Seismic performance of a major twin tunnel with unreinforced concrete lining: Expert witness study for the needs of arbitration

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Seismic performance of a major twin tunnel with unreinforced concrete lining: Expert witness study for the needs of arbitration

I. Anastasopoulos
ETH Zurich, Switzerland

G. Gazetas
National Technical University of Athens, Greece

ABSTRACT: The paper studies the seismic performance of a major twin tunnel, with unreinforced concrete (UC) lining. The work is part of an expert witness study for an arbitration. For confidentiality reasons, the key conclusions are presented, without revealing sensitive information. Situated in Greece, the tunnel is part of a major motorway, under construction at the time of the dispute. With a total length of almost 6 km, and overburden ranging from 30 to 300 m, the tunnel was constructed by conventional means of drill and blast. After its completion, the seismic safety of the UC lining was disputed by the owner. An expert witness of the owner assessed the adequacy of the UC lining through pseudo-static "pushover" analysis. Assuming elastic response, the earthquake-induced tensile stresses were found to exceed the tensile strength of concrete, concluding that the UC lining is insufficient. We were subsequently appointed by the contactor to conduct a more sophisticated expert witness study. The problem was analyzed with the finite element method, employing a nonlinear model for the rock mass, and the concrete damaged plasticity (CDP) model for the UC lining. An initial elastic pushover analysis, using the parameters as the owner's expert witness, gave compatible results. Then, nonlinear pushover analysis was conducted, with and without the surrounding rock mass. Without the beneficial compression offered by the rock mass, the UC lining fails at drift $\delta = 1.7$ mm. Confined by the surrounding rock mass, it is capable of sustaining much larger deformation without collapsing. Finally, nonlinear dynamic time history analyses revealed that tunnel damage is a function of shaking intensity and initial loading. Even with conservative assumptions, no collapse mechanism would develop and the residual drift is negligible.

1 INTRODUCTION

Situated in Greece, the studied tunnel consists of two branches and has a total length of 6 km and 300 m maximum overburden. According to the geological studies, it passes through the "Ossa" mountain rockmass, crossing through crystalline limestones and phyllites. Both formations are dipping towards the W to NNW with dip angles ranging from 10° to 30°. At specific locations the rock formations are overlain by soil-like rocks (scree deposits, weathered and disintegrated phyllites, and talus cone deposits). Two main fault systems are encountered in the broader area, one with a N-S to NW-SE strike, and another with an ESE-WNW strike.

The tunnel was constructed with conventional excavation of drill and blast. The temporary support consisted of shotcrete, combined with additional stabilization measures such as rock bolts, anchors, steel sets, spiles or steel tubes, depending on the geological conditions. The final support consists of an unreinforced concrete (UC) lining in areas where the rockmass is of good quality. However, the lining is reinforced in areas of poor ground conditions and/or low overburden, as well as near the portals and transverse connections.

After construction, the adequacy of the design was disputed by the State authorities, raising questions on the adequacy of the unreinforced concrete (UC) lining, especially for seismic

DOI: 10.1201/9781003348030-357

loading. Assigned by the state, a structural engineering expert (SEE, not disclosed due to confidentiality) assessed the adequacy of the UC lining. Seismic loading was simulated through pseudo-static "pushover" analysis, imposing a seismic shear strain at the top of the model. Assuming *elastic* UC lining response, the earthquake-induced tensile stresses were found to exceed the design tensile strength of concrete (f_t = 1.42MPa), leading to the conclusion that the UC lining is insufficient for seismic loading.

The scope of our study, which was assigned to us by the contractor (not disclosed due to confidentiality), was to assess the *seismic performance* of the UC lining of the tunnel using state-of-the-art procedures. Its adequacy against other load combinations, such as wedge failure, does not fall within the scope of this study.

