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The effect of creep-induced settlement and strength gain on flood embankments

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ABSTRACT: Kilometres of flood embankments have protected coastal and river banks for decades. Predicted sea level rise due to climate change is likely to require these earth embankments to be raised. Historically, empirical and semi-empirical methods were used in the design of flood embankments founded on soft alluvial clays. Such methods, however, are unable to account for the enhancement of the stiffness and strength properties by transient processes in the clay. This paper presents a numerical study on the behaviour of a flood embankment. The problem is analysed using a plane-strain coupled finite element model of a trial embankment constructed in the Thames Estuary in the 1970s. The foundation soil is modelled using an overstress-based elastic-viscoplastic constitutive model in the equivalent time framework. The predicted settlements and gains in undrained shear strength due to the transient processes of consolidation and creep are quantified and verified against site measurements. It is shown that the numerical model can reproduce accurately the time-dependent behaviour of the alluvial clay.

Keywords: earth embankment; creep; numerical analysis

1 INTRODUCTION

Kilometres of earth embankments have protected the Thames Estuary against floods for decades. As the region developed and tidal patterns evolved, most embankments would have been progressively raised or rebuilt. The height of each rise is often limited by the undrained strength of the foundation soft alluvial clays, on which these embankments are typically constructed. Conventional empirical or semi-empirical designs do not account for time-dependent gains in undrained strength that have been observed in the foundation clay. The gain in strength over time is attributed to transient process of consolidation and creep. Consolidation is characterised by the dissipation of excess pore pressures generated during construction, while creep accounts for time-related changes in the soil fabric under constant load. These time-dependent processes reduce void space in the soil, inducing large settlements which can take years to develop.

With numerical methods in geotechnical design gaining traction, a number of constitutive models have been developed to simulate the time-dependent behaviour of soils (e.g., Adachi & Oka, 1982; Bodas Freitas et al., 2011; Leoni et al., 2008). Their performance is best assessed when applied to boundary value problems with site measurements to validate model predictions. This paper presents a numerical model of an earth embankment based on an instrumented trial embankment that was monitored for more than 10 years.

2 TRIAL EMBANKMENT

Following the devastating 1953 North Sea floods, the then Building Research Establishment (BRE) conducted extensive research using a series of trial embankments on the alluvial clays in the Thames Estuary. The Trial Embankment II (TE II, Figure 1) was constructed at Dartford, UK, next to an existing embankment.

