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SEISMIC RESPONSE ANALYSIS OF AN IMMERSED TUNNEL USING IMPOSED DEFORMATIONS

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ABSTRACT

The seismic response of tunnels is governed by kinematic interaction between tunnel structure and surrounding soil, whereby inertia effect become negligible. The seismic analysis considers racking deformation of the rectangular cross section as well as axial and curvature deformations due to the snaking mode in the longitudinal direction. The first task is accomplished by means of solutions based on 2D continuum models. The second task requires some engineering approximation of the complex 3D problem that encompasses all the relevant features of the global tunnel response. A system consisting of a beam supported by non-linear springs that reflect the effects of soil-structure interaction is introduced for this purpose. The paper summarizes a case study by describing the methodology adopted to determine the seismic input motion acting on the tunnel, the main features of the structural system used for the analysis, the derivation of appropriate values for the soil springs, and the main findings with respect to the influence of element lengths on sectional forces and joints displacements.

Keywords: immersed tunnels, soil-structure interaction, seismic analysis

INTRODUCTION

Unlike the case of surface structures whose seismic response is governed by inertia effects, the response of tunnels embedded in the ground is primarily kinematic, i.e. it is caused by the compatibility of the tunnel deformation to that of the surrounding ground. Therefore, soil-structure interaction effects are essential to the design. In static design the loads are well-known and the analysis is carried out in terms of forces (force method). In contrast, seismic structural response due to the imposed seismic displacements (deformation method) strongly depends on structural details. Design is carried out for three principal types of deformation: i) Axial extension and compression due to wave motions parallel to the tunnel axis causing alternating compression and tension, ii) longitudinal bending due to curvature caused by those components of the seismic wave that produce particle motions perpendicular to the tunnel axis, iii) ovaling for circular, and racking for rectangular tunnels, resp., that develops when shear waves propagate normal or nearly normal to the tunnel axis, resulting in a distortion of the cross-sectional shape of the tunnel lining.

State-of-practice presentations may be found in the papers by St John and Zahrah (1987), Wang (1993), Kiyomiya, (1995), Hashash et al. (2001) and Vrettos (2005). Immersed tube tunnels consisting of several elements, usually of rectangular cross-section, connected by flexible joints

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require a more elaborated treatment. In addition, since this type of structure is mostly used to cross shallow waters, where soil conditions are often poor, particular attention to soil-structure interaction effects is required. In the following, some general design aspects are summarized fist. The methodology applied during the tender design of an immersed tube tunnel in a region of medium to high seismicity is outlined next. The structural model is presented, and typical results for sectional forces and displacements are given.

ANALYSIS OF AXIAL AND CURVATURE DEFORMATION

For running tunnel design simple equations have been derived for the axial and curvature deformation analysis enabling the hand-calculation of strains and sectional forces. They are based on the pioneering work by Newmark (1967) and Kuesel (1969). The basic assumptions are: loading by sinusoidal wave of wavelength λ, displacement amplitude D_0, angle of incidence ψ, and structure conforming to the ground motion. Maximum values for axial and bending strains ε_x and ε_b, axial forces N, and bending moments M are given by:

$$|\varepsilon_x| = \frac{2\pi}{\lambda} D_0 \sin\psi \cos\psi \quad ; \quad |\varepsilon_b| = \pm \frac{2\pi^2 D_0\, b \cos^3\psi}{\lambda^2} \tag{1}$$

$$|N| = \frac{2\pi}{\lambda} \sin\psi \cos\psi\, E_\ell A_\ell D_0 \quad ; \quad |M| = \left(\frac{2\pi}{\lambda}\right)^2 \cos^3\psi\, E_\ell I_\ell D_0 \tag{2}$$

where E_ℓ is the modulus of elasticity, A_ℓ the cross-sectional area, and I_ℓ the moment of inertia of the tunnel lining. The maximum values $N_{\max}$ and $M_{\max}$ are obtained for $\psi = 45°$ and $\psi = 0°$, respectively.

From the above equations it can be seen that axial strains and normal forces are governed by the particle velocity amplitude while bending strains and moments are controlled by acceleration. This implies that both kinematic quantities have to be estimated with the same accuracy in order to obtain realistic design values. This fact is essential in the selection of appropriate values of the seismic input motion.

