Seismic Design of “Qaf Murrizi” Tunnel: The Design of Tunnels in Heterogeneous Areas

Ervin Paci, Hektor Cullufi and Arben Dervishaj

Civil Engineering Faculty, Polytechnic University of Tirana, Tirana 1001, Albania

Abstract: Recently based in the performance of tunnels under seismic movements a good progress is made in the seismic design of tunnels. The majority of problems in the tunnel structures take place in near fault conditions and in the case of great variation of rock properties. Not having a previous experience in the seismic design of the tunnels that pass through tectonic zones with very large heterogeneity (strong rock-soil or very poor rock that behaves like soil) this article presents the general aspects of seismic calculation of tunnels and application in a specific example. The article presents the methods of seismic input modeling, design and particularities of numerical calculations. The geological conditions in which the case study tunnel will be constructed are very common in Albanian territory and represent one of the most difficult cases for the construction of road tunnels. The applied approach for using the longitudinal models for generating time histories of acceleration that can be used as input for transversal models is very simple and will help the designers for the seismic analysis of other tunnels that will be constructed in Albania.

Key words: Seismic design, tunnels, Albania, heterogeneous areas.

1. Introduction

Nowadays, due to the spread of these kinds of works, seismic response of tunnels and underground works are part of the concern of the population, both for transport and for other purposes. An eventual damage of the tunnel brings not only engineering and financial problems for the structure but affects the whole performance of the network infrastructure of a country [1, 2]. The calculation of underground tunnels is still one of the most difficult problems of engineering, especially in very seismic regions. The tunnels as underground structures must withstand both static and seismic loads. Seismic response of tunnels and underground structures is generally very different from that of the above surface structures as the total mass of the structure is usually small compared to the mass of the surrounding ground and terrain confinement provides high values of geometric damping. Because of these two effects an underground structure reacts in accordance with the surrounding

terrain response without resonance.

Historically underground tunnels have experienced a lower rate of damage compare to structures on the surface, however, same recent major earthquakes in major urban centers and mountain areas have resulted serious damage of underground structures especially in heterogeneous areas that have major contrast in physical, mechanical properties (strong rock-soil or very poor rock that behaves like soil). The first indications for developing more profound analysis for the seismic design of tunnels are taken from evaluation of seismic performance of existing tunnels.

This article presents a simple practical approach for the design of tunnel linings using longitudinal models to generate the seismic input for lateral models. In our application, we have try to estimate the shear deformation induced by vertical shear waves in tunnel lining considering soil-structure interaction and the influence of strong heterogeneities in the modification of time histories of acceleration. The approach consider two of the main factors that contribute to ground strains due to wave propagation, wave passage effects and site effects. The third factor is spatial incoherence.

Corresponding author: Ervin Paci, lector, research field: tunnels. E-mail: ervin.paci@fin.edu.al.

Spatial in coherence contributes significantly for the estimation of ground strains if the motion is predominated by body waves. If the motion is predominated by surface waves (shallow tunnels), the propagation effects overshadow those of incoherency [3]. According to AFTES (French Tunneling Association) recommendation, the spatial incoherence must be considered only for long tunnels ($L > 5$ km) but recently it is documented that spatial incoherency can contribute to the estimation of strains for an additional factor of 2-3, especially in the presence of strong heterogeneities [3]. The application presented in this paper that we have not considered the spatial incoherence of motion which will be one of our future objectives.

2. Criteria for Damage Classification

Several authors have made different classifications. Tables 1 and 2 give two of the most important: Classification Huang et al. (1999) and Classification FEMA 1999 [2].

3. Calculation Methods

The seismic design of tunnels can be made by

different methods. The choice of appropriate method depend upon several factors such as: tunnel importance, complexity of geological structural, available geotechnical and seismicity parameters, etc..

Calculation methods are Refs. [4-6]:

- pseudo-static analytical analysis;
- simplified dynamic analysis (beam on springs);
- detailed numerical dynamic analysis.

3.1 Pseudo-Static Analytical Analysis

These methods for continuous medium consider different modes of deformation (compression/extension; longitudinal bending; ovaling of the cross section) for longitudinal waves, transverse, and Reyleigh wave taking into account the soil-structure interaction by the flexibility coefficients.

