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SEISMIC FRAGILITY ASSESSMENT OF MULTISTORY UNDERGROUND STRUCTURES IN DIFFERENT SOIL SITES

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Abstract

Structural seismic fragility analysis, which establishes the probabilistic relationship between the seismic damage probability of structures and the intensity of ground motions, is a key component in the framework of performance-based earthquake engineering (PBEE). Compared to the increasing popularity of PBEE methodology in the seismic performance assessment of the aboveground structures, deterministic approaches prevail in the seismic response analysis of underground structures, which fails to explicitly consider the uncertainties of the ground motions. This paper presents a numerical procedure based on two-dimensional nonlinear incremental dynamic analysis (IDA) to develop seismic fragility curves for multistory underground structures buried in different types of soil sites. Equivalent linear model is adopted in the analysis to simulate the shear modulus degradation and damping characteristics of the soil under seismic excitations. The hysteretic behavior of the multistory underground structures under ground shaking is simulated using fiber beam-column elements. An ensemble of 21 ground motions recorded on the ground surfaces are firstly back-calculated through simplified one-dimensional site response analysis to obtain the seismic input motions at the level of engineering bedrock. Those bedrock motions are collectively scaled up to specific intensity levels based on the median peak acceleration and subsequently used as the input motions for nonlinear IDA. Based on statistical analysis of the IDA results, it is found that the peak acceleration at the ground surface is an efficient and appropriate intensity measure of the ground motions for shallowly buried multistory underground structures and used to construct the seismic fragility curves of underground structures for different soil sites. Besides, both the characteristics of input ground motions and soil conditions play important roles in the seismic response of the underground structures. The seismic fragility curves obtained from this numerical study are validated by comparing with the existing empirical and numerical seismic fragility functions of buried rectangular underground structures and can be used as an effective tool to quickly assess the seismic performance of the underground structures.

Keywords: seismic fragility analysis, incremental dynamic analysis, multistory underground structure, damage state

1. Introduction

The underground structures usually play a critical role in the urban infrastructure systems in maintaining the functionality and economic well-being of the region, Recent large earthquakes led to severe damage even total collapse of several underground structures, such as the 1995 Kobe Earthquake in Japan [1], the 1999 Kocaeli Earthquake in Turkey [2], the 1999 Chi-Chi Earthquake in Taiwan [3], the 2008 Wenchuan Earthquake in China [4], the 2016 Kumamoto Earthquake in Japan [5] and so on. Typical forms of seismic damage of the underground structures include concrete spoil, lining cracks, exposure and buckle of reinforcements, groundwater inrush, lining dislocation, portal failure, shear failure of central columns and complete collapse of the structures [1-5]. The retrofit and reconstruction of seismic damaged underground structures are often extremely difficult and challenging, which significantly impair the emergency response capability of the cities in facing natural disasters. Therefore, it is very important to effectively evaluate the seismic fragility of underground structures, especially the shallowly buried underground structures, which are vulnerable to seismic shaking.

Seismic fragility analysis, or seismic damage prediction, aims at establishing the relationships between the seismic damage probability of structures and the intensity of ground motions, which is an important component in the performance-based earthquake engineering (PBEE) methodology for engineering structures [6-8]. With the development of computational techniques in earthquake engineering research, more and more researchers develop seismic fragility curves for different types of underground structures generally following the probabilistic-based PBEE framework. Argyroudis and Pitilakis [9] investigated the seismic response of shallowly buried tunnels in alluvial deposits using a quasi-static approach by considering the uncertainties of ground motions and established PGA-based fragility curves for tunnels with both circular and rectangular cross sections in different types of soil. Zhong et al. [10] presented a simple framework for developing seismic fragility curves for buried straight segmented water pipelines with push-on joints using incremental dynamic analysis (IDA) method. The numerical results indicate that cured-in-place-pipe liner technology can effectively improve the seismic performance of buried segmented pipelines. Zhong et al. [11] also incorporated the IDA method [12] to assess the seismic performance of the Daikai subway station. Quantitative thresholds for the five limit states of the subway station, were proposed based on nonlinear static pushover analysis of the buried structure and failure probabilities of the subway station corresponding to different performance levels were established in this study. Qiu et al. [13] adopted uniform design method to consider the variabilities of rock property, burial depth, tunnel diameter, and lining thickness of the circular tunnel and performed seismic fragility analyses of mountain tunnels. Argyroudis et al. [14] studied recent developments on fragility assessment of critical transport infrastructures subjected to various natural hazards, presented a comprehensive review of the available fragility models, and discussed the main modelling challenges for the generation of analytical fragility functions for different infrastructures.

