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Extending the life of existing infrastructure

Prolonger la durée de vie des infrastructures existantes

L. Zdravković
Imperial College London, London, United Kingdom

D. M. Potts
Imperial College London, London, United Kingdom

T. M. Bodas Freitas
CERIS, Instituto Superior Técnico, Universidade de Lisboa, Lisboa, Portugal

ABSTRACT: Contemporary geotechnical design is increasingly faced with a demand for extending the life of ageing infrastructure. Typical examples are foundation systems for re-development projects in congested urban environments, road and rail infrastructure and flood defences. The latter structures in particular are also projected to have to sustain rising sea-levels due to the effects of climate change. To enable such a design, the changes in the mechanical behaviour of the foundation soil over the period from first construction to the current state must be quantified. This transient period is governed by the time-related process of consolidation and creep in the ground. Considering examples of earthfill embankments, this paper discusses the facets of soil behaviour governing both the short- and long-term design of existing infrastructure embankments, using advanced numerical analysis.

RÉSUMÉ: La domaine de la géotechnique contemporaine est de plus en plus confrontée à la nécessité de prolonger la durée de vie d'infrastructures vieillissantes. Des exemples typiques sont les systèmes de fondations pour les projets de re-développement dans les environnements urbains encombrés, les infrastructures routières et ferroviaires et les défenses contre les inondations. Ces dernières structures, en particulier, devraient également faire face à l'élévation du niveau de la mer en raison des effets du changement climatique. Pour pallier de telles contraintes, les changements dans le comportement mécanique du sol de fondation au cours de la période allant de la première construction à l'état actuel doivent être quantifiés. Cette période transitoire est régie par le processus temporel de consolidation et de fluage du sol. À l'aide d'exemples de remblais en terre, ce document de recherche traite des aspects du comportement du sol régissant la conception à court et à long terme des remblais d'infrastructures existants, à l'aide d'une analyse numérique avancée.

Keywords: earth embankment; anisotropy; creep; numerical analysis;

1 INTRODUCTION

Earthfill embankments are common geotechnical structures for supporting networks of road and rail infrastructure and serving as flood defences along river banks and coastlines. They are often erected on soft clay deposits whose undrained strength limits the construction height of a single-stage embankment to 3 to 4 m. Bjerrum (1973) postulated that the strength anisotropy of soft clay deposits may be the key parameter for the short-term embankment design, as its structure imposes large rotations of principal stresses in the foundation soil, promoting the mobilisation of variable soil strength along the potential failure surface.

The effect of strength anisotropy can only be reasonably quantified through the application of advanced numerical analysis which employs appropriate constitutive models (e.g. Karstunen et al., 2005; Zdravković et al., 2002; Kavvadas & Amorosi, 2000; Whittle & Kavvadas, 1994).

During the transient post-construction period the foundation soil is subjected to time-related processes of consolidation and creep, the former dissipating the construction-generated pore water pressures in the ground and the latter accounting for changes in the soil fabric. These processes can induce large settlements, thus requiring the raising of the embankment height so that it can continue to fulfil its purpose. The reduction of void space in the foundation soil leads to an increase of its undrained strength. Again, advanced numerical analysis and appropriate soil constitutive models are needed to quantify the likely gain in undrained strength and the likely height to which an embankment can be raised without failing (e.g. Yin & Graham, 1999; Bodas Freitas et al., 2011, 2015; Karstunen & Yin, 2010).

Using numerical analysis and a number of case studies, this paper demonstrates the effect that the soil strength anisotropy has on the short-term design of soft-ground embankments, and the effect of creep on its long-term behaviour and ability for subsequent life extension. All analyses were performed using the Imperial College Finite Element Program (ICFEP; Potts & Zdravković, 1999), employing a hydro-mechanically coupled formulation, with appropriate time steps to simulate realistic construction sequences.

2 SHORT-TERM DESIGN

Faced with the need to build road embankments on soft Champlain deposits in east Canada, researchers at Laval University developed in the 1970s a large research programme that involved construction and monitoring of a number of trial embankments on Champlain clays at the Saint-Alban site in Quebec. The test embankment ‘A’ (Fig. 1) was brought to failure and was the subject of the numerical study by Zdravković et al. (2002) which quantified the effect of strength anisotropy on the short-term design of this embankment.