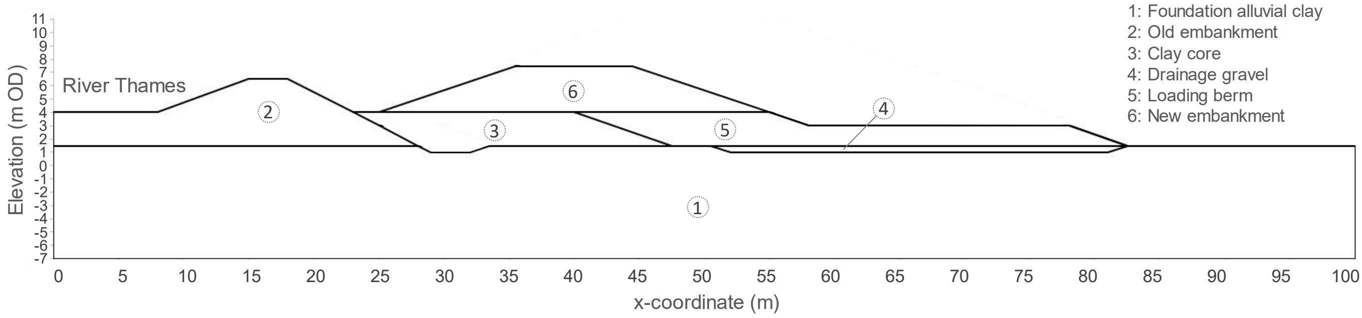

Figure 1. Cross-section of Trial Embankment II at Dartford

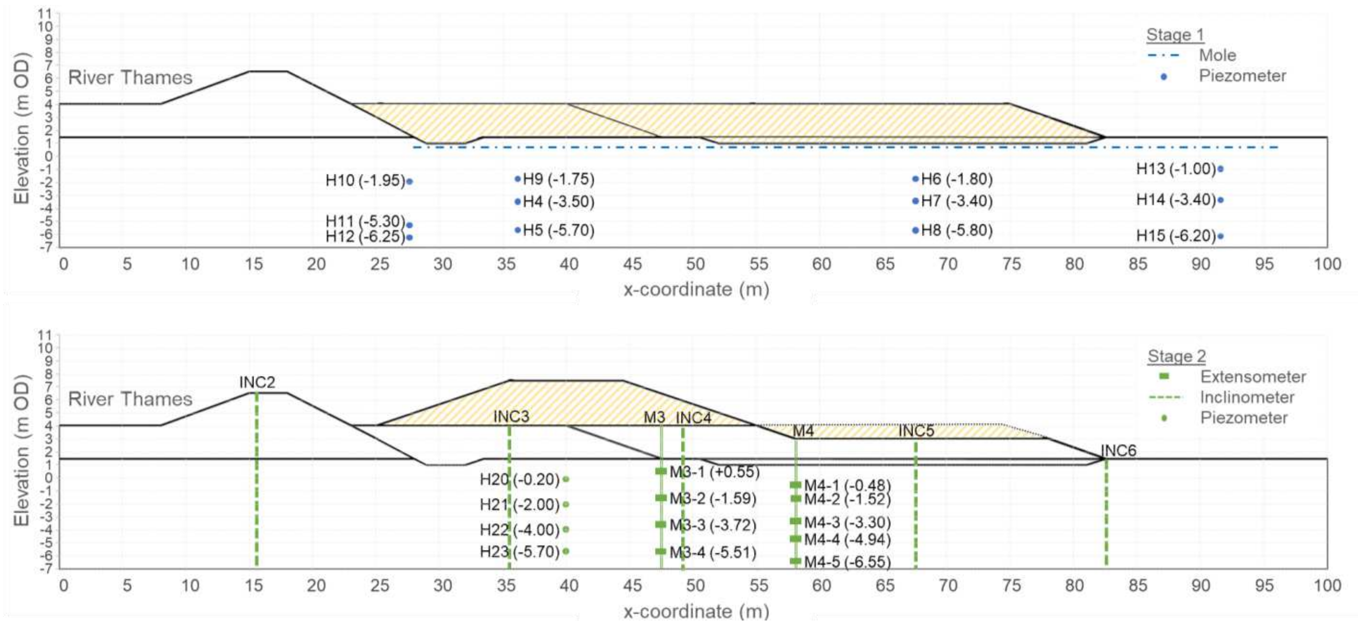

Figure 2. Instrumentation in (a) Stage 1 and (b) Stage 2

2.1 Ground conditions

A typical soil profile in the Lower Thames consists of 7 to 11 m of alluvial clay with slight organic content (numbered as 1 in Figure 1) overlying a gravel layer (Marsland, 1986). In the clay, there is a 1.5 to 2 m thick desiccated crust (Figure 3) due to the tidal variations in ground water table. Below the crust, it has been reported that there are slight degrees of overconsolidation and a gradual increase in undrained strength, from 5 – 12 kPa at the crust to 14 – 30 kPa at 10 m depth (Marsland & Randolph, 1978). The permeability of the clay ranges from 10^{-8} to 10^{-9} m/s.

The gravel layer, not shown in Figure 1, has a high permeability, in the range of 10^{-3} to 10^{-4} m/s, acting as drainage at the bottom of the alluvial clay.

The old embankment (no. 2 in Figure 1) was assumed to be a dense silty clay.

