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Numerical simulation of centrifuge seismic tests on tunnel in sandy soil

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ABSTRACT: Centrifuge seismic tests on small-scale physical models of tunnel in sandy soil are herein reproduced with a numerical calculation to investigate the simulation capability of a numerical model. The soil behavior under monotonic and cyclic loading is simulated fitting laboratory tests with a hypoplastic constitutive model (von Wolffersdorff 1996) coupled to the intergranular strain concept (Niemunis & Herle 1997). Four centrifuge tests have been back-analyzed, reproducing the combinations of tunnel at two depths in two different soil densities. The results from the numerical analyses are compared with those obtained from the tests and with other numerical simulations previously performed in a round robin tests. The simulation proves able to capture observations and to well reproduce ground deformation in all cases. Afterwards, a parametric analysis has been performed modifying the Coulomb friction coefficients at the soil-lining interface to show the relevance of this parameter on the stresses mobilized in the lining.

1 INTRODUCTION

The seismic response of tunnels generally receives less attention compared with other design requirements, possibly due to the belief that underground structures do not experience significant dynamic loading. Tunnels are considered as structures that move together with the shaken ground and thus inertia moment does not provoke high load. This belief turned to be misleading in several cases where damages (Wang et al. 2001) and even collapse (Hashash et al. 2001) were observed after seismic events.

Several methods are currently available in the literature for the seismic design of tunnel lining. A comprehensive review of all methods is given by Pitilakis & Tsinidis (2012) who stated that there is a large uncertainty in the different methods and scatter of results is significant even with the same assumptions. The limited knowledge on the dynamic response of tunnels makes it a difficult and challenging task for designers. The natural consequence is to perform numerical simulations with use of the advanced material models able to catch the dynamic response of soil. Nevertheless, even with the precise calibration of constitutive models based on the sophisticated laboratory tests, the accuracy of prediction still seems to be unsatisfactory (Bilotta et al. 2014).

In order to improve current knowledge on the behaviour of tunnels in sandy soil subjected to seismic events and improve accuracy of prediction, a numerical reproduction of the

centrifuge seismic tests on small-scale physical models (Lanzano et al. 2012) is herein presented. The tests were performed in the framework of a round robin tests where different class A predictions were carried out by several authors (Tsinidis et al. 2014; Hleibieh et al. 2014; Conti et al. 2014; Gomes 2014; Amorosi et al. 2014). The same experiments, together with the laboratory test performed to calibrate the soil, are here resumed to evaluate advantages and drawbacks of another numerical model, not implemented in the round robin test.

2 THE NUMERICAL SIMULATION

2.1 *Centrifuge seismic tests*

Centrifuge seismic tests on small-scale physical models have been performed in 2007 at the Schofield Centre of Cambridge University (UK) by Lanzano et al. (2012) within the framework of the ReLUIS Project (2005–2009). Based on those tests, a numerical Round Robin Tunnel Test (RRTT) was launched among researchers in the geotechnical community. All input data were known except the results of the centrifuge tests. In this paper two sets of laboratory tests are referred on dense and loose sand respectively denoted as T3 and T4. The geometrical layout of tests and instrumentation is presented in Figure 1.

Both tests were performed using same dry Leighton Buzzard sand characterized by triaxial, oedometer and resonant column-torsional shear tests (Lanzano et al. 2016). State properties of the Leighton Buzzard sand are given in Table 1. The material was pluviated into the laminar box obtaining two different relative densities Dr (50 and 80%). The tunnel lining was made from an aluminum-alloy tube with an external diameter of 75 mm and a thickness of 0.5 mm. The depth of the tunnel was fixed in both tests at 187.5 mm below the top surface.

