Design Issues for Immersed Tunnel Foundations in Seismic Areas

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Abstract

Immersed tunnels are increasingly used for crossings of shallow waters. One project has recently been completed in Greece while another one is in the tender process. The soil strata encountered are basically non-homogeneous and of poor quality, so that some ground improvement to prevent ground failure is necessary. Analysis of tunnel response is carried out by the seismic deformation method, requiring a reliable estimate of the kinematic quantities as well as of the spring constants accounting for the effects of soil-structure interaction. The paper addresses some design issues and briefly presents the foundation of the Preveza-Aktion immersed tube tunnel.

INTRODUCTION

Immersed tube tunnels are increasingly used for strait crossings in the broader Mediterranean area. One project recently completed in Greece is the Preveza-Aktion road fixed link. Another one, running along the coastline of Thessaloniki, is in the final tender process. Both immersed tubes are constructed in shallow waters. Some of the design issues that emerged during the author’s involvement in the foundation design of the above structures are summarized in the present paper. Immersed tunnels can be classified as structures between deep embedded lifelines and surface structures, thus requiring differentiated treatment. In addition, in most cases they are founded on poor ground conditions, so that some kind of improvement is necessary.

SEISMIC INPUT

The seismic environment for this size of project requires a site-specific seismic hazard analysis. The outcome of such a study is given in terms of maximum values for acceleration, velocity, and displacement. While the former two quantities can be determined by sufficient accuracy, displacement values are difficult to estimate. Seismic input parameters for common high-rise buildings are defined in terms of acceleration spectra whereas tunnel design is carried out by the deformation method requiring, as will be shown later, appropriate values of all three kinematic quantities.

Various Japanese codes for lifelines define the seismic input motion in terms of velocity response spectra, thus providing a valuable starting point for the estimation of design values for the seismic deformation method. Velocity response spectra in these codes are defined on the interface between the surface soil layer and the considerably stiffer base layer characterized by a shear wave velocity of 300 m/s. Spectral values are given in dependence of the eigenperiod of the surface layer, that in turn depends on the nonlinear dynamic soil properties. For the use of such type of specification in the design of near-surface structures like immersed tubes the corresponding values at the soil surface must be determined either by an appropriate seismic site response analysis or by assuming a prescribed depth variation of displacements within the soil layer. The adoption of this methodology and the definition of velocity response spectra for seismic regions other than in Japan is not an easy task: Local seismological experience refers to spectral...
values observed either at the surface of some typical soil formations or on outcropping rock.

A conversion method for deriving the maximum surface seismic displacement from the acceleration spectrum at the surface has been suggested by Sawada et al. (1999). The method also considers dispersion characteristics of the relevant SH-surface waves.

An alternative consists in defining an appropriate acceleration response spectrum at the surface of outcropping rock, transferring the motion first to the interface between soil layer and seismic bedrock (typically characterized by a shear wave velocity of 600 m/s) and subsequent to the level of the underground structure using a standard seismic site response analysis like SHAKE. By means of this procedure, however, one obtains merely the acceleration time history. The displacement time history is then derived by integration applying appropriate base-correction techniques. Often, however, both the peak value and the frequency spectrum of the computed seismic displacement at the surface of the "soft soil" don't correspond to those expected to occur based on earthquake records, thus requiring some adjustment, at least a scaling to a realistic peak value.

The simplest and more robust method consists in first defining a set of realistic acceleration, velocity, and displacement peak values for the prevailing soil conditions based on local seismological experience and then selecting from a database an appropriate earthquake record that meets the specified set of peak values.

Irrespective of the method adopted, the selection of a representative value for the seismic displacement still poses an inherent uncertainty of the seismic deformation method.

SOIL PROPERTIES

Soil stratigraphy can be determined with sufficient accuracy by standard exploration methods such as borings, SPT and/or CPT soundings, whereby CPT are by far more appropriate due to the strong variability and the nature of marine soils.