The traditional seismic performance assessment of underground structures based on expert judgment and empirical seismic fragility curves may not able to provide reliable vulnerability evaluation of the underground structures given the limited earthquake damage case studies and the different site conditions. Besides, the using a deterministic approach using several typical earthquake records as the input motions fails to explicitly consider of the randomness the ground motions. In this study, the probabilistic seismic vulnerability assessment of a typical three-story rectangular underground structure is introduced with explicit consideration of the uncertainties of the input ground motions. A series of nonlinear dynamic time history analyses are performed using the IDA to obtain the seismic response of the underground structures represented by the peak interstory drift ratio (IDR), which is one of the most straightforward and well-recognized damage indices (DIs) in the seismic performance assessment of structures [15-16]. The seismic fragility curves of the three-story underground structure embedded in two different typical engineering sites are established in this study using the PGA at ground surface as the seismic IM of the ground motions and the IDR as the DI of underground structures based on the IDA results.

2. Numerical modeling of soil-structure interaction system

The main section of a typical three-story rectangular underground structure, which is one of the most commonly seen structure types for underground subway station, selected for the case study. Fig. 1 presents the cross-sectional geometries and reinforcement arrangements of the subway station. The reinforced-concrete section has overall dimensions of 18.75 m by 22.0 m and central columns spaced 9.0 m in the longitudinal direction. The columns have a rectangular reinforced-concrete cross section of 0.8 m by 0.8 m. The burial depth from the top of the reinforced-concrete roof of the station to the ground surface is about 10 m. The reinforcement ratios for the slabs, central columns and the side walls are 1.1%, 1.0% and 0.65%. The Grade C30 concrete with the design compressive strength of 30 MPa and the Grade HRB235 reinforcement with the design yield strength of 235 MPa are used in the underground structure with detailed material properties listed in Table 1.

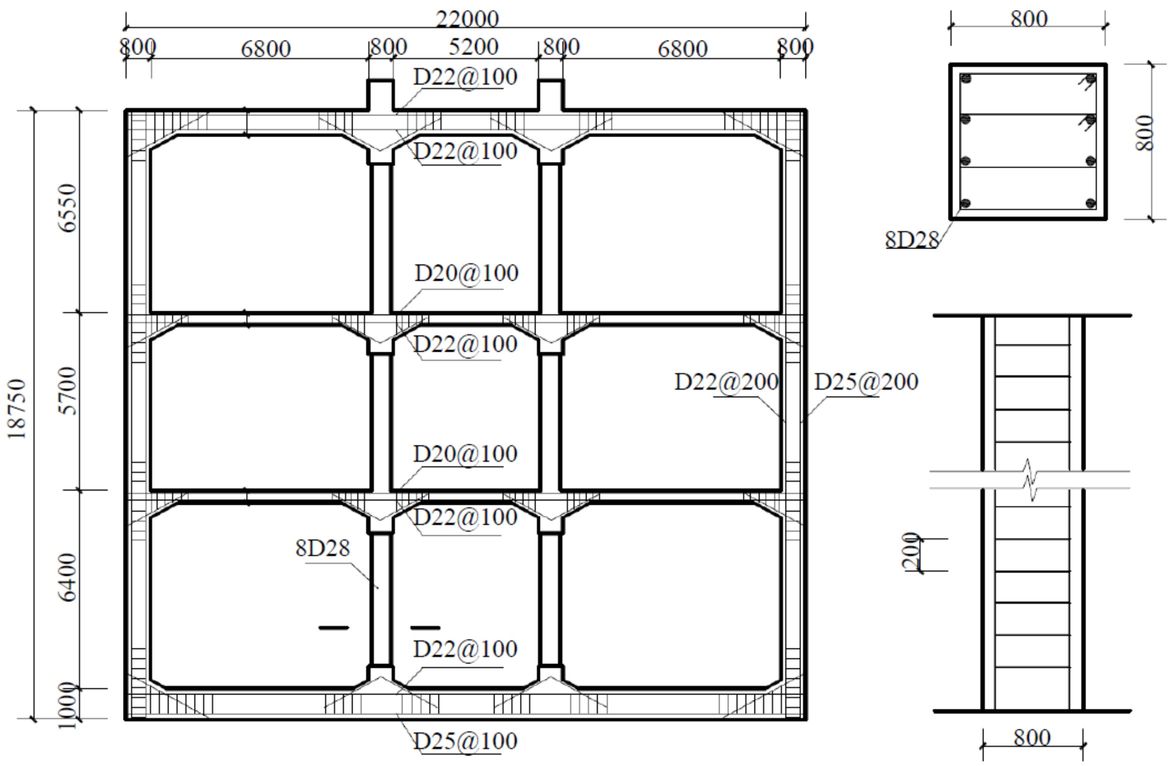


Fig.1 Schematic diagrams of cross-section of subway station structure (unit: mm)

Table 1 Material properties of concrete and steel reinforcement

Material	Density, ρ (kg/m^3)	Elastic modulus, E (GPa)	Poisson's ratio, v	Yield stress, f_y (MPa)	Axial compressive strength, f_{c0} (MPa)	Axial tensile strength, f_t (MPa)	Ultimate compressive strength, f_u (MPa)	Peak compressive strain, ε_{c0}	Ultimate compressive strain, ε_{cu}
Reinforcement	7800	200	0.3	235	—	—	—	—	—
Concrete	2500	24	0.15	—	14.3	2.01	12.2	0.001	0.0038

