

Article

Seismic Vulnerability Evaluation of Metro Station Complex Structures in TOD Mode Based on IDA Method

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Abstract: In order to further study the dynamic response and damage status of the subway station structure and promote the development of the TOD (transit-oriented development) mode structure system, this paper proposes a calibration method for the seismic performance index limit of the subway station complex structure in TOD mode. Taking a practical project in the Beijing city sub-center station integrated transport hub as the research background, the nonlinear analysis model of soil–structure interaction under different site types is established. Firstly, the limit value of the interstory drift ratio is determined by the pushover loading method of the inverted triangular distributed load for the three-dimensional numerical model. Secondly, different types of seismic waves are selected to analyze the seismic vulnerability of the simplified two-dimensional numerical model, and the exceedance probability of different damage states of the structure is quantitatively analyzed. By analyzing the pushover curve, the maximum interstory drift ratio limits corresponding to the five damage states of the subway station complex structure are 0.14%, 0.32%, 0.66%, and 1.12%, respectively. Under different site types and different types of seismic waves, the seismic response law of subway station structures in TOD mode is different. Using different types of ground motion as the input, the mean and discreteness of different IDA curve clusters are quite different. The near-field pulse-type ground motion has a greater impact on the ground motion of the structural system under the Class II site, and the far-field long-period ground motion has a greater impact on the structure under the Class III site. Damage decreases with the increase in the equivalent shear wave velocity of the site, that is, the harder the site's soil is, the less susceptible the structural system is to damage by underground motion. The established seismic vulnerability curve and seismic damage probability table can effectively evaluate the seismic performance of subway station complex structure in TOD mode. The research results can provide a valuable reference for the seismic performance evaluation of similar underground structures.



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Keywords: underground structure; subway station complex structures; seismic performance; seismic vulnerability; damage probability; IDA (incremental dynamic analysis) method

1. Introduction

In order to realize the intensive utilization level of land resources, in the planning and construction of underground subway stations, it is important to comprehensively consider the overall development mode of land around the station and underground subway stations to solve the shortage of urban land resources, improve traffic congestion,

and promote the integration of underground and aboveground space. The subway superstructure strengthens the integrated planning and comprehensive utilization efficiency of subway stations and the surrounding land, improves the comprehensive development and utilization level of subway stations, and makes rational use of urban resources [1]. The large space at the bottom of the TOD (transit-oriented development) mode of rail transit is an underground space complex with subway stations as the core and underground commercial development as the auxiliary. The upper tower can be used as office buildings, residences, hotels, etc., to meet people's housing, office, travel, and other living needs, as shown in Figure 1. Compared with the traditional subway station with an overlying soil layer, the upper part of the subway station in TOD mode is the tower, and its seismic response characteristics and failure mechanism are not clear.



Figure 1. TOD model architecture in China; (a) Shanghai Vanke Sky City; (b) Xiamen Caicuo Vehicle Base; (c) Chongqing Longhu Times Street.

The Pacific Earthquake Engineering Research Center (PEER) in the United States proposed the seismic vulnerability module within the probabilistic seismic hazard analysis (PSHA) framework to describe the damage levels of structures under various intensities of seismic action. This module quantitatively expresses the exceedance probabilities of different damage states of structures under different levels of seismic action. Research on the seismic vulnerability of aboveground structures has been extensive. Lu Xinzheng et al. [2] conducted a comparative analysis of the collapse processes of reinforced concrete (RC) frame structures and RC frame-core tube structures under unidirectional horizontal ground motion and three-dimensional ground motion using the incremental dynamic analysis (IDA) method. Their study indicated that structures exhibit more collapse modes under three-dimensional ground motion, whereas considering only unidirectional ground motion results in lower vulnerability assessments. Qiu Yongcun et al. [3] analyzed an RC frame structure using the IDA method and concluded that both the inherent properties of the structure and the type of ground motion significantly affect structural ductility, with stiffness being the primary controlling factor. Han Xuechuan et al. [4] investigated the differences in dynamic responses between integrated subway station structures with upper covers and standalone station structures under horizontal seismic action through numerical simulations. Yang Liansen et al. [5] performed a vulnerability analysis of a high-rise building in Suzhou under the combined effects of seismic and wind loads, demonstrating that under multi-hazard coupling, the exceedance probabilities of all damage states increase with the growth of coupled loads.

In recent years, with the growth and concentration of urban populations, the rapid development and utilization of underground space have accelerated the expansion of underground structures. Consequently, researchers have increasingly begun to study the seismic vulnerability of underground space structures [6]. The American Lifelines Alliance [7] and the HAZUS system [8] have developed empirical vulnerability curves for

various types of tunnels under different levels of peak ground acceleration (PGA), based on observed damage states from past seismic events. These curves summarize the exceedance probabilities of different damage levels and predict the extent of structural damage under earthquakes of varying intensities. Currently, numerical analysis methods are widely used for seismic vulnerability assessments of structures. Zhong Zilan et al. [9] conducted a seismic vulnerability analysis of a two-story, three-span subway station using the IDA method, providing quantitative exceedance probabilities for various damage states. Dong Zhengfang et al. [10] used the SAP2000 finite element software to model underground utility tunnel structures and investigated the effects of factors such as site classification, burial depth, and cross-sectional dimensions on their seismic vulnerability. Jiang et al. [11] conducted a study on the vulnerability of 18 existing metro stations under different site conditions, providing a basis for evaluating the seismic performance of existing urban underground frame structures in China. Duy et al. [12] conducted a vulnerability analysis of rectangular cut-and-cover metro tunnels and developed vulnerability curves for single-box, double-box, and triple-box metro systems. The study indicates that multi-box rectangular tunnels are more susceptible to seismic damage. Zhong et al. [13] proposed vulnerability curves for rectangular metro stations subjected to lateral seismic excitations in layered soils. The proposed method was validated using a large-span station and can serve as an effective approach for developing a seismic vulnerability database for underground structures. Liu et al. [14] conducted a comparative study on the seismic vulnerability of metro station structures before and after the addition of rubber bearings at the column tops. The results indicate that the structural performance is significantly improved after the installation of rubber bearings. Jiang et al. [15] studied several typical rectangular frame-section metro stations and performed vulnerability analyses to compare station performance. The results indicate that the average shear wave velocity of soil layers with the same height as the structure plays a critical role in the vulnerability analysis of the structure. Cheng et al. [16] investigated the seismic vulnerability of the interaction system of metro station structures in saturated soft soil. The proposed method can be applied to vulnerability analysis in saturated soft soil sites. Xu Minze et al. [17] analyzed the vulnerability of subway station structures in saturated sandy soil using the OpenSees platform, offering valuable insights for assessing the safety of such structures in these conditions. Huang et al. [18] developed vulnerability curves for circular tunnels in soft Shanghai soils under transverse seismic action based on IDA results for tunnels at different burial depths. Chen Jinnan et al. [19] utilized ABAQUS software to model prefabricated rectangular subway station structures and analyzed their seismic vulnerability under different site conditions using the IDA method. However, With the trend of urban development shifting toward densification, underground structures are growing larger in scale and are situated closer to surrounding aboveground buildings, resulting in significant interactions between structures. Existing studies on underground structures have been primarily focused on individual, shallow-buried underground structures. However, there is still a lack of research on the impact of adjacent or connected aboveground structures on the seismic vulnerability of underground structures.

With the rapid development of the transit-oriented development (TOD) model centered around metro stations, investigating the seismic vulnerability of metro station complexes under the TOD model is essential. This research not only provides exceedance probabilities for various damage states of the structural system under different seismic intensities but also offers a means to assess potential losses following an earthquake. Moreover, it serves as a guideline for seismic design of such structural systems. Accordingly, this study takes an actual engineering project in Tongzhou, Beijing, as the research object. Based on the incremental dynamic analysis (IDA) method, a seismic vulnerability analysis

is conducted for metro station complex structures under the TOD model in Class II and Class III site conditions. This aims to provide new insights for evaluating the seismic performance of similar underground complex structures.

