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## Model preparation for unsaturated soil testing in a centrifuge environment

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**ABSTRACT:** This paper presents selected aspects of the model preparation and the experimental procedures used for testing the unsaturated soil behaviour in centrifuge environment. The study is based on the results of centrifuge tests carried out to investigate the influence of partial saturation on the behaviour of laterally loaded piles. The soil used in the experimentation is a high permeability silty clay soil, named B-grade kaolin. The models were statically compacted at two different densities with the same water content. In order to reduce the after compaction suction, the samples were subjected to an imbibition process at  $1\times g$  connecting the bottom of the model with the water reservoir. The influence of the compaction process and the imbibition stage on the subsequent inflight soil conditions are discussed and analysed. The centrifuge tests were realised at the centrifuge centre of the University Gustave Eiffel, in Nantes, France, in the framework of the GEOTRANSALP project.

**Keywords:** Unsaturated Soils; Static Compaction; Water Retention Curve; Centrifuge Modelling; Hydro-Mechanical Stress Paths.

### 1 INTRODUCTION

Centrifuge modelling represents a valid methodology to investigate the performance of different types of geotechnical works such as shallow and deep foundations, tunnels, slopes and embankments. Traditionally, studies have been conducted on fully saturated or completely dry soil models. However, in some cases the variation of the water content in the soil due to the water table oscillation, rainfall events, interaction with the atmosphere and exceptional events (i.e. pipe rupture etc...) may be crucial for the geotechnical works behaviour. These and other problems have been recently investigated by centrifuge modelling (Casini *et al.*, 2009; Soranzo *et al.*, 2015; Thorel *et al.*, 2011). Scaling laws for unsaturated soils have been experimentally investigated, among others, by Depountis *et al.* (2001) and Caicedo *et al.* (2006) under different soil condition and  $g$ -levels. The authors found that capillary rise and diffusion time in the centrifuge could be scaled by  $1/N$  and  $1/N^2$ , respectively. Details of the analytical formulations can be found in Caicedo and Thorel (2014).

Compaction is the most common technique to prepare unsaturated soil samples (Caicedo and Thorel, 2014). Dynamic compaction allows to control the density but not the compaction stress. On the other hand, with the uniaxial static compaction procedure both density and compaction stress can

be measured although the horizontal stresses are in general unknown.

This paper presents the details of the model preparation of the experimental investigation conducted by Lalicata *et al.* (2019). The study aimed at assessing the influence of partial saturation on the response of laterally loaded piles installed in fine-grained soils. The centrifuge tests ( $N = 100\times g$ ) were carried out using the centrifuge facilities of the University Gustave Eiffel, located close to Nantes (France) in the framework of the GEOTRANSALP-PILE-UNSAT agreement.

### 2 MATERIALS

#### 2.1. The soil used

The material used is a commercial soil named B-Grade kaolin (90% fine silt and 10% clay). This soil has a low plasticity index ( $IP=14\%$ ) and a low liquid limits ( $w_L=42\%$ ), the plastic limit is  $w_P=28\%$ . A complete hydro-mechanical characterisation was conducted in the geotechnical laboratory using standard apparatus for unsaturated and saturated soils. Results are reported in Lalicata (2018) and Lalicata *et al.* (2020). For the sake of brevity, only the flooding oedometer tests and the suction controlled oedometer tests are presented here. The hydro-mechanical parameters of the soil are listed in Table 1.

**Table 1.** Hydro-mechanical properties of B-Grade kaolin.

$\rho_s$ (g/cm <sup>3</sup> )	$C_c$	$C_s$	$N_o^*$	$\phi'$ (°)	$c_v$ (mm <sup>2</sup> /s)
2.66	0.26	0.078	1.36	22	1.0

\* $N_o$  is the voids ratio at  $\sigma'_v=1$  kPa under oedometric conditions.

Several imbibition tests in the oedometer apparatus were undertaken to define the after compaction conditions ( $w$ ,  $e$ ) for centrifuge models. The samples (18 mm thick and 70 mm in diameter) were prepared with different compaction features and subjected to different total vertical stresses  $\sigma_v$  applied before the wetting stage. In the main series of the tests, the vertical stress was 150 kPa corresponding to the vertical stress active at half of the pile embedded length ( $z=8$ m). Samples were left equalise under the external load before filling the oedometer cell with water.