The seismic response analysis of the transverse seismic response of the subway station is conducted using the general-purpose finite element software ABAQUS [17]. The two-dimensional (2D) finite element model of the nonlinear soil-structure interaction system is illustrated in Fig. 2. The four-node quadratic reduction integral plane strain element (CPE4R) is used to simulate the seismic response of the surrounding soil. The equivalent linear models as shown in Fig. 4 are adopted in the numerical analyses to approximately consider the nonlinearity of the soil under earthquake excitations [18]. The nonlinear fiber beam-column element, PQ-Fiber [19] developed based on subroutine in ABAQUS is used herein to simulate the nonlinear behavior of the reinforced concrete structure. The constitutive models for the reinforcement and the concrete are illustrated in Fig. 3. It should be pointed out that the actual spacing of the columns (9.0 m) in the longitudinal direction of the station is taken into consideration in the numerical analyses through the reduced initial stiffness of the columns, by dividing the initial elastic modulus and the strength of the reinforcements and the concrete by 9.0.

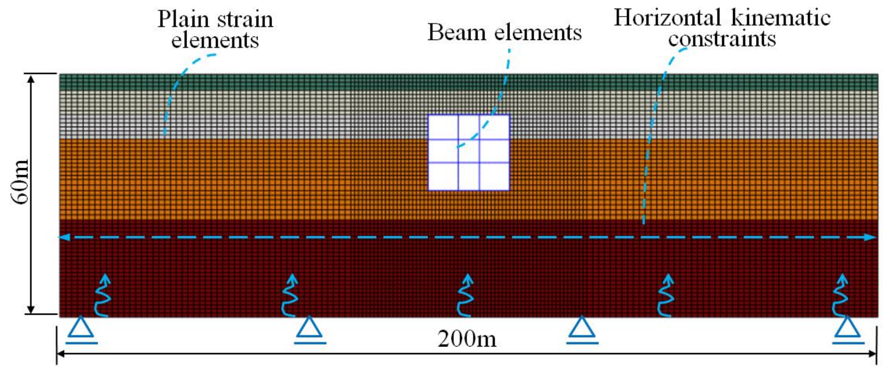
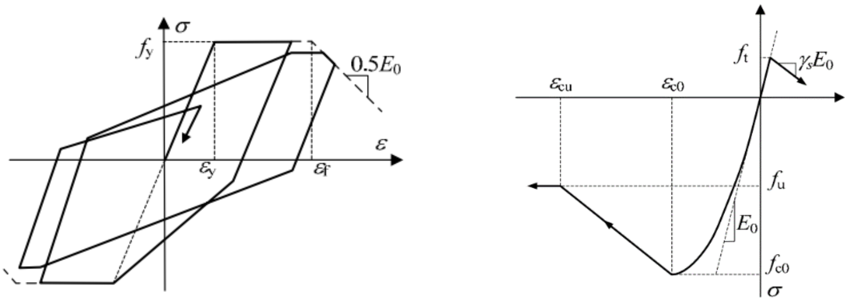


Fig. 2 Finite element model of soil-structure interaction system



(a) Steel [20] (b) Concrete [21]