In summary the strains, according to Wang [6] are given in Table 3:

In the case of taking into account, the soil-structure interaction are introduced the coefficients of relative flexibility and relative stiffness (compressibility). This interaction [6] should be taken into account in cases where the relative flexibility coefficient is $F < 20$. Simple equations that are derived from numerical

Table 1 Classification Huang et al. (1999).

Tunnel classification	Class	Damage level	Damage description	Traffic strategy
No immediate danger	A	No damage	No damage detectable by visual inspection	Normal operation
		Slight	Light damage detected on visual inspection, $w < 3$ mm, $l < 5$ mm; No effects on traffic	Operable with regulations
Dangerous	B	Moderate	Spalling, cracking of linings $w > 3$ mm, $l > 5$ mm, exposed reinforcement, displacement of segmental joints, leaking of water; Some disruption to traffic	Operable with regulations
Very dangerous	C	Severe	Slope failure at openings, collapse of main tunnel structure, up heave or differential movement of road and road shoulder, flooding, damaged ventilation and lighting system in long tunnels; Total disruption of traffic	Not operable

Table 2 Classification FEMA 1999.

Damage classification	Damage level	Damage description
ds1	No damage	
ds2	Slight/minor damage	Minor cracking of the tunnel liner, damage requires no more than cosmetic repair and some rock falling, or by slight settlement of the ground at a tunnel portal
ds3	Moderate damage	Moderate cracking of the tunnel liner and rock falling in inlined tunnels
ds4	Extensive damage	Major ground settlement at a tunnel portal and extensive cracking of the tunnel liner
ds5	Complete collapse	Major cracking of the tunnel liner, which may include possible collapse

Table 3 Strains for different type of waves [6].

Wave type	Max longitudinal strain	Max normal strain	Max shear strain	Max curvature
Wave-P	$\varepsilon_1 = \frac{V_P}{C_P} \cos^2 \phi$	$\varepsilon_n = \frac{V_P}{C_P} \sin^2 \phi$	$\gamma = \frac{V_P}{C_P} \sin \phi \cos \phi$	$K = \frac{a_P}{C_P^2} \sin \phi \cos^2 \phi$
	$\varepsilon_{bn} = \frac{V_P}{C_P} ; \phi = 0^\circ$	$\varepsilon_{nm} = \frac{V_P}{C_P} ; \phi = 90^\circ$	$\gamma_m = \frac{V_P}{2C_P} ; \phi = 45^\circ$	$K_m = 0.385 \frac{a_P}{C_P^2} ; \phi = 35^\circ 16'$
Wave-S	$\varepsilon_1 = \frac{V_S}{C_S} \sin \phi \cos \phi$	$\varepsilon_n = \frac{V_S}{C_S} \sin \phi \cos \phi$	$\gamma = \frac{V_S}{C_S} \cos^2 \phi$	$K = \frac{a_S}{C_S^2} \cos^3 \phi$
	$\varepsilon_{bn} = \frac{V_S}{C_S} ; \phi = 45^\circ$	$\varepsilon_{nm} = \frac{V_S}{2C_S} ; \phi = 45^\circ$	$\gamma_m = \frac{V_S}{C_S} ; \phi = 0^\circ$	$K_m = \frac{a_S}{C_S^2} ; \phi = 0'$
Wave-Rayleigh	$\varepsilon_1 = \frac{V_{RP}}{C_R} \cos^2 \phi$	$\varepsilon_n = \frac{V_{RP}}{C_R} \sin^2 \phi$	$\gamma = \frac{V_{RP}}{C_R} \sin \phi \cos \phi$	$K = \frac{a_{RP}}{C_R^2} \sin \phi \cos^2 \phi$
	$\varepsilon_{bn} = \frac{V_{RP}}{C_R} ; \phi = 0^\circ$	$\varepsilon_{nm} = \frac{V_{RP}}{C_R} ; \phi = 90^\circ$	$\gamma_m = \frac{V_{RP}}{2C_R} ; \phi = 45^\circ$	$K_m = 0.385 \frac{a_{RP}}{C_R^2} ; \phi = 35^\circ 16'$

analysis are given in the case of strong heterogeneities [3, 5]. In these cases, the design must be based upon numerical analysis.

