



ARTICLE

Dynamic Elastoplastic Analysis of a Long-Span Composite Structure Based on Cross-Platform Collaborative Modeling: A Case Study of a University Sports Building

Hongtao Yue¹, Xiang Yu^{2,*}, Chenghu Wang², Xiaona Kou¹, Zhongyuan Liu^{1,3} and Manli Ou¹

¹School of Civil and Environmental Engineering, Hunan University of Technology, Zhuzhou, China

²Hunan Zhimou Planning Engineering Design and Consulting Co., Ltd., Zhuzhou, China

³School of Civil Engineering, NingboTech University, Ningbo, China

*Corresponding Author: Xiang Yu. Email: yuxiangabcd2026@163.com

Received: 24 March 2026; Accepted: 13 May 2026; Published: 23 October 2026

ABSTRACT: Dynamic elastoplastic time history analysis is a critical methodology for evaluating the seismic performance and damage evolution mechanisms of complex long span structures. To address the modeling challenges inherent in hybrid frame space grid systems, this study implemented an efficient collaborative workflow utilizing the data interface between YJK and ABAQUS. A long span sports training building served as the research prototype for systematic multidimensional elastic and elastoplastic dynamic evaluations. The results indicate that under frequent earthquakes, the structure exhibits pronounced sensitivity to broadband excitations, with the artificial wave R1 inducing a maximum elastic inter story drift ratio of 1/605 and a peak base shear of 2442 kN. This response reveals a strong spatial torsional coupling effect triggered by the abrupt stiffness mutation at the corbel-truss interface. Under rare earthquakes, the maximum elastoplastic drift ratio reaches 1/159, remaining significantly below the code mandated collapse limit of 1/50. A hierarchical damage evolution mechanism was elucidated, where localized yielding at 270.7 MPa in the mid span top chords and corbel supports effectively functions as a structural fuse to dissipate seismic energy. Crucially, the maximum compressive strain in the concrete columns is maintained at 0.0019, which mobilizes the ultimate material strength while avoiding the strain softening region. This ensures the stability of the vertical load transfer path and demonstrates high seismic resilience as the structure remains repairable after extreme events. Furthermore, by extracting the three dimensional dynamic ultimate reaction forces, a shear dominated asymmetric loading characteristic was identified at the connection hubs, where the horizontal shear (681 kN) significantly exceeded the vertical reactions. Supplementary performance based verifications for the shear and sliding capacities of the corbels and truss supports were conducted to eliminate the risks of joint failure and roof unseating. The analytical framework of macroscopic overall evaluation, microscopic damage revelation, and critical joint verification established in this study provides a robust theoretical and engineering reference for the seismic design of similar complex spatial structures.

KEYWORDS: Collaborative modeling; long span hybrid structure; dynamic elastoplastic analysis; seismic resilience; multidimensional seismic excitation

1 Introduction

Dynamic elastoplastic analysis represents a key approach for evaluating the actual seismic performance of building structures subjected to strong earthquakes [1]. Traditional static or linear analysis methods, which are predominantly based on elastic assumptions, fail to accurately capture internal force redistribution, energy dissipation, and damage evolution once a structure enters the plastic stage during major seismic

events [2,3]. Dynamic elastoplastic analysis directly simulates the time-dependent effects of ground motion, accurately representing the complete response process of structural components from elastic behavior to yielding and potential failure [4]. These capabilities provide essential scientific support for performance-based seismic design, progressive collapse prevention, and life-cycle safety decision-making.

In recent years, the seismic safety of large-span spatial structures has drawn significant attention, especially following the observed structural vulnerabilities in recent seismic events [5]. Unlike regular multi-story buildings, large-span structures (e.g., gymnasiums, airport hangars) exhibit complex dynamic characteristics, dense natural frequencies, and a high sensitivity to spatial multi-support excitations [6]. Previous studies have extensively investigated the dynamic responses and failure mechanisms of these structures. The critical influence of the lower supporting substructures on the upper roof has been increasingly recognized. Nair et al. [7] and Zhao et al. [8] highlighted that the dynamic interaction between the flexible upper spatial grid and the rigid lower supporting frame significantly alters the internal force redistribution. In addition, Tian et al. [9] and Guo et al. [10] explored the progressive collapse mechanisms of long span spatial grids under dynamic loads. Under diverse seismic inputs, Zhou et al. [11] and Liu et al. [12] further explored the seismic behavior of large span steel trusses. To bridge the gap between numerical predictions and physical reality, Xu et al. [13] conducted shaking table tests to investigate the multi-dimensional seismic response of long-span grid structures with base isolation, and Galvis and Smith-Pardo [14] developed experimentally verified PMM interaction diagrams for shallow foundations. Chae et al. [15] further provided experimental validation for performance-based seismic design through large-scale real-time hybrid simulations.

In practical engineering applications, evaluating these complex spatial effects and connection interfaces remains challenging. Fang et al. [16] proposed an adaptive model order reduction method for nonlinear seismic analysis of civil structures based on the elastic-plastic states, which allows efficient yet accurate prediction of the global seismic response while accounting for nonlinearities at critical joints and members. Zhang et al. [17] investigated a two-dimensional isolation device for double-layer grid structures, demonstrating its effectiveness in mitigating horizontal and vertical seismic responses. Pang et al. [18] conducted shaking table tests and numerical simulations on a precast frame-shear wall structure with innovative untopped precast concrete floors, showing that the untopped floor system can achieve satisfactory seismic performance, with controlled inter-story drift and effective energy dissipation under earthquake excitations.

To accurately capture this upper-lower structural interaction under severe earthquakes, advanced coupling and integrated modeling methods are urgently required. Currently, structural engineering practice relies heavily on multiple isolated platforms: specialized design software (e.g., YJK, 3D3S) is utilized for structural layout and member section optimization to ensure code compliance [9,14], while high-fidelity finite element (FE) software (e.g., ABAQUS) is essential for severe-earthquake elastoplastic analysis. Existing coupling methodologies primarily focus on macro-micro multi-scale modeling within a single software, or rely on specific software combinations like SAUSAGE and ABAQUS. However, when transferring a complex, highly indeterminate large-span spatial model from a standard engineering design platform (like YJK) into an explicit FE environment (like ABAQUS), researchers typically resort to manual geometric reconstruction. This conventional process is not only extremely time-consuming but also highly prone to geometric distortion, discretization errors, and the loss of critical component properties.

Addressing these modeling barriers, there remains a lack of systematic research on achieving lossless underlying data conversion and collaborative modeling across multiple platforms for long span hybrid structures. To bridge this gap, this study introduces novel contributions to building design technology through an efficient cross platform collaborative modeling framework. Taking a long span university sports building as the research prototype, a dedicated data interface workflow is firstly developed to seamlessly map verified structural topology, section properties, and material nonlinearities from YJK/3D3S directly into the

ABAQUS/Explicit environment. Second, a comprehensive multidimensional dynamic elastoplastic evaluation framework is proposed to systematically analyze stress evolution and progressive collapse resistance under rare earthquakes. Third, a hierarchical damage evolution mechanism is elucidated for the composite structure, demonstrating how localized yielding acts as a structural fuse to dissipate seismic energy while maintaining the global load transfer path. Ultimately, these integrated findings provide a robust theoretical foundation and practical engineering references for the performance based seismic design of similar complex spatial structures.