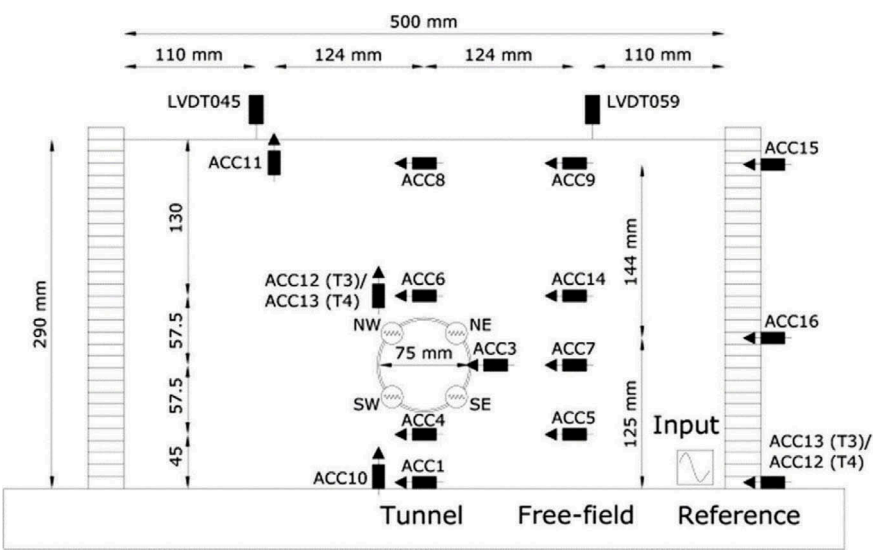

Figure 1. The layout of the T3 and T4 centrifuge test (Bilotta et al. 2014)

Table 1. State parameters of the Leighton Buzzard sand (Lanzano et al. 2016)

	G_s	e_{max}	e_{min}	d_{50}	d_{10}	d_{60}/d_{10}
Soil	-	-	-	μm	μm	-
Leighton Buzzard sand – fraction E	2.65	1.014	0.613	140	95	1.58

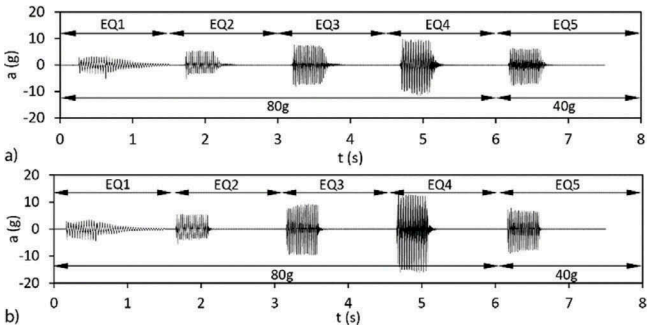

Figure 2. Acceleration time histories in model T3 (a) and T4 (b) (Lanzano et al. 2012)

The seismic loading at each test consists of four shaking sequences (EQ1-EQ4) at 80 g condition and one (EQ5) at reduced acceleration to 40 g. Examples of seismic signals are given in Figure 2.a for the T3 model and Figure 2.b for the T4 model.

Two linear variable differential transducers (LVDT) were used to record vertical displacements of the model top surface. Furthermore, a number of piezoelectric accelerometers were used to measure horizontal and vertical accelerations in different locations of the model. The model tube was equipped with four strain gauges to derive bending moments and hoop forces.

2.2 *Computational model*

The finite element code ABAQUS^{TM} (Hibbitt & Sorensen 2001) has been adopted to perform 2D plane-strain numerical simulations. The finite element mesh for the soil consists of about 3200 4-node quadratic elements, while for the lining 60 beam elements are used. A further reduction of the finite elements size resulted with a significant increase in calculation time with no effect on the obtained results. Therefore, the model discretization adopted in the analysis is presented in Figure 3 together with the boundary conditions.

The base boundary of the model is simulated as a bedrock thus all nodes along that surface are fixed. For the vertical boundaries, the kinematic coupling has been applied between two nodes at the same level of opposite sides, simulating the simplified condition of the laminar box.

