

VULNERABILITY ASSESSMENT OF SHALLOW METRO TUNNELS IN GREECE

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ABSTRACT

Fragility curves constitute an emerging tool for the seismic risk assessment of transportation system elements. They describe the probability of a structure being damaged beyond a specific damage state for various levels of ground shaking, thus they can be used for prioritizing retrofit, pre-earthquake planning and loss estimation studies. So far the vulnerability assessment of tunnels has been mainly based on empirical fragility curves, derived from actual damage from past earthquakes. In this paper, a simple yet comprehensive methodology and a numerical approach is proposed to construct fragility curves for shallow metro tunnel in alluvial deposits, when subjected to transversal seismic loading. The response of the tunnel is calculated under quasi static conditions applying the induced seismic ground deformations which are calculated through a 1D equivalent linear analysis. Defining the damage levels according to the exceedance of strength capacity, the fragility curves could be constructed, as a function of the level and the type of the seismic excitation, considering the related uncertainties. The methodology is applied for the fragility analysis of typical cross sections for Thessaloniki's and Athens' metro. The aforementioned approach allows the evaluation of reliable fragility curves in Greece considering the distinctive features of the tunnel geometries and strength characteristics, the input motion and the soil properties.

Keywords: tunnels, metro, earthquake, damage, fragility curves

INTRODUCTION

Transportation systems play a vital role in modern urban systems, while their importance is considered to be higher due to the lack of redundancy of the individual subcomponents. Although metro tunnels have been proven in most strong earthquakes less vulnerable than above-ground structures, they may still be susceptible to seismic damage, especially the shallow ones (Hashash et al., 2001, Power et al., 1998). The Kobe's metropolitan line failure is a typical example (Shinozuka, 1995). The damage of a metro underground line may produce important indirect effects to human lives and to the operation of the global transportation system of a big city. Hence, it is of high importance for risk managers, city planners and authorities to know the expected degree of damage of each transportation component due to various earthquake scenarios. Fragility curves, which describe the probability of a structure being damaged beyond a specific damage state for various levels of ground shaking, seem to be a valuable tool in order to estimate the damage level of particular element at risk and thus to contribute in the retrofitting decisions, emergency response planning and estimation of direct and indirect losses of lifeline systems.

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So far the vulnerability assessment of tunnels has been mainly based on empirical fragility curves, through the observations of several researchers (Dowding and Rozen, 1978; Owen and Scholl, 1981; Wang, 1985; Sharma and Judd, 1991), derived from actual damage of tunnels in past earthquakes all over the world. ATC13 (1985) developed damage probability matrices for alluvium, cut and cover and rock tunnels using the Modified Mercalli Intensity scale according to an expert opinion approach. HAZUS (NIBS, 2004) developed fragility curves for bored and cut and cover tunnels for ground shaking (i.e. peak ground acceleration) and ground failure (i.e. permanent ground displacement) based on expert judgment. ALA (2002) produced empirical fragility curves for peak ground acceleration for bored and cut and cover tunnels with poor-to-average and good construction, based on regression analysis of a worldwide damage database. The curves are based on PGA values which have been back-calculated at the tunnel location using attenuation models, while the uncertainties in the ground motion and in the tunnel performance are also considered. The definition of the damage states in these approaches is qualitative, mainly based on the extent of cracking of the tunnel liner. More recently, Salmon et al. (2003) presented analytical fragility formulations that were developed for the components of the San Francisco Bay Area Rapid Transit (BART) system based on combination of analytical and experiential data.

The objective of the present paper is twofold: (i) to present a comprehensive approach for the development of analytical fragility curves for shallow metro tunnels based on numerical simulation considering both structural parameters, local soil conditions and variation of input ground motion and (ii) to perform an application in two metro tunnels in Greece. The seismic ground deformations are computed and they are imposed at the boundaries of a plane strain ground model including the tunnel structure in order to calculate the response of the tunnel itself taking into account the soil-structure interaction (Argyroudis et al, 2005). The damage levels are defined according to the exceedance of strength capacity of the tunnel's lining, while the fragility curves could be constructed, as a function of the level of seismic excitation i.e. in terms of peak ground acceleration. Uncertainties related to (a) the input seismic excitation, (b) the lining strength and (c) the definition of the damage levels are also considered in the computation of the fragility curves. The methodology is applied for the fragility analysis of typical cross sections for Thessaloniki's metro due to specific seismic scenarios. In addition, generalized fragility curves that have been developed in recent study following the aforementioned approach (Argyroudis et al, 2006) are used for the vulnerability assessment of a particular line of Athen's metro.