2 Project Description

The sports training building of a university in Hunan Province constitutes a comprehensive facility that integrates athletic training and teaching functions. The building covers a total floor area of approximately 12,000 m² and includes one basement level and six above-ground stories, with a total height of 22.5 m. The basement accommodates equipment rooms and parking spaces, while the above-ground floors contain basketball courts, badminton courts, and auxiliary teaching facilities. The structural system adopts a cast-*in-situ* reinforced concrete frame, which provides favorable global integrity and seismic performance. The roof system consists of a steel space truss supported by corbels of the surrounding reinforced concrete frame columns. Rubber bearings are installed between the corbels and the space truss.

The sports training building is located in a region with a seismic precautionary intensity of 7 degrees (0.10 g), a design earthquake group of Group 1, and a site classification of Class II. According to the Standard for Classification of Seismic Protection of Building Constructions (GB 50223-2008) [19], school teaching buildings, including the present sports training building, require a seismic protection category not lower than the Key Protection Class (Class B). The General Standard for Seismic Protection of Building and Municipal Engineering (GB 55002-2021) [20] requires buildings classified as Key Protection Class to adopt seismic design measures corresponding to an intensity one degree higher than the local seismic precautionary intensity. The sports training building therefore adopts seismic design measures corresponding to an intensity of 8 degrees (0.20 g). In accordance with the Technical Specification for Space Grid Structures (JGJ 7-2010) [21], the steel space truss of the sports training building located in an 8-degree intensity region requires verification of vertical seismic effects to ensure structural safety under seismic actions.

3 Structural System Layout

The roof utilizes an orthogonal square pyramid space truss with projected plan dimensions of 44.7 m × 32.1 m. To optimize the internal force distribution through the arching effect, the truss depth varies from 3.1 m at the mid-span to 2.3 m at the supports, forming a shallow arch profile. Fig. 1 illustrates the detailed plan layout of the roof space truss.

The structural system of the finite element model adopts two primary cross-section types, including circular steel tube sections for the roof space truss and rectangular reinforced concrete sections for the lower frame columns. The section types were detailed in Fig. 2. The primary load-bearing circular steel tubes of the space truss adopt a representative cross-section with an outer diameter of 219 mm and a wall thickness (t) of 12 mm [22]. The members and bolted spherical joints of the roof space truss adopt Q235B steel. The reinforced concrete beams and columns of the lower frame adopt concrete with a strength grade of C35. The longitudinal and transverse reinforcement adopts HRB400-grade hot-rolled ribbed steel bars. The rectangular reinforced concrete (RC) columns of the lower frame are defined with a concrete cover (c) of 35 mm for durability and fire resistance. These members are interconnected via prefabricated bolted spherical joints, which ensure reliable force transfer and high global stiffness. The space truss is supported by RC frame columns through corbels positioned at a top elevation of 20.6 m. To manage environmental and

dynamic effects, rubber bearings are integrated at each support node. These bearings are physically designed to provide rotational capacity and horizontal deformation allowance, accommodating displacements from temperature fluctuations and seismic actions.

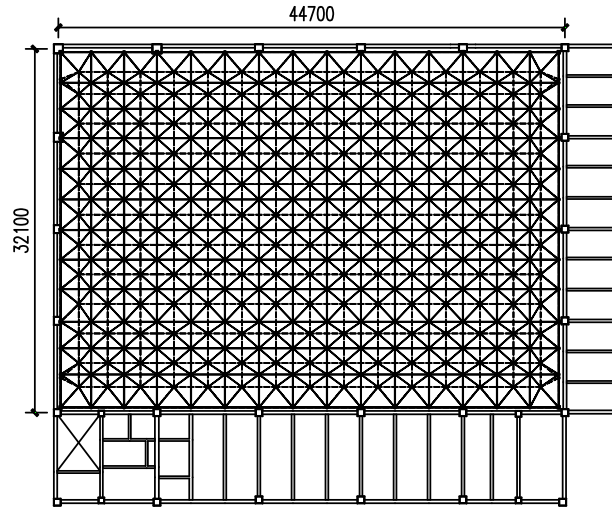


Figure 1: Plan layout of the roof truss (/mm).

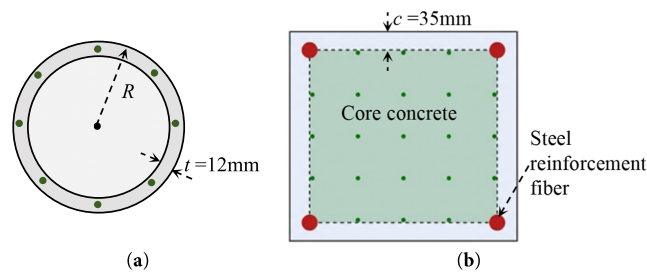


Figure 2: The distribution of integration points in beam element cross-sections. (a) Circular steel tube, (b) rectangular reinforced concrete beams/columns.

4 Structural Model

To evaluate the seismic performance of the sports training building, a hierarchical multi-platform analysis framework was implemented. Preliminary structural modeling and section optimization were performed using 3D3S 2023 and YJK 6.0.0 to ensure code compliance. Subsequently, a baseline dynamic verification was conducted in MIDAS Gen 2022. A key methodological innovation of this study is the implementation of an automated model-conversion workflow between the engineering design environment (YJK) and the high-fidelity analysis environment (ABAQUS 6.14.4/Explicit). As illustrated in Fig. 3, rather than utilizing error-prone manual modeling, a dedicated interface was employed to map the verified structural topology, section properties, and material assignments directly into the ABAQUS environment. This integrated approach ensures the geometric fidelity of the complex spatial grid and significantly enhances computational efficiency for large-scale indeterminate systems.

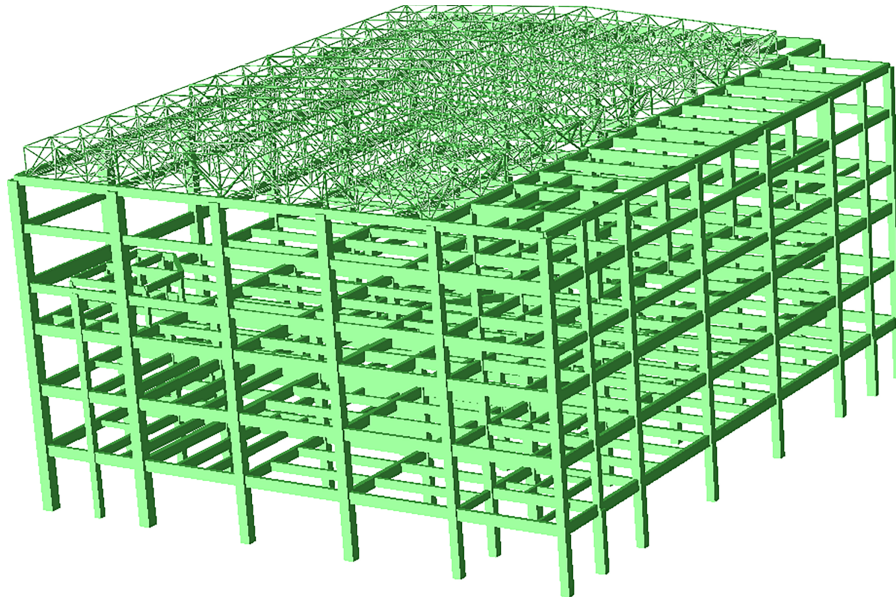


Figure 3: ABAQUS finite element model of the sports training building (/mm).