Fig.3. Constitutive models for concrete and reinforcements

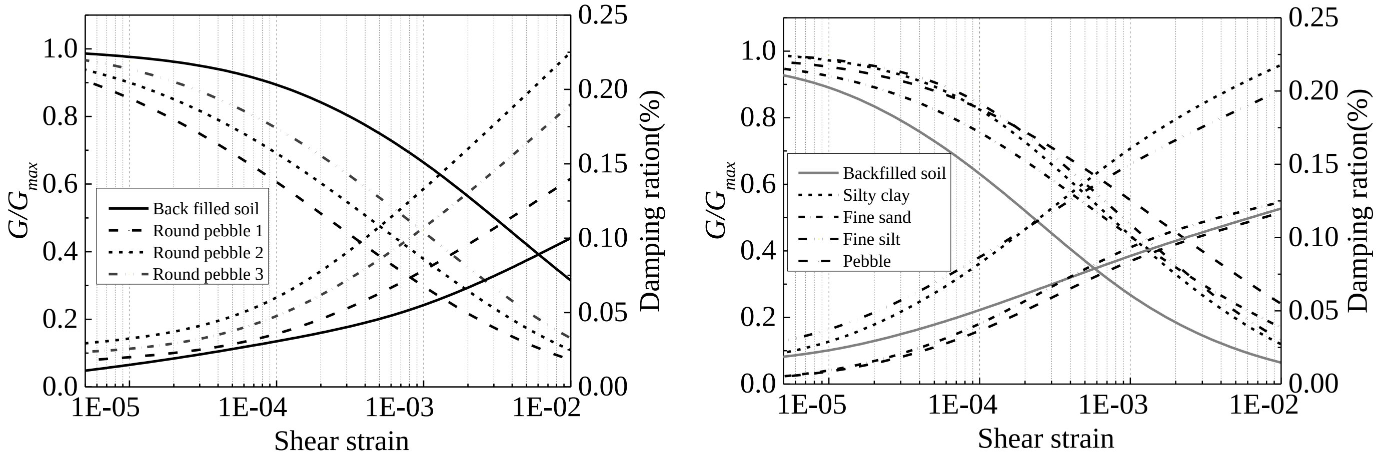
Two different soil profiles with the equivalent shear wave velocities of 400 m/s and 237 m/s are selected in this study to evaluate the influence of different site classes to the seismic response of the underground structures. Those soil sites can generally be classified as Classes B, C according to Eurocode 8 [22] and Classes I, II according to the Chinese code for seismic design of urban rail transit structures [23]. The overall depth of the soil deposit above the engineering bedrock for the two sites is 60 meters. Table 2 shows the geotechnical properties of different soil layers and the shear wave velocity distributions of the two sites. The shear modulus ratio reduction (G/G_{max}-γ) curves and the material damping increase (D-γ) curves of different soil layers based on laboratory test results [18] are illustrated in Fig. 4. The four-node quadratic reduction integral plane strain element (CPE4R) is used to simulate the seismic response of the surrounding soil. In order to consider the nonlinear behaviors of the soils, the equivalent shear moduli and the equivalent damping ratios of different soil layers are firstly obtained using the Equivalent-Linear Earthquake Site Response Analyses (EERA) program [24] and subsequently assigned to the soil model in ABAQUS in the nonlinear dynamic analyses.

The three-story and three-span rectangular underground structures is assumed to be buried in the aforementioned two sites with a burial depth from the top slab to the ground surface of 10 m. The interaction between the underground structure and the surrounding soil is achieved by the mechanical contact algorithm in ABAQUS. The mechanical contact property consists of the normal and tangential components with respect to the two contact surfaces. The contact pressure-overclosure relationship which only allows the transmission of normal contact pressure after the contact of the two surfaces, is used in the normal direction of the soil-structure interface. The tangential behavior between the two contact surfaces is simulated using the classical isotropic

Coulomb friction model with a constant friction coefficient, μ = 0.4 [25]. The finite element mesh sizes of the structure and the surrounding soils, which satisfies the accuracy requirement for the dynamic analysis, are shown in Fig. 1. In order to eliminate the influence of boundary effects on the seismic response of underground structures, the overall width of the truncated soil domain is 200 m, approximately 5 times of the width of the embedded structure [26]. The bottom boundary of the model is constrained in the vertical direction to remove rigid body motion. The horizontal kinematic constraint, or namely, the equal-displacement-boundary constraint [27], is adopted on the nodes at the same burial depth on the two side boundaries to impose the same horizontal displacements, which can effectively simulate the vertical seismic wave propagation from the engineering bedrock to the ground surface.

Table 2 Geotechnical properties of soils

Site class	Soil layer	Soil type	Depth (m)	Density (kg/m^3)	Shear wave velocity (m/s)	Poisson's ratio	Shear modulus (MPa)
I	1	Backfilled soil	4.0	1800	300	0.2	162
	2	Round pebble 1	12.0	2100	450	0.25	425
	3	Round pebble 2	20.0	2200	550	0.23	666
	4	Round pebble 3	24.0	2150	600	0.20	774
II	1	Backfilled soil	5.0	1750	180	0.25	57
	2	Silty clay	10.0	1900	250	0.3	118.8
	3	Fine sand	10.0	2000	300	0.25	180
	4	Fine silt	15.0	2000	320	0.25	205
	5	Pebble	20.0	2280	500	0.2	525



(a) Site I (b) Site II

Fig. 4 G/G_{max} -γ and D-γ curves of soil layers in two sites

3. Selection of input ground motions

The seismic ground motions with inherent randomness lead to significant uncertainties in the seismic response of engineering structures [28]. To account for the uncertainties of the earthquake motions, an ensemble of seismic records covering a wide range of PGAs. predominant frequencies, site classes and historical major earthquake events are used as the input motions in the IDA in this study. Following the guidelines in ATC-63 [29], 21 horizontal far-field seismic records are selected from the Pacific Earthquake Engineering Research Center (PEER) strong earthquake record database [30] based on the site conditions, seismic intensities and the epicenter distances, which are sufficient to reflect the inherent record-to-record uncertainties of the ground motions according to the study of Vamvatsikos and Cornell [12]. The selected earthquake moment magnitudes and epicentral distance were in the range of M_w = 6.0-8.0 and R = 5km ~ 135km, respectively. The detailed information for the seismic records used in this study is listed in Table 3. Fig. 5 presents the acceleration response spectra of the selected 21 seismic records. These selected ground motions recorded on the ground surfaces are firstly back-calculated through simplified one-dimensional site response analysis in EERA to obtain the seismic input motions at the level of engineering bedrock. Those bedrock motions are collectively scaled up to specific intensity levels with PGA from 0.05 g to 0.8 g based on the median peak acceleration and subsequently used as the input motions for nonlinear IDA. A total of 147 nonlinear time history analyses are conducted for each site class in IDA herein.

