6th International Conference on Earthquake Geotechnical Engineering
1-4 November 2015
Christchurch. New Zealand

Methodology to Derive Damage State-Dependent Fragility Curves of Underground Tunnels

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ABSTRACT

During past earthquakes, underground structures have sustained significantly less damage than above-ground constructions. Most likely, this may help to explain why, traditionally, tunnel designers, have the tendency to disregard the seismic performance of underground structures. To date, relatively little efforts have been dedicated to the seismic vulnerability of this kind of complex infrastructures. Analytical fragility functions represent a valuable tool to assess the seismic fragility of tunnels. Seismic vulnerability assessment based on analytical fragility functions typically follows a state-independent approach through which is possible to reproduce different levels of seismic damage on structures initially undamaged. The state-dependent approach, by considering the eventuality of a system that is initially damaged, has been developed to assess the variation of the capacity of a system within a seismic sequence. Although aftershocks are generally less severe than the mainshock, they may represent an additional source of hazard that is not accounted for with the traditional approach. This paper describes the methodology developed to derive damage state-dependent analytical fragility curves of underground tunnels by means of fully nonlinear dynamic analysis.

Introduction

The seismic vulnerability assessment based on analytical fragility functions is typically focused on the probabilistic evaluation of the damage occurred to intact structures as consequence of earthquakes. This means that the main objective of this analysis is to evaluate the susceptibility of undamaged structures to be damaged by ground motions. This approach refers as "state-independent" because each damage state achieved by the structure (e.g. slight or moderate) is independent from the previous levels of damage.

Since the return period associated to severe mainshock is usually high, it is assumed that a structure may be repaired between two seismic sequences. However, this seems only a weak justification because the low economic resources might increase the reparation time and the interval between two strong earthquakes could be insufficient to repair the structure. Furthermore, aftershocks represent an additional source of hazard that is not accounted for with the traditional damage state-independent approach. Although aftershocks are generally less severe than the mainshock, their effects on a post-mainshock damaged structure are neglected.

In this sense, the state-dependent (or time-dependent) approach has been developed specifically

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to capture the variation of the capacity of a structure within a seismic sequence (Luco et al., 2004; Bazzurro et al., 2004; Yeo & Cornell, 2005; Luco et al., 2011). Differently from the state-independent formulation, with the state-dependent approach is possible quantify the probability to achieve a worse level of damage in an already damaged structure subjected to a certain intensity of seismic action (i.e. PGA or PGV). This paper describes a numerical procedure to derive damage state-dependent fragility curves for underground deep tunnels subjected to seismic waves that propagate perpendicular to the tunnel axis (i.e. 2D response). For a structure that sustained only slight damage during the mainshock (e.g. from Damage State 0 or DS0 to DS2), with this methodology it is possible to assess the probability to gain a worse level of damage (e.g. from DS2 to DS3) during subsequent seismic events. Finally, the procedure has been validated with the derivation of a set of state-dependent fragility curves for a representative cross section of deep tunnels constructed in weak-rock with reinforced concrete final support. This is a methodological paper, the set of fragility curves presented here have been derived using a small sample of analysis only to setup, test and validate the numerical procedure.

Dynamic Analysis Accounting for Soil and Structure Nonlinearity

Usually, software oriented to geotechnical engineering applications is more suitable to simulate the seismic response of soil and rock with a rigorous formulation of soil-structure interaction. However, the nonlinear behaviour of structural elements is typically overlooked or oversimplified. The seismic vulnerability of tunnels is generally related to the damage of the support rather than to the surrounding ground and this is essentially due to the difficulties of inspection during post-earthquake investigations. In this sense, the first problem tackled in this study was to choose a nonlinear constitutive law not only for the ground but also for the underground structures because the linear-elastic behaviour is not compatible to the aim of this paper.

