AN EARLY-STAGE DESIGN PROCEDURE FOR CIRCULAR TUNNEL LINING UNDER SEISMIC ACTIONS

E. Bilotta[1], G. Lanzano[1], G. Russo[1], F. Santucci de Magistris[2] and F. Silvestri[1]

[1] *DIGA, University of Napoli Federico II, Naples,*
[2] *SAVA, Structural and Geotechnical Dynamics Laboratory, University of Molise, Campobasso, Italy*
Email: bilotta@unina.it

ABSTRACT :

The increments of the internal forces induced by an earthquake in the transverse section of a tunnel lining can be ascribed to the ovalisation of the section, induced by soil shear straining in the vertical plane. They can be assessed with several procedures at different levels of complexity. In this paper, two kind of analysis were per-formed on idealised geometry and soil conditions, considered representative of soil classes specified by Euro-code 8: pseudo-static analysis, where the seismic input was reduced to an equivalent peak strain amplitude, computed through a free-field pseudo-static analysis of the ground and then considered acting on the tunnel lining in static conditions; and full dynamic analysis, where the soil and tunnel responses were mechanically coupled and modelled by using FEM. Both were performed considering the soil as an equivalent linear medium.
On the basis of the comparison of the results of both approaches, modification factors of the usual pseudo-static formulae are proposed, which take into account the kinematic interaction between the tunnel and the ground during shaking. The method, based on the use of simple charts, can be easily adopted for early-stage design.

KEYWORDS: tunnels, lining, seismic actions, pseudo-static, dynamic

1. INTRODUCTION

Civil infrastructures and lifelines in seismic areas need to be designed to support the extra loading produced by earthquakes. Some indications for such design can be found in Owen & Scholl (1981), JSCE (1992), AFPS/AFTES Guidelines (2001), ISO TC 98 (2003). Design rules for tunnels are not introduced in Eurocode 8 (EN 1998-5, 2003). This maybe because earthquake effects on underground structures were deemed to be negligible, in spite of the different evidences from several case-histories (see for instance Lanzano *et al*., 2008). Research activities are in progress in Italy to refine the design methods for tunnels under seismic actions (e.g. Bilotta et al., 2007). The shear waves propagating during an earthquake perpendicularly to the tunnel axis, result in a distortion of the cross-section of the structure: in this paper a procedure to calculate the forces induced by ground shaking in the tunnel lining in simple subsoil conditions is illustrated. Such simplified procedure incorporates the results of finite elements dynamic analyses, which consider the kinematic interaction between the tunnel lining and the ground, in the framework of the pseudo-static approach commonly adopted for early-stage design.
Three idealized ground conditions (Fig. 1) were considered: a 30 m thick layer of soft clay, medium dense sand or gravel, overlying a compliant rock bedrock (V_r= 800 m/s, γ=22 kN/m3, D_0=0.5%). The tunnel has the following characteristics:

- circular shape with reinforced concrete lining (variable thickness from 0.1 to 1.3 m, diameter D=6 m);
- axis depth z_0=15 m;