The effects of soil-structure interaction are modelled through the introduction of springs. In a first approximation these springs are assumed frequency independent. Denoting with K_a and K_t the spring coefficients in the longitudinal and transverse direction, respectively, given in units of force/displacement per unit length of the tunnel (kN/m/m) the resulting expressions for the maximum sectional forces present:

$$N_{\max,SSI} = N_{\max} \frac{K_a}{\frac{E_\ell A_\ell}{2}\left(\frac{2\pi}{\lambda}\right)^2 + K_a} \quad ; \quad M_{\max,SSI} = M_{\max} \frac{K_t}{E_\ell I_\ell \left(\frac{2\pi}{\lambda}\right)^4 + K_t} \tag{3}$$

A possibility to eliminate the dependency on the wavelength is to maximize the sectional forces with respect to it. Another possibility is to derive spring coefficients as functions of the wavelength and estimate the wavelength by an appropriate procedure based on wave propagation in the soil deposit considered. The difficulty one faces in determining values of the spring coefficients becomes obvious when comparing the recommendation of the AFTES (2001) that suggest $K_a = K_t = G$, where G is the shear modulus of the surrounding soil, and that by Clough & Penzien (1993) to set $K_a = 3G$. Shallow immersed tube tunnels correspond rather to surface box foundations, so that one may resort to available solutions for impedance functions for embedded rectangular rigid foundations, as summarized for example for some standard cases by Gazetas (1991). The consideration of the effects of soil layering on the impedances requires, however, modelling via a 3D finite-element code. Since immersed tube tunnels are composed by discrete elements connected by joints the above equations are suitable mainly for preliminary estimates of sectional forces.

A first estimate for the influence of flexible joints between tunnel elements is obtained by the simplified method by Hamada et al. (1982) assuming that the axial strain due to push-pull deformation dominates the tunnel behavior. The relative displacement of the flexible joint δ_j is given by

$$\frac{\delta_j}{L_j} = \frac{\tanh(\beta L_j/2)}{(\beta L_j/2) + (k_j L_j / E_\ell A_\ell)\tanh(\beta L_j/2)}\varepsilon_g \qquad \text{with} \qquad \beta = \sqrt{K_a / E_\ell A_\ell} \tag{4}$$

where ε_g is the ground normal strain uniformly distributed along the tunnel axis, L_j is the distance between two adjacent joints, i.e. the tunnel element length, and k_j is the axial spring stiffness of the flexible joint.