2 SEISMIC HAZARD

According to the Greek Seismic Code (EAK 2000), the project lies within low seismicity Zone I, with a PGA of 0.16g (10% exceedance probability in 50 years). Based on the geology of the area, the site is classified in soil category A. Although a special seismo-tectonic study was not required according to EAK 2000, as part of the Technical Dispute two such studies were conducted (not disclosed due to confidentiality). According to the first one, a number of near-field possibly active faults should be investigated. However, according to the second study, their characterization as active is not adequately proven, and regardless of its validity, being at a distance of 15 km from the tunnel, these faults cannot be categorized "near-field". Hence, it was concluded that there is no active fault in the immediate vicinity of the tunnel, and no special study is required.

With respect to the estimation of seismic hazard, both seismo-tectonic studies applied state-of-the-art widely accepted methodologies. Their main difference lies in the assumed return period. Considering the project of "great significance", the first study assumed a return period of 4950 years (2% exceedance probability in 100 years), resulting to a PGA of 0.35 g. For the Maximum Design Earthquake (MDE) or Maximum Credible Event (MCE), the second study assumed a return period of 975 years (5% exceedance probability in 50 years), resulting to a PGA of 0.23 g.

Although both assumptions may be considered valid, in view of international engineering practice, a return period of 4950 years is considered overly-conservative. We briefly mention few examples of tunnels of "great signifficance" in areas of high seismicity: (1) the BART immersed tunnel in San Fransisco (an area of very high seismicity) has been designed for a DBE (Design Basis Earthquake) with a return period of 1000 years (Sun et al. 2008); (2) the Fort Mason tunnel, also in San Francisco, has also been designed for a DBE with a return period of 1000 years (Zafir et al. 2006); and (3) the Aktion–Preveza tunnel in Greece has been designed for a DBE with a return period of 949 years (Vrettos & Savvidis, 2004).

Based on such evidence, the return period of 975 years proposed by the second study is considered more rational. It should be noted, however, that the level of seismic safety is a decision of the owner, but at a much earlier stage, before the design – and definitely before construction – of the tunnel. In the absence of any statement, a return period of 975 years is considered rational, if not over-conservative. It should also be noted that the value given by EAK 2000 (0.16 g in this case) correspond to a return period of 475 years (10% exceedance probability in 50 years). For "important structures" a 30% increase is foreseen, implying a seismic coefficient of 0.16g x 1.3 = 0.21g, which is very close to the value of 0.23 g proposed by the second seismo-tectonic study.

3 SEISMIC SHAKING SCENARIOS

As discussed later on, the seismic performance of the tunnel is analyzed: (a) through pseudo-static "pushover" analysis, and (b) through dynamic time-history analysis. For the latter, appropriate seismic excitations are required. These are computed on the basis of acceleration response spectra for the study area, which are computed herein on the basis of the second seismo-tectonic study, combined with broadly accepted empirical attenuation relations. This is accomplished through a two step procedure:

Step 1: Application of attenuation relations (Ambraseys et al. 1996; Abrahamson & Silva 1997; Boore et al. 1997; Spudich et al. 1999) to derive acceleration spectra. Such attenuations correlate source characteristics to site characteristics (distance and ground conditions). The seismic hazard is assessed deterministically (DSHA), considering the worst case scenario, based on the second seismo-tectonic study: an earthquake of magnitude M_S 6.5 at 15 km distance from the tunnel.

Step 2: Mathematical modulation of real accelerograms recorded during earthquakes in Greece to realistically modify them, so as to become compatible with the elastic response spectra of the previous step. The acceleration time histories computed in this step are then used as input to the finite element (FE) model in the subsequent nonlinear time-history analysis.

For the worst-case scenario (M_S 6.5 at 15 km distance), the four attenuation relationships were used to estimate the response spectra and the envelope is shown in Figure 1a. The DSHA-compatible response spectrum exceeds the EAK (2000) design spectrum for the periods of interest ($T \leq 0.5$s). Different records were used to match the elastic response spectrum. An example of such a spectrum compatible record, based on the 1986 M_s 6.0 Kalamata earthquake, is shown in Figure 1b.

