 (Tavenas *et al.*, 1983; Mesri *et al.*, 1994b). According to Fig. 4(d) a typical value of c_v for Ballina soft clay is $3 \text{ m}^2/\text{year}$, which is a common value for many soft clays; however, the lower range of 0.3 – $0.5 \text{ m}^2/\text{year}$ is more consistent with the Ballina soft clay liquid limit of 110%.

In summary, the observed behaviour of Ballina soft clay, in general, is remarkably consistent with those of most soft clay deposits. Therefore, an excellent selection has been made for a field testing facility in Australia.

Authors' reply

The authors would like to thank Professor Mesri and Mr Kane (the discussion contributors) for their valuable contribution to the analysis of the behaviour of Ballina clay. In this reply the authors wish to provide some additional information and clarifications on the points raised by the discussion contributors.

To facilitate the discussion, the authors also refer to the interpretation of the information presented in Kelly *et al.* (2017) and Pineda *et al.* (2016) by academics and practitioners, to predict the behaviour of a trial embankment built on Ballina clay improved by prefabricated vertical drains. These predictions were presented in a symposium organised by the ARC Centre of Excellence for Geotechnical Science and Engineering in Newcastle (Australia) in September 2016 (CGSE, 2016). One lesson learned from this symposium was that the interpretation of the same set of laboratory and in situ data by different predictors may result in large variations in adopted soil properties, even among experienced predictors who used the same analysis approach. Hence the authors believe that it is worth elaborating more on the derivation of some key Ballina clay parameters, in the light of the discussion contributors' analysis and limited additional data. Detailed outcomes of the above-mentioned symposium will be reported in a special issue to be published in *Computers and Geotechnics* later in 2017. Note that, in the following, the same notation is used as employed by the discussion contributors, for the purpose of consistency.

Preconsolidation pressure

The discussion contributors have estimated values of preconsolidation pressure using three different methods proposed by Professor Mesri and his co-workers. Values reported by Pineda *et al.* (2016) (Table 2) were obtained at strain rates larger than the value imposed during IL tests which, for soft clays, typically ranges between $1.0 \times 10^{-7} \text{ s}^{-1}$ (Watabe *et al.*, 2012) and $2.5 \times 10^{-7} \text{ s}^{-1}$ (Mesri *et al.*, 1994a; Terzaghi *et al.*, 1996). Although a correction factor was suggested based on the work by Watabe *et al.* (2012), uncorrected values were intentionally reported in Pineda *et al.* (2016), allowing the predictors to make their own estimates.

Most of the predictors did not account for rate effects and that was, in the authors' opinion, a major contributor to under-prediction of embankment settlement. The authors attribute this to some of the predictors not being familiar with the interpretation of the CRS test. For the depth analysed by the discussion contributors ($5.5 < z < 6 \text{ m}$), the corrected preconsolidation pressure leads to $\sigma'_p/\sigma'_{v,0} = 1.49$ (similar to the lower bound estimated by the discussion contributors). Fig. 23(a) presents recent measurements from a creep-IL test carried out on a Sherbrooke (block) specimen obtained from borehole BH1 (depth 5.24–5.61 m), drilled 100 m north from borehole inclo 2. The results shown in Fig. 23 were obtained from long-term loading steps (14 days), and the maximum vertical effective stress reached was 500 kPa. Small loading increments were applied in order to obtain reliable estimates of the preconsolidation pressure. Fig. 23(b) shows the $\varepsilon_v - \log \sigma'_v$ curves for constant rates of strain between 10^{-8} and 10^{-6} s^{-1} , back-analysed from the creep test following the procedure described by Leroueil *et al.* (1985). The preconsolidation pressure ranges from 57 kPa to 68 kPa for strain rates of 10^{-8} s^{-1} and 10^{-6} s^{-1} , respectively, and a value of $\sigma'_p \approx 62 \text{ kPa}$ is obtained for a strain rate equal to 10^{-7} s^{-1} . This value is in agreement with the lower bound estimate proposed by the discussion contributors, as well as with the values reported in Pineda *et al.* (2016), once corrected by rate effects.

Undrained shear strength

The discussion contributors are right in the fact that laboratory tests from Jamiolkowski *et al.* (1985) and Ladd (1991) have been incorrectly replicated in Fig. 21. Owing to an error when digitising the corresponding figures, values of $s_u/\sigma'_{\text{yield}}$ as well as the trend lines were unintentionally shifted down compared with the original data. The corrected Fig. 24 presents the original test results together with triaxial data obtained from Ballina clay. Note that the general trends remain unchanged, but the triaxial extension tests from Ballina clay lie below the trend line drawn by Ladd (1991).

Coefficient of earth pressure at rest

Various approaches have been used to estimate the coefficient of earth pressure at rest from seismic dilatometer tests (Marchetti, 1980; Powell & Ugrow, 1988; Lacasse & Lunne, 1988; Kouretzis *et al.*, 2015). A range of K_0 profiles were obtained from each interpretation method, including independent estimations from a self-boring pressuremeter test (Gaone *et al.*, 2016). Interpretation of push-in pressure cell (PiPCs) measurements showed good agreement with the lower bound profiles obtained using the expressions proposed by Lacasse & Lunne (1988) and Kouretzis *et al.* (2015). Independent checks were made using similar expressions to those presented by the discussion contributors. From the analysis of the triaxial compression tests, a constant volume friction angle equal to 36° was adopted. This value is similar

Fig. 23. One-dimensional creep test on Sherbrooke specimen BH1-1B

Fig. 24. Variation of normalised s_u with PI (corrected). TC, triaxial compression; TE, triaxial extension; SS, simple shear

to the constant volume friction angle reported by Mayne (2016) for a wide range of natural soft soils. A comprehensive laboratory study using high-quality Sherbrooke (block) specimens is currently underway at the University of Newcastle in order to assess K_0 for Ballina clay specimens corresponding to various depths.

Compressibility

The authors agree with the discussion contributors that the strongly non-linear compressibility curves should be incorporated into settlement assessments – for example, using secant values of C_c over the appropriate stress range as proposed by Mesri & Choi (1985). For the prediction of the trial embankment behaviour, most symposium contributors adopted conventional bi-linear void ratio–log stress curves in

their analyses and used 'average' C_c values based on their own judgement. Although a wide range of values was finally employed by the various predictors, there was a tendency to use values measured at stress levels much higher than the one imposed by the embankment (excluding analyses where advanced constitutive models incorporating destructuration effects were employed). In the authors' opinion, this also contributed to under-prediction of embankment settlement. The authors agree with the discussion contributors that explicitly accounting for destructuration is preferable to the use of average values and judgement, particularly when high-quality laboratory tests are available.

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