The soil powder was dried at 105 °C for 24 hours before adding the demineralized water to reach the target water content. The moisture was manually mixed and then it was kept in sealed bags for 24-48 hours to homogenise in a temperature-controlled environment. The specimens were statically compacted ( $v=1.5$  mm/min) directly in the oedometric ring.

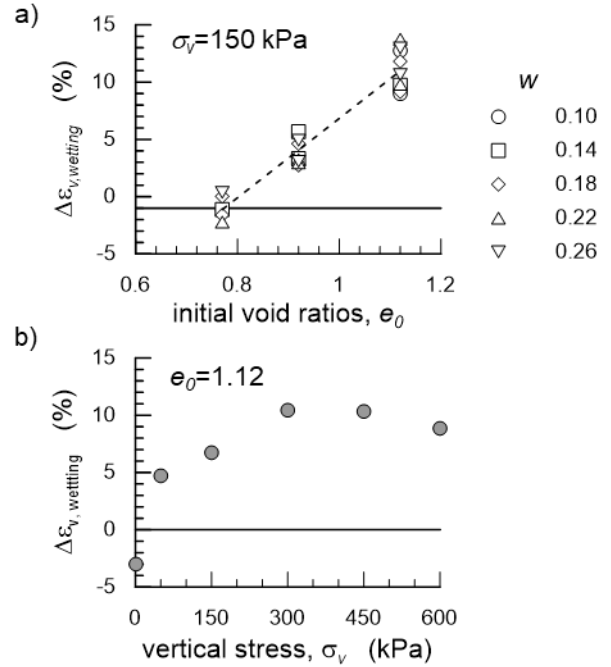
Three voids ratio are taken into account ( $e_0 = 0.77$ , 0.92 and 1.12), for water content ranging from 10% to 26%, with steps of 4%. For each grid point, at least two samples were tested.

Figure 1a shows the wetting induced deformation under  $\sigma_v = 150$  kPa in function of the compaction void ratios. Water content measurements at the end of the test have shown that all the samples reached degrees of saturation closed to one ( $S_{r,min}=0.95$ ). Results are grouped for the initial water content. For the material, the latter does not have a significant influence on the wetting deformation while the results strongly depends on the initial void ratios. The deformations reduce approximately linearly as the voids ratio decrease. Little swelling was observed for the more compacted samples ( $e_0=0.77$ ).

The influence of the applied external load during samples inundation is depicted in Figure 1b, for stresses ranging from 0 to 600 kPa. Samples were prepared at  $e_0=1.12$  and  $w=14\%$ . Soil swelling occurs only in absence of external load applied, while since from  $\sigma_v= 50$  kPa the soil experience positive deformations that increase up to  $\sigma_v= 300$  kPa and then they slightly reduce in magnitude. Thus, in the centrifuge, this soil would have experienced modest swelling in the shallower layer and volume reductions increasing with depth.

In order to explore the influence of the compaction conditions of soil in the centrifuge tests, two void

ratios are selected (0.93 and 0.75) prepared with the same water content  $w=15\%$ . The two densities are considered representative, respectively, of a collapsible soil and a swelling soil under wetting processes. The higher void ratios of 1.12 was discarded as showed deformation too elevated, Figure 1b.



**Figure 1.** Wetting induced deformation versus: a) initial void ratios, b) total vertical stress.

The soil water retention curve (SWRC) was obtained by means of the suction controlled oedometer (Romero et al. 1995). The data corresponding to the main wetting curve a two different void ratios, 0.93 and 0.75, are presented in Figure 2 in terms of suction  $s$  (in log scale) versus the degree of saturation  $S_r$ . In the suction range experimentally studied, the data exhibit a marked influence of porosity in the retention behaviour. In fact, as porosity decreases, air entry value increases and the slope of the transition zone reduces in the  $S_r-s$  plane. The Van Genuchten (1980) equation was used to fit the data:

$$S_r = S_{r,res} + (S_{r,sat} - S_{r,res}) \cdot \left( \frac{1}{1 + (\alpha s)^n} \right)^m \quad (1)$$

Where  $S_{r,res}$  and  $S_{r,sat}$  are respectively the residual and saturated degree of saturation;  $\alpha$  is the inverse of the air entry value,  $n$  governs the slope of the transition zone and  $m$  controls the inflection point at the residual values of suction. The values of the parameters are listed in Table 2. Neglecting the hysteresis of the SWRC, eq (1) can be used to

estimate the as-compacted suction and thus the initial effective stress in the model according to the Bishop effective stresses definition:  $\sigma' = \sigma + S_r \cdot s$  (Bishop and Blight, 1963). As the total stress is zero after the compaction the effective stresses in the soil reduces to the product  $S_r \cdot s$ , and are 387 kPa for  $e_0=0.93$  ( $S_{r0}=0.43$ ) and 1768 kPa for  $e_0=0.75$  ( $S_{r0}=0.52$ ) respectively.