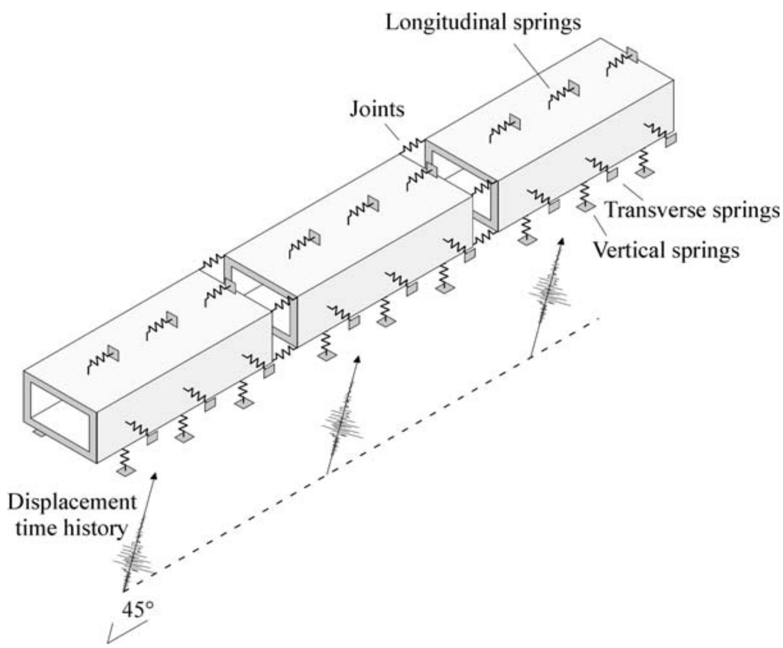

Figure 1. Model for global tunnel response

For the final structural design a tunnel model is needed describing as close as possible the global response of the complete system and also including the variation of the soil conditions along the tunnel alignment. The model depicted in Figure 1 is adopted consisting of a chain of elastic beams connected by springs representing the flexible joints and supported by vertical, longitudinal and transverse springs reflecting the effects of soil-structure interaction. The beam chain is subjected to a spatially varying free-field soil deformation pattern. Ignoring ground motion incoherence effects, the spatial variability of the ground motion can be approximately simulated by the passage of shear waves without the presence of wave scattering. The angle of incidence is set equal to 45° with respect to the longitudinal tunnel axis. The wave passage takes place with an apparent velocity *c*. The seismic motion is imposed in terms of the free-field displacements on the soil ends of the springs with a phase shift equal to the apparent wave-passage velocity, cf. Figure 1. Inertia effects of the beam are neglected. Axial force and bending moments on the tunnel cross-section increase with decreasing apparent wave velocity c. Typically, apparent wave passage velocities range between 1000 m/s and 2500 m/s, cf. Hamada et al. (1982).

ANALYSIS OF RACKING DEFORMATION

Since the size of a typical cross-section is small compared to the dominant wavelengths of the seismic motion, it can be assumed that the induced shear strain is constant over the height of the tunnel structure. Furthermore, it is assumed that inertia effects in soil-structure interaction are negligible, thus allowing a quasi-static analysis. The expected maximum free-field shear strain $\gamma_{\max}$ due to vertically propagating shear waves may be determined by the simplified Newmark method:

$$\gamma_{\max} = \frac{V_{s0}}{\bar{c}_s} \tag{5}$$

where V_{s0} is the peak particle shear wave velocity and $\bar{c}_s$ is the effective shear wave propagation velocity in the depth range of the structure accounting also for the effects of shear strain amplitude. In the preliminary design stage estimates for $\bar{c}_s$ can be obtained, for example, by applying to the small-strain values c_s an appropriate reduction factor, dependent on the maximum earthquake acceleration, as described for example by Eurocode EC8, Part 5. In the final design a numerical seismic site response analysis for vertically propagating shear waves is conducted using the code SHAKE or similar and the difference of maximum shear strains at the top and bottom elevations of the structure is determined directly from this analysis. The difference of the ground displacement between the top and bottom elevation of the structure is then $\Delta_{ff} = \gamma_{\max} h$ with h denoting the tunnel height. The relative stiffness between the lining and the medium governs the response. A solution that considers the soil-structure interaction effect is presented by Penzien (2000): The lining racking ratio R_r defined as the ratio of the diameter change of the lining Δ_ℓ to the respective value under free-field conditions Δ_{ff} is derived for a homogeneous elastic full-space:

$$R_r = \frac{4(1-\nu)}{1 + k_\ell (3-4\nu) / k_s} \tag{6}$$

where ν is the Poisson's ratio of the soil medium, k_s is the generalized stiffness of the soil displaced by the lining, and k_ℓ is the generalized stiffness of the lining. The soil stiffness is given in an explicit form $k_s = \overline{G}/h$, where $\overline{G}$ is the strain-compatible soil shear modulus of the surrounding soil , while the lining lateral stiffness k_ℓ is obtained from simple static analysis of the box structure without the surrounding soil under simple boundary conditions, whereby the stiffness coefficient k_ℓ is that shear stress which will produce the corresponding unit racking displacement of the lining under plane strain conditions, Penzien (2000). The racking displacement is finally applied to the structural model of the tunnel to calculate the sectional forces. The equations for the racking ratio ignore the influence of the free ground surface. For shallow tunnels with burial depths smaller that 1.5 times the tunnel height a correction is necessary. A reduction of the burial depth to half the tunnel height yields approx. a 20% increase in the racking ratio, cf. Wang (1993).

In the final design stage a 2D dynamic finite element analysis is carried out in addition to the above procedure by using an appropriate computer code based either on equivalent-linear frequency domain algorithms with strain-dependent shear modulus and damping or time-domain algorithms with nonlinear soil models.

PROJECT DESCRIPTION, SEISMIC ENVIRONMENT, AND SOIL CONDITIONS

The project is a four-lane road tunnel crossing a bay in the Mediterranean Sea. At both ends of the immersed part the approaches will be constructed as open trough/diaphragm wall systems. As the alignment descends the system will be transformed to a cut-and-cover tunnel diaphragm wall.

