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Seismic fragility and risk assessment of an unsupported tunnel using incremental dynamic analysis (IDA)

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Abstract. Seismic assessment of underground structures is one of the challenging problems in engineering design. This is because there are usually many sources of uncertainties in rocks and probable earthquake characteristics. Therefore, for decreasing of the uncertainties, seismic response of underground structures should be evaluated by sufficient number of earthquake records which is scarcely possible in common seismic assessment of underground structures. In the present study, a practical risk-based approach was performed for seismic risk assessment of an unsupported tunnel. For this purpose, Incremental Dynamic Analysis (IDA) was used to evaluate the seismic response of a tunnel in south-west railway of Iran and different analyses were conducted using 15 real records of earthquakes which were chosen from the PEER ground motion database. All of the selected records were scaled to different intensity levels (PGA=0.1-1.7 g) and applied to the numerical models. Based on the numerical modeling results, seismic fragility curves of the tunnel under study were derived from the IDA curves. In the next, seismic risk curve of the tunnel were determined by convolving the hazard and fragility curves. On the basis of the tunnel fragility curves, an earthquake with PGA equal to 0.35 g may lead to severe damage or collapse of the tunnel with only 3% probability and the probability of moderate damage to the tunnel is 12%.

Keywords: seismic response; fragility curve; incremental dynamic analysis; underground structures

1. Introduction

Estimation of dynamic response of tunnels and underground structures is an important issue in design and analysis of such facilities. Historical reports indicate that underground structures have experienced a lower rate of damage than the surface ones (Hashash *et al*. 2001), but some previous earthquakes have induced severe damages to underground openings, including the 1995 Kobe, Japan earthquake (Tohda 1996), the 1999 Chi-Chi, Taiwan earthquake (Wang *et al*. 2001), the 1999 Kocaeli, Turkey earthquake (Hashash *et al*. 2001), the 2004 Mid Niigata, Japan, prefecture earthquake (Jiang *et al*. 2010) and the 2008 Great Wenchuan earthquake, China (Wang *et al*. 2009). Underground structures exposed to seismic loading is now more of a concern for the designer than it used to be in the past (Kappos 2002). This is because the application range of these structures has been more developed.

Prediction of ground motions resulting from earthquake is very difficult and it is almost impossible to determine the characteristics of ground motion until earthquake actually occurs (Nejati *et al*. 2012a). Every earthquake causes some unique motions which depend on several factors, including disruption mechanism of fault at earthquake source, the wave's propagation media and geological features of earthquake site (Kramer 1996).

In geotechnical earthquake engineering, uncertainty and variability are inevitable while dealing with natural materials. This is because geotechnical structures are inherently heterogeneous and there is an insufficient amount of information available for site conditions. On the other hand, any seismic analysis was subjected to many sources of uncertainty in case of earthquake magnitude, frequency content, time duration, epicenter distance and etc. Therefore, there will always be some random variation in the various properties of soil and earthquake. (Hamidpour *et al*. 2017, Guellil *et al*. 2017) In these cases, uncertainty quantification is an appropriate tool for reduction of the ambiguity. In other words, the seismic response of structures should be determined by different earthquake records using different dynamic analyses (Vamvatsikos and Cornell 2002).

One of the appropriate methods for a parametric analysis of seismic behavior of structures is Incremental Dynamic Analysis (IDA) which was widely used for the seismic assessment of buildings and geotechnical structures like dams, embankments and bridges (Mander *et al*. 2007, Christovasilis *et al*. 2009, Zarfam and Mofid 2011, Alembagheri and Ghaemian 2013, Brunesi *et al*. 2015, Khorami *et al*. 2017). However, this method for seismic analysis of tunnels and underground structures has not yet been considered, while the uncertainty and variability in underground structures are more than the surface ones. Moreover, one of the most important outcomes of the IDA is fragility curve, which represents the probability of a structure reaching a certain damage state for an earthquake event. In the present study, IDA was used for seismic

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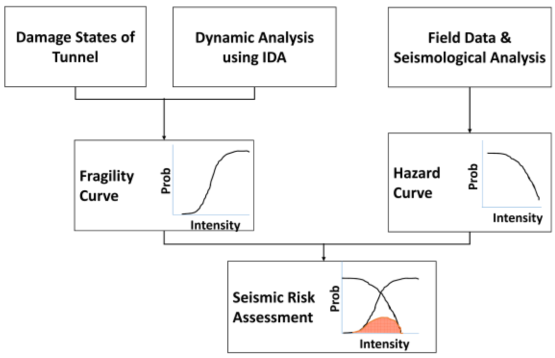

Fig. 1 schematic approach outline for seismic risk assessment of underground structures

assessment of an unsupported tunnel and the levels of induced damage due to different earthquakes were evaluated. In the next, seismic risk assessment of the tunnel under study was evaluated based on the IDA results and seismic hazard curve of the tunnel site.