In the TE II, the clay core (no. 3 in Figure 1) was formed by a compacted plastic clay of low permeability. It extended into the impervious alluvial clay layer, reducing seepage through the embankment. It was abutted by a landward loading berm of sandy gravel (no. 5 in Figure 1), deposited on top of a shallow drainage ditch excavated in the alluvial clay (no. 4 in Figure 1). The fill for the new embankment (no. 6 in Figure 1) was compacted London Clay.

2.2 Construction and instrumentation

TE II was constructed in two stages. Commencing in September 1973, Stage 1 (as shaded in Figure 2(a)) comprised the construction of the clay core (25 days), drainage ditch (25 days) and the loading berm (15 days), and a 3-year monitoring period with no works. The mole and the four clusters of three piezometers, each shown in Figure 2(a), were monitored from the start of the Stage 1 construction to end-1974 and to mid-1976 respectively. Subsequently, Stage 2 (as shaded in Figure 2(b)) commenced in September 1976, involving the reprofiling of the loading berm, the raising of the new embankment over 50 days and a 10-year monitoring period with no works. The two multi-point magnetic extensometers, five inclinometers and four piezometers shown in Figure 2(b) were similarly monitored from the start of the Stage 2 construction to the late 1980s.

3 NUMERICAL MODELLING

To model the transient processes in the foundation clay, an overstress-based elastic-viscoplastic (EVP) model (Bodas Freitas et al., 2011; Perzyna, 1963) using the concept of equivalent time (Yin & Graham, 1989) was employed. A small strain stiffness model was also employed in the model formulation. All analyses were performed using the Imperial College Finite Element Program (ICFEP; Potts & Zdravković, 1999). A coupled hydro-mechanical plane-strain finite element formulation was employed with appropriate time steps to simulate the complex construction history of the site. The soil was considered isotropic, due to the lack of experimental data to help quantify any inherent strength and stiffness anisotropy. This was considered acceptable as the focus of the study was the evaluation of the effects of creep on the two-stage construction of the trial embankment. The study of Zdravkovic et al. (2019) demonstrated the dominance of creep effects over anisotropy in the assessment of soft clay response to multi-stage long-term loading.

3.1 Initial conditions

A constant unit weight of 16 kN/m^3 was adopted for the foundation clay. The water table was assumed at 2 m depth, coinciding with the bottom of a surface crust. An initial void ratio of 1.85 and the distribution of the coefficient of lateral earth pressure at rest, K_0, in Figure 3(a) were assumed. Using simulated undrained triaxial tests, the numerical model was calibrated for the overconsolidation ratio, OCR, profile in Figure 3(b) to reproduce the initial S_u profile in the foundation clay in 1973, before the construction of TE II (Figure 3(c)).

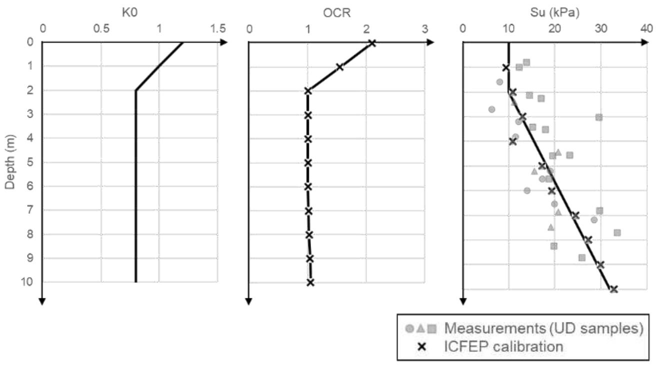

Figure 3. Initial conditions in foundation clay: (a) coefficient of lateral earth pressure at rest, K_0; (b) overconsolidation ratio, OCR; (c) undrained shear strength, S_u

3.2 Modelling of the foundation soil

Table 1 summarises the EVP model parameters when associated plasticity is assumed in the model. Due to limited soil investigation data, proxy parameters for the foundation alluvial clay were derived from other soft clays in the region.

