Seismic analyses of shallow tunnels by dynamic centrifuge tests and finite elements

Analyses sismiques des tunnels à faible profondeur par essais dynamique en centrifugeuse et éléments finis

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ABSTRACT

The increments of internal forces induced in a tunnel lining during earthquakes can be assessed with several procedures at different levels of complexity. However, the substantial lack of well-documented case histories still represents a difficulty in order to validate any of the methods proposed in literature. To bridge this gap, centrifuge model tests were carried out on a circular aluminium tunnel located at two different depths in dense and loose dry sand. Each model has been instrumented for measuring soil motion and internal loads in the lining and tested under several dynamic input signals. The tests performed represented an experimental benchmark to calibrate dynamic analyses with different approaches to account for soil-tunnel kinematic interaction.

RÉSUMÉ

Les variations de forces internes induites dans le revêtement d'un tunnel au cours de tremblements de terre peuvent être évaluées de plusieurs procédures à différents niveaux de complexité. De toute façon, le manque des cas bien documentés représente toujours une difficulté, afin de valider l'une des méthodes proposées dans la littérature. Pour combler cette lacune, essais en centrifugeuses ont été effectués sur des modèles d'un tunnel circulaire d'aluminium situé à deux profondeurs différentes dans sable sec à deux densités différentes. Chaque modèle a été instrumenté pour mesurer les mouvements du sol et le forces internes au revêtement et testé avec plusieurs signaux dynamiques d'entrée. Les essais effectués ont représenté une référence expérimentale pour étalonner les analyses dynamique avec approches différentes pour la pris en compte de l'interaction cinématique entre le revêtement et le sol.

Keywords : tunnel, centrifuge, earthquake

1 INTRODUCTION

Most of the earthquake damages observed on shallow tunnels in soft ground can be ascribed to the dynamic soil straining around the underground structure, rather than to the inertial interaction between soil and tunnel lining. Accordingly, the seismic design of tunnels should be based on accurate predictions of the soil-tunnel kinematic interaction (e.g. Paolucci & Pitilakis 2007; Bilotta et al. 2008).

Pseudo-static and simplified dynamic analyses with uncoupled approaches are suggested by State-of-the-art papers (e.g. Hashash et al. 2001) and guidelines (ISO TC98/SC3 N229 2003) for preliminary and intermediate design stages. Full dynamic analyses including soil-structure interaction are recommended for the final stages of the design.

In the uncoupled approaches, the kinematic interaction between the ground and the lining is usually neglected (e.g. Bilotta et al. 2007). The seismic increments of bending moment, M, and hoop force, N, along the transverse section of the lining are then computed using an average free-field shear strain and the relative tunnel/ground elastic stiffness (e.g. Wang 1993).

On the other hand, full dynamic analyses (Corigliano 2007; Amorosi & Boldini 2007; Lanzano et al. 2009) highlight the strong influence on the predictions of kinematic interaction and soil non-linear behaviour; such factors are not adequately considered in the uncoupled approaches. Such issues and the substantial lack of well-documented full-scale case histories still represent a difficulty in order to validate any of the proposed methods. To bridge this gap, centrifuge seismic tests on a model tunnel were carried out at the Schofield Centre of Cambridge University (UK) in the framework of an Italian research project (ReLUIS). The final aim was to calibrate and compare numerical predictions of different complexity on the basis of reliable experimental data. This paper illustrates an example of the use of the experimental results for the calibration of simplified and full dynamic analyses.

2 CENTRIFUGE TESTS

2.1 *Tunnel models*

The experimental activity was carried out at the Schofield Centre of the University of Cambridge (UK). Four models of tunnel were tested in the beam centrifuge under different seismic events, simulated by using the dynamic actuator SAM (Madabhushi et al. 1998).

The Leighton Buzzard Sand fraction E (d_{50}=140 µm, d_{max}=150 µm, U_C=1.47, G_S=2.65, e_{max}=1.014, e_{min}=0.613) was used for model making. Dry sand models (500 mm x 250 mm x 290 mm) were prepared according to two different procedures, to obtain values of the relative density, D_r, equal to 40% and 75%. The model tunnel was an alloy tube 200 mm long, having external diameter D = 75 mm and thickness t = 0.5 mm. It was installed at two different depths, corresponding to a tunnel cover, C, equal or twice the diameter. Table 1 shows the features of the four models tested in centrifuge.

Table 1. Tested models.