The site is characterized by moderate to high seismicity. The safety level, as finally defined by the client, corresponds to a design earthquake with return period of 2000 years. During the very early stages of the tender both the hazard level and the characteristics of the design earthquake were not specified, so that a seismic hazard study had to be commissioned. This study combined state-of-the-art methodologies and derived the characteristics of the design earthquake for different return periods and for the different soil classes as defined by the NEHRP 2000. The final specification of the client defined the design earthquake at outcropping of rock with the following values for peak ground acceleration, velocity and displacement: 0.35 g, 30 cm/s, and 15 cm, respectively. Essential for the analysis are the ratios acceleration-to-velocity and acceleration-to-displacement. A review of available databases revealed that no record satisfying the specified set of values of kinematic quantities exists for the broader region of the project area. The seismic hazard study carried out independently in the early stages of the tender yielded comparable values for the acceleration level but much lower values for the displacement.

For the tender design stage it was sufficient to use one accelerogram. An earthquake record with comparable set of values is taken from the PEER strong motion database: It refers to the Loma Prieta 1989 Earthquake recorded at Palo Alto SLAC Lab, USGS Station 1601. The site conditions correspond to rock. The characteristics of this earthquake record in the predominant horizontal direction are: peak ground acceleration (PGA) = 0.278 g, peak ground velocity (PGV) = 29.3 cm/s, and peak ground displacement (PGD) = 9.72 cm. For the subsequent seismic site response analysis the time history of acceleration is scaled by the factor 1.26 to a peak value of 0.35g to comply with the client's requirements.

Soil stratigraphy was relatively uniform along the tunnel alignment without abrupt changes, and soil classification indicates soil types that are often encountered in foundation design. Field investigations consisted of borings with SPT tests, and a number of CPT tests. Laboratory tests included soil classification tests and standard tests for soil strength and deformation properties. Some values of dynamic soil properties were also provided. These data were synthesized together with empirical relations to establish the design soil profiles along the tunnel alignment. Soil was divided into three units: Unit S1 consisting of loose silty sandy marine sediments ML, CL-ML, followed by unit S2 that is a transition zone of stiff clays and clayey sands CL-CH, SC underlain by unit S3 consisting of stiff to very stiff clays, and sandy clays to clayey sands CL-CH, SC. Average values for soil consistency index I_c in the three soil units were 0.55, 0.85, and 0.90, respectively.

SEISMIC SITE RESPONSE ANALYSES

Seismic site response analyses have been conducted by using the code SHAKE for a typical soil profile consisting of a 25 m thick soft surface layer underlain by a 35 m thick layer of stiffer soil resting on a layer of rock or firm soil. A depth-dependent soil stiffness is assumed for the top layer, cf. Figure 2. Two configurations have been analysed: "basic soil profile" and "soft soil profile" exhibiting in the surface layer a difference of 15% in the value of shear wave velocity. The non-linear behaviour of the soil has been estimated on the basis of its plasticity index from available data in the literature based mainly on the curves by Vucetic and Dobry (1991). The following peak values have been obtained at the surface of the soft soil: peak ground acceleration (PGA) = 0.50 g, peak ground velocity (PGV) = 50 cm/s, peak ground displacement (PGD) = 14 cm.