The determination of dynamic soil properties requires the performance of several in-situ and laboratory tests. Direct measurement of in-situ shear wave velocity offshore is still not an easy task. One promising technique is the Seismic Cone Penetration Test but the method will need some time to attain maturity. Thus, in most cases the designer's only alternative is the use of indirect methods based on empirical correlations between dynamic soil properties and SPT or CPT penetration resistance, such as those summarized by Kramer (1996). Attention shall be paid in clarifying whether the particular empirical relation refers to laboratory or in-situ values.

Variation of shear modulus and damping ratio with shear strain level is obtained from resonant column of cyclic triaxial tests supplemented by data from the literature. A comparison between laboratory and in-situ data is only possible for the small strain shear modulus. The designer shall be aware that laboratory values will in general be lower and match in-situ values only by coincidence. Still there is no consensus among experts on the value of this scaling factor, so that a variation of best-estimate small strain values within a range of at least ±30% is recommended.

Other soil parameters such as static compression moduli, shear strength parameters, and density can be determined by standard procedures. Difficulties and inaccuracies are, however, expected in the estimation of horizontal and vertical permeability values that are needed in the design of liquefaction countermeasures associated with dissipation of earthquake-induced excess pore-water pressure.

GROUND IMPROVEMENT

The major threat calling for ground improvement is related to liquefaction susceptibility of loose sediments in the top layers. Liquefaction potential can be estimated by standard empirical procedures as described in almost all modern seismic codes. A popular remedial measure consists in installing stone columns / gravel drains in a regular grid along the tunnel axis. Such ground structures show a stable behavior with no failure being reported so far. Some analysis methods are available today as presented later in the section describing the Preveza-Aktion tunnel. More sophisticated procedures incorporating coupled dynamic hydraulic-mechanic models but also robust homogenization techniques to determine the nonlinear mechanical properties of the
composite ground are still missing. While the former affect the layout of the stone-column grid system, the latter are crucial to the soil-structure interaction effects and the seismic loads exerted on the tunnel structure.

**ANALYSIS OF TUNNEL RESPONSE**

**Seismic Deformation Method**

The seismic deformation method is usually applied for the analysis. The fundamentals have been established by Newmark (1967) and Kuesel (1969). More recent reviews of available analytical methods are given by St. John and Zahrah (1987) and Hashash et al. (2001). The analysis is carried out for a traveling wave with wavelength $\lambda$, particle displacement amplitude $D_0$, and angle of incidence $\psi$ with respect to the tunnel axis.

Ignoring soil-structure interaction (SSI) effects the maximum axial force is obtained for $\psi = 45^\circ$ while maximum bending moment and shear force occur for $\psi = 0^\circ$. The corresponding values are:

$$M_{\text{max}} = \frac{\pi}{\lambda} E_t A_t D_0$$  \hspace{1cm} (1)

$$M_{\text{max}} = \left(\frac{2\pi}{\lambda}\right)^2 E_t I_t D_0$$  \hspace{1cm} (2)

where $E_t$ is the Young’s modulus of the lining material, and $A_t$ and $I_t$ are the cross section area and moment of inertia of the lining, respectively. Shear forces are determined from bending moments but are omitted here for the sake of brevity.

Note that equations (1) and (2) are valid for harmonic wave motion and imply that the axial force is controlled by particle velocity whereas bending moments are governed by particle acceleration. Thus, both kinematic quantities directly influence the structural tunnel response.

For near surface immersed tubes in soft soils the tunnel structure is stiffer than the surrounding ground, a fact that requires proper consideration of SSI effects. To accomplish this, the tunnel is modeled by an elastic beam supported by a series of idealized springs acting in the longitudinal and transverse direction, respectively, the support being subjected to the free-field motion. The supporting springs, often called moduli of subgrade reaction, are given in units of force/displacement per unit length of the tunnel (kN/m/m). Here we use $K_a$ for the longitudinal and $K_t$ for the transverse direction, respectively. Soil-structure interaction effects are then described by means of reduction factors $R_N$ and $R_M$ that are applied to the sectional forces from equations (1) and (2), respectively:

$$R_N = \frac{K_a}{E_t A_t \left(\frac{2\pi}{\lambda}\right)^2 + K_a}$$  \hspace{1cm} (3)

$$R_M = \frac{K_t}{E_t I_t \left(\frac{2\pi}{\lambda}\right)^4 + K_t}$$  \hspace{1cm} (4)

This procedure corresponds to a conservative design due to the assumptions made that the ground motion is produced by a single shear wave train, that the wave train impinges upon the structure at the most critical angle, and that the displacement amplitude $D_0$ is independent of the wavelength, while in general ground displacement amplitude decreases with the wavelength.