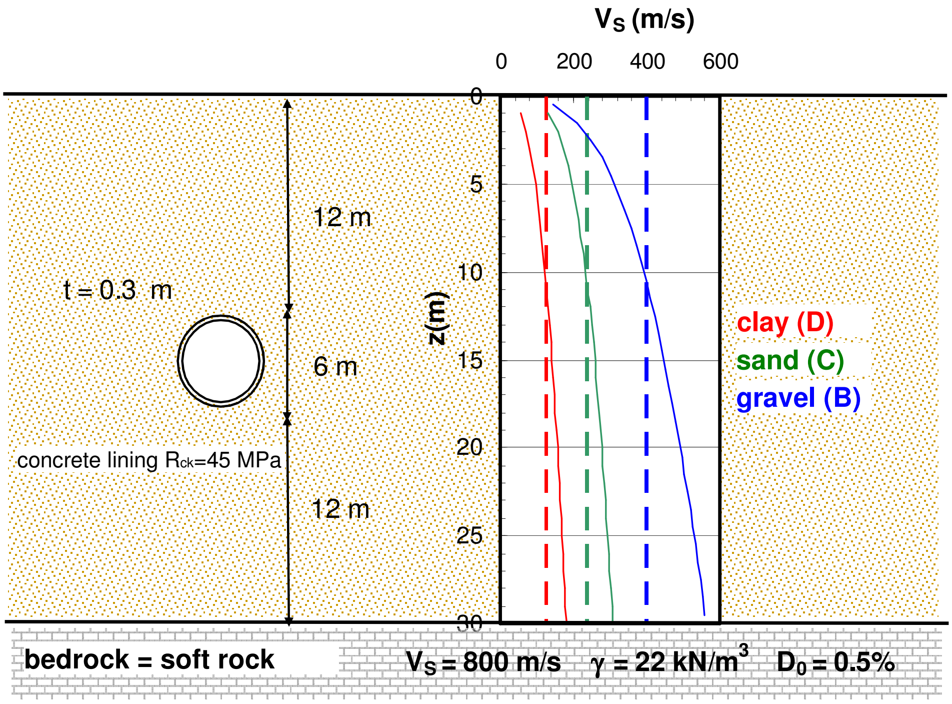

Figure 1 - Ground conditions

The values of small strain soil parameters have been chosen according to literature empirical relationships linking the shear modulus (G_0) and the damping ratio (D_0) to the lithostatic stress, the void ratio and intrinsic soil properties, such as particle size and plasticity index IP (Santucci de Magistris, 2005; d'Onofrio & Silvestri, 2001). The profiles of V_S with depth adopted for each soil type are shown in Fig. 1, where the dashed lines represent the value of the so called 'equivalent velocity' $V_{S,30}$ (EN 1998-1, 2003). Table 1 summarizes the geotechnical parameters and the ground type according to EC8.

Table 1: Ground parameters and classification according to EC8

Ground	type	ϕ' (°)	I_P (%)	γ (kN/m^3)	D_0 (%)	$V_{S,30}$ (m/s)
Clay	D	25	30	18	2.5	125
Sand	C	35	-	20	1.0	240
Gravel	B	45	-	21	1.0	400

Soil non-linearity and cyclic energy dissipation were taken into account through an equivalent linear approach which considers the variation of the shear modulus G and the damping ratio D with the shear strain γ. Therefore, the curves $G(\gamma)/G_0$ and $D(\gamma)$ for the three materials (Figure 2) have been assumed according to literature indications (Vucetic & Dobry, 1991; Stokoe, 2004).