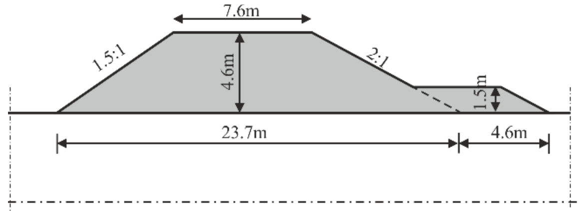

Figure 1. Cross-section of Saint-Alban embankment

2.1 Ground conditions

The soil investigation programme comprised a number of field and laboratory experiments on Champlain clays (e.g. Lefebvre et al., 1988; Leroueil et al., 1983), revealing highly anisotropic undrained strength, S_u, profiles. A typical desiccated crust exists up to 2 m depth, below which the clay is lightly overconsolidated, with $OCR = 2.2$ and $K_0^{OC} = 0.65$. The ground water level was found at 0.7 m depth and the permeability of the clay is in the range of 10^{-10} to 10^{-9} m/s, which indicates that undrained conditions are likely to exist in the foundation soil during embankment construction. The angle of shearing resistance was found to be $\phi'_{TXC} = 27^o$ and $\phi'_{TXE} = 25^o$, in triaxial compression and extension, respectively.

The embankment fill was a uniform medium to coarse sand, containing about 10% gravel, with an angle of shearing resistance $\phi'_{TXC} = 44^o$.

2.2 Numerical modelling

A limit equilibrium analysis by the research team at Laval University predicted that the embankment A, with its cross-section depicted in Fig. 1, would fail at 4.6 m height, on the side of the steeper slope (1.5:1 horizontal to vertical). In the actual event the failure height was 3.9 m.

The main aspect of the finite element analyses conducted by Zdravković et al. (2002) was the choice of a constitutive model to represent the foundation soil. The measured profiles of S_u in Fig. 2 are obtained from undrained triaxial compression (TXC) tests on isotropically-consolidated clay samples (CIU) and from a range of vane tests that were shown to give S_u values similar to direct simple shear (DSS) strengths (Lefebvre et al., 1988). The inclination of the major principal stress, σ_1', in the former tests is vertical (i.e. $\alpha = 0^o$), while in the latter tests the σ_1' is thought to be at around $\alpha = 45^o$ to the vertical. Consequently, there is a significant strength anisotropy in the soil from the profiles in Fig. 2.

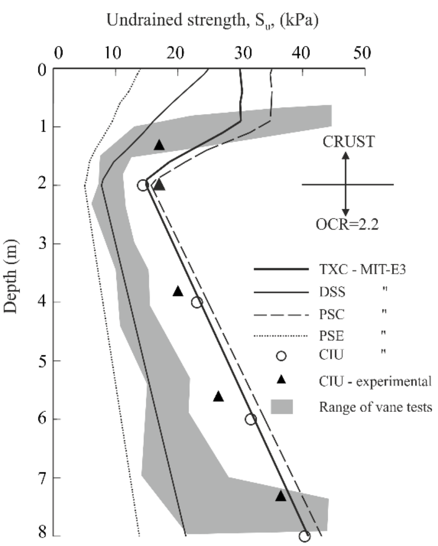

Figure 2. S_u profiles for the Champlain clay deposit

A critical state-based Modified Cam Clay (MCC) constitutive model (Roscoe & Burland, 1968) may be an obvious choice for modelling the behaviour of soft clays on the wet side of the critical state. However, this model can simulate only an isotropic undrained strength and in this study it was calibrated to reproduce either a TXC or a DSS S_u profile from Fig. 2. The former is most likely to be available from any site investigation, whereas the latter strength was recommended by Bjerrum (1973) as an appropriate average strength for the design of soft clay embankments. Conversely, an anisotropic MIT-E3 model (Whittle & Kavvadas, 1994), developed in the framework of bounding surface plasticity, was also employed in this investigation and calibrated to simulate the observed anisotropic nature of the soil's undrained strength. The calibrated MIT-E3 S_u profiles in Fig. 2 correspond to TXC (agreeing well with the measured CIU profile) and DSS (ploting in the range of vane tests) strengths. Also shown are the plane strain compression (PSC) and extension (PSE) profiles as limiting values (at $\alpha = 0^o$ and 90^o respectively) of S_u that bound the strengths that the foundation soil can mobilise during embankment construction. The calibration process and parameters derived for both models are given in Zdravkovic et al. (2002).