The parameters relating to soil strength, (α, μ and M_{in}) were referenced from the soft, inorganic Bothkennar Clay that was extensively researched and published in the 1992 *Géotechnique* symposium. However, due to the presence of peat in the alluvial clay, the parameters describing compressibility (λ and κ) and creep behaviour (ψ_0/V and ε_{vml}^{vp}) were better approximated from the organic Mucking Clay data (Losacco, 2007; Pugh, 1978; Wesley, 1975).

Table 1. Parameters for EVP model

Description	Symbol	Value
Over-consolidation ratio	OCR	variable
Mean effective stress on the reference time line	p'_{mi0}	10.0 kPa
Slope of the compression line in $\boldsymbol{v - \ln p'}$ space	λ	0.385
Slope of the swelling line in $\boldsymbol{v - \ln p'}$ space	κ	0.064
Poisson's ratio	υ	0.26
Loading surface / plastic potential surface parameters	α	0.05
	μ	0.85
	M_{in}	0.065
Time-dependent parameter	ψ_0/V	0.004133
Time corresponding to the reference time line	t_0	1.0 day
Maximum volumetric viscoplastic strain	ε_{vml}^{vp}	0.0609

3.2.1 Small strain stiffness modelling

The small strain stiffness was modelled after Bothkennar Clay and sources of measured data are referenced in the legend of Figure 4. The Imperial College General Small Strain Stiffness (IC.G3S) model (Taborda et al., 2016) was employed, with shear stiffness model parameters given in the inset in Figure 4.

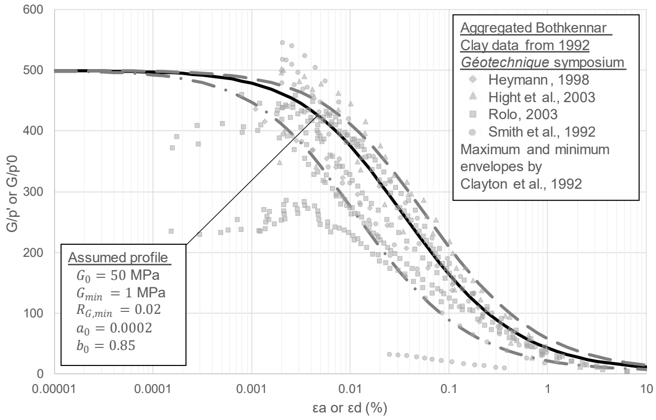

Figure 4. Stiffness degradation curve and parameters

3.2.2 Permeability parameters

A constant permeability of 10^{-8} m/s was adopted for the foundation clay as an initial assumption. No field or laboratory measurements of the variation of clay permeability were available for a more accurate representation.

3.3 Modelling of the remaining materials

The remaining material types in the numerical model (2 to 6 as shown in Figure 1) were simulated with a linear Mohr-Coulomb model. Assuming zero cohesion, c', in all soils, the parameters comprising the angle of shearing resistance, ϕ', and angle of dilation, ψ, are summarised in Table 2.

Table 2. Parameters for Mohr-Coulomb model

Soil type	$\boldsymbol{\phi'}$ (deg)	$\boldsymbol{\psi}$ (deg)
Old embankment (2)	35	11.7
Clay core (3)	22	7
Drainage layer (4)	32	16
Loading berm (5)	32	16
New embankment (6)	22	7

The permeability of the clay core and of the new embankment was assumed constant, at $k = 10^{-9}$ m/s, while the remaining materials were considered drained in a hydro-mechanically coupled analysis.

4 VALIDATION

The time-dependent behaviour of the foundation clay predicted by the model was compared with performance data from site instrumentation.

4.1 *Settlements*

During Stage 1, the surface settlement of the foundation clay was measured by a mole installed horizontally at 0.8 m depth under the footprint of the TE II. The short- and long-term surface settlements predicted by the model agreed well with the mole measurements (Figure 5). The model was able to capture the overall trend and spatial distribution of settlements, albeit generally underpredicting the settlement values by 10 to 20% in the first 6 months. A decrease in the rate of settlement was also observed. The average settlement rate in the short term (between Days 73 and 140 after the start of Stage 1 construction) was 2.8 mm/day and decreased to 0.87 mm/day in the long term (between Days 140 and 449).

