



Seismic response of a prefabricated substation equipment cabin with base isolation: Full-scale shaking table test and numerical analysis

Jiajun Du^{a,b}, Wei Wang^{a,b,1,*}, Jiachen Zhang^{a,b}, Xiaotian Liu^{a,b}, Theodoros L. Karavasilis^c

^a State Key Laboratory of Disaster Reduction in Civil Engineering & Department of Structural Engineering, Tongji University, Shanghai, 200092, China

^b Shanghai Engineering Research Center for Resilient Cities and Intelligent Disaster Mitigation, Tongji University, Shanghai, 200092, China

^c Department of Civil Engineering, University of Patras, Patras, 265000, Greece

ARTICLE INFO

Keywords:

Full-scale shaking table test
Experimental validation
Base isolation
Equipment response spectrum
Prefabricated substation equipment cabin
Seismic isolation bearing manufacturing error

ABSTRACT

This study explores a relatively under-investigated application of base-isolation to lightweight prefabricated steel substation cabins, characterized by a short isolation period and limited number of bearings, which are fundamentally different from conventional building-scale isolation systems. A bidirectional full-scale shaking table test was conducted to examine the seismic response of the prefabricated cabin structure and its internal equipment, verifying the effectiveness of the isolation strategy in controlling the response of the prefabricated cabin, particularly in reducing the equipment acceleration response. The finite element model developed in SAP2000 was validated by comparison with experimental data, confirming its validity for subsequent numerical analyses. Furthermore, the validated finite element model is used to construct an equipment acceleration response spectrum (EARS) tailored to prefabricated substation environments, offering a practical reference for equipment protection design in future modularized substations. A comparison of the EARS under both isolated and non-isolated configurations further confirmed and quantified the mitigation effect of the isolation strategy on the equipment's response. Additionally, the small footprint of the prefabricated cabin may amplify the impact of bearing manufacturing errors on its structural response; therefore, this paper evaluated the associated risks by analyzing structural differences across various possible errors and configurations. The analysis reveals that manufacturing errors indeed result in potential increases in the structural displacement response and peak bearing deformation, but the amplified responses remain within the permissible limits defined by the codes. Furthermore, the study also explored alternative isolation layer schemes to optimize structural performance and economic efficiency.

1. Introduction

In contemporary power-dependent societies, robust electrical networks constitute a vital foundation for sustaining critical societal functions spanning medical services, telecommunications, manufacturing sectors, and domestic electricity requirements. Substations, functioning as neural centers within power grids, perform indispensable roles through voltage regulation, energy allocation, and protective system operations - their reliable performance being intrinsically linked to overall network reliability [1]. This operational criticality has elevated seismic risk assessment to a key discipline within infrastructure resilience research. Empirical evidence from major earthquakes [2–6], including the 2008 Wenchuan earthquake [2,3] and the 2022 Luding earthquake [4] in China, the 2010/2011 Christchurch earthquake sequence [5], and the 2010 Chile earthquake [6], documented extensive damage to substation

equipment, large-scale power outages, and prolonged post-earthquake recovery, revealing that substation damage-induced power interruptions frequently precipitate cascading economic consequences and extended restoration periods. Such findings necessitate the strategic implementation of earthquake-resistant engineering solutions for critical power facilities located in seismically active regions, thereby ensuring continuous grid operations and mitigating potential socio-economic disruptions during tectonic events.

In view of this, enhancing the seismic resilience of substation facilities has attracted considerable attention in recent years. Zhu et al. [7] developed ground motion classification-adaptive fragility models for 500 kV transformers. Concurrently, Taghizadeh et al. [8] established an optimal seismic intensity measurement method for high voltage equipment, addressing both network-interconnected and standalone

* Corresponding author at: State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, Shanghai, 200092, China.

Email address: weiwang@tongji.edu.cn (W. Wang).

¹ Professor.

<https://doi.org/10.1016/j.istruc.2026.111328>

Received 6 November 2025; Received in revised form 1 February 2026; Accepted 5 February 2026

Available online 10 February 2026

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substation configurations. Beyond addressing the fragility of these facilities, engineering strategies for enhancing their seismic performance have attracted growing attention. Specifically, effective base isolation techniques have shown operational promise: Experimental validation by Li et al. [9] confirmed that dual friction pendulum systems significantly improve bushing-transformer dynamic responses, whereas the parametric study conducted by He et al. [10] quantified significant life cycle cost reductions in ultra high voltage bypass switches through optimized wire rope isolation deployment. These studies have primarily focused on critical facilities in conventional substations, where facilities are typically distributed outdoors. However, as electrical facilities have been progressively upgraded to intelligent and integrated configurations, indoor substations have gained increasing prevalence [11,12]. Relatively, indoor substations offer advantages such as improved land utilization, effective noise control, and better protection of internal equipment.

The seismic design of indoor substations typically involves applying the conventional building seismic design methodology to ensure that the main lateral resistance system remains elastic under the design-level earthquake, with additional measures to keep equipment responses within safe thresholds. Similar to its application in substation facilities, seismic isolation techniques—by reducing the transmission of ground seismic energy to the superstructure—have proven highly effective in meeting the design requirements of indoor substations, nuclear-industry structures, and similar critical industrial facilities. Moreover, prior research on base-isolated structural systems indicates that response mitigation may be sensitive to key isolator parameters [13]. In addition, when simplified nonlinear static evaluation procedures (e.g., pushover-based assessments) are adopted, the estimated demands can also be affected by modeling/analysis assumptions such as the selected lateral load pattern [14]. These observations further highlight the value of experimental validation and high-fidelity time-history analysis when extending isolation concepts to critical industrial facilities such as indoor substations. Congru et al. [15] conducted shaking table tests on a concrete indoor substation model equipped with a hybrid isolation system comprising lead-rubber bearings and viscous dampers. Chen et al. [16] investigated the effectiveness of partial isolation strategies in improving the seismic performance of existing fixed-base steel indoor substations. As for mitigating the seismic response of prefabricated industrial equipment structural frames, Liu et al. [17–19] explored multiple seismic isolation strategies, including shape memory alloy cable-restrained base isolation systems that provide self-centering capability [17], bearing-based hybrid isolation systems using linear natural rubber and lead rubber bearings to enhance hysteretic energy dissipation [18], and hybrid isolation systems combining elastomeric bearings with nonlinear fluid viscous dampers to achieve dual control of equipment acceleration and bearing deformation [19]. However, the indoor substations investigated in these studies have all been of conventional concrete or steel-frame construction, rather than the prefabricated cabin-type substations addressed in this study.

Prefabricated substation equipment cabin (PSEC) is generally designed, fabricated, and assembled using standardized modular units, offering significant advantages in rapid installation, low environmental impact, and high integration [20,21]. They constitute the primary form of next-generation intelligent substations promoted by the State Grid Corporation of China. Because the member layout, connection details and section sizes are largely standardized to satisfy modular manufacturing and functional/non-structural requirements of substations, the lateral flexibility of the superstructure and the feasible bearing geometry are both constrained, which may limit the achievable isolation-period elongation compared with building-scale isolation practice. Although the seismic behavior of some prefabricated structural systems under near- and far-field earthquakes has been investigated in recent years [22,23], current research on the PSEC [24] is very limited, and investigations into the effectiveness of seismic isolation

techniques for prefabricated cabins are lacking. A companion study by the authors reported the full-scale shaking-table test and numerical assessment of the same PSEC mock-up under fixed-base conditions, providing baseline observations on cabin response, equipment characteristics and design implications [25]. Building on that baseline, the present paper focuses on the base-isolated configuration and provides direct fixed-base versus isolated comparisons, including EARS-based evaluation and isolation-layer optimization under realistic displacement constraints. While base isolation is a mature technology, its application to PSECs presents unique challenges that have not been adequately addressed in existing literature. Firstly, the relatively small mass and stiff commercially available bearings result in a short-period isolation system, a scenario that deviates from conventional isolation design paradigms and warrants in-depth investigation into its efficacy and limitations. Secondly, the limited footprint of PSECs restricts the number of isolation bearings, potentially amplifying the structural sensitivity to manufacturing errors and inducing significant torsional responses—a critical issue rarely explored for small-scale isolated structures. Thirdly, a systematic methodology for evaluating the seismic demands on non-structural equipment within such isolated cabins is lacking.

Building on the preceding discussions, this paper aims to address the identified research gaps through a comprehensive program of full-scale shaking table testing and refined numerical analysis of a two-story PSEC with base isolation. The primary original contributions of this work are fourfold:

1. To provide full-scale experimental data quantifying the performance of a short-period isolation system tailored for a lightweight PSEC, with a focus on its efficacy in protecting internal equipment.
2. To introduce and validate the concept of Equipment Acceleration Response Spectra (EARS) as a design-oriented tool for precisely assessing equipment-specific seismic demands within isolated cabins.
3. To systematically quantify the sensitivity of the structural torsional response to realistic bearing manufacturing errors, a critical yet underexplored issue for small-scale structures with a limited number of isolation units.
4. To explore optimized isolation schemes that balance structural performance with economic efficiency, offering practical design alternatives.

To this end, the remainder of the paper is structured to logically develop these contributions. Section 2 details the full-scale shaking table test program, including the specimen, instrumentation, and loading protocols. Section 3 follows with the presentation and discussion of the shaking table test results. Subsequently, numerical analysis based on the experimentally validated finite element model is presented, with Section 4 focusing on the EARS of equipment in the PSEC. Additionally, Section 5 presents a detailed numerical investigation into the effects of bearing manufacturing errors, addressing the third contribution. Section 6 further explores the optimization of the seismic isolation layer scheme to obtain possible alternatives. Finally, the main findings are discussed in Section 7, and the key conclusions, study limitations, and directions for future research are summarized in Section 8.