2. Project Background and Site Conditions

2.1. Project Overview

The calculation model is based on the actual project of the Beijing city sub-center station comprehensive transportation hub. The project is a subway station complex structure system in TOD mode, as shown in Figure 2a. Its underground structure consists of three stories: the negative 1 story, which is a restaurant business area convenient for tourists; the negative 2 story is the public transfer area of train, subway, and bus connection; the negative 3 story is the track and platform story. The project is expected to complete structural construction by the end of December 2024. It is expected to be opened and put into use in 2025. The scale of underground construction will reach 1.28 million square meters. After completion, it will become the largest underground integrated transportation hub in Asia.



Figure 2. Schematic diagram of three-dimensional section of metro station complex structure in TOD mode (unit: m); (a) actual engineering effect drawing; (b) research model.

Due to the complexity of the actual project, and because this paper focuses on the study of the seismic performance of the subway station in the integrated building, the actual project is reasonably simplified, and the simplified model is shown in Figure 2b. The simplified structural system is divided into two parts: the upper cover frame structure and the subway station complex structure. The upper cover frame structure is 40 m high, 32 m long, and 24 m wide. It uses square section frame columns with a cross-sectional size of $0.7\text{ m} \times 0.7\text{ m}$ and is rigidly connected to the top plate of the subway station complex structure. The cross-sectional length and width of the subway station complex structure are 81.1 m and 27.1 m, respectively. It is divided into three stories, upper, middle, and lower, with each story's height being 8 m, 8 m, and 10 m; the thickness of the structural side wall is 1.1 m, and the thickness of the top plate, middle plate, and bottom plate are 1 m, 0.4 m, and 1.2 m, respectively. The size of the middle column of the complex structure is $0.8\text{ m} \times 1.2\text{ m}$, the distribution spacing is 8 m, and C45 concrete is used. The rest of the structural components are all C40 concrete.

2.2. Site Conditions

Site classification plays an important role in the vulnerability analysis of underground structures [12,18]. To analyze the impact of site classification on the seismic vulnerability curves of metro station complex structures, seismic vulnerability analyses were conducted

for the structure under medium-stiffness soil conditions (Class II site) and soft soil site conditions (Class III site). The site division is based on the “GB 50909-2014 Code for Seismic Design of Urban Rail Transit Structures” [20].

For the medium-stiffness site, the study used the site conditions of a metro station complex in Tongzhou, Beijing, where the equivalent shear wave velocity of the soil was calculated as 336.13 m/s, classifying the site as a Class II site. The soil layer thickness was 80 m. For the soft soil site, the study adopted the site conditions of a metro station complex in Suzhou, where the equivalent shear wave velocity of the soil was 198.58 m/s. The soil at this site was softer, with lower bearing capacity, and classified as a Class III site according to the classification system. The soil layer thickness was also set to 80 m. In both cases, similar soil layers with comparable physical properties were combined and simplified into six layers. The key parameters of the soil layers are presented in Table 1.

Table 1. Physical and mechanical parameters of soil layers.

Site Category	Soil Category	Soil Layer Thickness /m	Density /(kg·m ⁻³)	Cohesion /kPa	Friction Angle /°	Elastic Modulus /MPa	Shear Wave Velocity /(m·s ⁻¹)	Poisson's Ratio
Class II site	Miscellaneous fill	4	1750	10	8	92	180	0.35
	Silt fill	6	1800	14	30	171	220	0.37
	Silty clay	6	1900	36	18	472	280	0.41
	Clay	10	1950	20	30	213	300	0.43
	Medium coarse sand	24	2100	0	33	837	350	0.28
	Pebbly boulders	30	2150	0	14	1000	500	0.20
Class III site	Miscellaneous fill	4	1600	10	8	92	120	0.45
	Muddy soil	7.5	1680	10	12	120	140	0.42
	Muddy silty clay	8	1750	13	18	140	150	0.39
	Soft clay	12.5	1850	15	22	180	180	0.37
	Fine silt	28	1900	30	34	653	230	0.30
	Medium coarse sand	20	2050	31	33	837	280	0.28

When selecting ground motion records, the corresponding ground motion records should be selected according to the site category to which the project belongs. According to the equivalent shear wave velocity V_{s20} of the soil and the thickness of the cover layer, the site is divided into I, II, III, and IV categories in the GB50011-2010 Code for Seismic Design of Buildings [21]. The seismic design code of the United States divides the site into five categories A, B, C, D, and E based on the average shear wave velocity V_{s30} of the soil layer within 30 m below the surface. According to the research of LYU Hongshan et al. [22], the American site classification standard corresponds to the Chinese site classification standard as shown in Table 2 below.

Table 2. Correspondence between Chinese and American Standards.

Serial Number	Site Description	Chinese Code	American Code
1	Hard rock areas, which belong to hard rock areas in engineering, have fast ground motion propagation speed, small seismic wave attenuation, and strong seismic resistance.	Class I site	$V_{s30} < 510$ m/s

Table 2. Cont.

Serial Number	Site Description	Chinese Code	American Code
2	The general rock and better soil in engineering belong to the medium foundation, which has a certain seismic capacity, ground motion propagation speed, and moderate attenuation.	Class II site	$260 \text{ m/s} < V_{s30} \leq 510 \text{ m/s}$
3	Soft soil geology or poor rock belong to soft soil areas in engineering. The seismic capacity is poor, the ground motion propagation speed is slow, and the seismic wave attenuation is large.	Class III site	$150 \text{ m/s} \leq V_{s30} \leq 260 \text{ m/s}$
4	Very soft soil or very poor rock belong to very soft areas in engineering. The seismic capacity is very poor, the ground motion propagation speed is very slow, and the seismic wave attenuation is very large.	Class IV site	$V_{s30} < 150 \text{ m/s}$

3. Limit of Structural Performance Index

3.1. Definition of Complex Structure Performance Indicators

GB 50909-2014 The Code for Seismic Design of Urban Rail Transit Structures in China defines different damage states for structures. Du Xiuli et al. [23] analyzed a large number of underground metro station structures using the pushover method for underground structures and classified the structural system into four performance levels and five damage states based on interstory drift ratio (IDR) as the limit value. The interstory drift ratio is defined as the ratio of the difference between the upper and lower displacements of the middle column of the subway station to the height of the story, as shown in Equation (1):

$$\delta = \frac{u_1 - u_2}{H} \quad (1)$$

In the formula, δ is the interstory displacement ratio of the structure; u_1 and u_2 are the peak response displacement of the upper and lower section nodes, respectively; H is the interlayer height.

In this study, numerical simulations were conducted based on actual engineering projects. A three-dimensional numerical model of soil–structure interaction was developed using the finite element software ABAQUS 2024, as shown in Figure 3. The model dimensions are set to $560 \text{ m} \times 80 \text{ m} \times 64 \text{ m}$, with an overall width of 560 m , corresponding to three times the cross-sectional length of the station complex on each side [19]. The height is 80 m , extending to the bedrock at the base.

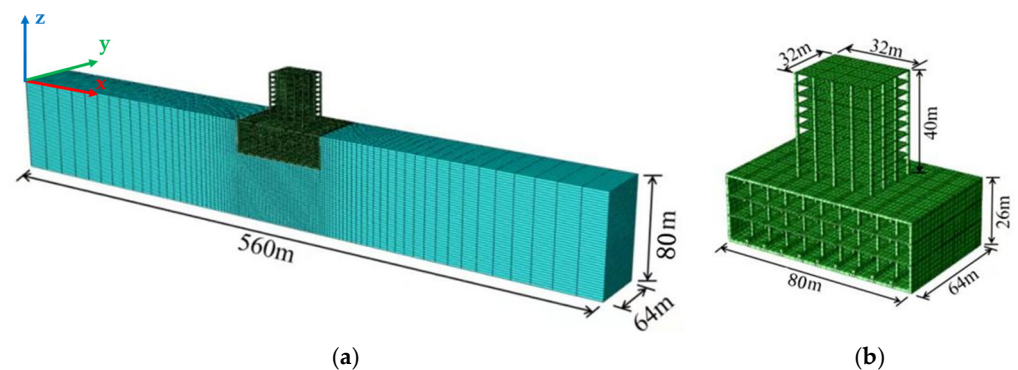


Figure 3. Three-dimensional soil–structure dynamic interaction analysis model (unit: m); (a) three-dimensional integral finite element model; (b) complex structure calculation model.