Seismic hazard and risk are two important concepts in engineering design. Seismic hazard describes the probability of occurrence of a disaster event generated by an earthquake, whereas seismic risk quantifies the probability of occurrence of a specific level of damage over a certain period of seismic hazard (Wang 2011, Erdik 2017). Fig. 1 shows schematic approach outline for seismic risk assessment of underground structures.

Risk is a forward looking concept and involves two parts of (1) the occurrence probability of a disaster and (2) the extent of induced damage due to the disaster. Hence, seismic risk assessment is quantifying how likely a specific damage due to an earthquake event could be happening.

Table 1 Influencing parameters on seismic response of underground structures

Parameters	Condition	Damage description	Reference
Earthquake Magnitude	>6 (Richter scale)	Severe damage	Sharma and Judd (1991)
Distance to epicenter	<25 Km	Severe damage	Sharma and Judd (1991)
Peak Ground acceleration (PGA)	>0.55 g	Severe damage	Sharma and Judd (1991)
	<0.19 g	No damage	Dowding and Rozen (1978)
	0.25-0.4 g	Minor damage	Dowding and Rozen (1978)
	<0.5 g	No collapse	Dowding and Rozen (1978)
	>0.5 g	Severe damage	Dowding and Rozen (1978)
Geological condition	Near fault	Severe damage	Li (2012); Yu *et al*. 2016
Rock mass quality	Poor rock	Severe damage	Li (2012)
	High quality	slightly to moderately damage	Li (2012)
Tunnel Depth	<50 m	Severe damage	Sharma and Judd (1991)
	>300 m	No serious damage	Sharma and Judd (1991)
Type of support	Supported tunnel	Safer than unsupported tunnel	Hashash *et al*. (2001), Sharma and Judd (1991)
Frequency content	High frequency	Local spalling	Hashash *et al*. (2001)
Construction method	Cut & cover	Rectangular structures more vulnerable than circular bored	Hashash *et al*. (2001)

2. Seismic response of underground structures

The seismic response of underground structures, e.g., caverns, tunnels and underground material storage, has been an important topic due to the damages of such structures in recent strong earthquakes. The earliest studies on the seismic response of underground structures were some empirical researches which presented summaries of case histories of damage to underground structures (Dowding and Rozen 1978, Owen and Scholl 1981, Sharma and Judd 1991, Power *et al*. 1998, Kaneshiro *et al*. 2000)

Wang *et al*. (2001) reported the damage in mountain tunnels after the 1999 Chi-Chi, Taiwan earthquake. They found that among the 57 tunnels investigated 49 of them were damaged and the degree of damage is associated with the geological condition and structural arrangement of the tunnel.

Jiang *et al*. (2010) assessed damaged tunnels resulted of the 2004 Mid Niigata, Japan, Prefecture Earthquake using a Geographic Information System (GIS) and summarized several influencing factors, such as the distance to epicenter, the geological conditions, the completion time, the overburden and the angle between the presumed fault and the tunnel axes which caused the tunnel damage.

Li (2012) analyzed the 2008 Wenchuan, China earthquake and illustrated the basic features of the various types of seismic damage and their relationship with distance from the epicenter, seismic wave propagation direction, presence and orientation of fault zones, bedrock-overburden interface, geo-stress, rock mass quality at the portal slope and the surrounding tunnel, and the depth and shape of the tunnel.

Almost all of the previous studies in the field of seismic response of underground structures limited to observational studies of the damaged tunnels resulted by the occurred earthquakes. Therefore, most of them are qualitative and only describe the induced damage due to the earthquakes. Some of the most important notions regarding the seismic response of underground structures summarized in Table 1.

Information on the seismic response of underground structures, compared to the surface structures, is relatively scarce (Wang 1993). Therefore, there is a significant lack of information regarding the seismic response of underground structures and a reliable seismic assessment of underground structures cannot be achieved by previous observations.

However, the effect of earthquake on underground structures can be categorized into two types of (1) ground shaking and (2) ground failure, such as fault displacement, liquefaction (Hashash *et al*. 2001). Also the damage induced by ground shaking are in three principle types of axial, curvature and hoop deformations (Owen and scholl 1981).

Hashash *et al*. (2001) presented a systematic approach for evaluating the seismic response of underground structures on the basis of used methods in earthquake engineering. This approach is including three major steps: (1) Definition of the seismic environment, (2) Evaluation of ground response to shaking and (3) Assessment of structure behavior due to seismic shaking.

Variability of rock mass geo-mechanical properties and unpredictable characteristics of a probable earthquake produce an enormous uncertainty in the seismic response of the underground structure. Deterministic dynamic analyses of underground structures are not capable to handle these kinds of uncertainties. Probabilistic analysis is an appropriate tool for quantification of the uncertainties in the seismic response of underground structures (Aslani and Miranda 2005).

However, for seismic assessment, although the common probabilistic approach is more applicable than the deterministic one, there are still remain many sources of uncertainty in the three mentioned steps of seismic analysis. For example, the design of earthquake criteria limited to two earthquake levels: maximum design earthquake (MDE) and operation design earthquake (ODE), while a reliable seismic assessment of an underground structure needs some more dynamic analyses.