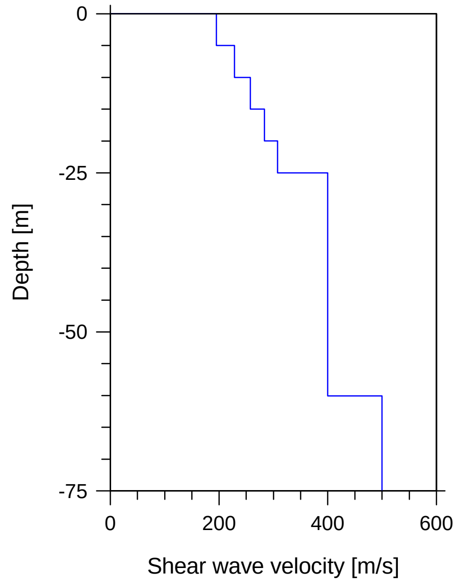

Figure 2. Soil profile for seismic site response analysis

In a next step an appropriate earthquake record is selected for use in the structural analysis of the tunnel. The client's specifications required a simultaneous excitation in the two horizontal and the vertical direction. A record from the PEER database is selected that shows approximately the relationship between acceleration, velocity, and displacement of the above set of values in both directions. The record is taken from the Imperial Valley 1979 Earthquake at the Station 5058 El Centro Array #11 with magnitude M_s = 6.9. The site conditions correspond to soil with an average shear wave velocity to a depth of 30 m ranging from 180 m/s to 360 m/s. These records are then scaled to yield in the predominant horizontal direction of excitation a peak ground acceleration of 0.50g, i.e. both horizontal records are multiplied by 0.50/0.38=1.316. The vertical component is scaled to yield a PGA equal to 70% of the horizontal PGA of 0.5g. This data set was finally used as the design earthquake for the analysis of the global tunnel response and is summarized in Table 1. The respective time-histories of the two horizontal components are depicted in Figure 3. As can be seen, the values of the velocity in the horizontal direction compare well with the value of 50 cm/s as determined in the project-specific seismic site response analysis.

For the transverse racking analysis of the tunnel a first estimate for the maximum free-field shear strain γ_{max} was obtained using equation (5) and the reduction factor to account for the effects of acceleration level according to EC8, Part 5. The shear strain as computed directly from the seismic site response analysis for the typical soil profile varies in the relevant depth of the surface layer between 0.15% and 0.25%. Based on these findings an average value γ_{max} = 0.2% is finally used for the design.

Table 1. Data set of maximum values for the design earthquake

Component		PGA [g]	PGV [cm/s]	PGD [cm]
Horizontal	H-E11 140	0.479	45.4	21.1
Horizontal	H-E11 230	0.500	55.4	24.5
Vertical	H-E11 UP	0.350	27.8	17.1

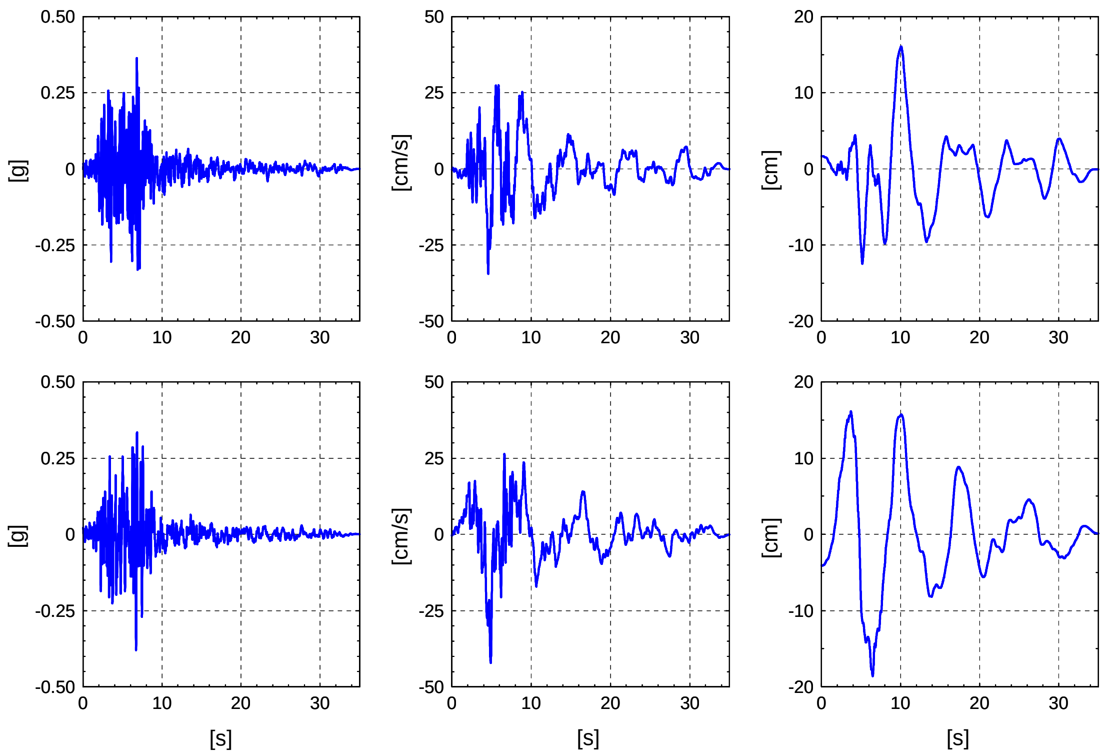

Figure 3. Acceleration, velocity, and displacement time histories of the two horizontal components of the design earthquake

