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A Coupled 3D Nonlinear Modeling Approach for Evaluating SSI Effects in Large Stepback Buildings on Stepped Slope Site

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ABSTRACT: Large and complex structures on stepped slope sites are still commonly evaluated using simplified fixed-base assumptions, which may not adequately reflect the effects of soil-structure interaction (SSI). To address this issue, this study develops a coupled three-dimensional nonlinear finite element model in which the surrounding soil domain, foundation system, and superstructure are considered simultaneously, so as to capture soil-structure interaction effects under stepped topographic conditions. A corresponding fixed-base model is further established for comparison, and the approach is applied to the Ground Transportation Center (GTC) at Kunming Airport, a large transportation hub founded on a multi-bench stepped slope. The seismic responses predicted by the two models are compared in terms of slab damage, story drift ratio, story shear force, overturning moment, and energy dissipation characteristics. The results show that the fixed-base model can generally capture the overall response trend, but noticeable differences remain in local response characteristics. Compared with the SSI model, the fixed-base model predicts a wider extent of tensile damage, larger story drift ratios at most middle and upper floors, and larger overturning moments. In contrast, the SSI model shows more unfavorable local responses at lower floors near the soil-contact region, including larger story drift and local story shear force. In addition, SSI changes the energy dissipation pattern by allowing a considerable portion of the input energy to be transmitted into and dissipated through the surrounding soil, thereby reducing the energy demand on the structure and dampers. These results indicate that the effect of SSI should be explicitly considered in the seismic analysis of large irregular structures on stepped slope sites, as relying solely on conventional fixed-base assumptions may lead to inaccurate structural assessments and potential vulnerability at embedded levels.

KEYWORDS: Soil-structure interaction; stepped slope site; seismic response; fixed-base; large-scale structure

1 Introduction

Driven by rapid urbanization and the scarcity of flat land, modern infrastructure projects are increasingly expanding onto slope sites and other topographically irregular regions. Consequently, a growing number of large-scale facilities, such as airports and high-speed railway stations, are being constructed on stepped sites [1,2]. Owing to their large mass and complex configurations, such as long spans and split levels, these structures are particularly vulnerable to earthquake-induced torsional effects and local stress concentrations caused by structural asymmetry, abrupt elevation changes, and differences in the participating mass of surrounding soil [3]. When such structures are founded on slope sites, the spatially varying amplification and nonuniform seismic input induced by topography may further modify their

seismic response. Therefore, the effects of slope-site should be fully considered in the seismic response analysis of such structures.

Previous studies have demonstrated that slope sites can significantly amplify seismic motions, and this amplification exhibits strong spatial variability governed by topographic position, geological conditions, incident-wave characteristics [4–7], slope geometry, and soil properties [8,9]. This understanding has been partly reflected in current seismic design provisions. For example, Eurocode 8 [10] recommends a topographic factor of 1.4 for slopes steeper than 30° and higher than 30 m, while the Chinese code JGJ/T 472-2020 [11] suggests amplification factors ranging from 1.1 to 1.6 according to slope inclination and height. However, these simplified approaches are still largely based on the fixed-base assumption and do not adequately account for soil-structure interaction (SSI). In practice, SSI can significantly modify the dynamic characteristics and seismic response of structures. Shabani and Ghanbari [12] reported that directly applying slope-amplified ground motions to a fixed-base model may overestimate structural responses such as base shear and inter-storey drift ratio. Moreover, even for structures on level ground, no consensus has yet been reached on whether SSI plays a beneficial or detrimental role in seismic response, because it may reduce seismic demand through period lengthening and additional damping, while it may also amplify response due to foundation softening, rocking, and deformation compatibility effects [13–17]. This issue becomes more critical for structures extending over a long distance along a slope, because different parts of the structure may be subjected to different levels of topographic amplification.

Accordingly, increasing attention has been paid to the seismic response of buildings on slope sites considering SSI. For example, Song et al. [18] employed a finite element model to analyze the seismic response of a six-storey reinforced concrete frame structure on a slope, and found that slope effects mainly alter the deformation characteristics of the structure, causing its lateral displacement to develop clearly toward the slope, with this tendency becoming more pronounced as the slope ratio increases. Bahuguna and Firoj [19] investigated the differences in seismic response among six-storey reinforced concrete buildings located at the top, middle, and bottom of a slope, and reported notable differences in lateral displacement, floor acceleration, and interstorey drift ratio. Kourkoulis et al. [20] investigated the seismic response of frame structures located near the slope crest under different foundation types, and showed that a properly designed raft foundation can effectively enhance the seismic performance of the structure under the combined effects of slope instability and earthquake shaking. Fatahi et al. [21] developed a three-dimensional time-domain finite element model for buildings near shallow slopes and showed that decreasing the distance between the foundation and the slope crest can intensify foundation rocking, lateral deformation, and interstorey drift due to slope-foundation-structure interaction. More recently, Islam et al. [22] investigated steel MRF buildings on slopes considering earthquake frequency content and demonstrated that the structural acceleration, lateral displacement, base shear, and foundation rocking are strongly affected when the dominant frequency components of the input motion approach the site, topographic, or structural frequencies. In general, existing studies have demonstrated that the participation of the surrounding soil mass and the corresponding SSI effects profoundly influence the seismic behavior of buildings on slope sites. However, the research has mainly focused on ordinary frame structures with relatively regular configurations, and whether these findings can be directly extended to large-scale structures spanning slopes remains to be further verified.

Compared with ordinary frame buildings, large-scale structures usually possess larger dimensions, more complex spatial configurations, and more pronounced stiffness irregularities. In recent years, increasing attention has been paid to the seismic response of large and complex structures. Zhang and Chen [23] reported that a high-speed railway hub station exhibits a distinct dynamic feature of being relatively flexible in the upper part and stiff in the lower part when pile-soil interaction is considered. Sun et al. [24] showed, through a shaking table test on a large-span high-rise composite building in a mountainous area, that terrain-induced time-delay and amplification effects of ground motions can significantly alter seismic input and structural response. Guo et al. [25] found that friction-damped self-centering tension braces can significantly reduce story drift, residual story drift, and roof displacement of a large-span terminal building, while also mitigating torsional effects. Although the above studies have revealed several characteristics of the seismic response of large and complex structures, targeted comparative studies are still lacking for large-scale buildings located on slope sites, especially transportation hub structures extending along a slope, when both soil-structure interaction and structural nonlinearity are considered. In particular, the differences between such results and those obtained from fixed-base analysis using amplified slope ground motions as input remain insufficiently clarified.

To address this critical issue, this study develops a comparative modeling framework for evaluating the seismic response of large stepback structures on stepped slope sites. A fully coupled three-dimensional nonlinear finite element model considering the surrounding soil domain is established to capture soil-structure interaction (SSI) effects, together with a corresponding fixed-base model for comparison. The framework is then applied to the Ground Transportation Center (GTC) at Kunming Airport, a large-scale transportation hub situated on a stepped slope. Under structural nonlinear conditions, the slab damage distribution, story drift ratio, story shear force, overturning moment, and energy dissipation characteristics predicted by the two models are comparatively analyzed to clarify the influence of SSI on the dynamic characteristics and key response features of large-scale buildings spanning stepped slopes, and to evaluate the applicability of the fixed-base simplified method using amplified slope ground motions as input.

2 Representative Prototype and Numerical Modeling Procedure

2.1 Representative Prototype: GTC at Kunming Airport

The GTC at Kunming International Airport is selected as the prototype structure in this study. As shown in Fig. 1a,b, the GTC is built on a stepped fill slope and serves as a major interchange hub connecting the airport with high-speed rail, metro, and surface transportation systems. The high-speed railway passes through the terminal district, with the airport station located beneath the GTC, while the metro lines pass underneath and intersect the railway obliquely, forming a complex three-dimensional structural system integrating above-ground transportation hub buildings, underground rail transit facilities, and multi-level transfer spaces. Moreover, the functional floors are arranged in accordance with the site elevation, resulting in distinct stepped and backstep characteristics, local floor discontinuity, and nonuniform support conditions, as illustrated by the sections along L1-L1 and L2-L2 in Fig. 1c,d, respectively.

From a modeling perspective, the GTC can be abstracted as a representative prototype consisting of three coupled subsystems from bottom to top, namely the stepped-slope ground domain, the foundation-retaining system, and the superstructure. The ground domain includes the multi-bench slope geometry and nonuniform soil strata, while the foundation-retaining system consists of piles, pile caps, and retaining walls embedded in the slope. The superstructure combines lower reinforced concrete components, middle transition floors, and an upper steel frame system, together with large floor openings and local discontinuities that produce evident mass and stiffness irregularities. Consequently, these features make the GTC a representative prototype, rather than a site-specific case, for developing a comparative modeling strategy

involving coupled SSI and fixed-base approaches for large-scale structures on stepped slope sites. This section presents the finite element model of the GTC and the stepped slope, together with the seismic input motions used in the numerical analysis.