The structural components are discretized using three-dimensional B31 beam elements, based on Timoshenko beam theory, which accounts for transverse shear deformations. To capture the progression of material nonlinearity through the member depth, a fiber section discretization scheme is implemented. Unlike traditional empirical plastic hinge models, the fiber-based approach explicitly evaluates sectional internal forces by integrating the uniaxial stress-strain responses of individual fibers across the cross-section (Fig. 2). This enables the model to accurately simulate the migration of the neutral axis and the complex interaction between axial force and biaxial bending (P - M - M interaction). Specifically, Fig. 2a illustrates the circular steel tube section where fibers follow the wall thickness and circumferential directions, while Fig. 2b presents the rectangular RC section, distinguishing the concrete cover, core concrete, and longitudinal reinforcement fibers to account for confinement effects.

The reliability of the dynamic elastoplastic analysis hinges on the selection of robust material constitutive laws, as detailed in Fig. 4. The RC frame columns are simulated using the Concrete Damaged Plasticity (CDP) model, which incorporates a non associated flow rule and scalar damage variables to characterize irreversible stiffness degradation and energy dissipation. As illustrated in Fig. 4a, the uniaxial behavior of concrete is defined by critical stress and strain parameters. Here, $f_{c,r}$ and $\epsilon_{c,r}$ represent the representative peak compressive stress and its corresponding peak strain, respectively, while $\epsilon_{c,u}$ denotes the ultimate compressive strain. Similarly, $f_{t,r}$ and $\epsilon_{t,r}$ characterize the peak tensile strength and the initial cracking strain. This formulation allows for the precise tracking of the transition from the elastic phase to the strain softening region, a method that has been extensively validated against experimental data for reinforced concrete members [23]. For the Q235B steel truss members and HRB400 reinforcement, a Kinematic Hardening Model is employed, as shown in Fig. 4b. This model is superior for cyclic seismic analysis as it accurately simulates the Bauschinger effect, representing the evolution of the yield surface during plastic strain reversals. The constitutive curve is defined by the yield strength f_y at the initial yield strain ϵ_0 followed by a hardening phase until reaching the ultimate strain ϵ_u . This approach is essential for tracking cumulative plastic damage in the roof system and has been proven effective in simulating the dynamic stability of spatial grid structures [24].

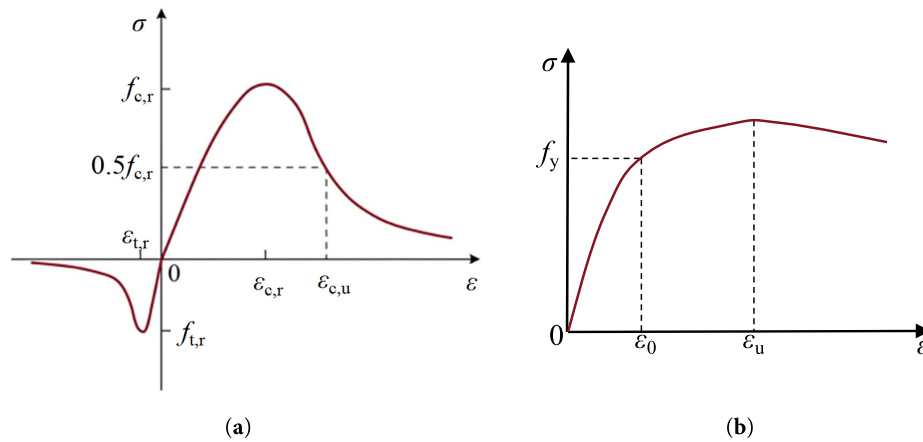


Figure 4: material constitutive laws of concrete and steel. (a) Concrete damaged plasticity mode, (b) kinematic hardening model for steel.

The boundary conditions and loading protocols are defined to reflect the physical constraints of the integrated system. The column bases are modeled as fully fixed at the foundation level, justified by the high rotational stiffness of the pile foundation. The critical interface between the roof truss and the RC corbels is illustrated in Fig. 5, where the space grid is supported by standard elastomeric rubber bearing pads. It should be emphasized that these bearings are not utilized as seismic isolation devices; rather, their primary function is to accommodate thermal deformations and release parasitic bending moments at the nodes. Accordingly, this connection is modeled using beam-element constraints to represent an ideal hinged boundary condition. Specifically, the translational degrees of freedom (U_x , U_y , U_z) are coupled to transfer vertical loads and horizontal shear, while all rotational degrees of freedom (U_{Rx} , U_{Ry} , U_{Rz}) are completely released. The analysis initializes with the gravity load representative value. In accordance with GB 50011-2010, the snow load contribution is taken as 0.225 kN/m^2 . This precise load initialization ensures that seismic excitations are applied to a pre-stressed structural state that accurately reflects the mass distribution during the design-level seismic event.

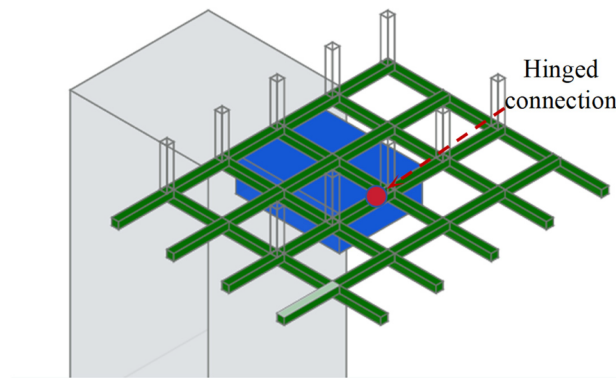


Figure 5: Connection between the roof truss and column corbels.

The dynamic equations of motion are solved using the Central Difference Scheme within the Abaqus/Explicit solver. This numerical integration algorithm is specifically justified for this highly indeterminate system due to its superior convergence stability. By utilizing a diagonal mass matrix, the explicit procedure avoids the computationally intensive inversion of the global stiffness matrix required by implicit

solvers. Furthermore, the explicit approach is inherently robust when dealing with high-frequency responses and localized instabilities, such as member buckling or concrete crushing, providing a stable numerical path for evaluating the overall progressive collapse resistance of the structure.