The soil is modeled using the Mohr–Coulomb constitutive model, while the concrete is simulated using the Concrete Damage Plasticity (CDP) model. The reinforcing steel within the concrete is modeled with an ideal elastic–plastic constitutive model. The schematic diagrams of the constitutive models for various materials are shown in Figure 4. A three-dimensional solid model was developed for the different soil layers and the overall structure, with the reinforcement embedded in the concrete structure as truss elements. The contact interface between the structure and the soil is modeled as surface-to-surface contact, where the master surface and slave surface are assigned to the structure’s exterior surface and the soil surface, respectively. The contact properties are defined with “hard” contact for normal behavior and “penalty friction” for tangential behavior, with a friction coefficient set to 0.4 [24].

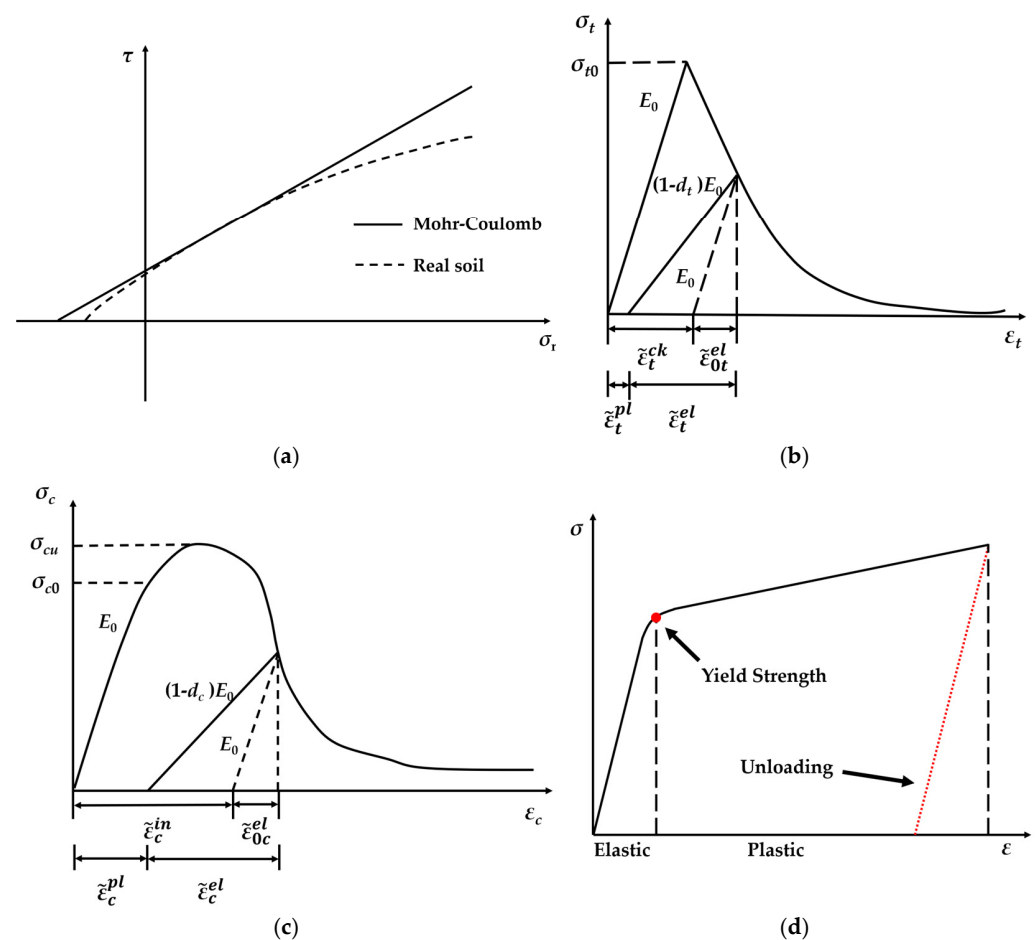


Figure 4. Constitutive model diagram of each material; (a) Mohr–Coulomb yield criterion; (b) uniaxial tensile stress–strain diagram of concrete; (c) uniaxial compression stress–strain curve of concrete; (d) ideal elastic–plastic constitutive model of reinforcement.

In this paper, the pushover analysis of the structure is carried out when defining the limit state of the structure. This method is also recommended by the European standard EuroCode8 [25] as the definition of the limit state. The pushover analysis is divided into two analysis steps: Step 1, involves calculating the geostatic stress equilibrium to obtain the stress field distribution under gravity loading. Step 2 applies the pushover analysis method for underground structures proposed by Liu Jingbo et al. [26]. In this step, an inverted triangular distribution of absolute acceleration loads is applied laterally to the structure to reach the target displacement. For the gravity analysis, the boundary condition of the model is set as the fixed constraint at the bottom, and only the vertical degrees of freedom

are released on both sides of the boundary. In the pushover analysis, the bottom boundary condition is unchanged, and the boundary on both sides is set as the roller boundary. The upper structure is converted into a uniformly distributed load applied to the roof of the subway station, and the horizontal displacement of the inverted triangle distribution is loaded on both sides to analyze the underground structure by the pushover method. The loading method is shown in Figure 5.

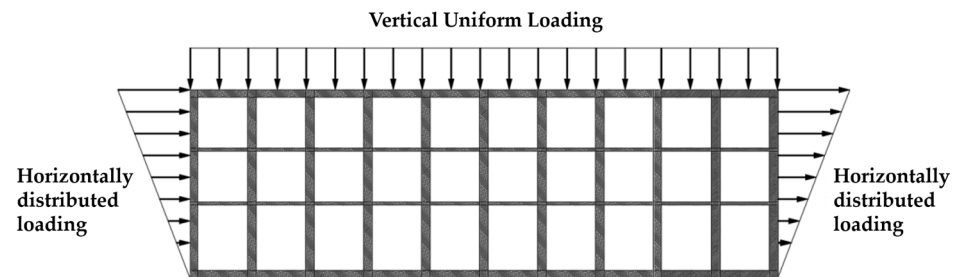


Figure 5. Pushover method analysis loading diagram.

In the process of defining the limit state, the force–displacement curve of the sum of the shear forces at the bottom of the middle column and the interstory displacement ratio is obtained by the pushover method, and then the four limit state points C1, C2, C3, and C4 of the subway station structure are divided, as shown in Figure 6a. The 0–C1 stage represents that the structure is basically in a linear elastic state, and C1 is the node where the slope of the initial stage of the curve begins to change; the C1–C2 stage represents the gradual transition of the structure from the linear elastic state to the elastic–plastic state. The abscissa of C2 is determined by the intersection of the straight extension line of the initial linear stage of the pushover curve and the shear capacity of the middle column. C3 is the interstory drift ratio corresponding to the peak shear point of the middle column of the pushover curve; C4 corresponds to the interstory drift ratio when the shear capacity of the middle column of the pushover curve decreases to 85% of the peak value. Since any two failure states are not independent, the starting point of each failure state is taken as the interstory displacement ratio corresponding to the median value of the adjacent two limit state points, and the occurrence stage of each failure state is characterized.