APPLICATION OF THE PROPOSED METHODOLOGY TO THESSALONIKI'S METRO

The main steps of the proposed methodology are outlined hereinafter, through the application in typical cross sections at three locations of Thessaloniki's metro for two particular seismic scenarios; the first with mean return period equal to 475 years and the second with 950 years. The construction of Thessaloniki metro base line is recently started and will be approximately 9.6 km long with 13 stations. The system will be served by two single-track tunnels, each one being approximately 6.4km long, which will be bored using two Tunnel Boring Machines (TBMs), while the two tunnels at the eastern and western ends of the line, 1.45km long in total, will be constructed by means of the Cut and Cover method. The depth of the tunnels (top-of-rail) ranges from -14m measured from the ground surface at the west end (New Railway Station), to -18m for most of the project's length, reaching the maximum depth (approx. -31m) at the area of the University and climbing again to -13m at the east end (Nea Elvetia Station).

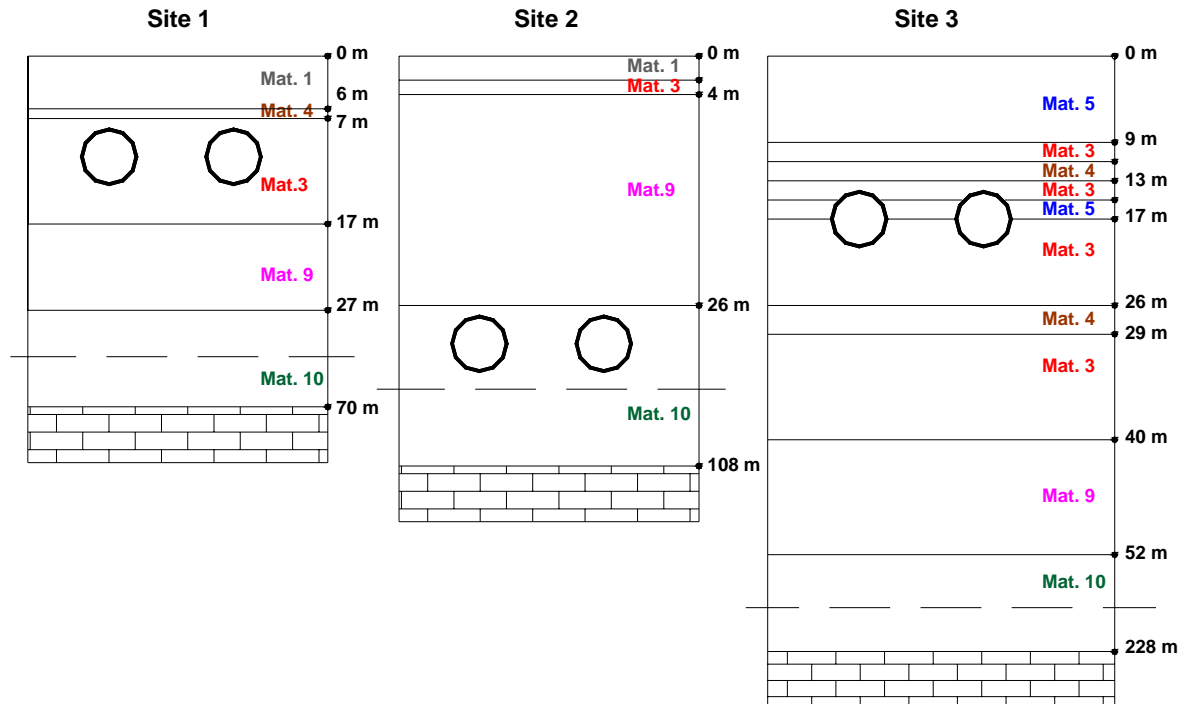
Tunnel and soil model

In the present study a typical cross section of the two bored tunnels is considered. The minimum internal diameter of each tunnel is 5.25m and the minimum thickness of the precast reinforced concrete segment lining is 0.3m. The distance between the two tunnels (wall to wall) is 7m. The characteristics of the lining are shown in table 1.