The model used in this study was discussed more in detail in Andreotti and Lai (2014a; 2014b). It is a 2D plane strain numerical model that, by using nonlinear constitutive models both for the tunnel lining and the surrounding ground, allows to assess the level of seismic damage generated on the tunnel lining. This damage model, implemented in FLAC2D using FISH (ITASCA, 2011), is capable to reproduce the most important failure mechanisms of the tunnel lining. Material nonlinearities of the support have been introduced using the lumped plasticity approach with the insertion of potential plastic hinges based on a bilinear hysteretic model with strength and stiffness reduction (Matsushima, 1969). The axial failure of the lining is governed by interaction diagrams (M-N). Bending failure is based on the flexural capacity θ_u (i.e. ultimate rotation) of the lining section while shear failure is imposed to the model in terms of forces. When, during the analysis, shear forces exceeds the shear strength (V_u) of the cross-section, the tunnel section is tagged as severely damaged. The moment-rotation hysteretic relation, assigned prior to the analysis, governs the behaviour of the regions where most of the permanent rotations occur (i.e. plastic hinges). The dependency from the degree of damage (i.e. strength and stiffness degradation) is introduced by considering the number of hysteretic cycles achieved (Matsushima, 1969) while the variation of M_u with N is governed by the moment-thrust (M-N) interaction diagram.

Definition of Damage States

The definition of damage states for tunnels is usually based on a qualitative damage description from past earthquakes and, although various damage indexes and related parameters have been proposed for the vulnerability of buildings and bridges, no such information is available for tunnels (Argyroudis and Pitilakis, 2012). Regarding flexural mechanisms, the deformations induced by the ground surrounding the tunnel support can be accommodated only until certain thresholds beyond which the triggering of a kinematic failure mechanism is simulated by the activation of at least four plastic hinges. At this stage, the lining is no longer able to contrast the deformation of the ground which may potentially induce uplift of the floor and falling of materials. Brittle failures in the model occur when, in at least one control node of the lining, the value of axial or shear forces achieve its limit value. The ultimate value for the axial force is related to the flexural mechanisms by the moment-thrust (M-N) interaction diagram. This means that failures in compression or tension ($N \geq N_u$) could happen only with the activation of at least one plastic hinge. On the other hand, shear failure is imposed to the model when the shear force in the lining exceeds its limit value ($V \geq V_u$) and can occur even without the activation of any flexural plastic hinge. Control nodes (see Figure 2) are the nodes of the lining where the potential activation of plastic hinge during the analysis is possible and where the axial and shear verification is carried out at each step of the analysis. Currently the model implements 10 control nodes along the tunnel lining. The position of these nodes is fixed and it has been defined in locations where, after a careful literature and experimental review, the damage is most likely to occur. Finally, M-N interaction diagrams and the ultimate value for the shear force (V_u) of the lining section have been defined according to the specifications of Eurocode 2 and Eurocode 8 for buildings. According to these considerations, all damage states can be defined numerically as follow:

1. DS0 (no damage): no plastic hinge activated and $V<V_u$ in all control nodes of the lining.
2. DS1 (slight damage): at the activation of the 1st plastic hinge and $V<V_u$ in all control nodes of the lining.
3. DS2 (moderate damage): at the activation of the 2nd plastic hinge and $V<V_u$ in all control nodes of the lining.
4. DS3 (extensive damage): when the number of active plastic hinges is $i \geq 3$ (flexural mechanism) or when, in at least one control node, the axial force (N) or shear force (V) exceeds their limiting value, respectively N_u and V_u.

Definition of State-Independent and State-Dependent Fragility Curves

Seismic fragility curves relate the probability of reaching or exceeding a particular damage state (DS_i) given a particular level of ground motion intensity measure (i.e. $IM=PGV$). The use of the lognormal distribution enables easy development and expression of these curves and their uncertainty (Kennedy et al., 1980). With this formulation, it is assumed that all uncertainty in the fragility curves can be expressed through uncertainty in its median alone. Hence, only two parameters are needed to plot the curves. The analytical relation of the standard Gaussian cumulative function used to draw the curves is represented in Eq. (1) and (2) respectively for the state-independent and for the state-dependent formulation:

$$P\left[IM \geq \overline{IM}_{c,DSi}\right] = \phi\left(\frac{\ln IM - \ln \overline{IM}_{c,DSi}}{\beta_{tot,DSi}}\right) \tag{1}$$

$$P\left[IM \geq \overline{IM}_{c,DSj} \mid DS = DS_i\right] = \phi\left(\frac{\ln IM - \ln \overline{IM}_{c,DSj}}{\beta_{tot,DSj}}\right) \tag{2}$$

where:

$\overline{IM}_{c,DS}$: is the median value of the intensity measure (*IM*) which generates the i^{th} or the j^{th} damage state (i.e. DS_i or DS_j) on the system, according respectively to the state-independent or the state-dependent formulation. The intensity measure used in this study is the Peak Ground Velocity (*IM=PGV*).

$\beta_{tot,DS}$: is the lognormal standard deviation parameter that describes the total uncertainty associated to a certain damage state. Following the HAZUS methodology (NIBS, 2004) $\beta_{tot,DS}$:

$$\beta_{tot,DS} = \sqrt{\left(\mathrm{CONV}\left[\beta_C, \beta_D, \overline{IM}_{c,DS}\right]\right)^2 + \left(\beta_{IM,DS}\right)^2} \tag{3}$$

where:

β_C: is the lognormal standard deviation parameter that describes the variability of the capacity curve.

β_D: is the lognormal standard deviation parameter that describes the variability of the demand or hazard.

$\beta_{IM,DS}$: is the lognormal standard deviation parameter that describes the uncertainty in the estimate of the median value of the thresholds of damage states.

The two terms in Eq. (3) are assumed to be independent and the total uncertainty is calculated by the SRSS (Square Root of the Sum of the Squares). The function "CONV" in Equation (3) implies the process of convolving probability distributions of the demand and the capacity curve (NIBS, 2004). In this paper, "CONV" has been used with its general meaning of concomitance and mutual conditioning. The best estimate of the median value of the threshold capacity of each damage state in terms of ground motion intensity measure ($\overline{IM}_{c,DS}$) can be obtained following different methods. One is based on linear regression (e.g. Shome and Cornell, 1999; Gehl et al., 2013; Argyroudis and Pitilakis, 2012) or quadratic regression (e.g. Pan at al., 2007) between damage index (*DI*) and intensity measure (*IM*). The other method is based on the nonlinear least square optimization performed on the observational frequencies of exceedance ($N_{f,DS}/N$) of the predefined thresholds of damage states (e.g. Rota et al., 2008). Theoretically both approaches require a certain number of simulations (*N*) to be reliable but in the observational frequencies approach, *N* has to be fairly large because it is assumed that the effective probability can be approximated by the numerical frequency (Gehl et al., 2013).

When there is lack of information about seismic damage, as in the case of tunnels, the variability of the capacity (β_C) and $\beta_{IM,DS}$ (median value of the thresholds of damage states in terms of intensity measure) are judgmentally assigned using published data, such as in HAZUS (NIBS, 2004), as reference (e.g. Salmon et al., 2003). However, the use of the judgmental approach into the analytical formulation introduces subjectivity into the analysis, makes less clear the role of uncertainties on the results and finally leads to the definition of hybrid rather than analytical fragility curves. The numerical procedure proposed in this study tries to overcome this problem. The estimates of β_D and β_C at different limit states are simultaneously obtained according to the results of a regression fit of the data obtained from the mechanical analysis (i.e. nonlinear dynamic analysis).

The thresholds of the damage states in terms of damage index ($\overline{DI_{c,DS}}$) and the associated variability ($\beta_{DI,DS}$) are defined through numerical approach using the data on damage states from the whole sample of analyses. As described in Figure 1, from the linear empirical relation it is possible to define the best estimate of the threshold of each damage state in terms of intensity measure ($\overline{IM_{c,DS}}$). By using different input ground motions and systems with different mechanical parameters, the linear fit gives an approximation of the process of convolution between β_D, β_D and $\overline{IM_{c,DS}}$. As showed in Figure 1, the lognormal standard deviation parameter ($\beta_{IM,DS}$) that describes the uncertainty in the estimate of the median value of the thresholds of damage states in terms of intensity measure has been derived from a relation between $\beta_{DI,DS}$ and the slope of the linear regression. Finally the total lognormal standard deviation ($\beta_{tot,DS}$) has been computed by the SRSS since $CONV[\beta_D, \beta_C, \overline{IM_{c,DS}}]$ and $\beta_{IM,DS}$ can be assumed to be independent (NIBS, 2004).