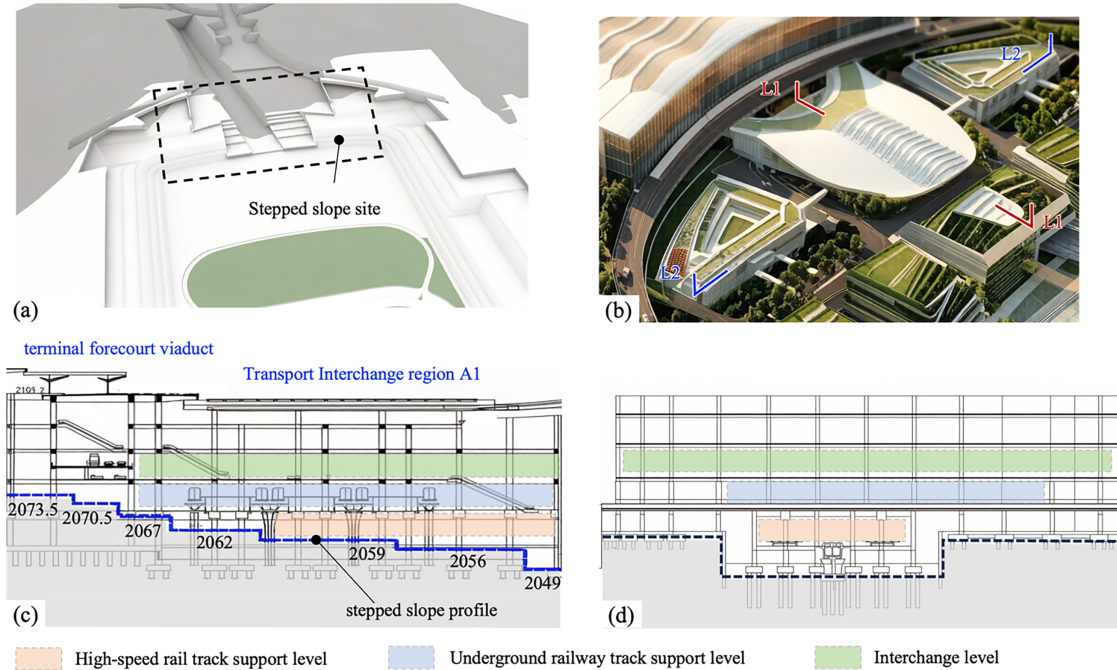


Figure 1: Engineering overview and structural configuration of the Ground Transportation Center on the stepped fill slope: (a) Site location; (b) Building view of the Ground Transportation Center; (c) Section along L1-L1; and (d) Section along L2-L2.

2.2 FE Model of GTC on Stepped Slope

As shown in Fig. 2, a three-dimensional finite element model comprising the superstructure, foundation system, and multi-bench stepped slope was established for seismic response analysis of the Ground Transportation Center. The overall model has plan dimensions of approximately $600 \text{ m} \times 550 \text{ m}$. In particular, the 600 m extent in the downslope direction (i.e., the Y direction shown in Fig. 2) was selected with consideration of the seismic input direction, so as to cover the principal range of slope effects under along-slope seismic excitation. The lowest elevation of the slope site is approximately 35.5 m , and the slope has a vertical height difference of about 24.5 m . It is divided into six benches, with retaining walls arranged at each bench to represent the retaining system adopted in the actual project. The above-ground GTC and the multi-bench stepped slope were modeled together to reflect the global mechanical behavior and interaction of the soil-structure system under topographic conditions. The main structure is approximately 264 m in length and 156 m in width, with a total height of about 55.5 m . To clearly illustrate the model configuration, Fig. 2b also presents three-dimensional views of the main structure at different floor levels.

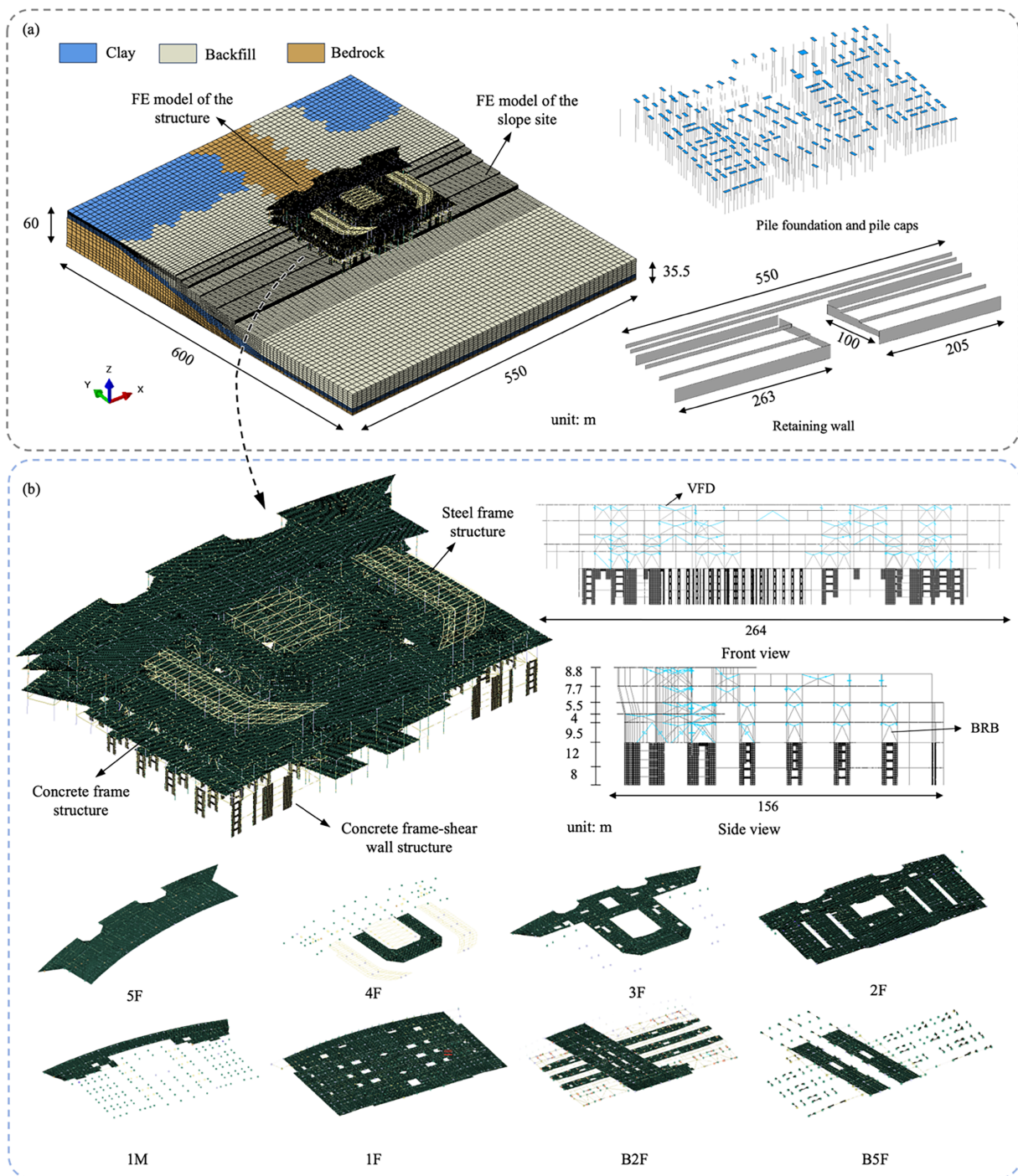


Figure 2: 3D finite element (FE) model of the Ground Transportation Center on the stepped slope: (a) FE model of the stepped slope and foundation system; and (b) FE model of the superstructure and views at different floor levels.

2.2.1 Modeling of the Stepped Slope

The three-dimensional stepped-slope site model was established based on the engineering site conditions, in which the soil layers were modeled as nonuniform strata varying with the terrain (Fig. 3). The slope site mainly consists of backfill, clay, and bedrock, and their basic mechanical parameters are listed in Table 1.

The nonlinear behavior of soil under seismic loading was approximately considered using the equivalent linear method. In this method, the strain-dependent shear modulus and damping ratio of soil are represented by strain-compatible equivalent parameters. The dynamic soil curves adopted in this study were obtained from the seismic safety evaluation report of the project site, and the corresponding modulus reduction and damping ratio curves are shown in Fig. 4. For the bedrock, no shear modulus degradation was assumed within the considered strain range, namely $G/G_{\max} = 1.0$, and a constant damping ratio of $\xi = 0.05$ was used. To determine representative dynamic soil parameters, the soil profile at the center of the structural area was selected for the equivalent linearization analysis. The equivalent linearization analysis was performed using SHAKE91 as a preprocessing tool, through which the strain-compatible equivalent shear modulus and damping ratio of each soil layer were obtained. These equivalent parameters were then introduced into the Abaqus finite element model. During the time-domain seismic response analysis, the soil was treated as an equivalent linear elastic material, and the soil properties were not iteratively updated with the development of strain. The soil damping was represented by Rayleigh damping, and the corresponding Rayleigh damping coefficients were calibrated based on the first and second frequencies of the free-field site model.

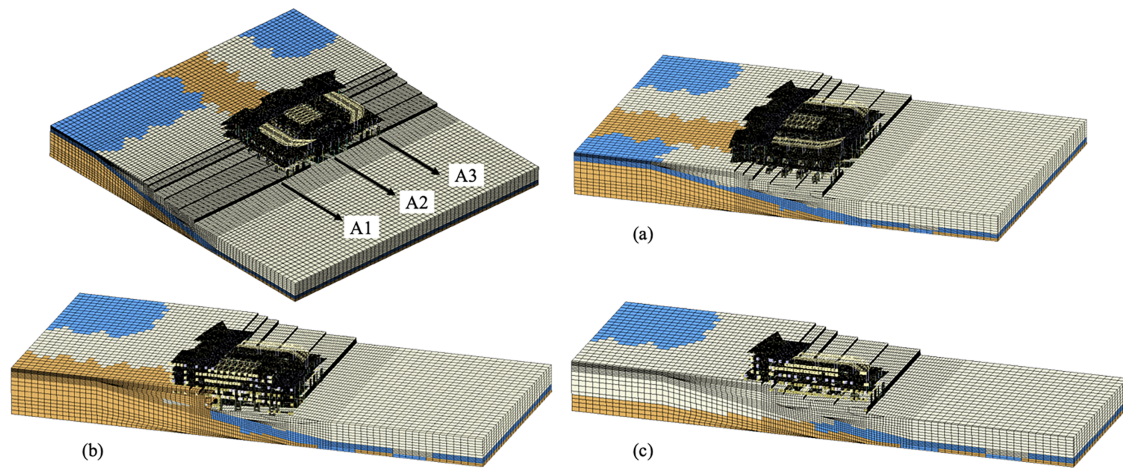


Figure 3: Nonuniform soil distribution of the stepped-slope site in different sections: (a) Section A1; (b) Section A2; and (c) Section A3.

Table 1: The material parameters of the soil.