5 Selection of Ground Motion Records

The seismic response of the structural system depends strongly on the characteristics of the input ground motions, including spectral properties, duration, and peak ground acceleration. The selection of appropriate ground motion records therefore constitutes a critical prerequisite for the accuracy of time-history analysis results and the seismic safety evaluation of the structure [25,26]. The Code for Seismic Design of Buildings (GB 50011-2010) [27] requires the selected ground motions to satisfy statistical compatibility criteria. The average seismic influence coefficient curve of the selected ground motions shall differ from the seismic influence coefficient curve adopted in the modal response spectrum method by no more than 20% at the periods corresponding to the dominant vibration modes of the structure, which typically include the first several modes contributing most significantly to the base shear.

In strict accordance with these principles, an ensemble comprising two natural ground motion records (T1 and T2) and one artificial record (R1) was selected from the built-in seismic wave library of the YJK software. The selected records satisfy the code requirements and strictly match the specific site soil category and the design earthquake group of the project. Fig. 6 presents a comparison between the design response spectrum specified by the code and the mean response spectrum of the three selected ground motions. The two curves show excellent agreement in the vicinity of the fundamental period of the structure, and the difference at the dominant modal periods remains within 20%. Although the inset indicates a localized deviation of approximately 19%, this specific spectral dip reflects the inherent stochastic variability of the natural records rather than a deficiency in input energy. To further quantitatively verify the statistical compatibility of this ensemble, structural base shears obtained from the Time-History Analysis (THA) were compared with those from the Modal Response Spectrum Method (RSM). Under individual excitations, the minimum base shears in the *X* and *Y* directions reached 80.7% and 68.6% of the RSM results, respectively, successfully satisfying the 65% lower-bound code limit. Moreover, the average base shears of the three records accounted for 99.2% (*X*-direction) and 91.3% (*Y*-direction) of the RSM results, well exceeding the 80% requirement. These rigorous validations confirm that the selected seismic inputs possess appropriate frequency content and energy characteristics for the reliable seismic evaluation of the structure.

To ensure compatibility with the design spectrum across different seismic hazard levels, the acceleration histories were amplitude-scaled to match the target Peak Ground Acceleration (PGA), in accordance with Article 5.1.2 of the Code for Seismic Design of Buildings. For the elastic time-history analysis under frequent earthquakes (minor earthquakes), the PGA was scaled to 70 cm/s^2 . For the elastoplastic time-history analysis under rare earthquakes (major earthquakes), the PGA was scaled to 400 cm/s^2 . The analysis applied the ground motion input in three dimensions to capture the spatial variability of seismic action. In the global coordinate system, the *X*-axis and *Y*-axis are defined as the longitudinal and transverse directions of the structural plan, respectively, while the *Z*-axis represents the vertical direction. The code-recommended PGA ratios were 1:0.85 for the two horizontal directions (*X* and *Y*) and 0.65 for the vertical direction (*Z*) relative to the larger horizontal component. Two input cases were considered: Case 1 (*X*:*Y*:*Z* = 1:0.85:0.65) and Case 2 (*X*:*Y*:*Z* = 0.85:1:0.65).

Due to the variable depth of the space truss and the spatial coupling between the upper grid and lower RC frame, the structure lacks a single absolute weak axis. Exchanging the major horizontal components ensures a rigorous envelope approach, capturing the pronounced cross-directional redistribution of base shear. This multidimensional input enabled a comprehensive assessment of the structural response under

complex seismic loading, guaranteeing that the most unfavorable internal forces in all localized members were safely evaluated regardless of the earthquake's incidence angle.

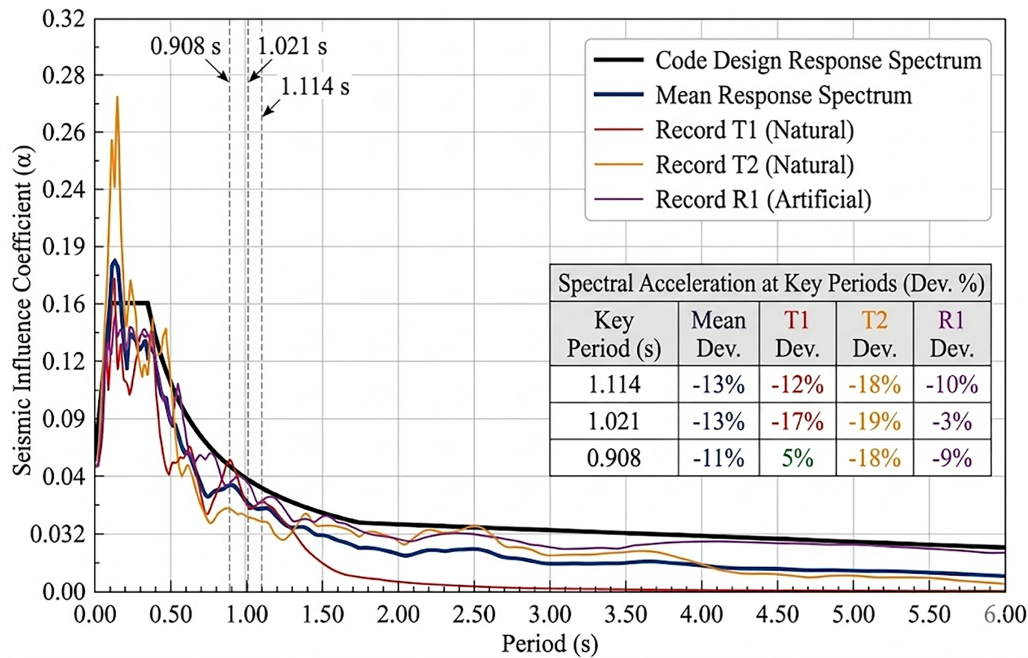


Figure 6: Comparison chart of seismic influence coefficient curves: code spectrum vs. response spectra.

6 Elastic Seismic Response Analysis

6.1 Modal Response Spectrum Analysis

To establish a dynamic baseline for the sports training building, the modal response spectrum method (MRSA) was implemented using Midas Gen 2022. The analysis evaluated the structural seismic effects under frequent earthquake (minor earthquake) conditions. To capture the complex dynamic behavior of the hybrid system, the analysis incorporated torsional coupling effects and included a sufficient number of vibration modes to ensure that the effective mass participation factor strictly exceeded 90%. The resulting base shear forces are summarized in [Table 1](#), where the off diagonal values (211 kN) objectively reflect the orthogonal shears induced by this torsional coupling. The primary base shear force in the transverse (Y) direction slightly exceeds that in the longitudinal (X) direction, reaching a maximum of 2414 kN. These static equivalent values serve as a critical benchmark for validating the subsequent time-history analyses.

Table 1: Base shear forces from modal response spectrum analysis.

