

Article

Influence of Story Mass Ratio on Seismic Dynamic Response of Embedded Modular Buildings

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Abstract

In the seismic design of embedded modular buildings, the dynamic coupling effect between the main structure and embedded modules is generally simplified or neglected. This study investigates the influence of story mass ratio on the seismic response of a six-story steel frame structure with embedded box modules. A coupled finite element model was established, and the equivalent lateral stiffness simulation method was theoretically verified. Elastic time-history analyses were conducted using three representative ground motions. Eight story mass ratios were considered, and the floor displacement, story shear force, and floor acceleration response spectra were compared between coupled and appended-mass models. The results indicate that the story mass ratio is an important parameter affecting the magnitude of module–main-frame coupling effects within the investigated prototype. For the investigated structural prototype and selected seismic inputs, the differences between the coupled and appended-mass models are relatively small at relatively low mass ratios, while the influence of coupling becomes progressively more significant with increasing μ . With increasing μ , lower-story displacement and story shear increase noticeably (maximum first-floor displacement ratio: 1.157; maximum shear ratio: 1.139), while upper-story responses decrease, with a maximum reduction in top-story shear of 16.9%. In addition, the short-period spectral peak of the floor acceleration response spectra is influenced by the mass ratio. For the investigated cases, noticeable differences in local floor spectral peaks are observed at relatively higher mass ratios, with relative spectral differences exceeding 5% in some floors based on the adopted comparison criterion and significant reductions occurring at larger mass ratios. These results provide preliminary insights into model selection and seismic response evaluation of the investigated embedded modular building prototype.

Keywords: embedded modular building; dynamic coupling; story mass ratio; seismic response; seismic design



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1. Introduction

In seismic analysis of building structures, floor mass distribution is a fundamental factor governing dynamic characteristics and seismic responses. Variations in mass distribution can significantly influence natural periods, modal participation, inertia-force distribution, inter-story drift, story shear, and floor acceleration responses. This effect becomes more pronounced in primary–secondary structural systems, where secondary components are modeled as additional lumped masses and interact with the primary structure through stiffness and connection constraints, thereby affecting the overall dynamic

response. Accordingly, the story mass ratio μ , defined in this study as the ratio of module mass to total floor mass, is introduced to characterize the intensity of dynamic interaction and to evaluate whether secondary components can be simplified as appended mass or should be explicitly considered in coupled analysis. In this study, the variation of story mass ratio was achieved by changing the module mass while maintaining identical module geometry, material properties, stiffness characteristics, and connection conditions among different cases. This controlled parameter setting was adopted to isolate the influence of mass ratio on the module–main-frame coupling response.

Modular construction represents an important direction for industrialized and sustainable development in the construction industry. Modular buildings are composed of volumetric units integrating structural, enclosure, mechanical, electrical, and finishing systems, which are manufactured in factories and assembled on-site. This approach improves construction efficiency, reduces waste, and shortens construction periods [1]. Owing to advantages such as low self-weight, high strength, and recyclability, steel modular buildings have been widely applied in hospitals, high-rise residential buildings, and hotels [2–6].

According to existing specifications, steel modular buildings can be classified into self-supporting stacked-box systems, modular–steel frame composite systems, modular–shear wall composite systems, and embedded modular structural systems [7]. Among these, embedded modular buildings place volumetric modules within a steel frame, where the main frame resists gravity and lateral loads while modules serve functional and partially structural roles. This configuration provides a promising solution for increasing building height and improving functional flexibility. However, in engineering practice, embedded modules are often simplified as equivalent floor masses in design, neglecting their stiffness contribution and interaction with the main structure. Such simplification may lead to inaccuracies in predicting dynamic responses, including story shear distribution, inter-story drift, and floor acceleration characteristics, particularly for public buildings where coupling effects may be non-negligible [8]. Variations in the story mass ratio can influence the degree of inertial coupling and modify the energy transfer path between the primary and secondary structures [9].