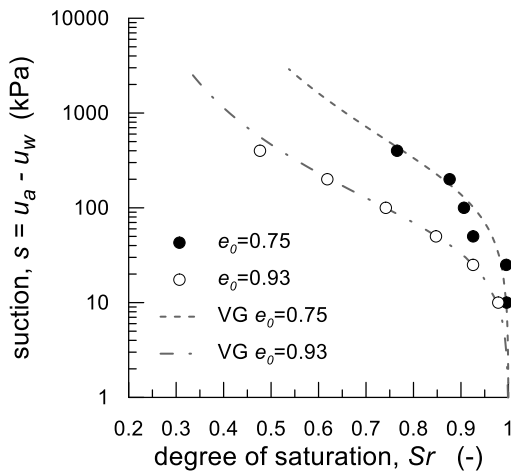


Figure 2. Soil water retention curves, SWRC, for the two densities used in the centrifuge tests.

Table 2. Van Genuchten parameters for different void ratios.

$e_0$	$S_{r,sat}$	$S_{r,res}$	$\alpha$ (1/kPa)	$n$	$m$
0.93	1.0	0.16	0.02	1.4	0.25
0.75	1.0	0.16	0.006	1.3	0.23

## 2.2. The model pile

The model pile (scale 1/100) is a close-ended aluminium tube (external diameter  $D=12$ mm, thickness  $t=1$ mm) embedded into the soil for 150 mm. The pile was instrumented with 10 pairs of strain gauges arranged every 15 mm, providing the bending moment profile along the pile length. The strain gauges were calibrated in the laboratory adopting the cantilever scheme with point load. The experimental arrangement for the calibration is presented in Figure 3.

## 2.3. The experimental apparatus

The axial symmetric models 180 mm deep were prepared in 300 mm diameter rigid tub. As shown in Figure 4, two twin containers were spin up at the same time: one was used for CPT measurement before, during and after the test. The other was used for the load test on the pile that was bored in the centre of the model, at  $1 \times g$ , by means of a manual screwing system. The screw was mounted into a rigid frame to maintain position and verticality.

The model was extensively instrumented in order to follow both the equalisation phases and pile loading (Figure 5).



Figure 3. Model pile and calibration set-up.

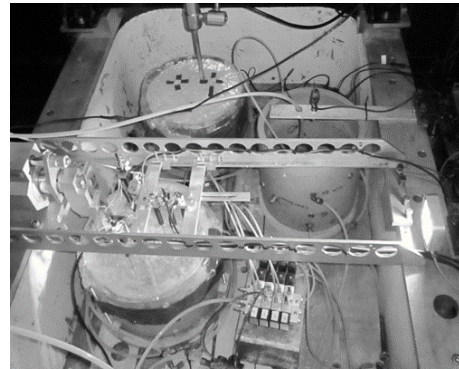


Figure 4. Model assembled in the strongbox

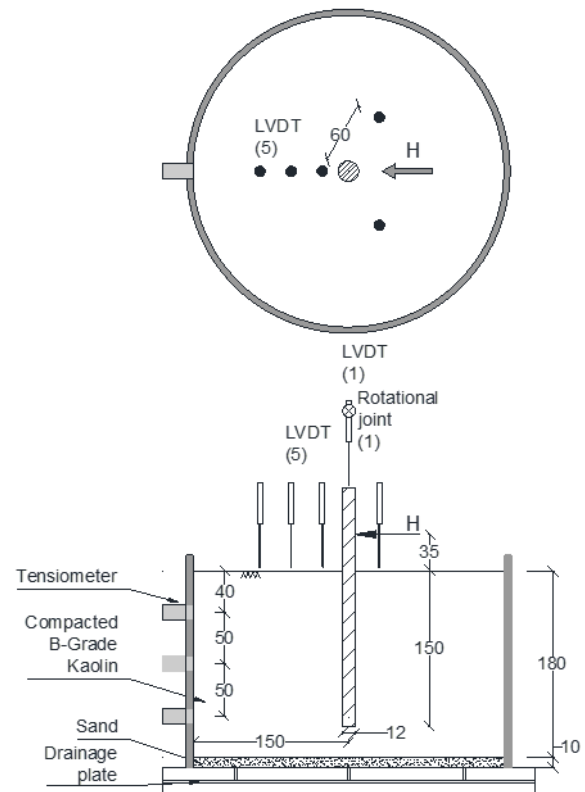


Figure 5. Model instrumentation.