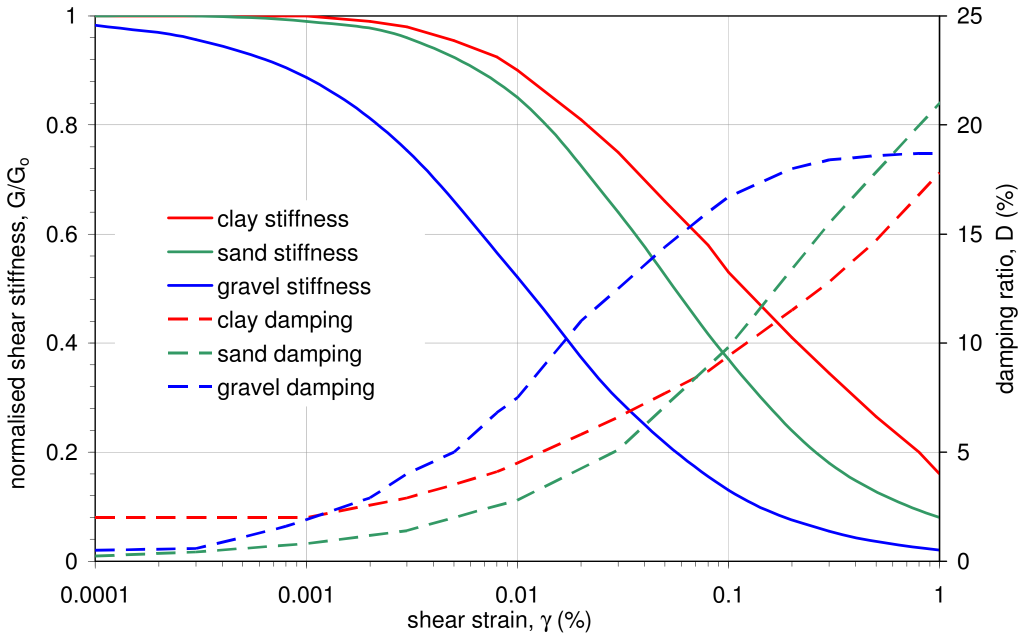

Figure 2 - Variation of shear modulus and damping with shear strain level

2. 2 PSEUDO-STATIC *VS* FULL DYNAMIC CALCULATION

In the usual simplified methods the kinematic soil-structure interaction is neglected as the free-field displacements are applied to the tunnel boundary (e.g. Hashash *et al.,* 2001) and the seismic force increments in the lining are calculated by means of the closed-form elastic solutions (Wang, 1993) for a tunnel surrounded by a homogeneous and isotropic half-space, using the average shear deformation γ_{PS} of the ground at the tunnel depth as input:

$$N(\theta)=\pm\frac{1}{2}K_2 G_m D\gamma_{PS}\cos 2\left(\theta+\frac{\pi}{4}\right) \quad (1a)$$

$$M(\theta)=\pm\frac{1}{12}K_1 G_m D^2\gamma_{PS}\cos 2\left(\theta+\frac{\pi}{4}\right) \quad (1b)$$

where:

$$K_1 = \frac{12(1-\nu_m)}{2F+5-6\nu_m} \tag{2a}$$

$$K_2 = 1 + \frac{F[(1-2\nu_m)-(1-2\nu_m)C] - \frac{1}{2}(1-2\nu_m)^2 + 2}{F[(3-2\nu_m)+(1-2\nu_m)C] + C\left[\frac{5}{2}-8\nu_m+6\nu_m^2\right] + 6 - 8\nu_m} \tag{2b}$$

The dimensionless parameters F and C represent the relative soil/tunnel stiffness and refer to a tunnel with diameter D, lining thickness t, and elastic parameters E_l and ν_l, in an elastic ground (G_m and ν_m):

$$F = \frac{G_m(1-\nu_l^2)D^3}{2E_l t^3} \tag{3}$$

$$C = \frac{G_m(1-\nu_l^2)D}{E_l t(1-2\nu_m)} \tag{4}$$

Based on the equilibrium of a deformable soil column from the surface to a given depth z, several procedures can be adopted to evaluate the average shear deformation γ_{PS}, as discussed in Bilotta *et al*. (2007).
A first class of methods is based on the specification of a vertical profile of peak acceleration a_{max}. Then, the maximum shear stress τ_{max} is computed by integration as:

$$\tau_{\max}(z) = \int_0^z \rho a_{\max}(z)dz \tag{5}$$

Another class of methods follows an approach similar to those adopted for the evaluation of liquefaction susceptibility based on simplified procedures to define the seismic induced shear stress profile. Hence, the shear stress distribution with depth is calculated according to the following equation:

$$\tau_{\max}(z) = r_d(z)\frac{a_{\max,s}}{g}\sigma_v(z) \tag{6}$$

In Eq. (6), σ_v is the total vertical stress, and r_d is a reduction parameter which takes into account the deformability of the soil column. Several empirical relationships (Iwasaki *et al.,* 1978; Liao & Whitman, 1986; Power *et al*., 1996; Idriss & Boulanger, 2004) to define r_d are reported in literature.
All the considered pseudo-static methods require a preliminary evaluation of the peak acceleration at surface. In the paper this value has been computed as:

$$a_{\max,s} = S \cdot a_g \tag{7}$$

where a_g is the peak acceleration on outcropping rock site and S the site response factor. Its value has been originally specified by EC8 part 1 (EN 1998-1, 2003), followed by several proposed of updating (i.e. Italian OPCM 3274, 2003; ETC12, 2006) for each ground type (Table 2). In this paper, the average value has been assumed. Alternatively, a non-linear response factor can be used, varying with the ground motion amplitude, as proposed by Ausilio *et al.* (2007).