Previous studies on modular steel structures have mainly focused on global seismic performance and structural systems, including the influence of module arrangement, lateral-force-resisting mechanisms, and overall dynamic characteristics [10–12]. Another research direction emphasizes inter-module connections and joint behavior, highlighting the influence of connection stiffness and load transfer mechanisms on structural response [13–15]. In addition, studies on primary–secondary structural systems have shown that interaction between main structures and secondary components can significantly affect dynamic responses, particularly floor acceleration and response spectra when the mass or stiffness of secondary systems is not negligible [16,17]. More broadly, research on soil–structure interaction, adjacent structural interaction, and vibration control systems has demonstrated that interaction effects can substantially modify seismic demand and response distribution [18–22]. In addition, advanced data-driven and machine-learning-based methods have recently been used to estimate seismic damage and response characteristics of building structures, including interpretable machine learning for cumulative seismic damage assessment and machine-learning-based seismic damage classification of reinforced concrete buildings [23,24]. Although these studies are not specifically focused on embedded modular buildings, they collectively indicate that interaction mechanisms should be explicitly considered when secondary components possess non-negligible mass and stiffness.

Despite these advances, research on embedded modular buildings remains limited, particularly regarding the lack of quantitative criteria for determining whether embedded modules can be simplified as appended mass or should be modeled as a coupled subsystem.

In particular, the influence of story mass ratio on the dynamic behavior of embedded module–frame systems has not been systematically investigated. This motivates the present study to focus on the role of story mass ratio in governing coupled seismic responses of embedded modular buildings.

Compared with existing studies, the present work takes the story mass ratio as the key control parameter and systematically compares coupled finite element models with simplified appended-mass models. The influence of mass ratio on floor displacement, story shear redistribution, and floor acceleration response spectra is investigated through nonlinear time-history analyses. The results aim to provide a quantitative basis for evaluating when coupled modeling is necessary and to improve simplified seismic design approaches in which embedded modules are treated only as equivalent floor masses.

2. Finite Element Model Development and Verification

2.1. Basic Model Information

In this paper, a six-story steel frame structure is taken as the main structure, with embedded box modules as the substructure, to investigate the influence of different story mass ratios on the dynamic response of the structure. The main structure has a plan dimension of 17.2 m × 15 m and a story height of 4.5 m. The column-base nodes of the main steel frame were fully fixed at the ground level, with all translational and rotational degrees of freedom constrained. The box height is 3.3 m, leaving a 1.2 m overhead space for mechanical ducts and electrical lines. A reinforced concrete floor slab with an actual thickness of 120 mm is modeled. The dead loads and live loads applied to the beams and slabs were determined according to the Load Code for the Design of Building Structures (GB 50009-2012) [25] and the seismic gravity load representation is specified in the General Code for Seismic Precaution of Buildings and Municipal Engineering (GB 55002-2021) [26]. To focus on the influence of the story mass ratio on the coupled seismic response of the embedded modular building, the self-weight of structural members was calculated from the material density, while the additional permanent loads and live loads were converted into equivalent uniformly distributed loads and applied to the corresponding slab regions. In the dynamic analysis, the gravity load representative value was considered based on the self-weight of structural components and the combination value of variable loads, consistent with the seismic design requirement. The cross-sectional dimensions of the beams and columns of the main structure are given in Table 1. The steel material is Q355, with a Poisson's ratio of 0.3 and an elastic modulus of 206,000 MPa.

Table 1. Dimension table of beams and columns of the main structure.