Since the sectional forces depend on the wavelength $\lambda$, we now seek the wavelengths that maximize axial force, and bending moment. For simplicity we assume here that the moduli of subgrade reaction are independent of the wavelength $\lambda$ of the seismic wave. Setting the derivatives of the sectional forces with respect to the wavelength equal to zero and inserting the maximizing wave length into the equations for the sectional forces we obtain the following maximum values

$$\tilde{N}_{\text{max,SSI}} = \frac{1}{4} \left(2K_a E_t A_t\right)^{1/2} D_0$$ \hspace{1cm} (5)

$$\tilde{M}_{\text{max,SSI}} = \frac{1}{2} \left(K_t E_t I_t\right)^{1/2} D_0$$ \hspace{1cm} (6)

It is obvious, that the application of these upper bound values leads to over-conservative
design.

If, instead, one wants to use equations (1) and (2) multiplied by equations (3) and (4), respectively, a realistic value for the wavelength $\lambda$ of the relevant incident seismic wave is needed. Selecting an appropriate value for $\lambda$ is not an easy task. Following various code specifications for underground facilities in Japan and assuming that the subsoil is composed by a surface layer of thickness $H$ and shear wave velocity $V_{s1}$ and a base layer of shear wave velocity $V_{s2}$ the effective wavelength $\lambda$ is given by

$$\lambda = \frac{2 \lambda_1 \lambda_2}{\lambda_1 + \lambda_2} \quad (7)$$

where $\lambda_1 = V_{s1} \cdot T$, $\lambda_2 = V_{s2} \cdot T$, and $T = 4H/V_{s1}$ is the fundamental period of the single layer system.

An alternative has been proposed by Matsubara et al. (1995) assuming surface waves of Love (SH) type propagating in the surface layer. This proposal yields smaller values for the effective wave length $\lambda$.

At this point it should be mentioned that wave propagation theory indicates that propagation of Love (SH) surface waves is only possible in soil deposits with increasing shear modulus with depth (layered or continuously non-homogeneous). Thus, if a ground improvement has been carried out, the higher stiffness of the top layer will prohibit the occurrence of SH surface waves, rendering the above procedure not applicable.

In summary, when using the above formulation based on monochromatic wave loading the sectional forces strongly depend on the assumed wave length for which a reliable and physically sound estimation is still difficult.

**Soil Structure Interaction**

The next source of design uncertainty refers to the value of the spring constants $K_a$ and $K_f$ that account for the effects of tunnel-soil interaction. These spring constants can not be evaluated rigorously for a given soil/tunnel system. Some proposals can be found in the literature but show large scatter, the equations depending on the method adopted and the system geometry. St. John and Zahrah (1987) derived an expression based on the two-dimensional, plane-strain solution to the Kelvin’s problem, i.e. the response of an infinite elastic, homogeneous, and isotropic medium to a static point load. The solution for a sinusoidal load is then determined by an approximate procedure. The resulting equation that applies both for the axial and the transverse horizontal direction reads:

$$K_a = K_f = \frac{16\pi G(1-\nu)}{(3-4\nu)} \frac{d}{\lambda} \quad (8)$$

where $G$ and $\nu$ denote the shear modulus and Poisson’s ratio of the soil, respectively, and $d$ is the equivalent tunnel diameter. Note that in this expression the moduli of subgrade reaction depend on the wavelength, a fact that introduces an additional uncertainty factor.