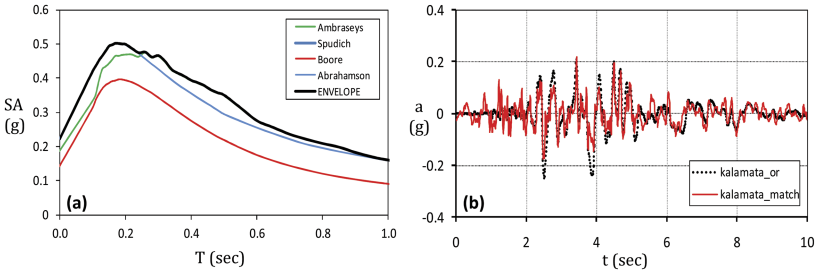

Figure 1. (a) The four computed acceleration response spectra and their envelope, used as target spectrum; and (b) example of modified DSHA spectrum compatible motion, based on the Kalamata 1986 record.

4 ANALYSIS METHODOLOGY

The seismic performance of the tunnel is analyzed employing the FE method, assuming plane-strain conditions, modeling both the rock-mass and the UC lining with quadrilateral continuum elements. As depicted in Figure 2, the FE mesh is finer along the periphery of the tunnels. The lining is connected to the surrounding rock-mass through interface elements, allowing separation and sliding.

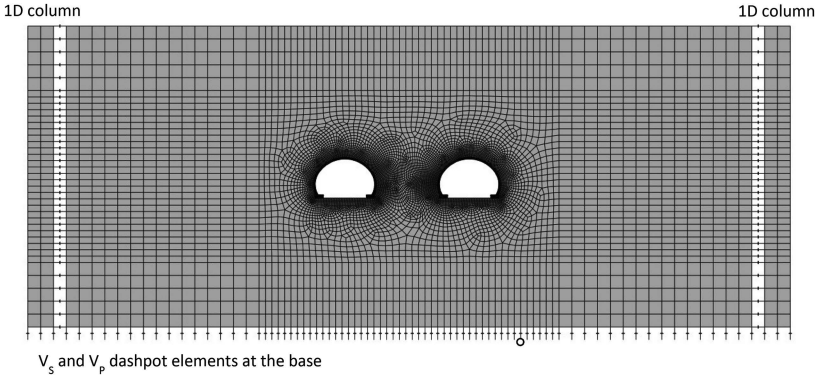

Figure 2. Finite element model: The discretization is finer in the area close to the tunnels.

Absorbing boundaries are installed at the base of the model, consisting of viscous dampers. With respect to the lateral boundaries, the most common approach is the application of kinematic constrains. However, this type of boundary is inherently imperfect: it generates wave reflections which modify the response, while refracted waves propagate within the surrounding medium. To overcome such drawbacks, a free–field boundary is employed herein, consisting of two 1D soil columns connected to the lateral boundaries of the 2D model through dampers.

4.1 *Constitutive models*

The rock formation is not expected to exhibit substantial nonlinearity during seismic shaking. However, to account for possible failure mechanisms, a nonlinear constitutive model with Mohr-Coulomb failure criterion is employed. The latter is certainly not state-of-the-art, but it was deemed reasonable given the absence of detailed data, allowing calibration of more advanced models. Model parameters are assessed conservatively on the basis of the corresponding design parameters of the rock mass: $\varphi = 30^{o}$, $c = 250$ kPa, and $E = 800$ MPa. A 2% Rayleigh damping is introduced, and a coefficient of earth pressures at rest $K_o = 0.5$ is assumed.

The UC lining is modelled employing the *Concrete Damaged Plasticity* (CDP) model. Based on the models proposed by Lubliner et al. (1989) and Lee & Fenves (1998), the model is incorporated in Abaqus. Developed for quasi-brittle materials, the model combines isotropic damaged elasticity with isotropic tensile and compressive plasticity. It considers two failure mechanisms: tensile cracking and compressive crushing. Under uniaxial tension, the stress-strain response is linear until the failure stress f_t, followed by softening due to micro-cracking. Under uniaxial compression, the response is elastic until the yield stress f_{c0}, followed by hardening up to the failure stress f_{cc}, and subsequent strain softening.