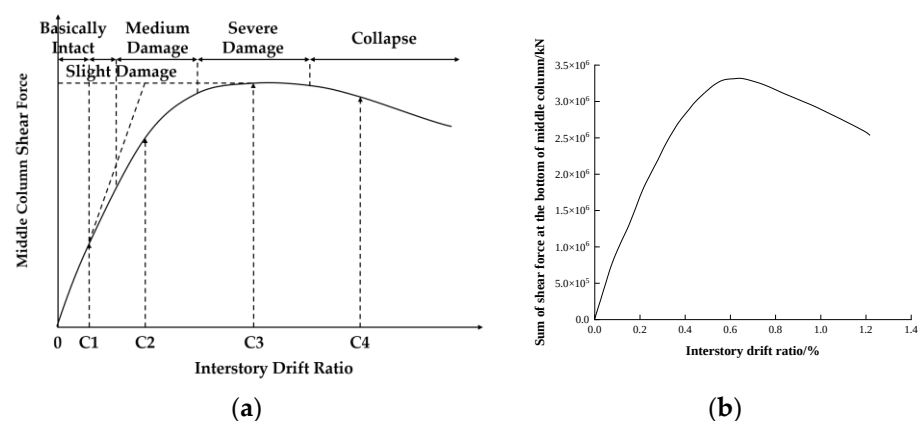


Figure 6. Pushover curve of interstory drift ratio and shear force of middle column; (a) damage index calibration method; (b) structural seismic performance curve.

During the earthquake, the middle column in the bottom story of the structure is more likely to be damaged due to its larger buried depth and greater pressure. Referring to the research of Du et al. [27], the insufficient deformation capacity of the middle column in the bottom of the structure is an important factor in the collapse of the structure, which can

characterize the seismic performance of the structure. The relationship curve between the sum of the bottom base force of the middle column of the lower story of the subway station complex structure in TOD mode and the interstory drift ratio is shown in Figure 6b.

Du et al. [23] divided the subway station structure into four performance levels according to the functional status of the subway station and qualitatively described its functional status, as shown in Table 3.

Table 3. Structural performance level classification and definition for subway stations.

Performance Level	Functional Status	Qualitative Description
LS1	Normal use	The central column of the structure is not damaged, the side walls and top and bottom plates have no visible cracks, and the structure is in normal use.
LS2	Can be used	The structure was slightly damaged, with slight cracks on the columns, top and bottom plates, and side walls.
LS3	Use after repair	The structural components are severely damaged, cracks appear on the components that penetrate the protective story, but the structure has not collapsed.
LS4	Cannot be used	Structural components are severely damaged, strength is degraded, and a fracture zone is formed, which may lead to collapse.

Based on the four performance levels mentioned above, this study defines the damage states of the metro station complex structure under seismic loading using the maximum interstory drift ratio $\theta_{\max}$ as the structural demand parameter. The corresponding damage levels, performance states, and their respective $\theta_{\max}$ ranges are shown in Table 4.

Table 4. Definition of structural damage state.

Damage Level	Basically Intact	Minor Damage	Moderate Damage	Severe Damage	Collapse
Performance Status	Normal use	Temporary use	Repair use	Safety assurance	Loss of function
Range of Evaluation Indicators (%)	$\theta_{\max} \leq 0.14$	$0.14 < \theta_{\max} \leq 0.32$	$0.32 < \theta_{\max} \leq 0.66$	$0.66 < \theta_{\max} \leq 1.12$	$1.12 < \theta_{\max}$

3.2. Two-Dimensional Finite Element Model

Considering that the complex structure is a double symmetrical structure, the soil layer in the site is evenly distributed, and the seismic wave is only input in one direction along the horizontal X direction. At the same time, in order to save computing resources, the transverse seismic problem of the structure is simplified into a two-dimensional plane strain problem during the elastic–plastic time history analysis, and the column of the tower is selected as the cross section. Based on this, this paper establishes a two-dimensional soil–structure integral finite element model for IDA analysis, as shown in Figure 7.

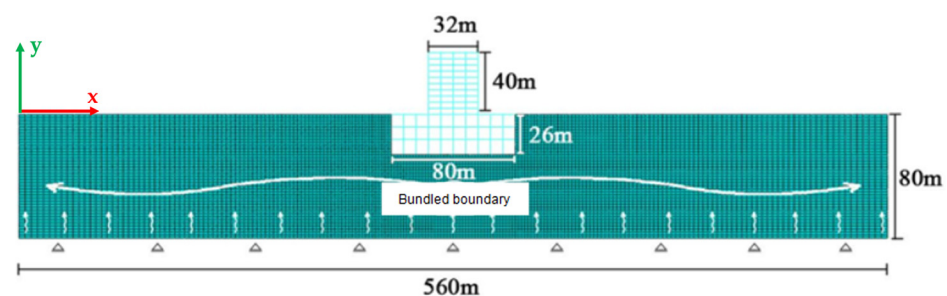


Figure 7. Soil–structure finite element model.

The lateral calculation width of the soil–structure interaction finite element model is set to 560 m, with a vertical height extending to 80 m down to the bedrock. Soil elements are modeled using four-node bilinear plane strain elements (CPE4R), and the soil around the structural system is refined with a denser mesh. The components of the overhead frame structure and the metro station complex structure are modeled using beam elements, with the internal reinforcement defined using the way of rebar layer. The contact interface between the soil and the structure is modeled as surface-to-surface contact, with the outer surface of the structure set as the primary surface and the soil surface as the slave surface. The contact properties are defined with “hard” contact for normal behavior and frictional contact for tangential behavior, using the “penalty” method with a friction coefficient of 0.4. The model’s side boundaries are constrained using a bundled boundary so that the same height on both sides of the boundary of the soil can only do horizontal shear motion under the action of ground motion, and there is no vertical relative motion, different from the fixed constraint, while the bottom boundary is set to allow horizontal freedom but restrict vertical movement. First, gravity loading is applied for gravity analysis, followed by inputting horizontal seismic waves at the model’s base for time history analysis.

3.3. Selection of Seismic Wave

Ground motion is the result of energy release within the Earth’s interior, and its wave characteristics play a critical role in earthquake engineering. Seismic waves with different frequencies and different peak accelerations (PGA) will affect the seismic response of underground integrated subway station structures [28,29]. In the existing ground motion record data, there are two types of ground motions that are considered to be special, namely the near-fault pulse type and far-field long-period type. They show very different wave characteristics near the source and away from the source. To better explore the seismic performance of structures under different types of ground motion, this study utilizes the design ground motion acceleration response spectrum derived from the Seismic Design GB 50909-2014 Code for Urban Rail Transit Structures in China. Using this, appropriate ground motion records are selected from the Pacific Earthquake Engineering Research Center (PEER) database and defined as ordinary ground motions.

In this study, the fault distance is used as the key parameter to divide the near-fault pulse-type ground motion and the far-field long-period ground motion. The fault distance is usually defined in the range of 20–60 km. The ground motion with a fault distance less than 20 km is defined as a near-fault pulse-type ground motion, and the ground motion with a fault distance greater than 60 km is defined as a far-field long-period ground motion. This division is based on the propagation characteristics of ground motion at different fault distances. Near-fault pulse-like ground motion has high frequency and amplitude, while far-field long-period ground motion has rich low-frequency components and a small amplitude. The selection criteria of near-fault pulse ground motion and far-field long-period ground motion are based on Tables 5 and 6, respectively.

Table 5. Selection criteria of near-fault pulse ground motion.

Earthquake Magnitude	Rupture Distance/km	PGV/(cm·s ^{−1})	Pulsed Index	Fourier Frequency Domain/Hz
M _w ≥ 6.5	R ≤ 20	PGV ≥ 30	PI ¹ > 0.85	0~2

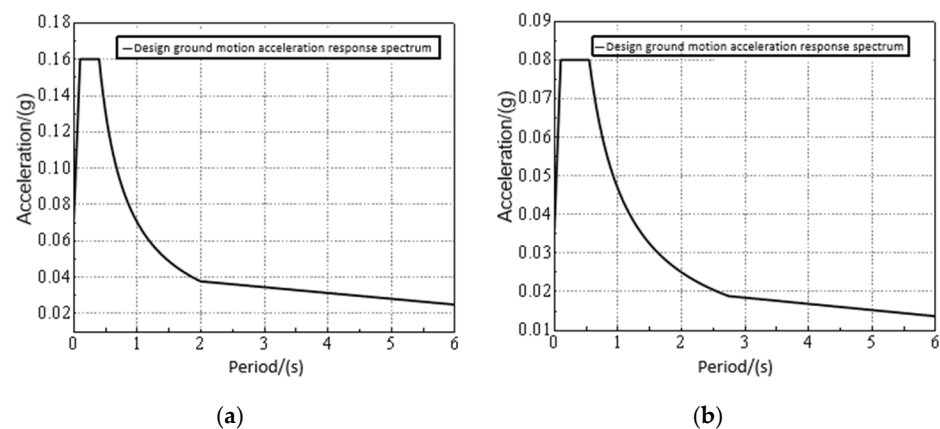
¹ Baker [30] proposed the index PI of peak velocity ratio and energy ratio to characterize the degree of pulse characteristics of pulse-type ground motion. When the ground motion pulse index PI > 0.85, it is defined as pulse-type ground motion.