Material	Density (kg/m^3)	Elastic Modulus (MPa)	Poisson's Ratio
Backfill	2200	370	0.3
Clay	1850	253	0.4
Bedrock	2500	2775	0.2

In the finite element model, all soil layers were simulated using C3D8 solid elements. To ensure effective wave propagation in the ground while maintaining computational efficiency, the mesh size was set to 0.5 m in the vicinity of the structure and 2–5 m in regions away from the structure.

The foundation consists of piles, pile caps, and retaining walls arranged along the slope. The piles transfer the superstructure loads to the deeper soil, while the pile caps connect the piles to the superstructure. The retaining walls are arranged along each bench of the stepped slope to maintain slope stability and restrain the deformation of the backfill. In the finite element model, the piles were simulated using two-node

three-dimensional beam elements (B31). The pile caps and retaining walls were simulated using four-node reduced-integration shell elements (S4R). The pile caps have a thickness of 2.5 m, and the piles are circular members with a diameter of 1.2 m. Both were modeled as linear elastic concrete. The geometric and material parameters of the foundation and retaining-wall components are summarized in Table 2. As shown in Fig. 5, the underground portions of the retaining walls were connected to the surrounding soil using the embedded region constraint, whereas the above-ground portions were tied with the slope. The piles and pile caps were also connected to the soil through the embedded constraint.

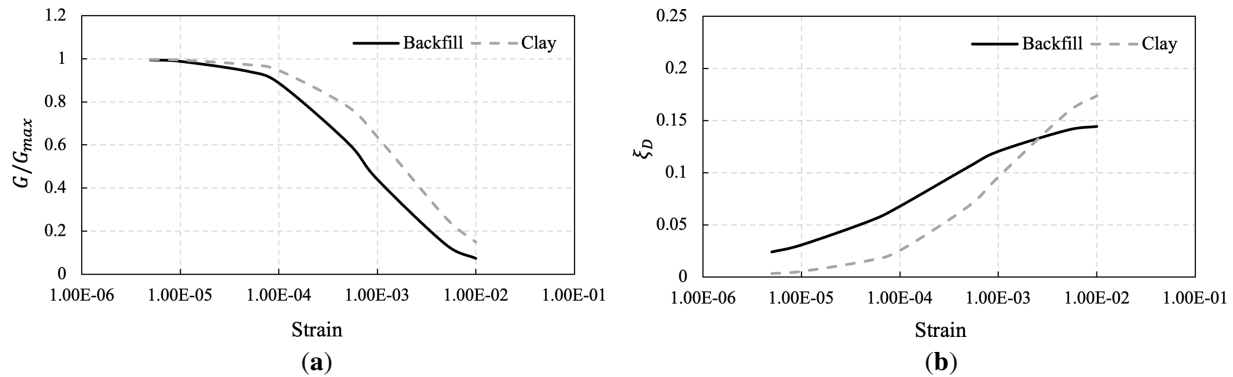


Figure 4: Dynamic soil curves adopted in the equivalent linear analysis: (a) normalized shear modulus reduction curves; and (b) damping ratio curves.

Table 2: Geometric and material parameters of the foundation and retaining-wall components.

Component	Element Type	Diameter (m)	Thickness (m)	Length (m)	Concrete Type
Pile	B31	1.2	\	30	C40
Pile cap	S4R	\	2.5	\	C30
Retaining wall S1	S4R	\	0.5	5.5	C35
Retaining wall S2			0.5	5.5	
Retaining wall S3			2.5	15	
Retaining wall S4			0.5	5.5	
Retaining wall S5			0.5	5.5	
Retaining wall S6			3.0	21	

2.2.2 Modeling of the GTC Structure

The main structure of the GTC consists of a lower reinforced concrete frame-shear wall system (B5F and B2F), a middle reinforced concrete frame system (1F and 1M), and an upper steel frame system (2F, 3F, 4F, and 5F), while all floor slabs are made of reinforced concrete. In addition, buckling-restrained braces (BRBs) and viscous fluid dampers (VFDs) are arranged at key locations in the middle and upper parts of the structure, as shown in Fig. 2b. In the finite element model, columns, braces, and floor beams were simulated using B31 beam elements, whereas floor slabs and shear walls were modeled using S4R shell elements. For the B31 beam elements, the longitudinal reinforcement was explicitly modeled using the “*REBAR” keyword in Abaqus by specifying the reinforcement material, cross-sectional area, and position relative to the local beam-section axes. Details of the keyword definition and the corresponding parameter meanings

are provided in [Appendix A](#). The VFDs were simulated using the dashpot element in Abaqus by defining the nonlinear relationship between resisting force and relative velocity [26]. As shown in [Fig. 6a](#), the force-velocity relationship of the damper can be expressed as $F_{VFD} = CV^\alpha$, where C is the damping coefficient, V is the relative velocity between the two connected nodes, and α is the nonlinear velocity exponent.

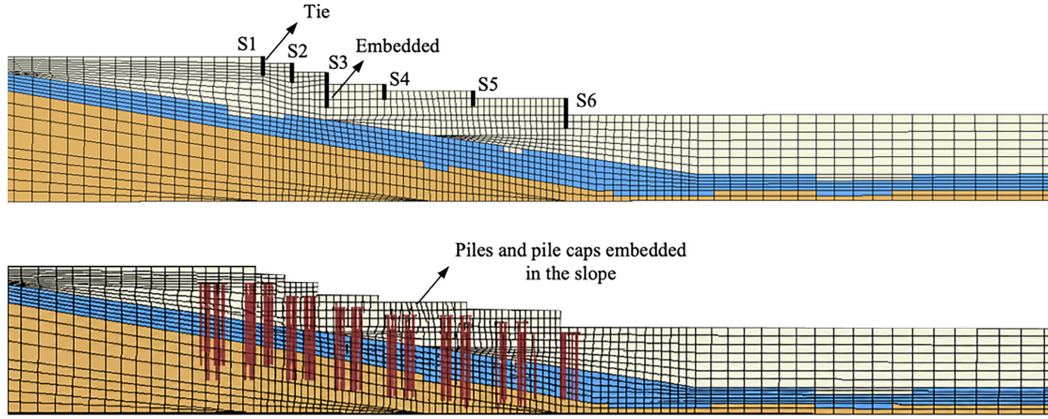


Figure 5: Contact relationship of the retaining walls and pile foundation system with the surrounding soil.

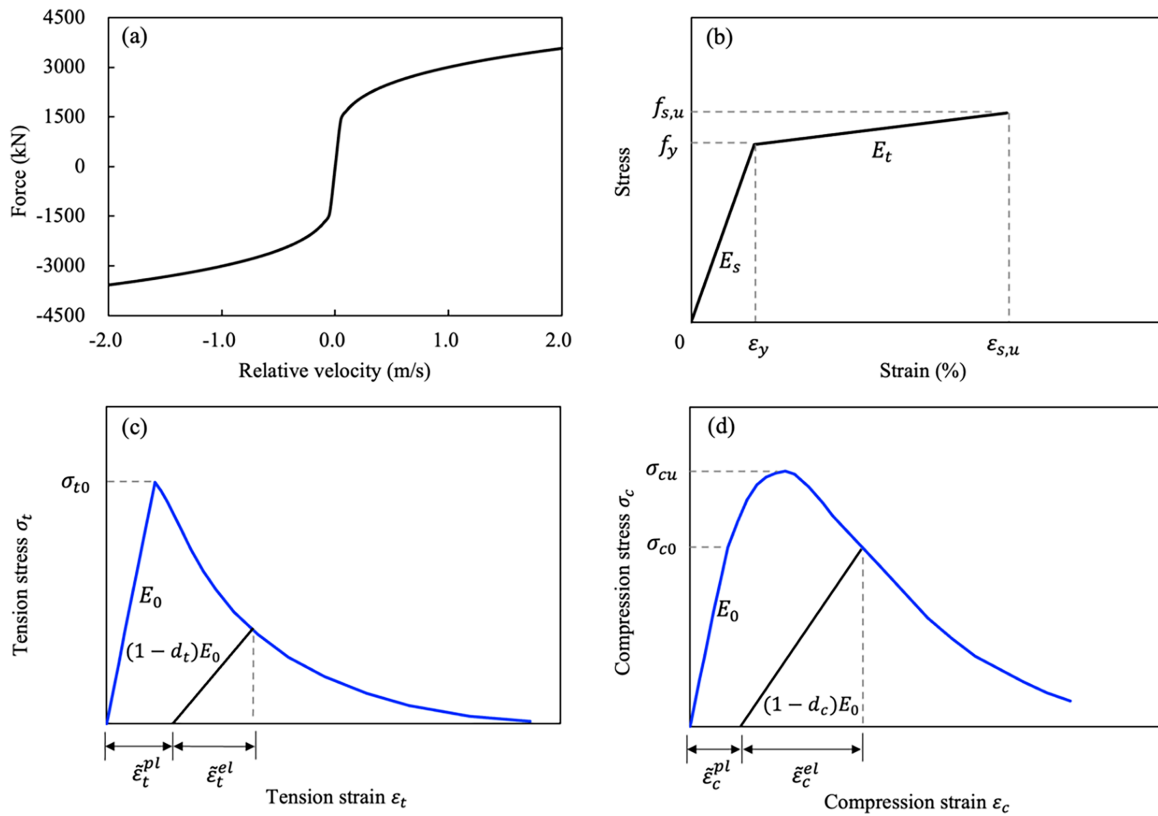


Figure 6: Constitutive models of the VFD, steel, and concrete used in the numerical model: (a) Force-velocity relationship of the VFD; (b) Bilinear elastoplastic constitutive model with kinematic hardening for steel; (c) Uniaxial tensile constitutive relationship of concrete in the CDP model; and (d) Uniaxial compressive constitutive relationship of concrete in the CDP model.

For material modeling, concrete was adopted for the lower frame-shear wall system and all floor slabs, while the core concrete of the concrete-filled steel tubular (CFST) members in the middle and upper structural zones was modeled separately. The upper steel frame adopts Q355 and Q420 steel, while the longitudinal reinforcement in the reinforced concrete frame adopts HRB400 and HRB500 steel bars. Both were modeled using the built-in elastoplastic model with kinematic hardening in Abaqus. As shown in Fig. 6b, the steel constitutive relationship is represented by a bilinear model, in which E_s is the elastic modulus, f_y is the initial yield stress, $f_{s,u}$ is the ultimate stress, ε_y is the yield strain, and $\varepsilon_{s,u}$ is the corresponding ultimate strain.

