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Small scale slope failure benchmark test. Modelling and prediction

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Abstract

A benchmark was designed with the aim of evaluating the capabilities of current modelling techniques and computational codes to reproduce slope failures. Laboratory tests and three instrumented small scale experiments of slopes initially unsaturated and subjected to a controlled rainfall up to failure were performed. The objective of the benchmark was to predict one of the slope failures knowing the rest of the data. The paper presents the modelling strategy and the results obtained using the finite element code Code_Bright and the Barcelona Basic Model as the constitutive model for the unsaturated soils.

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Keywords: unsaturated soil; small scale experiment; slope failure; parameter callibration; modelling.

1. Introduction, benchmark description and modelling approach

Natural slopes in Campania region are often covered by loose pyroclastic deposits underlain by fractured carbonate rock. Internal friction angle of these materials is close to 38º, which is lower than the slope angle, around 40º. In such conditions, cohesion induced by suction plays a key role to ensure equilibrium.

Shallow landslides involving pyroclastic soils are shown to be a sudden phenomenon in most cases not preceded by major visual evidence of impending failure. An experimental program involving laboratory tests on samples and small scale slope failure tests was designed to prepare a benchmark test in an effort to establish the capabilities of current modelling approaches to predict slope failure conditions.

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Two information packages were delivered to participants in a benchmark modelling exercise: the first package included results of laboratory tests on samples of pyroclastic soil. The second one describes the performance of two small scale experiments of slopes subjected to simulated rainfall (tests D3 and D4 in the remaining). The objective of the benchmark is to predict the time of failure of a third experiment (test C4) performed also on a small scale slope.

This paper describes the modelling strategy adopted to reproduce the laboratory results and the available small scale slope experiments. A prediction of the failure time of the additional experiment is given. Analyses were performed with FE computer code "Code Bright"[2,3]. The code solves coupled THM problems in deformable saturated-unsaturated porous media. A BBM constitutive model[4] was adopted to interpret the mechanical tests provided.

2. Model parameters

Isotropic compression tests performed on a suction controlled triaxial cell were analyzed to derive BBM compressibility parameters and to estimate the yielding conditions. Samples had a high void ratio (2.13-2.14) and were subjected to the stress paths indicated in Figure 1 (inset). The figure shows a comparison of sample response and model approximations for two initially unsaturated samples (at an estimated suction of 40kPa) and a saturated sample. The saturated preconsolidation isotropic net stress was estimated as p_0^*=31kPa. Estimated model parameters are given in Table 1.

Table 1. Model parameters

Elastic parameters for non-linear elasticity model : $\frac{\Delta e}{1+e} = a_1 \Delta \ln(-p') + a_2 \Delta \ln((s+p_{atm})/p_{atm})$			Plastic parameters for BBM					
a_1	a_2	Poisson ratio, ν	λ(0) – κ	r	β	p^c	k	M
–0.01	0	0.2	0.107	0.5	8MPa^{-1}	0.001MPa	0.25	8

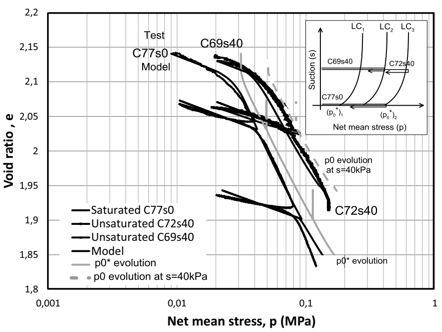

Fig. 1. Experimental results and model predictions for compressibility tests.

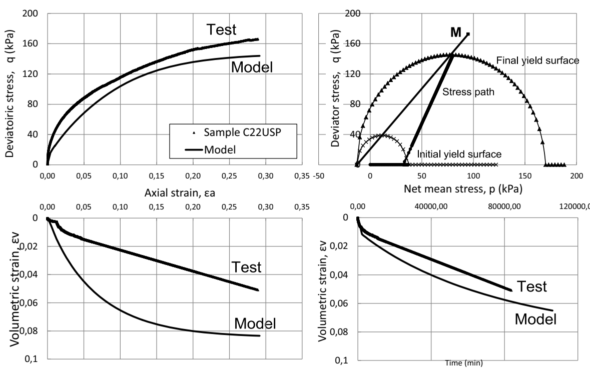

Fig. 2. Experimental and modelling results corresponding to test C22USP s43.

Ten suction controlled triaxial tests were also available. Six of them, those performed at initial suctions in the range 43-74 kPa, were analyzed. The analysis involved a relatively lengthy period of adjusting parameters with the purpose of simulating all the experiments with a single set of values (those given in Table 1). A comparison of model performance and actual measurements for one of the tests is given in Figure 2.

The hydraulic part of the model required an estimation of the water retention properties of the soil, the saturated permeability and its variation with suction. Data relating volumetric water content and suction was obtained from triaxial tests, evaporation data from reconstituted samples, data derived from pressure plate and infiltration tests. Data from flume test D4 (wetting path) was included in the analysis. The set of data points are collected in Figure 3 together with a van Genuchten approximation and the parameters used. The approximation selected is closer to data derived from flume test D4 which is considered more reliable because it corresponds to a wetting path. Also included in the figure is the estimated water retention curve for the pervious drains installed at the lower end of the small scale slope models.