Direction	F_x/kN	F_y/kN
X-direction	2113	211
Y-direction	211	2414

To quantitatively validate the effectiveness of the time-history inputs, the ETHA base shears ([Table 2](#)) were compared against the MRSA baseline ([Table 1](#)). Under individual seismic excitations, the minimum base shears in the X and Y directions reached 80.7% and 68.6% of the MRSA results, respectively, successfully satisfying the 65% lower-bound limit specified by the design code. Furthermore, the average base shears of

the three records in the X and Y directions accounted for 99.2% and 91.3% of the MRSA results, respectively, well exceeding the 80% requirement.

Table 2: Base shear under minor earthquake action.

Ground Motion Records	Acceleration Ratio	F_x/kN	F_y/kN
T1	1:0.85:0.65	2012	1924
	0.85:1:0.65	1706	2291
T2	1:0.85:0.65	2001	1655
	0.85:1:0.65	1741	1880
R1	1:0.85:0.65	2278	1903
	0.85:1:0.65	1777	2442
Maximum Average Base Shear		2097	2204

While direct experimental testing of the full-scale sports training building is physically unfeasible, the reliability of the overall numerical framework is established through a two-step verification process. First, the aforementioned 99.2% and 91.3% base shear matches confirm the accuracy of the baseline dynamic characteristics and global mass-stiffness distribution prior to the nonlinear stage. Second, for the subsequent severe-earthquake stage, the adopted fiber-based discretization and constitutive models (Concrete Damaged Plasticity and Kinematic Hardening) have been extensively benchmarked against experimental data in existing literature [28,29]. This strictly complies with the provisions of the Code for Seismic Design of Buildings (GB 50011-2010) and guarantees a reliable foundation for further analysis.

6.2 Elastic Time-History Analysis Results

Elastic time-history analysis was subsequently performed using YJK 6.0.0. Three site-compatible ground motions (two natural records, T1 and T2, and one artificial record, R1) were selected, with their peak ground accelerations (PGA) amplitude-scaled to 70 cm/s^2 to represent frequent earthquake conditions. To account for spatial multidimensional effects, two input cases with orthogonal acceleration ratios were considered: Case 1 ($X:Y:Z = 1:0.85:0.65$) and Case 2 ($X:Y:Z = 0.85:1:0.65$). The base shear results extracted from the ETHA are summarized in Table 2.

A comparative analysis of the ETHA results reveals distinct dynamic characteristics of this hybrid structure. First, a pronounced broadband amplification effect is observed. The base shears induced by the artificial wave (R1) comprehensively envelop those of the natural waves (T1 and T2). This is attributed to the broadband frequency characteristics of the artificial wave, which more readily excite the densely distributed higher-order modes typical of complex frame-space grid systems [30]. Second, the analysis captures a strong spatial synergistic coupling. Altering the major component of the orthogonal seismic excitations triggers a significant cross-directional redistribution of the base shear, indicating complex coupled in-plane and out-of-plane deformations driving internal force transfers between the flexible roof and rigid substructure [31].

Finally, the global lateral stiffness was evaluated. As presented in Table 3, the maximum elastic inter-story drift ratio occurs in the Y -direction under the artificial wave R1, reaching $1/605$. This deformation strictly satisfies the $1/550$ elastic limit mandated by the code, guaranteeing that the structure remains in a fully safe working state under frequent earthquakes. Notably, the high limit utilization rate (approximately 90.9%) indicates a highly optimized layout of the lateral stiffness system. This optimal design prevents the excessive

waste of materials and the undesirable amplification of seismic inertial forces associated with over-stiffening, achieving an excellent balance between structural safety and economic efficiency.

Table 3: Maximum inter-story drift angle under minor earthquake action.

Ground Motion Records	Direction	Acceleration Ratio	$\theta_{\max}$
T1	X-direction	1:0.85:0.65	1/703
	Y-direction	0.85:1:0.65	1/665
T2	X-direction	1:0.85:0.65	1/693
	Y-direction	0.85:1:0.65	1/755
R1	X-direction	1:0.85:0.65	1/631
	Y-direction	0.85:1:0.65	1/605

7 Dynamic Elastoplastic Time-History Analysis

7.1 Base Shear and Inter-Story Drift Angle

ABAQUS 6.14.4 software performed dynamic elastoplastic time-history analysis of the structure under rare earthquake (major earthquake) conditions. The analysis used the same ground motions as in the minor earthquake study (T1, T2, R1). In accordance with Table 5.1.2-2 of the Code for Seismic Design of Buildings, the peak ground acceleration was scaled to 400 cm/s^2 for three-dimensional input. By processing the field variable output from the ABAQUS results, the maximum base shear and maximum inter-story drift angle under rare earthquake action were extracted, as summarized in [Table 4](#).

Table 4: Maximum base shear and maximum inter-story drift angle under rare earthquake action.

Ground Motion Records	Acceleration Ratio	F_x/kN	F_y/kN	$\theta_{\max}$
T1	1:0.85:0.65	6659	6221	1/182
	0.85:1:0.65	5842	6875	1/171
T2	1:0.85:0.65	6328	5734	1/193
	0.85:1:0.65	5237	6133	1/178
R1	1:0.85:0.65	7335	6322	1/167
	0.85:1:0.65	6231	7636	1/159

As shown in [Table 4](#), under the most unfavorable load case (R1 wave), the maximum base shear is 7636 kN, and the maximum elastoplastic inter-story drift ratio is 1/159. By cross-comparing with the elastic calculation results under frequent earthquakes (70 cm/s^2), significant nonlinear response characteristics can be observed: the peak input acceleration of the rare earthquake increases to 571.4% ($400/70$) of that of the frequent earthquake, whereas the base shear of the structure only increases to 312.7% ($7636/2442$), and the inter-story drift ratio increases to 380.5% ($(1/159)/(1/605)$). The disproportionate growth of internal forces and deformations provides direct quantitative evidence that the overall structure has entered the plastic energy dissipation stage. Through stiffness degradation and plastic deformation, the structure effectively dissipates a large amount of seismic input energy. Meanwhile, the extreme value of 1/159 accounts for only 31.4% of the 1/50 collapse limit specified by the code. It indicates that the system still possesses sufficient

lateral drift control capacity after plastic damage occurs, aligning with current performance-based seismic design theories [10,31].

7.2 Analysis of Roof Truss Yielding

Fig. 7 illustrates the Von Mises stress contour of the roof space grid members at the moment of peak response under the R1 wave. Under severe multidimensional earthquakes, the vast majority of the roof members maintain an elastic state with stresses remaining below 235 MPa. However, highly localized yielding is observed, with maximum stresses reaching 270.7 MPa and successfully entering the strain-hardening stage.