The soil-lining interface is simulated using contact model available in the adopted code. The behaviour in the normal direction is modelled as an exponential pressure-overclosure relationship with prevention from surfaces separation. Contact pressure is transmitted between surfaces when the clearance between them has reached a threshold value; then the pressure

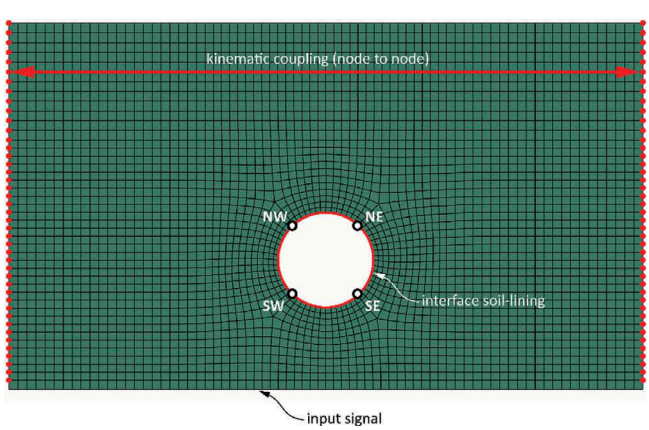

Figure 3. Geometry and boundary conditions of the numerical model

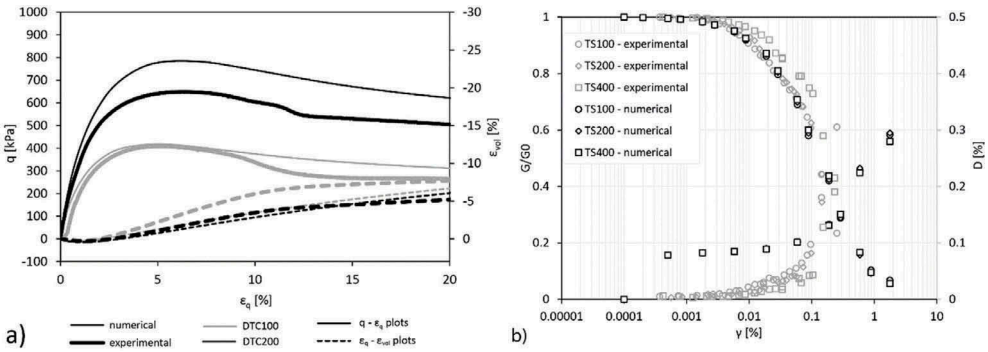

Figure 4. Numerical simulation of laboratory tests for: a) large-strain and b) small-strain range

Table 2. Parameters of the von Wolffersdorff hypoplastic model with the ISC for Leighton Buzzard sand

φ_c °	h_s MPa	N -	e_{d0} -	e_{c0} -	e_{i0} -	α -	β -	R -	β_r -	X -	m_R -	m_T -
31.79	600	0.47	0.504	1.01	1.21	0.2	0.8	$1e^{-4}$	0.07	1	6	3

increases exponentially as the clearance fades out. In the tangential direction, contact has been described using the basic Coulomb friction model with the friction coefficient μ=0.29 which corresponds to half of the soil friction angle ($0.5 \cdot \varphi_c = 15.9°$).

The analysis is performed in four steps. In the first step, the initial stress state is obtained assuming gravity load equal to 1g, while in the second one gravity is increased from 1 g to 80 g as in the experimental test. In the following step, seismic input (EQ1-EQ4) is applied at the bottom boundary of the model with the time history obtained from the experiment (Figure 2). Finally, gravity loading is decreased from 80 g to 40 g and the last shaking cycle is applied (EQ5).

Calculations have been performed on one computational node of the Polish Grid Infrastructure (PL-GRID) equipped with two Intel® Xeon® X5670 processors (48 threads) and 72 GB of memory and even though it took around 72 hours for each of them.

The nonlinear mechanical behaviour of soil during shaking was modelled using von Wolffersdorff (1996) hypoplastic model with an extension called Intergranular Strain Concept (ISC) introduced by Niemunis & Herle (1997). Von Wolffersdorff model is an advanced rate-independent constitutive model including irreversible non-linear stress-strain response with stiffness anisotropy, while ISC extension gives the ability to include past strain history and effects related with the cyclic loading. The calibrated response of the model in the large-strain amplitude is presented in Figure 4a, while in the small-strain range in Figure 4b. The parameters of the calibrated model are shown in Table 2.