2.3 Implications for short-term design

Fig. 3 shows the variation of the normalised horizontal movement, u, at the toe, with increase in embankment height, indicating also the predicted failure heights. Recalling that the trial embankment failed when reaching 3.9 m height, none of the analyses that modelled the foundation soil as isotropic (MCC) were able to predict the correct failure. Adopting the TXC profile of S_u is clearly unsafe, predicting that the embankment can be 1 m higher. Conversely, the DSS-based design is overly conservative, predicting a 0.6 m lower failure height from that observed in the field.

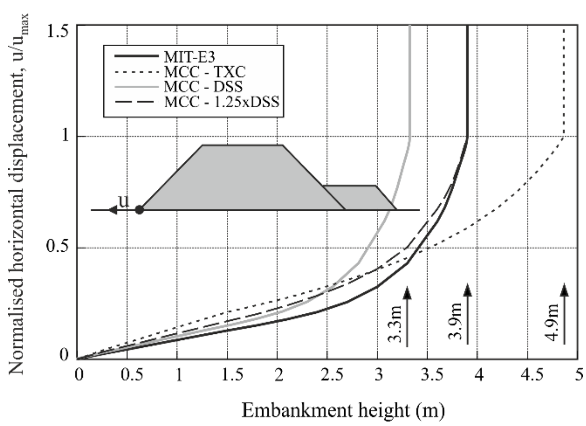

Figure 3. Predicted failure of St Alban embankment

The MIT-E3 model, calibrated only from the soil data (i.e. not back-calculating any of the parameters using the observed failure) predicts accurately the failure height. An accurate prediction of the failure height with the MCC model was obtained for an S_u profile that is 25% larger than the DSS strength profile (Fig. 3). Zdravković et al. (2002) further demonstrated that using such a back-calculated isotropic strength to predict the failure height of a different shape embankment built on the same foundation soil (e.g. adding another berm as in Fig. 4), did not produce the same result as the analysis of the same embankment using the originally calibrated MIT-E3 model. The isotropic prediction was again unsafe, giving 4.9 m failure height, compared to 4.4 m predicted when simulating anisotropic strength (Fig. 4).

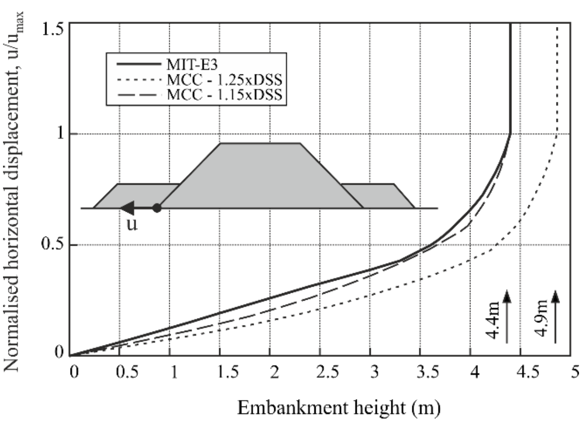

Figure 4. Predictions of modified embankment

3 LONG-TERM BEHAVIOUR

Having established in the previous study that the anisotropy of the undrained strength in soft clay foundation soils has a governing effect on the short-term design of earth embankments, the next step was to apply the established modelling procedure to the assessment of the long-term behaviour of embankments. A case study of a trial embankment constructed at the Mucking Flats test site in the Thames Estuary in the UK is used for this purpose, as it was subjected to a period of post-construction monitoring.

3.1 Ground conditions

Extensive laboratory and field characterisation of the site and the behaviour of the Mucking clay were reported in Pugh (1978) and Wesley (1975). Profiles of measured S_u in Fig. 5 indicate a desiccated crust, similar to the soft Champlain clays, to about 1.5 m depth. The clay beneath is lightly overconsolidated with an $OCR = 1.35$ and a $K_0^{OC} = 0.55$, while the permeability was characterised as anisotropic, with $k_h = 1.3 \cdot 10^{-9}$ m/s and $k_v = 2.6 \cdot 10^{-9}$ m/s.

The embankment material was a clean sand with $\phi'_{TXC} = 38^{\text{o}}$.

