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Finite element modelling of direct simple shear tests with pre-failure episodic shearing and consolidation

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ABSTRACT: Element-scale experimental direct simple shear (DSS) tests can be used to quantify the degree of softening and hardening of normally and lightly over consolidated clay under episodic loading sequences to characterise whole-life geotechnical response. However, little attention has been given to element-scale numerical analysis of DSS tests for characterising whole-life softening and reconsolidation responses, although numerical modelling provides a powerful tool to augment experimental element test campaigns. This paper presents a finite element model simulating DSS tests of a normally consolidated clay under (i) displacement-controlled undrained monotonic shear to failure and (ii) load-controlled undrained episodic monotonic pre-failure shear with intervening consolidation. The numerical results are compared with experimental DSS test results to assess the realism of the numerical model. Two critical state constitutive models are considered, (i) Modified Cam Clay (MCC) and (ii) Clay Hypoplasticity. It is demonstrated that the MCC model can capture the stress–strain–volume response of undrained monotonic shear to failure, and a single episode of pre-failure shear and subsequent consolidation, while the Clay Hypoplasticity model can also capture the softening and hardening response in subsequent episodes of pre-failure shear and consolidation, as a result of the inclusion of pre-yield pore pressure generation. The finite element model enables interrogation of aspects of soil response that are not available from the experimental element test output, and enables exploration of an extensive sweep of parameter combinations that would be impractical in the laboratory.
Keywords: Numerical Analysis; Geotechnics; Direct simple shear; episodic loads; Clay.

1 INTRODUCTION

It is well recognized that cyclic loading may reduce geotechnical capacity through soil softening while subsequent consolidation can improve it through soil hardening (Gourvenec, 2020), and episodes of pre-failure shearing intervened with consolidation periods have been shown to cause strength and stiffness increases in soft clays (Laham, et al., 2021) (Laham, et al., 2022). Such loading sequences could be caused by intervals of environmental or operational activities (Gourvenec, 2018). A design approach that allows for these progressive changes in soil parameters is referred to as 'whole-life geotechnical design'.

Experimental DSS element scale testing has recently been used to quantify the degree of softening and hardening of soft soils under these conditions, where evolving soil properties are captured through a whole-life design approach (Laham, et al., 2021). However, for such loading patterns, involving multiple episodes of shear and intervening consolidation, test times can be extensive, with each consolidation phase lasting up to 24 hours. Numerical modelling of the DSS can overcome this challenge, reducing the current burden on capacity of experimental laboratories to meet the demand for testing for foundation and anchor design to meet global offshore wind ambitions.

This paper presents a finite element (FE) DSS model that is capable of replicating experimental DSS tests under undrained episodic monotonic pre-failure shear with intervening consolidation. The model supports whole-life design and enables scrutiny of aspects of soil response that are not accessible in experimental DSS tests (e.g., the distribution of stresses and strains within a soil sample). The paper presents the theory of DSS and the constitutive models adopted in this study (Section 2), the setup of the finite element model, including geometry and boundary conditions, loading sequence, soil parameters, and testing programme (Section 3), and results in terms of undrained monotonic shear to failure and episodic monotonic soil response (Section 4). An important aspect of this paper is validation of the finite element DSS model against experimental DSS results comparing the stress distributions and different stress–strain behaviour, as demonstrated in Section 4.

2 THEORY

2.1 *Direct simple shear (DSS) test*

Direct simple shear is an experimental protocol to determine the stress–strain behaviour of soils under loading conditions that closely simulate plane strain and determine the associated shear strength parameter. During testing, the specimen is constrained axially between the two platens and is sheared by displacing one horizontal platen relative to the other at a constant rate of displacement and the resulting shear force is measured (Airey, et al., 1984).