Table 1. Geometrical and material properties of the tunnel lining.

EA (kN/m)	EI (kNm ² /m)	d (m)	w (kN/m)	v	Reinforced Steel	Reinforcement percentage (%)	Concrete
4.52E+08	3.14E+06	0.30	7.	0.20	S 500	0.6-1.25	C40/50

The analysis is performed at three sites (Figure 1) of the metro line: 1. New Railway Station, 2. Panepistimio (University), 3. Patrikiou, using the soil profiles that derived from the microzonation study of Thessaloniki (Anastasiadis et al. 2002). The upper point of the circular section is in a depth of 8, 27 and 14m, while the total depth from the surface to the bedrock is 70, 108, 228m for each of the three locations respectively. The parameters of the soil materials that are used in the present study are given in Table 2. It is noted that the three soil profiles are characterized from a fundamental period that is equal to 0.5, 0.72 and 0.8 seconds respectively.

**Figure 1. Examined soil models at the three sites of Thessaloniki metro line****Table 2. Soil material characteristics**

Material	E (kPa)	c (kN/m ²)	ϕ (°)	γ_{dry} (kN/m ³)	γ_{wet} (kN/m ³)	v	Vs (m/sec)
1	1.80E+05	0.1	28	15.4	18.5	0.3	220-225
3	2.80E+05	29	8	16.6	19.9	0.3	225-380
4	2.50E+05	5	31	17.2	20.6	0.3	200-300
5	1.40E+05	40	23	16.7	20.0	0.3	200-270
9	6.00E+05	60	25	18.0	21.6	0.3	350-450
10	1.00E+06	110	26	18.3	22.0	0.3	400-750
Bedrock	5.50E+06						1100

Ground motion characteristics

The acceleration time histories that are used as input motion in outcrop conditions are obtained from the Microzonation study of Thessaloniki. Real records from five different earthquakes are used: (a) Kozani, Greece, 13-5-1995 ($M=6.5$, $R=17\text{km}$, $0.142g$), (b) Thessaloniki, Greece, 20-6-1978 (City Hotel, $M_s=6.0$, $R=29\text{ km}$, $0.13g$, deconvolution), (c) Umbria-Marche, Italy, 5-4-1998 (Cubbio-Piene, $M=4.8$, $R=8\text{ km}$, $0.235g$), (d) Montenegro, former Yugoslavia, 15-4-1979 (Hercegnovi Novi, $M=6.9$, $R=65\text{ km}$, $0.256g$), (e) N. Palm Springs, USA, 8-7-1986, (5072 Whitewater Trout Farm, $M=6.2$, $R=7.3\text{km}$).

The main criteria for choosing these time histories are the following:

- (i) The soil conditions of the records are similar to soil class A of Eurocode 8.
- (ii) The PGA values are between 0.20 and $0.30g$.
- (iii) The shape of the acceleration response spectra is close to the one of Eurocode 8 for soil class A. (very stiff soil or seismic bedrock with $V_s > 750\text{m/s}$).

The fundamental periods of the selected input motions ranges from $T_0=0.20\text{sec}$ to 1.0sec .

The recorded input motions were scaled properly according to the seismic hazard study of Thessaloniki for 475 and 950 years return period and applied to the base of the subsoil models. In particular they scaled to values equal to $0.20g$ (Sites 1, 2) and 0.24 (Site 3) for the 475 years scenario and to $0.28g$ (all sites) for the 950 years scenario.