Based on a synthesis of earlier proposals by various authors the AFPS/AFTES Guidelines (2001) recommend

$$K_a = K_f = G \quad (9)$$

On the other hand, Clough and Penzien (1993) propose

$$K_a = 3G \quad (10)$$

This discrepancy clearly demonstrates the difficulty in assessing realistic values. For critical projects it is recommended to compute the response of the actual soil-structure system by means of a finite-element analysis considering non-linear soil behaviour at least in a static formulation. This procedure automatically yields different values for the axial and the transverse springs.

An alternative, approximate method based on a quasi-static formulation is presented hereafter. We consider a single tunnel element with a given aspect ratio in the order of 5:1 to 10:1. Interaction between the tunnel elements is neglected. Perfect bonding between plate (tunnel) and foundation soil, and linear-elastic theory are assumed. The shear modulus of the foundation soil is taken equal to the one determined from the equivalent non-linear free-field seismic site response analysis of the soil deposit, i.e. it is a reduced shear modulus including the nonlinear effects associated with cyclic loading. With these values the modulus of
subgrade reaction is determined first for a surface foundation. The effects of tunnel embedment are then taken into account leading to an increase of the values of subgrade reaction moduli. An additional increase is associated with the assumption made that the shear modulus degradation of the soil underneath the tunnel will be smaller than under free-field conditions due to the effects of the tunnel-soil interaction.

The equation for the spring constant per unit tunnel length reads:

$$K_m = \alpha_m \cdot \bar{G}_m$$

(11)

where \(m\) refers to the vibration mode, i.e. longitudinal, transverse horizontal, and vertical. \(\bar{G}_m\) is the mean effective shear modulus of the ground in the vicinity of the tunnel element, its value depending on the vibration mode, since vertical vibration modes reach deeper regions than horizontal and rocking ones.

The factor \(\alpha_m\) is composed of three terms:

$$\alpha_m = \alpha_{0,m} \cdot \alpha_{1,m} \cdot \alpha_{2,m}$$

(12)

where \(\alpha_{0,m}\) represents the response of the surface foundation, the factor \(\alpha_{1,m}\) reflects the effect of embedment, and \(\alpha_{2,m}\) the effect of reduced seismic strain amplitude of the near surface ground due to the presence of the rigid tunnel structure.

The formulas given by Gazetas (1991) may be used to determine numerical values for the static stiffness of the rectangular plate, i.e. \(\alpha_{0,m}\) and \(\alpha_{1,m}\). These formulas have been derived by approximating rigorous solutions of the soil-structure interaction problem. For the calculation of the embedment factor the actual sidewall-soil contact surface is used, which consists only of the two sidewalls of the tunnel elements. For horizontal motions the parameter \(\alpha_2\) is roughly in the order of 1.1 to 1.2, e.g. Kuesel (1969) reports that analyses for San Francisco Trans-Bay Tube yielded a 15% reduction of the free-field deformation due to the tunnel rigidity. For vertical motions \(\alpha_2\) will be higher due to the lower level of seismic excitation and accordingly the weaker shear modulus degradation. A reasonable estimate would be an increase of 20% with respect to the value for horizontal motions.

This approach ignores coupled impedances that may become appreciable if foundation embedment is large. In most practical cases, however, these effects are negligible. Note that in the above approximate methodology the subgrade modulus is independent of the wave length.

If no upper bound is imposed on the ground spring constant, large sectional forces will result due to the high stiffness of the support. In reality, slippage between tunnel and soil will occur as soon as the shear stress exceeds a critical value defined by the effective shear strength at the tunnel/soil interface that in turn depends on the normal effective overburden stress. Note that when selecting partial safety factors for this case the mobilized shear strength corresponds to an action since it increases the sectional forces. If the spring constant is assumed to depend on the wave length, the critical value will also vary with wave length requiring some averaging procedure to obtain a single mean value.

**Joints**

Transverse flexible joints of various types are placed between tunnel elements to cope with ground differential settlement and seismic displacements. Gina type rubber gaskets and shear key systems are widely used. Often, prestressed tendons are placed across the joints to prevent opening due to excessive extension that would cause leakage at the joint. Steel joints of meander-like shape have also been developed to absorb large displacements. In the structural analysis joints are modeled by nonlinear springs.