3. Incremental Dynamic Analysis (IDA)

As mentioned before, quantification of the uncertainties in the seismic response of underground structures needs different dynamic analyses using different earthquake records and incremental dynamic analysis is a reasonable approach for this purpose.

The concept of IDA was developed by Bertero (1977) to evaluate the strength and deformation capacities of buildings and Vamvatsikos and Cornell (2002) established a common frame of reference for IDA which can be used for the assessment of demand and capacity of structures.

Table 2 Limit states of tunnel at different value of convergences

Limit state	Tunnel convergence (%)
No Damage to Minor Damage	1
Moderate Damage	2.5
Severe Damage to Collapse	5

IDA is a parametric analysis method that involves subjecting a structural model to a sufficient number of ground motion records, which scaled to multiple levels of intensity, thus producing different curves of response parameterized versus intensity levels (Vamvatsikos and Cornell 2002). The mentioned levels are appropriately selected to force the structure through the entire range of behavior, from elastic to inelastic and finally to collapse. The result of IDA was reported in several IDA curves which are presented by a scalar Intensity Measure (IM) versus Damage Measure (DM).

In the present study, Peak Ground Acceleration (PGA) was chosen as seismic intensity measure and maximum displacement at tunnel crown was chosen as damage measure. PGA is a simple and easy-to-use index which is suitable for short-period structures such as tunnels (Ye *et al*. 2013).

3.1 Definition of damage state

Damage state of structure is an important ingredient of IDA and instability risk assessment. Different criteria were proposed for determination of damage state of tunnels. The critical deformation of tunnel, which is always lower than the failure strain, can be used as damage state of tunnel. For example, Sakurai (1981) suggested a criterion for determination of critical deformation on the basis of elastic modulus of the tunnel surrounding rock mass

$$\log \varepsilon_c = -0.25 \log E - 0.85 \tag{1}$$

Table 3 properties of selected records for IDA

No.	Event	Magnitude (R)	Distance (km)	PGA (g)	PGV (cm/s)	PGV/PGA (m/s/g)	Time Duration (Sec)
1	San Fernando, 1971	6.61	0.00	1.23	93.23	0.76	7.00
2	Coyote Lake, 1979	5.74	10.21	0.12	9.21	1.28	5.75
3	Whittier Narrows, 1987	5.99	6.78	0.11	10.80	1.00	3.50
4	Loma Prieta, 1989	6.93	8.84	0.52	41.85	0.81	3.60
5	Landers, 1992	7.28	2.19	0.77	31.49	0.41	13.75
6	Northridge, 1994	6.69	4.92	0.43	42.10	0.99	4.40
7	Northridge, 1994	6.69	15.11	0.18	11.91	1.50	6.75
8	Kobe, Japan, 1995	6.90	0.90	0.41	22.22	0.54	4.75
9	Kocaeli,Turkey, 1999	7.51	3.62	0.22	35.07	1.56	13.5
10	Northridge, 1994	5.28	18.53	0.05	2.50	2.12	8.75
11	Loma Prieta, 1989	6.93	3.22	0.44	86.28	0.50	4.00
12	Tottori, japan, 2000	6.61	15.23	0.20	9.15	2.16	17.00
13	Tottori, japan, 2000	6.61	15.58	0.25	25.53	0.96	9.25
14	Parkfield, CA, 2004	6.00	4.66	0.25	13.19	1.88	8.75
15	Iwate, japan, 2008	6.90	16.26	0.28	23.59	1.20	24.00

$$\log \varepsilon_c = -0.25 \log E - 1.22 \quad (2)$$

$$\log \varepsilon_c = -0.25 \log E - 1.59 \quad (3)$$

where ε_c is critical strain and E is elastic modulus of tunnel surrounding rock mass. Eqs. (1), (2) and (3) represent the top, middle and bottom limits of the critical deformation, respectively. However, the Sakurai criterion is restricted to the elastic range of deformation and then is not applicable for nonlinear seismic assessments. Further, the Sakurai criterion does not present a damage level of tunnel over the induced deformations.

The damage or limit state of tunnel has been precisely considered for squeezing rocks in several previous research works (Jethwa *et al*. 1984, Singh *et al*. 1992, Aydan *et al*. 1993, Goel *et al*. 1995, Singh and Goel, 1999, Hoek and Marinos 2000). In the present study, based on the proposed classification for squeezing potential in tunnels, a reliable limit state was suggested which can be used for evaluation of the seismic response of tunnels (Table 2).

3.2 Record selection

Record selection is one of the most important issues of IDA and significantly influences on the result of analyses. Several methods were proposed for record selection, and there are three fundamental effective factors on seismic time histories selection as follow: (Kramer 1996)

1. Earthquake magnitude
2. Distance from the site
3. Site condition characteristics

On the basis of the mentioned effective factors, 15 real records of earthquakes were selected from the PEER ground motion database (PEER Database 2014). Magnitude of the selected earthquakes varies in the range of 4-9 in the Richter scale. Distance of the selected earthquake from the site are smaller than 20 Km and the shear wave velocities in the selected sites are in the range of 800-4000 m/Sec. Table 3 shows the characteristics of selected records for IDA. Time duration of the chosen records as seismic ground strong motion was determined over a range of 5-95% of the total Arias Intensity (AI) (Arias 1970).