3 RESULTS

Vertical settlements, bending moments and hoop forces obtained from numerical simulations during all 5 shaking sequences (from EQ1 to EQ5) are compared with the measurements taken during experimental tests and with the results from other numerical simulations within the RRTT (Bilotta et al. 2014). The features of five numerical predictions of RRTT are summarized in Table 3.

The vertical settlements measured at the location of LVDT059 for the T3 and T4 models are compared with their simulation in Figure 5. Here the results of simulation obtained by various authors in the round robin are reported for reference. All curves show a progressive increase of settlements during each shaking induced by accumulation of plastic volumetric strain. This trend is more and less captured by all numerical simulations, although there is a

Table 3. Main features of the numerical predictions with RRTT (Bilotta et al. 2014)

Group	Adopted constitutive model	Numerical code
AUTH	Visco-elastic-plastic model	ABAQUS
TUD	Hypoplastic	TECHNOG
TVG	M1 -bounding Surface plasticity M2 – perfect plasticity with embedded hysteretic behaviour	FLAC
UTL	Elastoplastic multimechanism	GEFDYN
BaBo	'Small-strain' elasto-plastic hardening soil model	PLAXIS

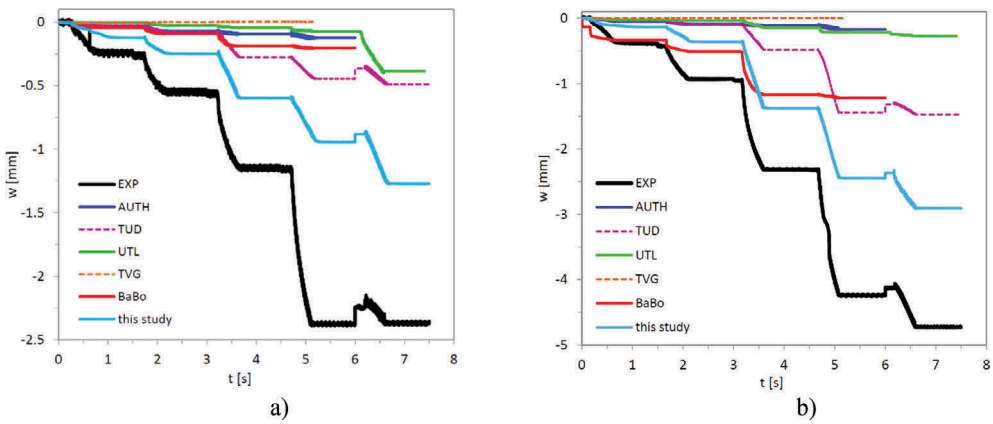

Figure 5. Comparison of the vertical displacement time histories between centrifuge tests (LVDT059) and various numerical simulations for T3 (a) and T4 (b) models.