Undrained simple shear parameters are determined by performing drained tests at constant volume by maintaining constant height of the specimen throughout shearing (Dyvik et al, 1987). For the sample to change volume under direct simple shear, it is necessary for the ends of the apparatus to be relatively free from shear stress. As a result, the distribution of shear stress over the horizontal boundaries at the top and the bottom of the sample become non-uniform at the start of shearing (Roscoe, 1953), where the applied shear stress falls towards zero at the edges as illustrated in Figure 1. The resulting unbalancing couple has to be counteracted by an opposite couple generated by a non-uniform distribution of normal stress on the top and bottom surfaces (Airey, et al., 1985).

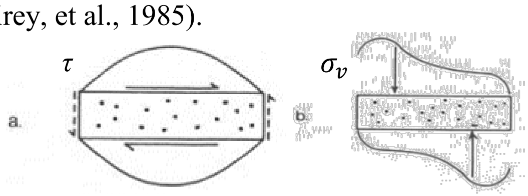

Figure 1. Stress distribution within DSS apparatus at the start of shearing (a) non-uniform distribution of shear stresses, τ, from absence of complementary shear stress on ends of sample, (b) non-uniform distribution of normal stress, σ_v, to preserve moment equilibrium of the sample (Airey, et al., 1985)

2.2 Constitutive models

Two constitutive models are considered in the finite element DSS to describe the behaviour of clays under undrained monotonic shear to failure and episodic monotonic pre-failure shear intervened with consolidation: the MCC elastoplastic model (Roscoe & Burland, 1968) and the Clay Hypoplasticity model (Masin, 2005). A plane strain finite element mesh was used to capture the simple shear conditions of the DSS experiment. The MCC model has been successfully used in the past to capture the evolution of soil properties to undrained episodic monotonic shear to failure with intervening consolidation (Hodder, et al., 2013) (Laham, et al., 2021).

However, MCC cannot capture soil hardening due to pre-failure shear followed by a consolidation as the yield surface separates elastic deformations from plastic deformations; such that strain occurring within the yield surface is treated as elastic deformation and no hardening is detected. In contrast, Clay Hypoplasticity defines a general asymptotic state boundary surface that can capture plastic strain increments within its surface, which allows capturing soil hardening due to consolidation after pre-failure shearing.

This paper presents the performance of the Clay Hypoplasticity model in capturing whole-life softening and hardening under pre-failure shear with intervening consolidation; alongside illustrating the capability of MCC in capturing undrained monotonic DSS to failure, or an initial episode of undrained monotonic DSS followed by consolidation. Results from all the FE analysis are compared with previously published experimental DSS results (Laham et al., 2021).

3 MODEL DESCRIPTION

A two-dimensional plane strain FE model was constructed to reflect conditions in a suite of previous experimental DSS tests capturing whole-life softening and reconsolidation (Laham, et al., 2021). The FE analyses were carried out in Plaxis 2D (Bentley, 2020).

3.1 Model geometry & boundary conditions

The soil sample was modelled as 25 mm in height and 100 mm in breadth, representing the dimensions of the mid-cross section of the experimental sample size. The external boundary conditions and applied loading during the experimental test set up for the consolidation and shear stages are shown in Figure 2. During the consolidation stage (Figure 2a), a uniform distributed vertical load was applied on the top of the sample, vertical settlements were permitted, and drainage was also permitted from the top and bottom boundaries. During the shearing stage (Figure 2b), the top of the sample was prevented from vertically displacing and the bottom of the sample was moved uniformly horizontally to capture sample shearing at a constant volume. The vertical sides of the sample were constructed with five smooth (frictionless) non-rotating plates of high stiffness, and end-to-end ties on either side of the sample (Figure 2c). Each plate was modelled as 1/5 of the sample height . to idealise the metal disks that radially confine the sample in the physical experiment. These plates were thicker than the metal disks that are used in the experiment, however, they enabled linear shearing at the boundaries to replicate the stress distribution expected for a soil sample under DSS loading conditions.