Permeability is also a key parameter to estimate the time evolution of pore pressures and displacements of the model slopes. Data on the variation of saturated permeability with confining stress was also provided in the exercise. This data is shown in Figure 4. However, the average confining stress in model slopes is very small (the thickness of the soil layer was only 10 cm) and it is essentially outside the range of applied stresses in Figure 4. There were two additional sources of information: (a) global observations of the model slope reaction to rain and (b) the fitting of pore water pressure evolution at some piezometers monitoring the response of the model slopes subjected to continuous rain infiltration.

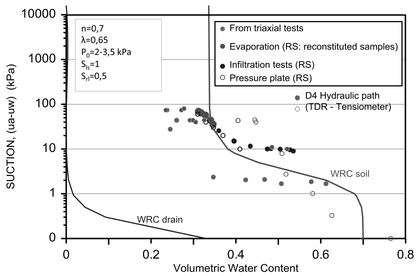

Fig. 3. Water retention curves for pyroclastic soil and base drain. Experimental data and curves estimated using van Genuchten Model is indicated.

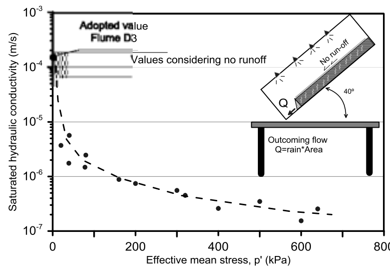

Fig. 4. Saturated hydraulic conductivity of remolded pyroclastic soil.

The first comment refers to the absence of observed runoff in the model slopes and the development of (small) positive pore pressures at the base of the slope prior to failure. These conditions forced to narrow the estimated range of saturated soil permeability around 10^{-4} m/s. The adopted value is shown in Figure 4 for the very small confining stress expected in the slope tests.

The variation of relative permeability with degree of saturation follows a power law $k_{rl} = A\, Sr^{\lambda}$. A value of λ between 4 and 5 was back calculated from the evolution of water pressure reported from tensiometers in model tests D3 and D4.

3. Modelling slope tests D3 and D4

According to the received information soil was mixed with water to the desired water content and then tamped in layers to the target dry density. Little or even null compaction was applied in test D3 but some tamping was needed in D4 resulting probably in a different initial position for the LC yield surface, a change which was not introduced in the model. After compaction and once equilibrium conditions were achieved (based on suction measurements) models where tilted to the desired slope angle and prepared for rainfall testing. In the calculation model, the emplacement procedure was not simulated exactly as performed and then the initial state of stress and the corresponding position of the LC yield surface were estimated. This estimation was made assuming that the soil was not overconsolidated and taking into account the given value of the void ratio and the available compressibility curves.

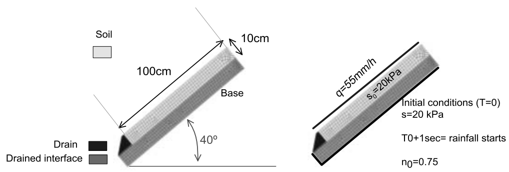

Fig. 5. Model geometry, mesh, boundary and initial conditions (Slope model test D3).

The following phases define the modelling work: (i) first a tilted base is built, (ii) then the soil is emplaced over the sloped base and (iii) finally the infiltration phase is simulated (rain). The drain located at the toe of the slope was modeled as a granular rigid material having a constant hydraulic conductivity value one order of magnitude higher than the one of the soil. Model geometry, mesh, boundary and initial conditions adopted for test D3 are shown in Figure 5. Material was emplaced considering an isotropic compressive net mean stress of 0.01MPa and then stress redistribution to equilibrium was permitted prior to the start of rainfall.

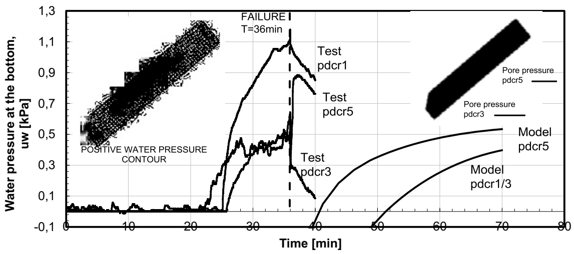

Fig. 6. Evolution of measured and calculated water pressure at the base of test D3.

3.1. Slope test D3

Consider first the response of the pore pressure sensors installed at the base of the test. They are able to record only positive water pressures. Measurements are compared with model results in Figure 6. Positive pore pressures develop in the test after 22-25 minutes after the initiation of rainfall. The model predicts a negative pore pressure for a longer time and pressures begin to change to positive values beyond 36 minutes. The sudden change in the rate of (positive) water pressure increase marks the beginning of failure in the measured record.

Negative water pressures were however recorded in tensiometers located inside the soil mass. This is shown in Figure 7 for sensors T3 and T6. Sensor T3 was closer to the bottom and it reacts slowly compared with the near surface sensors. The model reproduces quite well the history of suction dissipation. Note also, comparing Figures 6 and 7, that the calculated values of positive pore pressure, once the base of the slope reaches saturation, are much smaller than the absolute value of suction measured and calculated at early stages inside the soil mass.