Structural Member	Cross-Sectional Dimensions (mm)
Columns	B 300 × 300 × 12
main beam	HM 300 × 200 × 10 × 8
secondary beam	HM 300 × 175 × 8 × 6

The embedded box modules were modeled according to their actual component dimensions. The box floor slab was converted into an equivalent surface load applied to the corresponding main-structure floor slab, while the box columns and top beams were explicitly modeled and connected to the main frame at the corresponding positions. In the coupled models, kinematic coupling constraints were established between the reference points of the embedded modules and the corresponding floor regions of the main frame to maintain displacement compatibility during the time-history analyses. In this study, this constraint was adopted to represent an idealized displacement-compatible connection condition between the embedded modules and the main frame. This assumption enables

the influence of story mass ratio on the global module–main-frame coupling response to be isolated while avoiding additional parameter variations associated with connection stiffness. In the uncoupled models, the embedded modules were not explicitly coupled with the main frame but were simplified as appended masses applied to the corresponding floor slabs. For the cases where coupling effects are not considered, the boxes are applied to the corresponding floor slabs based on the lumped mass system assumption.

The story mass ratio was used to quantify the relative mass contribution of the embedded modules at each module floor. In this study, the story mass ratio of the i -th floor is defined as the ratio of the embedded module mass to the total mass of the embedded module and the corresponding main-structure floor:

$$\mu_i = \frac{m_{b,i}}{m_{b,i} + m_{s,i}} \quad (1)$$

where

μ_i —story mass ratio of the i -th floor;

$m_{b,i}$ —mass of the embedded box module assigned to the i -th floor;

$m_{s,i}$ —mass of the corresponding main-structure floor.

The finite element models, frequency extraction analyses, and direct-integration time-history analyses were implemented using Abaqus 2019 [27]. The beams and columns of the main steel frame were modeled using 2-node linear Timoshenko beam elements (B31), and the columns and beams of the embedded box modules were also modeled using the same type of beam elements. The reinforced concrete floor slab was explicitly modeled using four-node doubly curved shell elements with reduced integration (S4R), with an actual thickness of 120 mm. No additional rigid diaphragm constraint was introduced at the floor level. The in-plane stiffness of the floor system was represented by the membrane behavior of the shell elements, while displacement compatibility between connected components was enforced through coupling constraints at the beam–slab interface. The typical mesh size was 100 mm for beam members and 50 mm for floor slabs. The same mesh configuration was adopted for all mass-ratio cases to ensure consistency of comparison. The design drawings of the main structure and the finite element model are shown in Figure 1.

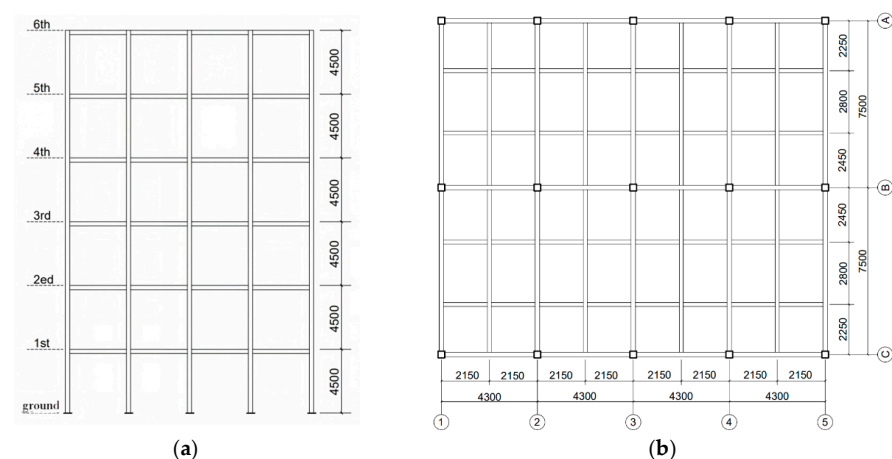


Figure 1. Design drawing of the main structure: (a) Structural elevation drawing; (b) structural plan drawing.