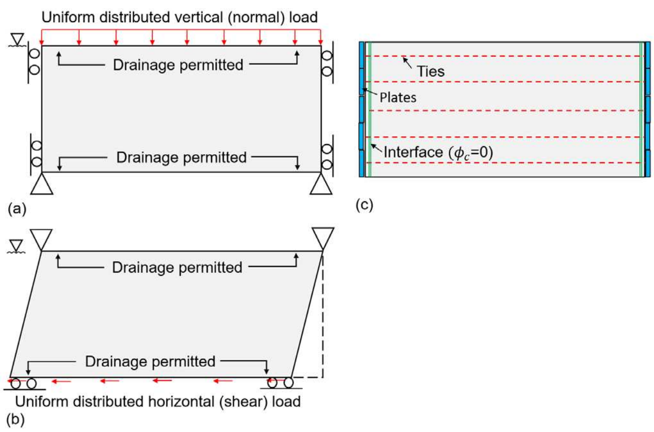

Figure 2. FE DSS (a) boundary conditions (BC) and applied loading during consolidation stage and (b) BC and applied shearing during shear stage and (c) model configuration

Capturing the displacement of the sides in concert is complex in plane strain since they are not physically connected, as is the case in 3D. A variety of options were explored, and this configuration was chosen as it resulted in normal and shear stress distributions that matched the theoretical distributions shown in Figure 1.

3.2 Testing programme & scope of loading

One FE analysis with displacement-controlled monotonic shear to failure and one with load-controlled episodic monotonic pre-failure shear with intervening consolidation were carried out to compare the observed results with those from existing experimental tests (Laham et al. 2021), validating the model to provide confidence in simulating more complex loading sequences or for a parametric analysis.

The loading procedure adopted in the episodic monotonic pre-failure shear test is illustrated in Figure 3 and consists of six episodes of undrained half pre-shear cycles with intervening consolidation stages, followed by a final monotonic, undrained shear stage (half-cycle, 7) to failure. The soil samples were pre-consolidated to 65 kPa representing an element of soil around 10 m below the mudline. A shear stress, τ, of 11 kPa, equivalent to 70% of the monotonic strength s_{ui}, is considered, representative of soil elements close to a loaded anchor. Constant height conditions were applied during the shearing stages in both monotonic and episodic monotonic tests to ensure constant sample volume, according to the method outlined in Dyvik et al. (1987), and as was applied in the experimental tests.

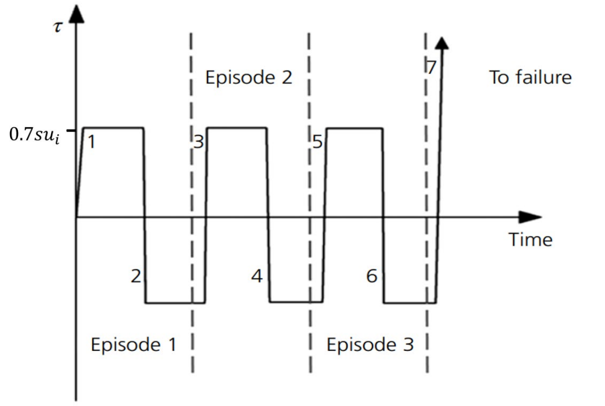

Figure 3. Shear stress loading pattern with time for undrained pre-failure episodic tests (Laham, et al., 2021)

3.3 Soil parameters

The soil parameters used in the FE analyses simulate a normally consolidated kaolin clay, based on the parameters of the material used in the experimental tests that the FEA are compared with, and are summarised in Table 1. These parameters are different in MCC and Clay Hypoplasticity because of the different formulations of the two constitutive models. The MCC parameters, derived from the experimental testing programme (Laham, et al., 2021) were calibrated for Clay Hypoplasticity by translating the MCC isoNCL and CSL into the hypoplastic asymptotic state boundary by converting from $e - ln(\sigma_v')$ space to $ln(1+e) - ln(\sigma_v')$ space. The critical state friction angle that was adopted in the MCC simulations is based on the ratio τ/σ_v' reached at critical state during experimental DSS tests on normally consolidated Kaolin (Zografou et al., 2018 Laham et al., 2021). An equivalent critical state friction angle was used in the Clay Hypoplastic simulations, which was determined from using a Lode angle dependent analytical approach for the Matsuoka-Nakai failure criterion as described in Medicus et al. (2022). The Poisson's ratio, ν, parameters were selected based on the best fit of the model predictions with the experimental monotonic shearing results from Laham et al. (2021). Saturated and dry soil densities were set equal to 18 and 16 kN/m^3, representing the measured densities of soil tested in the experimental testing programme (Laham et al. 2021) according to the phase relationships equations using unit weight of water γ_w, density of soil particles, G_s, and specific volume, ν, as an input.