Displacements (horizontal and vertical) were also calculated in model D3. They are compared with measurements in Figure 7. Displacements sensors became inoperative a few minutes before the reported failure time. Calculations tend to overestimate the measurements and they show a distinct failure at time 44 minutes. This is about 10 minutes longer than the actual failure instant. The results of test D4 are collected in Figure 8 in terms of suction dissipation in four tensiometers (T2, T3, T4 and T6). Again the deep sensor reacts in a slower manner compared with surface devices. This is also well captured by the model.

4. Predicting test C4

Data in Figure 6 indicates that failure is recorded for small values of positive pore pressure (0.4 – 0.6 kPa) at the bottom of the test. Failure is achieved around 10 minutes after the beginning of the development of positive water pressures. Model predictions for case C4 have been based on the calculated evolution of water pressures at the bottom of the test. The calculated dissipation of suction in a few points and the development of positive pressures at the base of the test are shown in Figures 9 and 10. Figure 9 in particular indicates that positive pore water pressures in the order of 0.5kPa develop at 34-39 minutes after the beginning of the rainfall test.

5. Discussion and conclusions

This paper presents a short account of the entire modelling and backanalysis performed. The approach followed may be described as “canonical” in the sense that it followed a “rational” set of stages: selecting a reasonable elastoplastic model to represent the soil mechanical behaviour, deriving constitutive parameters from sample tests, predicting/backanalyzing two "prototype" tests, identifying failure with a rapid increase in time of calculated displacements and applying the model to the "benchmark" case.

Things are not so linear and simple, however. The determination of hydraulic conductivity is hardly reliable if it is based on samples. A similar comment can be made on water retention. The marked drying-wetting hysteresis makes very unreliable predictions based on drying branches if the real problem involves soil wetting (which is the case), displacement calculations are not reliable if based on parameters derived from sample tests and, on top of it all, computations are plagued with convergence issues. Time to failure of test C4 was predicted on the basis of calculated evolution of pore water pressures in three sensors and it is summarized in Figure 10. It is predicted to be 34 to 39 minutes after the beginning of the rainfall action.

Despite the difficulties mentioned, the case is still capable of a more refined analysis which could incorporate in a more precise manner some test construction details and a further evaluation of the available information.

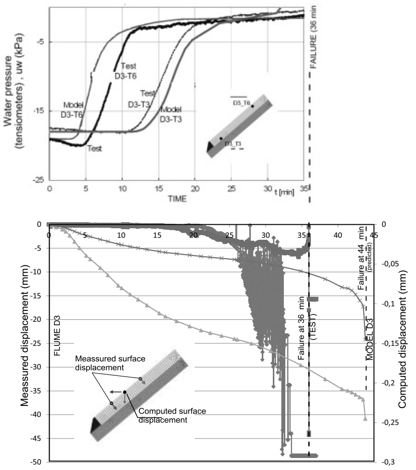

Fig. 7. Suction dissipation in tensiometers located inside the soil in test D3. Also plotted are model predictions.

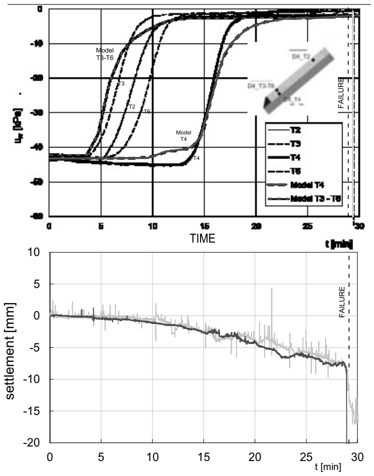

Fig. 8. Suction dissipation in tensiometers located inside the soil in test D4. Also plotted are model predictions and measured displacements.

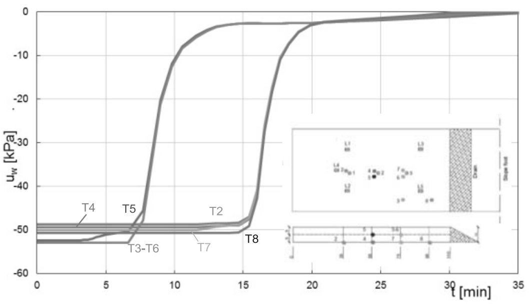

Fig. 9. Calculated suction dissipation in tensiometers located inside the soil in test C4.

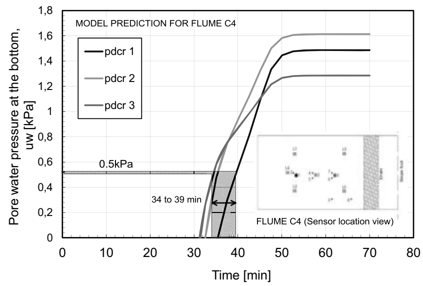

Fig. 10. Calculated evolution of water pressures at the bottom of test C4 in the positions marked. The estimation of failure time is also indicated.

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