The bottom of the model was connected to a water reservoir, the level of which was governed by an electro-pneumatic valve system remotely controlled by the operator from the centrifuge control room. A laser sensor measured the water height in the tank. Five LVDT sensors measured soil settlements, three tensiometers, installed on the side of the container, measured the pore water pressure (negative and positive) in the soil. The lateral load was applied at 35 mm from ground level by a hydraulic actuator. The loading phase was displacement-controlled ( $v=0.003$  mm/s at model scale), and a load cell measured the lateral load. One LVDT built in on a rotational joint gave the pile's vertical displacement and rotation.

### 3 CENTRIFUGE TEST PROCEDURE

In total four centrifuge tests were undertaken, varying both the elevation of the water table,  $z_w$ , and the initial void ratio of the soil,  $e_0$  (Table 3). In the two tests where the water table was 70 mm from the ground level the pile was loaded until a lateral displacement,  $y$ , of 30-40% of the diameter,  $D$ , was reached. In the following step, the water table level was raised to ground level keeping the load constant. For the sake of comparison, two additional tests were conducted up to soil failure in fully saturated condition ( $z_w=0$ ) on soil samples prepared at the same voids ratio of the main tests. Measurements of pile displacements, lateral load and bending moments were recorded.

In order to reduce the as compacted suction, the samples were subjected to an imbibition process at 1g, connecting the bottom of the model with the water reservoir.

Before pile loading, equalisation of pore pressures in the soil was allowed. The load was applied with a very slow displacement rate to avoid the creation of excess pore pressures. At the end of each test, undisturbed specimen were sampled to obtain the water content and voids ratio distribution with depth.

**Table 3.** Initial conditions of the centrifuge tests.

Test ID	$e_0$	$z_w$ (mm)	$w_0$	$\sigma_{vc}$ (kPa)	$Sr_0$
T 05	0.93	0	15.03	580	42.02
T 06	0.93	70	14.67	559	42.01
T 08	0.75	70	14.72	1395	51.03
T 09	0.75	0	14.72	1395	51.03

#### 3.1. Model preparation

Samples were prepared by static compaction at two different voids ratios ( $e_0=0.93$  and 0.75) and the same water content ( $w=15\%$ ).

The moisture was prepared following the same procedure used for the flooding tests. After the moisture homogenisation, the soil was then passed in a mechanical mixer to size down the aggregates and stored again in sealed plastic bags.

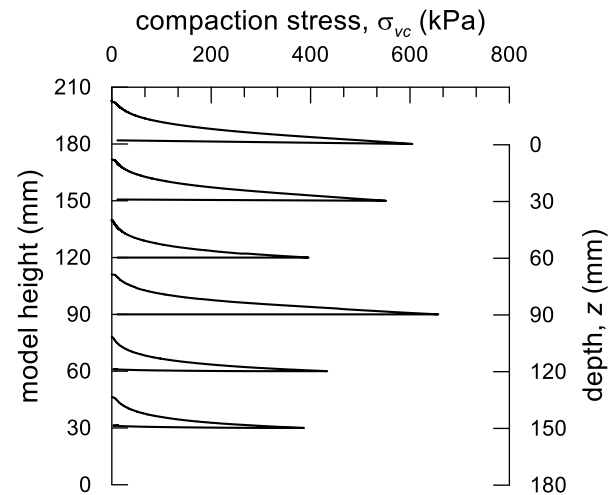
#### 3.2. Compaction

The soil models were statically compacted, in a rigid tub of 300 mm in diameter. An extension is required to compact the soil to the target density. A 10 mm thick sand layer, surrounded by geotextile, allowed the drainage at the bottom of the model. A 2 mm thick plastic sheet coated the inner side to reduce skin friction during model preparation and during the test.

The soil is compacted in six layers of final height 30 mm with a displacement rate of 1.5 mm/min. The compaction is a displacement-controlled process: the press stops when the target displacement was reached. In Figure 6 typical results of the compaction process are presented for one loose soil sample ( $e_0=0.93$ ).