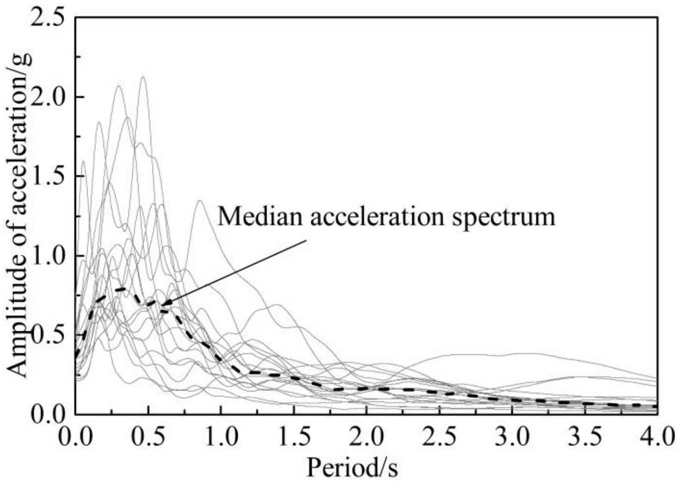
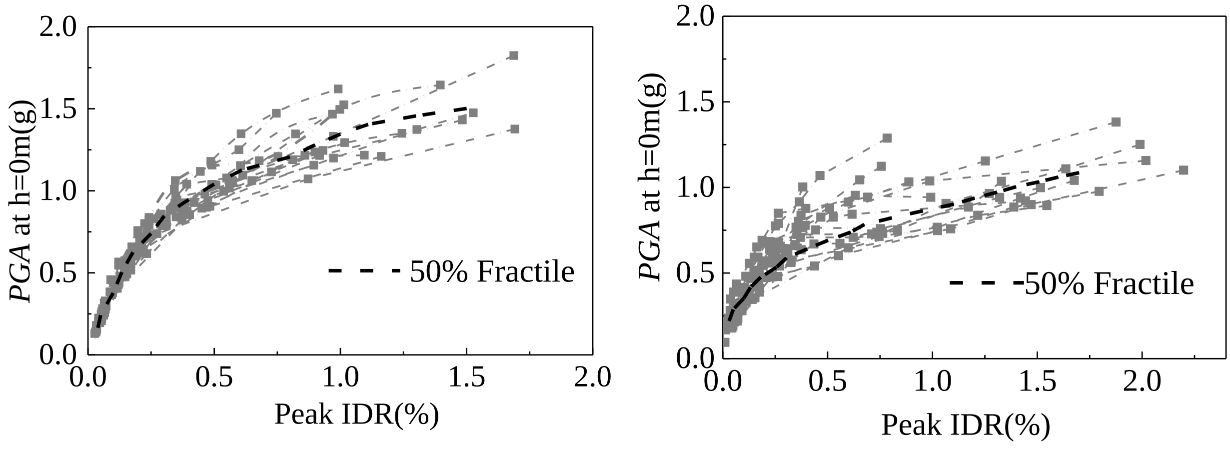


Fig. 5 Spectral acceleration of selected ground motions with damping ratio of 5%

4. Results of nonlinear incremental dynamic analysis

In this study, the peak acceleration at the ground surface is adopted as the ground motions intensity measure (IM), and the peak interstory drift ratio, θ_{max} , of the underground structure is used as the structural damage measures (DM). Fig. 6 (a) and (b) presents the IDA results of three-story and three-span subway station structures for different soil sites. When different ground motion records are used as inputs, there are large differences between different IDA curves, indicating the seismic response of the underground structure is closely related to the characteristics of the input motions. Moreover, it can be seen that the dispersion of the IDA curve is much smaller at the initial stage. As the seismic intensity increases, the dispersion of the IDA results shows a gradually increase trend.

It can also be seen from the median response curves that the median maximum IDR of the structure under site class II is 2.25%, and the median maximum IDR of the structure under site class I is 1.75%. Since the underground structure is constrained by the surrounding soil, the seismic response of the structure generally follows the ground deformation. Generally, under the same intensity of ground motions, the site response is smaller with the as the engineering site changes from site class II to site class I. Therefore, the peak deformation of the structure decreases with the improvement of the engineering site conditions.



(a) site class I (b) site class II

Fig. 6. IDA curves

5. Fragility analysis

5.1 Definition of damage states

The interstory drift ratio (IDR) is one of the most straightforward and well-recognized damage indices (DIs) in the seismic performance assessment of structures. Du et al. [31] performed a series of static pushover analyses on 18 different rectangular underground structures considering nonlinear soil-structure interactions to statistically quantify the threshold values for the different damage states. Five structural damage states from 'no damage' to 'completely collapse' are quantitatively defined based on the average IDR of the 18 underground structures obtained from the pushover analyses. In this paper, the DIs for the three- story three-span underground structure are defined based on Du et al.'s work [31] as shown in Table 4.

