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# Outcomes of RRTT, a predictive exercise on the behavior of tunnels under seismic actions

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ABSTRACT: This paper outlines the main outcomes of RRTT, a Round Robin numerical prediction exercise of centrifuge Tests on Tunnel models subjected to seismic loading, which was launched in Rome at the 7th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground in 2011, and jointly promoted by TC204 together with TC104 and TC203 of ISSMGE.

## 1 INTRODUCTION

The seismic behavior of urban tunnels can be predicted by simplified pseudo-static approaches and dynamic methods of analysis, i.e. procedures that take into account the dynamic nature of the seismic loads and the cyclic soil behavior. These latter methods can either uncouple the analysis of free-field soil response from that of the tunnel ('simplified dynamic analysis'), or use more complex procedures ('full dynamic analysis') accounting for soil-structure interaction (e.g. Hashash et al. 2001).

The calibration of all such methods should require validation against experimental data, which are seldom available at the prototype scale. Centrifuge modelling is definitely an alternative powerful tool to produce 'artificial case histories' for calibration, back-analysis or benchmarking among different analytical approaches. Use of centrifuge modelling to investigate the seismic behaviour of tunnels was first carried out at Cambridge University (Cilingir & Madabhushi, 2011).

On the other hand, the evolution of internal forces in the lining with time is crucial for the engineering assessment of the seismic performance of a tunnel.

If suitable constitutive laws for the soils are well-calibrated on laboratory and field tests and the most appropriate geometrical and physical description of the boundary value problem is performed, finite element or finite difference methods analyses can provide a reliable evaluation of both free-field and soil-tunnel dynamic response.

In the framework of Italian ReLUIS-DPC Project, dynamic centrifuge tests were carried out at Cambridge on tunnel models in sand (Lanzano et al. 2012). After the end of the research project, the experimental data have been made available online to the scientific community, to be used for benchmarking simplified to complex dynamic numerical methods. The paper describes, compares and discusses the main outcomes of such a predictive exercise, as developed by five European research groups, operating with different advanced constitutive and numerical models. The detailed results of each group were individually reported in specific papers collected in a Special Issue of the Journal Acta Geotechnica. The comparisons summarized in this conference paper highlight that, overall, all the models could well capture the kinematic characteristics (i.e. acceleration and shear strain) of soil dynamic behavior; the predictions of internal tunnel lining forces (hoop forces and bending moments) were instead much more variable, being more significantly affected by the assumptions on the soil-lining interface.

## 2 NUMERICAL MODELLING

### 2.1 *Experimental benchmark*

The details on the centrifuge benchmark tests have been already presented in Bilotta & Silvestri (2012, 2013) and in details in Lanzano et al. (2010, 2012), therefore they will not be shown hereafter.

The models T3 and T4 were made of dry Leighton Buzzard Sand fraction E ($D_r \cong 75\%$ and 40% respectively). Figure 1 shows the layout of the two models with the measuring devices deployed in the sand layer: accelerometers (ACC) were used to measure horizontal and vertical component of acceleration time histories; LVDTs were used to measure surface settlements. In addition the aluminum lining of the model tunnel was instrumented with strain gauges at four different locations to measure bending moment and

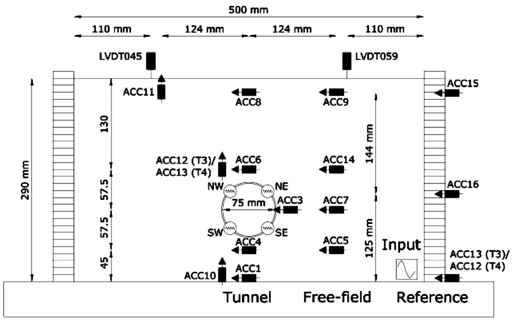


Figure 1. Model layout with instrumentation.