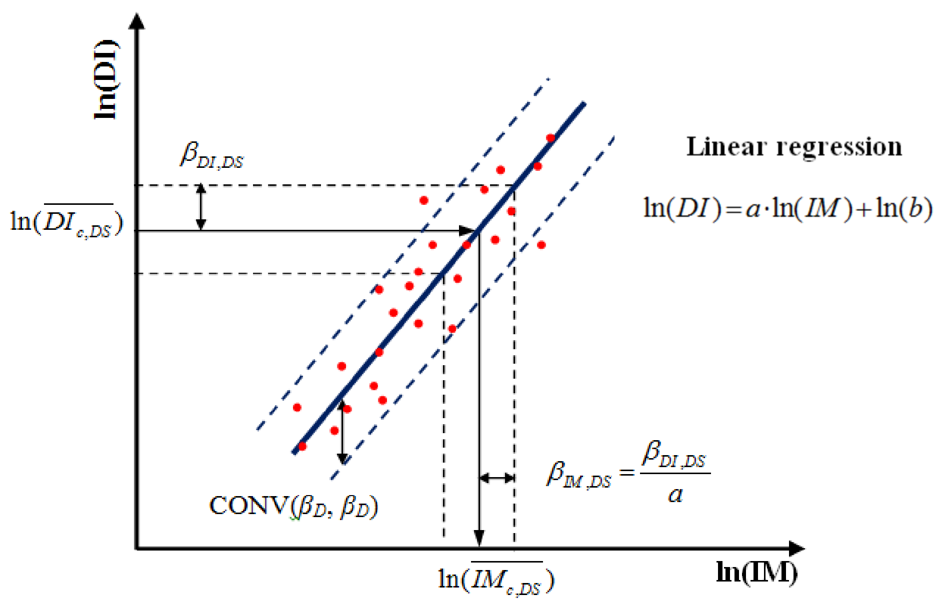


Figure 1: Approach used in this work to define the parameters required to draw the fragility curves.

Each damage states (DS_i) can be defined on the bases of the global normalized cumulative ratio ($GNCR$) damage index (DI) using Eq. (4) as defined in Andreotti and Lai (2014a; 2014b):

$$DS_i = GNCR_i = \sum_{i=0}^{4} \sum_{n}^{j} \frac{\left|\theta_{m,in} - \theta_{y,in}\right|}{\theta_{pl,u}} \tag{4}$$

where $\theta_{m,in}$ and $\theta_{y,in}$ are the maximum rotation and the yield rotation at the plastic hinge i in the cycle n while $\theta_{pl,u}$ represents the ultimate plastic rotation which can be computed with the formulation proposed by the Eurocode 8. *GNCR* returns the summation of the total plastic rotation accumulated at each cycle and in all active plastic hinges.

Methodology for the Derivation of State-Dependent Fragility Curves for Tunnels

The methodology developed to derive state-dependent fragility curves for underground tunnels follows the framework proposed by Luco et al. (2004), Bazzurro et al. (2004), Yeo & Cornell (2005) and Luco et al. (2011) which was specifically proposed for buildings. It has been adapted in this study to underground tunnels. The procedure can be subdivided in the following steps:

1. <u>Selection of time histories</u>. Selection of a suite of earthquake records with suitable levels of scaling in amplitude. This suite of records should contain both mainshocks and aftershocks. From the review of seismic damage occurred to underground structures it was found that the near-fault is the most critical condition for tunnels. The final set of accelerograms used in this study to derive fragility functions is composed by 11 signals recorded on outcropping rock, with a range of PGV that goes from 0.38 m/s to 1.26 m/s with epicentral distance lower than 40 km. The velocity records have been applied at the base of the model with half intensity to remove the free surface effect. All records have been used here indifferently as mainshocks and aftershocks.