Model	tunnel cover, C (mm)	relative density, D_r (%)
T1	75	~75
T2	75	~40
T3	150	~75
T4	150	~40

A typical layout of an instrumented model (T1) is shown in Fig. 1. Each model was monitored with miniature piezoelectric accelerometers positioned along three vertical alignments: two arrays were located in the soil ('tunnel' and 'free-field' in Fig. 1) and another ('reference') along the model container. The surface settlements were measured by LVDTs, placed in two gantries above the model.

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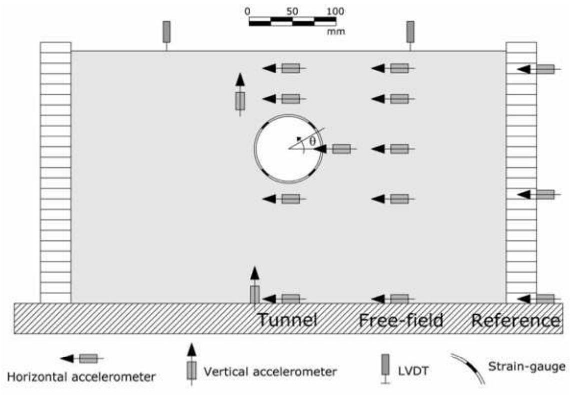

Figure 1. Schematic assembly of model T1.

Strain gauges (Wheatstone bridges) were positioned on the external and internal surface of the tube. They allowed to measure the hoop force and the bending moment in four points of the tunnel cross section, located along two perpendicular diameters at angles θ=45°+n90° (n=0 to 3).

Each model was stepwise spun up to 80g, thereafter underwent four seismic events (EQ1 to EQ4) at increasing frequency and acceleration amplitudes. According to the centrifuge scaling laws (Schofield 1980), the dimensions of the model, multiplied by a factor of N=80, correspond to a 6 m diameter tunnel with an equivalent concrete lining thickness of about 6 cm in a 23 m deep sand layer at the prototype scale. Table 2 shows the values of amplitude, nominal frequency and duration of each signal, with bracketed figures referring to the prototype scale.

Table 2. Model earthquakes.

Input signal	Frequency, f (Hz)		Duration (s)		Amplitude (g)	
EQ1	30	[0.375]	0.4	[32]	4.0	[0.05]
EQ2	40	[0.5]	0.4	[32]	8.0	[0.10]
EQ3	50	[0.625]	0.4	[32]	9.6	[0.12]
EQ4	60	[0.75]	0.4	[32]	12.0	[0.15]

2.2 *Experimental results*

The sequence of earthquakes generated by the SAM consisted of pseudo-harmonic input motions with non-symmetric irregular cycles. As an example, Figure 2 shows the acceleration time history recorded at the base of the reference vertical alignment of the dense sand model T1 during the earthquake EQ2. This test was taken as the experimental benchmark for the calibration of the numerical predictions, and will be analysed in detail in the following.

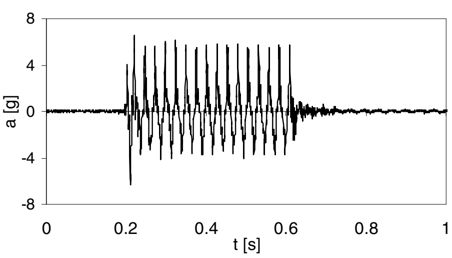

Figure 2. Input motion (model T1, EQ2).

The surface/base transfer functions along the vertical alignments were calculated as the ratio between the Fourier spectra of the recorded signals. Figure 3 shows the comparison between the amplification factor at the reference accelerometer array (black solid line) and that along the tunnel axis (grey solid line). The amplification peaks appear significantly reduced along the tunnel vertical, highlighting the wave-screening effect of the tunnel structure.

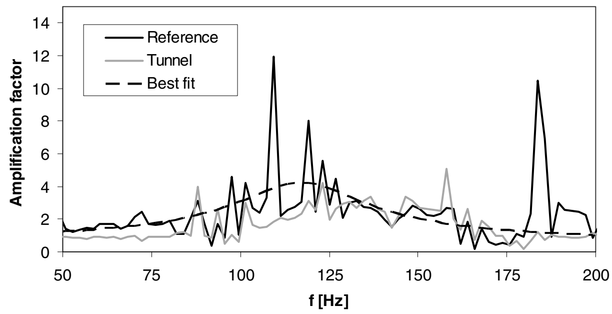

Figure 3. Surface/base transfer functions (model T1, EQ2).