After the compaction of each layer, its surface was scarified to achieve a good connection with the sequent one.

The soil surface was covered with a plastic film to prevent water evaporation in the upper part of the model.



**Figure 6.** Typical results of the compaction process for one of the low density models.

The reasonable homogeneity of the distribution of vertical compaction stresses, Figure 7, and the good repeatability of the results in the different tests confirm the validity of the procedure used. It has to be noticed, however, that the dispersion increases with the soil density. The compaction stress increases with dry density from a mean value of 500 kPa for the loose soil ( $e_0 = 0.93$ ) to 1400 kPa for the dense soil ( $e_0 = 0.75$ ).

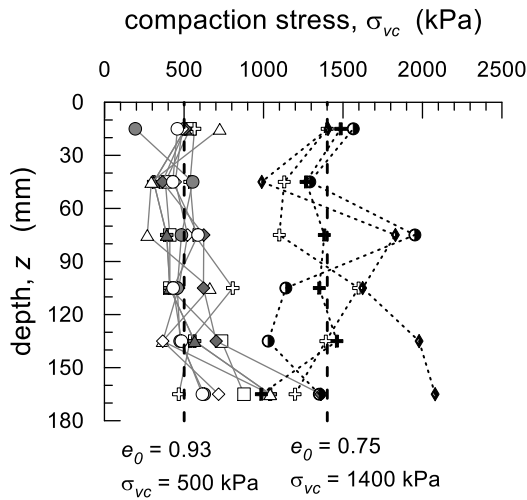


Figure 7. Compaction stress profiles in the tests.

#### 4 ANALYSIS OF THE RESULTS

In this section, selected results are presented to illustrate first the influence of the centrifuge test procedure on the soil behaviour. The complete results are presented in Lalicata et al. (2019) and Lalicata (2018).

##### 4.1. 1g imbibition

The absorbed water volume in this stage is presented in Figure 8a. Figure 8b shows the temporal evolution of the vertical displacements at the soil surface for the two densities considered. The wetting process induces the development of vertical displacements directed upwards attaining a final values of approximately 3 mm for the loose soil and ~6 mm for the dense soil. The water infiltration into the soil layer entails an increase in water volume by around 2 lt in both cases.

The vertical displacements increase as the wetting front advances. Despite the absorbed water volume is approximately coincident in the two cases, the wetting-induced swelling in the dense soil is roughly the double of that found in the loose soil. This outcome reflects the relevant difference in the as-compacted suction and the degree of saturation (see Table 8). Because of the different shape of the SWRC, (see Figure 2), the same approximate variation in  $S_r$  leads to a suction decrease and, thus to a stress unloading, greater in the dense than in the loose soils.

The overall imbibition process induced by capillary rise at 1g with zero pore pressure at base had not still reached stationary conditions, because the main aim of this phase was to increase the suction to speed-up the equilibrium in the following in-flight phase.

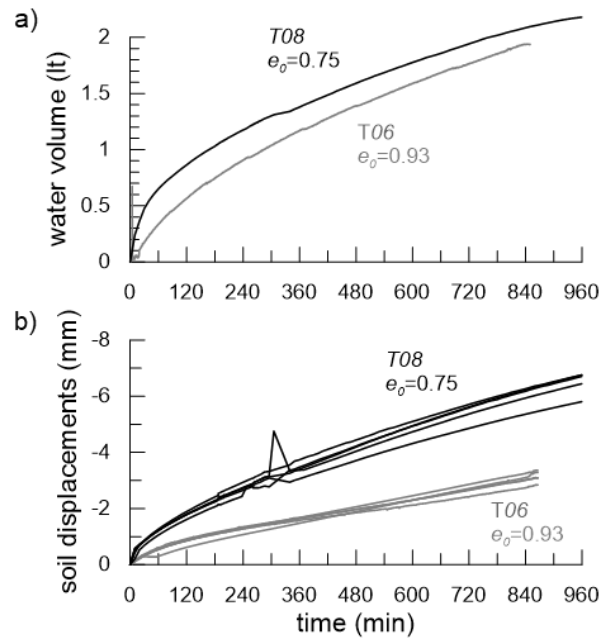


Figure 8. 1g imbibition: a) adsorbed volume, b) vertical displacement of soil.