**Numerical Analysis of Global Tunnel Response**

Japanese practice for this type of analysis as described by Kiyomiya (1995) uses a multi-mass spring model whereby the surface layer is assumed to vibrate in shear mode. The surface layer is divided into a number of slices perpendicular to the tunnel axis. Each slice is represented by an equivalent mass-spring system, so that the natural period of the subsystem coincides with the first mode of shear vibration of the slice. Springs connect the tunnel structure to the surrounding ground capturing soil-structure interaction effects. Thus, this approach considers inertia effects and in
In the strict sense does not correspond to a pure deformation method.

An alternative is to use a beam supported by springs and 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. Fig 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, Hamada et al. (1982). A reliable estimate is difficult since the type of wave expected to occur at the tunnel location is in general unpredictable, and available information from other long structures is sparse. For example, if the seismic motion is predominantly produced by Rayleigh waves apparent velocity will be low yielding considerably higher sectional forces and moments. Local incoherencies in the free-field motion will further increase the forces.

There are also some proposals to use for the tunnel design instead of the deformation method inertia-controlled design procedures developed for long-span structures, i.e. multi-support response spectra methods together with some coherency model describing spatial variability of earthquake ground motion, Der Kiureghian & Neuenhofer (1992), Der Kiureghian (1996). Although deformation methods are physically more sound for underground structures (soil-structure interaction controls the response of embedded tunnels) such methods are appealing since they can also capture effects due to rapidly varying soil profiles, a situation that is often encountered along straits. However, in contrast to bridges with a limited number of distinct supports, tunnels have a continuous ground support requiring a close spacing of the idealized springs, a fact that poses some restrictions on the applicability of such methods.

**PREVEZA – AKTION IMMERSED TUNNEL**

**Project Description**

As part of a major highway system the immersed tunnel of the Preveza-Aktion strait on the north-western coast of Greece has been completed in 2002. It was constructed by a 50/50 Joint Venture of British Christiani & Nielsen Ltd and Greek T.E.G.C. (now Empedos S.A.). The design of the tunnel itself has been carried out by Comar Engineers A/S, Copenhagen. GuD Consult GmbH, Berlin (to which the author was affiliated with at that time) provided consultant services to Comar Engineers A/S for geotechnical earthquake design.

The site is located within the area of highest seismicity in Greece. The geotechnical exploration program during the tender phase has revealed an irregular stratigraphy along the tunnel axis and also the existence of loose silty/sandy layers that would liquefy in the event of a major earthquake. The installation of stone columns below the entire length of the tunnel has been proposed as an effective countermeasure.

The immersed part 909 m in length consists of eight precast rectangular concrete elements approximately 60 m to 135 m long casted in a dry dock, towed and immersed in a trench excavated on the sea bottom at a depth of 27 m.
The overall section of the elements is 12.6 m wide and 8.75 m high. On the Preveza side a 509 m long cut-and-cover tunnel was constructed using the top down technique between diaphragm walls. On the Aktio side part of the dry dock was converted into a 150 m cut-and-cover section. Open ramps on both sides connect to the national roads. Fig 2 shows a longitudinal section. Construction details are given by Loukakis and Sørensen (2004).

**Seismic Environment**

A project specific seismological study established design seismic parameters (acceleration, velocity, and displacement), as well as proper artificial accelerograms and response spectra. The selected maximum seismic acceleration of 0.33 g corresponds to a return period of 475 years and is assigned to rock outcrop with shear wave velocity of 600 m/s. The design earthquake with the same return period has a magnitude of 7.3.

**Ground Conditions and Soil Parameters**

The tunnel is constructed in recent marine, lacustrine-marine and swamp deposits. An extensive in-situ and laboratory testing program has been carried out on behalf of the contracting JV both off-shore and on-shore. The bedrock was not identified during the ground investigation that reached an exploration depth of 80 m in the center of the strait. Standard tests included soil classification, soil shear strength, and compressibility as well as in-situ tests such as SPT at all borehole locations. In order to obtain a continuous profile of the soil conditions a large number of CPT tests with pore pressure measurement have been performed on land and at sea. In addition some seismic cone penetration tests have been conducted on land to gain information on the in-situ shear wave velocity.