The transfer function at the reference vertical was back-analysed, to derive the equivalent stiffness and damping parameters of the sand mobilised during the centrifuge test. The experimental curve was best-fitted with the analytical expression of the amplification factor of a visco-elastic soil column (dashed line). An equivalent shear modulus G=26.5 MPa and a damping ratio D=15.2 % were calculated in this particular case. The extensive application of the same procedure to all tests resulted into equivalent stiffness ranging between 8 and 30 MPa and damping ratio between 5.5% and 32%, consistently with the sand density and the earthquake amplitude (Lanzano 2009).

The horizontal displacements u(t) were obtained by double integration of the acceleration time histories. The records were filtered prior to each of the numerical integrations, to avoid an artificial increase of the signal noise. The shear strains along vertical arrays were then calculated by differentiating with space the displacement time histories, according to Brennan et al. (2005).

The bending moment, M, and hoop forces, N, in the lining were continuously monitored by the strain gauges during each swing-up stages and seismic event. It was observed that the residual values of the internal lining forces after the shaking were significantly different from the initial conditions. This behaviour was recorded almost systematically for any event in all the models (Lanzano 2009), and indicates that permanent deformations of the soil and/or the tunnel occurred during shaking. A densification of the sand was in fact shown by the settlements read by the surface LVDTs (Lanzano et al. 2009).

The average values of the peak-to-peak amplitude of the oscillations of both bending moment, $M_{pk\text{-}pk}$, and hoop force, $N_{pk\text{-}pk}$, in the time histories were taken to compute the seismic increments $\Delta M=0.5 \cdot M_{pk\text{-}pk}$ and $\Delta N=0.5 \cdot N_{pk\text{-}pk}$. Such values were taken as experimental reference for comparison with the predictions of the numerical analyses.

3 NUMERICAL SIMULATION

In the numerical simulations shown hereafter, the soil was modelled as linear visco-elastic, by assuming the shear stiffness G and the damping ratio D constant with depth, and corresponding to the values back-figured from the experimental amplification curve. A comparison of test data with equivalent linear analyses is shown elsewhere (Lanzano et al. 2009).

The input signal for the analyses was taken equal to the record of the accelerometer located at the base of the reference array (Fig. 1). A frequency domain analysis was performed by EERA (Bardet et al. 2000), to obtain a free-field solution under the hypothesis of 1D wave propagation; the base boundary condition corresponds to that of a rigid outcropping bedrock.

Full dynamic linear visco-elastic analyses of the coupled ground-tunnel system undergoing shaking were performed by the FE codes PLAXIS V8 (Brinkgreve 2002) and ABAQUS v6 (HKS 2003).

The geometry of the centrifuge model T1 was reproduced by the finite element meshes shown in Figure 4. The PLAXIS mesh consisted of triangular 15-node elements, the ABAQUS mesh of 8-node rectangular ones. In both cases, the mesh density was high enough to ensure the frequency content of the input signal not to be artificially filtered. The lining was modelled by using beam elements.

The two vertical boundaries were linked by rigid 'node-to-node anchors' or 'pins', forcing them to have identical displacements as in the rigid laminar box in the centrifuge tests. The interface between the tube and the soil was modelled in two different ways, according to different software options: in ABAQUS a full slip (zero friction) contact was defined; in PLAXIS elastic "interface elements" were activated with shear stiffness set much lower than the surrounding soil. A rigid bottom boundary was assumed in both FE models. The viscous damping was modelled through a Rayleigh formulation, using the 'double frequency approach' (Park & Hashash 2004).

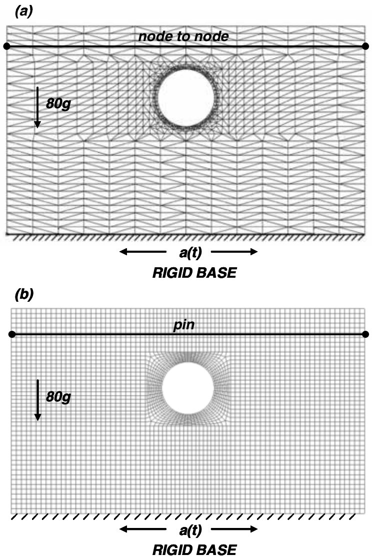

Figure 4. Finite element mesh of test T1 in PLAXIS (a) and ABAQUS (b).