3.2 Simplified Dynamic Analysis (Beam on Springs)

Modeling as a beam supported on springs is one of the most practical methods of solving the problem. In this type of analysis, we can model both transient and non-transient ground movements. In general, the model form will have the appearance shown in Fig. 1 where springs on one side have a shift from the fault displacement. The seismic loading can be applied by time history of acceleration [7]. In this case, springs should be associated with dampers.

3.3 Detailed Dynamic Analysis (Numerical Methods)

Numerical methods are in many cases the only methods by which you can get the solution. Numerical methods can simulate the seismic motion and dynamic soil-structure interaction in any condition. The main numerical methods are “finite element”, “finite difference” and the less used “boundary element” method. The numerical methods recently have made significant progress in wave propagation solution techniques, boundary simulation, material behavior modeling and soil structure interaction allowing them to solve the complex problem from the source (far field) to the analyzed structure including all the possible soil-rock conditions.

We emphasize that the parameters used in these methods can not always be assessed in the laboratory or in the site through various tests and therefore they must be validated through real physical models or

during the tunnel construction. After the measurement of the performance of structures through the “back analysis” procedures, we must re-run the analysis with the new parameters. Influence of each parameter is tested by sensitivity analysis.

In Finite element methods, numerical modeling of wave motion can be performed by means of direct integration in time or through solutions based on frequency calculations. The latter due to their simplicity is widely used in the analysis of structures, but in modeling of ground wave motion are not very suitable. Numerical modeling through time integration allows easier implementation of the laws of advanced material behaviors that take into account the time dependency and material nonlinearity. In all these formulations, we should take into account a number of factors which will influence the numerical analysis such as: formulation of the mass matrix, the determination of the damping matrix ($|C| = a_R|M| + b_R|K|$), boundary conditions, critical time integration step, finite element size, etc., the damping coefficients must be chosen to match 1D wave propagation in elasto-plastic soil columns. The boundaries can be modeled by adaption of material properties of boundary elements, infinite elements and viscous

Fig. 1 Beam on springs model.

boundaries. The last two methods are used mostly and the response of the system in both cases is the same [8]. Details are given in the application case.

4. Seismic Input

The analysis will be made in time domain, so the best way for representing the seismic input load is by time histories of acceleration. Different types of time histories can be used for the definition of the seismic input:

- artificial accelerograms compatible to a reference response spectrum;
- artificial accelerograms compatibles with some seismogenic constraints such as magnitude and epicentral distance;
- synthetic accelerograms generated through complex mathematical models of seismic source and propagation phenomena [9];
- real accelerogram selected from earthquake strong motion databases such as the ESD (European Strong Motion Database), the PEER (Pacific Earthquake Engineering Research Centre), etc.;
- artificial accelerograms generated through hybrid procedures.

For seismological and geotechnical applications real accelerograms are preferred because they are more realistic for frequency content, number of cycles, correct correlation between the vertical and horizontal components of ground motion and for the energy content in relation to the seismogenic parameters (EN 1998-1-5, 2004; Bommer and Acevedo, 2004). However, in order to use a real accelerogram, we should take into consideration the near-fault conditions. For near fault conditions, it is required for the time histories to include directivity effects and fling step [9], in other words, they should refer to real, near-field earthquakes.

5. Seismic Design of “Qaf Murrizi” Tunnel

5.1 Introduction of Qaf Murrizi

The Qaf Murriz tunnel is part of a new road from

Tirana to Dibra in the center of Albania. The position of the tunnel is shown in Fig. 2. The tunnel has two lanes with a cross section of 105-120 m² and a length of 2.6 km. It passes through a very disturbed tectonic zone, where the carbonatic rocks (Cretaceous) of Krasta-Cukali zone over thrust the flysch rocks (Paleocene) of Kruja-Dajti zone. It has a closed shape in the flysch zones with a 90 cm thick inverter in the most difficult part. The lining thickness is from 50 cm to 90 cm for the section in the flysch and 35-60 for the section in the carbonatic rocks.