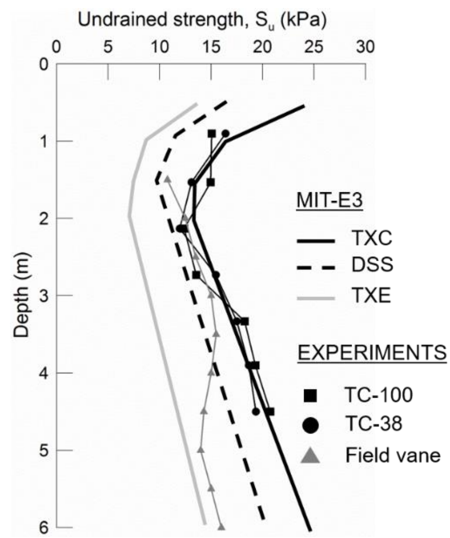

Figure 5. S_u profiles for the Mucking clay deposit

3.2 Numerical modelling

The same MIT-E3 constitutive model used in the Saint-Alban study was selected for the simulation of the Mucking clay behaviour. The calibration process and derived model parameters are given in Sheehy (2005). Fig. 5 shows that the strength anisotropy of the Mucking clay is less well defined experimentally compared to Champlain clays. However, the TXC S_u profiles are reasonably well reproduced by the model.

The embankment, termed Bank 2, was selected for this analysis, as it was erected to 2.8 m height and then monitored for a limited period post-construction. A set of settlement gauges was placed at a 1 m deep horizontal level underneath the embankment body. Their measurements are compared with numerical predictions in Fig. 6.

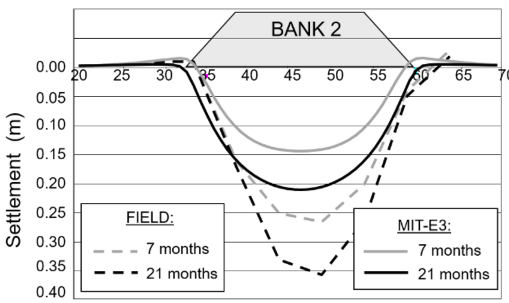

Figure 6. Post-construction settlements of Bank 2; predictions accounting for consolidation

Fig. 6 shows substantial embankment settlements measured at 7 and 21 months post-construction. However, the settlements predicted in the analysis employing the MIT-E3 model, which in the transient stage accounts only for consolidation (i.e. dissipation of excess pore water pressures), are significantly smaller, by some 40%, than measured. Adjusting the model parameters within the scatter of experimental data and repeating the analysis did not alter notably these predictions. It was evident that, for the transient/long term modelling of the embankment, additional facets of time-dependent soil behaviour, such as creep or viscosity, need to be considered.

A number of constitutive models have been proposed to simulate the time-dependent behaviour of soils (mainly clays) that follow isotach viscosity (Liigaard et al., 2004). Most of these are elastic-viscoplastic models, based on Perzyna (1963) overstress theory. The model selected for the current study adopts the same concept, in combination with the Yin (1999) equivalent time (ET) framework. Details of the model, termed IC-ET, its generalisation in stress space and its implementation in ICFEP are given in Bodas Freitas et al. (2011, 2012).

Calibration of the IC-ET model for the behaviour of Mucking clay is given in Losacco (2007). The model is isotropic and was therefore calibrated to reproduce the TXC S_u profile in Fig. 5, which is similar to that reproduced by the MIT-E3 model. The main difference is that in the time-dependent framework the S_u profile has to be consistent with the strain rate at which the shearing in the experiments was conducted, which was 0.33 %/hour (Wesley, 1975).

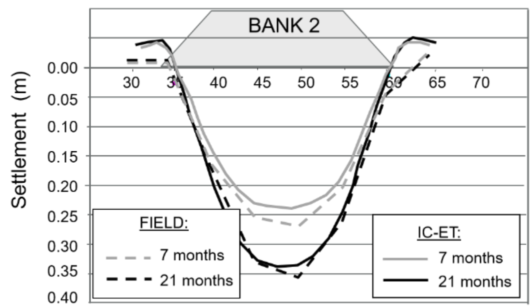

Figure 7. Post-construction settlements of Bank 2; predictions accounting for creep and consolidation

The finite element analysis of the Mucking Flat embankment Bank 2 was repeated, with the IC-ET model now simulating the behaviour of the foundation soil and accounting for both consolidation and creep. The predictions of its settlement at the same time instances of 7 and 21 months are shown in Fig. 7. The agreement with field measurements is excellent.