Table 6. Selection criteria of far-field long-period ground motion.

Rupture Distance/km	PGA/g	PGV ¹ /PGA ²	Define Indicators	Fourier Frequency Domain/Hz
$R \geq 60$	$PGA \geq 0.02$	$PGV/PGA^3 < 0.2$	$\beta_l^4 > 0.4$	0~2

¹ PGV is used to represent peak ground velocity. ² PGA is used to represent peak ground acceleration. ³ Liao [31] thought that $PGV/PGA < 0.2$ indicates that the pulse is not obvious. ⁴ Li [32] proposed the square weighted average of the period β_l as the defining parameter of the long-period ground motion, and the ground motion with $\beta_l > 0.4$ is a long-period ground motion.

For the selection of ordinary ground motion, GB 50909-2014 Code for seismic design of urban rail transit structures in China points out that the difference between the acceleration response spectrum curve of the selected ground motion and the design acceleration response spectrum curve at the three main characteristic period points should be less than 20%. The three main characteristic period values of the model in this paper are obtained by calculating the “frequency” under the “linear perturbation” program in ABAQUS software. According to the code, the seismic fortification intensity of the study site in Tongzhou District, Beijing, where the selected structural system is located, is Intensity 8. The site is classified as a Class II site, with a design peak ground acceleration (PGA) of 0.2 g and a characteristic period of 0.4 s. In subsequent comparative analyses of the vulnerability differences of the structural system under different site categories, a project site in Suzhou is selected as the research object. This site is classified as a Class III site with a seismic fortification intensity of 7, a design PGA of 0.2 g, and a characteristic period of 0.35 s. The design acceleration response spectrum curve is shown in Figure 8.

**Figure 8.** Design ground motion acceleration response spectrum; (a) Class II site; (b) Class III site.

Vamvatsikos et al. [33] suggested that when performing IDA analysis, it is necessary to select 10 to 20 ground motions to eliminate the influence of ground motion variability on seismic vulnerability results. In this study, based on key indicators such as site classification, epicentral distance, Fourier frequency domain, earthquake magnitude, and pulse index, 15 records of each type of ground motion—near-field pulse-type, far-field long-period, and ordinary ground motions—were selected from the PEER ground motion database for analysis. Figures 9 and 10 present the acceleration response spectrum of the selected ground motion records. The selected recorded ground motion records are used as the original ground motion data, and the selected ground motion data are subjected to amplitude scaling. Palanci et al. [34] used different scaling methods and scaling limits to study the influence of amplitude scaling on important earthquake demand parameters (EDPs). The evaluation shows that if the spectral shape compatibility is ensured, then similar average responses can be obtained regardless of scaling limits, methods, and construction topologies.

In this paper, the equal step amplitude modulation method is used to adjust the amplitude of ground motion in incremental dynamic analysis [35]. This method is to scale the ground motion amplitude according to a certain step size, so as to obtain a series of ground motions with different intensities. The amplitude of ground motion can be adjusted according to Equation (2):

$$A(t) = \frac{A_{\max}}{a_{\max}} a(t) \quad (2)$$

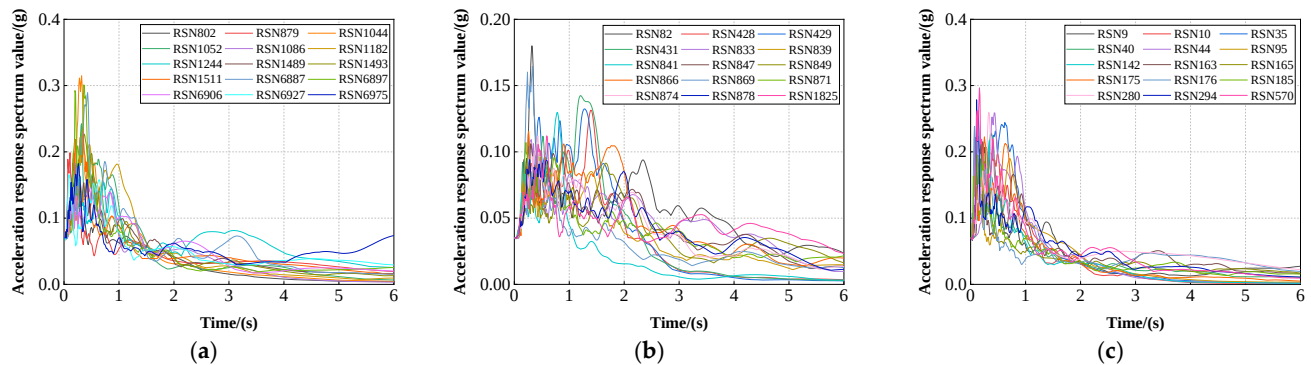


Figure 9. Class II site ground motion acceleration response spectrum (damping = 5%); (a) near-field type; (b) far-field type; (c) ordinary type.

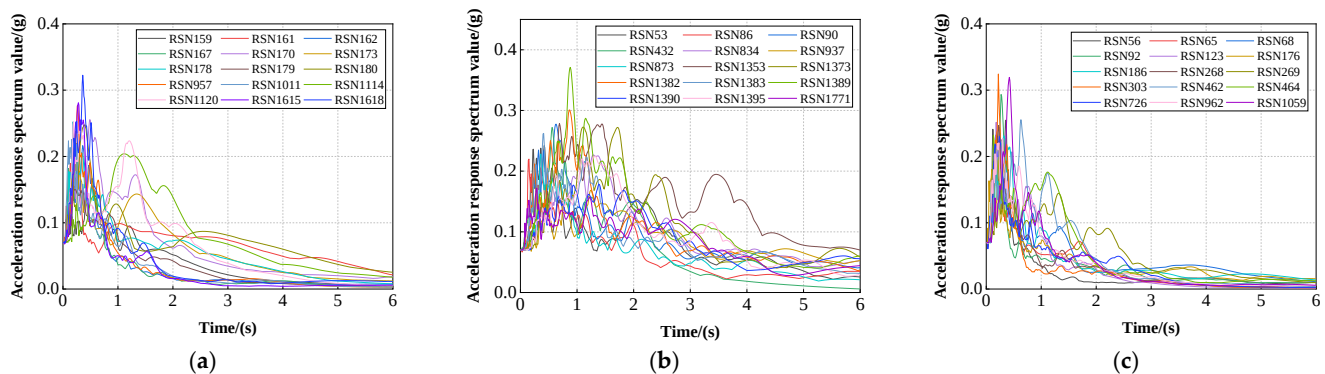


Figure 10. Class III site ground motion acceleration response spectrum (damping = 5%); (a) near-field type; (b) far-field type; (c) ordinary type.

In the formula:

$a(t)$, $A(t)$ —The time history of ground motion acceleration before and after amplitude modulation.

$a_{\max}$, $A_{\max}$ —The peak acceleration of ground motion before and after amplitude modulation.