5.2 Seismic Input

It is well known and widely accepted that due to many uncertainties of a seismic event, the best way for seismic input evaluation is performing probabilistic seismic hazard analysis. Based upon the results of this analyze, we can have the desired parameters of seismic input (ground motion parameters) in any point. For numerical analysis, we need the time histories of this motion that are compatible with the design spectra given by the codes. We have evaluated the ground motion peak acceleration through a hazard analysis with life time of structure $T_L = 100$ years and compare the results by a simplified approach taking the value of PGA (peak ground acceleration) from expression [10]:

$$a_{gdes} = g_i a_{gR}$$

where, g_i is the importance factor, a_{gR} is the peak ground acceleration foran reference action with

Fig. 2 Position of “QafMurriz” tunnel.

associated reference probability of exceedance $P_{NCR} = 0.1$ in 50 years.

According to EC8 seismic hazard should be given only with one parameter a_{gR} on ground type “A” that correspond to rock or rock like geological formations, including 5 m weak formations (soil) at surface. The values of a_{gR} are taken from the probabilistic hazard map of PGA given by “Geoscience Institute” [11]. The return period of the reference event is $T_R = 475$ years corresponding to a life time of 50 years.

For important buildings structures, lifetime is 100 years and consequently the return period will be $T_R = 949$ years [1, 2, 12]. For evaluation of Hazard, we have used the computer program CRISIS 2007. Since the data for seismic parameters must be derived from analysis of earthquakes catalog, we have accepted the zones (seismic sources) characterized of the data given by “Geoscience Institute” [11]. The most influential zone for the site where the tunnel will be constructed is the Pan Adriatic zone with the lower and upper limit of magnitude $m_o = 4.5$ and $m_u = 6.9$. Gutenberg-Richter model parameters of seismic event recurrence for this zone are $b = 1.031$ [11], $a = 4.599$ (number of events per year that have the magnitude $> m_o$). Division of the

zone was made with 10 km grid step and the grid is corrected considering the points where real earthquakes have been registered. We consider also as seismic source one tectonic line and the seismicity model for this tectonic lines are taken the characteristic earthquake model (not Gutenberg-Richter) with magnitude $M = 6.5$ [11].

After the evaluation of the PGA, the seismic input is taken as two real accelerograms from ESD database using the software Rexel [13] and scaling the accelerograms with the software Seismosoft [14]. The chosen accelerogram according to EC8 spectra is compatible with PGA taken from probabilistic seismic hazard analysis for $T_R \approx 950$ years. For taking into account the convulsion from surface to the depth in which is generated the input in the model a scaled factor is applied to the accelerograms. The selected accelerograms are given in Figs. 3 and 4.

The second accelerogram is chosen to have the same specter of acceleration but different time duration and maximum acceleration.

5.3 Numerical Calculations

For numerical calculation we have used the software

Fig. 3 Time history of acceleration in x, y and z direction for the Friuli earthquake.

Fig. 4 Time history of acceleration in x, y and z direction for the Montenegro earthquake.

“PLAXIS 2D” [15].

The longitudinal model is used to calculate axial strains, deformation and to generate the seismic input for lateral model considering soil-structure interaction and the influence of strong heterogeneities in the modification of time histories of acceleration. The model is taken 2,800 m long and at maximum height 1,000 m. It has 1,668 elements with average length 48 m ($< 1/4$ - $1/8$ wave length) [8, 15] with coarse mesh triangular 15 node finite elements. In PLAXIS, any material model can be used in dynamic calculations but for large scale models even the classic elasto-plastic material model (Mohr-Coulomb) gives accurate results because local site effect that can be modeled only with advanced material models predominates in near surface layers that represent only a very small part of the model. The Mohr-Coulomb material model parameters are considered constant and are estimated from site tests, laboratory tests and geophysical tests. The tunnel lining is modeled as elasto-plastic beam element with normal and bending stiffness (EA, EI) of the concrete lining shape for two typical cross section. The soil tunnel interaction is modeled by interface elements with

assumption of no sliding among them. The construction sequences and water are not taken in consideration. The model boundaries in Plaxis are created by absorbent viscous boundaries that give the same results as infinite element boundaries [8]. In our model, the relaxation coefficients that determined the normal (C1) and shear (C2) stresses absorbed by the damper are taken 1 and 0.25 [16]. The geological formations in our case are rock-poor rock and so the material damping Rayleigh coefficients are taken the default values ($a_R = b_R = 0$). The longitudinal model is shown in Fig. 5.