3.3 Life extension

One option for extending the life of ageing infrastructure embankments, in particular those that serve as flood defences, is to raise the embankment height. The issue is then to determine how high can the embankment be raised before it fails, which depends on the current S_u distribution that is altered by the time-dependent processes in the soil discussed above. This distribution is difficult to quantify empirically, hence standard methods of analysis cannot be used.

Having shown in Fig. 7 that, taking account of consolidation and creep reproduces accurately the time-dependent response of soft-clay foundations (albeit over a limited time-span, as available in the case study), the finite element analysis of Bank 2 was extended to allow a period of 30 years to pass from the end of its construction. Both consolidation and creep were allowed to take place during this time, although pore water pressures stabilised after about 5 years. Bank 2 was then subjected to two hypothetical, but realistic, raising schemes shown in Fig. 8.

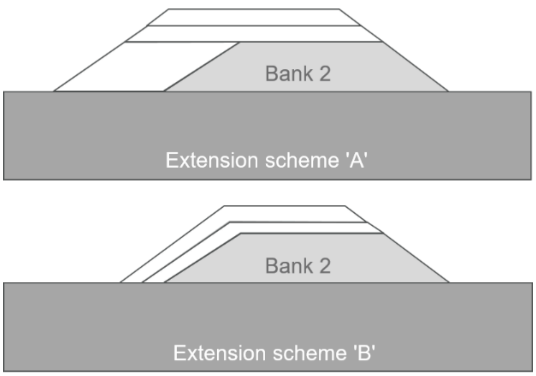

Figure 8. Raising schemes applied to Bank 2

The predicted raised height of Bank 2 just before failure was 5.3 m and 5.6 m for schemes ‘A’ and ‘B’, respectively, from the analyses that employed the IC-ET constitutive model for the foundation soil. This is nearly 100% increase of the height of the original embankment). The same extension exercises were applied in the analysis that employed the MIT-E3 model, although the transient soil behaviour was not well reproduced by this model (Fig. 6). The predicted raised heights, just before failure, were 3.9 m and 4.7 m for schemes ‘A’ and ‘B’, respectively.

4 VALIDATION

The soundness of the finite element predictions discussed above was demonstrated by comparisons of analyses results with available field measurements from the two case studies on infrastructure embankments. The analysis employing the MIT-E3 model was shown to reproduce accurately the soft-clay embankment construction to failure, emphasising the importance of strength anisotropy in the short-term design. The analysis that employed the IC-ET model demonstrated the accuracy in predicting the transient, post-construction behaviour of soft clay foundations. However, the raising of the embankment following the latter analysis was not possible to verify as this was not done in the field and hence the data is not available.

Consequently, to gain confidence in the ability of the IC-ET model to predict accurately the changes in the mechanical behaviour of the foundation soil that would enable life extension of infrastructure, a case study of two footings constructed on the soft Bothkennar clay (Jardine et al., 1995; Lehane & Jardine, 2003) is considered in this section.

4.1 Geometry and ground conditions

The footing experiment comprised two identical square footings (2.2 x 2.2 m) constructed at the Bothkennar site in the UK and instrumented to measure displacements, stresses and pore water pressures in the ground. The footing ‘A’ was loaded to failure (undrained) immediately after construction in 1990 (test A), as shown by the load-displacement curve in Fig. 9. The ultimate foundation pressure was estimated at 138 kPa. Footing ‘B’ was also loaded in 1990 (test B), but only to 89 kPa (equivalent to a factor of safety of about 1.5 applied to the bearing capacity) and was then left under this load for 11 years. In 2001 footing ‘B’ was further loaded to failure (test C), mobilising 204 kPa of ultimate pressure. The gain in capacity of footing ‘B’ after 11 years of consolidation and creep in the soil was 48% compared to the capacity of the virgin soil. The solid lines in Fig. 9 represent experimental data.

The behaviour of the Bothkennar clay was extensively examined through laboratory and field experiments in the 1990s and reported in the special issue of *Géotechnique* in 1992. The profile of undrained strength from TXC tests in Fig. 10 shows consistent data from samples obtained with different sampling techniques. The angle of shearing resistance is $\phi'_{TXC} = 37^{o}$ and $\phi'_{TXE} =$

42° and the permeability is about $5 \cdot 10^{-9}$ m/s. The clay is lightly overconsolidated with $OCR = 1.5$ and $K_0 = 0.65$.