According to FEMA [36], during incremental dynamic analysis, each ground motion can be scaled to seven intensity levels. To better compare the influence of different site conditions on the seismic performance of structures, this study scales the PGA of ground motions for Class II and Class III sites to 0.1 g, 0.2 g, 0.4 g, 0.6 g, 0.8 g, 1.0 g, and 1.2 g sequentially. It is set to input along the X direction of the bedrock, and it is assumed that the earthquake only propagates along the vertical direction of the bedrock, and there is no oblique propagation.

4. Seismic Vulnerability Analysis

4.1. Probabilistic Demand Analysis

Structural vulnerability refers to the conditional probability of exceeding a certain degree of damage to a building under different disaster intensities [37]. In the 1970s, vulnerability was first used as a ground motion function to evaluate the seismic performance

of structures. In recent years, with the progress of finite element analysis technology, vulnerability analysis has become an effective means to evaluate the disaster prevention performance of structures [38]. The structural vulnerability analysis based on IDA method is to input a set of ground motion records with gradually increasing strength to the structure until the collapse of the structure occurs, and the total number of ground motions is defined as N_{total} . If there is a $N_{destruction}$ strip ground motion under a certain ground motion intensity IM (Intensity Measure) that causes structural damage, then the damage rate of the structure under IM is recorded as $N_{destruction}/N_{total}$. If the number of ground motion records is large enough and representative enough, then the seismic capacity of the structure can be quantitatively evaluated by analyzing the failure rate of the structure, and then the corresponding suggestions for seismic design can be put forward. Scholars at home and abroad have carried out a lot of successful research work in this field [39–43].

In this study, the maximum interstory drift ratio (θ_{max}) is chosen as the structural damage measure (DM), and the peak ground acceleration (PGA) is selected as the intensity measure (IM) of the ground motion. The calculated results are organized into a set of IDA curves, and their mean and standard deviation are used to generate the 50%, 16%, and 84% quantile curves, representing the central tendency and variability of the data, as shown in Figure 11.

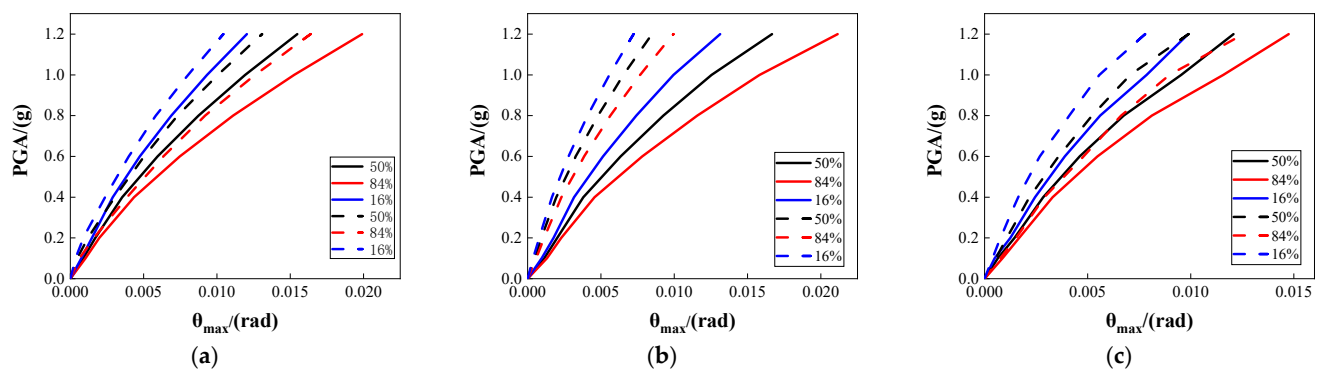


Figure 11. IDA quantile curve (dotted line—Class II site; solid line—Class III site); (a) near-field type; (b) far-field type; (c) ordinary type.

Due to the large discreteness of the maximum interstory displacement ratio response of the station structure obtained by IDA analysis, it is necessary to fit it. Cornell et al. [44] pointed out that the relationship between the structural damage index DM and the ground motion intensity parameter IM approximately satisfies the power function relationship (3):

$$DM = a \cdot IM^b \quad (3)$$

By substituting the selected ground motion intensity index (PGA) and structural damage index (θ_{max}) into Equation (3), Equation (4) is obtained as follows:

$$\theta_{max} = a(PGA)^b \quad (4)$$

Or the logarithm of both sides can be expressed as Equation (5):

$$\ln(\theta_{max}) = a \ln(PGA) + b \quad (5)$$

In the formula:

a, b —linear regression coefficients, which are constants.

Figures 12 and 13 illustrate the relationship between DM and IM, along with the linear regression equations, for the metro station complex structure under the TOD model in Class II and III site conditions.

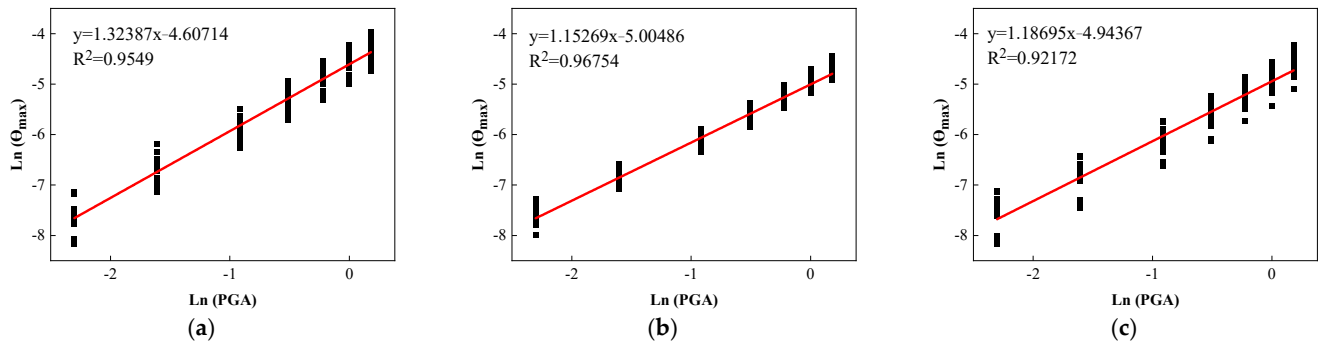


Figure 12. DM–IM linear fitting curve of type II site; (a) near-field type; (b) far-field type; (c) ordinary type.

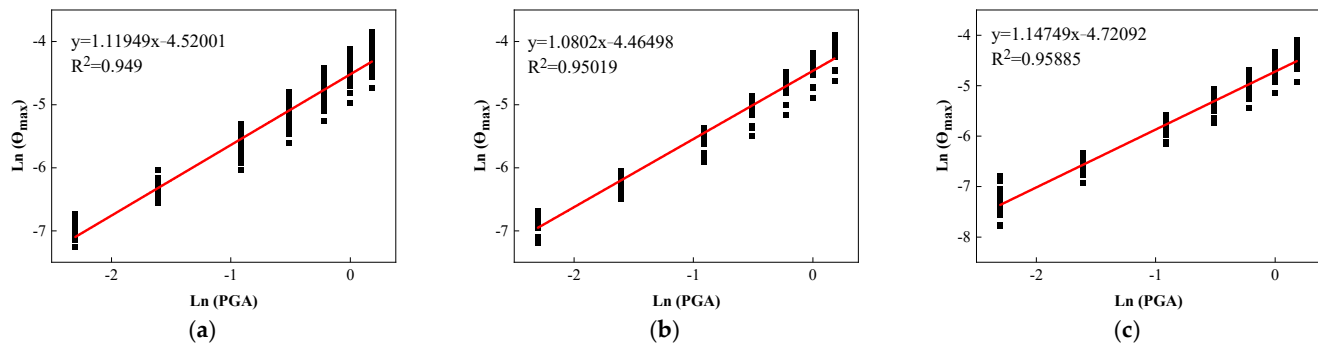


Figure 13. DM–IM linear fitting curve of type III site; (a) near-field type; (b) far-field type; (c) ordinary type.