Table 4 Definition of damage states for interstory drift ratio of underground structure [31]

Damage state	Functional status	Range of DI	Median value of DI
No damage	Fully operational	$\theta_{max}\leq 1/1223$	—
Minor damage	Immediate occupancy	$1/1223<\theta_{max}\leq 1/343$	0.19%
Moderate damage	Functional with moderate reparation	$1/343<\theta_{max}\leq 1/161$	0.46%
Extensive damage	Life safety	$1/161<\theta_{max}\leq 1/105$	0.79%
Completely collapse	Complete loss of function	$\theta_{max}>1/105$	—

5.2 Fragility curves

It is generally assumed that the seismic fragility curves of engineering structures follow a two-parameter lognormal distribution [9, 32]. The conditional probability of the structural response exceeding the structural demand capacity parameter is defined in the failure stage under different intensity earthquake. The formula can be expressed as:

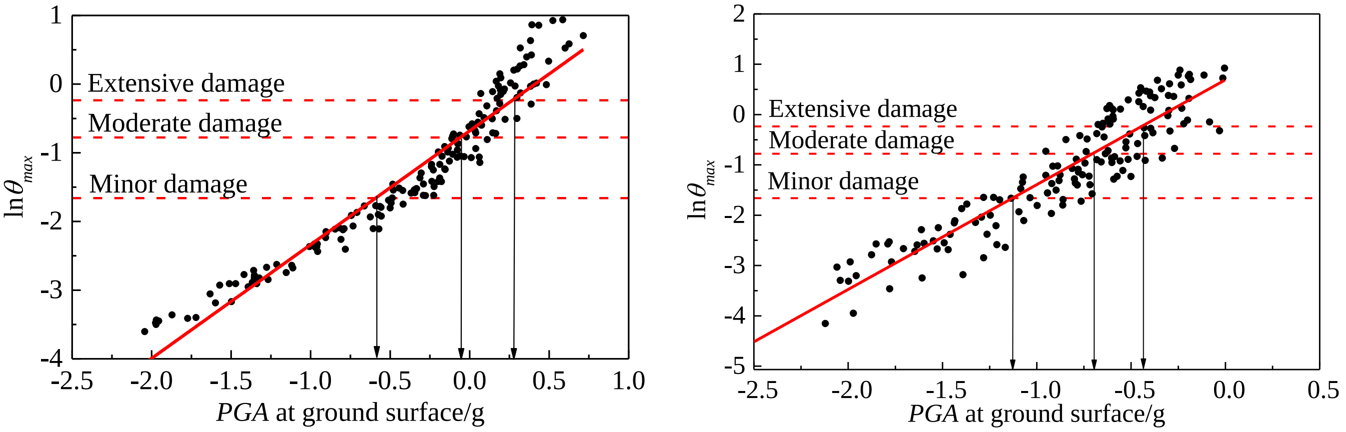
$$P_f(d_s \geq d_{si} \mid S) = \Phi[\frac{1}{\beta_{\mathrm{tot}}}\ln(\frac{S}{S_{mi}})] \tag{5}$$

In the formula, P_f represents the probability that the response of a structure under a certain seismic intensity exceeds a certain performance level, d_{si}. d_s represents a certain performance level of the structure under a certain earthquake intensity, and S refers to the selected ground motion IM in this paper. Φ represents the standard normal cumulative probability function. S_{mi} represents the intermediate critical value of the ground motion IM under a certain damage state. β_{tot} represents the total lognormal standard deviation, where β_{tot} represents the total lognormal standard deviation and can be estimated using Eq. 6.

$$\beta_{tot} = \sqrt{\beta_{DS}^2 + \beta_C^2 + \beta_D^2} \tag{6}$$

where β_{DS} is the uncertainty in the definition of different damage states, which is assumed to be equale to 0.4 according to the requirements of the earthquake disaster loss risk assessment software (HAZUS) [33] β_C represents the influence of structural form uncertainty on the bearing capacity and is neglected in this study, because the structural form in this paper is well-defined; β_D is the mean standard deviation of the structural damage induced by the ground motions at each intensity level in the linear logarithmic regression coordinate system ln(IM)-ln(DM). In this study, both β_D and S_{mi} are obtained by linear regression of nonlinear IDA results using the least-squares method.