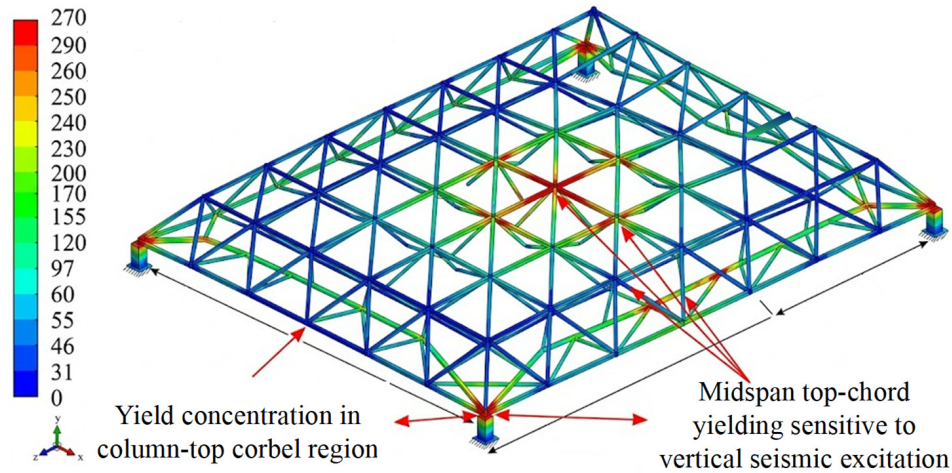


Figure 7: Von Mises stress distribution in roof truss members at a specific time under R1 wave action (/MPa).

Spatial distribution analysis indicates that this yielding is concentrated at two types of critical mechanical nodes. First, at the corbel support interfaces, the connections between the flexible spatial grid and the rigid RC columns experience immense horizontal seismic shear transfer, where the abrupt stiffness mutation triggers a pronounced spatial whiplash effect [11]. Second, yielding occurs at the mid-span top chords, which directly reflect the dynamic response sensitivity of the long-span system to vertical seismic excitations. Instead of brittle failure, this localized dynamic yielding represents a managed hierarchical damage evolution. In the context of performance based seismic design [1], these yielding mid span members effectively function as structural fuses under severe vertical earthquakes. By entering the strain hardening stage, they dissipate a substantial portion of the seismic energy through plastic work, thereby dampening the dynamic demand before it can jeopardize the primary vertical supports. Because this maximum stress (270.7 MPa) accounts for only 73.1% of the ultimate tensile strength of Q235 steel, the highly localized yielding does not sever the global load transfer path. This verifies that the highly indeterminate space grid retains robust internal force redistribution capacity to effectively resist progressive vertical collapse [2].

7.3 Column Yielding Analysis

As the primary load-bearing substructure, the performance of the RC columns under rare earthquakes dictates the global collapse resistance. Severe yielding or crushing of these columns would inevitably lead to an irreversible loss of vertical bearing capacity. By extracting the field variable outputs from the Concrete Damaged Plasticity (CDP) model, the maximum compressive strains of the critical ground-floor concrete columns were obtained, as summarized in Table 5.

Table 5: Maximum strain in columns under major earthquake action.

Ground Motion Records	Acceleration Ratio	Maximum Column Strain
T1	1:0.85:0.65	0.0009
	0.85:1:0.65	0.0012
T2	1:0.85:0.65	0.0012
	0.85:1:0.65	0.0015
R1	1:0.85:0.65	0.0015
	0.85:1:0.65	0.0019

Under the most severe load case (R1 wave), the extreme compressive strain reaches 0.0019. Mapping this value onto the concrete constitutive curve reveals that it mobilizes 95.0% of the peak compressive strain (0.002) for C35 concrete. This numerical characteristic embodies profound physical significance: the concrete in the extreme compressive zone has fully mobilized its ultimate strength and entered the plateau phase of plastic deformation, demonstrating an extremely high material utilization rate. Crucially, because the strain remains strictly below the peak value, the core concrete avoids entering the strain softening descending branch. This maintenance of the concrete compressive strain at 0.0019 embodies a critical indicator of seismic resilience. As established by Bruneau et al. [28], structural resilience is defined by the ability of a system to mitigate hazards and recover functionality. Physically, the vertical load transfer path remains elastic and uncrushed, implying that the damage is largely repairable. From a dynamic damage perspective, while the columns experience expected tensile damage and microcracking, which is an energy dissipation mechanism permitted by design codes under rare earthquakes, their core compressive bearing capacity remains uncompromised [29,30]. Ultimately, this hierarchical damage evolution demonstrates that the structure possesses the necessary redundancy and post earthquake residual capacity to fulfill recovery objectives of high performance seismic design [31].

8 Design Check of Corbels and Truss Supports

To ensure the reliable realization of the aforementioned hierarchical damage mechanism, the structural integrity of the connection hubs between the roof and the columns, specifically the RC corbels and the steel truss supports, must be strictly guaranteed. Historical reconnaissance following earthquakes has demonstrated that the support zones of spatial grids often represent critical vulnerabilities. These connections may experience premature brittle failure through the tearing of base plates, shearing of anchor bolts, weld fracture, or the sliding of rubber bearings. Consequently, to prevent such catastrophic local damage and prevent progressive global instability, comprehensive design verification were conducted on the corbels and supports utilizing the enveloping reaction forces extracted from the dynamic elastoplastic time history analysis.

8.1 Design Check of Corbels

The critical interface between the flexible roof and the rigid supporting frame is located at the top of the column corbels at an elevation of 20.600 m. Based on the field variable outputs from the ABAQUS severe earthquake simulations, the most unfavorable internal joint reaction forces were extracted. The maximum design reaction forces transmitted to the corbels reached 681 kN in the longitudinal direction, 327 kN in the transverse direction, and 222 kN in the vertical direction. This specific force distribution, where the horizontal dynamic shear significantly exceeds the vertical gravity and seismic reactions, highlights a loading state heavily dominated by shear induced by the spatial whiplash effect at the structural boundary. It reveals

that for hybrid structures with high stiffness contrast, the stiffness mutation at the structural interface is the primary driver of joint vulnerability. Consequently, the safety of the connection hub cannot be guaranteed by conventional gravity centric design alone. This necessitates a performance based verification that prioritizes bidirectional shear resistance and sliding stability to prevent the premature unseating of the roof grid under extreme multidimensional excitations.

To withstand these extreme shear transfers, the corbels were rigorously checked using specialized structural design suites compliant with the Code for Design of Concrete Structures (GB 50010-2010). The theoretical component level evaluations focused on three primary aspects: diagonal section crack resistance under combined bidirectional horizontal and vertical forces, normal section flexural capacity treating the corbel as an eccentrically compressed member, and the configuration of the horizontal stirrup area required to resist dynamic shear and confine the core concrete. The verification results confirm that the geometric dimensions and reinforcement detailing of the corbels possess sufficient ultimate bearing capacity under rare earthquakes, effectively preventing progressive instability induced by shear failure at the joints. As detailed in the design schematic (Fig. 7), the RC corbel extends 600 mm horizontally from the 700 mm wide main column. The vertical height of the upper straight section is 800 mm, supported by a lower beveled section with a 600 mm vertical projection. Furthermore, Fig. 8 illustrates the critical reinforcement configurations essential for resisting the aforementioned loads, including the layout of the top longitudinal flexural rebars, the dense distribution of horizontal stirrups, and strict adherence to the required anchorage length. These comprehensive geometric and detailing parameters guarantee the load transfer safety of the connection hub.