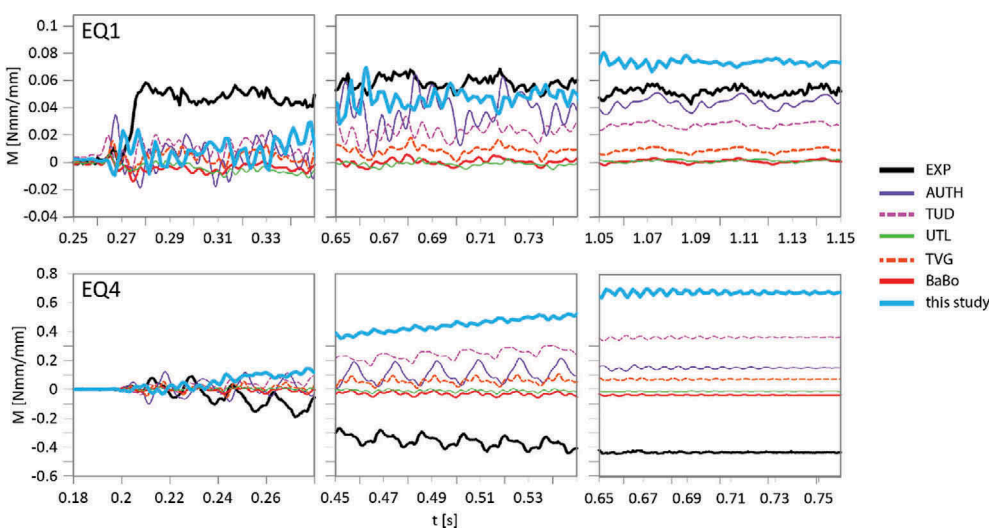

Figure 6. Comparison between time histories of bending moments (NW) obtained from experiments and numerical simulations for the EQ1 and EQ4 shaking sequence for the T3 model (dense sand).

significant variability and all of them underestimate the measured settlements. The solution obtained from the presented simulation depicted with the light blue color shows some improvement comparing to other ones. The final settlement from the numerical simulation is equal to about 50% of the measured ones for both T3 and T4 models.

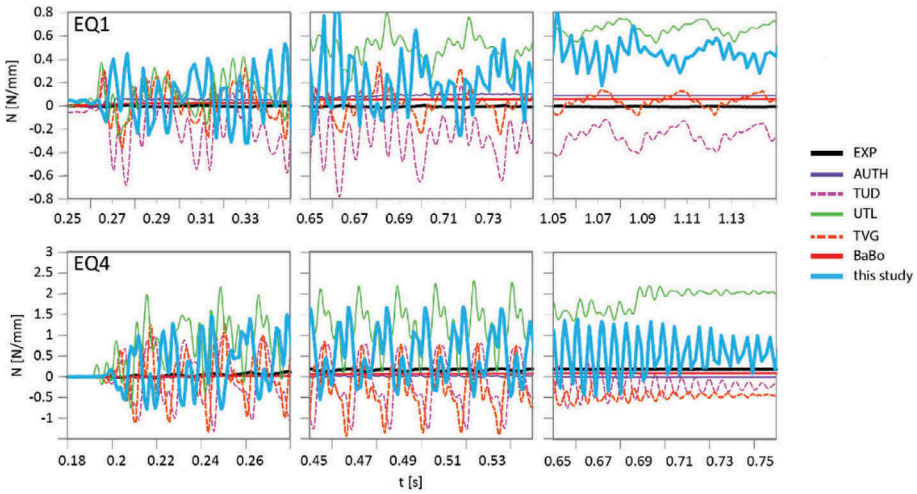

Figure 7. Comparison between time histories of hoop forces (NE) obtained from experiments and numerical simulations for the EQ1 and EQ4 shaking sequence for the T3 model (dense sand).

The time history of the bending moments at the NW measuring point (see Fig.3) and hoop forces at the NE measuring point are shown for the weakest (EQ1) and strongest (EQ4) seismic events. The recorded time histories are given for both tests, respectively for the T3 model in Figures 6 and 7, and for the T4 model in Figures 8 and 9. Those results are plotted in three time ranges representing the initial, central and final part of the signal, thus giving the representative comparison. During both seismic events (EQ1 and EQ4) there is a significant change in bending moments and hoop forces. For the EQ1 event obtained results shows progressive accumulation of bending moments from the beginning of the event with the maximum value similar to the measured one. For the EQ4 event numerical calculation shows progressive accumulation of bending moments but with the opposite sign. The experimental time histories show always negative values, while all the numerically obtained curves show positive results. However, the absolute values computed in the present analysis are similar in both cases to the experiments with maximum values equal to around 0.6 Nmm/