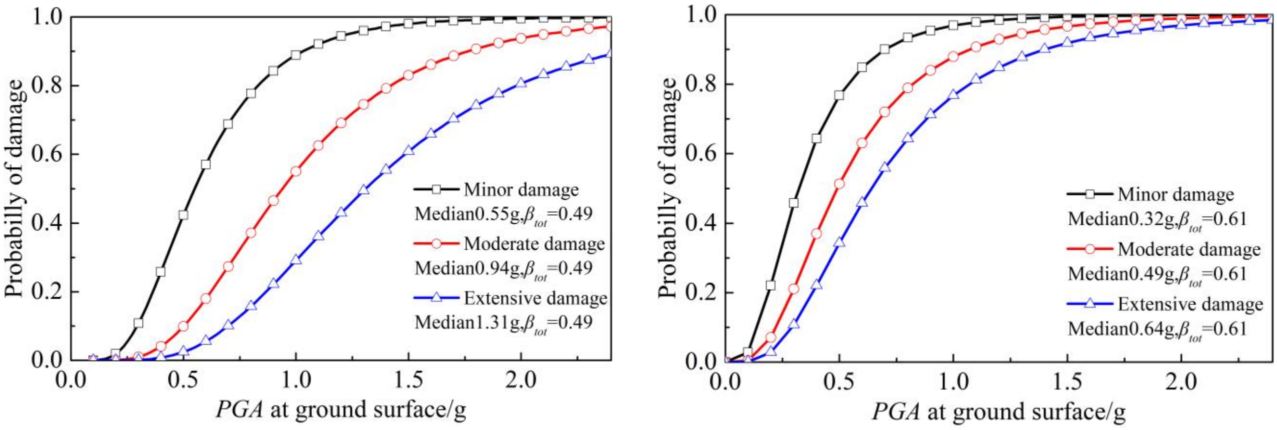
Through the statistical linear regression analysis of the IDA results of the soil-structure interaction systemas shown in Fig. 7, the intersections of the intermediate values of the different limit states as listed in Table 5and the linear fitting curve are obtained, which are the values of S_{mi} corresponding to different limit states. The S_{mi} values of the site class I are 0.55, 0.94, 1.31, and the normal standard deviation β_{tot} is 0.49. The S_{mi} values of the site class II are 0.32, 0.49 and 0.64, and the normal standard deviation β_{tot} is 0.61.



(a) Site class I (b) Site class II

Fig. 7. Estimation of median threshold values of *PGA* at ground surface for each damage state of the underground structure

Fig. 7 presents the seismic fragility curves of the three-story and three-span subway station structure with the *PGA* at the ground surface as the abscissa and the structural failure probability as the ordinate on the two types of sites. The probabilities of the underground structure under the performance levels of slight damage, medium damage and serious damage under different intensities of ground motions can be directly obtained from the seismic fragility curves.



(a) Site class I (b) Site class II

Fig. 8. Fragility curves for the underground structure

As shown in Fig. 8, for site class I, the *PGA* for minor damage, moderate damage and extensive damage with a corresponding probability of 50% are 0.55 g, 0.94 g and 1.31g. For site class II, the *PGA* of minor damage, moderate damage and extensive damage with a probability of exceeding 50% are 0.32 g, 0.49 g and 0.64 g respectively. It can be seen that the threshold *PGAs* at the ground surface corresponding to extensive damage of the subway station embedded in two different sites are very large with a median amplitude of 1.31 g and 0.64 g, respectively, based on the numerical results, which is consistent with the limited seismic damage records for the extensive damage of the tunnels in history. It can be seen from the above comparison that the engineering site conditions have significant influence on the seismic fragility curves of the same underground structures in additional to the characteristics of the ground motions. The subway station is less vulnerable to the same intensity of earthquake motions when it is embedded in a better engineering site conditions (Site class I).

Those fragility curves of the three-story and three-span rectangular subway station developed based on IDA in this study are compared with the empirical fragility curves developed by the American Lifelines Alliance (ALA) [34] based on rectangular cut-cover tunnel damages from past earthquakes and the numerical fragility curves of rectangular tunnels produced by Argyroudis and Pitilakis [9] in Figs. 9 and 10. It is assumed the underground structure is in good quality construction before the earthquake, and the strength and the stiffness degradation of the structures due to aging and corrosion are not considered in the comparison. The empirical fragility curves from ALA were derived based on the past seismic damage observations without consideration of the characteristics of the site conditions and the damage records of the rectangular cut-cover tunnels with extensive damage or collapse are very limited, which leads to significant variation in the statistical results. It should also pointed out that the empirical fragility curves corresponding to extensive damage of the underground tunnels are not currently available in ALA. Besides, the probabilities of seismic damage to the three-story and three-span underground structure from this study are also different from the seismic fragility curves developed by Argyroudis and Pitilakis [9] for rectangular tunnels in two different sites. It is believed that those differences between the numerical fragility curves can be mainly attributed to the selection of types and ranges of damage indices, the detailed configurations of the engineering sites, the burial depths of the structures and the numerical analysis methods. Argyroudis and Pitilakis [9] adopted a quasi-static approach in the numerical analysis and used the ratio between actual and capacity bending moment of the tunnel cross section as the damage index in developing the fragility curves. However, this study performed full dynamic time-history analysis and used the interstory drift ratio as the damage index in the development of fragility curves for the subway station. In addition, the central columns are generally believed to be the weakest link of the underground structures during the earthquakes, which lead to the three-story and three-span underground structure more vulnerable to earthquake excitations compared to the box structures without central columns used in Argyroudis and Pitilakis's study [9].