1D linear equivalent soil response analysis

The response of each soil profile is calculated using the code EERA (Bardet et al., 2000) taking into account the inelastic behavior of the soil through appropriate G/G_{\max} - γ and D - γ degradation curves. For each earthquake and each of the aforementioned levels of peak ground acceleration, the maximum shear strain time history that corresponds to tunnel's depth in each site is calculated. The distribution of the displacement amplitudes with depth is obtained at the time instance where the shear strain takes the maximum value in order to be imposed statically at the boundaries of the tunnel plain strain model. In addition the peak ground acceleration on the surface is defined as it constitutes the main index to correlate the tunnel damage and define accordingly the fragility curves.

Tunnel response analysis

The earthquake effects on tunnel structures can be due to ground failure and ground shaking. The majority of damage records has been associated with permanent ground displacements due to ground failure, i.e., fault rupture through a tunnel, landsliding (especially at tunnel portals), and soil liquefaction. While ground shaking has produced a lower degree of damage, especially to tunnels in rock, shallow tunnels in alluvial deposits have been vulnerable to transient seismic lateral ground displacements. The response of tunnels to seismic shaking motions may be demonstrated in terms of two principal types of deformations; the first occurring along the longitudinal axis of the tunnel, which include both axial and curvature deformations; the second perpendicular to the longitudinal axis in the plane of the tunnel cross section, resulting in "ovaling" deformations of a circular tunnel cross section and "racking" deformations of a rectangular cross section (Wang, 1993, Hashash et al., 2001).

The seismic response of the tunnel lining, due to the second type of deformation caused by seismic waves propagating in planes perpendicular to the tunnel axis is examined hereinafter based on a quasi-static analysis. A plane strain ground model and the two tunnels cross sections are simulated using the Plaxis finite element code (Plaxis, 2002) for the three different sites of Thessaloniki's metro. The construction of the tunnel is performed with the TBM method. The analysis starts with the definition of the initial conditions and continues with the staged construction of the tunnel and the calculation of the stresses under static loading. The lateral boundaries of the soil model are located in a distance equal to about three times of the tunnels' diameter from the outer sides of the two tunnels. The shear deformations that were calculated in the previous step, through the one dimensional wave propagation theory for the different levels of peak ground acceleration (corresponding to the maximum shear strain between top and bottom of the tunnel) are imposed on the boundaries of the plain strain model. Therefore, stresses and deformations of tunnel lining can be calculated due to the shear distortion of

the surrounding ground. An example of the output is given in Figure 2. During analysis the behavior of the tunnel lining was set to be linear elastic. Regarding the soil, a Mohr - Coulomb behavior is considered as the inelastic behavior was taken into account through during the procedure of seismic one dimensional analysis.