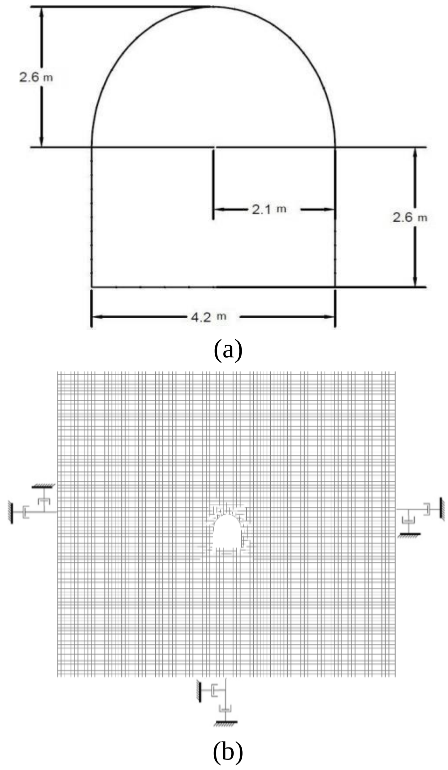

Fig. 2 (a) Dimensions and (b) numerical modeling of the studied tunnel section

Table 4 Geo-mechanical properties of rock mass in the under study tunnel (Fahimifar 2015)

Properties (unit)	Value
Q	11.1
GSI	60
RMR	55
RQD (%)	75
Uniaxial compressive strength (MPa)	88
Elastic modulus (GPa)	10
Tensile strength (MPa)	2
Cohesion (MPa)	4
Friction angle (Degree)	33
Density (Kg/m^3)	2600

4. Case study

The case study of this research is an unsupported railway tunnel in south-west of Iran. The south-west railway route consists of 131 tunnels, approximately 60 Km, which located in Zagros Mountains. Some of the excavated tunnels were unsupported and the tunnel excavated in weak rock was supported with stone masonry lining. The construction of these tunnels started in 1925 and then several instability problems occurred during the operation period of the tunnels. Rock mass weathering and erosion of the unsupported sections, cracking of the stone masonry linings and water seepage into the tunnels are some of the instability problems which reported in Zagros-region railway tunnels.

Whereas Zagros fold-and-thrust belt is amongst the world's most seismically active mountain ranges, seismic response of the excavated tunnel, especially unsupported tunnels, is necessary. However, in the present study, an unsupported section of the Zagros-region railway tunnel No. 68 in south-west of Iran was considered. The geo-mechanical properties of the studied tunnel were summarized in Table 4.

5. Numerical modeling

Numerical dynamic analysis of the tunnel under study was conducted using two-dimensional finite difference code (Nejati *et al*. 2012b). The mechanical behavior of the tunnel surrounding rock was described by an elastic-perfectly plastic material with a Mohr-Coulomb constitutive model. After simulation of static model, some modifications should be done in order to prepare necessary conditions for dynamic analysis. These changes include conversion of fixed boundary condition to viscose, defining the dynamic damping for the complete system and applying earthquake load as a time history at the base of the model. Dimensions and numerical modeling of the studied section of the tunnel is illustrated in Fig. 2. The viscous boundary was applied to the numerical model in normal and shear directions to prevent the reflection of outward propagating waves back into the model (Lysmer and Kuhlemeyer 1969). The dynamic damping in the geotechnical material is a parameter that is difficult to determine. Rayleigh damping is usually used as a damping of systems in dynamic analyses and often 2-5% of the critical damping is chosen as the material damping value (Gardien and Stuit 2003). In analyses that use one of the plasticity constitutive models (e.g., Mohr-Coulomb) a significant amount of energy can dissipate during plastic flow. Hence, for the seismic analyses that involve large-strain, only a minimal percentage of damping (e.g., 0.5%) may be required (Itasca 2000). So, in this study the value of dynamic damping is considered as 0.5% of the critical damping.

Rayleigh damping is frequency-dependent but it will be independent in the range of predominant frequency (Itasca 2000) and usually the natural frequency of the system could be used as predominant frequency. There are several methods to determine natural frequency of model. One of them is applying the gravity force, to the numerical model with zero damping. Upon applying the gravity force, the model begins to oscillate and the natural frequency of oscillation with recording history of vertical displacement in the special time interval is determinable. The frequency of oscillation can be used as a natural frequency of model. Fig. 3 depicts the time history of vertical displacement for the model under study in no damping condition. It can be observed, in this figure, that the frequency of oscillation is approximately 9 Hz. This can be adopted as natural frequency of the model.