Figure 5 shows the comparison between the profiles of a_{max} and γ_{max} predicted by different dynamic analyses and the experimental data. Along the reference vertical alignment (Fig. 5a, left), the amplitude motion computed by EERA in 1D conditions (thin solid line) and ABAQUS (thick solid line) are very close each other and to the measurements (triangles). PLAXIS (dashed line) predicted an almost straight profile of a_{max} and slightly overestimated the amplification. Along the tunnel alignment (Fig. 5b, left), the two codes computed different profiles of a_{max}, the values calculated by ABAQUS being closer to the experimental data. Both codes significantly over-predicted the peak acceleration above the tunnel.

The peak shear strains computed along the reference alignment (Fig. 5a, right) by EERA and ABAQUS are similar and close to the average experimental value of about 0.1% (grey line), while those predicted by PLAXIS are much lower. Along the tunnel vertical (Fig. 5b, right) both FE codes compute γ_{max} higher than those along the reference alignment, due to the high deformability of the lining. Above the tunnel, the numerical methods predicted shear strain significantly lower than the average experimental data (about 0.25%), while the agreement improves below the tunnel. Across the tunnel lining, the shear strain predicted are significantly reduced by the interface conditions, being zero those relevant to the full-slip hypothesis assumed in ABAQUS.

Finite element analyses allowed the peak seismic increments of bending moment, ΔM, and hoop forces, ΔN, to be computed along the nodes of the tunnel lining, and plotted against the anomaly, θ (see Fig. 1), as shown in Figure 6.

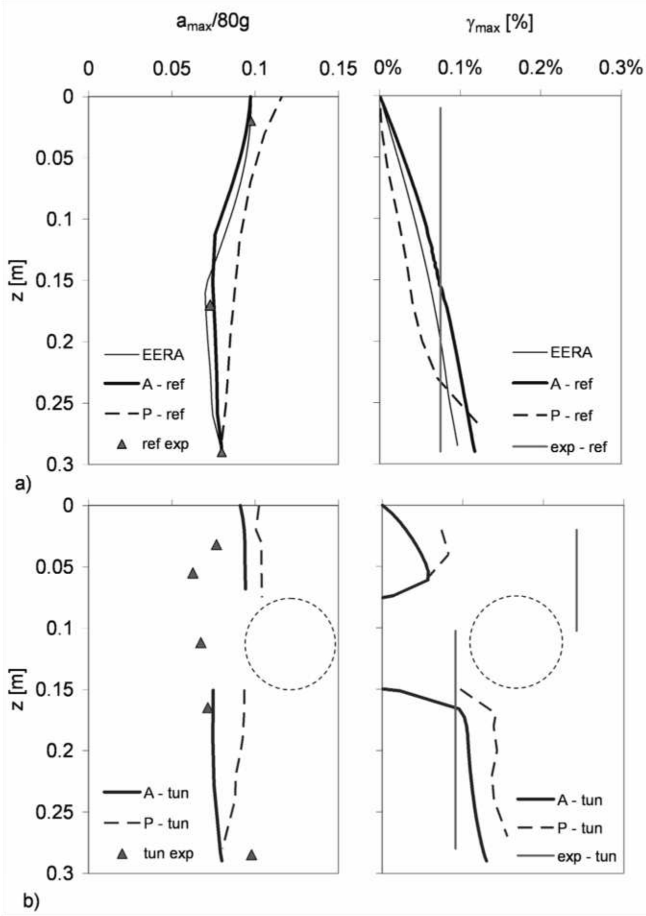

Figure 5. Comparison between measured and predicted profiles of a_{max} and γ_{max} for the reference (a) and tunnel (b) verticals (model T1, EQ2).

The values measured along the tunnel cross section are shown with symbols for comparison. In the same plots, the thin solid lines represent the internal force increments computed with the uncoupled approach, i.e. by introducing in the formulas by Wang (1993) the average value of the peak shear strain γ_{max} calculated by EERA at the tunnel depth.