2. <u>Cloud analysis</u> (i.e. Shome and Cornell, 1999; Gehl et al, 2013) <u>on the intact system</u> (from DS0 to DS_i). Using the selected set of unscaled mainshock, the undamaged model is subjected to several nonlinear time-history analyses in order to obtain a suite of damaged models of tunnel. The aim of this step is to generate different levels of damage on the intact models and bring the system from DS0 to all other damage states DS_i (i.e. DS1, DS2 and DS3). The output consists of multiple realizations of post-mainshock damaged systems according to the damage state (DS) and the damage index previously defined (*GNCR*). The data on damage are given in terms of intensity measure of ground motion (i.e. *IM=PGV*) which brings the system at a certain damage states (i.e. DS1, DS2 or DS3). In order to quantify the damage, at each damaged structure is associated the intensity measure of ground motion (IM), the value of damage index and the damage state achieved (DS_i). The output data of this step are sufficient to derive only state-independent fragility curves.

3. <u>Cloud analysis on the post-mainshock damaged systems</u> (from DS_i to DS_j). Using as input the set of post-mainshock damaged systems obtained in the previous step, the nonlinear time-history analyses are performed again using the unscaled aftershocks. The aim of this step is to bring a system from a damage state DS_i (e.g. DS2) into a worse damage state DS_j, with $j > i$ (e.g. DS3). This step is essential to derive state-dependent fragility curves. Formally, this step is identical to the previous one, what it changes is the type of numerical

model used as input in the dynamic analysis and theoretically also the input motion that, in this case, should be an aftershock. Now, the vulnerability analysis is performed with the damaged system (DS_i). The aim of this phase is to bring the systems from the damage state DS_i (i.e. DS2) to a worse damage state DS_j (i.e. DS3).

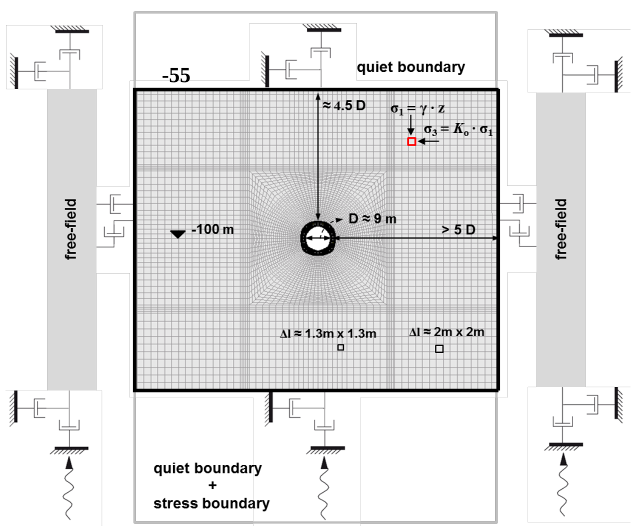
4. Derivation of fragility curves. Processing the data obtained in step 2 and 3, it is possible to define all the parameters required to plot the state-independent and the state-dependent fragility curves. The parameters are the median threshold values in terms of intensity measure ($\overline{IM_{c,DSj}}$), which generates the i^{th} or the j^{th} damage state (DS_i or DS_j), and the lognormal standard deviation $\beta_{tot,DS}$ that quantify the total uncertainty associated to the expected value.

Results

To validate the proposed procedure we have chosen the project of an important European base tunnel of high speed railway line (LTF, 2013) as a case study.