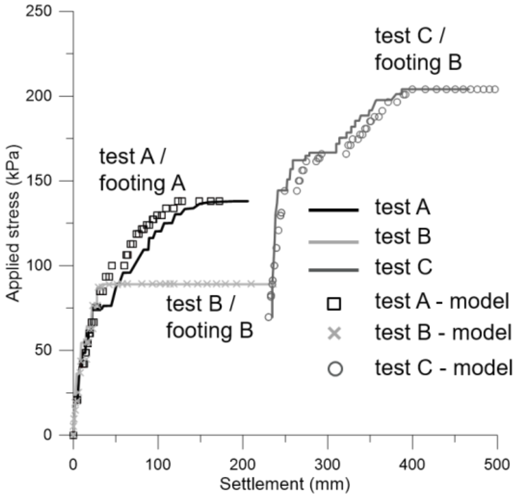

Figure 9. Load tests on footings at Bothkennar

4.2 Numerical modelling

The IC-ET model is employed in these analyses to simulate the behaviour of the Bothkennar clay. The calibration of the model and characterisation of ground conditions are detailed in Bodas Freitas et al. (2011, 2015). Fig. 10 shows that the calibrated TXC S_u profile, derived for the experimental shearing strain rate of 4.5 %/day, matches well the test data. This profile is marked as 'virgin strength' that exists in the soil before the construction of the footings and their loading.

The symbols in Fig. 9 show the predicted load-displacement curves from numerical analysis. Excellent reproduction of the experimental load-displacement curve in test A demonstrates consistency between the model calibration and characterisation of the initial ground conditions. The simulation of test B also shows excellent agreement with the test, mobilising a similar amount of settlement under the maintained load as experimentally observed. This is shown in more detail in Fig. 11, depicting the evolution of settlement with time. It was noted in the experimental study that consolidation was completed within a year under maintained load, implying that measured total settlement after 11 years is a combination of both consolidation and creep processes in the ground. A slight discrepancy between the predicted and measured settlements at 11 years is attributed to the observation that many instruments had stopped working since the previous reading at 2 years post-initial loading, as discussed in Lehane & Jardine (2003), thus questioning the interpretation of that last measurement.

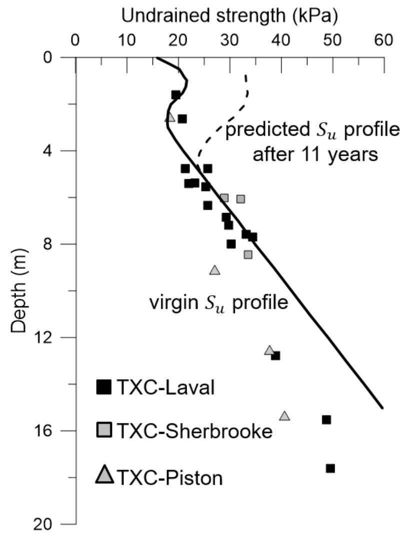

Figure 10. S_u profile for Bothkennar clay

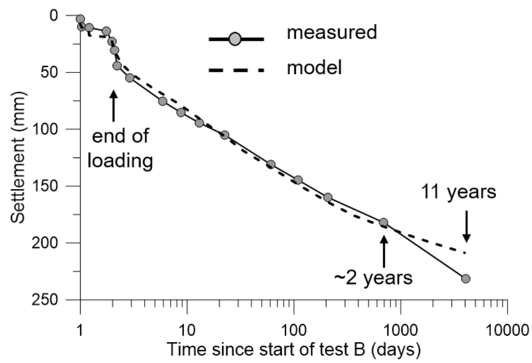

Figure 11. Evolution of settlement under the footing

The simulated further loading of the footing in test C, Fig. 9, is also in excellent agreement with test data, reproducing accurately the ultimate load and the gain in undrained capacity of the footing after 11 years of maintained load. This test demonstrates that the model was capable of accurately predicting the gain in strength due to consolidation and creep over those 11 years. The S_u profile under the footing at 11 years post-initial construction and loading is shown as a dashed line in Fig. 10. This verification therefore gives confidence that the predictions of raised embankment heights with the IC-ET model, after 30 years of maintained load, are likely to be correct.

5 CONCLUSIONS

The studies of earth embankments discussed here indicate that the short-term design of this structures needs to consider the anisotropy in the foundation soil induced by principal stress rotation during construction. For the long-term design it would appear that creep in the soil is more dominant than strength anisotropy. The applied equivalent time constitutive model is able to accurately predict the long-term changes in the soils mechanical behaviour.

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