Table 2: Response factors in pseudo-static methods

Soil	EC8	OPCM 3274	ETC12	average
Clay	1.35	1.35	1.1	1.27
Sand	1.15	1.25	1.15	1.18
Gravel	1.2	1.25	1.3	1.25

The maximum shear strain at a depth z is therefore calculated from the maximum shear stress, $\tau_{max}(z)$, according to the Ramberg & Osgood (1943) model:

$$\gamma_{max}(z) = \frac{\tau_{max}(z)}{G_0} + C\left[\frac{\tau_{max}(z)}{G_0}\right]^R \qquad (8)$$

where the parameters C and R have been calibrated on the curves of Fig. 2.
On the other hand, in the full dynamic analysis of the coupled ground-tunnel system undergoing shaking, the incremental internal forces in the lining are computed using a numerical model. The finite elements software Plaxis v8 (Brinkgreve, 2002) was used to perform two-dimensional free-field and soil-structure interaction dynamic analyses. A set of input acceleration time histories was selected from a database of records of Italian seismic events (Scasserra *et al.*, 2006). All the signals have been scaled to the same conventional value of a_g (0.35g) and applied at the base of the model.
The bedrock has been assumed as a rigid boundary, whereas lateral mesh boundaries were located at a distance about 8 times the layer thickness (Visone *et al.*, 2008) and were modelled with dampers according to the Lysmer & Kuhlemeyer (1969) formulation (Fig. 3). Ground conditions and soil behaviour have been modelled according to Figs. 1 and 2. As the FE analyses were performed with a linear elastic model for the soil, the dependency of the soil stiffness and damping ratio on the strain level has been first considered by a secant equivalent approach. Therefore, preliminary one-dimensional SSR analyses have been performed by means of the code EERA (Bardet *et al.*, 2000), which operates in the frequency domain. The material properties calculated as output from the SSR analysis were hence used as input to the FE analyses. The continuum was divided into the same number of sublayers as specified in EERA and different materials were defined for each sublayer. The soil damping was modelled through the Rayleigh formulation, according to the double frequency method, assuming an almost constant damping ratio between the first natural frequency of the deposit and a frequency n times larger; n is the first odd integer which approximates by excess the ratio between the fundamental frequency of the seismic signal and the first natural frequency of the deposit (Lanzo *et al.*, 2004).