By choosing different point for graphical presentation, we take the time histories of acceleration that serves as input for transversal model seismic calculation.

We have made five transversals models, two in deep flysch formations, one in carbonatic formations, one in contact zone and one in right portal zone. The flysch model for the maximum depth (280 m) is represented in Fig. 6. This model has 359 finite elements with average length 38 m ($< 1/4$ - $1/8$ wave length) with medium fine mesh triangular 15 node finite elements

Fig. 5 Calculation model in longitudinal direction (Flysch formation are shown with green color, and carbonatic formation are shown with light brown color).

Fig. 6 Transversal model of the tunnel.

near the tunnel lining.

The most suitable material model in dynamic analysis in Plaxis is Hardening soil with small strain stiffness (HS small) because it can simulate hysteric behavior of the ground and stress level dependency of stiffness. In our case due to lack of appropriate data for generating, the curves of shear modulus dependency from dynamic cyclic small shear strains, we have used Mohr-Coulomb material model with the material damping Rayleigh coefficients values $a_R = 0.08$, $b_R = 0.0042$. For small models because we have only considered wave passage effect and not damping in longitudinal model, we must take non zero values for Rayleigh coefficients. The values of Rayleigh coefficients are taken by assuming the same damping

(z) for natural soil frequency (w_1) and input motion predominant frequency (w_2) [17].

All other modeling aspects are the same as in longitudinal model.

For comparison, we have done also a beam on spring model with longitudinal “ k_l ” and transversal “ k_t ” springs every 50 m. Spring behavior law is taken from AFTES [1] recommendation. In our case, the tectonic fault can not be directly linked with seismic permanent movements, so we have considered only the transient movements [4, 7].

6. Results

Each model is taken by strains, deformations, stresses and internal forces in the structure. In

longitudinal model, we can control only global stability [4], strains and stresses in surrounding terrain and tunnel deformations. The time histories of acceleration taken from the longitudinal model in the case of free field propagation and soil-structure interaction are shown in Fig. 7. A difference of about 20% in the peak values is seen between the two cases. The vertical deformation in the contact area is shown in Fig. 8. The left portal zone is very close to the boundaries, has generated unrealistic deformation and strains and is not taken in consideration. In transversal models, we can take directly internal forces in tunnel lining. The

diagram of moment for the transversal section at the end of seismic load phase for flysch zone is shown in Fig. 9. The maximal enveloped moments values are about 300 KNm. Additional forces that will be applied in the structure of the tunnel are up to 75-85% of static forces in the flysch zone and 10-12% of static forces in the limestone zone.

7. Conclusions

From analyzing the time histories of acceleration, it is seen that differences for strong heterogeneities contribution cannot be evaluated accurately in depth

Fig. 7 Comparison of time histories of acceleration for free field (blue) and soil structure interaction (red) conditions.

Fig. 8 Tunnel deformation in the contact zone. Max value in the contact zone 5.7 cm.

Fig. 9 Moments in tunnel lining for transversal section at the end of load phase. Max moment 80 kNm at the end of left lateral side.

for the case of strong rock over weak rock (reverse fault conditions). The values of the peak strains in different points of longitudinal model do not match the values taken from the analytical equation given by O'Rourke and Liu (1999). The differences can be estimated with accuracy only for near surface tunnels and normal very strong heterogeneities (soil over strong rock). For “in depth” tunnels a 2D numerical longitudinal model is necessary to consider all the effects and generate the appropriate time history of acceleration. The displacement profile of tunnel axis gives the realistic shape of deformation [18] and the results for displacement and internal forces obtained from the analysis that are consistent with theoretical expectations [19]. In longitudinal direction, numerical model gives much less internal forces than predicted with pseudo-static and simplified dynamic models due to more accurate soil-structure interaction estimation. For the design practice for the tunnels in Albania ($M_w < 6.5$), we concluded that for the design earthquake: in good rock formations additional seismic forces are very small [19] and the tunnels will not have any damage (ds1); in soft or weak formations additional seismic forces influence the design of lining but are within acceptable values for normal dimensions of road tunnels and the tunnels will have slight damages (ds2); at the contact zone of strong with weak

formations (sheared flysch) additional interior forces and permanent deformations are greater and special joint details are needed to withstand [19] them. If such measures will not be applied, moderate to extensive damage must be expected (ds3-ds4).