Table 1. Earthquakes fired in test T3.

| earthquake # | g-level | main frequency (Hz) | duration (s) | nominal PGA (g) |
|---|---|---|---|---|
| 1 | 80 g | 30 [0.375] | 0.4 [32] | 4 [0.05] |
| 2 | 80 g | 40 [0.5] | 0.4 [32] | 8 [0.10] |
| 3 | 80 g | 50 [0.625] | 0.4 [32] | 9.6 [0.12] |
| 4 | 80 g | 60 [0.75] | 0.4 [32] | 12 [0.15] |
| 5 | 40 g | 50 [1.25] | 0.4 [16] | 6 [0.15] |

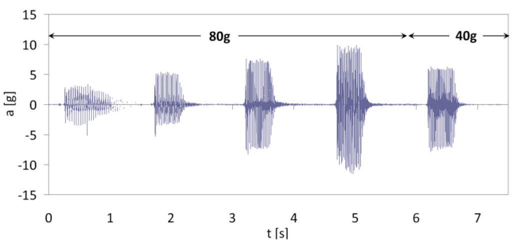


Figure 2. Typical input signals (T3).

hoop force in the transverse section (NE, NW, SE and SW).

The main characteristics of the input signals applied to the models are shown in Table 1 (bracketed figures at equivalent prototype scale) and Figure 2.

### 2.2 *Modeling strategies*

The five papers to which this paper refers show the predictions of teams belonging to academic Departments of several European countries. Each group adopted a different numerical code and a different constitutive model for the soil, as shown in Table 2.

A significant issue for all the participants was the calibration of the constitutive model on the results of both laboratory and centrifuge tests. Since the results of an extensive campaign on the sand were available to the participants (Visone & Santucci de Magistris, 2009), most groups initially calibrated their models on laboratory tests only and performed 'blind' predictions of dynamic behavior observed in the centrifuge models.

Table 2. Main features of RRTT numerical analyses.

| Team | Reference paper | Adopted constitutive law | Numerical code |
|---|---|---|---|
| AUTH | Tsinidis et al. (2014) | Visco-elasto-plasticity | ABAQUS (*FEM*) |
| TUD | Hleibieh et al. (2014) | Hypoplasticity | TOCHNOG (*FEM*) |
| TVG | Conti et al. (2014) | EPP with embedded hysteretic behaviour | FLAC (*FDM*) |
| UTL | Gomes (2014) | Elastoplastic multi-mechanism | GEFDYN (*FEM*) |
| BaBo | Amorosi et al. (2014) | "Small strain" elasto-plastic hardening soil model | PLAXIS (*FEM*) |

However, the direct interpretation of the centrifuge tests, as a back-analysis of a non-linear boundary value problem, incorporates in the calibration the detail of the actual history and stress paths, leading to a more reliable estimate of the geotechnical behavior (*cf.* Lanzano et al., 2014). Therefore most groups repeated the analyses by tuning the calibration of the constitutive models on the basis of a back-analysis of the centrifuge tests.

The numerical analyses were hence performed by the different groups with different strategies, which justify the differences observed among the results. First, rather different constitutive laws were adopted (see Table 2) modeling in different ways the transition from linear/reversible to non-linear/irreversible behavior of sand. In particular each model has different ability to model plastic accumulation of both shear and volumetric strain. Second, the selection of the initial stiffness profile in the sand layer was not univocal. Finally, the assumptions on the contact between the sand and the tunnel lining were different.

## 3 MAIN RESULTS

A short selection of the results achieved by the numerical analyses are summarized in this section and compared to the experimental benchmark.

### 3.1 *Accelerations*

A comparison between the experimental time-history of acceleration (black line) measured at the top of the reference array (ACC15 in Figure 1) and the corresponding calculated time histories (colored lines) is shown for model T3 and the strongest event EQ4, as an example, in Figure 3.

In general the predictions of four out of five teams are satisfactory, since the main features of the experimental record are caught, both in terms of amplitude and frequency content (Figure 4).