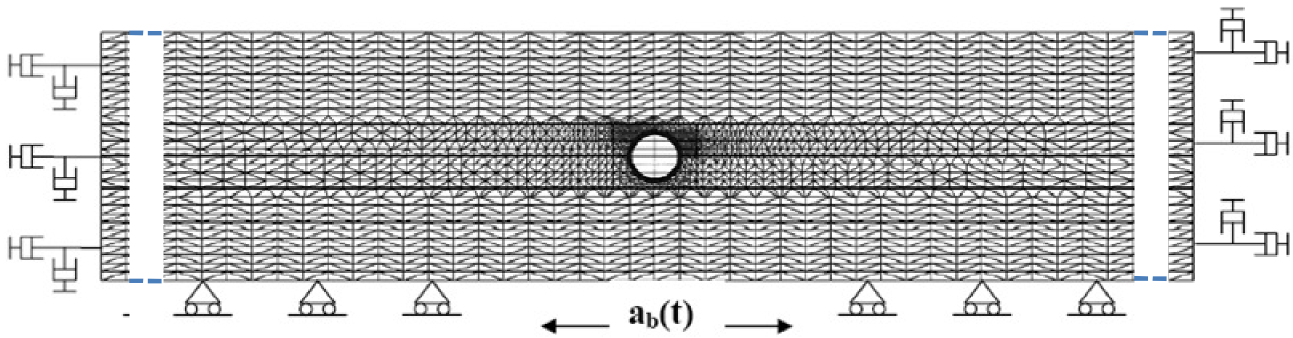

Figure 3 - Sketch of the mesh used for FE Plaxis analyses

3. ANALYSIS OF THE RESULTS

Depending on the selected method, the pseudo-static analyses allowed the maximum hoop force $N_{max,PS}$ and bending moment $M_{max,PS}$ to be calculated neglecting the kinematic interaction. On the other hand, the FE analyses allowed calculating the maximum hoop force $N_{max,DYN}$ and bending moment $M_{max,DYN}$ accounting for kinematic interaction.
The results of the pseudo-static and full dynamic analyses where combined together and the following parameters were defined, having the dimensions of a compliance:

$$k_N^* = \frac{N_{\max,PS}}{G_m \gamma_{PS}} \frac{\gamma_{DYN,FF}}{N_{\max}^{DYN}} \qquad (9a)$$

$$k_M^* = \frac{M_{\max,PS}}{G_m \gamma_{PS}} \frac{\gamma_{DYN,FF}}{M_{\max}^{DYN}} \qquad (9b)$$

In Eqs (9) the values:

$$\frac{N_{\max,PS}}{G_m \gamma_{PS}} = \frac{1}{2} K_2 D \qquad (10a)$$

$$\frac{M_{\max,PS}}{G_m \gamma_{PS}} = \frac{1}{12} K_1 D^2 \qquad (10b)$$

are in fact representative of the relative stiffness between soil and lining. In the same equations the ratios $N_{\max,dyn} / \gamma_{DYN,ff}$ and $M_{\max,dyn} / \gamma_{DYN,ff}$ are factors which quantifies the effects of kinematic soil-tunnel interaction in the numerical analyses.