Rock parameters								
GSI	E	ν	K	G	ϕ	c	γ	K_o
	[MPa]		[MPa]	[MPa]	[°]	[MPa]	[kN/m^3]	
15	400	0.35	444.44	148.15	20	0.3	22	0.6
25	1100	0.30	916.67	423.08	22	0.5	24	0.6
35	2600	0.30	2166.67	1000	24	0.6	25	0.6

Final lining of reinforced concrete					
Section	Thickness	Width	Height	Steel rebars	Stirrups
	[m]	[m]	[m]	[kg/m^3]	[cm^2]
S1	0.7	1.0	1.0	50	14.13
S2	1.0	1.0	1.0	80	14.13



Materials			
Concrete		Steel	
C30/37			B450C
Ec	f_c	Ec	f_s
[MPa]	[MPa]	[MPa]	[MPa]
33000	25.5	210000	450

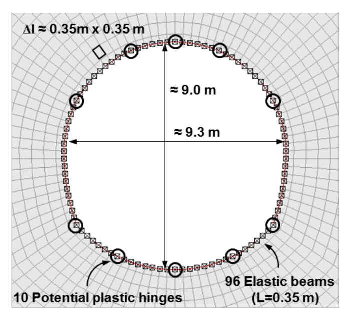


Figure 2: Numerical model and parameters used to define fragility curves.

About 90 fully nonlinear dynamic analyses are carried out to define the median value of the thresholds of damage index (DI_i) and the associated variability (lognormal standard deviation $\beta_{DI,DS}$) for each damage states (DS_i). The analysis are performed using 11 different time histories, 2 different sections of tunnel support and 3 different types of surrounding ground (see Figure 2).
In order to understand the choice of the mechanical parameters reported in Figure 2 it is important to underline that even if underground tunnels are less vulnerable than above-ground

constructions, the data collected during the post-event surveys show that there are critical conditions in which the tunnel systems can undergo severe damage. Focusing on deep tunnels, the main critical condition is: proximity to the active fault and weak or high fractured rock (e.g. Corigliano, 2007; Miyabayashi et al., 2008; Corigliano et al., 2011). High contrast between stiffness of ground and tunnel support and low ductility of tunnel supports are other important aspects that may be underestimated in the design phase of tunnels because the seismic verification is not a common practice, at least in Europe. The methodology presented in this paper is tested in relation to this main critical condition that it is assumed to exist at least in one cross-section of the tunnel. The other situations are not considered here because, generally, they are not cause of concern for deep tunnels bored in rock. For this reason, the mechanical parameters of the ground reported in Figure 2 refers to weak or highly fractured rock as for example flyshoid or varicoloured clay-shales formations. Even if possible, because of the complexity of the problem (i.e. effective stress approach), the influence of water has not been considered in this analysis.

Some of the results from the vulnerability analysis are reported in Figure 3. In order to assess the uncertainties of the capacity, demand and damage states, in the proposed approach the regression analysis was performed using all the data available without distinguish between type of structural support or ground type (i.e. rock class in terms of GSI). For comparison, the regression analysis on the damage data coming from the numerical models with different linings (i.e. section S1 and S2) but only with one type of surrounding rock (i.e. GSI 25) shows little variability and higher coefficients of determination (Figure 3a). To achieve a practical end use of the fragility functions (e.g. typological fragility curves) it is better to consider all the data available from the mechanical analysis. However this obviously translates into the introduction of variability in the model, as showed in Figure 3b. Finally, from the regression analysis on systems with different level of initial damage (i.e. DS0, DS1 and DS2), for each type of system the two parameters required to plot the fragility curves have been defined (Figure 4 and 5).