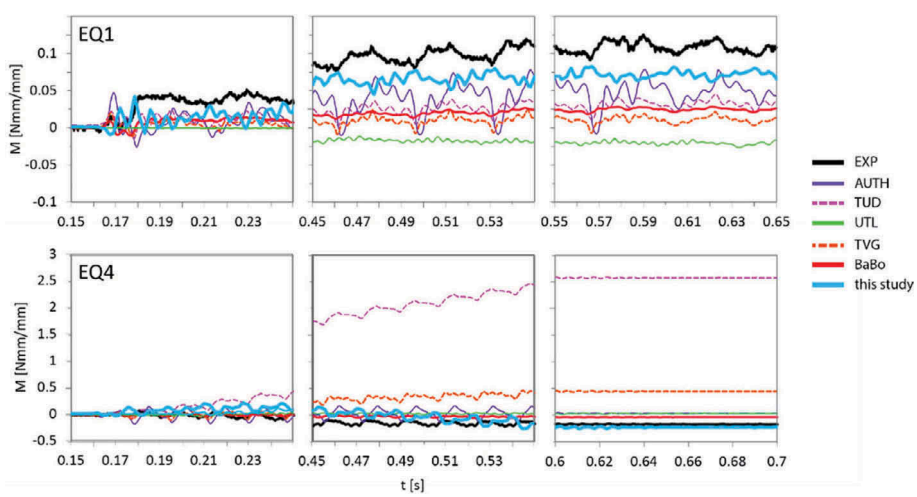

Figure 8. Comparison between time histories of bending moments (NW) obtained from experiments and numerical simulations for the EQ1 and EQ4 shaking sequence for the T4 model (loose sand).

mm. On the contrary, the normal forces depicted in Figure 7 are much higher than those experimentally recorded. The comparison with the other numerical simulation carried out in the round robin tests shows a large scattering of results.

Time history of the bending moments for the T4 model depicted in Figure 8 also shows a similar response to the observed one. Initially, during the EQ1 event, there is a progressive accumulation of bending moments with the maximum value reaching 70% of the observed one. On the other hand, bending moments accumulated during the EQ4 event are satisfyingly well reproduced by the numerical simulation. Normal forces depicted in Figure 9 are much higher than those experimentally recorded similarly as most of the other numerical simulations of the RRTT.

4 INFLUENCE OF THE SOIL-LINING INTERFACE

In order to investigate the influence of the soil-lining interface three values of the Coulomb friction coefficient μ has been considered for the T3 model, 0.0, 0.29 and 1.0 respectively for full-slip, frictional and no-slip condition.

The comparison of vertical displacement time histories between centrifuge tests (LVDT059) and numerical simulations for different value of μ coefficient is depicted in Figure 10. There is a clear evidence that the friction at the soil-lining contact has a negligible influence on settlements as only a very small difference is observed among results.

The structural forces in the lining obtained from numerical simulations with different friction coefficients are compared only for the EQ1 event and depicted in Figure 11. The difference in the accumulation of bending moments is significantly affected by friction coefficient and for the full-slip condition, value is maximum for whole time history. On the other hand, for the no-slip condition bending moments have a negative value indicating that there is a change in the lining deformation mechanism. Hoop forces are also affected by the friction coefficient of the soil-lining interface however, there is no clear trend as for the intermediate value μ=0.29 hoop force is maximum and with for two other values reaches the minimum (opposite sign) with the similar absolute value.

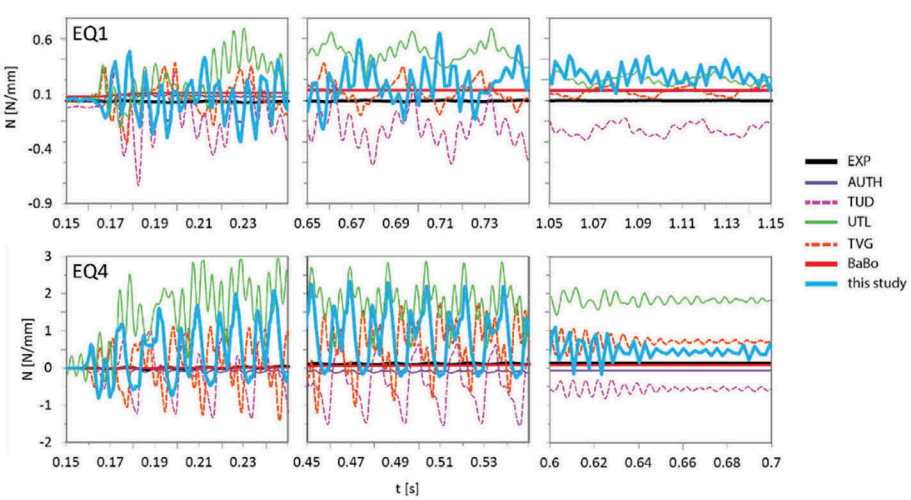

Figure 9. Comparison between time histories of hoop forces (NE) obtained from experiments and numerical simulations for the EQ1 and EQ4 shaking sequence for the T4 model (loose sand).