2. Shaking table test of the PSEC with base isolation

2.1. Test model

The test structure is a full-scale model of a PSEC from an actual engineering project. It is designed for a seismic intensity of 8 (peak ground acceleration (PGA) corresponding to a 475-year return period, namely PGA_{475y} , is 0.3 g), with a site characteristic period of 0.65 s [26]. The model dimensions, as well as the connection points between and within modules, are shown in Fig. 1. In the test, the short direction

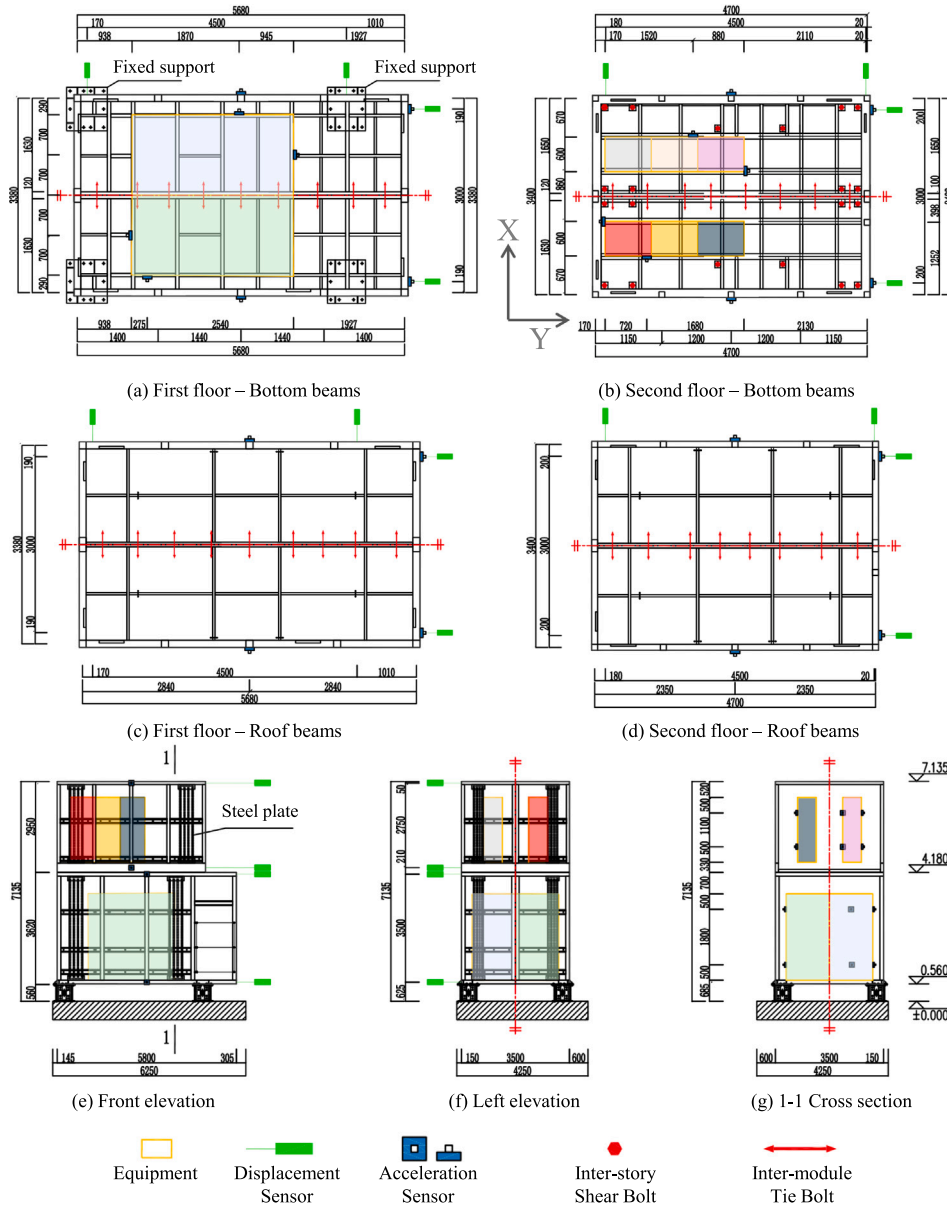


Fig. 1. The illustration of test model.

of the structure is defined as the X -direction, and the long direction as the Y -direction. The superstructure of the prefabricated cabin consists of a two-story modular steel structure (with story heights of 3.62 m and 2.95 m, respectively). Each story is composed of two modules that are symmetric about the Y -direction. The modules are connected both horizontally and vertically by friction-type high-strength bolts. The section sizes of structural members are listed in Table 1. Additionally, the structure includes lintels (horizontal short beams used to mount wall panels, made of 2.5 mm thick cold-formed C-shaped thin-walled sections, 100 mm in height and 70 mm in width) and lateral steel plates (320 mm wide in the X -direction, 440 mm wide in the Y -direction, 5 mm thick, each stiffened by four $40 \times 40 \times 3$ square steel tubes). The model is constructed using Q355 (yielding strength $f_y = 355$ MPa) structural steel. As indicated by the color blocks in Fig. 1, two pieces of switchgear cabinets are mounted on the first floor, each weighing approximately 2.8 tons, for a total of 9.8 tons. Six smaller pieces of panel cabinets are placed on the second floor, each weighing about 0.2 tons, totaling 4.5 tons. The superstructure is connected to the shaking table

via four bearings. The bearings can be configured for either fixed-base or base-isolated conditions by installing or removing rigid connecting plates fabricated from Q355 structural steel (as shown in Fig. 2). An overall view of the full-scale shaking table test setup for the PSEC is shown in Fig. 3. The detailed dimensions and installation layouts of the steel plates placed above and below the isolation bearings, as well as the plates used for fixed-base conversion, are also explicitly shown in Fig. 3. For the fixed-base configuration, the rigid connecting plates fully constrain the horizontal movements at the base, resulting in a fixed boundary condition in both horizontal directions. Vertical displacement remains restrained in both configurations to ensure stable axial load transfer. Rotational degrees of freedom at the isolation level are not explicitly restrained, allowing potential torsional response to develop under asymmetric bearing properties. The four bearings weigh approximately 1.6 tons in total, bringing the total weight of the model to around 16 tons. The fixed-base test results of this full-scale mock-up have been reported in detail in [25] and are used herein as the benchmark for evaluating the isolation effectiveness.

Table 1
Cross-section of the test model.

Name	Cross-sectional specification
Column of the 1st-floor module	□120×120×6
Column of the 2nd-floor module	□100×100×4
Lower beam beneath 1st-floor column	□120×120×8
Main beam on 1st floor	□120×60×4
Secondary beam on 1st floor	□40×40×3
Upper beam of the 1st-floor column	□120×120×6
Other top beams on 1st floor	□80×40×5
Lower beam beneath 2nd-floor column	□300×150×6
Transverse beam in Y-direction (2nd floor)	H100×100×6×8
Secondary beam on 2nd floor	□100×50×3
Upper beam of the 2nd-floor column	□100×100×4
Other top beams on 2nd floor	□80×40×5

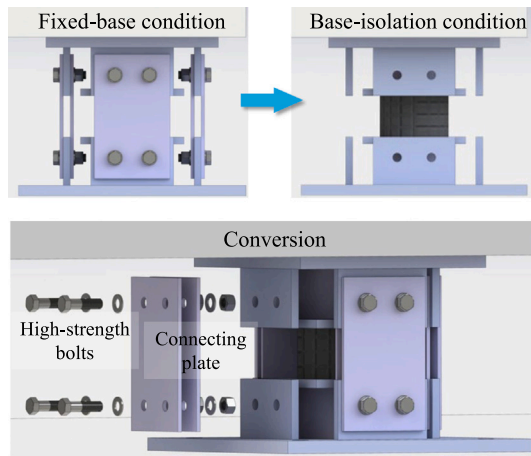


Fig. 2. Non-isolation and isolation bearing conversion.

2.2. Design of isolation system

The seismic isolation system is seismically designed in line with the current Chinese codes [26,27]. More specifically, under rare earthquake conditions, the horizontal displacement of the isolation layer must be limited to the lesser of 0.55 times the bearing diameter or 3.0 times the total rubber layer thickness. Considering both the scale of the test model and the commercially available bearing diameters (typically ≥ 200 mm), lead rubber (LR) bearings and linear natural rubber (LNR) bearings with a diameter of 200 mm were preliminarily selected. The former provides greater lateral stiffness and energy dissipation capacity. According to the aforementioned specification, the permissible horizontal displacement

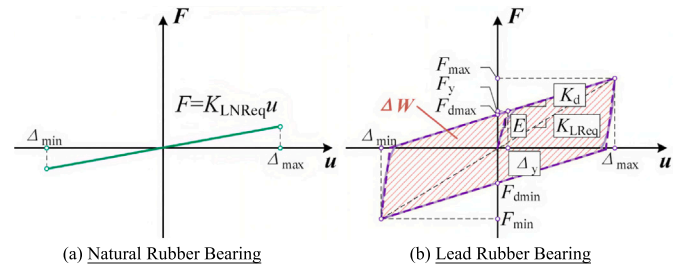


Fig. 4. Illustration of the dynamic behavior of LR and LNR bearings.

limit for these isolation bearings under rare earthquakes is 110 mm. By simplifying the calculation of the distinct period characteristics and damping ratios of the two isolation systems, the displacement response spectrum values can be determined using the design response spectrum for rare earthquakes specified in the Chinese codes [26,27]. Comparing these values with the stipulated displacement limit allows for a rational analysis of the feasibility of the isolation schemes. As illustrated in Fig. 4, the equivalent stiffness of the LR can be calculated using Eq. (2). The horizontal equivalent stiffness of each 200 mm diameter LNR can be referred to in previous research and is taken as 200 N/mm [18]. Then, the fundamental periods of the two isolation systems were determined using Eq. (2) to be approximately 0.73 s and 0.89 s for the LR and LNR systems, respectively.

$$K_{LReq} = \frac{F_{yLR} + (\Delta_0 - \Delta_y) (F_{yLR} / \Delta_y) r}{\Delta_0} \quad (1)$$

$$T = 2\pi \sqrt{\frac{M}{K_{eq}}} \quad (2)$$

where F_{yLR} is the total strength of the LR bearings, taken as 60 kN according to data from the manufacturer; Δ_y is the design yield displacement of the LR bearing, taken as 8 mm [18]; Δ_0 is the design peak deformation of the isolators, taken as 90 mm; r is the design post yield stiffness ratio of the LR bearings, taken as 1/13 [18]; M is the total mass supported by the isolation bearings; K_{eq} is the total horizontal equivalent stiffness of the LR or LNR bearings.