5.1 IDA curves

Dynamic loads are usually applied to the model boundaries as a record of acceleration, velocity, stress and force. One restriction when applying velocity or acceleration input to model boundaries is that these boundary conditions cannot be applied along the same boundary as a viscous boundary condition because the effect of the boundary would be nullified (Itasca 2000). For removing this problem, velocity record is transformed into a stress record using the following formula (Itasca 2000)

$$\sigma_s = 2(\rho \cdot C_s) \cdot v_s \tag{4}$$

where σ_s is applied shear stress, ρ is mass density, C_s is speed of *s*-wave propagation through medium and v_s is input shear particle velocity and obtained by integration of the acceleration time history.

Whereas the IDA curves were produced on the basis of PGA, all of the acceleration time histories were scaled to different levels of intensity (0.1-1.7 g) and transformed into the shear time histories according to Eq. (4). In the next step, shear stresses of the earthquake records were applied to the base of the model and the corresponding

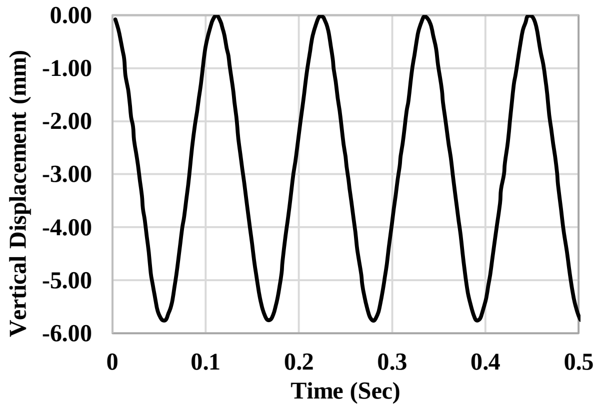

Fig. 3 vertical displacement versus time, for sudden gravity loading of no damping model