The stress-strain relationships are defined in tabular form, adopting widely used relations for confined concrete under compression (Mander et al., 1988) and tension (Chang & Mander, 1994). Under cyclic loading, the model has "memory" of damage sustained in previous cycles, and can capture the effect of opening and closing of cracks. Most importantly, it captures the interaction between axial load and bending moment, and its effect on stiffness and bending moment capacity.

4.2 *Modeling of initial tunnel convergence*

The seismic performance of the tunnel is sensitive to the static rock-mass confining pressures acting on the lining. These are a function of initial convergence, before installation of the UC lining. The initial convergence is a function of the excavation method, the delay in the installation of the temporary support, the stiffness of the temporary support, the properties of the surrounding rock-mass, and several other parameters, the realistic estimation of which would require much more detailed information, that was not available. Therefore, the initial convergence is parametrically investigated considering two scenarios:

- *Lower bound*, assuming initial convergence of 6 mm, leading to relatively large pressures acting on the lining: 350 kPa at the crown, compared to 180 kPa assumed in design (Figure 3a).
- *Upper bound*, assuming initial convergence of 14 mm, resulting in reduced pressures on the lining: 120 kPa at the tunnel crown, compared to 180 kPa assumed in design (Figure 3b).

4.3 *Types of analysis*

The seismic performance of the tunnel is simulated with two different methods:

1) Pseudo-static "*pushover" analysis,* applying the seismic shear strain in terms of angular deformation at the top of the FE model (St. John & Zahrah 1987; Wang, 1993). The angular deformation γ is computed for two acceleration levels: (i) $a_g = 0.24$ g, following the second seismo-tectonic study; and (ii) $a_g = 0.35$ g, based on the more conservative first

study. With respect to particle velocity v_g and v_g/a_g ratio, we conservatively adopt the values proposed by the study of the SEE, which yield: $\gamma \approx 0.0006$ for $a_g = 0.35$ g.

2) Dynamic time-history analysis, using as input ground motions the previously discussed DSHA-computed acceleration time histories.

In both cases, the tunnel is analyzed: (i) assuming elastic lining response, and (ii) accounting for inelastic lining response, employing the CDP model.

5 PSEUDO-STATIC ANALYSIS

5.1 *Pseudo-static analysis assuming elastic UC lining response*

Although not realistic, we start with such analysis to show that our pseudo-static elastic models are compatible with those of the SEE. We assume exactly the same model parameters as the SEE, for the rock mass (E = 800 MPa, ν = 0.2) and for the UC lining (E = 32 GPa, ν = 0.2). We also adopt the angular deformation: $\gamma \approx 0.0006$ for $a_g = 0.35$ g. Both initial convergence scenarios are analyzed, but the results are discussed focusing on the lower-bound scenario.

The contact pressures acting on the lining before application of seismic loading are substantially larger than the ones assumed in design: 350 kPa at the tunnel crown, instead of 180 kPa of the design. As depicted in Figure 3, there exist a few UC lining elements in which the maximum principle stress exceeds the design tensile strength of concrete (f_{ctd} = 1.42 MPa), even before application of pseudo-static pushover loading – totally in accord with the SEE. Based on the results of the analysis, it is concluded that:

(a) At specific UC lining locations, the developing principle tensile stress exceeds the design tensile strength of concrete (f_{ctd} = 1.42 MPa), even before application of seismic loading;
(b) The location of maximum tensile stressing compares well with the study of the SEE;
(c) The maximum tensile stress is substantially larger than the one of the SEE (4.6 MPa instead of 2.2 MPa), revealing the conservatism of our analysis. The difference is attributed to the simulation of nonlinear rock-mass response, which results to soil yielding close to the foundation of the UC lining; and
(d) Seismic loading does not lead to any substantial increase of the tensile stresses, and could be claimed to be almost negligible compared to static loading.