Based on the experimentally measured damping ratios reported for LR and LNR bearings [18,19], and considering the need for conservative assumptions at the design stage, the damping ratios were assumed to be 20% for the LR system and 8% for the LNR system. Subsequently, the displacement design spectra for rare earthquakes, corresponding to damping ratios of 20% and 8%, are illustrated in the Fig. 5. It can be observed that the displacement response spectrum value for the LR is

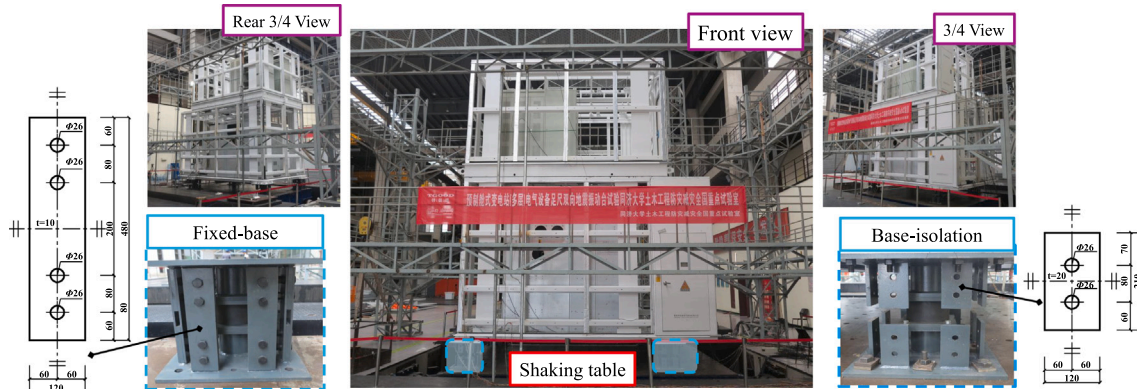


Fig. 3. Overall view of the full-scale prefabricated substation equipment cabin on the shaking table.

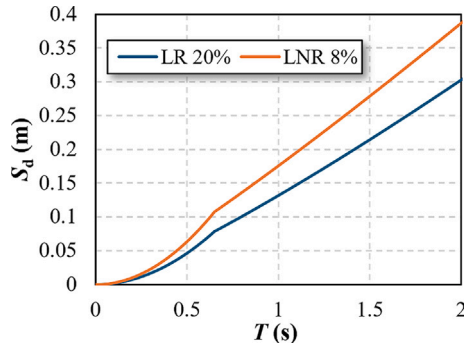


Fig. 5. Displacement design response spectrum corresponding to rare earthquakes.

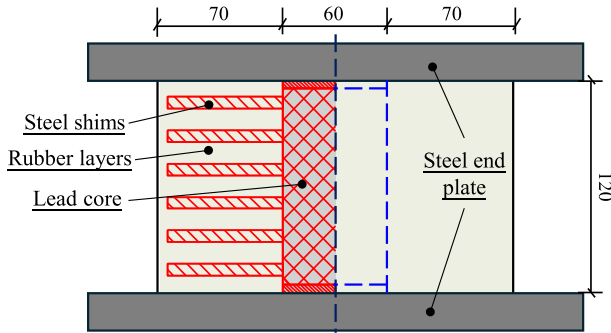


Fig. 6. Schematic illustration of the selected LR bearing.

approximately 90 mm (matching the design displacement, Δ_0), while that for the LNR is about 154 mm. Given that the allowable horizontal displacement limit for the isolation layer is 110 mm, the LR bearing isolation scheme was deemed more reasonable and was therefore selected for subsequent experimental investigation. A sketch of the selected LR bearing is provided in Fig. 6.

2.3. Mechanical performance test of the isolation bearing

In accordance with Chinese code JG/T 118–2018 [28], one isolation bearing was selected prior to the shaking table tests to undergo a quasi-static horizontal hysteresis test (Fig. 7) to investigate its mechanical properties. The test was conducted under a constant vertical compressive stress of 12 MPa. Loading was applied at a frequency of 0.05 Hz with displacement amplitudes of 25 mm, 50 mm, 70 mm, and 90 mm, respectively. For each displacement amplitude, three fully reversed loading cycles were applied to obtain the corresponding hysteresis loop (Fig. 8).

The horizontal hysteretic behavior of the bearing can be represented using a bilinear model as shown in Fig. 9. Key mechanical parameters were determined from the third stable hysteresis loop for each amplitude and calculated according to Eqs. (3)–(7) [18]. The resulting mechanical properties are presented in Table 2. As indicated in Table 2, the equivalent stiffness of the isolation bearing decreased from 562.1 N/mm at a displacement amplitude of 25 mm to 292.4 N/mm at 90 mm, corresponding to an overall reduction of approximately 48%. The equivalent damping ratio also showed a moderate decreasing trend, reducing from 0.31 to 0.28 over the same displacement range. In contrast, the yield force increased monotonically from 7.7 kN to 13.2 kN, representing an increase of about 71%. These variations quantitatively reflect the characteristic nonlinear behavior of LR bearings, in which stiffness degradation and strength mobilization become more pronounced with increasing deformation demand. Notably, previous studies have shown that the interaction between vertical load and horizontal deformation



Fig. 7. Horizontal quasi-static hysteresis test on the isolation bearing.

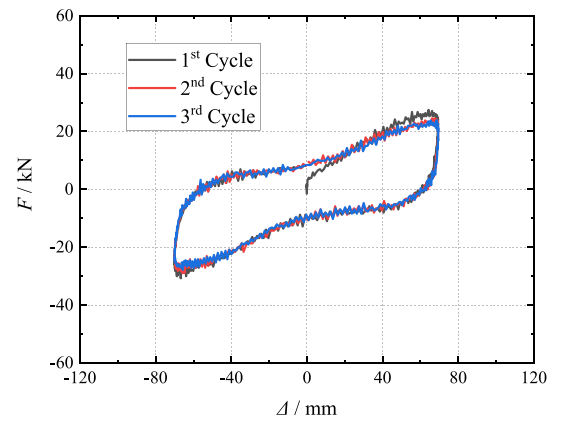


Fig. 8. Horizontal load-displacement curve of the isolation bearing.

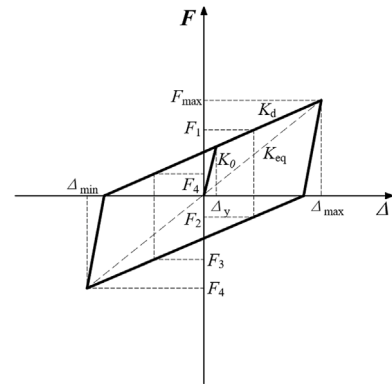


Fig. 9. Horizontal mechanical properties of the isolation bearing.

may influence the stability and stiffness of elastomeric bearings, particularly at large shear strains or near critical load conditions [29–31]. In the present study, the vertical load applied to the isolation bearings was kept constant throughout the tests, and the measured horizontal displacements remained well below the critical instability threshold. Therefore, the coupling effect between vertical load and horizontal response is considered secondary, and the observed responses are primarily governed by the horizontal shear behavior of the bearings.

$$K_{eq} = \frac{F_{max} - F_{min}}{\Delta_{max} - \Delta_{min}} \quad (3)$$

Table 2
Mechanical parameters under different loading amplitudes.

Loading Amplitude (mm)	Horizontal Equivalent Stiffness (N/mm)	Equivalent Damping Ratio	Post-Yield Stiffness (N/mm)	Yield Force (kN)
25	562.1	0.31	213.3	7.7
50	375.7	0.33	174.4	9.9
70	320.4	0.31	172.0	11.9
90	292.4	0.28	165.5	13.2

$$\zeta = \frac{1}{4\pi} \frac{\Delta W}{S_{\Delta}} \quad (4)$$

$$S_{\Delta} = \frac{1}{2} F_{\max} \Delta_{\max} \quad (5)$$

$$K_d = \frac{1}{2} \left[\frac{F_1 - F_4}{(\Delta_{\max} - \Delta_{\min})/2} + \frac{F_2 - F_3}{(\Delta_{\max} - \Delta_{\min})/2} \right] \quad (6)$$

$$Q_d = \frac{1}{2} \left[\frac{\Delta_{\min} F_1 - \Delta_{\max} F_4}{\Delta_{\min} - \Delta_{\max}} - \frac{\Delta_{\min} F_2 - \Delta_{\max} F_3}{\Delta_{\min} - \Delta_{\max}} \right] \quad (7)$$

where $F_{\max}$ and $F_{\min}$ are the maximum and minimum horizontal loads of the hysteretic curve, respectively; $\Delta_{\max}$ and $\Delta_{\min}$ are the maximum and minimum horizontal displacements, respectively; ΔW is the area enclosed by the hysteretic curve; F_1 and F_2 , F_3 and F_4 are the horizontal forces at half of the maximum and minimum deformations of the bearing, respectively.

2.4. Test setup and instrumentation

The shaking table test was carried out in the multifunctional shaking table laboratory at Tongji University's Jiading Campus. The test platform features a table measuring 6.0 meters by 4.0 meters, capable of bearing a maximum load of 30 tons. It offers a rated acceleration capacity of $\pm 1.5 g$ and a maximum displacement amplitude of ± 500 mm, operating effectively across a frequency band ranging from 0.1 to 50 Hz.

To monitor structural accelerations, one accelerometer was placed in both the X and Y directions at the centroidal height of the upper and lower beams on the first and second stories. To capture the dynamic response of the mounted equipment, two additional accelerometers were located at opposite corners along the equipment height on each floor. Furthermore, Linear Variable Differential Transformers (LVDTs) were installed at the same elevation as the accelerometers to track lateral and rotational displacements. Two sensors were arranged in each principal direction to accurately capture both translational and torsional floor movements. The configuration of all acceleration and displacement sensors used during the shaking table tests is illustrated in Fig. 1.

2.5. Input seismic ground motions and test cases

Bidirectional ground excitations were applied to the test model. According to Chinese code GB 50,011–2010 [26], three ground motion records were selected and scaled, including one artificial ground motion and two natural ground motions. Besides, the detailed information and time histories are presented in Table 3 and Fig. 10. Their response spectra, along with the design spectrum, are shown in Fig. 11.