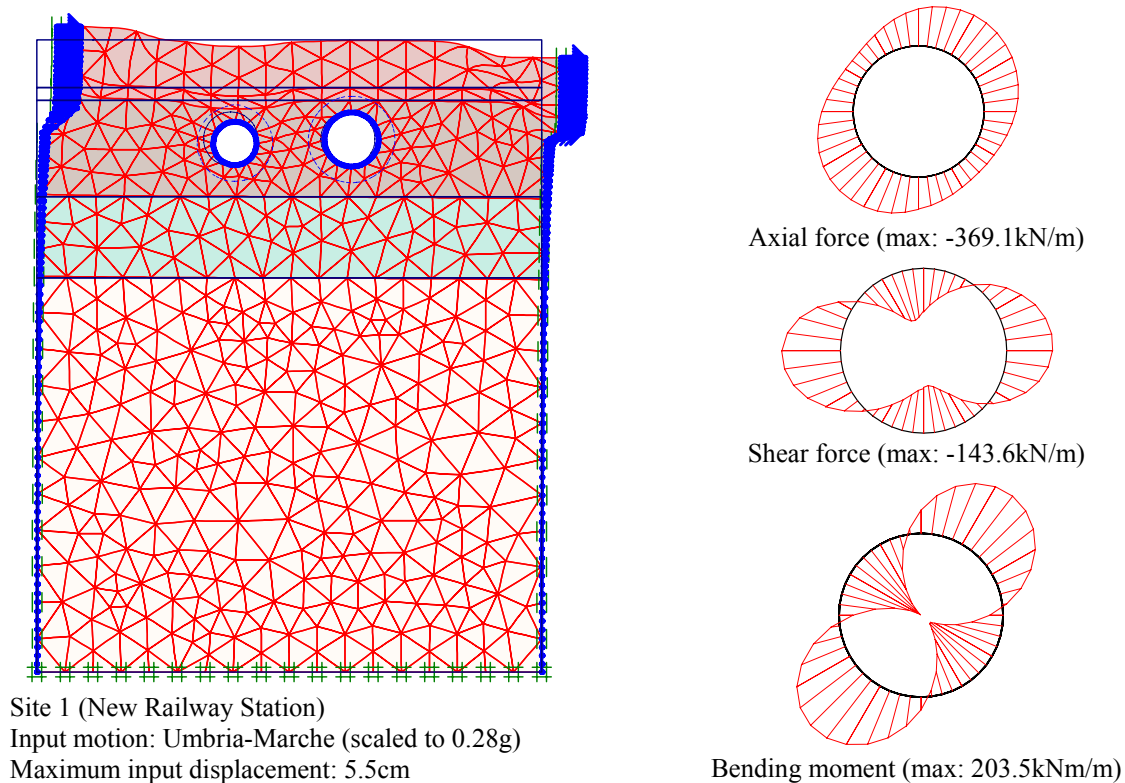


Figure 2. Example of tunnel-soil model and analysis results.

Fragility analysis

The earthquake consequences in lifeline components are mainly described in terms of damage states (i.e. no damage, slight/minor, moderate, extensive, complete). This qualitative approach requires an agreement about the meaning and content of each damage state, but in general is a rather subjective definition. In the present study, the quantification of the damage states is based on a damage index (DI) that is defined as the ratio of the developing moment (M) to the moment resistance (MR_d) of the tunnel lining. In this way it is possible to establish a relationship between the damage index (M/MR_d) and peak ground acceleration (PGA) as it is derived from the analysis results that outlined in the previous section. The definition of damage states is then based on the range of damage index values. According to previous experience of damages in tunnels and engineering judgment, three different damage states are considered due to ground shaking. These concern minor, moderate and extensive damage in tunnel linings and are described in table 3. Although the proposed limits of damage index are not well documented they give a realistic and qualitative view for the expected damage.

Table 3. Relationship between the damage index (DI) and the damage state.

DAMAGE INDEX - DI	DAMAGE STATE
$DI \leq 0.7$	No damage
$0.7 < DI \leq 1.0$	Minor damage
$1.0 < DI \leq 1.3$	Moderate damage
$1.3 < DI \leq 1.8$	Extensive damage

Figure 3 illustrates the results of the vulnerability analyses performed for the three sites of the Thessaloniki metro line in terms of damage index and PGA values. The plotted points are referred to

the response of the two tunnels cross sections, under the five input motions and the two seismic scenarios. The tunnel lining cross section is formed by a set of precast segments that are connected with bolts. In the present application the resistance of the tunnel lining segments is considered to be similar with the one of a girder element; however this is a rather conservative assumption and a more detailed calculation should be made.

It is seen that in some cases the results present considerable dispersion, still a general trend is figured if the extreme values will be removed. Thus, the expected damages in site 1 (New Railway Station) range between none and minor, while in sites 2 (Panepistimio) and 3 (Patrikiou) are mainly between minor and moderate level for both seismic scenarios. No extensive damages are reported even for the extreme case of the 950years return period earthquake. The discrepancy of the results in each site is explained due to the different frequency content of the input motions and the resulted variation in the response of the soil and consequently of the tunnel structure. More precisely, the analysis of the site 2 utilizing the Umbria-Marche input motion resulted in the highest values of damage index. This has to be correlated to the frequency content of the particular seismic excitation that is close to the predominant frequency of the specific soil profile. Furthermore, in site 3, the highest damage index values are derived due to the Umbria-Marche and Palm Springs input motions that are characterized by a frequency content that is near to the predominant frequency of the particular soil profile.