SOIL-STRUCTURE INTERACTION SPRING COEFFICIENTS

There is still no consensus among design engineers with respect to the derivation of the spring coefficients accounting for the effects of soil-structure interaction. The method followed herein was initially applied in the design of another immersed tunnel in seismic region and is presented in detail by Vrettos and Savidis (2004). Some other proposals from the literature are used for verification purposes. Parameters to be considered are the geometry of the foundation, the embedment depth, and the non-linear soil-structure interaction. Spring values of are determined for a rigid single tunnel element with aspect ratios 5:1 and width equal to 36 m. The basic assumptions are: rigid rectangular box structure, static loading conditions, perfect bonding between foundation and soil, and linear-elastic theory. The shear modulus is taken equal to the one determined from the equivalent-linear free-field seismic site response analysis of the soil. For the effects of embedment, that is taken equal to the height of the tunnel elements, the actual sidewall-soil contact surface (only two sidewalls) is used. The Poisson's ratio is assumed equal to 0.45. The values obtained are: $k_x = 2.35\,\overline{G}$ in the longitudinal, $k_y = 2.78\,\overline{G}$ in the transverse horizontal and $k_z = 3.64\,\overline{G}$ in the vertical direction in units of MN/m per meter tunnel length. The values for the longitudinal springs compare well with the suggestion by Clough and Penzien (1993) to set $k_x = 3\,\overline{G}$.

The accuracy of the formulae used is checked against the numerical solution of the 3D dynamic-soil-structure interaction problem obtained by the finite-element code SASSI. Two soil models have been considered to assess the effects of soil layering. Comparison of the solution for a single tunnel element with that for a group of two elements showed that the cross-interaction effects are small when perfect boning between tunnel and soil is assumed.

An independent method is based on the proposal by St. John and Zahrah (1987). They determine the longitudinal spring dependent on the wavelength L of the shear wave travelling in the soil deposit yielding $k_x = 830\,\overline{G}\,/\,L$. The wave length L is estimated here as the average of the shear wave lengths in the surface and base layer in a two layer system consisting of a soft layer with and a stiff base, cf. Matsubara et al. (1995). For a ratio of shear wave velocities of 3 and an effective thickness of the surface layer of H = 40 m we obtain L = 240 m yielding $k_x = 3.5\,\overline{G}$, that is in the same order of magnitude as the values obtained via impedance functions for embedded foundations. However, this method is very sensitive to the value of the wave length and was used only for verification purposes. Axial springs have also been derived by Matsubara and Hoshiya (2000) but the solution is difficult to apply. Inserting $\overline{G}$ = 45 MPa into the solutions via impedance functions we obtain k_x = 106 MN/m/m, k_y = 125 MN/m/m, k_z = 164 MN/m/m. The values derived by the above procedure are regarded as "best estimates". For the final design a min/max consideration with a variation in the order of ± 30% is considered necessary, even if the detail level of subsoil investigation will be much higher.

Due to the finite shear strength of the soil, a limiting spring force must be considered for movements in the longitudinal direction. It is calculated for the standard cross section by assuming a buoyancy safety factor of 1.04, a 1 m thick rockfill, a traffic load in the tunnel of 10 kPa, an angle of friction of the soil equal to 35°, and earth pressure at rest at the two side walls of the tube element. The calculated value is 820 kN/m. A safety factor of 1.5 is applied to consider the effects of vertical earthquake motion, uncertainties in the values of soil parameters and in the value of shear strength in dependence of mobilized earth pressure yielding a limiting spring force of $T_{\max} =$ 1230 kN/m.

STRUCTURAL ANALYSIS OF GLOBAL TUNNEL RESPONSE

The aim of the design is to assess two effects that have an important influence on the construction and the cost of the tunnel, namely i) the maximum tensile stress induced by the design earthquake in the tunnel elements that defines the necessary pre-stressing to avoid decompression of the joints between the 25m long sub-elements, and ii) the deformation of the flexible joint between elements. This effect reflects the adequacy of the Gina profile to keep the joint watertight. The magnitude of both above effects increases with the increase of the length of the monolithic elements of the tunnel. Therefore a further target of this design was to investigate the influence of the monolithic element lengths. To this end, two cases of element length for a total tunnel length of 1350 m have been analysed: i) 225 m length with each element consisting of 9 x 25m long sub-elements, and ii) 150 m length with each element consisting of 6 x 25m long sub-elements. The cross-section of the tube with area A_ℓ = 123.29 m^2 and moment of inertia I_ℓ = 1326 m^4 / 15740 m^4 is shown in Figure 4.