The univariate linear regression equation for Class II sites is:

$$\ln(\theta_{max}) = 1.32387\ln(PGA) - 4.60714$$

$$\ln(\theta_{max}) = 1.15269\ln(PGA) - 5.00486$$

$$\ln(\theta_{max}) = 1.18695\ln(PGA) - 4.94367$$

The linear regression equation for Class III sites is:

$$\ln(\theta_{max}) = 1.11949\ln(PGA) - 4.52001$$

$$\ln(\theta_{max}) = 1.0802\ln(PGA) - 4.46498$$

$$\ln(\theta_{max}) = 1.14749\ln(PGA) - 4.72092$$

4.2. Plotting of Seismic Vulnerability Curves

The structural vulnerability curve describes the relationship between the performance parameters of the structure and the seismic intensity. Argyroudis et al. [45] and Zhong et al. [13] pointed out that the damage probability calculation formula for four different performance levels of the structural system is shown in the following Equation (6):

$$P(LS) = \Phi \left[\frac{a + b\ln(IM) - \ln(D_c)}{\sqrt{\beta_{DM|IM}^2 + \beta_C^2}} \right] \quad (6)$$

In the formula:

$P(LS)$ —Probability of structural limit state failure;

a, b, θ_{max} and PGA linear regression coefficient;

IM —Seismic intensity index;

D_c —Interstory drift ratio limits for five failure states;

Φ —Standard normal distribution function;

β_c —The logarithmic standard deviation of the disaster prevention capability;

$\beta_{DM|IM}$ —The standard deviation of the logarithmic normal distribution of the structural disaster prevention performance requirements.

When using PGA as the seismic intensity index, according to the HAZUS99 user manual recommendations [46], $\sqrt{\beta_{DM|IM}^2 + \beta_c^2} = 0.5$.

Using Equation (6), the vulnerability probabilities of the structure were calculated, and the vulnerability curves for the metro station complex structure under Class II and III site conditions were plotted, as shown in Figure 14.

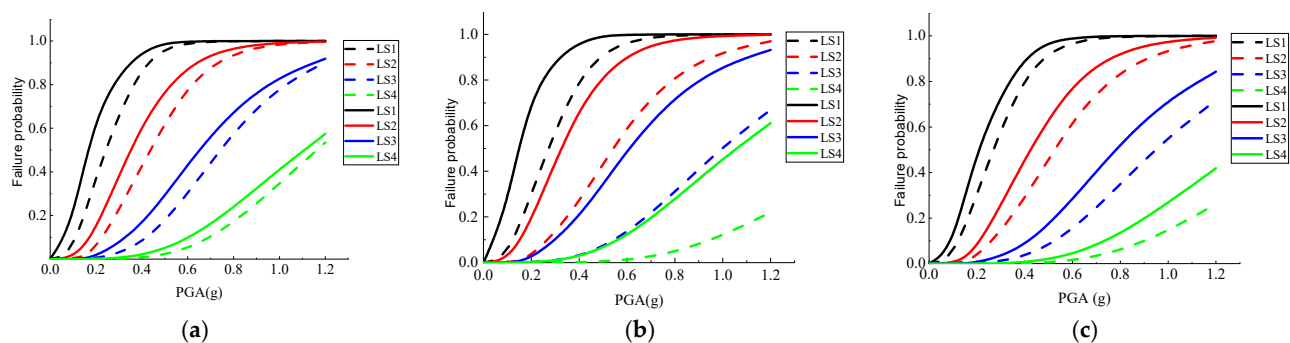


Figure 14. Structural vulnerability curve (dotted line—Class II site; solid line—Class III site); (a) near-field type; (b) far-field type; (c) ordinary type.

As shown in Figure 14, the vulnerability curves of the underground complex structure exhibit a similar trend under different site types. The excess probabilities for the four performance levels increase with increasing seismic intensity. As the site class improves (with an increase in equivalent shear wave velocity), the vulnerability curves become flatter, indicating that the structure is less likely to be damaged in Class II site conditions. In Class II site conditions, the structure is more significantly affected by near-field pulse-type ground motions, resulting in the highest exceedance probabilities across the four performance levels. Comparing the influence degree of Class II and Class III sites under different types of seismic waves, the influence of far-field long-period seismic waves on Class III sites is much greater than that of Class II sites. This is because far-field long-period ground motions are primarily dominated by mid-to-low frequencies. Weak soil layers tend to filter out high-frequency components while amplifying low-frequency components, resulting in a several-fold amplification of the ground motion amplitude, which leads to more severe structural damage.

4.3. Probabilistic Assessment of Structural Damage States

The structural damage state probability $P(DS)$ represents the probability of the structure being in different damage states under varying seismic intensities. Its calculation formula is expressed as shown in Equation (7).

$$P(DS_i) = \begin{cases} 1 - P(LS_i), i = 1 \\ P(LS_{i-1}) - P(LS_i), i = 2, 3, \dots, N \\ P(LS_N), i = N + 1 \end{cases} \quad (7)$$

In the formula:

N —The number of performance levels divided into structural system;

$N + 1$ —The number of damage states defined for the structural system.

Through Equation (7), the function curve corresponding to the transcendental probability of the five failure states of the subway station complex structure in TOD mode is obtained by calculating the function curve in Figure 14, and the change rule is shown in Figure 15. That is, the exceedance probability of the five failure states is calculated by the difference between every two adjacent functions in the six function curves $P = 1$, $P(LS_1)$, $P(LS_2)$, $P(LS_3)$, $P(LS_4)$, $P(LS_5)$, and $P = 0$. For example, the probability of the failure state of “basically intact” is the difference between the function “ $P = 1$ ” of the probability of 100% occurrence and the failure probability “ $P(LS_1)$ ” corresponding to the performance standard of “normal use”, and so on.

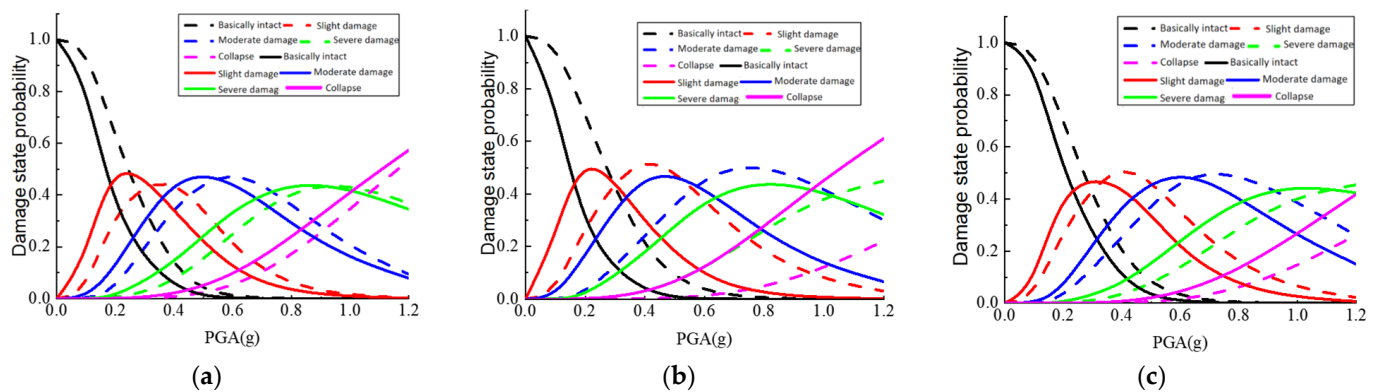


Figure 15. The probability diagram of structural failure state (dotted line—Class II site; solid line—Class III site); (a) near-field type; (b) far-field type; (c) ordinary type.

As shown in Figure 13, under Class II and III site conditions, the variation trend of the “essentially intact” state of the metro station complex structure decreases monotonically under the influence of the three types of ground motions. The “slight damage” and “moderate damage” states exhibit a trend of initially increasing and then decreasing, while the “collapse” state shows a monotonically increasing trend. Under near-field pulse-type ground motions, the “severe damage” state first increases and then decreases, whereas under far-field long-period ground motions and ordinary ground motions, the “severe damage” state increases monotonically.