Derivation of fragility curves

Fragility curves describe the damage probability corresponding to a specific damage state, for various levels of ground shaking and they are usually represented as a two-parameters (median and log-standard deviation) lognormal distribution functions. The median values of PGA that correspond to each damage state can be defined as the threshold values of the aforementioned damage indexes, based on the relationship between the damage index (M/MR_d) and PGA. A standard deviation value (β) that describes the total variability associated with each fragility curve has to be estimated. Three primary sources contribute to the total variability for any given damage state (NIBS, 2004), namely the variability associated with the discrete threshold of each damage state, the capacity of each structural type and the earthquake ground motion. As an example, in HAZUS (NIBS, 2004) the uncertainty in the definition of damage states for buildings is assumed to be equal to 0.4, while in the BART system for bored tunnels (Salmon et al., 2003) the variability of the capacity is assumed to be equal to 0.3. The last source of uncertainty associated with seismic demand could be defined by calculating the variability in the results of inelastic dynamic analyses carried out for the different input motions at each level of PGA in bedrock.

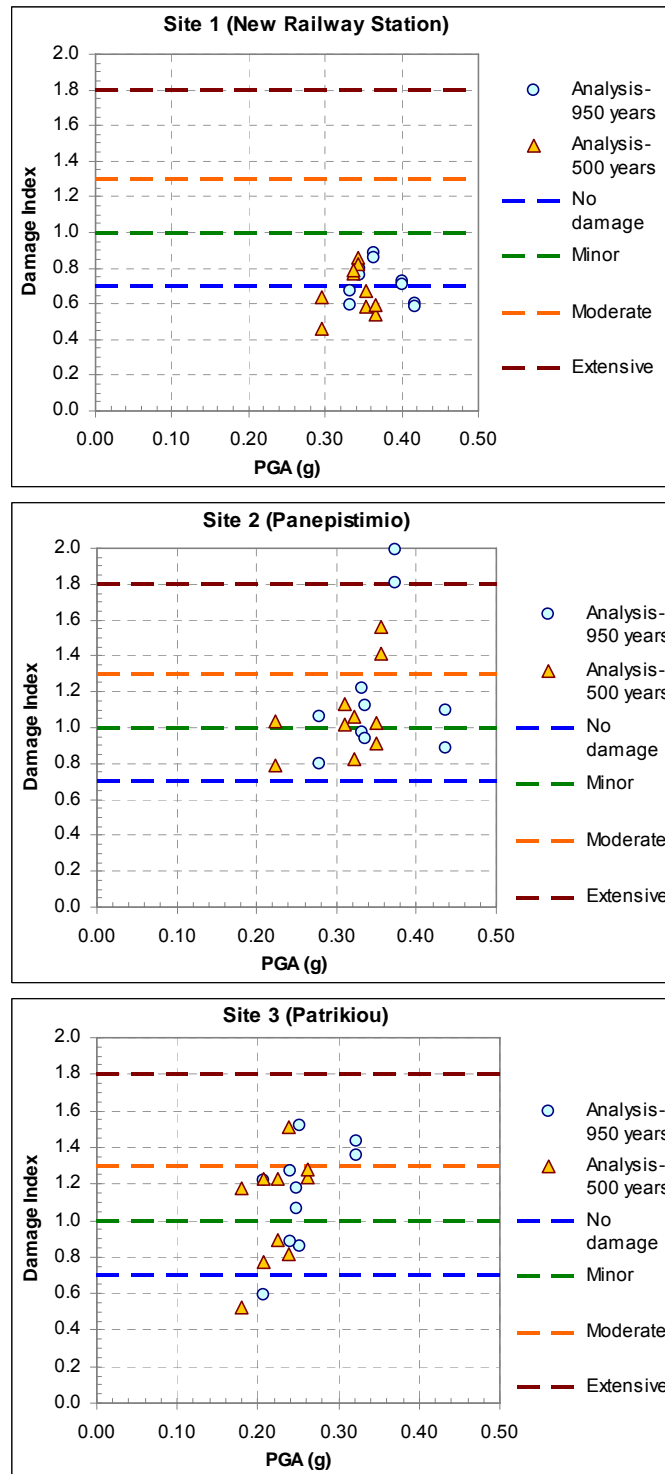


Figure 3. Fragility analysis results for the three sites of Thessaloniki metro line.