Concrete was modeled using the built-in concrete damaged plasticity (CDP) model in Abaqus. As shown in Fig. 6c,d, the uniaxial tensile and compressive behaviors of concrete are defined separately, where the subscripts t and c denote tension and compression, respectively. The initial elastic modulus is denoted by E_0 ; σ_{t0} and σ_{c0} represent the initial yield stresses in tension and compression, respectively; σ_{cu} denote the corresponding ultimate compressive strengths. The corresponding inelastic behavior is characterized by the equivalent tensile and compressive plastic strains, denoted by $\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$, respectively. As damage develops, the unloading stiffness is reduced to $(1 - d_t)E_0$ in tension and $(1 - d_c)E_0$ in compression, where d_t and d_c are the tensile and compressive damage variables, respectively. The materials and constitutive models of different structural components are summarized in Table 3, and the corresponding key model parameters are given in Tables 4–6. The structural components located below the ground surface were treated using the embedded region constraint (Fig. 7).

Table 3: Material and constitutive models adopted for different structural components.

Component	Material	Grade	Constitutive Model	Key Parameters
VFD	Damper	–	nonlinear force-velocity relationship	C, α
Frame-shear wall	concrete	C60	CDP	$\rho_c, E_0, \sigma_{t0}, \sigma_{c0}, \sigma_{cu}$
Floor slabs	concrete	C30	CDP	$\rho_c, E_0, \sigma_{t0}, \sigma_{c0}, \sigma_{cu}$
Steel frame	Steel	Q355, Q420	Elastoplastic model with kinematic hardening	$\rho_s, E_s, \nu, f_y, f_{s,u}, \varepsilon_{s,u}$
Reinforcement	Steel bar	HRB400, HRB500	Elastoplastic model with kinematic hardening	$\rho_s, E_s, \nu, f_y, f_{s,u}, \varepsilon_{s,u}$

Note: “ ρ_c ” and “ ρ_s ” denote the densities of concrete and steel, respectively.

Table 4: Parameters of the nonlinear force-velocity model for the VFD.

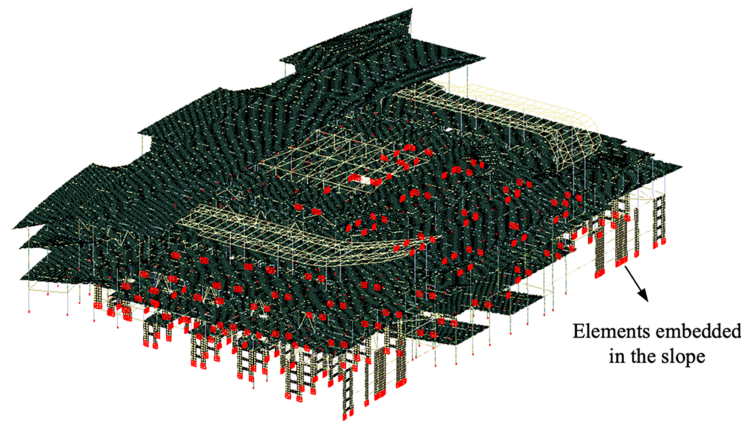
Device	Constitutive Model	C (N)	α
VFD	$F_{VFD} = CV^\alpha$	3×10^6	0.25

Table 5: Parameters of the concrete damaged plasticity.

Material	ρ_c (kg/m ³)	E_0 (GPa)	ν	σ_{t0} (MPa)	σ_{c0} (MPa)	σ_{cu} (MPa)
C30	2500	30.0	0.2	2.412	14.07	20.10
C60	2500	36.0	0.2	3.420	26.95	38.50

Table 6: Parameters of the bilinear elastoplastic model with kinematic hardening for steel materials.

Material	ρ_s (kg/m ³)	E_s (GPa)	ν	f_y (MPa)	$f_{s,u}$ (MPa)	$\varepsilon_{s,u}$
Q355	7800	206	0.25	355.0	759.9	0.09631
Q420	7800	206	0.25	420.0	823.6	0.09600
HRB400	7800	206	0.25	400.0	804.0	0.09610
HRB500	7800	206	0.25	500.0	902.0	0.09562

**Figure 7:** Schematic view of the GTC elements embedded in the slope.

2.3 Seismic Input Motions and Boundary Conditions

A ground motion with a peak acceleration of 0.20 g on level ground was taken as the reference input. According to JGJ/T 472-2020 [11], and considering the slope angle of the site, the seismic amplification factor for the slope site was taken as 1.1, corresponding to a target surface peak ground acceleration of 0.22 g. For the fixed-base model, the amplified ground motion with a peak acceleration of 0.22 g was directly applied at the base of the superstructure model. For the SSI model, the bedrock motion was input at the bottom boundary of the model and amplitude-scaled so that the surface peak ground acceleration at the center of the GTC structure was approximately 0.22 g. The input motions adopted for the two models are shown in Fig. 8, and the excitation was applied in the Y direction. The bedrock input motion was artificially synthesized using the target bedrock response spectrum and peak ground acceleration as control parameters. For the fixed-base model, the corresponding surface input motion was obtained through one-dimensional site response analysis based on typical borehole profiles, using the synthesized bedrock motion as the input.

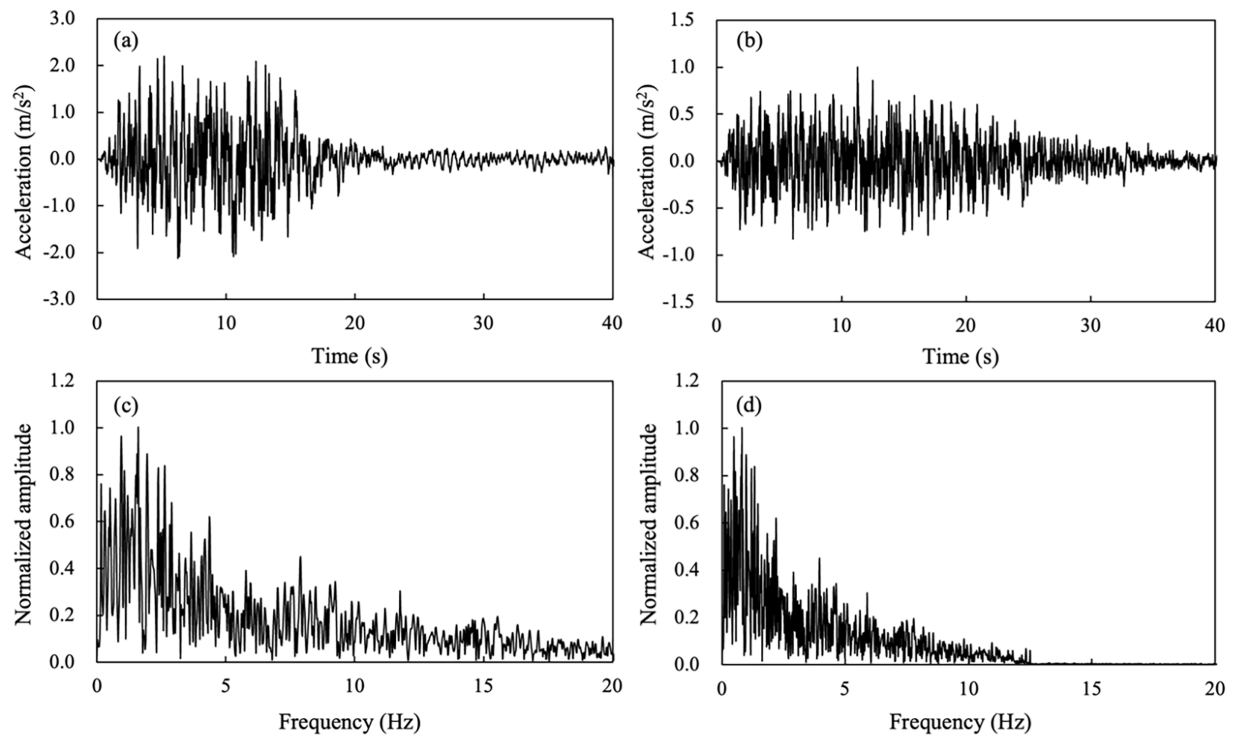


Figure 8: Input earthquake motions and Fourier spectrum for the fixed-base and SSI models: (a) fixed-base motion; (b) SSI motion; (c) fixed-base spectrum; and (d) SSI spectrum.

Prior to the dynamic analysis, a static analysis step was conducted to establish the initial stress state of the integrated soil-structure system. During this step, gravity was applied to the entire soil-structure system, while the dead and live loads of the superstructure were simultaneously incorporated through equivalent concentrated and distributed surface loads. Meanwhile, normal displacement constraints were imposed on the lateral boundaries and bottom of the site model to establish the initial static equilibrium state of the system. In the subsequent dynamic analysis, the nodal reaction forces obtained from the static analysis were equivalently applied to the model boundaries to maintain the initial equilibrium state. Considering the soil-layer distribution and the stepped-slope site conditions, the bottom boundary was modeled as a rigid boundary to represent the underlying bedrock, and the seismic motion was applied at this boundary. For the lateral boundaries, since the elevations of the two sides of the stepped-slope site are different, only the z -direction displacement was constrained during the dynamic analysis. To reduce possible artificial wave reflections from the lateral boundaries, a sufficiently large soil domain was adopted along the excitation direction to minimize the influence of boundary effects on the structural response. As shown in Fig. 9, the comparison of acceleration time histories at representative monitoring points near the lateral boundaries indicates that the influence of lateral boundary reflections on the structural response region is limited.