The shaking table tests for the base-isolated prefabricated cabin were conducted using bidirectional horizontal seismic input ($X:Y = 1:0.85$ or $0.85:1$), and divided into five loading stages based on PGA levels: 0.1 g , 0.2 g , 0.3 g , 0.4 g , and 0.51 g . Here, 0.51 g corresponds to the maximum PGA associated with the rare-level earthquakes under the seismic design intensity of 8 ($PGA_{475y} = 0.3 g$). The loading cases are detailed in Table 4. Except for case 1, a white noise test ($X:Y = 1:1$, $PGA = 0.05 g$) was performed after each stage to assess changes in the dynamic characteristics of the structure. For the non-isolated configuration, the test cases were the same as those listed in Table 4, except for the omission of the 0.4 g PGA stage.

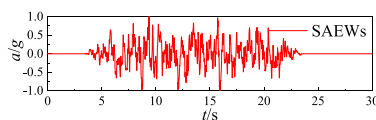
3. Shaking table test results

3.1. Dynamic characteristic

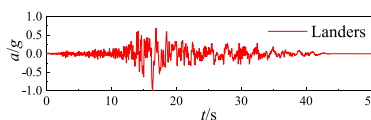
The fundamental dynamic characteristics of the structure, namely the natural period and damping ratio, were obtained from the white noise excitation case. Specifically, the absolute acceleration time history of the shaking table was used as the input, and the absolute acceleration at the top of the second story was used as the output to construct the transfer function and generate the amplitude-frequency diagram. The structural natural frequency was then identified from the peaks of the curve, from which the natural period was derived. The damping ratio was further determined using the half-power bandwidth method. The resulting natural periods and damping ratios of both fixed and isolated base conditions are summarized in Table 5. It can be observed that the isolation measures extended the fundamental period of the fixed-base structure by approximately 2.4 times in both principal directions. Given this relatively limited period elongation, the mitigation of spectral accelerations is primarily attributed to the significant increase in the equivalent damping ratio (from 6.8%–8.7% to 12.7%–15.5%), which highlights the dominant role of the damping reduction coefficient in the seismic protection of such lightweight prefabricated structures. This mechanism is further evidenced in Fig. 12, where the elastic response spectra are marked with vertical dashed lines representing the identified periods of both fixed-base and isolated configurations. However, the relatively small mass of the prefabricated cabin structure limits the effectiveness of even the smallest commercially available LR bearings (with a diameter of 200 mm). In light of the potential excessive displacement identified during preliminary analysis when using LNR bearings

Table 3
Summary of ground motion information.

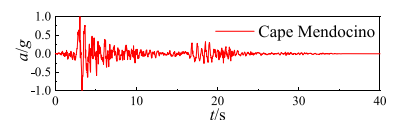
Ground Motion	Type	Year	Magnitude	Station	PGA (g)	PGV (cm/s)	Time Step (s)	Duration (s)
SAEWs	Artificial	—	—	—	0.30	46.5	0.01	20.00
Landers	Far-field	1992	7.3	Yermo Fire	0.24	52.0	0.02	44.00
Cape Mendocino	Near-fault	1992	7.0	Petrolia	0.63	82.1	0.02	36.00



(a) Record of SAEWs.



(b) Record of Landers.



(c) Record of Cape Mendocino.

Fig. 10. Acceleration history of ground motions.

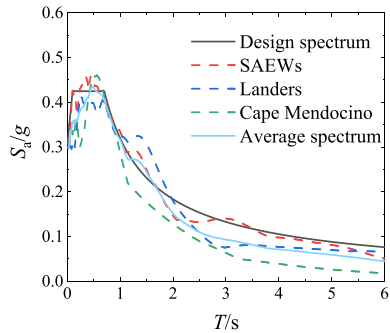


Fig. 11. Acceleration spectra of selected ground motions.

Table 4
Test cases.

Loading Condition	PGA /g	Input Excitation	Amplitude/g X Direction	Y Direction	Duration /s
1	0.1	White noise	0.050	0.050	100.0
2		SAEWs	0.100	0.085	20.0
3		Landers	0.100	0.085	44.0
4		Cape mendocino	0.100	0.085	36.0
5		SAEWs	0.085	0.100	20.0
6		Landers	0.085	0.100	44.0
7		Cape mendocino	0.085	0.100	36.0
8		White noise	0.050	0.050	100.0
9	0.2	SAEWs	0.200	0.170	20.0
10		Landers	0.200	0.170	44.0
11		Cape mendocino	0.200	0.170	36.0
12		SAEWs	0.170	0.200	20.0
13		Landers	0.170	0.200	44.0
14		Cape mendocino	0.170	0.200	36.0
15	0.3	White noise	0.050	0.050	100.0
16		SAEWs	0.300	0.255	20.0
17		Landers	0.300	0.255	44.0
18		Cape mendocino	0.300	0.255	36.0
19		SAEWs	0.255	0.300	20.0
20		Landers	0.255	0.300	44.0
21	0.4	Cape mendocino	0.255	0.300	36.0
22		White noise	0.050	0.050	100.0
23		SAEWs	0.400	0.340	20.0
24		Landers	0.400	0.340	44.0
25		Cape mendocino	0.400	0.340	36.0
26		SAEWs	0.340	0.400	20.0
27	0.51	Landers	0.340	0.400	44.0
28		Cape mendocino	0.340	0.400	36.0
29		White noise	0.340	0.400	100.0
30		SAEWs	0.510	0.434	20.0
31		Landers	0.510	0.434	44.0
32		Cape mendocino	0.510	0.434	36.0
33	0.51	SAEWs	0.434	0.510	20.0
34		Landers	0.434	0.510	44.0
35		Cape mendocino	0.434	0.510	36.0
36		White noise	0.050	0.050	100.0

Table 5
Structural natural period and damping ratio.

	Isolated-base		Fixed-base	
	X direction	Y direction	X direction	Y direction
Period/s	0.488	0.399	0.202	0.165
Damping ratio/%	15.5	12.7	8.7	6.8

exclusively, numerical analyses based on an experimentally validated finite element model were further conducted in Section 6 to explore the feasibility of adopting a hybrid isolation system incorporating both LNR and LR bearings.

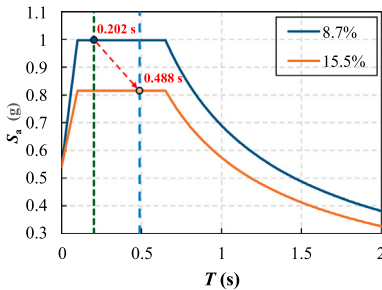


Fig. 12. Comparison of spectral accelerations reduction mechanisms based on test-identified periods and damping ratios.

3.2. Acceleration response

3.2.1. Acceleration of the structure

Fig. 13 illustrates the peak absolute acceleration responses at representative measurement points on the structure under various loading cases. It can be observed that, during the five loading levels corresponding to increasing PGA levels, the mean peak acceleration responses at the top of the first floor are approximately 0.25 g, 0.40 g, 0.55 g, 0.70 g, and 0.80 g, respectively, while those at the top of the second floor are approximately 0.30 g, 0.45 g, 0.70 g, 0.90 g, and 1.00 g. The corresponding acceleration amplification factors relative to the input ground motion range from about 1.0 to 2.0. In general, the measured acceleration responses exhibit a consistent increasing trend with rising PGA. Under cases with PGA less than 0.30 g, the peak acceleration at the second floor is close to that of the first floor, showing no clear evidence of increasing acceleration with structural height. This indicates that the isolation bearings function effectively, resulting in predominantly translational motion of the superstructure. At elevated PGA levels, the isolation bearings demonstrated an insufficient capacity to attenuate seismic energy transmission, which was manifested as a significant increase in the acceleration response on the second floor of the prefabricated cabin.

For completeness, detailed discussions on the fixed-base response trends can be found in [25]; the present study emphasizes the isolation-induced changes and the equipment-demand mitigation. Taking the Y-direction measurement point at the top of the second floor as a representative example, a comparison of the peak absolute acceleration responses between the isolated and non-isolated structures under various loading conditions is presented in Fig. 14. It is evident that the isolated cabin generally exhibits lower acceleration responses at this location compared to the non-isolated cabin, with the reduction ratio ranging from approximately 5% to 30% across different cases.

3.2.2. Acceleration of equipment

Providing structural support for electrical equipment is the most critical function of the prefabricated cabin structure. Therefore, conducting experiments to obtain the actual acceleration responses of the equipment under seismic excitation is essential for evaluating the protective performance of the structural system. During the tests, no significant damage to the equipment was observed under all loading cases. Fig. 15 illustrates the time history comparison of absolute acceleration between the measurement point atop the first-floor structure and the upper section of the equipment located on the same level. It is evident that the equipment tends to exhibit higher-frequency and larger-amplitude vibrations compared to the main structure. This behavior is primarily attributed to the relatively flexible frames and panels of the equipment, which lead to amplified local responses at the sensor locations.

Fig. 16 illustrates the peak acceleration responses of the equipment under different test cases. It can be observed that the peak acceleration responses in the two directions for the equipment on the first floor differ significantly. This discrepancy occurs primarily because the Y-direction sensors on the first-floor equipment were installed on the

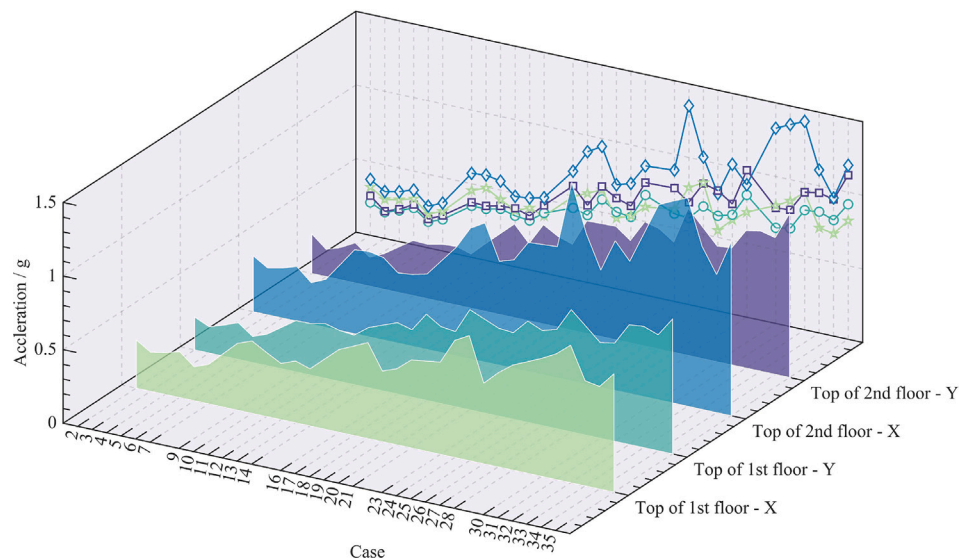


Fig. 13. Peak absolute acceleration response of the structure.