During Stage 2, subsurface settlements in the foundation clay were measured by multipoint magnetic extensometers and inclinometers. As an example, the predicted subsurface settlements under the crest of the new embankment, at extensometer M3, are shown in Figure 6. The predicted evolution of settlements with time is in a reasonably good agreement with measurements, in particular in the long term. Initially, within the first year from the start of Stage 2 construction, the predicted settlements at all depths are larger than measured. Although not shown here for brevity, a similar trend was observed for the subsurface settlements beneath the toe of the new embankment, at extensometer M4 (see Figure 2(b)).

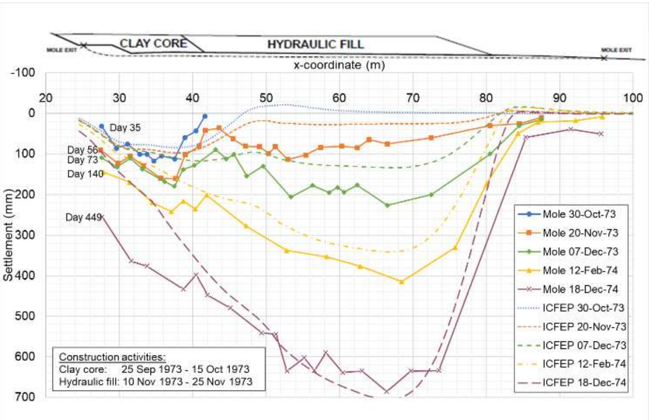

Figure 5. Stage 1 surface settlements, mole

Overall, the model was able to reproduce the time-dependent settlement behaviour of the alluvial clay. It performed comparably well in the short- and long-term, evidenced by settlement predictions over the two construction stages. This suggests that the values of ψ_0/V – proportional to the coefficient of secondary compression, C_α – are well captured and appropriate for the large creep deformations that persisted after the dissipation of excess pore pressures.

The discrepancy in the predicted shape of extensometer settlements with time is likely attributed to the simplified assumption of constant permeability in the foundation soil. Although not shown here, preliminary parametric analyses, in which the permeability was modelled as stress level (void ratio) dependent, have confirmed this. The magnitude of permeability reduces as effective stresses in the foundation soil increase during primary consolidation post-embankment construction, resulting in an initially slow rate of settlement. Once the effective stresses stop changing, the creep component of the model becomes more dominant, continuing to increase viscose settlements.

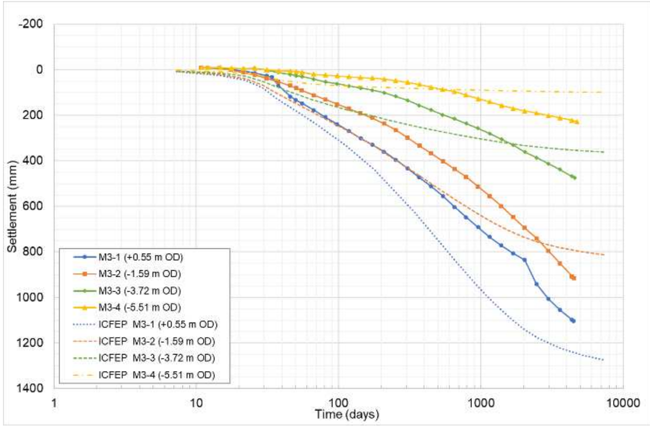

Figure 6. Stage 2 subsurface settlements, extensometer M3

4.2 *Lateral displacements*

Subsurface lateral displacements were measured by inclinometers at five points in the embankment cross-section during Stage 2 (see Figure 2(b) for inclinometer positions). The measurements showed that, as the new embankment was constructed, the underlying geomaterials moved laterally away from the centreline of the new embankment. The movements were relatively small and far from failure.