Table 1. Constitutive parameters for kaolin clay (Laham, et al., 2021) (Medicus, et al., 2022)

Kaolin Properties	**Modified Cam Clay**	**Clay Hypoplastic**
Slope of NCL, λ	0.15	0.09
Slope of swelling line, κ	0.05	0.01
Void ratio at $\sigma_v' = 1\ kPa$ on NCL	-	1.26
Critical state friction angle, φ_c°	24	25.9
Poisson's ratio, ν	0.2	0.3
Slope of CSL, M	0.704	-
Ratio of horizontal & vertical shear moduli α_G, (-)	-	1.4

3.4 Testing procedure

The tests were initiated by fixing the water table at the top of the sample to ensure fully saturated conditions. The samples were considered normally consolidated and the K_0 stress generation was performed automatically due to the zero lateral strain boundary condition.

For simulating DSS, three different phases were implemented: (i) the initial stress generation phase where sample equilibrium was established, (ii) the consolidation phase, in which a constant vertical load was applied to the top of the model replicating the initial effective vertical stress and (iii) constant volume shearing. For monotonic shear to failure, a horizontal displacement was applied uniformly to the bottom boundary of the sample, sufficient to exceed 15% strain to ensure failure (similar to displacement control in the lab experiments); and for pre-failure shear, a distributed shear load was applied instead to mimic load-controlled conditions.

4 RESULTS

4.1 *Insitu stress state*

Under in-situ conditions after initial consolidation and before the start of shear ($\gamma_s = 0\%$), the model is shown to exhibit a stress state as expected: shear stress, τ, is zero (Figure 4a); normal effective stress, $\sigma_v{}'$ is uniformly distributed throughout the sample and equals to the applied consolidation stress, σ_v, (Figure 4b); and excess pore water pressure, Δu, is zero through the specimen having dissipated during consolidation.

4.2 *Undrained monotonic shear to failure*

The stress distribution within the model changes gradually under undrained monotonic shear. With shearing ($\gamma_s > 0\%$), shear stress is maximum in the centre of the sample and falls towards zero at the corners (Figure 4a). As a result, the stresses are counteracted by an opposite asymmetric distribution of normal stresses on the top and bottom surfaces (Figure 4b). The distributions obtained show agreement with previous theoretical studies on stress distribution in DSS (Airey, et al., 1985).

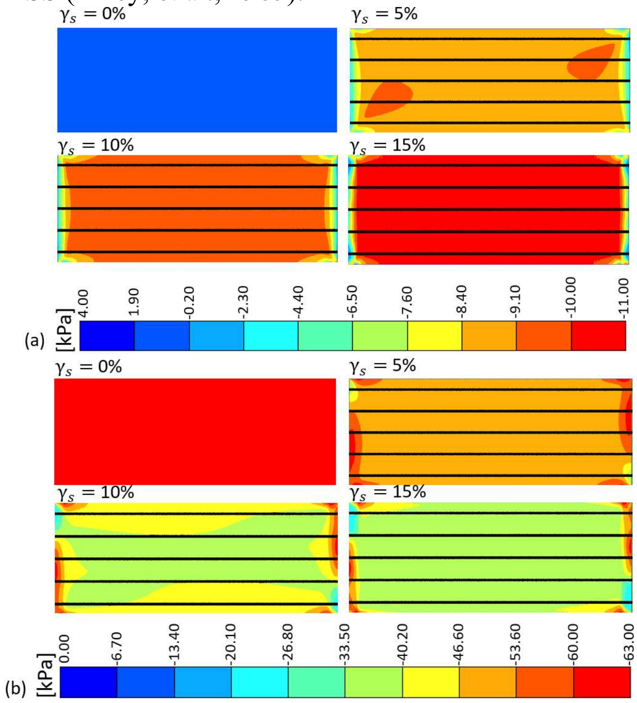

Figure 4. (a) Shear stress and (b) Normal effective stress (compression) distribution within soil sample during displacement-controlled undrained monotonic shear to failure