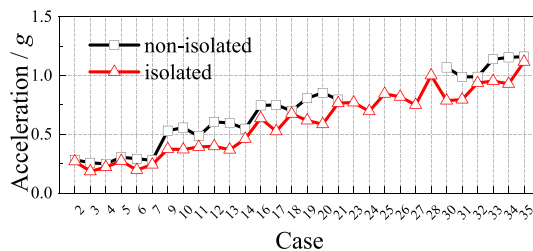


Fig. 14. Comparison of peak absolute acceleration response of the structure.

relatively weak cabinet skin, making them more susceptible to local vibrations at the mounting locations. In terms of magnitude, except for the Y -direction measurement point on the upper part of the first-floor equipment—which is notably affected by panel vibrations—the acceleration amplification factors of the remaining points relative to the input ground motion generally fall within the range of 1.0 to 3.0. This is comparable to the structural amplification factor range of 1.0 to 2.0, further indicating that the isolation bearings were partially but notably effective in isolating the superstructure and internal equipment from seismic energy.

Fig. 17 presents a comparison of the peak absolute acceleration responses at the Y -direction measurement point on the upper part of the second-floor equipment under various loading cases. It can be observed that, in most cases, the isolated structure exhibits lower peak acceleration responses compared to the non-isolated structure, with reduction

ratios ranging from approximately 10% to 50%. In most conditions, the reduction exceeds 20%, indicating that the isolation measures are effective in controlling the equipment acceleration response.

3.3. Displacement response of the structure

The inter-story drift ratio measured during the tests remained small throughout. Even under the condition with a PGA of 0.51 g , the maximum drift ratio was only 1/400, indicating that the structural response was well controlled within the code-specified limit of 1/100. Additionally, during the tests, no significant torsional response was observed in the superstructure. However, the recorded displacement time-history curves indicated some discrepancies in the displacement responses at measuring points located at the same height and direction under certain high-PGA conditions, suggesting the occurrence of certain torsional motion. For instance, Fig. 18 shows the absolute displacement time histories at the top of the first and second floors under case 30. For the first-floor roof, the peak displacement responses at two X -direction measurement points on the same side are 101.24 mm and 94.72 mm, respectively, with a difference of 6.52 mm. For the second-floor roof, the peak displacement responses at two Y -direction measurement points on the same side are 84.44 mm and 78.82 mm, respectively, with a difference of 9.62 mm. Thus, in Section 5, further investigation is conducted on the possible torsion caused by the manufacturing errors of seismic isolation bearings in such small-scale prefabricated cabin.

The primary objective of the prefabricated cabin structure is to ensure the functional safety of the electrical system; therefore, the control of equipment acceleration is the key focus of the seismic isolation

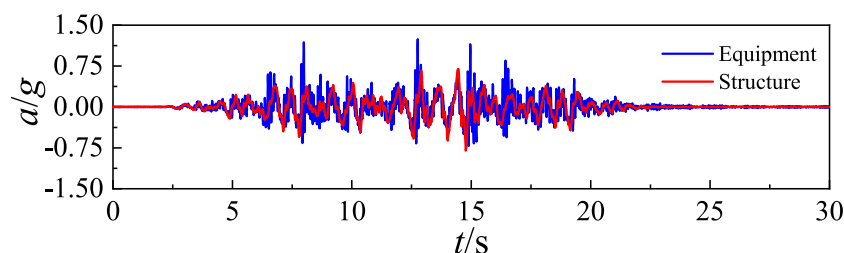


Fig. 15. The time history comparison of absolute acceleration between the measurement point atop the first-floor structure and the upper section of the equipment located on the same level.

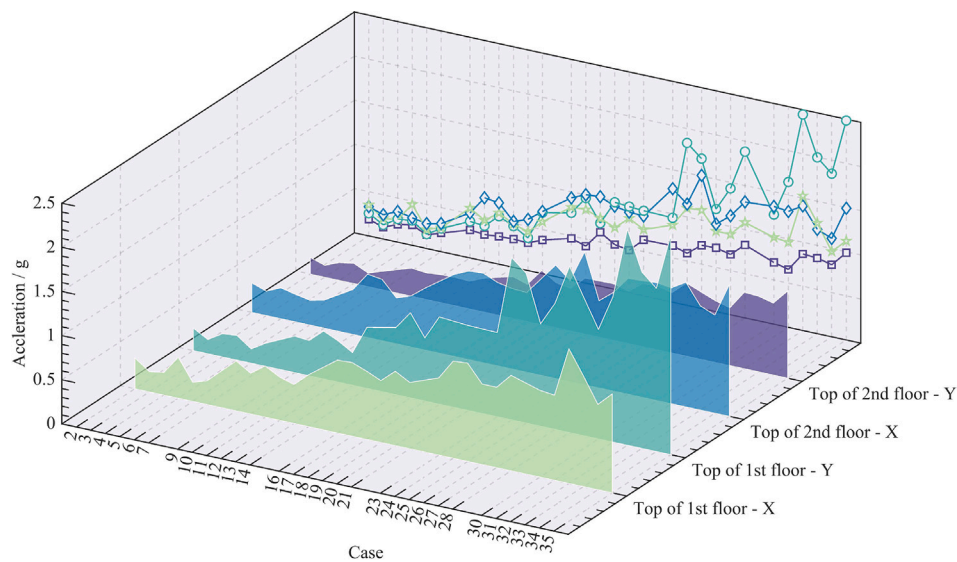


Fig. 16. Peak absolute acceleration response of the equipment.

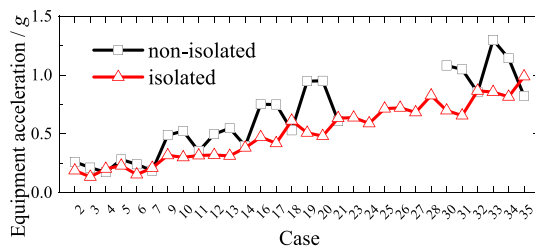


Fig. 17. Comparison of peak absolute acceleration response of the equipment.

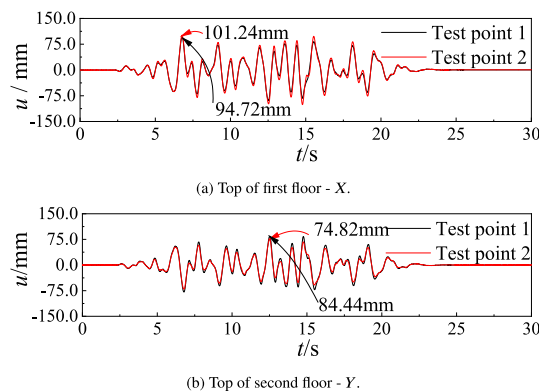


Fig. 18. Relative displacement time histories of the structure.

design. As shown in Table 5, the fundamental period of the isolated configuration is 0.488 s, which remains within the acceleration-sensitive region of the response spectrum. This relatively short isolated period is mainly attributed to the lightweight nature of the prefabricated cabin and the design constraints imposed by the safety and stability requirements of the isolation bearings. However, the introduction of the LR bearings significantly increases the damping ratio of the system, so the spectral acceleration under this damping ratio is significantly lower than the spectral acceleration when the fixed support has a damping of about 8%. Experimental results indicate that, although the period elongation effect provided by the seismic isolation bearings is limited, the equipment acceleration response is effectively

controlled, with reductions ranging from approximately 10% to 50%. This observation suggests that the seismic performance improvement of the isolated system is not governed solely by period shift. In the present system, the lead-core rubber bearings exhibit stable hysteretic behavior under cyclic loading, providing substantial energy dissipation and effectively increasing the equivalent damping of the isolation layer. Such enhanced damping, together with the filtering of high-frequency vibration components, contributes significantly to the reduction of acceleration transmission to the internal equipment. These results indicate that the isolation strategy adopted in this study functions primarily as a vibration mitigation mechanism tailored for lightweight equipment cabins, rather than as a conventional long-period base isolation system for building structures. In the next section, the mitigation effect of the isolation strategy on equipment acceleration responses will be further investigated through the analysis of equipment acceleration response spectra.

4. Equipment acceleration response spectra

Similar to floor response spectra [32], the equipment acceleration response spectra (EARS) presented here are also obtained by inputting the absolute acceleration time histories at a given floor into dynamic models of equipment with varying natural periods. For equipment of the same type, the mass is generally fixed, while the natural frequency can be adjusted by modifying the stiffness of the equipment cabinets. In this study, the finite element model of the prefabricated cabin was developed using SAP2000, as illustrated in Fig. 19. The beams and columns of the prefabricated cabin are idealized by 'frame' elements, whereas the lateral bracing steel plates are represented by 'thin-shell' elements. The seismic isolation bearings are simulated by 'Plastic Wen' model. All components in the test model are constructed from Q355 steel, which has a yield strength of 355 MPa. Beam-to-column connections are treated as rigid, while each equipment-to-structure interface—connected via a single friction-type high-strength bolt—is simulated as a hinged support by restricting all three translational degrees of freedom. The equipment is simplified as shell elements at this stage, with the mass distribution adjusted by altering the element thickness. The fundamental periods of the finite element models for the fixed-base system and the isolated system in the X- and Y-directions were 0.211 s, 0.152 s, 0.517 s, and 0.495 s, respectively, which align closely with the experimentally measured values. Furthermore, the time-history analyses also demonstrated good agreement between the finite element model predictions and the

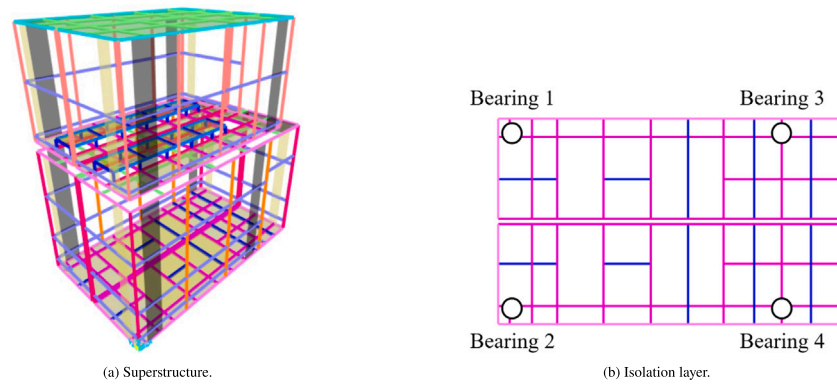


Fig. 19. Finite element model for PSEC with base isolation.