A comparison of the relative magnitudes and directions of the measured and the predicted lateral displacements after 180 – 220 days is shown in Figure 7. The model was able to capture the global trend of lateral displacements. However, it underpredicted the magnitudes of the movements, particularly those further away from the centreline (INC2, INC5 and INC6). This may be attributed to the simplistic modelling of soil permeability as isotropic and constant. Variation of permeability with stress level / void ratio may be more appropriate.

The numerical model also could not adequately reproduce the lateral movements caused by the interactions between the clay core, the loading berm and the foundation alluvial clay (INC3 and INC4). However, rather than being indicative of the inadequacies of the EVP model and parameters selected, this points to the need to refine the modelling of the remaining geomaterials (2 to 6) within the discretised domain. With no data to characterise these materials it was not possible to further enhance the analysis.

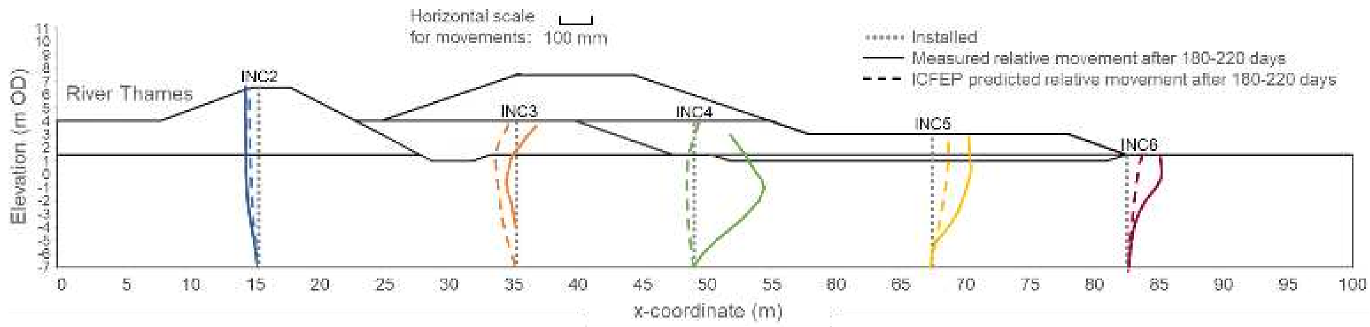

Figure 7. Relative magnitudes and directions of lateral displacements in Stage 2

4.3 Pore pressures

Piezometers were used to measure the change in pore pressures in the foundation clay during and after the construction activities in Stage 1 (Figure 8(a)) and Stage 2 (Figure 8(b)). Overall, the predicted pore pressures exhibited similar trends to the measurements. In both stages, there were rapid increases in pore pressures that corresponded to the placement of embankment fill above the various locations, followed by a gradual dissipation of these excess pore pressures.