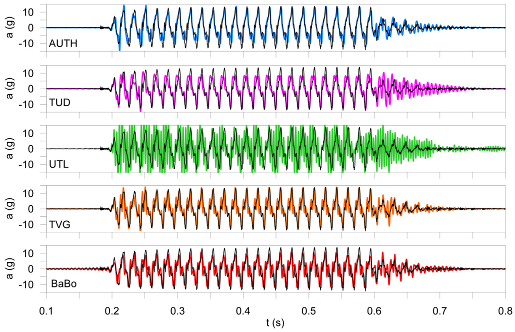


Figure 3. Acceleration time histories (T3, EQ4).

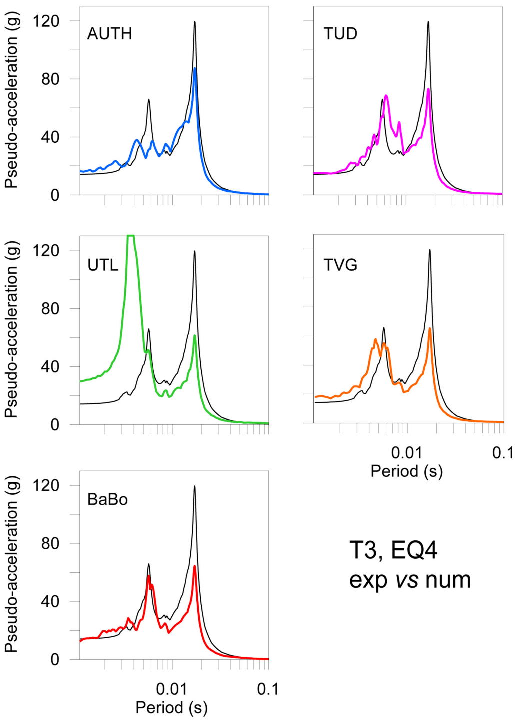


Figure 4. Response spectra at surface (T3, EQ4).

Only one numerical prediction (UTL) shows an unexpected amplification of the signal at high frequencies after a few cycles, particularly in the weakest event (EQ1). This team actually did not model the initial non-null damping ratio $D_0$, neither filtered out preliminary from the input the frequency content higher than 180 Hz as BaBo did, for instance, preventing over-amplification at high frequency.

Profiles of $a_{max}$ with depth in T3 during EQ4 are shown in the plots of Figure 5, as measured and calculated in free field conditions and along the tunnel array. For the weaker earthquakes the dispersion among the numerical predictions is lower.

In spite of the differences among the models, the dynamic response in terms of acceleration of the sand layer was well captured in most cases.

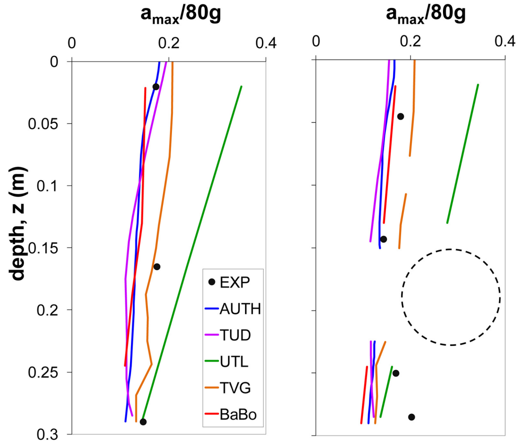


Figure 5. Comparison between the experimental and numerical profiles of peak acceleration (T3, EQ4).

On the other hand, a deeper insight into the results highlights the influence on the results of the way the small strain damping is accounted for. Since all the adopted constitutive models dissipate energy by reproducing the hysteretic behavior only, a small Rayleigh damping is needed to smooth the over-amplified signal at higher frequency (Bilotta et al., 2014).

### 3.2 *Shear strains*

The experimental time histories of the displacements, u(t), were obtained from double integration of the accelerograms (Brennan et al., 2005). Hence the experimental time histories of shear strain have been calculated by differentiating such displacements, u(t), with respect to depth, z, using a second order approximation over three instruments positioned in the same vertical.