For the analysis the tunnel is modelled by 3-D beam elements that are supported by springs subjected to the seismic motion as schematically depicted in Figure 1. The apparent propagation velocity (phase velocity) in the project area is conservatively assumed c = 1500 m/s.

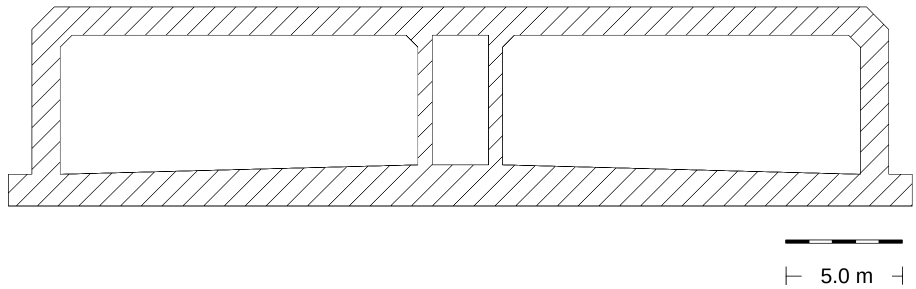

Figure 4. Tunnel cross section

The flexible joints between tunnel elements were modelled taking into account the actual non-linear force-deformation diagram of the Gina gasket ETS 200-260 SN, cf. Figure 5. In order to reflect with sufficient accuracy the strongly non-linear behaviour of the Gina-profile, its length was lumped at 20 points along the perimeter of the gasket. Due to the non-linear behavior of the gasket, the stiffnesses of the joint (longitudinal and two rotational stiffnesses) depend on the hydrostatic pressure and the amount of pre-stressing applied to the joint. The pre-stressing force was assumed 39.26 MN. This force corresponds to approx. 40% of the yield force of 17 tendons 19 Ø 0.6''. This reduced pre-stressing force was selected because it leads - in combination with the maximum force due to hydrostatic pressure (35.2 MN)- to a significant compression (approx. 123 mm) of the Gina gasket (see Figure. 5). Following this assumption, pre-stressing of the joints (along with hydrostatic pressure) is considered as a loading. Therefore the model adopted in the tender design phase does not reflect the stiffness of the tendons themselves but only that of the Gina-gasket. The tendons stiffness shall be included in the final model when the final amount of required pre-stress of the joints is selected.

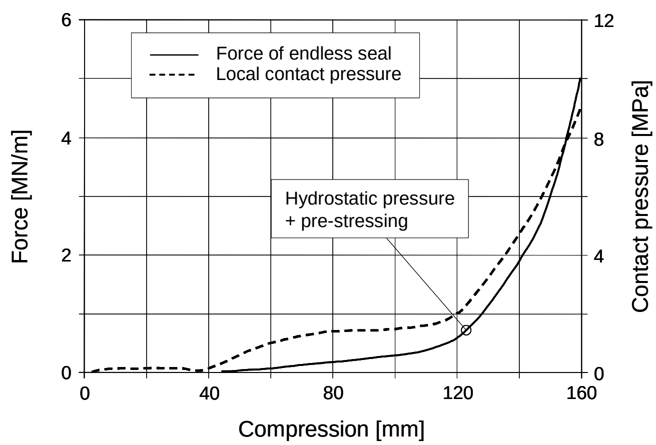

Figure 5. Force-deformation diagram of Gina-gasket

Ignoring ground motion incoherence effects, the above assumption allows to transform the displacement time history to snapshot of the ground displaced position along the length by replacing the time intervals $\Delta t = 0.005$ s of the recording by distance intervals $\Delta x = c\Delta t$ =7.5 m. For convenience, the nodes of the model were selected at these distance intervals, so that the corresponding ground displacements could be imposed at the corresponding soil ends of the soil springs representing the soil-structure interaction in the model.