To improve the reproducibility of the finite element models and to clarify the influence of the coupling treatment on the global dynamic characteristics, frequency extraction analyses were conducted for both the coupled models and the appended-mass models under different story mass ratios. The finite element configurations of the main structure and the

embedded modules are illustrated in Figure 2. In the coupled models, the embedded modules were connected to the main frame through kinematic coupling constraints, whereas in the appended-mass models, the embedded modules were simplified as lumped additional mass applied to the corresponding floor slabs. The geometry, material properties, boundary conditions, mass assignment method, and module layout were kept consistent with those used in the time-history analyses. Table 2 summarizes the first three natural periods and the corresponding dominant modal characteristics of the two modeling schemes. The comparison provides the basic dynamic properties required for reproducing the subsequent time-history analyses and also reflects the influence of module–main-frame coupling on the global modal characteristics.

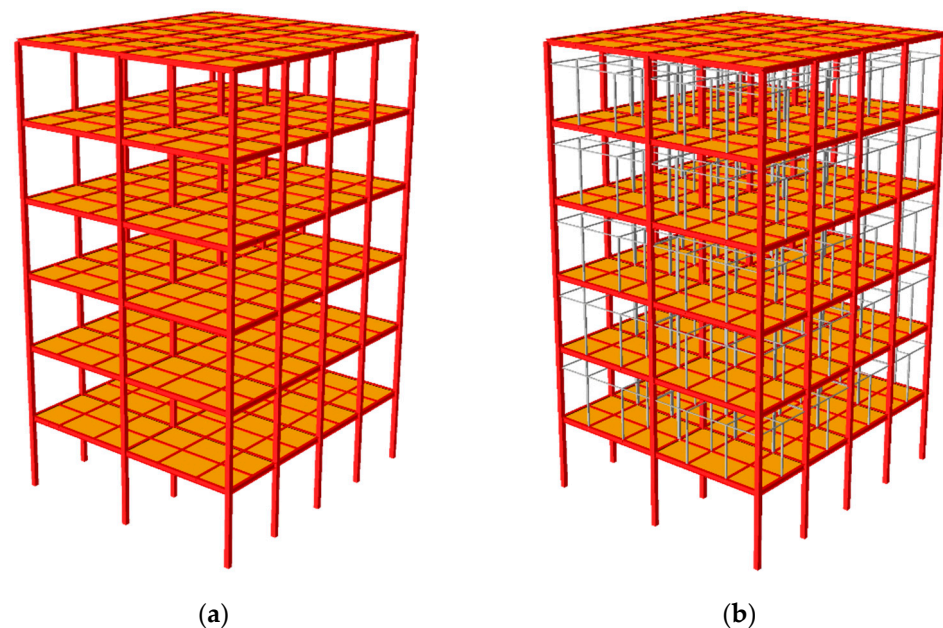


Figure 2. Finite element structural model diagram: (a) Model diagram of the main structure; (b) model diagram of the embedded module and main structure.

Table 2. Modal analysis results of the coupled and appended-mass finite element models.

Story Mass Ratio	Model Type	T_1/s	T_2/s	T_3/s	Dominant Modal Characteristics
5%	coupled	2.024	1.766	1.475	Mode 1: X translation; Mode 2: Y translation; Mode 3: torsion
	uncoupled	2.040	1.770	1.479	
7.50%	coupled	2.046	1.785	1.492	Same as above
	uncoupled	2.062	1.789	1.494	
8.50%	coupled	2.056	1.794	1.499	Same as above
	uncoupled	2.071	1.797	1.500	
10%	coupled	2.072	1.808	1.511	Same as above
	uncoupled	2.086	1.810	1.510	
15%	coupled	2.122	1.852	1.549	Same as above
	uncoupled	2.133	1.851	1.543	

Table 2. Cont.