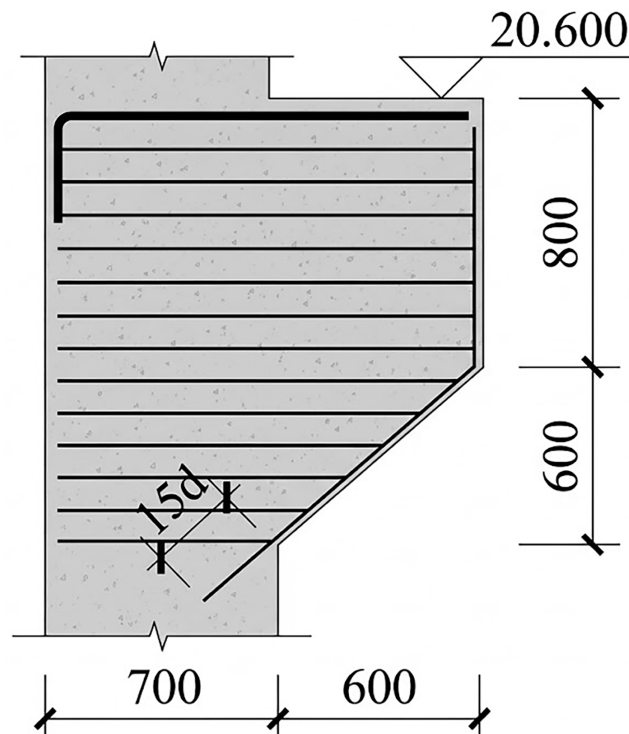


Figure 8: Corbel design schematic/mm.

8.2 Truss Support Design Check

According to the Technical Specification for Space Grid Structures (JGJ 7-2010), standard design procedures typically mandate vertical seismic checks for peripherally supported spatial grids. However,

this conventional code requirement may be insufficiently conservative for complex hybrid structures where horizontal seismic forces generate severe tension and shear demands on the boundary supports. Addressing this gap, this study utilized the 3D3S design platform to perform a comprehensive multidirectional design verification of the truss supports. This check was driven directly by the spatial support reactions, encompassing both vertical forces and bidirectional horizontal shears, obtained from the three dimensional ABAQUS elastoplastic time history analysis. The rigorous verification encompassed the local bearing capacity of the concrete beneath the base plates, the flexural stiffness and strength limits of the steel base plates, the geometric length and tensile strength of the critical weld connections between the supports and embedded parts, the stress capacities of the anchor bolts under combined tension and shear coupling, and the sliding stability of the elastomeric rubber bearings under peak horizontal forces.

Following this comprehensive evaluation, the designed truss supports demonstrated sufficient safety reserves to withstand the complex spatial multidimensional stresses under all considered rare earthquake scenarios, effectively eliminating the risk of connection failure and roof unseating.

9 Conclusion

In this study, a refined collaborative modeling and dynamic evaluation framework was implemented to investigate the seismic performance of a long span sports training building. By integrating the engineering design environment of YJK with the high-fidelity analysis capabilities of ABAQUS, systematic multidimensional elastic and elastoplastic time history analyses were conducted. The primary findings and contributions are summarized as follows:

- (1) The results validated that the proposed cross software modeling workflow achieves geometric and mechanical fidelity during data migration. By utilizing a dedicated data interface, the structural topology and section properties were directly converted from YJK into ABAQUS INP files. This methodology effectively eliminated the discretization errors inherent in manual reconstruction, ensuring that the critical material assignments and fiber based discretization for both steel and reinforced concrete members were preserved without loss.
- (2) The elastic response analysis revealed that the frame-space grid hybrid structure is highly sensitive to the frequency content of ground motions. Under frequent earthquakes, the artificial wave R1 induced a maximum base shear of 2442 kN and an inter story drift of 1/605, both of which enveloped the responses from natural waves. This broadband amplification effect is attributed to the densely distributed higher order modes of the long span roof. The analysis confirms that the structure possesses an optimized lateral stiffness layout with a 90.9% utilization rate of the code mandated elastic drift limit, achieving a precise balance between structural safety and material economy.
- (3) The dynamic elastoplastic simulations elucidated a hierarchical damage evolution mechanism under severe earthquakes. Although the structure entered the nonlinear stage, the maximum drift ratio of 1/159 remained significantly below the collapse threshold of 1/50. Localized yielding at 270.7 MPa was observed in the mid span top chords and corbel supports, which effectively acted as structural fuses to dissipate seismic energy. Crucially, the maximum compressive strain in the RC columns reached 0.0019, which mobilized 95% of the concrete compressive strength without entering the strain softening region. This demonstrates that the vertical load transfer path remained stable, successfully preventing brittle progressive collapse.
- (4) The study addressed the vulnerability of connection hubs by performing performance based design checks using dynamic ultimate reaction forces. The analysis identified a shear dominated loading state at the supports, where the horizontal dynamic shear reached 681 kN, nearly three times the vertical reaction force of 222 kN. By overcoming the limitations of conventional codes that prioritize

vertical verification, this study implemented supplementary shear and sliding checks for the corbels and truss supports. The results guarantee that the critical load transfer hubs can withstand the complex multidimensional stresses induced by the spatial whiplash effect.

In summary, although the quantitative findings are specific to the investigated building, the numerical modeling framework is highly generalizable. Based on fundamental material mechanics, the integrated fiber based methodology can be readily extended to evaluate the nonlinear behavior and collapse mechanisms of various complex structures, such as large span domes, airport terminals, and hybrid high rise structures. Furthermore, the established dynamic analysis framework is inherently adaptable to diverse seismic conditions globally by substituting site specific ground motion inputs. Through this physics informed approach, the structural resilience can be quantified, providing a robust reference for the performance based design of similar long span composite systems.

Acknowledgement: Not applicable.

Funding Statement: The authors received no specific funding for this study.

Author Contributions: The authors confirm contribution to the paper as follows: Hongtao Yue: Conceptualization, Methodology, Validation, Formal analysis, Investigation, Writing—original draft, Writing—review & editing; Xiang Yu: Conceptualization, Methodology, Software, Verification, Investigation, Supervision, Writing—review & editing; Chenghu Wang: Verification, Visualization, Writing—review & editing; Xiaona Kou: Visualization, Writing—review & editing; Zhongyuan Liu: Verification, Visualization, Writing—review & editing; Manli Ou: Visualization, Writing—review & editing. All authors reviewed and approved the final version of the manuscript.

Availability of Data and Materials: The data supporting the findings of this study are available from the corresponding author upon reasonable request.

Ethics Approval: Not applicable.