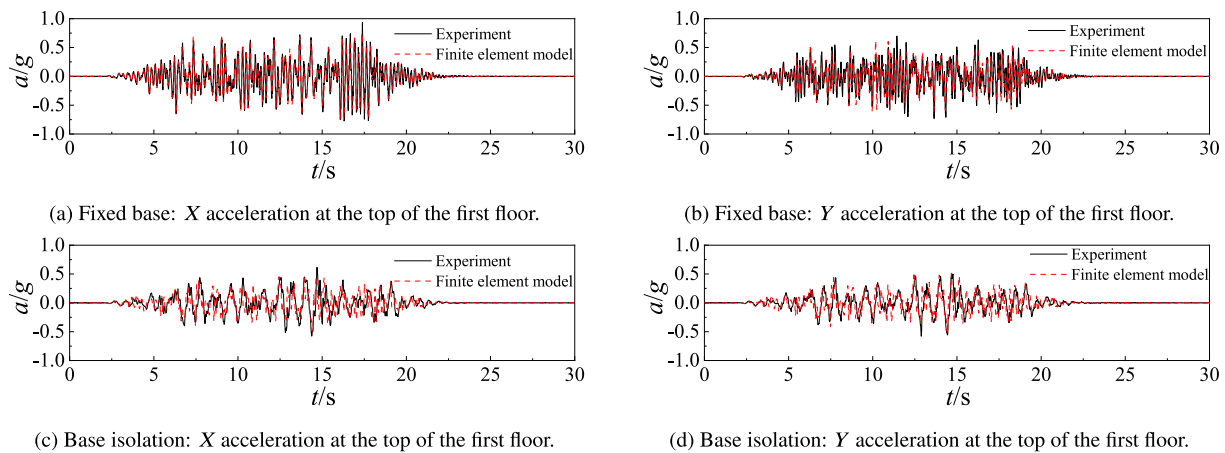


Fig. 20. Comparison of finite element simulation and experimental acceleration time history under case 30.

test results. As illustrated in Fig. 20, the numerical model accurately replicates the experimental acceleration time histories in terms of peak amplitudes, frequency content, and phase synchronization. The minor discrepancies in peak values (basically under 5%) confirm that the validated FE model provides a robust foundation for evaluating the seismic demands within the isolation system.

Building upon the validated finite element model, the equipment representation is further improved to derive the EARS. As depicted in Fig. 21, each piece of equipment is represented using a configuration consisting of two shell elements and four rectangular columns, ensuring that the total height aligns with the actual equipment dimensions. The equipment mass is assigned to the top and bottom surface elements, evenly divided in a 1:1 ratio so that each surface supports half of the total mass. Adjustments to the natural period of the equipment are made by modifying the cross-sectional sizes of the four columns. This modeling strategy is predicated on the fact that the lateral stiffness of substation equipment cabinets is fundamentally governed by their primary structural frames. By adjusting the column dimensions, the model effectively captures the global dynamic sensitivity of the equipment to floor motions across a targeted period range. It should be noted that while this simplified model does not account for localized effects such as high-frequency panel vibrations, it represents a strategic trade-off to ensure computational feasibility for extensive EARS analysis. Developing a fully-coupled, high-fidelity model for each cabinet type would require exhaustive experimental calibration and is often unnecessary for establishing general seismic design spectra. Thus, the adopted approach provides a reliable and efficient means to quantify the acceleration demands for equipment protection in modularized substations.

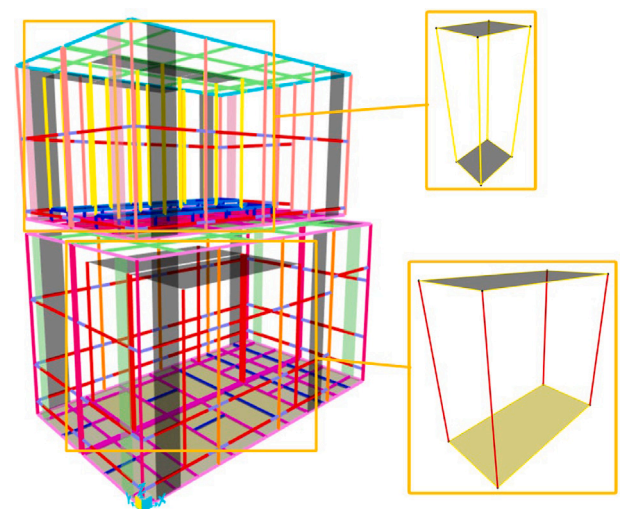


Fig. 21. Finite element model for EARS analysis.

The peak acceleration responses of the equipment corresponding to varying natural periods were then obtained through a series of time history analyses. Drawing upon previous studies [33–37], the natural period range for the equipment within the prefabricated cabin was defined as being between 0.05 s and 0.5 s, with an increment of 0.025 s

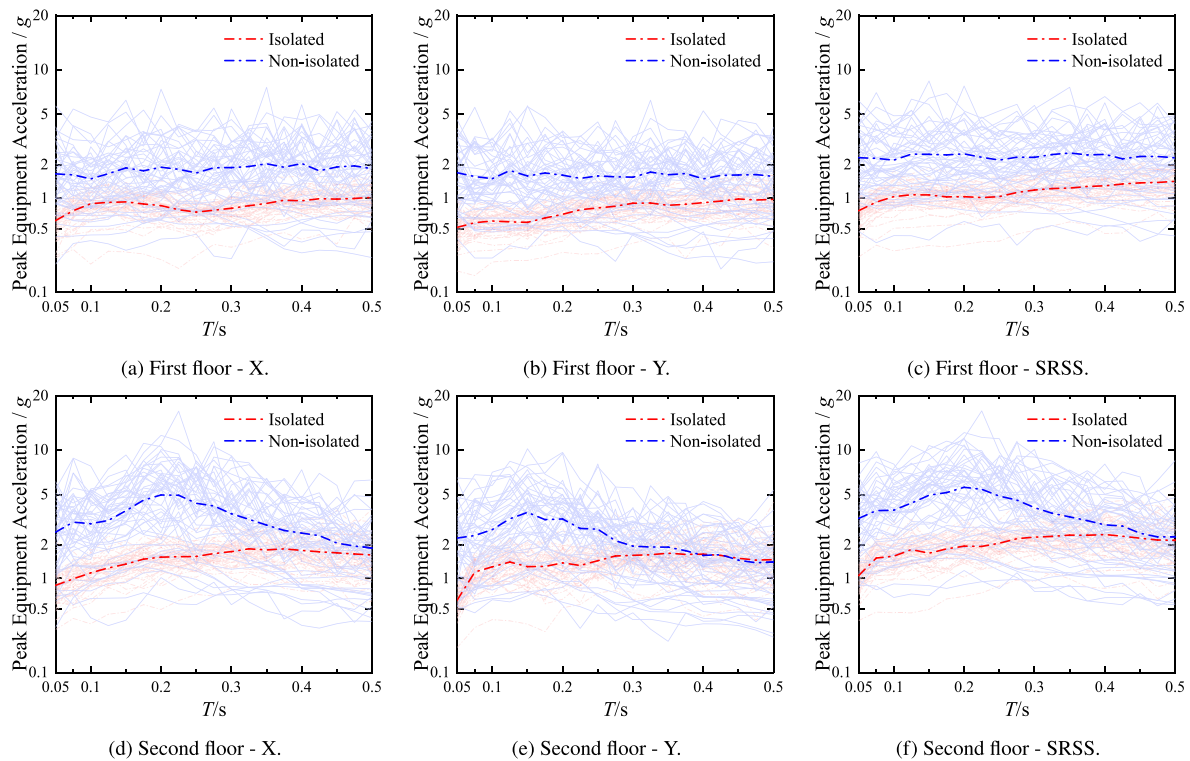


Fig. 22. Equipment acceleration response spectra.

used in the analyses. The input excitations comprised 50 earthquake records recommended by FEMA P695 [38], including 22 far-field and 28 near-field ground motions. These ground motions, scaled to a PGA of 0.51 g (corresponding to rare-level earthquakes with seismic intensity of 8 [26]) with an X:Y directional ratio of 1:0.85, were applied to the finite element model. The resulting peak acceleration curves for equipment located on the first and second floors, under both isolation and non-isolation conditions, are presented in Fig. 22.

It is observed that for the average Equipment Acceleration Response Spectra (EARS) under the non-isolated condition, the curves for the first-floor equipment in the X, Y, and biaxial square root of the sum of the squares (SRSS) directions exhibit certain degrees of fluctuations. The corresponding ranges are approximately 1.5–2.0 g for the X-direction, 1.5–1.8 g for the Y-direction, and 2.2–2.5 g for the SRSS combination. However, no significant increasing or decreasing trends are observed across the considered period range. For the second-floor equipment without isolation, the EARS curves generally display a single-peaked profile: the X-direction curve reaches a maximum value of 5.0 g at 0.2 s, the Y-direction curve peaks at 3.7 g at 0.15 s, and the SRSS curve attains a maximum of 5.6 g at 0.2 s.

Furthermore, under isolated conditions, the EARS curves for the first-floor equipment reach their respective minimum values of 0.6 g, 0.5 g, and 0.8 g at 0.05 s in the X, Y, and SRSS directions, respectively. Between 0.05 s and 0.25 s, these curves exhibit distinct behaviors: a gentle peak in the X-direction, a plateau followed by a monotonic increase in the Y-direction, and a monotonic increase leading to a plateau at approximately 1.1 g for the SRSS combination. Beyond 0.25 s, all curves demonstrate an overall increasing trend, with the SRSS combination reaching a maximum of 1.4 g. Similarly, for the second-floor equipment under isolated conditions, the X, Y, and SRSS EARS curves reach minimum values of 0.9 g, 0.6 g, and 1.1 g at 0.05 s, respectively. The X-direction curve forms a gentle peak, reaching a maximum of 1.9 g at 0.325 s; the Y-direction curve increases monotonically until 0.125 s before stabilizing at approximately 1.4 g, and after 0.275 s, another plateau emerges at approximately 1.7 g. The SRSS combination curve

also presents a peak profile with a steeper slope compared to the X-direction curve, reaching its maximum of 2.4 g at 0.4 s.