Story Mass Ratio	Model Type	T ₁ /s	T ₂ /s	T ₃ /s	Dominant Modal Characteristics
17.50%	coupled	2.155	1.881	1.574	Same as above
	uncoupled	2.163	1.877	1.564	
20%	coupled	2.182	1.905	1.595	Same as above
	uncoupled	2.187	1.898	1.581	
25%	coupled	2.246	1.962	1.645	Same as above
	uncoupled	2.245	1.949	1.621	

As shown in Table 2, the first three natural periods of both modeling schemes generally increase with increasing story mass ratio, which is consistent with the increase in effective structural inertia. The dominant modal sequence remains stable for all mass-ratio cases, with the first mode dominated by X-direction translation, the second mode by Y-direction translation, and the third mode by torsion. The difference between the coupled and appended-mass models indicates that the explicit module–main-frame coupling treatment slightly changes the global modal properties, indicating that a comparison between the two modeling schemes is necessary for subsequent seismic response analysis.

The steel material was modeled as a linear elastic material, with an elastic modulus of 206,000 MPa and a Poisson’s ratio of 0.3. Since this study focuses on the influence of the story mass ratio on the global seismic response under frequent earthquakes, material yielding and damage evolution were not considered. Therefore, the time-history analyses conducted in this study were elastic direct integration analyses. However, geometric nonlinearity was activated in the analysis step to account for the influence of structural deformation on the equilibrium state, including possible P-Δ effects caused by gravity loads and lateral displacements. In this way, the numerical model considered geometric nonlinearity while maintaining a linear elastic constitutive relationship for the steel members.

Rayleigh damping is adopted to model the energy dissipation of the structural system. A target modal damping ratio of 2% is assigned to the first two dominant translational modes of each numerical model. The corresponding Rayleigh coefficients are determined based on the modal frequencies of each model, as detailed in the following equations. For different story mass ratio cases, the Rayleigh coefficients are recalculated according to the corresponding modal characteristics while maintaining the same target damping ratio. This treatment ensures a consistent damping level among different cases and avoids variations in modal damping caused by changes in structural dynamic properties.

Based on this approach, Rayleigh damping is used to establish the structural damping matrix in the finite element analysis, and its expression is

$$[C] = \alpha[M] + \beta[K] \quad (2)$$

where

α —mass damping coefficient;

β —stiffness damping coefficient;

$[M]$ —mass matrix;

$[K]$ —stiffness matrix.

α calculation formula:

$$\alpha = \frac{2w_i w_j (\xi_i w_j - \xi_j w_i)}{w_j^2 - w_i^2} \quad (3)$$

β calculation formula:

$$\beta = \frac{2(\xi_i w_j - \xi_j w_i)}{w_j^2 - w_i^2} \quad (4)$$

where

w_i, w_j —circular natural frequencies of the selected i -th and j -th modes;

ξ_i, ξ_j —target damping ratios of the selected i -th and j -th modes.

Since the same target damping ratio was adopted for the first two modes, namely $\xi_i = \xi_j = \xi$, and $\omega = 2\pi f$, the above equations can be simplified as follows to maintain constant target damping ratios under varying dynamic properties:

$$\alpha = \frac{4\pi f_i f_j \xi}{f_i + f_j} \quad (5)$$

$$\beta = \frac{\xi}{\pi(f_i + f_j)} \quad (6)$$

where

f_i, f_j —natural frequencies of the selected first and second modes;

ξ —target modal damping ratio of the assembled structural system.

Because the natural periods vary with the story mass ratio, the Rayleigh damping coefficients were calculated separately for each mass-ratio case based on the corresponding first two modal frequencies.

2.2. Model Credibility Assessment

In this paper, the credibility assessment of the finite element model primarily focuses on the equivalent lateral stiffness of the box and its connected spring model. Based on the Euler–Bernoulli beam element stiffness method, a formula for the equivalent lateral stiffness of a single column considering a rotational spring at the column base is derived. The equivalent lateral stiffness of the combined system consisting of a single column and a column-base spring can be defined as

$$k_{col}(K_\theta) = \frac{V_1}{\Delta} \quad (7)$$

$$k_{col}(K_\theta) = K_{\Delta\Delta} - \frac{K_{\Delta 1} K_{1\Delta}}{K_{11}} \quad (8)$$

$$k_{col}(K_\theta) = \frac{12EI_c(EI_c + K_\theta h)}{h^3(4EI_c + K_\theta h)} \quad (9)$$

The formula satisfies both limit cases of an ideal hinge and an ideal rigid connection at the column base. For the entire box, the equivalent story stiffness can be obtained by summing the individual column stiffnesses in parallel.