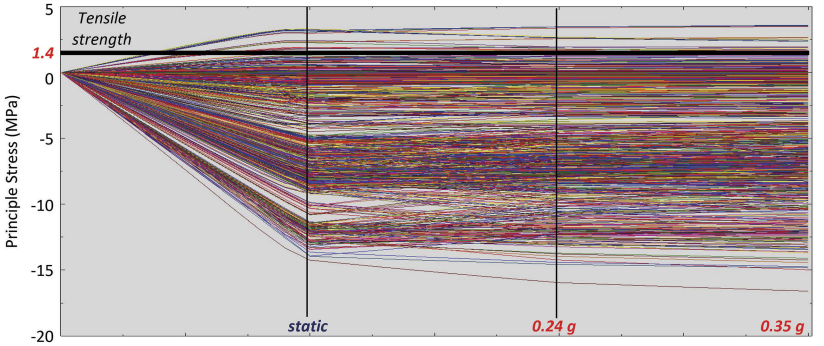

Figure 3. Pushover analysis assuming elastic response of UC lining and lower bound initial convergence: evolution of principle stresses with seismic deformation (expressed in terms of pseudo-static acceleration).

5.2 *Pseudo-static analysis accounting for nonlinear UC lining response*

The purpose of this analysis is to assess the degree of UC lining damage due to seismic shaking. As for the elastic analysis, we adopt the parameters of the SEE:

- Rock mass: E = 800 MPa, ν = 0.2, φ = 30°, and c = 250 kPa;
- UC lining: E = 32 GPa, ν = 0.2, f_{cd} = 21 MPa, and f_{ctd} = 1.42 MPa; and
- Angular deformation (shear strain): $\gamma \approx 0.0006$ for a_g = 0.35 g.

With respect to initial tunnel convergence, the analysis is conducted for both scenarios. The CDP model is calibrated conservatively, assuming the aforementioned design values of f_{cd} and f_{ctd}. The degradation rule is also calibrated conservatively, assuming very rapid degradation of strength and stiffness with accumulated tensile deformation (i.e., with cracking).

To illustrate the degree of conservatism with respect to concrete model calibration, we first conduct a displacement-controlled pseudo-static "pushover" analysis of the lining standing alone, neglecting the confining pressures of the surrounding rock-mass. Since the beneficial initial compression of the lining is completely ignored, the only compression is associated with the dead load of the lining itself. As illustrated in Figure 4, the reaction force P drops sharply at a drift $\delta \approx 1.7$ mm. Observe in Figure 4b that the tensile crack (just on top of the right foundation) penetrates into the lining for about 2/3 of the width, with the concrete tensile damage index reaching almost 1.0, which is indicative of a complete failure of the UC section. In accord with the SEE, without the beneficial confining pressures of the surrounding rock-mass, the UC lining fails at $\delta \approx 1.7$ mm.

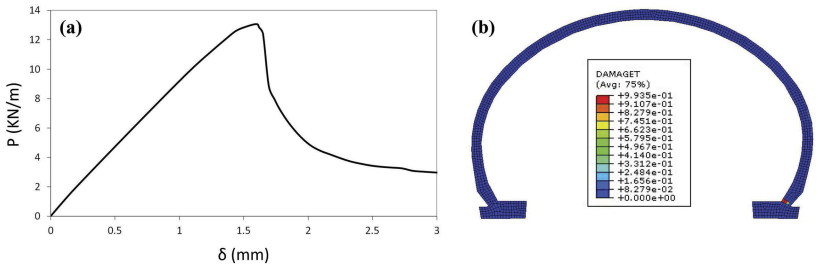

Figure 4. Pushover analysis of the UC lining alone, neglecting the surrounding rock-mass, accounting for nonlinear UC response: (a) evolution of reaction force P with imposed drift δ; and (b) concrete tensile damage index. Without the beneficial confining pressures of the surrounding rock-mass, the UC lining would fail at $\delta \approx 1.7$ mm.