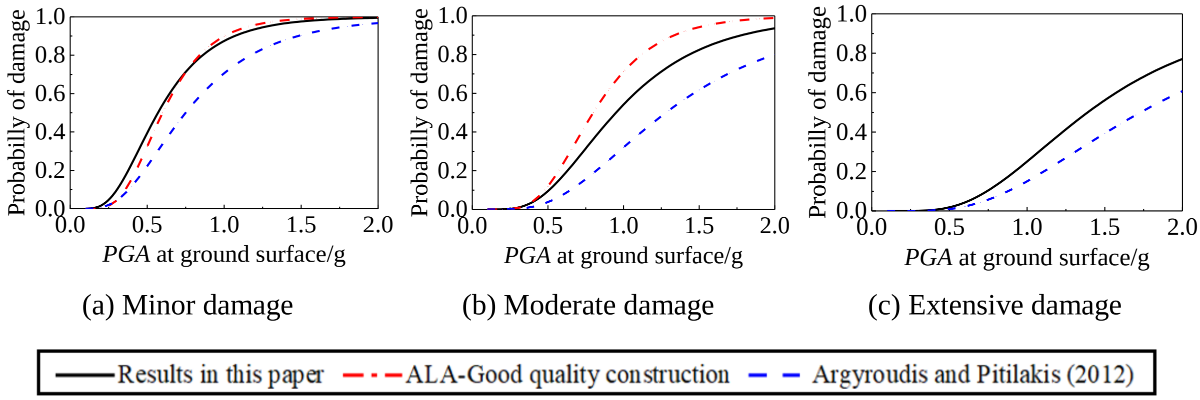


Fig. 9. Comparison between numerical and empirical fragility curves for rectangular cut-cover tunnels in site class I

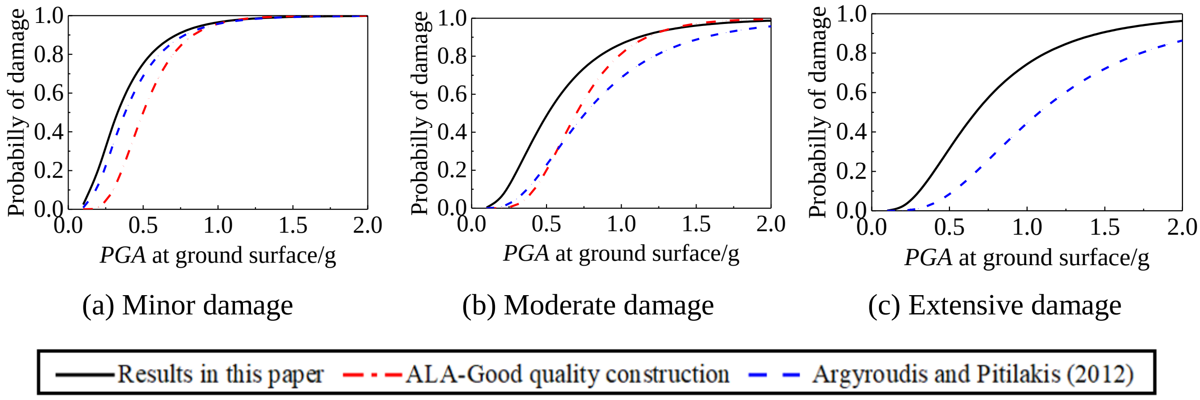


Fig. 10. Comparison between numerical and empirical fragility curves for rectangular cut-cover tunnels in site class II

6. Conclusions

A numerical procedure based on two-dimensional nonlinear incremental dynamic analysis is proposed in this paper to develop seismic fragility curves for multistory underground structures buried in different types of soil sites. Equivalent linear model is adopted in the analysis to simulate the shear modulus degradation and damping characteristics of the soil under seismic excitations. The hysteretic behavior of the multistory underground structures under ground shaking is simulated using fiber beam-column elements. The seismic fragility curves are established in this paper as a function of the peak ground acceleration at the ground surface for the underground structures based on the results from the incremental dynamic analysis. The following conclusions can be drawn from this study.

(1) The seismic fragility curves of the underground structure developed using the proposed method are validated with available empirical curves and numerical results, and can be used to approximately evaluate the earthquake vulnerability of the rectangular subway station buried in similar engineering sites.

(2) Based on statistical analysis of the IDA results, it is found that the peak acceleration at the ground surface is an efficient and appropriate intensity measure of the ground motions for shallowly buried multistory underground structures and used to construct the seismic fragility curves of underground structures for different soil sites.

(3) Both the characteristics of input ground motions and engineering site conditions play important roles in the seismic response of the underground structures.

7. Acknowledgements

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8. Copyrights

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