Profiles of the maximum measured and calculated values of free-field shear strain, $\gamma_{max}$, with depth in T3 are shown for EQ4 in the plots of Figure 6.

AUTH was able to match the average shear strain mobilized in the layer by reducing the initial shear stiffness of sand in the model layer compared to the element tests. Lanzano et al. (2014) have actually shown that the values of small strain shear stiffness in the centrifuge models were on average lower than those measured in the laboratory element tests.

BaBo over-predicted shear strain, although they assumed a $G_0$ profile very close to that from the laboratory element tests. This group was the only predicting, quite singularly, shear strains decreasing with depth.

All the other teams generally underestimated the shear strain, predicting values of about 20% to 30% of the experimental amplitudes.

### 3.3 *Surface settlements*

Measured and calculated time histories of surface settlement are shown in Figure 7 for all the earthquakes applied to model T3. The measured settlement (black

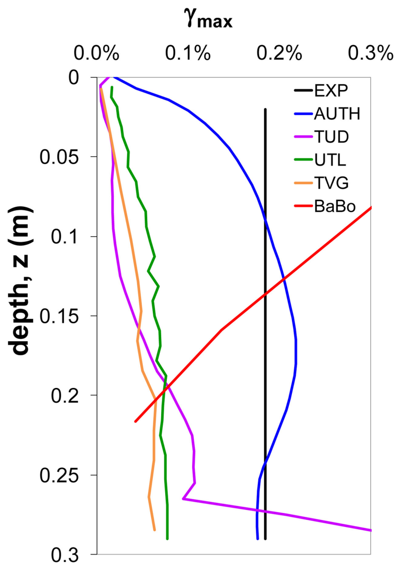


Figure 6. Comparison between the experimental and numerical profiles of maximum shear strain (T3, EQ4).

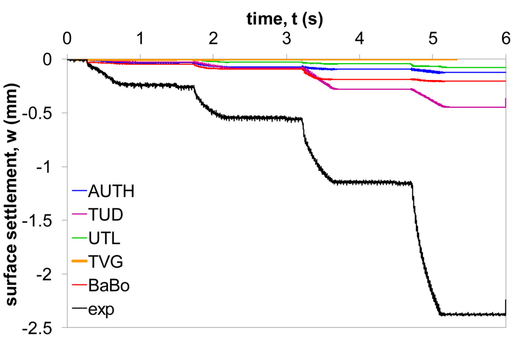


Figure 7. Comparison between the experimental and numerical time histories of settlement (T3).

line) cumulates during the events, indicating the densification of the sand layer due to the accumulation of plastic volumetric strain during shaking.

The amount of predicted densification is extremely variable among the different numerical models, indicating their different ability to simulate the volumetric plastic straining induced by cyclic shear loads.

Although the mobilized shear strain in the experiments was high enough to produce irreversible volumetric change, either the volumetric threshold predicted by the constitutive models was too high or the calculated shear strain was too lower than the experimental one. As a matter of fact the accumulation of surface settlements was largely under-predicted although using suitable constitutive laws.

In addition, local non-homogeneities of the sand density may be responsible of larger permanent changes of measured volume strains compared to the computed ones.