APPLICATION OF THE PROPOSED METHODOLOGY TO ATHEN'S METRO LINE

The Athen's metro has been gradually been in operation since 2000, with 24 stations and a total length of 26km; the extension of the lines was recently initiated including 12.7km and 13 stations. In the present paper an application of vulnerability assessment is presented, for the extension of metro line 3 (Ethniki Amina - Stavros) based on fragility curves that have been developed recently (Argyroudis et al., 2006) following the previously outlined methodology. The derived curves refer to three typical soil profiles corresponding to classes B, C and D according to EC8 and two typical cross sections of the Athens' metro, a circular (bored) and a rectangular (cut & cover). In Figure 4 representative curves are illustrated for the two sections and for minor and moderate damage state in comparison with the empirical ones of HAZUS (NIBS, 2004) and ALA (2002). It is seen that the role of the soil is important as the curves are modified when the soil effect is taken into account, while the empirical curves rather describe an average response of the tunnels independently to soil conditions. Based on the obtained analyses, moderate or extensive damages are expected only for the soil class C in case of the circular section, while in case of the rectangular one, moderate damage is possible to occur for all soil classes and extensive only for class C. This can be attributed to the increased damping of the soil class D and the higher stiffness of soil class B.

The metro line under study is passing through four principal groups of geomaterials: a) recent fills at the top, b) Athens Schist (Metasandstone-Metasiltstone, with varying degree of fracturing and weathering), c) conglomerate (with varying degree of cementation and looseness), d) stiff to hard clay-silt with varying percentage of sand and gravel, in places silty/clayey sand (locally cemented). Thus the general geotechnical conditions of the study area are classified to soil category B (according to EC8).

According to the proposed fragility curves, the circular cross section in soil class B is expected to sustain minor damage state, while moderate or extensive damages are not likely to occur. The median value of peak ground acceleration at which the tunnel section reaches the threshold of minor damage (i.e. 50% probability to exceed minor damage state) is 0.96g. During the 1999 Athens earthquake ($M_s=5.9$) the peak ground acceleration that was recorded to Halandri station (Cultural Center) nearby the metro line under study was equal to 0.16g. Hence, the estimated probability to exceed minor damage state is negligible for a seismic scenario similar to the one of the aforementioned seismic event. Table 4 illustrates the calculated probabilities to exceed each damage state for different soil conditions and various levels of seismic excitation. It is seen that even for an extreme seismic scenario (0.8g) in soil class B, the probability to occur minor damage is still low (35%).

Table 4. Exceedance probabilities for different levels of peak ground acceleration in case of circular tunnel cross section.

	Probability to exceed damage state (%)								
Damage state	minor			moderate			extensive		
Soil class	B	C	D	B	C	D	B	C	D
PGA (g)									
0.20	0.1	3.6	8.0	N/A	1.00	N/A	N/A	0.5	N/A
0.40	4.0	32.0	46.3	N/A	16.06	N/A	N/A	10.9	N/A
0.60	17.4	62.3	75.0	N/A	41.60	N/A	N/A	32.5	N/A
0.80	35.8	80.7	88.8	N/A	63.3	N/A	N/A	53.9	N/A