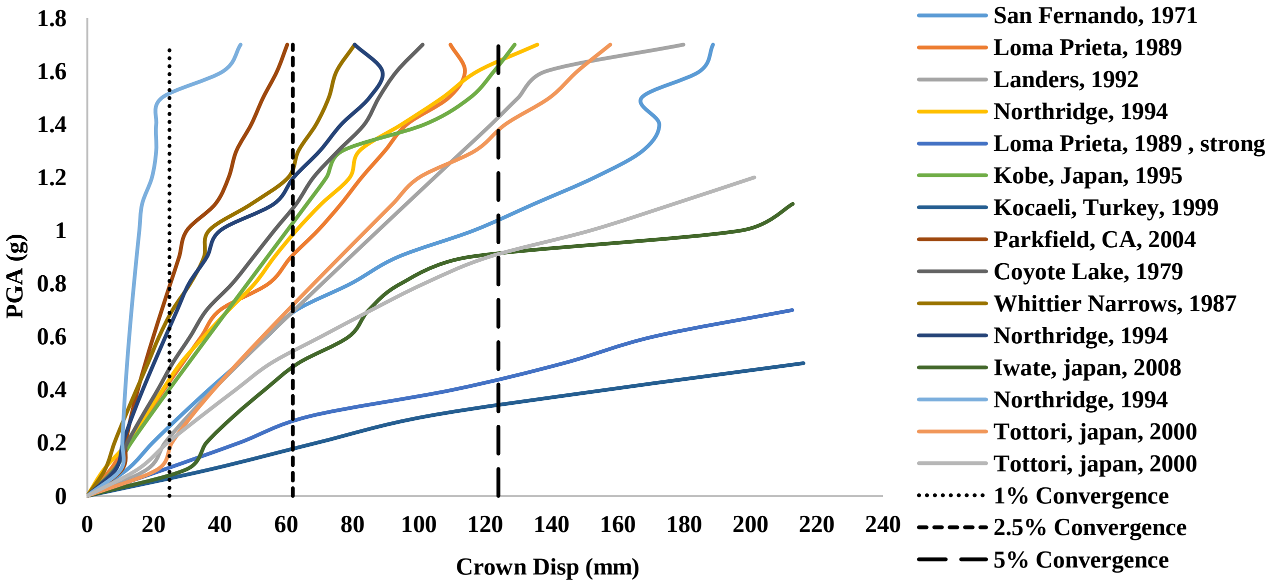

Fig. 4 IDA curves with three defined limit states

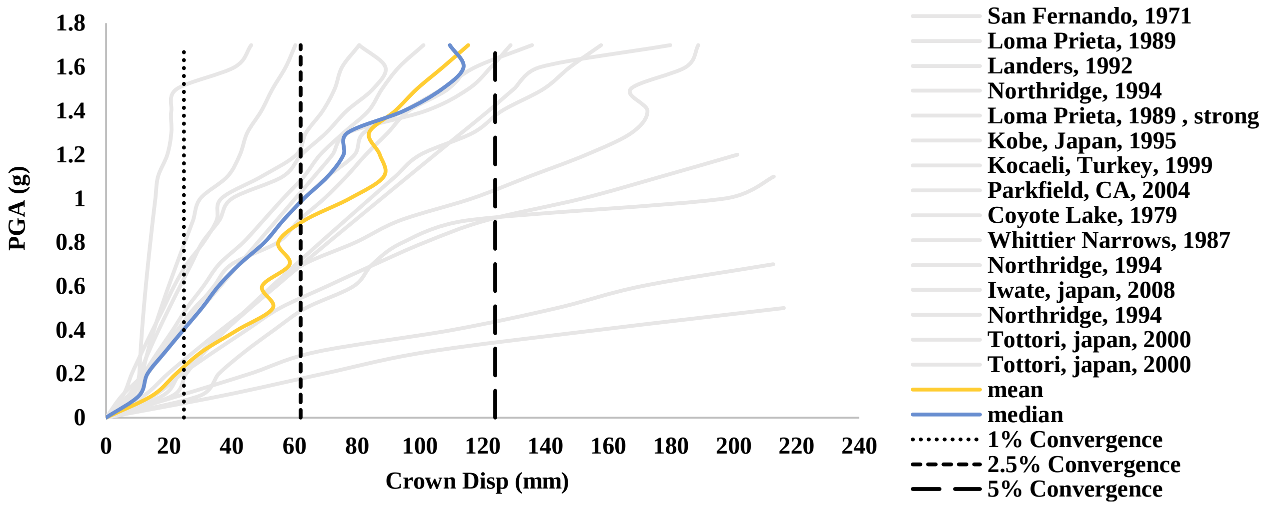

Fig. 5 Summarizing IDA curves for the tunnel under study

displacement at the tunnel crown was recorded.

Intensity of the applied load to the model should be increased to a level which can identify the seismic performance and structural capacities (Vamvatsikos and Cornell 2002). This level in surface structures is usually in range of 0.7-1 g, but in underground structures more intensity is required to achieve the ultimate structural capacity. However, in the present study, for saving the timeand reaching a reasonable output, intensities of the chosen earthquakes were increased to 1.7 g.

Fig. 4 illustrates IDA curves extracted for each of the 15 chosen earthquake records. Also the damage states of the tunnel are shown in Fig. 4 as 1%, 2.5% and 5% of convergences (See Table 2). The first limit state as one percent convergence of tunnel is corresponding to the damage initiation. The second one attribute to 2.5% convergence known as moderate damage and finally the third damage state according to 5% convergence is related to severe damage or collapse initiation.

As shown in Fig. 4, among the 15 IDA curves, only two earthquake records cannot force the tunnel to converge more than 2.5%. In other words, two earthquake records cannot produce moderate damage in the tunnel. In the same way, nine earthquake records, were extremely excited the tunnel and lead to severe damage or collapse.

As shown in Fig. 4, the number of IDA curves are high and seismic assessment of structures with all of the IDA curves is complicated. Then, mean or median of the IDA curves, as a representative curve, can be used for summarizing the IDA data.

Fig. 5 shows mean and median of the 15 IDA curves and the three damage states. It is inferred form Fig. 5, the records possessing PGA less than 0.5 g could not exceed limit state 2 (moderate damage) and also records with PGA less than 0.19 g will produce no damage in the tunnel, because they could not exceed first damage state. This finding has a good agreement with empirical classification suggested by Dowden and Rozen 1978 (Table 1). However, on the basis of suggested empirical classifications, an earthquake with PGA more than 0.5 g will excite a severe damage to tunnel (Dowding and Rozen 1978, Sharma and Judd 1991), while the results of IDA curves showed that only 13% (2:15) of the earthquakes with PGA equal to 0.5 g induce a moderate damage to the tunnel. This is because, in addition to PGA, there are some more influencing parameters on the seismic response of the underground structures and have to be considered in seismic assessments.

Although nine earthquake records excited the tunnel to severe damage or collapse, the mean and median of the 15 records, over the PGA which varies in the range of 0.1-1.7 g, cannot force the tunnel to collapse. This is because the IDA curves display a wide range of behavior which are completely dependent on the selected records, and the outputs are different from one record to the other. Many parameters such as earthquake duration, energy and frequency content of the record influence on the analysis results. Hence, it is important to use a powerful probabilistic method to achieve a distribution of DM given IM.

6. Seismic fragility modeling

As mentioned before, IDA only determines the earthquake intensity which excites the structure to a specific state of damage. However, it will be more applicable if we can determine an undesirable outcome as a function of excitation. This concept is the basis of fragility model (Porter 2014). In other words, a fragility model can be defined as a probability of exceeding a specific state of damage as a function of an intensity measure (Ellingwood *et al*. 2004).

$$Fr(x) = P[D(x) \geq R \mid IM = x] \quad (5)$$

where P represents a conditional probability, IM is random variable intensity measure describing the intensity of the demand on the system, $D(x)$ is demand on the system at intensity measure of x (e.g., PGA), and R indicates the resistance of component or system. Indeed, the seismic fragility model is a cumulative distribution function (CDF) of the capacity of a structure to resist an undesirable limit of instability.