Figure 5 shows the effective stress path and shear stress–strain response of soil observed in the FE models and compared with the experimental results under undrained monotonic shear to failure. A reduction in effective stress is observed in response to generation of excess pore pressure during undrained shearing, until reaching the critical state line (CSL) (Figure 5a) marking the point of continued deformation at constant shear stress, $\tau_{ult,mon}$, (Figure 5b), then the state moves to the critical state line (CSL). The location of the CSL differs slightly in the MCC and Hypoplastic formulation and can be approximately fitted within a chosen range of normal stresses, with the ratios τ/σ_v' at critical state given by (Medicus, et al., 2022):

In MCC: $\tau/\sigma_v' = \sin(\varphi_c)$ (1)

In Clay Hypoplasticity: $\tau/\sigma_v' = \sqrt{\frac{4\sin^2\varphi_c}{3+\sin^2\varphi_c}}$ (2)

where, φ_c is the Mohr Couloumb and Matsuoka-Nakai critical state friction angles, the latter accounting for intermediate principal stresses.

Comparison of the observed effective stress paths and stress–strain responses from the FE models with the experimental test results, shown in Figure 5, validates the FE DSS model in both MCC and Clay Hypoplasticity for undrained monotonic shear to failure, and provides confidence in applying the model to simulate soil response in DSS, extending to more complex load paths.

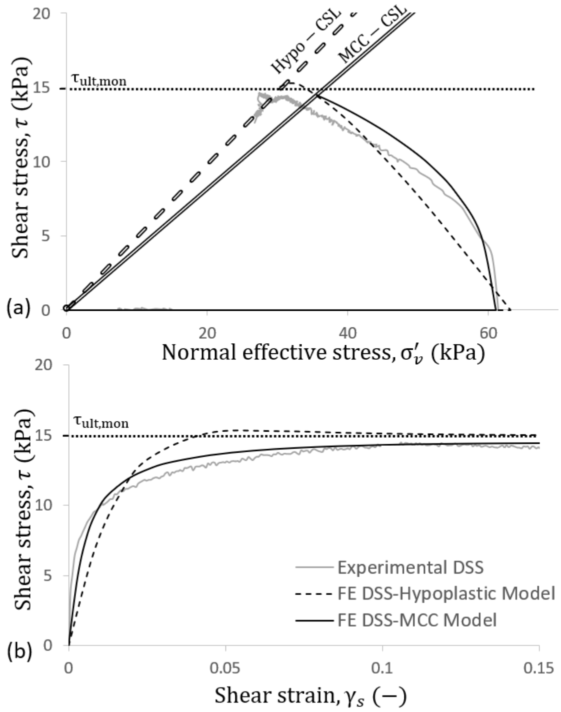

Figure 5. (a) Effective stress path and (b) shear stress–strain response under undrained monotonic shearing: FE results with MCC & Clay Hypoplasticity models compared with experimental results

4.3 *Undrained episodic monotonic pre-failure shear with intervening consolidation*

Figures 6 and 7 show the effective stress path and shear stress–strain response observed in the FE models in response to the applied sequence of episodic monotonic pre-failure shear and intervening consolidation (as illustrated schematically in Figure 3), and compared to the experimental DSS results.