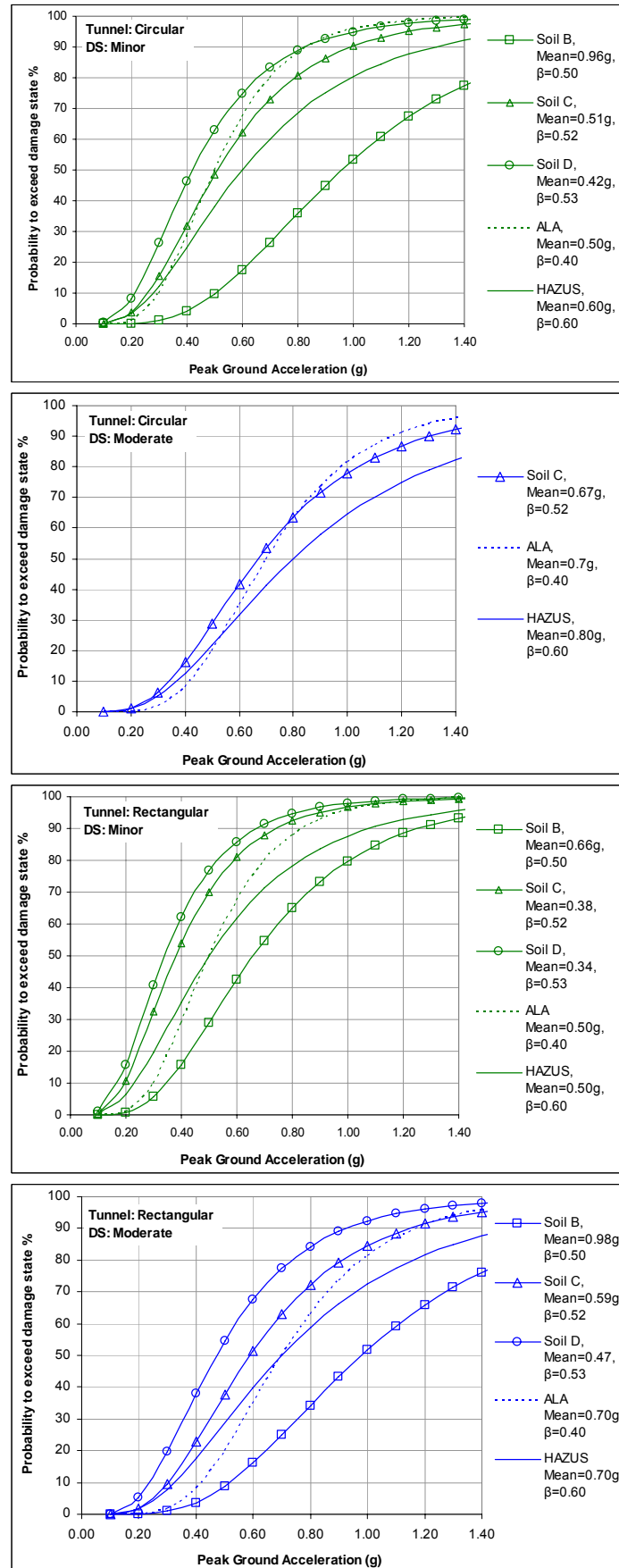


Figure 4. Analytical fragility curves of circular and rectangular tunnel section for minor and moderate damages and for three soil classes.

CONCLUSIONS

The scope of the present paper is to assess the vulnerability of shallow tunnels in alluvial deposits due to the transversal ground shaking, considering the distinctive features of the geometry and strength capacity of the tunnel, the local soil conditions and the input ground motion characteristics. The methodology is based on the combination of one dimensional seismic inelastic analysis in different soil profiles and a quasi-static two-dimensional plane strain analysis that includes both the soil and tunnel. The procedure is applied for the fragility analysis of typical cross sections in 3 sites of Thessaloniki's metro, using 5 different input motions scaled appropriately according to the seismic hazard study of Thessaloniki for 475 and 950 years return period. The results show a variation in the tunnel's response from site to site due to the modified soil conditions and the different depth of tunnels. Moreover, the response of the tunnels and consequently the derived damage indexes, vary in each site under the different input motions. This has to be attributed to the frequency content of the seismic excitations compared to the predominant frequency of each soil profile. However, a general trend is pointed out concerning the expected level of damages in each case.

Based on fragility curves recently developed following the proposed methodology for typical soils and tunnel sections of Athens metro, an application is made in the present paper for a particular metro line in Athens. The obtained fragility curves are modified according to the soil type (B, C, D /EC8) and the tunnel geometry (circular, rectangular), while the comparison with existing empirical ones shows that the latest may overestimate or underestimate the damage probability as they are not considering for the soil conditions.

In the proposed approach reasonable hypotheses were made for the method of analysis, the definition of damage index, damage states and beta values, minor modification of which can alter the results. These assumptions are made due to the complex nature of the seismic fragility problem. Besides, the purpose of the fragility curves is to make a preliminary assessment of the structure response for different seismic scenarios in order to define the seismic risk of the transportation components and the entire network.

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