##### 4.2. In flight phases

Figure 9 shows the temporal evolution of the vertical displacements at the ground surface during the in-flight phase for T06 ( $e_0=0.93$ ) and T08 ( $e_0=0.75$ ). The increase in the gravity loading, together with the hydraulic boundary condition imposed at the base, leads to an instantaneous settlement of about 3 mm and 7 mm for the dense and the loose soil respectively. The vertical displacements increase in time up to 3.5 mm and 10 mm in the two cases.

After the pile loading, the water table was raised up of 70 mm saturating the whole model. The loose sample globally settles as the volumetric collapse of the unsaturated part is greater than the volumetric swelling of the deeper soil layer. On the other hand, the LVDTs measured very little increase in soil volume in the dense sample.

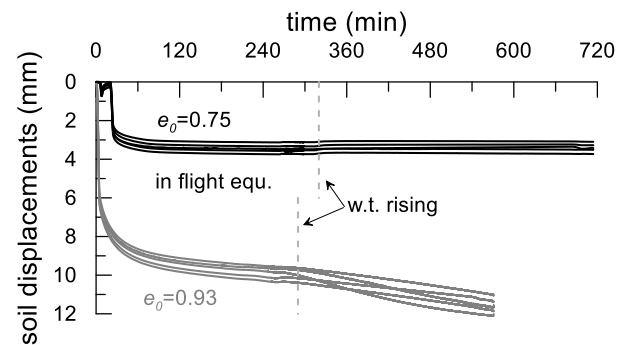


Figure 9. Soil vertical displacements versus time in test T06 ( $e_0=0.93$ ) and T08 ( $e_0=0.75$ ).

The instantaneous soil volume reduction observed as  $g$  increases is possible because the soil is not fully saturated. Indeed, the increment of total stresses is taken all by the effective stresses, as the presence of air in pores allows for a soil skeleton compression. According to the previous imbibition stage, the soil degree of saturation is not uniform in the soil but reduces moving towards the ground surface (far from the bottom boundary condition). Thus, the initial settlement is likely to be related to the compression of the upper part of the model. Conversely, the consolidation settlement has to be attributed to the excess pore pressures dissipation in the lower part of the model.

The pore pressure evolution for the test T06 shown in Figure 10 qualitatively confirms these findings. In fact, in the lower part of the model ( $z=140\text{mm}$ ), the pore pressure rises as  $g$  increases and then reduces with time. In the upper part ( $z=40\text{mm}$ ), it reduces slightly at the beginnings. After four hours of consolidation the trends seem to be stabilised but the absolute values are different from those expected. The tensiometers correctly capture the pore pressure variation during the rising of the water table; they measured approximately +70 kPa at the end of the test. Since the instruments worked well at 1g environment both before and after each test, further investigations are needed to understand the validity of these tools in high gravitational fields.

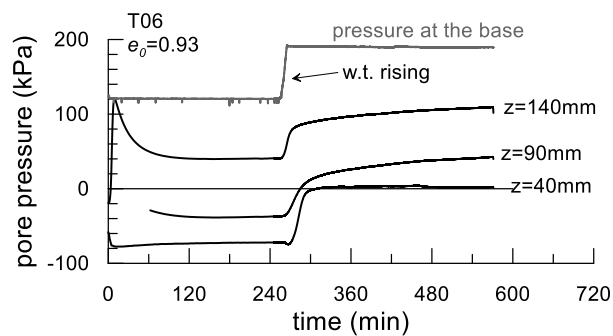


Figure 10. Pore pressures versus time in T06 ( $e_0=0.93$ ).

The pile head displacement and force evolution versus time are presented in Figure 11 a-b for the two soil densities. During the saturation at constant load, the pile head displacements increased more for the sample with  $e_0=0.93$  than the one with  $e_0=0.75$ . In both cases, the inundation event provoked the collapse of the pile. In fact, the displacements keep growing even if the pore pressures tend to the equilibrium (Figure 10).

#### 4.3. End of test measurements

The voids ratio profiles with depth, at the end of the tests, in Figure 12 allows drawing some general

considerations about the soil state at the end of the tests. The reference test on saturated soil conditions for the dense soil, T09 ( $e_0=0.75$ ), shows a profile of voids ratio approximately constant with depth to a value of  $\sim 0.75$ . For the loose soil (T05) the voids ratio decreases with depth from a value of  $\sim 0.9$  at the ground surface to a value of 0.76 for a depth of 170 mm. The latter distribution well fits the oedometric normal consolidation line, NCL, below 40 mm of depth, while the denser ones intercept it at  $z=120$  mm. The models were subjected to the same experimental procedure (1g imbibition, increase of total stress and in-flight equalisation). Thus, these differences are related both to the different initial void ratio and the different shape of the SWRC that controls the variation of mean effective stress and the preconsolidation pressure during hydro-mechanical stress paths.