Overall, the EARS curves for the first-floor equipment in the isolated structure consistently lie below those for the non-isolated configuration, indicating that isolation measures effectively reduce equipment acceleration on the first floor. Except for the Y-direction on the second floor, where the two systems exhibit relatively close response levels in certain period ranges, the EARS curves for the second-floor equipment under isolated conditions also generally remain below those of the non-isolated case. From the results, it can be concluded that for second-floor equipment, isolation measures significantly reduce acceleration responses when the equipment's natural period is less than 0.3 s; however, their effectiveness becomes limited when the period exceeds 0.4 s. Nevertheless, considering the EARS trends on both floors, implementing isolation remains the preferred strategy for mitigating equipment acceleration.

5. Effect of manufacturing errors in isolation bearings on structural torsional response

Conceptually, due to the relatively small size of the prefabricated cabin, the number of isolation bearings is limited. As a result, manufacturing errors are more likely to induce eccentricity within the isolation layer, leading to structural torsion. Therefore, this study proceeds with a detailed analysis to explore the effect of manufacturing errors in isolation bearings on structural response. According to the bearing manufacturer, the mechanical property deviations for the LR-200 bearings are within approximately $\pm 15\%$ of the target values specified prior to production (Table 6). To investigate the maximum potential torsional effects caused by manufacturing errors, configurations combining 'strong bearings' (with +15% deviation) and 'weak bearings' (with -15% deviation) in various quantities and locations were analyzed. The structural responses under these configurations were compared to the benchmark case to quantify the influence of manufacturing errors. The bearing layout schemes are illustrated in Fig. 23, with Layout 0

Table 6
Range of mechanical property parameters of isolators.

Bearing	Initial stiffness K_0 (N/mm)	Post-yield stiffness K_d (N/mm)	Yield force Q_d (kN)
Benchmark	2990.0	175.0	10.0
+15%	3438.5	201.3	11.5
−15%	2541.5	148.5	8.5

taken as the baseline for comparison. Based on the number of −15% deviation (weak) bearings, three categories of layouts were defined, and various arrangements were generated through combinatorial permutations. Since the prefabricated cabin structure is symmetrical along its short axis, only one of each pair of symmetric layouts was retained for analysis.

According to Chinese code [27,39], 11 ground motions were selected as seen in Table 7, including 2 artificial ground motions and 9 natural ground motions (from FEMA P695 [38]). The selected ground motions were applied to the finite element model using a bidirectional horizontal loading scheme (PGA = 0.51 g, X:Y = 1:0.85). The scaled acceleration response spectra of the selected ground motions, along with the target design spectrum, are presented in Fig. 24.

Table 8 summarizes the mean peak torsional angles at the base of the first story under different bearing layout schemes. It can be observed that the peak torsional angles remain relatively small across all configurations. However, in specific layouts, such as layouts 2–1 to 2–3, the peak torsional angles are comparatively larger. This is because an arrangement with stronger bearings concentrated on one side and weaker bearings on the opposite side tends to cause smaller deformations on the strong side and larger deformations on the weak side, thereby generating greater displacement differentials across the structure.

Furthermore, Fig. 25 presents the displacement and acceleration analysis results obtained from the time history analyses for each bearing

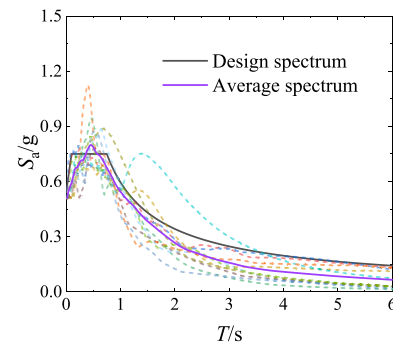


Fig. 24. Response and design spectra of selected ground motions.

Table 8
Peak torsion angle at the bottom of 1st floor of the structure with different arrangements of isolation bearings.

Isolator arrangement	1–1	1–2	2–1	2–2
Torsion angle (°)	0.103	0.102	0.130	0.071
Isolator arrangement	2–3	2–4	3–1	3–2
Torsion angle (°)	0.303	0.166	0.078	0.071

layout. The mean inter-story drift ratio at the first story generally shows an increasing trend under most bearing layouts, with the maximum increase reaching 9.3%. This can be attributed to the eccentricity of the isolation layer caused by bearing manufacturing errors, which introduces additional torsional effects on top of the overall translational motion. At the second story, the inter-story drift ratios are relatively smaller, and the influence of bearing errors is also less significant compared to the first story. The maximum bearing deformation is notably

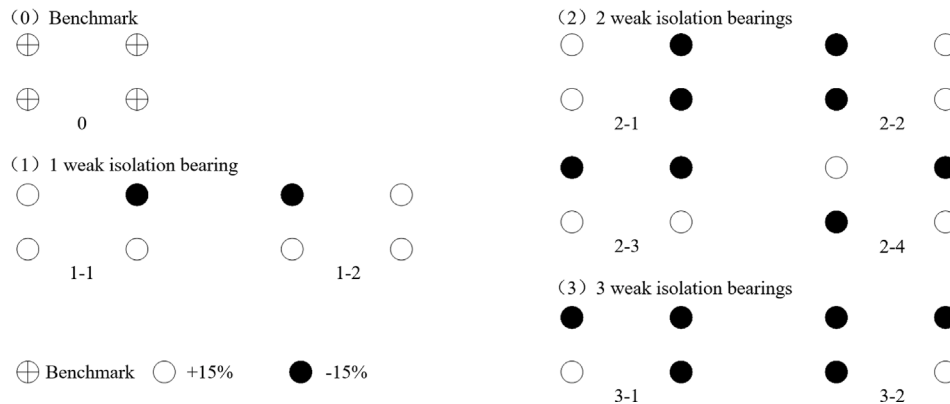


Fig. 23. The schemes of bearing layout.

Table 7
Summary of ground motions information.

Ground motion	Year	Location	Magnitude	Recording station	PGA/g	PGV/(cm/s)	Duration/s
SAEWS-01	—	—	—	—	0.51	230.3	20.00
SAEWS-02	—	—	—	—	0.51	167.0	20.00
Northridge-01	1994	Northridge	6.7	Beverly hills-mulhol	0.34	41.0	29.99
Kobe	1995	Kobe	6.9	Shin-osaka	0.26	42.0	40.96
Landers	1992	Landers	7.3	Yermo fire station	0.24	51.0	44.00
Superstition hills	1987	Superstition hills	6.5	Parachute test site	0.42	106.8	22.31
Cape mendocino	1992	Cape mendocino	7.0	Petrolia	0.63	82.1	36.00
Northridge-02	1994	Northridge	6.7	Rinaldi	0.87	167.3	19.91
Duzce	1999	Duzce	7.1	Duzce	0.52	79.3	25.89
Imperial valley	1979	Imperial valley	6.5	Chihuahua	0.28	30.5	40.00
Northridge-03	1994	Northridge	6.7	Northridge-saticoy	0.42	53.2	29.99

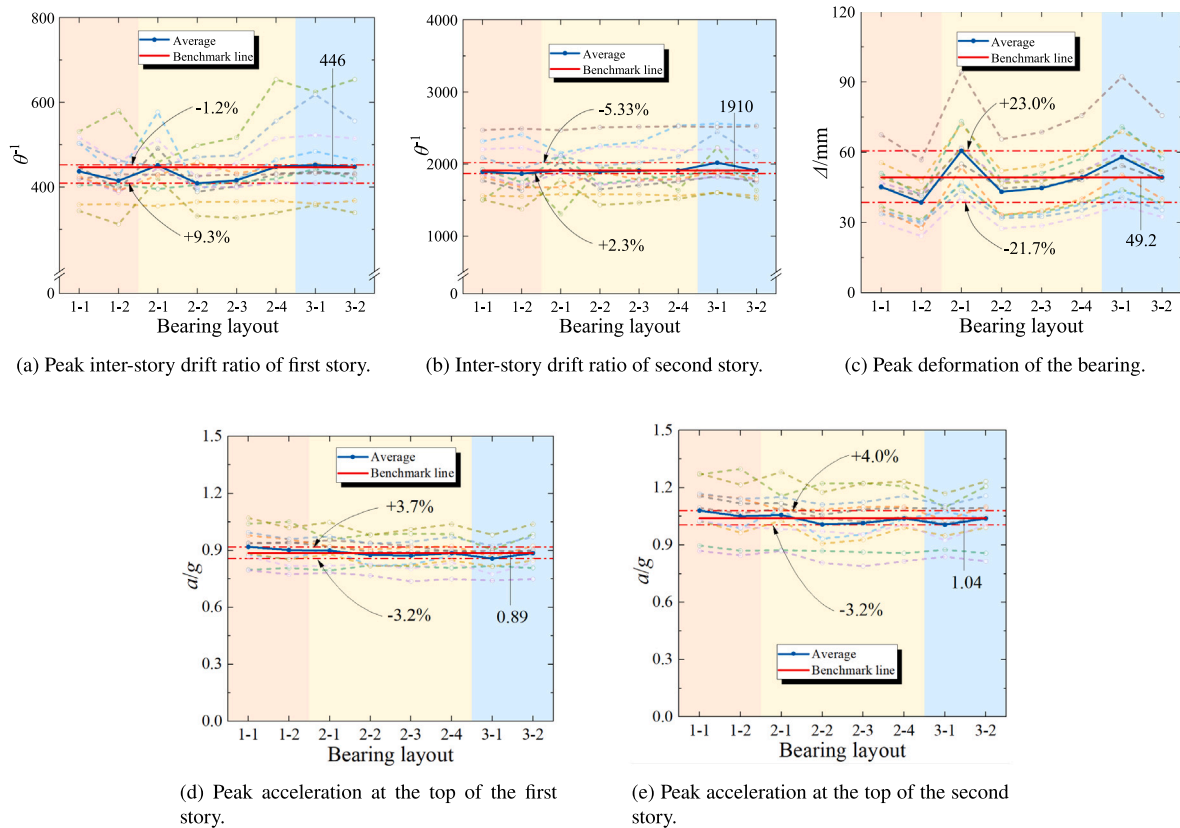


Fig. 25. Finite element simulation results.

affected by manufacturing errors, with variations ranging from -21.7% to 23.0% . This is due both to direct changes in bearing dynamic properties and to the additional effects induced by structural torsion. However, considering that the isolation layer composed of LR-200 bearings is relatively stiff for the small prefabricated cabin structure, even with a 23% increase, the maximum bearing deformation (60.5 mm) remains well below the code-specified limit of 110 mm [27]. Overall, given the sufficient safety margins in the baseline values for the first-story inter-story drift ratio (1/433) and bearing deformation (49.2 mm), relative to the code limits (1/100 and 110 mm, respectively), the maximum increases of 9.3% in drift ratio and 23% in bearing deformation due to manufacturing errors are considered acceptable. In terms of acceleration response, the influence of bearing errors is relatively minor, with variations ranging from approximately -3.2% to 4.0% .