A finite element model that conforms to the spatial configuration of the embedded box in this paper is established, and the finite element calculation results are compared with the theoretical calculation results. The comparison is shown in Table 3. The results in Table 3 show that the relative differences in lateral displacement between the simulated and theoretical values under all cases are within 5%, with a maximum of only 4.67%. This indicates that the finite element modeling method established in this paper can accurately reflect the lateral mechanical behavior of the box, and the calculation results are reliable.

Table 3. Comparison of lateral displacement: theory vs. simulation.

Dimensionless Stiffness $\lambda = K_{\theta}h/(EI)$	Per Column Base K_{θ} (kN·mm/rad)	Equivalent Stiffness K_{frame} (kN/mm)	Theoretical Lateral Displacement	Simulated Lateral Displacement	Difference %
0 (Hinged connection)	0	0.29808	100.64	103.8	3.14
0.5	4.92×10^4	0.39745	75.48	78.08	3.44
1	9.84×10^4	0.47694	62.9	65.18	3.62
2	1.97×10^5	0.59617	50.32	52.24	3.82
5	4.92×10^5	0.79489	37.74	39.32	4.19
10	9.84×10^5	0.93684	32.02	33.43	4.40
50	4.92×10^6	1.12610	26.64	27.88	4.65
100	9.84×10^6	1.15794	25.91	27.12	4.67
∞ (Rigid connection)	∞	1.19233	25.16	26.14	3.90

2.3. Ground Motion Selection

According to the Code for Seismic Design of Buildings (GB 50011-2010) [9], the seismic ground motion parameters should be determined based on the seismic precautionary intensity, design basic acceleration, design earthquake group, and site class. In this study, the structure was assumed to be located on a Site Class II site and subjected to a frequent earthquake corresponding to a seismic precautionary intensity of 7. To ensure compatibility with the target design spectrum while maintaining spectral diversity, three acceleration time histories were selected, including two natural records and one artificial record. The EI Centro record was selected because it contains relatively balanced frequency components, whereas the Taft record was used to represent a ground motion with more pronounced low-frequency energy. The artificial wave was adopted to improve the compatibility of the selected ground-motion set with the code design spectrum. It was generated using a spectrum-compatible method based on the GB 50011-2010 design spectrum for Site Class II and seismic precautionary intensity 7, followed by baseline correction and scaling to a peak ground acceleration (PGA) of 0.35 m/s^2 . All records were scaled by the peak ground acceleration (PGA) scaling method to 0.35 m/s^2 , corresponding to the frequent earthquake level adopted in this study. The metadata of the selected ground motions and the corresponding PGA scaling factors are summarized in Table 4.

Table 4. Metadata and scaling factors of selected ground motions.

Ground Motion	Record and Component	Magnitude/Distance/Site	PGA Scaling
EI Centro wave	1940 EI Centro earthquake, El Centro Array Station 9, 180 component	Mw 6.9; hypocentral distance 12.2 km; alluvium site	$0.34169 \rightarrow 0.35 \text{ m/s}^2$; SF = 0.102
Taft wave	1952 Kern County earthquake, Taft Lincoln School, 111 component	Mw 7.5; closest distance to fault 36.2 km; alluvium site	$0.17595 \rightarrow 0.35 \text{ m/s}^2$; SF = 0.199
Artificial wave	Target-spectrum-compatible artificial wave, X direction	Site Class II; generated based on GB 50011-2010 design spectrum	Target PGA = 0.35 m/s^2