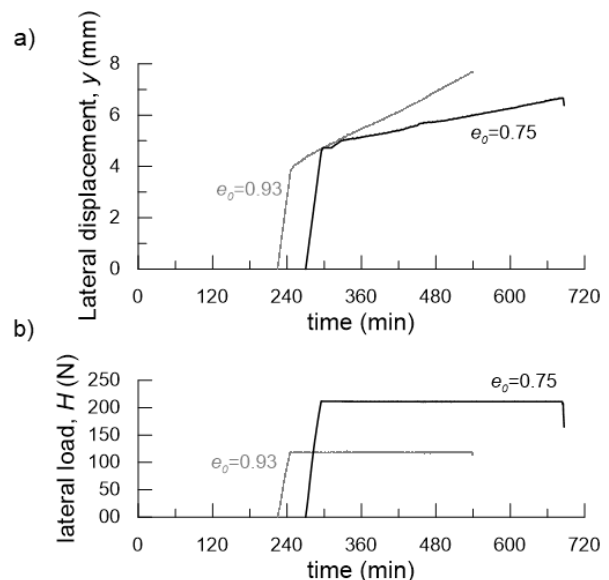


Figure 11. Pile loading and rising of the water table stages: a) lateral displacement versus time, b) lateral load versus time.

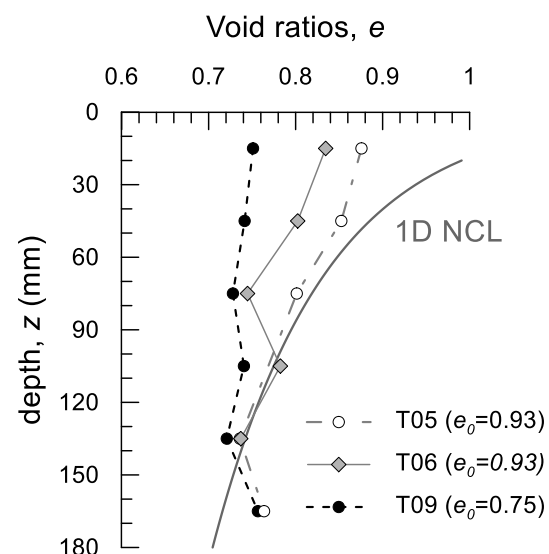


Figure 12. Void ratios profiles with depth at the end of the tests.

In test T06, after the water table rising, below  $z=90$  mm, the voids ratio is the same of T05 (saturated reference test). From 40 to 90 mm of depth the effects of the volumetric collapse upon wetting are very significant while this has a small influence in the shallower part of the model ( $z<40$  mm).

## 5 CONCLUDING REMARKS

The paper presents the experimental details of the centrifuge testing on an unsaturated fine graded soil focusing on the soil behaviour from the model preparation to the end of the test.

The study demonstrate the feasibility of centrifuge modelling in unsaturated soils highlighting the influence of the initial density on the soil response during the flight:

- Low compacted soils experience significant positive deformations during both flight and the rising of the water table.
- Dense soils show a moderate compression during the g-level increase and the application of the hydraulic boundary condition at the bottom. Moderate swelling was observed during the water table rising.

Useful insights on the soil response in flight can be collected from simple flooding oedometer tests. On the other hand, the knowledge of the SWRC, and its dependence from void ratios, is crucial to analyse the hydraulic behaviour of the soil during the centrifuge tests.

The 1g imbibition phase prior testing efficiently reduces the soil suction and speed up the in-flight consolidation time. Some attention is required to not over saturate the material before the centrifuge test beginning.

The tensiometers correctly reproduced the pore pressures variations in-flight but more work must be done to investigate the shift observed from the expected values.

To date, the procedure for the model preparation requires a significant effort from the researcher that limits the applicability to large experimental campaigns. Further reflections on possible improvements of the experimental procedures are needed.

However, the findings of this study encourage the use of centrifuge modelling to investigate the soil structure interaction in unsaturated soils.

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