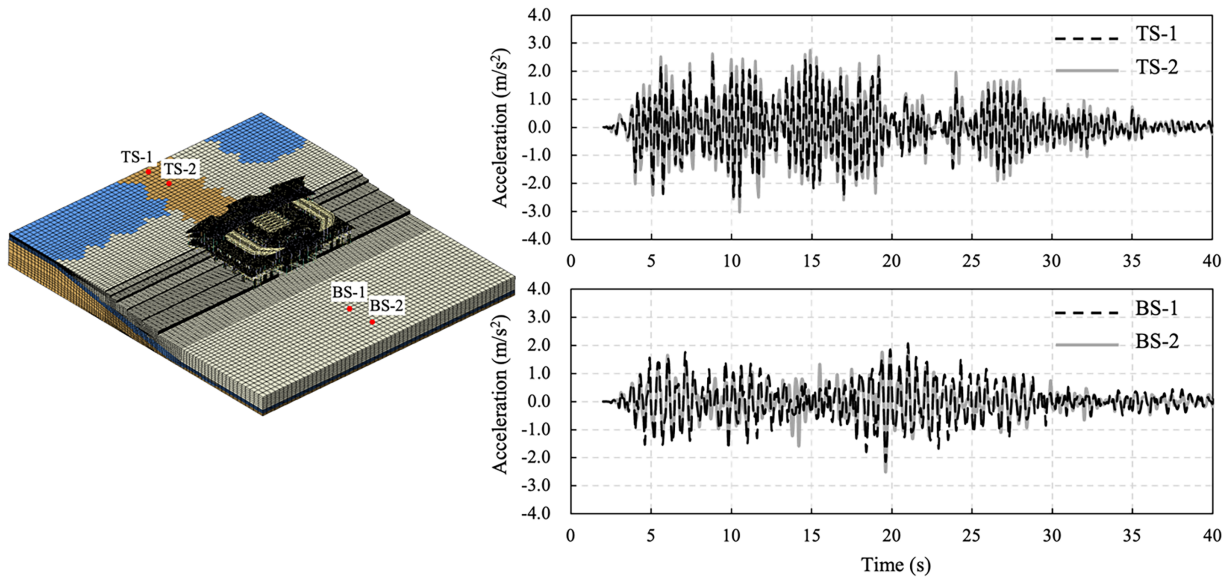


Figure 9: Layout of representative monitoring points and acceleration time histories near the lateral boundaries.

3 Effects of SSI on Seismic Response of the Structure

Based on the established three-dimensional finite element model, a comparative analysis of the structural seismic responses with and without SSI is conducted in this section. Particular attention is given to the effects of SSI on structural damage distribution, story drift ratio, story shear force, overturning moment, and energy dissipation characteristics.

3.1 Effect of SSI on Structural Damage Distributions

Fig. 10 presents the overall slab damage distributions of the SSI model and the fixed-base model. The results show that, in both models, slab damage under earthquake excitation is dominated by tensile damage, whereas compressive damage remains relatively limited. From the overall distribution, no obvious difference is observed in the slab damage pattern between the SSI and fixed-base models. Since the overall slab damage is governed by tensile damage, the following analysis focuses on the comparison of tensile damage distributions at different floor levels.

Fig. 11 shows the tensile damage distributions of slabs at the representative upper floors. Overall, the tensile damage at 5F is relatively minor and is only sporadically distributed near the edge strips on both sides and local cantilever edges, with limited differences between the SSI and fixed-base models at this floor. In contrast, the tensile damage at 3F becomes significantly more severe and is mainly concentrated around the large openings, in the connecting slab strips beneath the openings, and at local corner regions, making it one of the most damaged upper floors. A further comparison indicates that the main locations of tensile damage at 3F are generally similar in the two models, whereas the fixed-base model exhibits a wider damage range and more continuous damage distribution around the large openings, connecting slab strips, and local corner regions.

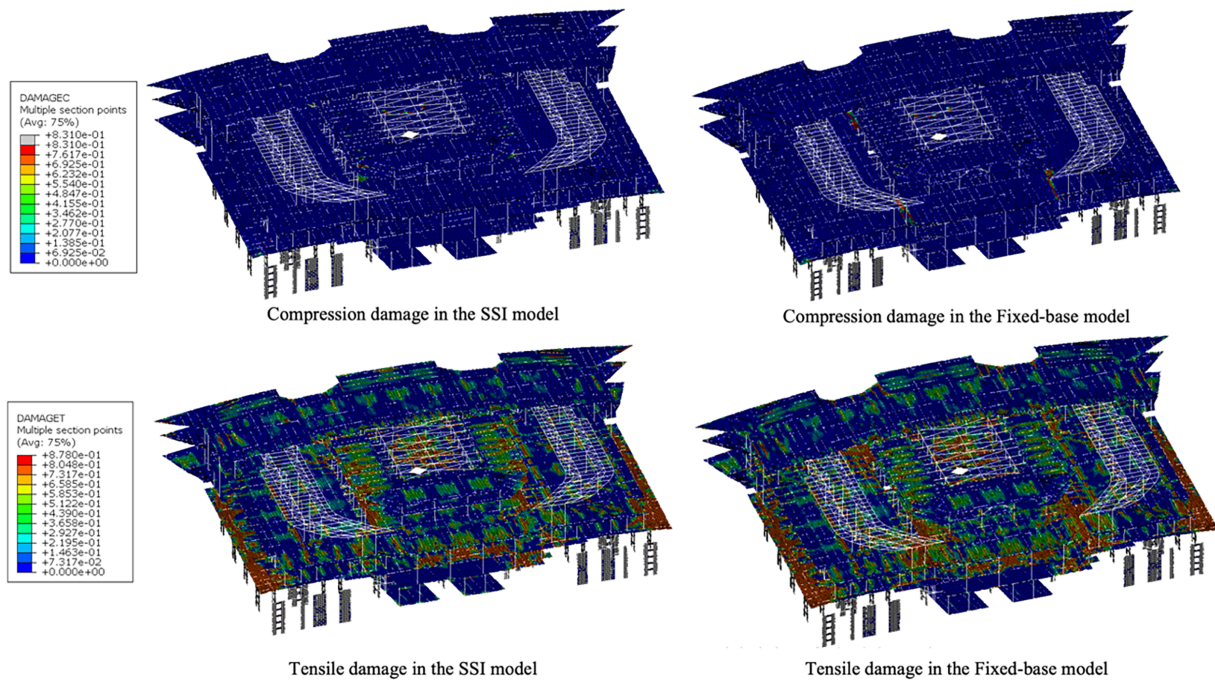


Figure 10: Comparison of tensile and compressive damage distributions between the SSI and fixed-base models.

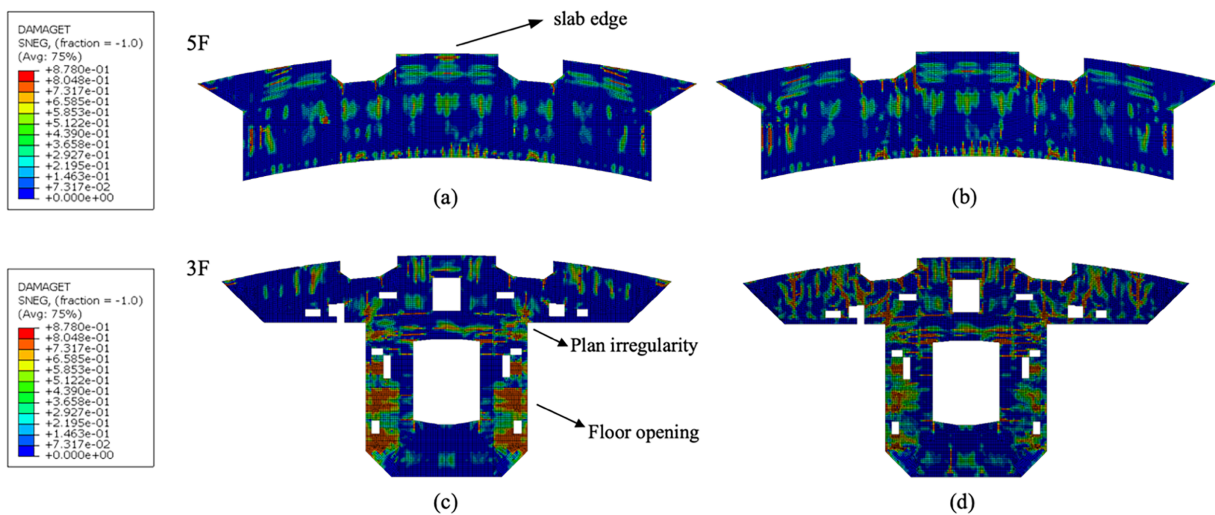


Figure 11: Comparison of tensile damage distributions at upper floors between the SSI and fixed-base models: (a) 5F in the SSI model; (b) 5F in the fixed-base model; (c) 3F in the SSI model; and (d) 3F in the fixed-base model.

Fig. 12 shows the tensile damage distributions of slabs at the middle floors. The tensile damage at 2F is mainly concentrated around the large openings and further extends to the narrow slab strips on both sides of the openings, with severe tensile damage concentrated along the opening edges. The tensile damage at 1F spreads over a wider range and is evident not only in the middle slab strips and near local openings, but also along both side edges, where continuous damage development can be observed. A comparison between the two models shows that the main locations of tensile damage at 2F and 1F are generally similar, whereas the

fixed-base model exhibits more continuous damage bands and a larger overall damage extent around the openings, along the narrow slab strips, in the middle transition zones, and near both side edges.