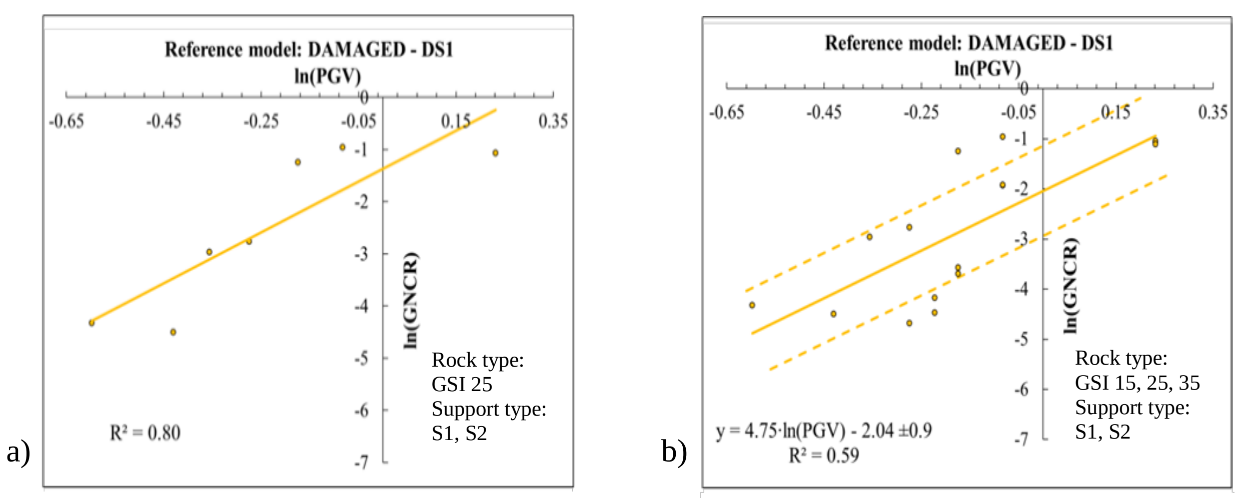
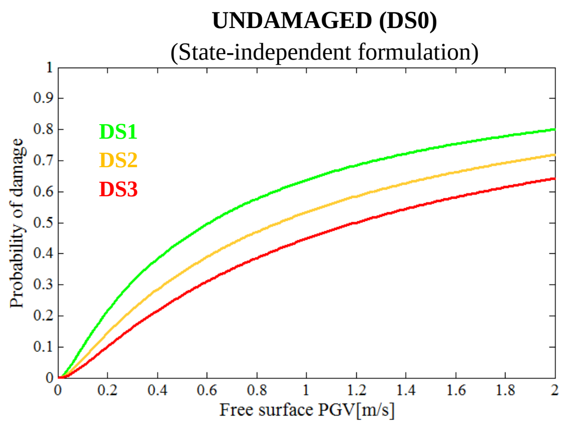
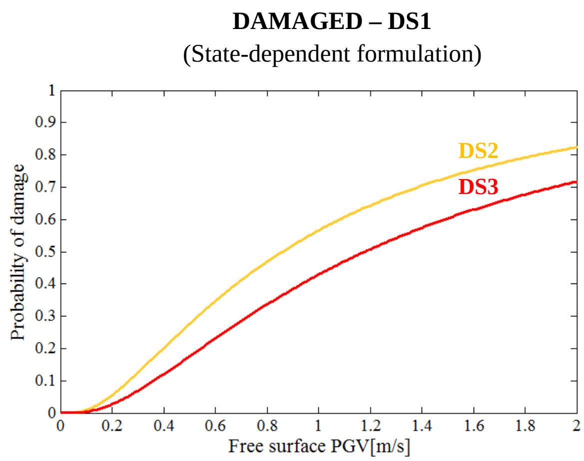


Figure 3: Example of linear fit to define the parameters required to draw the fragility curves (here are reported only data related to a system with an initial level of damage equal to DS1).

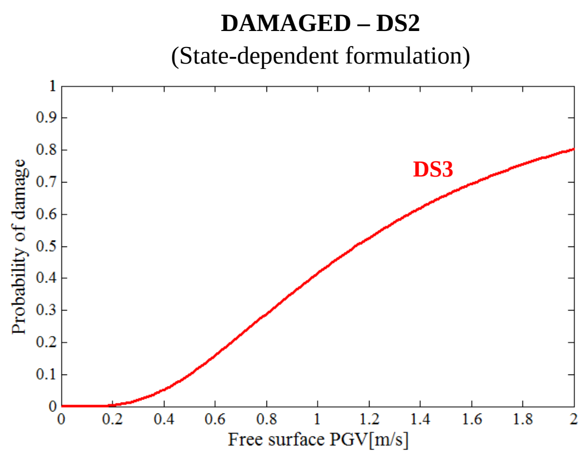


Reference Structure: UNDAMAGED (DS0)			
DS_i	level of damage	$\overline{IM}_{c,DS}$ [m/s]	$\beta_{tot,DS}$
DS0	None	-	-
DS1	Minor/slight	0.61	1.41
DS2	Moderate	0.89	1.40
DS3	Extensive/Severe	1.20	1.40

Figure 4: State-independent fragility curves obtained to validate the proposed procedure.