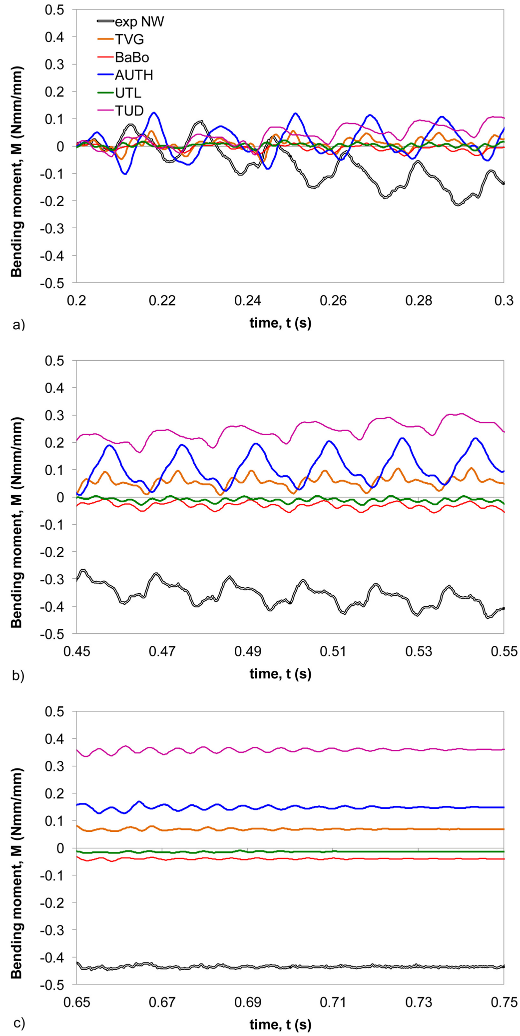


Figure 8. Model T3 (dense): comparison between the experimental (NW) and numerical time-history of bending moments during EQ4: a) initial, b) central, c) final window.

## 3.4 *Internal forces*

A representative selection of time histories of bending moment and hoop force is plotted in Figure 8 and 9. The experimental data in Figure 8 (black line) refer to the NW transducer, shown in Figure 1. Since all the numerical analyses were performed by assuming a positive acceleration at the base opposite to the test convention, in the following figures the calculation at the mirror point (symmetrical with respect to the vertical axis of the tunnel) was used for comparison. For the sake of representation only three significant portions of the time histories are plotted.

The comparisons show a trend to increase during shaking, which is generally larger for the stronger events. In addition reversible change of bending moment can be observed, which is associated to the cyclic loading.

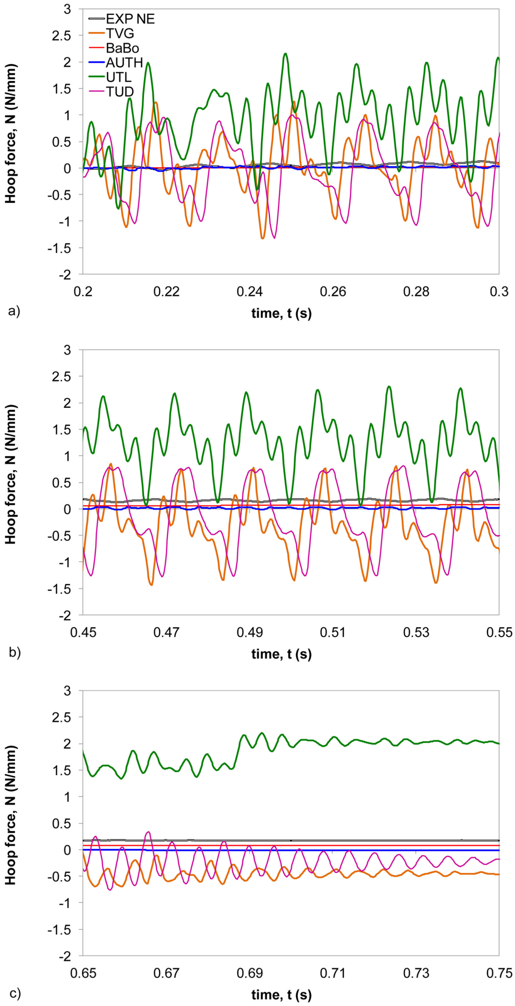


Figure 9. Model T3 (dense): comparison between the experimental (NE) and numerical time-history of hoop forces during EQ4: a) initial, b) central, c) final window.

The models have different ability to reproduce the permanent accumulation of bending moment at the end of the event, which seems to correlate well with the ability to predict sand densification.

Reversible and permanent changes of hoop forces with different magnitude can be observed in the experimental data and in the relevant numerical predictions (Figure 9).