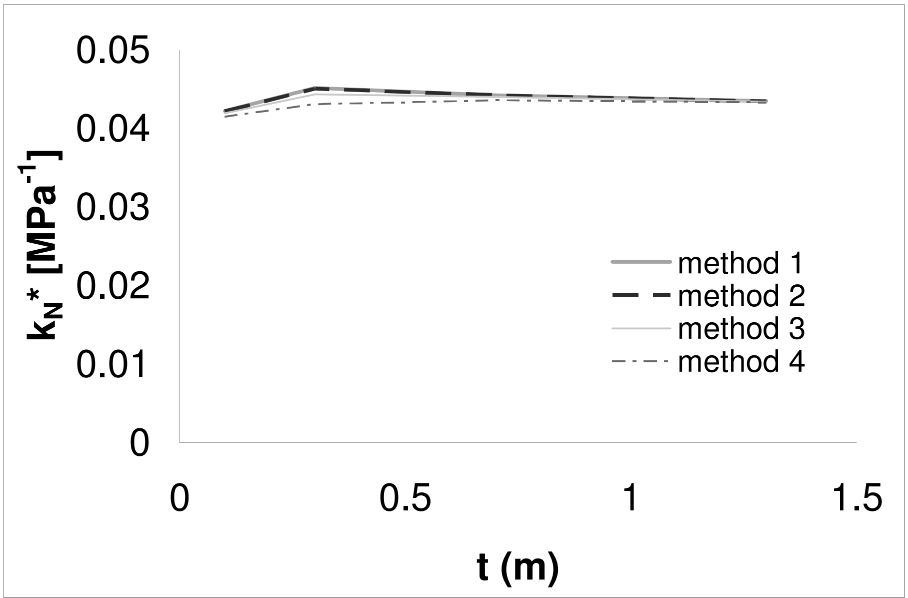

Figure 4 - Kinematic interaction parameter k^*_N *vs* lining thickness

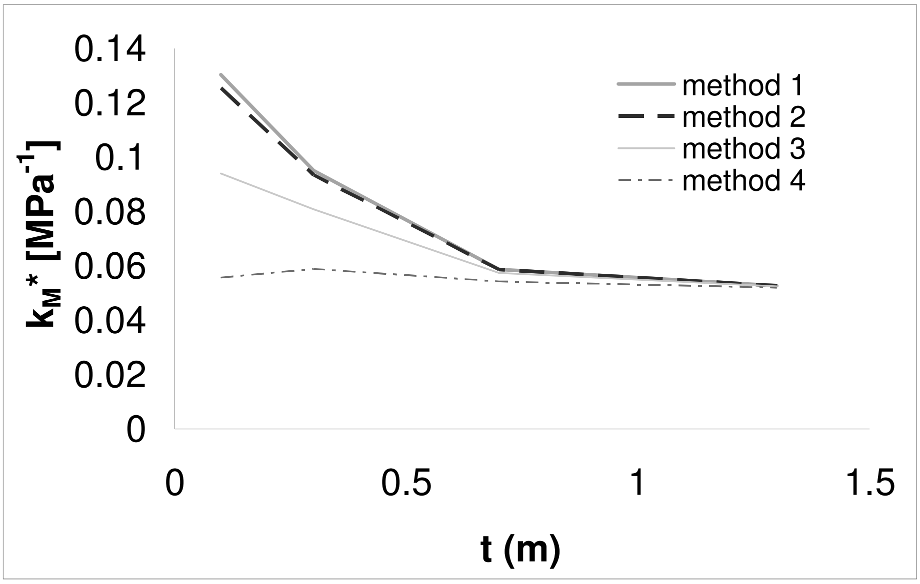

Figure 5 - Kinematic interaction parameter k^*_M *vs* lining thickness

The free-field estimations of the shear strain γ_{PS}, computed by pseudo-static methods were obviously different from the corresponding finite element solution $\gamma_{DYN,FF}$. The following dimensionless parameter was defined to quantify such a difference:

$$\alpha = \frac{\gamma_{PS}}{\gamma_{DYN,FF}} \qquad (11)$$

In Table 3 the values of α are shown, as computed by Eq. (11) using the average shear strain of each of the four method proposed by Bilotta *et al.* (2007). For each method, the values of α for sand and gravel are very close. Different is the case of clay, for which the average values are about twice as larger.

Table 3: Average values of ratio α

	Gravel	Sand	Clay
method 1	2.5	2.2	5.8
method 2	2.3	2.1	5.4
method 3	1.4	1.2	3.5
method 4	0.4	0.5	0.9

By means of any of the above mentioned pseudo-static methods, the following expressions may be used to evaluate the maximum bending moments and hoops, taking into account the possible kinematic interaction:

$$N(\theta) = \pm \frac{1}{2 \cdot \alpha \cdot k_N^*} K_2 D \gamma_{PS} \cos 2\left(\theta + \frac{\pi}{4}\right) \qquad (12a)$$

$$M(\theta) = \pm \frac{1}{12 \cdot \alpha \cdot k_M^*} K_1 D^2 \gamma_{PS} \cos 2\left(\theta + \frac{\pi}{4}\right) \qquad (12b)$$

where α is given for each method in Table 3 and the modification factor for a given lining thickness t, k_N^* and k_M^*, can be obtained (for sand) from the charts in Figs.10 and 11.

4. CONCLUSIONS

The simplified procedure proposed in the paper was derived from the comparison of the results of a series of full dynamic analyses of circular tunnels embedded in a schematic subsoil with four pseudo-static methods proposed in a previous conference paper (Bilotta *et al.,* 2007). It allows a simple modification to improve the accuracy of widely used closed form elastic solutions to calculate the increment of internal forces on a tunnel lining due to a seismic action (Wang, 1993). The improvement is based on a simplified way to introduce the kinematic interaction between the tunnel and the ground into a pseudo-static approach.
Efforts are currently performed in order to improve the reliability of such procedure on the basis of the results of more sophisticated numerical models calibrated on centrifuge tests. This might lead also to produce simplified methods to estimate the magnitude of tunnel displacements during seismic loading.

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