In the time-history analyses, the selected ground motions were applied along the global X direction of the finite element model, which corresponds to the principal horizon-

tal analysis direction of the main steel frame. To isolate the influence of the story mass ratio and to ensure consistency between the coupled and uncoupled models, only one horizontal component of each ground motion record was used in this study. For the two natural records, the horizontal component was selected based on its compatibility with the target design spectrum after PGA scaling and its representative frequency-content characteristics. The artificial wave was also treated as a single horizontal input component generated or selected to match the target code design spectrum. The vertical component and the orthogonal horizontal component were not included in the present analysis. The time step of the input records was 0.02 s, and the significant duration satisfied the requirements for time-history analysis. The acceleration time histories and power spectra of the selected ground motions are shown in Figure 3. Their acceleration response spectra are compared with the code design spectrum in Figure 4, and the comparison shows good agreement within the main period range of interest. Therefore, the selected ground motions can provide different frequency-content characteristics for comparing the coupled and uncoupled seismic responses under different story mass ratios.

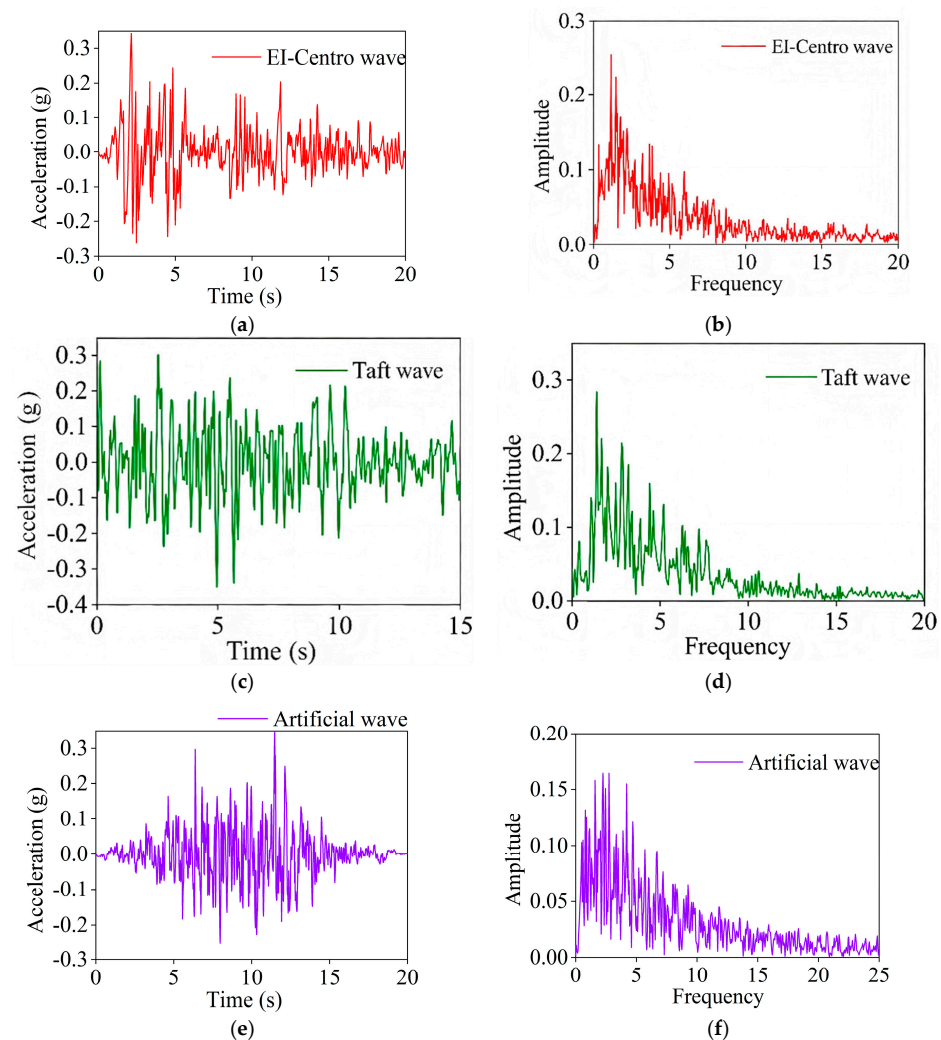


Figure 3. Seismic wave time-history curves and power spectra: (a) time-history curve of EI-Centro wave; (b) power spectrum of EI-Centro wave. (c) Time-history curve of Taft wave, (d) power spectrum of Taft wave, (e) time-history curve of artificial wave, (f) power spectrum of artificial wave.

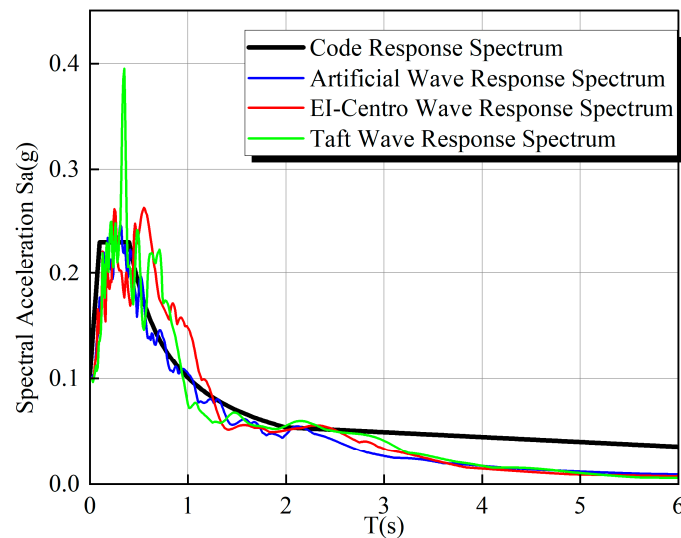


Figure 4. Matching of selected ground motions with the code spectrum.

2.4. Case Setup