The value of the hoop force is particularly influenced by the roughness of the interface between the aluminum lining and the sand, this contact being very smooth. Therefore different models predict very different hoop forces.

The prediction of plastic soil strain is crucial from the point of view of the correct assessment of post earthquake irreversible overload on a tunnel lining. All the analyses showed that yielding near the tunnel may

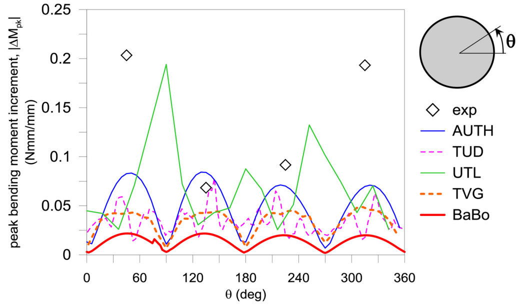


Figure 10. Model T3 (dense): experimental and numerical distribution of reversible increments of moments during EQ4.

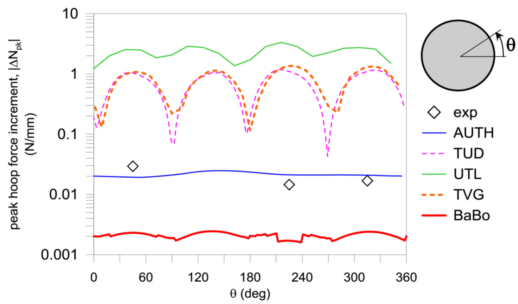


Figure 11. Model T3 (dense): experimental and numerical distribution of reversible increments of hoop forces during EQ4.

result in stress redistribution which affects the static performance of the tunnel lining.

A comparison between the experimental and numerical distribution of reversible increments of bending moments in T3 during EQ4 is shown in Figure 10.

All the numerical predictions underestimate the experimental measurements, being however quite different each other.

The different prediction of shear strain should in part justify the different assessment of peak increments of bending moment: the larger the average shear strain, the larger such increments.

In Figure 11 the similar comparison between the experimental and numerical distribution of reversible increments of hoop forces is shown, in a logarithmic scale, due to the large difference among the numerical predictions. Two groups (AUTH and BaBo) modeled a very smooth contact between the sand and the aluminum tube. In this way they matched better the experimental measurements. The way a frictionless contact is modeled by different numerical codes largely affect the calculated hoop forces for very thin lining such as the one at hand (Bilotta et al. 2009).

## 4 CONCLUSIONS

This paper summarized and discussed the results of several sets of numerical analyses which were collected from five published journal papers. These calculation had been performed with different constitutive models and numerical codes in the framework of a Round Robin numerical prediction exercise. The experimental benchmark was a set of centrifuge models of tunnels under seismic loading in sand.

Only a selection of the whole set of numerical results has been shown in this paper, being a more detailed presentation available in a journal paper published in a special issue containing all the original papers from RRTT (Bilotta et al, 2014).

The analysis of the results leads to make the following remarks:

- the amplification of ground acceleration is relatively well matched by very different constitutive models, if the correct level of shear stiffness and damping ratio is mobilized;
- the calculation of shear strain is instead pretty much affected by the choice of the constitutive behavior;
- reversible changes of internal forces during shaking seem to depend on the cyclic changes of shear stresses around the tunnel lining and not by the actual shear strain distribution in the ground; therefore they are essentially driven by the inertial forces which develop in the soil mass;
- irreversible load increments arise on the tunnel lining after shaking as a consequence of cumulated plastic strain in the ground (Lanzano & Bilotta, 2014);
- plastic volume strains in the soil (densification) may induce a significant change of bending moments and hoop forces in the lining after earthquake; plastic shear strains have a minor influence;
- observed sand densification is better reproduced with models which associate to shear a reasonable amount of permanent volume changes;
- hoop forces (both reversible and irreversible changes) show a major dependence on the assumption about the contact between the lining and the sand.

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