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

References

1. Lin H. Elastoplastic dynamic analysis and damage evaluation of reinforced concrete structures based on time histories. *Buildings*. 2025;15(6):971. doi:10.3390/buildings15060971.
2. Pelucco S, Doan VS, Vincenzi S, Preti M. Seismic performance and modelling of reinforced concrete frames infilled with sliding joint ductile infills. *Procedia Struct Integr*. 2026;78(1):591–8. doi:10.1016/j.prostr.2025.12.076.
3. Liang Q, Zhao C, Hu J. A new elastoplastic time-history analysis method for frame structures. *Adv Civ Eng*. 2020;2020(1):8818187. doi:10.1155/2020/8818187.
4. Malekizadeh M, Fanaie N, Pirasteh AA. Probabilistic seismic assessment of steel gabled frames with web-tapered members under near-fault ground motions along strike-normal and strike-parallel components. *Structures*. 2021;34(1):4142–57. doi:10.1016/j.istruc.2021.10.007.
5. Málaga-Chuquitaype C. Machine learning in structural design: an opinionated review. *Front Built Environ*. 2022;8:815717. doi:10.3389/fbuil.2022.815717.
6. Bai FY, Liu JQ, Li L. Elasto-plastic analysis of large span reticulated shell structure under multi-support excitations. *Adv Mater Res*. 2012;446–449:54–8. doi:10.4028/www.scientific.net/amr.446-449.54.
7. Nair D, Ichihashi K, Terazawa Y, Sitler B, Takeuchi T. Higher mode effects of multistorey substructures on the seismic response of double-layered steel gridshell domes. *Eng Struct*. 2021;243(7):112677. doi:10.1016/j.engstruct.2021.112677.
8. Zhao Z, Li X, Liu R, Xue S, Liu T, Fan Q, et al. Influence of considering substructures on seismic performances of suspen-dome structure. *Proc Inst Civ Eng Struct Build*. 2024;177(1):53–75. doi:10.1680/jstbu.21.00187.

9. Tian LM, Wei JP, Bai R, Zhang XC. Dynamic behavior of progressive collapse of long-span single-layer spatial grid structures. *J Perform Constr Facil.* 2021;35(2):04021002. doi:10.1061/(asce)cf.1943-5509.0001566.
10. Guo S, Xu D, Shang K, Yang S, Wang D, Li G, et al. Analysis of progressive collapse of a super-long span latticed steel arch structure. *J Theor Appl Mech.* 2023;2023:103–17. doi:10.15632/jtam-pl/157311.
11. Zhou L, Huang Y, Chen B, Hu S, Zhang C. Seismic behavior of large-span open-web floors consisting of I-shaped steel chords and steel tube webs. *Adv Civ Eng.* 2020;2020(1):8872447. doi:10.1155/2020/8872447.
12. Liu Z, Ma X, Lv J. Seismic response of a large-span steel truss arch bridge under nonuniform near-fault ground motions. *Buildings.* 2024;14(8):2308. doi:10.3390/buildings14082308.
13. Xu W, Du D, Wang S, Liu W, Li W. Shaking table tests on the multi-dimensional seismic response of long-span grid structure with base-isolation. *Eng Struct.* 2019;201(8):109802. doi:10.1016/j.engstruct.2019.109802.
14. Galvis FA, Smith-Pardo JP. Axial load biaxial moment interaction (PMM) diagrams for shallow foundations: design aids, experimental verification, and examples. *Eng Struct.* 2020;213(2):110582. doi:10.1016/j.engstruct.2020.110582.
15. Chae Y, Ricles JM, Sause R. Large-scale real-time hybrid simulation of a three-story steel frame building with magneto-rheological dampers. *Earthq Eng Struct Dyn.* 2014;43(13):1915–33. doi:10.1002/eqe.2429.
16. Fang M, Wang J, Li H. An adaptive model order reduction method for nonlinear seismic analysis of civil structures based on the elastic-plastic states. *Int J Numer Methods Eng.* 2021;122(15):3985–4006. doi:10.1002/nme.6689.
17. Zhang T, Xu ZD, Huang XH, Dong YR, Shi QX. Seismic performance analysis of the two-dimensional isolation device for double-layer grid structure. *Int J Struct Stab Dyn.* 2024;24(5):2450051. doi:10.1142/s0219455424500512.
18. Pang R, Sun YY, Xu Z, Xu K, Cui J, Dang LJ. Shaking table test and numerical simulation of a precast frame-shear wall structure with innovative untopped precast concrete floors. *Eng Struct.* 2024;300(4):117162. doi:10.1016/j.engstruct.2023.117162.
19. GB 50223-2008. Standard for seismic fortification category of building construction. Beijing, China: China Architecture & Building Press; 2008. (In Chinese).
20. GB 55002-2021. General code for seismic design of buildings and municipal engineering. Beijing, China: China Architecture & Building Press; 2021. (In Chinese).
21. JGJ 7-2010. Technical specification for space frame structures. Beijing, China: China Architecture & Building Press; 2010. (In Chinese).
22. Amini SN, Rajput AS. Seismic performance of ductile and non ductile reinforced concrete columns under varied axial compression. *Struct Eng Mech.* 2024;91(5):427–41. doi:10.12989/sem.2024.91.5.427.
23. Lin Y, Chen Y, Zong Z, Bi K. A state-of-the-art review of bridges subjected to earthquake fault ruptures. *Eng Struct.* 2025;330(2):119889. doi:10.1016/j.engstruct.2025.119889.
24. Montuori R, Nastri E, Piluso V. Influence of the bracing scheme on seismic performances of MRF-EBF dual systems. *J Constr Steel Res.* 2017;132:179–90. doi:10.1016/j.jcsr.2017.01.018.
25. Wei JP, Tian LM, Hao JP. Parameter analysis of progressive collapse simulation of long-span spatial grid structures. *Int J Steel Struct.* 2019;19(6):1718–31. doi:10.1007/s13296-019-00241-3.
26. Tian LM, Wei JP, Huang QX, Ju JW. Collapse-resistant performance of long-span single-layer spatial grid structures subjected to equivalent sudden joint loads. *J Struct Eng.* 2021;147(1):04020309. doi:10.1061/(asce)st.1943-541x.0002904.
27. GB 50011-2010. Code for seismic design of buildings (2016 edition). Beijing, China: China Architecture & Building Press; 2010. (In Chinese).
28. Bruneau M, Chang SE, Eguchi RT, Lee GC, O'Rourke TD, Reinhorn AM, et al. A framework to quantitatively assess and enhance the seismic resilience of communities. *Earthq Spectra.* 2003;19(4):733–52. doi:10.1193/1.1623497.
29. Baloević G, Radnić J, Harapin A. Numerical dynamic tests of masonry-infilled RC frames. *Eng Struct.* 2013;50(1):43–55. doi:10.1016/j.engstruct.2012.11.034.
30. Alshaikh IMH, Abadel AA, Sennah K, Nehdi ML, Tuladhar R, Alamri M. Progressive collapse resistance of RC beam-slab substructures made with rubberized concrete. *Buildings.* 2022;12(10):1724. doi:10.3390/buildings12101724.
31. Cui Y, Yang X, Liu H, Yamada S. Seismic behavior of steel space truss connections to reinforced concrete supporting columns. *Adv Struct Eng.* 2022;25(8):1714–29. doi:10.1177/13694332211062331.