It can be seen that the MCC model adequately captures the first full-cycle of shear, with the stress path bending to the left from the initial state O towards the

CSL but reversing at A when the target shear stress $0.7s_u$, equivalent to 11 kPa, is achieved. The MCC model also captures the response during the period of consolidation where the excess pore pressure dissipates, and the normal effective stress increases back to the initial value at A_1. However, no excess pore pressure generation can be captured with subsequent cycles of pre-failure shear from A_1 to A_2, as a result of the elastic assumptions within the yield surface. The intial cycle of shearing followed by consolidation causes an expansion of the yield surface, and some soil hardening occurs. This is manifested by the higher maximum shear stress mobilised during undrained shear to failure following the applied load path, i.e. $\tau_{ult,epi-mcc}$ in Figure 6a is greater than monotonic shear to failure without pre-shearing and consolidation, i.e. $\tau_{ult,mon}$, as shown in Figure 5a; but is less than observed after multiple episodes of hardening in the experimental DSS ($\tau_{ult,epi}$). Figure 7a reiterates the consequence of the elastic assumption inside the MCC yield envelope, on the stress–strain behaviour, showing no hardening along the strain axis after the first cycle of shear and consolidation.

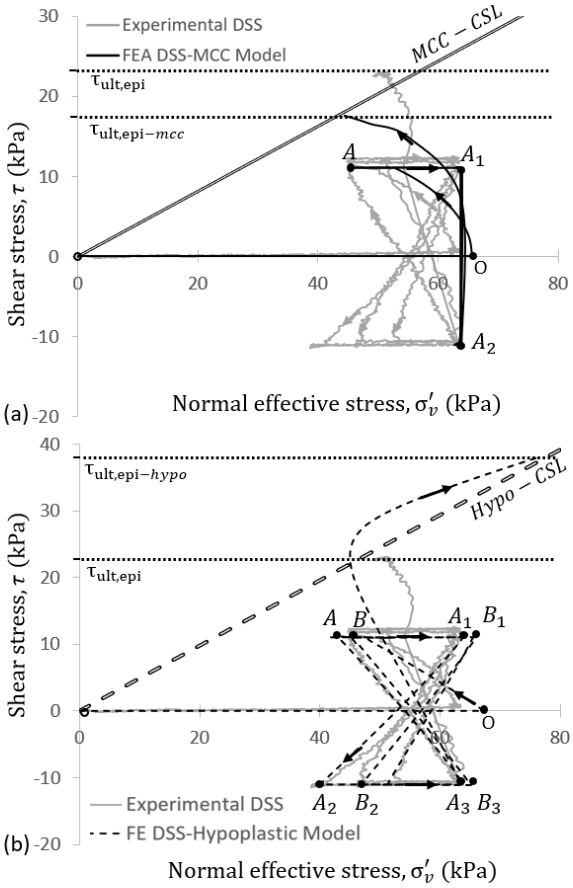

Figure 6. Effective stress path under undrained episodic monotonic shearing with intervening consolidation: FE results with (a) MCC and (b) Clay Hypoplasticity models compared with experimental results

Figure 6b and 7b show that the general asymptotic state boundary surface in Clay Hypoplasticity that can capture plastic strain increments within its surface, enables soil hardening due to multiple episodes of pre-failure shear and intervening consolidation. The stress path during pre-failure shear and intervening consolidation with the Clay Hypoplasticity model is consistent with the experimental results (i.e. path from O to $A, A_1 \ldots$).

However, the stress path during undrained monotonic shear to failure, following the sequence of episodic monotonic pre-failure shear and consolidation, i.e. $\tau_{ult,epi-hypo}$, indicates dilation to the CSL (i.e., bends to the right), and results in a higher shear strength than observed in the experimental DSS test, i.e. $\tau_{ult,epi}$. The shear stress–strain response from the Clay Hypoplasticity FE model, shown in Figure 7b, also captures the trend of continued plastic hardening as observed in the experimental DSS tests.