In the presence of the surrounding rock-mass, the performance of the UC lining is completely different. For the more conservative upper-bound initial convergence scenario, the results are summarized in Figure 5. For the most probable assumption of a_g = 0.24g (Figure 5a), the tensile damage index does not exceed a mere 0.06. For the extreme scenario of a_g = 0.35 g (Figure 5b), corresponding to an overly-conservative return period of 4950 years, the tensile damage index reaches 0.99, indicating severe damage. Even in this extreme scenario, tensile damage does not penetrate all the way through the section. Most importantly, tensile failure takes place at the outer periphery of the lining, in contact with the rock-mass: a collapse mechanism cannot easily develop.

To further explore the collapse capacity of the UC lining, an extreme pushover analysis is conducted, gradually increasing the angular deformation to $\gamma \approx 0.006$ – an order of magnitude larger than the one corresponding to the conservative scenario of a_g = 0.35 g. To be on the safe side, the analysis is performed assuming the *upper-bound* initial convergence scenario, which has been shown to lead to increased tensile concrete damage. As illustrated in Figure 6a, the shear deformation is so extreme that the rock-mass fails close to the model boundary. Substantial plastic straining of the rock-mass takes place around the tunnel. As it would be expected, the damage is now extensive, with the tensile concrete damage index reaching 1.0 at various locations (Figure 6b).

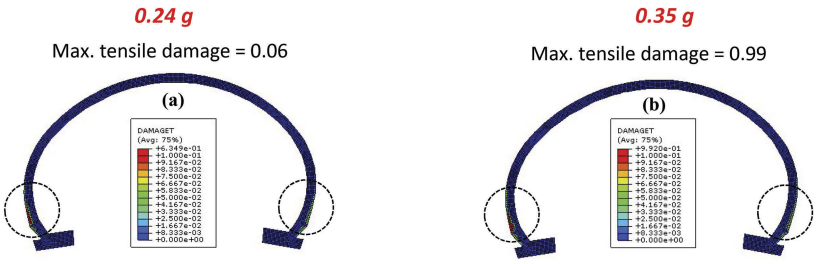

Figure 5. Pushover analysis accounting for *nonlinear* UC lining response, for *upper bound* initial tunnel convergence. Concrete tensile damage index for: (a) a_g = 0.24g; and (b) a_g = 0.35 g.

But even under such extreme deformation, no collapse mechanism develops. This is simply due to the beneficial role of the confining pressures offered by the rock-mass. As the system deforms, the pressures acting on the lining increase dramatically, reaching almost 1300 kPa at the right lateral boundary of the tunnel, and exceeding 3000 kPa at its foundations. Such increase of the confining pressures acts as a significant restraint, providing a beneficial restoring force to the system. In summary, whereas without the beneficial effect of the surrounding rock-mass, the UC lining would fail at drift δ of merely 1.7 mm, when confined by the rock-mass, the UC lining would probably sustain very large angular deformation without collapsing.

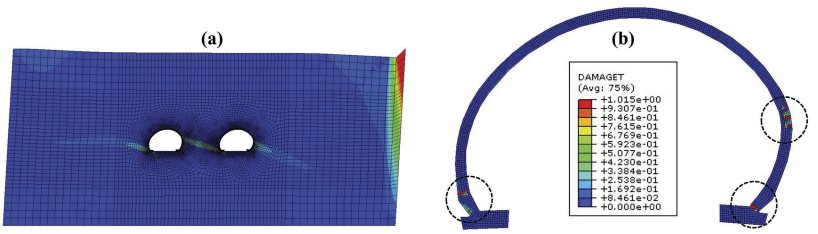

Figure 6. Extreme pushover analysis, imposing angular deformation $\gamma \approx 0.006$ – an order of magnitude larger than the one corresponding to the conservative scenario of a_g = 0.35 g (upper-bound initial convergence): (a) deformed mesh with plastic strain contours; and (b) concrete tensile damage index.