The snapshot of ground deformation moves along the tunnel with the apparent phase velocity *c*. The relative positions of the tunnel to the soil deformation snapshot that lead to the maximum value of a certain action effect were selected as follows based on the derivations presented in the second chapter of the present paper: Axial effects (axial force, axial joint deformation) depend on the ground strain in longitudinal direction that in turn is proportional to the particle velocity of the seismic motion. Therefore the most onerous relative position is selected by positioning the investigated point of the structure (middle of an element or element to element joint) at the position of the snapshot corresponding to max particle velocity. On the other hand, bending effects (bending moments about vertical and horizontal axis and corresponding joint rotations) depend on the relevant ground curvature that in turn is proportional to the ground acceleration in the corresponding direction (horizontal or vertical).

The main results are presented in Table 2 for the 225 m and the 150 m long elements, respectively, in terms of maximum/minimum stresses in the tunnel elements and of the range of joint compressive deformation. The columns of the tables correspond to the component of motion whose maximum/minimum values are investigated, and contain also the concurrent effects of the other components. As already mentioned, due to the non-linearity of the response the action effects of each load case include, in addition to earthquake, the effects of hydrostatic pressure and pre-stressing of joints. In order to show the influence of each component of the seismic motions, a separate line of the table gives the effects corresponding only to the investigated component (including hydrostatic pressure), and one more line those of hydrostatic pressure only.

Table 2. Results of structural analysis

Maximum stresses of beams [MPa]						
Dominant component	Longitudinal response		Transversal response		Vertical response	
Longitudinal	S (max. vel.)	-S (max. vel.)	W (conc.)	-W (conc.)	-S (conc.)	-S (conc.)
Transversal	W (conc.)	W (conc.)	S (max. acc.)	S (max. acc.)	W (conc.)	W (conc.)
Vertical	V (conc.)	V (conc.)	V (conc.)	V (conc.)	V (max. acc.)	V (max. acc.)
Total	-4.85 [-3.73]	0.65 [0.23]	-2.02 [-1.34]	0.40 [0.01]	-1.57 [-1.28]	0.13 [0.03]
Only dominant component	S (max. vel.)	-S (max. vel.)	S (max. acc.)	S (max. acc.)	V (max. acc.)	V (max. acc.)
	-4.49 [-3.51]	0.79 [0.37]	-1.54 [-1.10]	-0.08 [-0.09]	-0.79 [-0.85]	-0.08 [-0.09]
Hydrostatic	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35
Deformation of joints [mm]						
	-161 to -150 [-157 to -123]	-104 to -60 [-120 to -76]	-143 to -120 [-137 to -113]	-127 to -100 [-131 to -105]	-146 to -124 [-141 to -123]	-120 to -96 [-122 to -105]

Note: S, W, and V indicate the strong horizontal, the weak horizontal and the vertical component of design earthquake motion, respectively. Positive values of stresses denote tension, negative values of deformation denote closing of the joint. Numerical values outside brackets are for a system of 6 elements of 225 m in length, values between brackets are for a system of 9 elements of 150 m in length.

The following conclusions can be drawn from Table 2:

i) The longitudinal component dominates by far the response of the tunnel, both from the point of view of tensile stresses of the elements and from that of the joint deformations.
ii) The tensile stresses induced by the seismic action in the elements range (including the beneficial effect hydrostatic pressure): from 0.79 MPa for the 225 m long elements to 0.37 MPa for the 150 m long elements. These stresses define the demand for longitudinal pre-stresssing to be applied on the elements.
iii) The range of compressive deformation of the Gina-gaskets is as follows: -60 mm to –161 mm for the 225 m long elements, and -76 mm to –157 mm for the 150 m long elements. These deformations include the effects of hydrostatic pressure and the (assumed as constant) pre-stressing force of 39.26 MN at each joint.
The analysis has shown that both element types meet the technical requirements. However the shorter type seems to offer substantial advantages with respect to the seismic response.

CONCLUSIONS

The methodology presented incorporates all essential features for calculating with reasonable effort the response of shallow tunnels in soft soils. A refinement is necessary for the final design referring mainly to the variation of the spring values along the tunnel alignment due to the variability of soil stratigraphy but also to the characteristics of the seismic input motion and the apparent wave velocity.

It has been shown that particle velocity is equally important to acceleration when calculating seismic sectional forces and displacements in this type of structure.

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