The reverse fault conditions (rock upon soil) analysis for adapting the analytical equations of peak strain will be one of our future objectives of research.

References

- [1] Earthquake Design and Protection of Underground Structures, AFPS/AFTES (French Association for Seismic Engineering/French Tunneling Association), French, 2001.
- [2] Guidelines for Design of Buried Steel Pipe, ALA (American Lifelines Alliance), ASCE-FEMA (American Society of Civil Engineers-Federal Emergency Management Agency), 2001.
- [3] A. Zerva, Transient seismic ground strains: Estimation, modeling and simulation, in: Proceedings of the Workshop on the Effect of Earthquake-Induced Transient Ground Surface Deformations on at Grade Improvements, USA, 2003.
- [4] M. Corigliano, C.G. Lai, G. Barla, Seismic response of rock tunnels in near-fault conditions, in: 1st European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland, 2006.
- [5] L. Scandella, Numerical evaluation of transient ground strains for the seismic response analysis of underground structures, Ph.D. Thesis, Department of Structural Engineering, Polytechnic University of Milan, Italy, 2007.

- [6] J.J. Wang, Seismic Design of Tunnels: A State of Art Approach, Monograph 7, Parsons Brinckerhoff Quade and Douglas Inc., 1993.
- [7] G. Bouckovalas, Geotechnical Division, Lectures on Geotechnical Earthquake Engineering, School of Civil Engineering, National Technical University of Athens, 2010.
- [8] M. Hariri-Ardebili, A comparative study of seismic stability of coupled arch dam foundation reservoir systems using infinite elements and viscous boundary models, *International Journal of Structural Stability and Dynamic* 13 (6) (2013) DOI: 10.1142/S0219455413500326.
- [9] Y. Hisada, J. Bielak, A theoretical method for computing near-fault ground motions in a layered half-spaces considering static offset due to surface faulting—With physical interpretation of fling step and rupture directivity, *Bulletin of Seismological Society of America* 93 (3) (2003) 1154-1168.
- [10] Eurocode 8, Design of Structures for Earthquake Resistance, Part 4: Silos, Tanks and Pipelines, The European Union Per Regulation, 2004.
- [11] N. Kuka, E. Sulstarova, L. Duni, S. Aliaj, Seismic hazard assessment of Albania using the spatially smoothed seismicity approach, in: *International Conference in Earthquake Engineering*, Skopje, Aug. 26-29, 2003.
- [12] Eurocode 7, Geotechnical design, European Committee for Standardization, 2006.
- [13] I. Iervolino, C. Galasso, E. Chioccarelli, *Rexel 3.3 (beta): Computer Aided Code-Based Real Record Selection for Seismic Analysis of Structures*, Dipartimento di Ingegneria Strutturale, Università degli Studi di Napoli Federico II, Naples, Italy, 2008-2013.
- [14] Seismosoft Programs Suite, Earthquake Engineering Software Solutions, Seismosoft Ltd., 2012.
- [15] Plaxis BV, Plaxis 2D 2011 Dynamic module, Finite Element Code for Geotechnical Engineering Analysis, 2011.
- [16] C. Visone, E. Bilotta, F.S. de Magistris, Remarks on Site Response Analysis by Using Plaxis Dynamic Module, *Plaxis Bulletin* No. 23, 2002, pp. 13-17.
- [17] D. Park, Y.M.A. Hashash, Soil damping formulation in nonlinear time domain site response analysis, *Journal of Earthquake Engineering* 8 (2) (2004) 249-274.
- [18] D. Park, M. Sagong, D. Kwak, C. Jeong, Simulation of tunnel response under spatially varying ground motion, *Soil Dynamic and Earthquake Engineering* 29 (2009) 1417-1421.
- [19] M. Corigliano, L. Scandella, G. Barla, C.G. Lai, R. Paolucci, Seismic analysis of deep tunnels in rock: A case study in southern Italy, in: *4th International Conference on Earthquake Geotechnical Engineering*, Thessaloniki, Greece, 2007.