Reference Structure: DAMAGED - DS1			
DS_i	level of damage	$\overline{IM}_{c,DS}$ [m/s]	$\beta_{tot,DS}$
DS0	None	-	-
DS1	Minor/slight	-	-
DS2	Moderate	0.86	0.91
DS3	Extensive/Severe	1.18	0.92



Reference Structure: DAMAGED - DS2			
DS_i	level of damage	$\overline{IM}_{c,DS}$ [m/s]	$\beta_{tot,DS}$
DS0	None	-	-
DS1	Minor/slight	-	-
DS2	Moderate	-	-
DS3	Extensive/Severe	1.15	0.65

Figure 5: State-dependent fragility curves obtained to validate the proposed procedure.

Concluding Remarks

Analytical fragility functions represent a valuable tool to assess the seismic vulnerability of tunnels. The vulnerability assessment of tunnels is generally based on simple empirical fragility curves, without properly considering depth, soil type and lining details. Seismic vulnerability assessment based on analytical fragility functions typically follows a damage state-independent approach through which it is possible to reproduce damage states caused by specific ground motion on undamaged structures. Therefore this approach is not conservative because does not consider the influence of repeated seismic events (i.e. aftershocks), typical of real seismic sequences, or the eventuality of system already damaged by previous ground motions. Although aftershocks are generally less severe than the mainshock, they may represent an additional source of hazard that is not accounted for with the traditional approach. This paper describes a procedure to derive state-dependent analytical fragility curves by means of fully nonlinear dynamic analysis. The main outcome of this study is the setting up of a methodology to carry out seismic vulnerability analysis of tunnels. This method allows to assess the probability to achieve or exceed a certain level of damage on the support by considering different initial levels of damage in the system. The procedure has been tested using, as a case study, the project of an important European base tunnel of high speed railway line (LTF, 2013). A set of seismic fragility curves has been obtained but they should be considered only as a preliminary result that should be substantiated by a larger number of numerical simulations, as in this study they were somehow limited (on the order of 90). Furthermore, the work done so far for the validation of the methodology allowed to highlight some potential improvements to the constitutive law of plastic hinges. The activities already carried out for the enhancement of the cyclic behaviour of the structural components lead to the reduction of the epistemic uncertainty associated to the damage index (GNCR) which, by influencing the lognormal variability, it has been recognized to affect the reliability of tails of the fragility curves. Effectively, if we compare the results of state-independent approach in Figure 4, we can recognize that the mean values of threshold of damage state ($\overline{IM_{c,DSj}}$) are in line with the values of the traditional fragility curves present in literature and derived with the empirical approach (0.53 m/s for DS1 and 0.86 m/s for DS2 in Corigliano et al. 2007; ≤0.8 m/s for DS1 and >0.8 for DS2 in Dowding and Rozen, 1978). The most important differences concern the lognormal variability. In part because of the use of a different approach but, more heavily, for the small sample of analysis and because the updated version of constitutive law for plastic hinge has been developed at the end of this study.

Acknowledgments

This study has been carried out with the financial contribution of the European Commission within the framework of the FP7 REAKT ("Strategies and tools for Real Time Earthquake Risk Reduction") Project, Working Package 5, Task 5.2.4 (“Time dependent fragility curves for critical transportation system components”) and the Department of Civil Protection of Italian Government entitled “Seismic vulnerability of shallow and deep tunnels” (Project DPC-EUCENTRE S.3.9 – 2014). Both these supports are gratefully acknowledged by the authors.

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