Under Class II site conditions, the impact of near-field pulse-type seismic waves on the structure is more significant. As the PGA increases, the probability of severe damage and collapse of the structure significantly increases. In Class III sites, under the same PGA action, the probability of structural system failure increases. The damage probability of each damage level increases significantly under the action of far-field long-period seismic, indicating that the subway station complex structure under a weak site is more susceptible to the influence of far-field long-period seismic.

Tables 7–9 give the probability of the damage state of the subway station complex structure under different defense standards under three different types of seismic waves. The data indicate significant differences in the damage state probabilities of the metro station complex structure under different types of ground motions.

Table 7. Probability table of structural failure state under near-field pulse-type earthquake.

Site Classification	Fortification Standards	Damage State Probability				
		Basically Intact	Slight Damage	Moderate Damage	Severe Damage	Collapse
Class II site	Frequent seismic	98.26% ¹	<u>1.74%</u> ²	0	0	0
	Earthquake resistance	60.84%	<u>36.95%</u>	2.19%	0.02%	0
	Rare earthquake	5.94%	51.05%	<u>37.66%</u>	5.12%	0.23%
	Extremely rare earthquake	0.42%	18.05%	51.98%	<u>25.52%</u>	4.03%
Class III site	Frequent seismic	84.01%	<u>15.67%</u>	0.32%	0	0
	Earthquake resistance	28.88%	59.21%	<u>11.47%</u>	0.44%	0.01%
	Rare earthquake	1.75%	33.72%	50.13%	<u>13.25%</u>	1.15%
	Extremely rare earthquake	0.13%	9.89%	46.12%	<u>35.24%</u>	8.62%

¹ The probability sum is 100% under the same fortification standard, and the maximum probability data are displayed in bold font, and their corresponding failure state is defined as “main failure state”. ² The italic plus underlined font represents “secondary damage state”.

Table 8. Probability table of structural failure state under ordinary earthquake.

Site Classification	Fortification Standards	Damage State Probability				
		Basically Intact	Slight Damage	Moderate Damage	Severe Damage	Collapse
Class II site	Frequent seismic	98.43% ¹	<u>1.56%</u> ²	0.01%	0	0
	Earthquake resistance	69.41%	<u>29.35%</u>	1.23%	0.01%	0
	Rare earthquake	12.76%	59.76%	<u>25.39%</u>	2.04%	0.05%
	Extremely rare earthquake	1.78%	33.99%	50.01%	<u>13.09%</u>	1.13%
Class III site	Frequent seismic	93.65%	<u>6.30%</u>	0.05%	0	0
	Earthquake resistance	47.41%	47.86%	<u>4.64%</u>	0.09%	0
	Rare earthquake	4.89%	48.31%	40.31%	<u>6.16%</u> ³	0.33%
	Extremely rare earthquake	0.48%	19.28%	52.31%	<u>24.30%</u>	3.63%

¹ The probability sum is 100% under the same fortification standard, and the maximum probability data are displayed in bold font, and their corresponding failure state is defined as “main failure state”. ² The italic plus underlined font represents “secondary damage state”. ³ In order to better compare the probability of “secondary failure state”, the secondary failure state under the rare earthquake fortification standard in Class III sites is defined here as serious damage, with a probability of 6.16%.

It is pointed out that the peak acceleration of frequent ground motion should be determined by not less than 1/3 of the peak acceleration of basic ground motion in the GB 18306-2015 Ground Motion Parameter Zoning Map in China [47]. The peak acceleration of rare ground motion should be determined by 1.6~2.3 times the peak acceleration of basic ground motion. The peak acceleration of extremely rare ground motion should be determined by 2.7~3.2 times the peak acceleration of basic ground motion. In order to better compare the influence of different sites on the seismic performance of the structure, the basic seismic acceleration of the design of Class II site and Class III sites is 0.2 g, so the frequent, fortification, rare, and extremely rare earthquakes are 0.1 g, 0.2 g, 0.4 g, and 0.6 g.

Table 9. Far-field long-period seismic action structure failure state probability.

Site Classification	Fortification Standards	Damage State Probability				
		Basically Intact	Slight Damage	Moderate Damage	Severe Damage	Collapse
Class II site	Frequent seismic	98.29%	<u>1.70%</u>	0.01%	0	0
	Earthquake resistance	69.83%	<u>28.96%</u>	1.19%	0.02%	0
	Rare earthquake	14.04%	60.42%	<u>23.72%</u>	1.77%	0.05%
	Extremely rare earthquake	2.20%	36.88%	48.57%	<u>11.45%</u>	0.90%
Class III site	Frequent seismic	75.93%	<u>23.34%</u>	0.73%	0	0
	Earthquake resistance	21.37%	61.33%	<u>16.42%</u>	0.85%	0.03%
	Rare earthquake	1.10%	27.85%	52.11%	<u>17.11%</u>	1.83%
	Extremely rare earthquake	0.08%	7.55%	42.55%	<u>38.60%</u>	11.22%

By comparing the main failure states of fortification standards at all levels under the two types of sites, it is found that under frequent earthquakes (0.1 g), the station complex structure is basically in good condition. Under the fortification earthquake (0.2 g), the structural system under the Class II site is mainly in a basically intact state, while the structure under the Class III site is mainly in a slight damage state. Under the rare earthquake (0.4 g), the structural system under the Class II site is mainly slightly damaged, and the structural system under the Class III site is mainly in the medium damage state. Under the fortification standard of extremely rare earthquake (0.6 g), the two types of sites are mainly medium damage, but the structural system under the Class II site and the Class III site also has a large probability of minor damage and serious damage, respectively. The analysis results show that the structural system under the Class III site is always more likely to be in a higher failure state than the Class II site. This shows that damage decreases with the increase in the shear wave velocity of the site soil, that is, the harder the site soil is, the lower the degree of damage is.

Because the main failure states of similar sites under different fortification standards are basically the same, the higher failure state of the main failure state in the table is defined as the secondary failure state. Comparing the probability of secondary failure state of different types of underground ground motions in the same field, it is found that the near-field pulse-type ground motion makes the probability of secondary failure state of type II site much larger than that of the other two types of ground motions. The far-field long-period ground motion makes the probability of the secondary failure state of the Class III site much larger than the other two types of ground motions. This indicates that the site type and ground motion type significantly affect the seismic response of the structure.

5. Discussion and Conclusions

The innovation of this paper lies in the use of the IDA method to conduct a seismic vulnerability analysis of the subway station complex structure under the TOD (transit-oriented development) model, considering different site conditions and types of seismic waves. The findings can provide valuable references for assessing the seismic performance of similar structural systems. However, the subway station complex structure exhibits significant spatial effects, and the seismic vulnerability analysis in this paper is based on a two-dimensional numerical model, which has certain limitations. Further work is needed to develop an appropriate three-dimensional numerical model for a more in-depth vulnerability analysis. According to the above analysis, the main conclusions are as follows:

- (1) Based on the actual project, a three-dimensional numerical model of the subway station complex structure in TOD mode is established, and the pushover loading of the underground structure is carried out. Through the analysis of the interstory drift ratio–middle column shear pushover curve, the maximum interstory drift ratio limits corresponding to the five damage states of the subway station complex structure are defined as 0.14%, 0.32%, 0.66%, and 1.12%, respectively.
- (2) For the simplified two-dimensional numerical model, using different types of ground motion as inputs, the mean and discreteness of different IDA curve clusters are quite different. The near-field pulse-type ground motion has a greater impact on the ground motion of the structural system under the Class II site, and the far-field long-period ground motion has a greater impact on the structure under the Class III site. Site type and ground motion type significantly affect the seismic response of the structure.
- (3) Site classification has a significant impact on the seismic response behavior of the subway station complex structure. With the increase in the equivalent shear wave velocity of the site, the soil of the site is harder, and the seismic vulnerability curves of the subway station complex structure under the TOD model become increasingly flatter. This indicates that the structural system is less susceptible to damage under seismic waves, with a relatively lower risk of damage.

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