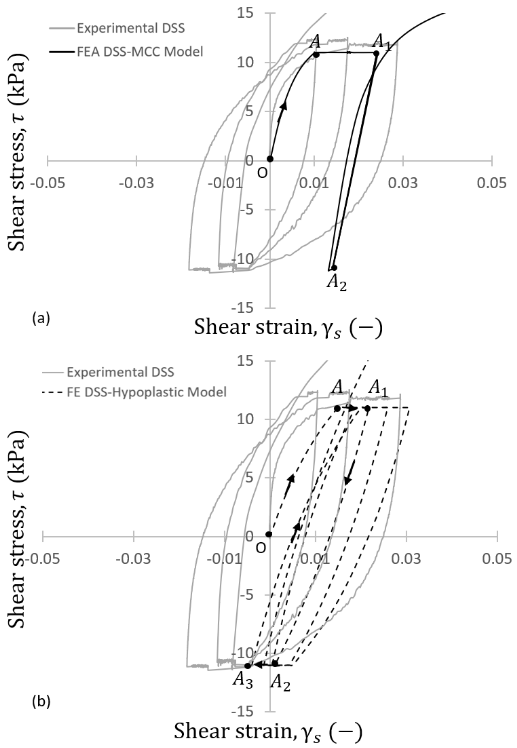

Figure 7. Shear stress–strain behaviour under undrained episodic pre-failure monotonic shear with intervening consolidation: FE results with (a) MCC and (b) Clay Hypoplasticity models compared with experimental results

4.4 Shear strength evolution from episodic pre-failure shear with intervening consolidation

Evolution of shear strength during cycles of undrained episodic monotonic pre-failure shear with intervening consolidation has been predicted through observation of the pore pressures and the constitutive relationship in $e - ln(\sigma'_v)$ space (Yasuhara & Andersen, 1991), due to the impracticality of monotonically shearing to failure at the end of each episode of shear and consolidation. Figure 8 shows the predicted increase in undrained strength with each half cycle from the Clay Hypoplasticity FE model is similar to that predicted from the experimental tests.

Further, the predicted strength at the last half cycle is close to the measured monotonic strength from the experimental DSS test. The increase in strength observed in the MCC FE model can be seen to plateau after the first half cycle of shearing and consolidation. The results show consistent patterns of increasing undrained strength during episodic monotonic pre-failure shear with intervening consolidation, as the soil become denser.

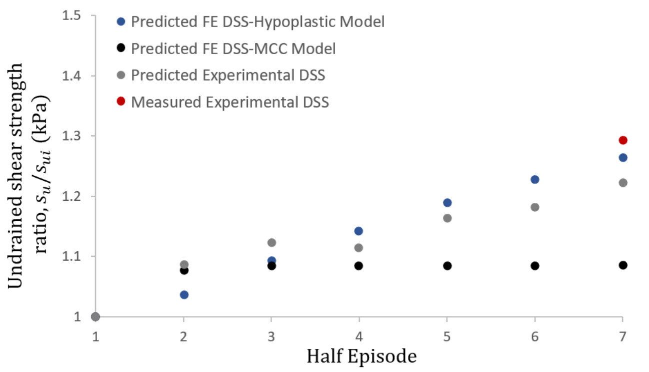

Figure 8 Predicted gains in undrained strength from FE and experimental DSS tests, compared with final measured undrained strength in experimental DSS (red)

5 CONCLUSION

This paper investigates the efficacy of a 2D FE model to capture soil response observed in experimental DSS tests, under both undrained monotonic shear to failure and episodic monotonic pre-failure shear with intervening consolidation of a normally consolidated kaolin clay. The numerical results are compared with the experimental results of equivalent tests.

The results show that the boundary conditions of a 3D experimental DSS test can be adequately replicated with a 2D plane strain FE model by incorporating constraints along the vertical edges in concert with connecting elements through the specimen, to force simple shear displacement. It has also been shown that Clay Hypoplasticity can capture the key features of whole-life softening and hardening due to episodic pre-failure shear with intervening consolidation.

The FE model enables scrutiny of the soil response throughout the soil sample within the DSS apparatus under whole-life conditions that are not accessible from experimental DSS test results. This enables exploration of an extensive sweep of parameter combinations that would be impractical in the laboratory. The validated FE model provides the opportunity to complement experimental DSS campaigns, to alleviate capacity challenges of experimental laboratories to meet the demand for testing for foundation and anchor design to meet global offshore wind ambitions. Such a model also supports routine use of consolidation-driven changes in soil response for whole-life geotechnical design, contributing to the optimization of reliable, cost-effective renewable offshore infrastructure.

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