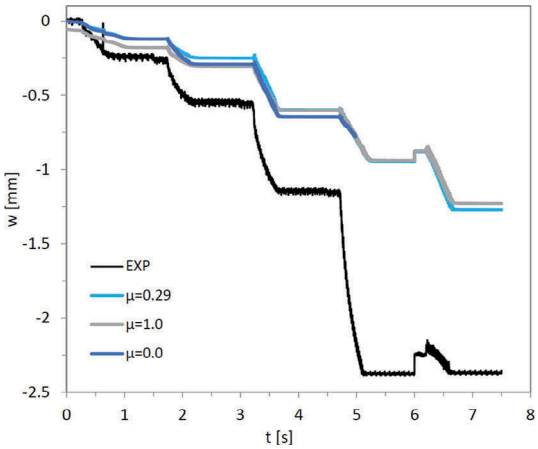

Figure 10. Comparison of vertical displacement time histories between centrifuge tests (LVDT059) and numerical simulations for T3 model with different friction coefficients.

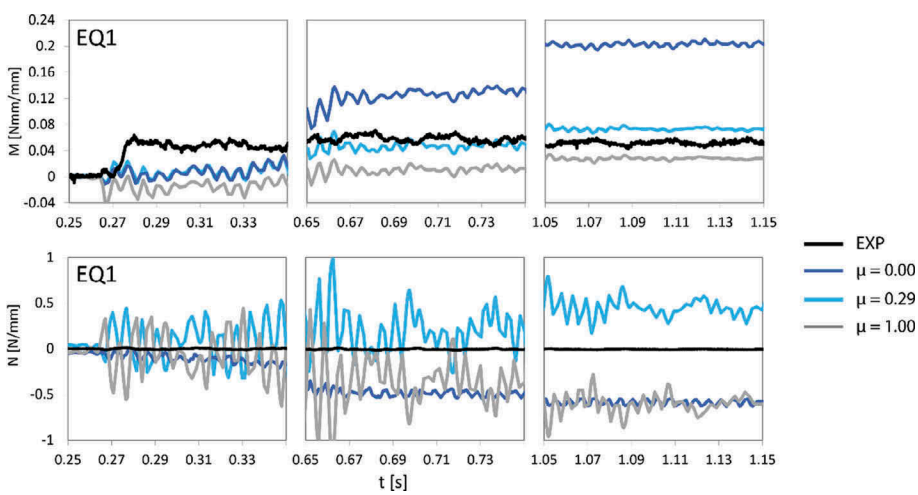

Figure 11. Comparison between time histories of bending moments (NW) and hoop forces (NE) obtained from numerical simulations for the EQ1 shaking sequence for the T3 model with different friction coefficients.

5 CONCLUSIONS

The presented study has shown that response of the tunnel in dry sand under cyclic seismic loading can be qualitatively and in some part quantitatively predicted by the numerical analysis with use of the hypoplastic constitutive model. It is confirmed that irreversible increments of internal forces are found in the tunnel lining after shaking. They stem from cumulated plastic strains, that produce sand densification and ground level settlement. The potential to capture such behaviour depends on the ability of the adopted constitutive model to predict plastic volumetric strain associated to shaking. In comparison to the previous numerical calculation in RRTT that made use of the same hypoplastic model, although in a different FE code (see Table 3), in this work a better prediction of the experimental results was achieved by refining the calibration of the model parameters. Hence this study further confirms that the improved

simulation of centrifuge tests stems from the higher ability of the adopted hypoplastic model to capture plastic volumetric strain associated with shear in sand, compared to other models also used in RRTT.

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