Figs. 25(a), (d)-(e) illustrate that the structural response with one weak bearing is higher than the benchmark value, while the response with three weak bearings is generally lower than the benchmark. Meanwhile, the response with two weak bearings fluctuates slightly around the benchmark value. As the proportion of lower-stiffness (“weak”) bearings increases, the effective lateral stiffness of the isolation system decreases, which elongates the fundamental period, i.e., $T = 2\pi\sqrt{M/K_{\text{eff}}}$; similar effects of isolator stiffness variability and isolator characteristic parameters on dynamic response have been reported in previous studies [40,41]. In this case, the acceleration response spectrum of the applied ground motion falls within the descending portion of the spectrum (Fig. 24, which theoretically results in a reduction in structural acceleration response as the number of weak bearings increases).

In summary, under the seismic scenarios considered in this study, the bearing manufacturing errors and layouts still allow for a certain safety margin in the structural performance, and the resulting impact is deemed acceptable. However, for small structural systems such as prefabricated substations, which have short periods and a limited number

of isolation bearings, the amplification effects of bearing manufacturing errors on the overall response should be thoroughly considered during structural design, especially in high-intensity seismic zones, to ensure sufficient safety margins.

6. Seismic isolation layer optimization

The above studies indicated that the prefabricated cabin structure isolated with four 200 mm diameter LR bearings possesses a considerable safety margin. To maximize economic efficiency and allow the structural response to approach the code-specified limits as closely as possible, the isolation layer was optimized in this section. Given the small footprint of the prefabricated cabin, two alternative schemes were proposed and designated as LR-LNR and LR-100. In the LR-LNR scheme, two LR bearings and two LNR bearings, each with a diameter of 200 mm, were arranged diagonally. The LR-100 scheme, on the other hand, utilizes four LR bearings with a diameter of 100 mm. The parameters of the bearings used in the finite element analysis are summarized in the Table 9, and the resulting structural natural periods are listed in the Table 10.

The subsequent analysis employed the same ground motions and bidirectional loading protocol as detailed in Section 5. The mean peak structural responses under various seismic conditions are summarized

Table 9
Mechanical performance parameters of considered bearings.

Bearing type	Initial stiffness K_0 / (N/mm)	Post-yield stiffness K_d / (N/mm)	Yielding force Q_d / kN
LR-200	2990.0	175.0	10.0
LNR-200	—	192.5	—
LR-100	759.0	169.0	1.0

Table 10
Natural vibration period of the structure.

Isolation Layer	X-direction / s	Y-direction / s
LR-LNR	0.71	0.70
LR-100	0.84	0.83

Table 11
Summary of peak structural responses for different isolation layers.

Isolation layer	Bearing deformation /mm	Maximum inter-story drift ratio	1st floor peak acceleration/g	2nd Floor peak acceleration/g	Maximum stress/MPa
LR-200	49.2	1/446	0.9	1.0	361.5
LR-LNR	71.7	1/443	0.8	0.8	306.1
LR-100	170.8	1/320	0.9	0.9	362.2

in the Table 11. It is observed that although the stiffness of the LR-100 scheme is significantly lower than that of the LR-200 scheme, the fundamental periods increase only moderately (to approximately 0.84 s). Consequently, the system still fails to enter the long-period range characteristic of conventional building isolation. More critically, such an overly compliant isolation layer results in excessive bearing deformation (170.8 mm), which far exceeds the prescribed displacement constraints, while the effectiveness of acceleration mitigation is not significantly enhanced. Compared to the configuration with four LR-200 bearings, the LR-LNR scheme exhibited increased bearing deformation due to the reduced overall stiffness of the isolation layer, bringing it closer to the code-specified displacement limit of 110 mm. Meanwhile, the maximum inter-story drift ratio is basically the same as before. Moreover, reductions of 15.7%, 21.1%, and 15.3% were observed respectively in the peak accelerations at the first floor, the second floor, and the maximum structural stress. For lightweight, small-footprint modular facilities isolated by only a few bearings, the optimization is typically governed by the isolator deformation capacity rather than by story drift. In such cases, a hybrid isolation layer that combines units with different stiffness and energy-dissipation characteristics (e.g., LR and LNR) provides a practical tuning strategy to balance (i) code-compliant isolator displacement demand, (ii) transmitted floor accelerations, and (iii) overall cost. In the present study, the LR-LNR scheme achieved lower floor accelerations and stresses while maintaining the bearing deformation within the code-specified limit, indicating that mixed-bearing configurations can be a viable and economical alternative for similar modular cabins. In contrast, simply reducing bearing size (LR-100) markedly decreases the allowable displacement (which scales with bearing diameter) and may lead to excessive bearing deformation and increased response demands.

7. Discussion

The present study highlights several design-critical characteristics of base-isolated prefabricated substation equipment cabins (PSECs) that differ from conventional building-scale isolation systems. Owing to the lightweight nature of the cabin (≈ 16 t), lightweight PSECs may present a mass-stiffness mismatch with commercially available bearings, such that the achieved isolation system remains in a short-period regime ($T \approx 0.4\text{--}0.5$ s). Nevertheless, the combined full-scale evidence and validated simulations indicate that this constrained period shift can still be effective in meeting the primary objective of PSECs, i.e., protecting acceleration-sensitive internal equipment, while satisfying the deformation-related response limits. The EARS-based assessment further suggests that the isolation benefit is most pronounced for equipment with relatively short natural periods (e.g., below ~ 0.3 s), which is typical for cabinet-type components considered in this study.

The measured global response features, including moderate acceleration amplification and small inter-story drift, indicate that short-period

isolation can be a practical strategy for modular substations when both drift control and equipment protection are required. This observation also motivates the use of performance checks that explicitly target equipment acceleration demands, rather than relying solely on period-elongation expectations commonly emphasized in building-scale isolation design.

For small-footprint PSECs, the limited number of bearings makes the global response more sensitive to bearing-to-bearing property scatter. The parametric analyses show that realistic manufacturing deviations (e.g., $\pm 15\%$) can noticeably amplify displacement-related demands, primarily by altering the effective stiffness and load-sharing of the isolation layer. This underscores the importance of stringent bearing quality control, rational bearing selection/arrangement, and adequate displacement/drift safety margins in design.

The isolation-layer optimization further indicates that a hybrid LR-LNR configuration (two 200 mm LR and two 200 mm LNR bearings in this study) can provide a more favorable balance between period extension, response control, and economic efficiency while meeting the response requirements. By contrast, a more flexible configuration (e.g., the LR-100 scheme examined herein) may lead to excessive bearing deformation exceeding the code limit and thus be unsuitable for the studied PSEC application. Overall, these findings offer design-oriented guidance for base-isolated lightweight modular substations, where equipment protection and constructability constraints jointly govern the selection of an appropriate isolation solution.

8. Conclusions

This study conducted a bidirectional full-scale shaking table test and validated numerical analyses for a base-isolated prefabricated substation equipment cabin. The main conclusions are as follows:

- For lightweight PSECs, commercially available bearings may result in a constrained short-period isolation system ($T \approx 0.4\text{--}0.5$ s), which still achieves substantial reduction of equipment acceleration demands (up to $\sim 50\%$ in the tests).
- For the lightweight PSEC, the base isolation system achieves acceleration mitigation through a combination of period elongation and increased energy dissipation. Specifically, since the period elongation ratio is limited to approximately 2.4, the damping reduction coefficient plays a more decisive role than period shift in reducing the spectral acceleration demands.
- The isolated cabin showed moderate acceleration amplification ($\approx 1.0\text{--}2.0$) and small inter-story drift ($\approx 1/400$ at the maximum test intensity), demonstrating satisfactory global structural performance for the considered inputs.
- Bearing property deviations (e.g., $\pm 15\%$) can amplify displacement-related demands and potentially aggravate torsional response in small-footprint cabins with few bearings, highlighting the importance of quality control, layout rationality, and adequate safety margins.
- A hybrid LR-LNR configuration can improve the trade-off between response mitigation and economic efficiency, whereas undersized bearings may cause excessive bearing deformation and are not recommended.

Despite the above findings, the study is limited to LR and LNR bearings and does not directly evaluate equipment functionality during and after earthquakes. Future work should examine other isolation solutions, such as elastomeric bearings combined with sliding interfaces, which can provide restoring stiffness together with displacement-dependent energy dissipation. In addition, future work should incorporate functional-performance assessment of critical equipment and connections under operational conditions, together with a lifecycle-oriented evaluation of cost versus risk reduction.

CRedit authorship contribution statement

Jiajun Du: Writing – original draft, Visualization, Validation, Software, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **Wei Wang:** Writing – review & editing, Supervision, Resources, Project administration, Funding acquisition, Conceptualization. **Jiachen Zhang:** Writing – original draft, Visualization, Validation, Software, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **Xiaotian Liu:** Writing – review & editing, Visualization, Validation, Software, Resources, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **Theodoros L. Karavasilis:** Writing – review & editing, Supervision, Resources, Project administration, Funding acquisition, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgments

The financial support from the [National Natural Science Foundation of China](#) (NSFC) with Grant No. 52378182, the Collaborative Research Project under International Joint Research Laboratory of Earthquake Engineering at [Tongji University](#) and the Shanghai 2022 Science and Technology Innovation Action Plan Social Development Science and Technology Research Project with Grant No. 22dz1201700 are gratefully acknowledged.

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