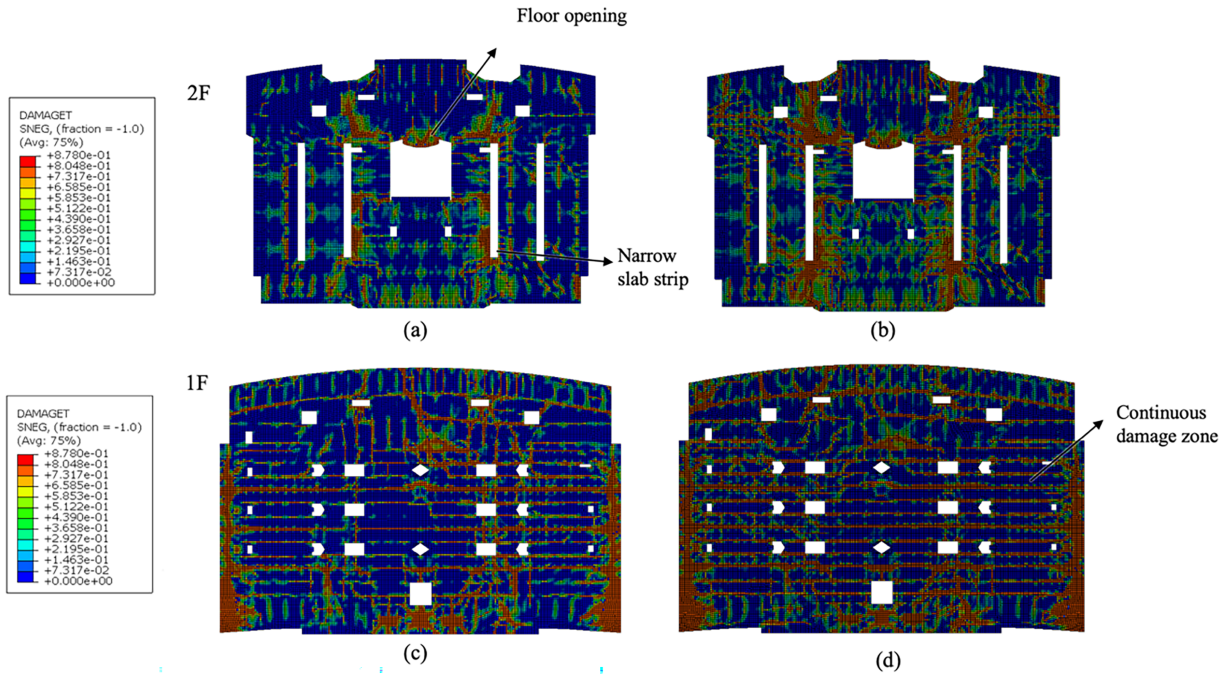


Figure 12: Comparison of tensile damage distributions at middle floors between the SSI and fixed-base models: (a) 2F in the SSI model; (b) 2F in the fixed-base model; (c) 1F in the SSI model; and (d) 1F in the fixed-base model.

Fig. 13 shows the tensile damage distributions of slabs at the lower floors. The tensile damage at B2F is mainly concentrated around the large openings and at the intersection region between the diagonal slab and the high-speed rail-bearing slab. In comparison, the overall damage at B5F remains relatively slight, with only a small amount of tensile damage appearing in local strip regions and near the openings. A further comparison indicates that the SSI model shows more pronounced local tensile damage at B2F, particularly at the openings in the diagonal metro track-bearing slab and the transverse rail-bearing openings. Although the overall damage at B5F is slight in both models, the SSI model still exhibits relatively more obvious local strip-like tensile damage.

These results indicate that, although the fixed-base model tends to predict a larger extent of slab tensile damage at most above-ground floors, soil-structure interaction may lead to more concentrated or intensified local slab damage in the lower floors near the slope and the split-level region, resulting in damage response characteristics different from those of the upper floors.

3.2 Effect of SSI on Story Drift Ratio

This section further examines the effect of SSI on the story drift ratio of the structure. Fig. 14 shows the monitoring points for story drift ratio arranged along the structural height, with their locations selected in representative regions of the major floors. Specifically, the monitoring points on the upper floors are mainly placed in the slope-top edge region, those on the middle floors are arranged near the large floor openings, and those on the lower floors are concentrated in the open area at the slope toe, so as to reflect the effects of local stiffness weakening caused by floor openings and the influence of slope topography on the distribution of interstory deformation.

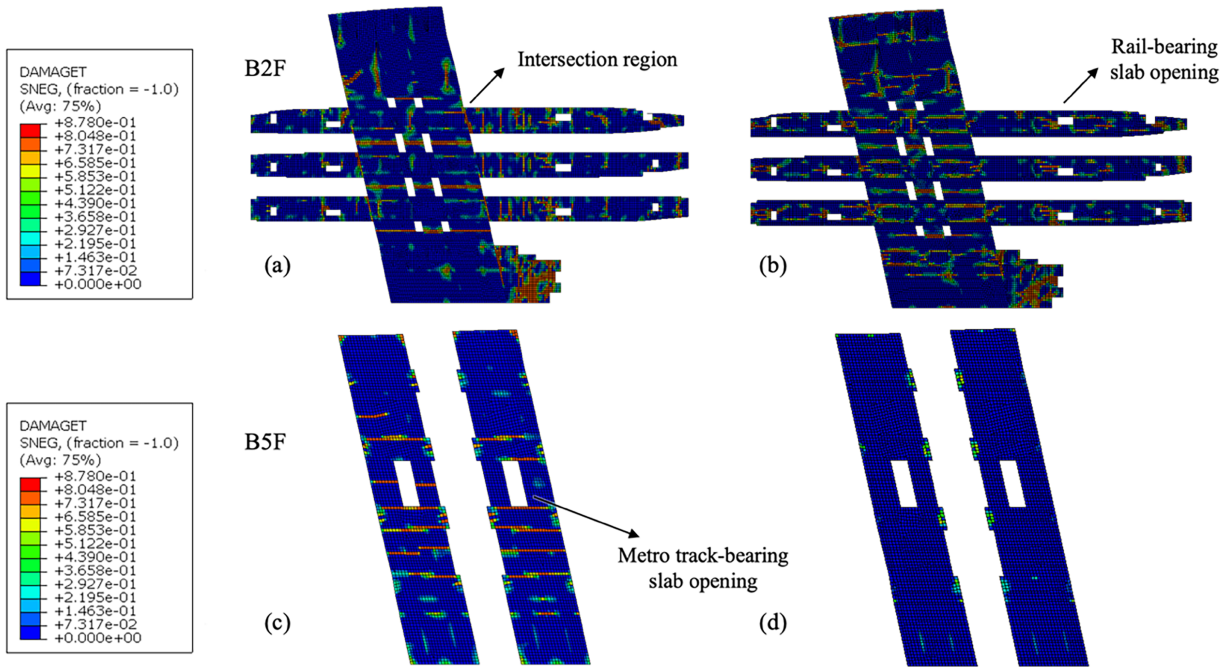


Figure 13: Comparison of tensile damage distributions at lower floors between the SSI and fixed-base models: (a) B2F in the SSI model; (b) B2F in the fixed-base model; (c) B5F in the SSI model; and (d) B5F in the fixed-base model.

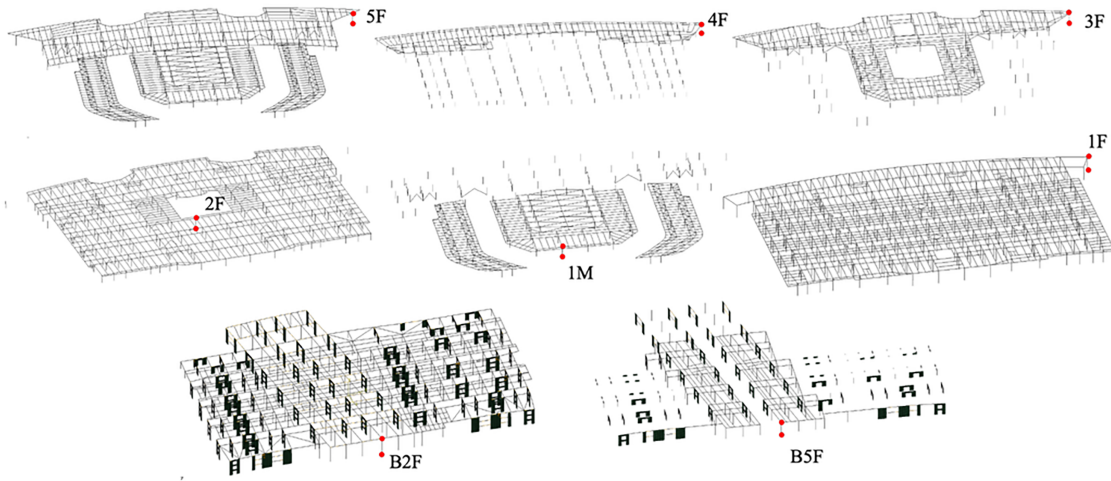


Figure 14: Layout of monitoring points for story drift ratio along the structural height.

Fig. 15 presents the comparison of story drift ratio between the fixed-base and SSI models. While both models share a trend where deformations are generally smaller at the base and amplified in the middle elevations, their specific distribution profiles exhibit notable discrepancies. The fixed-base model consistently overestimates the story drift ratios compared to the SSI model across the middle floors, with the most severe deviations concentrated between 1F and 3F. Notably, the fixed-base model predicts a broad peak deformation zone (ranging continuously from 1M to 3F) with a maximum story drift ratio of approximately 0.72×10^{-2} . In contrast, the fully-coupled SSI model yields a much sharper, reduced peak of 0.51×10^{-2} localized at levels 2F and 3F, while dropping significantly below that. This clearly demonstrates that incorporating SSI effectively

dissipates energy and dramatically alleviates the story deformation demands on the middle floors, although the response at the topmost floor (5F) remains largely unaffected.

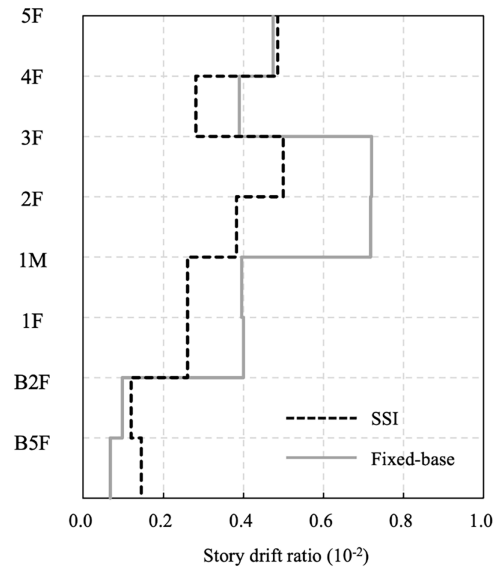


Figure 15: Comparison of story drift ratio at the monitoring points between the SSI and fixed-base model.