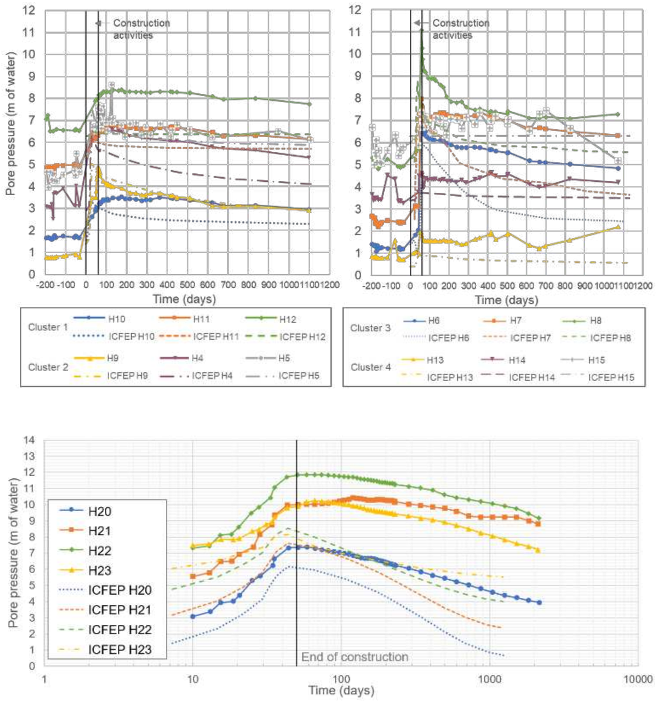

Figure 8. Pore pressures in the foundation clay: (a) in Stage 1, (b) in Stage 2

4.4 Undrained shear strength

During Stage 1, several profiles of S_u were obtained from undrained triaxial tests of undisturbed samples after the end of construction activities: three months (February 1974), one year (November 1975), and three years (September 1976). The profiles, taken in the middle of the Stage 1 platform, showed increasing strength with time, reflective of the effects of consolidation and creep on the undrained shear strength of the clay. The model similarly predicted time-dependent gains in S_u (Figure 9), although differing in the pattern of strength gain.

Three months after the end of construction, the predicted and measured S_u profiles agree. Both profiles are bow-shaped with the higher shear strengths at the top and bottom of the clay layer, indicative of ongoing consolidation with two-way drainage. One year after construction, the predicted S_u profile regained its linear increase with depth, whereas the measured profile retained its bow-shape. When considered with the trends in settlements and dissipation of excess pore water pressures, these differences in strength gain further evidence the need for a variable permeability function, as being more appropriate when considering the time-dependent behaviour of the foundation clay.

Three years after the end of construction, the model predicted an increase in shear strength of 12 kPa regardless of depth, resulting in a parallel rightward shift of the initial S_u distribution. The final predicted strength profile lies at the upper end of the measured strengths.

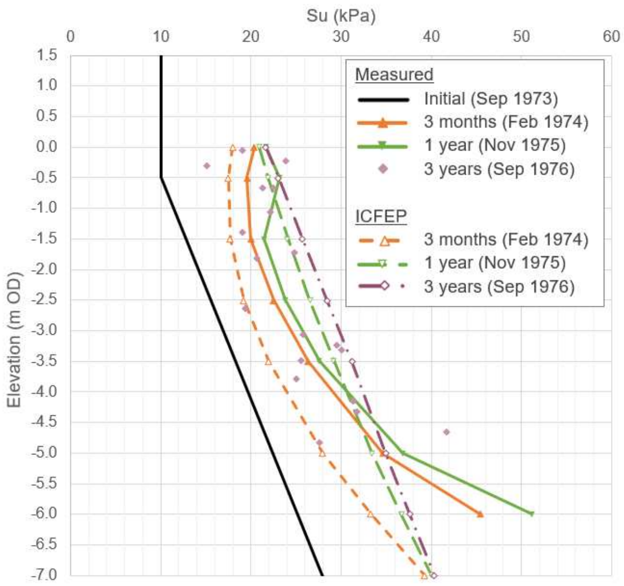

Figure 9. Evolution of Su profile with time in Stage 1

It is noteworthy that the predicted rate of increase in shear strength is higher in the first year (10 kPa/year) than in the subsequent two years (1 kPa/year). If consolidation (i.e. the dissipation of construction-generated excess pore pressures) is considered to have completed within one year, then the average rate of strength gain within the primary creep period is one magnitude larger than that within the secondary creep period. The extent to which secondary creep processes can further enhance soil strength will have to be assessed by validating model predictions of present-day strength (i.e. almost 50 years after the TE II works) against site investigation data.

5 CONCLUSIONS

The numerical model of a flood embankment developed here has been verified against site measurements over its two-stage construction. It was shown capable of reproducing the transient processes of consolidation and creep in the foundation alluvial clay in response to historical embankment raising works, despite adopting soil parameters from similar soft clays. Moving forward, the validated model can provide a preliminary understanding of predicted soil response to future embankment raising. Its performance in long-term predictions of settlements and lateral movements could be improved by performing site-specific characterisation of the alluvial clay for derivation of parameters for the mechanical constitutive model, but also by introducing time-dependent effects on other soil properties such as permeability.

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