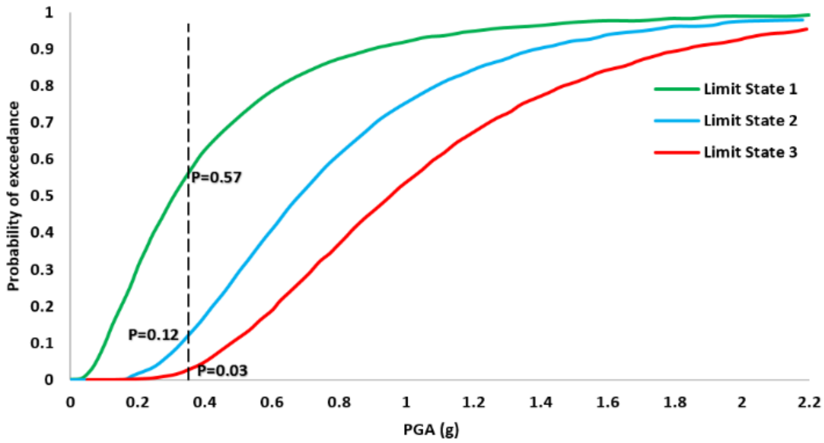

Fig. 6 fragility curves of the tunnel under study at different limit states

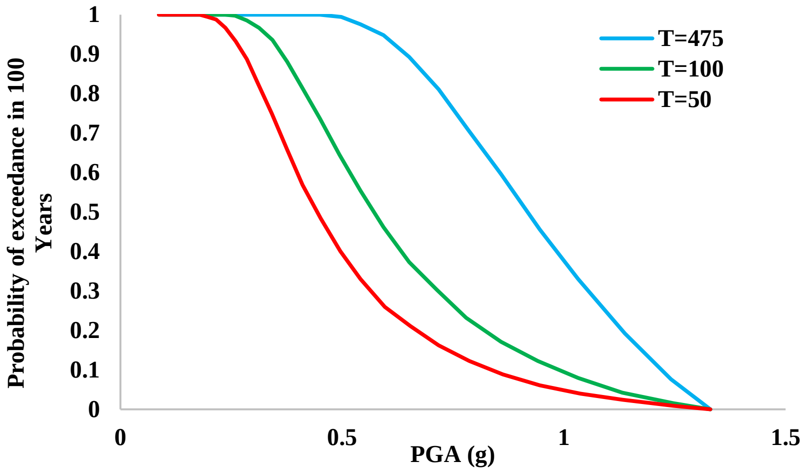

Fig. 7 Hazard curves for site of the tunnel at 50, 100 and 475 years of return periods in 100 years of the tunnel life time

The concept of fragility function in earthquake engineering has been firstly used by Kennedy *et al*. (1980), who define a fragility function as a probabilistic relationship between frequency of failure at a component of a nuclear power plant and peak ground acceleration in an earthquake.

The required data for determination of fragility curve can be collected by expert opinions, empirical or analytical equations and numerical or experimental modeling.

Argyroudis and Pitilakis (2012) proposed a numerical approach for construction of seismic fragility curves for shallow metro tunnels in alluvial deposits and the results were compared with a closed form solution. The comparison between the proposed fragility curves and the close form ones highlights the important role of the local soil conditions, which is not adequately taken into account in the empirical curves.

Masoomi and Lindt (2016) have investigated the performance of a masonry building subjected to tornado wind loads using a fragility methodology and tried to provide fragility functions applicable to different locations throughout the United States.

Huang *et al*. (2017) suggested an analytical method to develop seismic fragility analysis for rock mountain tunnels based on support vector machine (SVM) and they concluded that SVM can provide accurate estimation of fragility curves considering multiple uncertainties.

As mentioned before, in the present study, PGA was chosen as IM and a lognormal distribution of PGA taken as Probability Density Function (PDF). Indeed, seismic fragility curve is a Cumulative Distribution Function (CDF) of the IM variable e.g., PGA and can be expressed as the integral of its PDF

$$F(X) = \int_{-\infty}^{x} f(t)\, dt \tag{6}$$

where $F(X)$ is *CDF* of a continuous random variable X and $f(t)$ is *PDF* of variable t (i.e., PGA). Based on Eq. (5) the fragility curves can be obtained for each limit states (R), and then for the three mentioned limit states of 1%, 2.5% and 5% of tunnel convergences, three different seismic fragility curves were derived. Fig. 6 illustrates the seismic fragility curves of the tunnel under study at different limit sates.