It is noteworthy that the story drift ratios at B2F and B5F are larger in the SSI model than in the fixed-base model. From the perspective of structural layout, these two floors, as the lower floors closest to the soil-contact region, are more susceptible to the influence of the slope free surface and local stiffness variations in the lower shear wall system. Therefore, after SSI is taken into account, the local story deformation at these lower floors shows a certain degree of amplification. As a result, although the fixed-base model can reasonably capture the overall distribution trend of story drift along the height, its evaluation of deformation at local floors close to the soil-contact region may still involve some deviation.

3.3 Effect of SSI on Story Shear Force and Overturning Moment

To further reveal the differences in force response between the fixed-base model and the SSI model, Fig. 16 compares the distributions of story shear force and story overturning moment along the structural height. The results show that the story shear force generally increases from the upper floors to the lower floors, although the peak values occur at different locations in the two models. The maximum story shear force of the SSI model occurs at B2F, whereas that of the fixed-base model occurs at B5F, indicating that consideration of soil-structure interaction changes the force distribution in the lower part of the structure. In general, the fixed-base model yields larger story shear forces than the SSI model at most middle and upper floors, while at B2F the SSI model produces a larger story shear force. Considering the structural layout, the B2F floor spans across four stepped slope benches, and the differences in topographic amplification at different benches become more pronounced. This leads to stronger nonuniform seismic input along the structural length of this floor, thereby increasing the local story shear response in the SSI model.

Compared with story shear force, the story overturning moment shows a more pronounced cumulative increase toward the lower floors, and the peak values of both models occur at the lowest floor, B5F. In terms of magnitude, the story overturning moments of the fixed-base model are generally larger than those of the SSI model at all floors. These results indicate that, although the fixed-base model can reasonably capture the overall distribution pattern of structural force demand along the height, it is less sensitive to the redistribution

of force demand at lower local floors under complex stepped topographic conditions, especially the local increase in story shear at B2F. Therefore, for irregular structures founded on complex slope sites, the fixed-base assumption still has limitations when used alone for force-response evaluation.

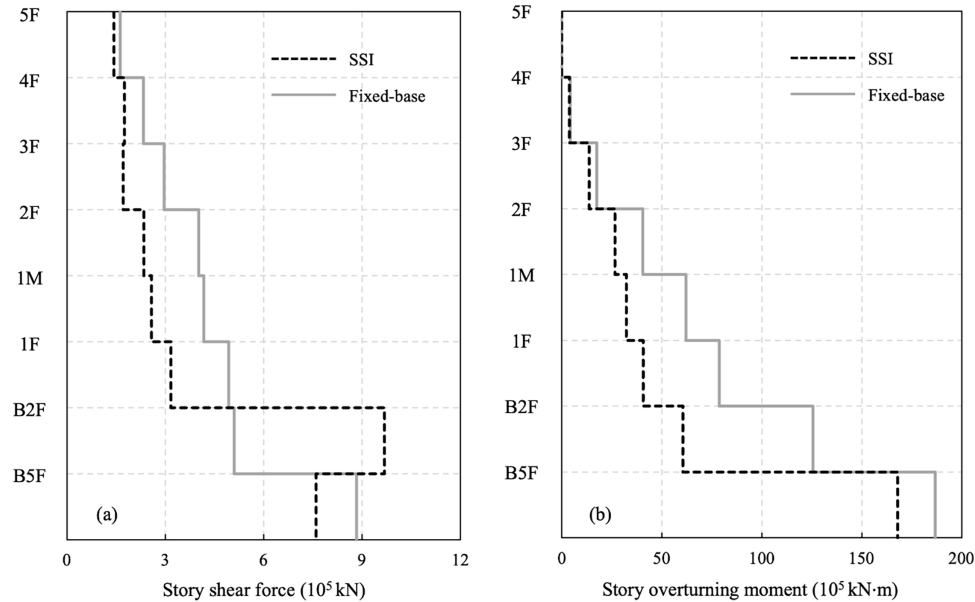


Figure 16: Comparison of story shear force and story overturning moment between the SSI and fixed-base model.

3.4 Effect of SSI on Energy Dissipation Characteristics

Fig. 17 presents the cumulative time-history responses of different energy dissipation components for the fixed-base model and the SSI model. In both models, the energy dissipation components increase rapidly during the period of concentrated seismic energy input and gradually approach stable values after about 20 s. Compared with the SSI model, the fixed-base model exhibits larger structural damping energy dissipation, damper energy dissipation, and structural nonlinear energy dissipation. In contrast, the SSI model additionally shows pronounced soil damping energy dissipation, indicating that the input energy is redistributed among the structure, dampers, and surrounding soil when soil-structure interaction is considered.

Regarding the individual energy dissipation components, Fig. 17a shows that the structural damping energy dissipation in the fixed-base model is larger than that in the SSI model, with final cumulative values of approximately 1.87×10^4 and 1.49×10^4 kJ, respectively. Fig. 17b indicates an even more pronounced difference in damper energy dissipation, with the fixed-base model reaching about 1.25×10^4 kJ, whereas the SSI model reaches only about 3.9×10^3 kJ. Fig. 17c further shows that the structural nonlinear energy dissipation reaches approximately 1.04×10^5 kJ in the fixed-base model and about 7.71×10^4 kJ in the SSI model, suggesting that under the fixed-base assumption the structure undergoes a higher degree of nonlinear development, accompanied by more significant plasticity and damage evolution of structural components. Overall, the presence of SSI significantly modifies the energy dissipation pattern of the system, leading to lower structural damping energy, damper energy, and nonlinear energy dissipation in the superstructure. This difference may be attributed to the fact that, when SSI is considered, part of the incident seismic wave energy is scattered, radiated, and damped in the surrounding soil before being transmitted to the superstructure. Therefore, the fixed-base model may tend to overestimate the energy demand borne by the superstructure and the supplemental energy dissipation devices.

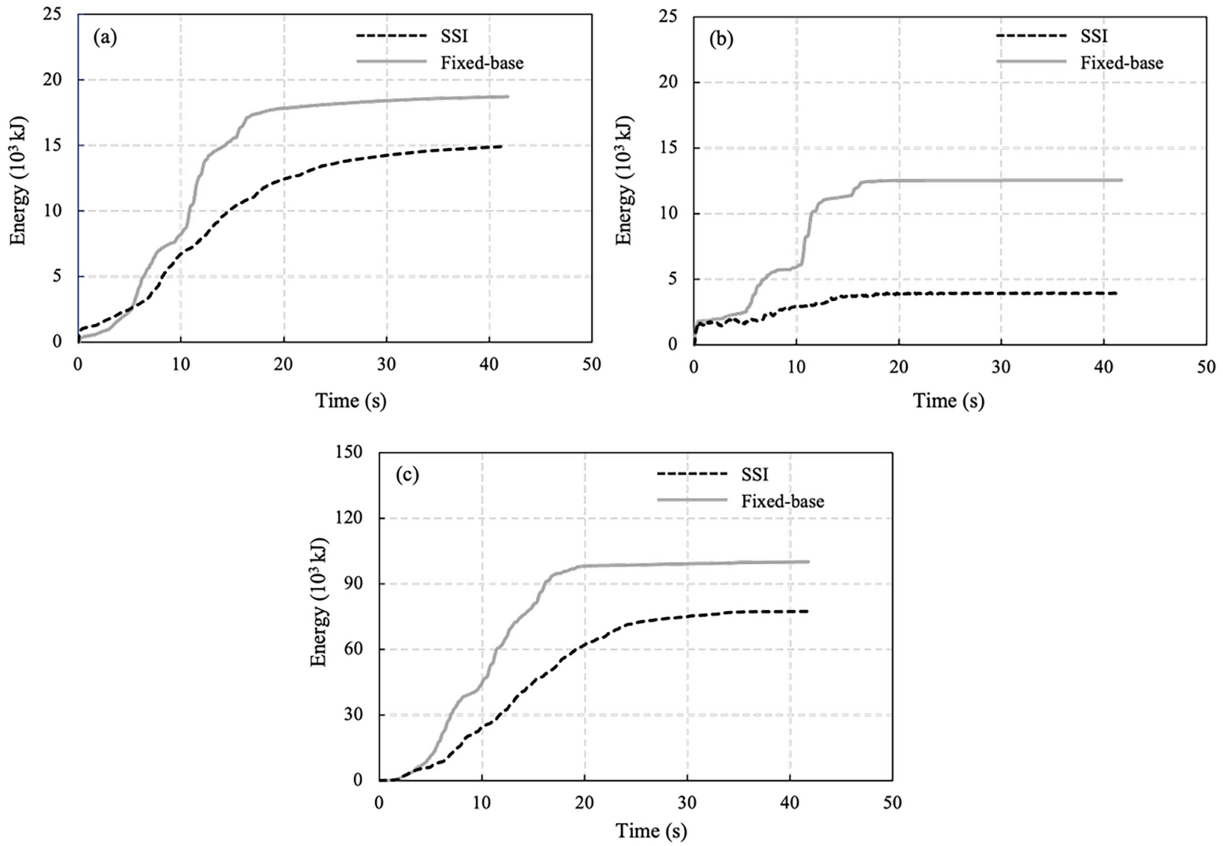


Figure 17: Cumulative time-history responses of different energy dissipation components: (a) structural damping energy dissipation; (b) damper energy dissipation; and (c) structural nonlinear energy dissipation.

4 Conclusions

This paper develops a fully coupled 3D nonlinear finite element model based on a large-scale transportation hub situated on a multi-bench stepped slope. The effects of dynamic soil-structure interaction (SSI) on the structural response are systematically investigated and compared with a conventional fixed-base model subjected to topographically amplified ground motions. Four key conclusions can be drawn from this research.