6 DYNAMIC TIME HISTORY ANALYSIS

The dynamic analysis is conducted with both initial convergence scenarios, assuming elastic and nonlinear UC lining response. We focus on nonlinear response, with the more conservative upper bound initial convergence scenario, and the spectrum-compatible Kalamata record (Figure 1b). As depicted in Figure 7a, the maximum drift δ reaches 4.8 mm, while the permanent drift is negligible. As shown in Figure 7b, the tensile damage index does not exceed 0.16. Compared to the pseudo-static analysis (Figure 5a), the degree of damage is substantially larger: 0.16 instead of 0.06. This is attributed to the cyclic nature of seismic loading, which leads to degradation. However, the tensile damage does not penetrate deep in the lining and no collapse mechanism develops.

Based on the results of all the dynamic analyses, it is concluded that: (i) subjected to dynamic loading, the UC lining sustains tensile damage at various locations; (ii) for the lower-bound initial convergence scenario, the concrete tensile damage does not exceed 0.16, for all seismic excitations examined; and (iii) for the upper-bound initial convergence scenario, the concrete tensile damage does not exceed 0.33. The areas of tensile damage do not penetrate all the way through the UC lining, the residual drift is practically zero, and no collapse mechanism develops.

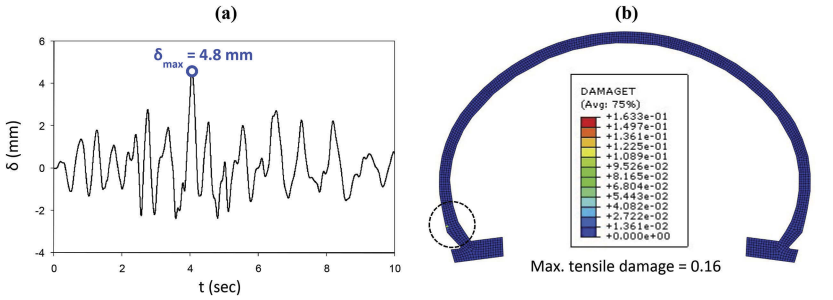

Figure 7. Dynamic time history analysis accounting for *nonlinear* UC lining response, *lower bound* initial tunnel convergence, and seismic excitation with the modified Kalamata record: (a) time history of tunnel drift δ; and (b) concrete tensile damage index.

7 CONCLUSIONS

The key conclusions of our study are summarized as follows:

1) Based on the pseudo-static analysis with elastic UC lining, seismic loading does not lead to an appreciable increase of tensile stresses, being almost negligible compared to static loading.
2) Based on the pseudo-static analysis with nonlinear UC lining, for the upper-bound initial convergence scenario, the concrete tensile damage does not exceed a mere 0.06 for the rational assumption of a_g = 0.24 g. Even under extreme assumptions, no collapse mechanism develops.
3) Without the beneficial confining pressures offered by the surrounding rock-mass, the UC lining would fail at a drift δ = 1.7 mm. However, with the confining pressures offered by the surrounding rock-mass, it can sustain an order of magnitude larger angular deformation.
4) Based on the dynamic time-history analysis, accounting for nonlinear UC lining response, the concrete tensile damage does not exceed 0.33, for all seismic excitations examined. Even in the worst case scenario, the areas of tensile damage do not penetrate all the way through the UC lining, the residual drift is negligible, and no collapse mechanism develops.

Adding reinforcement to an already constructed UC lining would be extremely challenging and costly, if not impossible. Thanks to our analysis, which considered the beneficial effect of the confining pressures offered by the rock-mass, it was possible to prove the adequacy of the UC lining, which was crucial for the rational settlement of the technical dispute.

ACKNOWLEDGEMENT

We would like to acknowledge Dr. R. Kourkoulis and Dr. F. Gelagoti for their contribution in the seismic hazard analysis and the determination of shaking scenarios for the needs of the study.

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