The layering sequence was taken from the borehole logs and verified from the results of the CPTU tests by adopting empirical charts. Agreement between borehole logs and CPTU interpretation was very good.

Cyclic soil resistance of sandy-silty and sandy soil samples was determined by stress-controlled cyclic simple shear tests. Various combinations of cyclic shear stress and effective consolidation stress have been investigated.

A large number of resonant column tests have been carried out to determine dynamic soil stiffness and soil damping. Particular attention has been paid to the nonlinear properties of the medium plasticity clays encountered along the tunnel alignment.
Results for this type of soil are summarized by Vrettos and Savidis (1999). Various design equations available in the literature were additionally used to establish the design values. A typical profile is given in Fig 3.

Liquefaction potential was assessed by means of the classical total stress method, Seed and Idriss (1982). The cyclic stress developed in the soil due to the earthquake shaking was calculated by SHAKE at representative locations along the tunnel axis. The cyclic resistance of the soil was determined on the basis of field tests, i.e. SPT blow count number and/or CPT tip resistance, the results of cyclic tests being used mainly as supporting evidence. Appropriate scaling factors were applied to account for the earthquake magnitude and for the effects of overburden stress. The screening procedure yielded a liquefiable zone of approx. 15 m thickness along the entire length of the immersed and most of the cut-and-cover parts of the tunnel.

Design of Stone Columns

The simplified method proposed by Seed and Booker (1977) was used for the design. A maximum allowable pore pressure ratio was specified equal to 0.6. The analysis yielded for a column diameter of 0.60 m a minimum center to center spacing between columns in a square grid equal to 1.80 m.

The grading of the stone column material is selected so as to satisfy two requirements:

i) the difference in permeability of the drain material and the surrounding soil should be large enough to permit a hydraulic gradient, and
ii) the particle sizes of the drain material should be small enough to prevent clogging. Usually, in geotechnical practice the criteria suggested by Terzaghi are applied. However, they would lead to a non-practicable solution. Some laboratory tests indicate that for dynamic loading a coarser drainage material is needed to guarantee effective permeability and a less restrictive lower limit for clogging may be applied. The corresponding design equation suggested by Saito et al. (1987) was adopted for the design.

Soil-Structure Interaction

The stiffness of the composite ground system was determined by applying the concept of stress concentration. The simplified analysis proposed by Priebe (1995) was adopted. The method was developed primarily for static conditions, but was applied here also for dynamic loading by using the respective soil parameters properly adjusted to account for the nonlinearity of soil behaviour. Dynamic shear modulus values for the gravel material have been derived from data in the literature. The different behaviour of gravel and natural soil material with respect to shear modulus reduction with increasing shear strain has been considered by means of an approximate averaging procedure. The profile of the improved ground has then been subjected to the design earthquake and analysed by SHAKE.
yielding the effective shear modulus depth profile at each specific location along the tunnel axis. A detailed description is given by Vrettos and Savidis (2004).

The ground springs needed as input in the global tunnel response analysis have been determined by means of the procedure described in the previous section of the paper.

**Monitoring**

A structural and seismic monitoring system has been installed after completion of works. On August 14, 2003 a magnitude 6.4 earthquake struck close to the island of Lefkada in the Ionian Sea. The epicenter was located about 25 km south-west of the site. Records obtained on the tunnel are presently analysed. No damage to the tunnel structure was observed.

**CONCLUSIONS**

State-of-the-art methods are available for analyzing immersed tunnels. Efforts to improve accuracy will concentrate on design issues referring to the modeling fidelity with respect to the seismic motion impinging on the tunnel and the dynamic soil–structure interaction effects. The advancement of high performance computers will facilitate the application of finite-element codes with proper consideration of wave radiation to compute the response of the entire embedded structure.

**REFERENCES**