1. SSI has limited influence on the overall distribution pattern of the structural response, but it noticeably modifies localized responses, especially at lower floors near the slope and soil-contact region. In terms of slab damage and interstory drift ratio, the fixed-base model generally predicts larger tensile damage and larger drift ratios at most above-ground floors, whereas the SSI model exhibits relatively more unfavorable local responses at some lower floors close to the slope and soil-contact region.
2. The topographical irregularities of the stepped slope lead to dramatic internal force redistribution, which can only be captured through SSI analysis. Because the B2F floor spans four stepped slope benches, the nonuniform topographic amplification at different bench locations results in a localized increase in story shear force at this level. This feature is not clearly reflected by the fixed-base model, highlighting the importance of SSI analysis for identifying unfavorable local force responses in similar structures on stepped slope sites.
3. Incorporating SSI changes the energy dissipation pattern of the soil-structure system. Compared with the fixed-base model, in which energy dissipation is mainly concentrated in structural damping,

supplemental viscous fluid dampers, and structural nonlinearity, the SSI model alters the energy transmission path and re-allocates the seismic energy demands. Consequently, the energy dissipation demand of the superstructure and protective devices is reduced to some extent.

4. The overall seismic assessment of complex step-back structures differs fundamentally from that of conventional flat-site buildings. Simply applying simplified topographic amplification factors to a fixed-base model is inherently inadequate. Such an approach leads to uneconomical over-design for the upper superstructure while concurrently introducing critical safety hazards at the lower embedded regions. This study provides a robust basis for revising current seismic design strategies for large-scale infrastructure on stepped terrains.

It should be noted that the three-dimensional numerical model established in this study involves certain simplifications. The equivalent linear soil model and the embedded-region constraint adopted at soil-structure interfaces may simplify complex material nonlinearity and suppress local interface mechanisms. In future work, more refined nonlinear soil models, nonlinear interface models, and experimental validation will be considered to better capture these local nonlinear mechanisms and further improve the proposed modeling framework.

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Availability of Data and Materials: The data used to support the findings of this study are available from the authors upon request.

Ethics Approval: Not applicable.

Conflicts of Interest: The authors declare no conflicts of interest.

Appendix A

For the B31 beam elements, the longitudinal rebars were defined using the *REBAR keyword in Abaqus. A representative input definition is given as follows:

```
*REBAR, ELEMENT = BEAM, MATERIAL = HRB400, NAME = RB1
BEAM_SET_1, 3.14159E-4, 0.11, 0.21
```

In this definition, “BEAM_SET_1” denotes the set of beam element to which the rebar is assigned, “3.14159E-4” is the cross-sectional area of the rebar, and “0.11” and “0.21” are the coordinates of the rebar location with respect to the local beam-section axes X_1 and X_2 , respectively, as shown in Fig. A1. The rebar area was determined from the nominal bar diameter, while the coordinates X_1 and X_2 were determined according to the beam-section dimensions and the designed rebar layout.

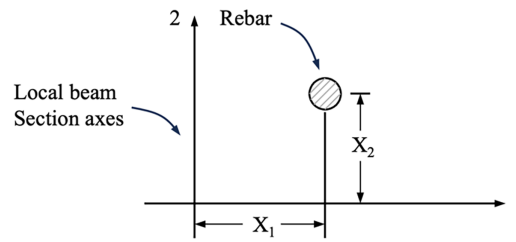


Figure A1: Definition of rebar location in the local coordinate system of a B31 beam section.

References

1. Shabani MJ, Shamsi M, Ghanbari A. Dynamic response of three-dimensional midrise buildings adjacent to slope under seismic excitation in the direction perpendicular to the slope. *Int J Geomech*. 2021;21(11):04021204. doi:10.1061/(asce)gm.1943-5622.0002158.
2. Shamsi M, Shabani MJ, Vakili AH. Three-dimensional seismic nonlinear analysis of topography-structure-soil-structure interaction for buildings near slopes. *Int J Geomech*. 2022;22(3):04021295. doi:10.1061/(asce)gm.1943-5622.0002301.
3. Das S, Maheshwari BK. Influence of slope topography on soil-structure interaction during earthquakes. *Acta Geotech*. 2024;19(7):4715–30. doi:10.1007/s11440-023-02186-8.
4. Ma Q, Wang F, Tao D, Xie Q, Liu H, Jiang P. Topographic site effects of Xishan Park ridge in Zigong city, Sichuan considering epicentral distance. *J Seismol*. 2021;25(6):1537–55. doi:10.1007/s10950-021-10048-7.
5. Gallipoli MR, Bianca M, Mucciarelli M, Parolai S, Picozzi M. Topographic versus stratigraphic amplification: mismatch between code provisions and observations during the L'Aquila (Italy, 2009) sequence. *Bull Earthq Eng*. 2013;11(5):1325–36. doi:10.1007/s10518-013-9446-3.
6. Luo Y, Del Gaudio V, Huang R, Wang Y, Wasowski J. Evidence of hillslope directional amplification from accelerometer recordings at Qiaozhuang (Sichuan—China). *Eng Geol*. 2014;183(4):193–207. doi:10.1016/j.enggeo.2014.10.015.
7. He J, Qi S, Zhan Z, Guo S, Li C, Zheng B, et al. Seismic response characteristics and deformation evolution of the bedding rock slope using a large-scale shaking table. *Landslides*. 2021;18(8):2835–53. doi:10.1007/s10346-021-01682-w.
8. Li Y, Wang G, Wang Y. Parametric investigation on the effect of sloping topography on horizontal and vertical ground motions. *Soil Dyn Earthq Eng*. 2022;159(9):107346. doi:10.1016/j.soildyn.2022.107346.
9. Nguyen KV, Gatmiri B. Evaluation of seismic ground motion induced by topographic irregularity. *Soil Dyn Earthq Eng*. 2007;27(2):183–8. doi:10.1016/j.soildyn.2006.06.005.
10. CEN. Eurocode 8: design of structures for earthquake resistance: part 5: foundations, retaining structures and geotechnical aspects. Brussels, Belgium: European Committee for Standardization; 2004 [cited 2026 Jun 26]. Available from: <https://www.phd.eng.br/wp-content/uploads/2014/11/en.1998.5.2004.pdf>.
11. JGJ/T 472-2020. Standard for design of building structures on slopes. Beijing, China: Ministry of Housing and Urban-Rural Development of the People's Republic of China; 2020.
12. Shabani MJ, Ghanbari A. Comparison of seismic behavior of steel building adjacent to slope topography by considering fixed-base, SSI and TSSI. *Asian J Civ Eng*. 2020;21(7):1151–69. doi:10.1007/s42107-020-00266-8.
13. Bhaumik L, Raychowdhury P. Seismic response analysis of a nuclear reactor structure considering nonlinear soil-structure interaction. *Nucl Eng Des*. 2013;265(9):1078–90. doi:10.1016/j.nucengdes.2013.09.037.
14. Jarernprasert S, Bazan-Zurita E, Bielak J. Seismic soil-structure interaction response of inelastic structures. *Soil Dyn Earthq Eng*. 2013;47(3):132–43. doi:10.1016/j.soildyn.2012.08.008.
15. Baidya S, Roy BK. Response mitigation of building isolated by shape memory alloy rubber bearing considering soil structure interaction effect. *J Vibr Eng Technol*. 2024;12(2):1299–316. doi:10.1007/s42417-024-01476-z.
16. Carbonari S, Dezi F, Leoni G. Nonlinear seismic behaviour of wall-frame dual systems accounting for soil-structure interaction. *Earthq Engng Struct Dyn*. 2012;41(12):1651–72. doi:10.1002/eqe.1195.

17. Pitilakis KD, Karapetrou ST, Fotopoulou SD. Consideration of aging and SSI effects on seismic vulnerability assessment of RC buildings. *Bull Earthq Eng*. 2014;12(4):1755–76. doi:10.1007/s10518-013-9575-8.
18. Song P, Guo S, Zhao W, Xin Q. Seismic response analysis of reinforced concrete frame structures considering slope effects. *Appl Sci*. 2023;13(8):5149. doi:10.3390/app13085149.
19. Bahuguna A, Firoj M. Numerical simulation of seismic response of Slope-Foundation–Structure interaction for mid-rise RC buildings at various locations. *Structures*. 2022;44(5):343–56. doi:10.1016/j.istruc.2022.08.011.
20. Kourkoulis R, Anastasopoulos I, Gelagoti F, Gazetas G. Interaction of foundation-structure systems with seismically precarious slopes: numerical analysis with strain softening constitutive model. *Soil Dyn Earthq Eng*. 2010;30(12):1430–45. doi:10.1016/j.soildyn.2010.05.001.
21. Fatahi B, Huang B, Yeganeh N, Terzaghi S, Banerjee S. Three-dimensional simulation of seismic slope-foundation–structure interaction for buildings near shallow slopes. *Int J Geomech*. 2020;20(1):04019140. doi:10.1061/(asce)gm.1943-5622.0001529.
22. Islam MR, Van Nguyen D, Park SJ, Kim D, Choo YW, Tessari A. Seismic resilience of steel moment resisting frame (MRF) structures on slopes considering earthquake frequency content. *Structures*. 2025;82(6):110720. doi:10.1016/j.istruc.2025.110720.
23. Zhang N, Chen Z. Seismic response characteristics of high-speed railway hub station considering pile-soil interactions. *Buildings*. 2025;15(14):2466. doi:10.3390/buildings15142466.
24. Sun L, Bai Y, Lai Z. Shaking table test on seismic performance of a large-span high-rise building. *Sci Rep*. 2024;14(1):6580. doi:10.1038/s41598-024-57068-0.
25. Guo T, Wang J, Ji X, Song L, Sun Y, Zhang Y. Seismic resilience enhancement of irregular space structure using friction-damped self-centering tension braces. *J Struct Eng*. 2024;150(3):04024005. doi:10.1061/jsendh.steng-13024.
26. Seleemah AA, Constantinou MC. Investigation of seismic response of buildings with linear and nonlinear fluid viscous dampers. Buffalo, NY, USA: National Center for Earthquake Engineering Research, University at Buffalo; 1997 [cited 2026 Jul 16]. Available from: <https://www.eng.buffalo.edu/mceer-reports/97/97-0004.pdf>.