Probability of exceedances (P) for an earthquake with *PGA*=0.35 g are shown in Fig. 6 at different limit states. As depicted in Fig. 6, the probability of exceedance of the tunnel instability is different for various limit states. For example, an earthquake with PGA equal to 0.35 g can lead the tunnel under study to collapse or severe damage (limit

Table 5 probability of the exceedance at three limit states and different return periods

Limit states	Return period of Hazard								
	T=475			*T*=100			*T*=50		
	LS 1	LS 2	LS 3	LS 1	LS 2	LS 3	LS 1	LS 2	LS 3
P_f^{100}	0.544	0.273	0.148	0.293	0.115	0.055	0.206	0.066	0.031
P_f	0.0078	0.0032	0.0016	0.0035	0.0012	0.0006	0.0023	0.0007	0.0003

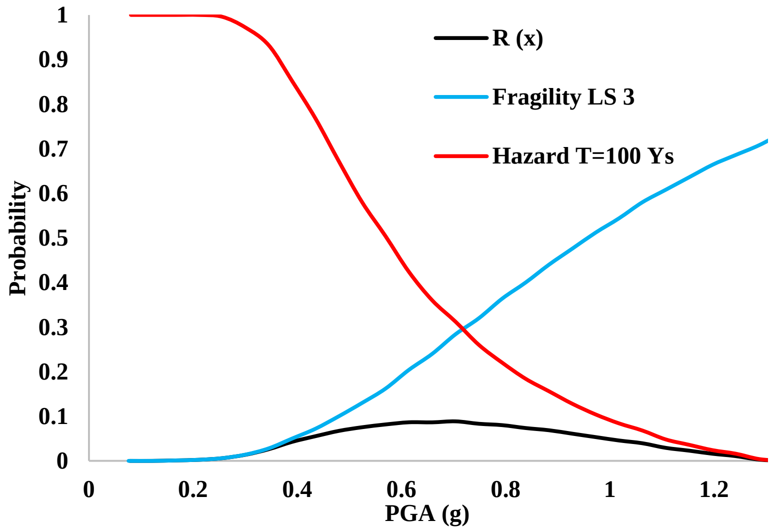

Fig. 8 Risk curve of the tunnel by convolving fragility and hazard curves

state 3) with only 3% probability and the probability of moderate damage to the tunnel (limit state 2) is 12%. Also the fragility curve corresponding to the limit state 1 shows that the mentioned earthquake induces no damage or minor damage to the tunnel with 57% probability.

7. Seismic risk assessment

Seismic risk assessment as a quantitative estimation of damages or losses resulting from a probable earthquake event requires calculations of two components of risk: (1) the magnitude of damage due to a hazard, and (2) the probability that the hazard will occur. Hence, it is necessary to have a hazard curve related to the tunnel site. Fig. 7 illustrates hazard curves for site of the tunnel at different return periods.

However, the risk curve of damage from an earthquake occurrence to the tunnel can be obtained by convolving fragility with hazard curve

$$P_f^{100} = \int_{x=0}^{x=\infty} Fr(x).H_{IM}(x)dx = \int_{x=0}^{x=\infty} R(x)dx \tag{7}$$

where $Fr(x)$ is the fragility function (Fig. 6), $H_{IM}(x)$ is the hazard function (Fig. 7), $R(x)$ is risk curve and P_f^{100} is probability of exceeding the critical limit in 100 years.

The probability of exceeding each limit state can be obtained by convolving the corresponding fragility curve of the tunnel and hazard curve. Fig. 8 shows risk curve of the tunnel by convolving the fragility curve of limit state 3 and hazard curve in 100-years return period. The area under the curve of $R(x)$ represents the probability of the tunnel collapse or severe damage in 100-years which is equal to 5.5%, which is approximately equal to an 1800-year return period. If the hazard curves were developed based on annual probabilities it would be the exceedance probability per year (P_f). For determining the n-year probability of exceeding at a certain limit state, it can be written

$$P_f^n = 1-(1-P_f)^n \tag{8}$$

Table 5 presents 100-years and annual probability of the exceedance at different limit states of the tunnel.

Table 5 summarizes probability of the exceedance at the three limit states and different return periods. All of the probabilities were calculated for the age of 100 years for the tunnel under study. Based on the data reported in Table 5, the annual probability of severe damage or collapse is only 0.06% in a 100 years.

8. Conclusions

In the present study, seismic response of an unsupported tunnel has been considered at different limit states using IDA. Also seismic fragility and instability risk of the tunnel were evaluated on the basis of IDA results and hazard curve of the tunnel site. The results of this study summarized as follow:

On the basis of IDA curves results, the records possessing PGA less than 0.5 g could not exceed limit state 2 (moderate damage) and also the records with PGA less than 0.19 g will produce no damage or minor damage in the tunnel, because they could not exceed first limit state. This finding is agree with previous reported observations in the field.

Although empirical studies on the basis of some case histories indicated that the earthquake with PGA>0.5 g produce a severe damage to tunnels, the results of IDA curves showed that only 13% (2:15) of the earthquakes with PGA equal to 0.5 g induce a severe damage or collapse to the tunnel under study. This is because, in addition to PGA, there are some more influencing parameters on the seismic response of the underground structures and it will vary from case by case.

Irrespective of the earthquake magnitude and frequency content, an earthquake with PGA equal to 0.35 g can lead to severe damage or collapse of the tunnel under study with only 3% probability and the probability of moderate damage to the tunnel is 12%. Also the earthquake with PGA=0.35 g induces no damage or minor damage to the tunnel with 57% probability.

The tunnel under study will exceed damage state 3 (severe damage or collapse) with a probability of 5.5% in 100-years which is approximately equal to an 1800-year return period, also the annual probability of the collapse is equal to 0.06%.

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