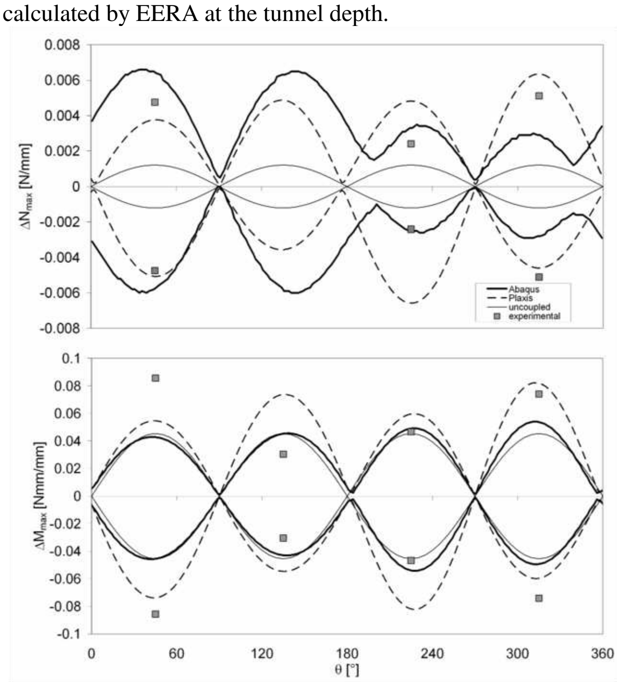

Figure 6. Increments of bending moment M and hoop forces N (EQ2 in model T1, at model scale)

The internal forces, as resulting from the full dynamic analyses, are in substantial agreement with the measured values. Not negligible differences arise between the two FE solutions in terms of hoop forces, the experimental values being somehow intermediate between the two predictions. Furthermore, the hoop forces calculated via the uncoupled approach are much lower than the experimental ones. On the other hand, the bending moments calculated with the uncoupled approach are close to both .the predictions of FE analyses, being about equal to those by ABAQUS.

It can be noted that the peak increments of hoop force computed modelling a full-slip contact (ABAQUS) are generally higher in the upper arch (θ=0-180°) than in the invert of the tunnel (θ=180°-360°); instead, they result more uniform when modelling the interface as an elastic layer of very low stiffness around the tunnel (PLAXIS).

The variability and the large scatter of the hoop forces predicted with the different approaches induced to carry out an additional set of sensitivity FE analyses, by simply modifying the thickness of the lining. In fact, the model tunnel adopted for the centrifuge tests was characterised by a very flexible lining, to allow sufficient accuracy in the measurement of the strains along the tube. Hence, two different lining stiffness were considered, by increasing the original thickness t=0.5 mm to 1 and 2.5 mm. The results obtained through this numerical investigation can be summarized as follows:

– at increasing thickness, the difference between the trends of the hoop force along the upper arch and the invert tend to disappear;
– the thicker the lining, the more satisfactory the agreement between the hoop forces obtained by the uncoupled approach and the FE analyses;
– a general agreement among the predicted bending moments was kept for all the thickness considered.

Table 3 reports the dimensionless scatter between the maximum hoop forces obtained with simplified (subscript 'SD') and full ('FD') dynamic analyses, for the three values of thickness selected. As expected, the tendency of the uncoupled approach to over-predict the internal forces increases with the lining stiffness, i.e. with the influence of kinematic interaction.

Table 3. Thickness of the lining and scatter between hoop forces.

t (mm)	$(\Delta N_{FD}-\Delta N_{SD}) / \Delta N_{SD}$ (ABAQUS)	$(\Delta N_{FD}-\Delta N_{SD}) / \Delta N_{SD}$ (PLAXIS)
0.5	313%	378%
1	47%	96%
2.5	9%	-49%

4 CONCLUSIONS

Uncoupled and coupled dynamic analysis of the seismic behaviour of a circular tunnel in sand were calibrated with reference to an 'artificial' case-history, generated by using a geotechnical centrifuge.

The results of numerical predictions were first compared in terms of profiles of peak acceleration a_{max}, and shear strain γ_{max}. The experimental profiles are quite matched by the predictions by EERA and ABAQUS, while larger differences arise with PLAXIS code, which seems to over-predict accelerations and under-predict shear strains.

The seismic increments of the hoop forces and bending moments predicted by FE analyses were again compared with experimental data, and also with a simplified (uncoupled) approach, based on the prediction of free-field strains and the use of Wang's formulas. Not negligible differences between PLAXIS and ABAQUS were obtained, probably due to the different model adopted for the soil-tunnel interface.

All the computational methods yielded realistic predictions of the bending moments, while for the hoop forces the scatter between experimental results and the different numerical predictions deserves further investigation.

The role played by the unusually small thickness of the lining adopted in the centrifuge tests has been analysed with a set of additional numerical analyses. It has been shown that the agreement between the different methods largely improves when thicker lining are considered.

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