In this paper, a full-floor vertical distribution scheme is adopted. Specifically, the embedded modules were arranged from the second floor to the sixth floor, and all figures and tables in the following response analyses correspond to this layout scheme. To better account for the influence of box coupling on the response of the main structure, corresponding finite element models without considering coupling effects (i.e., boxes treated only as appended mass applied to the corresponding floors) are also established.

The dimensions and material parameters of the box components and the main structural model remain unchanged, and the bottom connection between the boxes and the main structure is assumed to be rigid in the present study to represent an idealized rigid connection condition. This assumption is adopted to focus on the influence of story mass ratio on the module–main-frame interaction. The possible influence of semi-rigid connection behavior and connection stiffness variation is not considered in the present study and is recognized as a limitation of the current numerical model. The story mass ratio μ is varied by changing the appended mass on the embedded boxes. Eight story mass ratio cases are considered: $\mu = 5\%$, 7.5% , 8.5% , 10% , 15% , 17.5% , 20% , and 25% . Finite element models with and without coupling (i.e., only appended mass) are established separately, and the differences in structural dynamic response under different story mass ratios are compared and analyzed.

3. Results

3.1. Influence of Story Mass Ratio on Structural Dynamic Displacement Response

The variation curves of the average floor displacement ratio (coupled/uncoupled) with the mass ratio under different story mass ratios are shown in Figure 5.

The influence of different story mass ratios μ on the displacement response of each floor is summarized in the form of the displacement ratio (coupled/uncoupled) under fixed layout conditions. The displacement ratio directly reflects the degree to which box coupling affects the structural displacement. Meanwhile, averaging the results from the three ground motion records can, to some extent, reduce the chance caused by differences in the frequency spectra of individual ground motions, thereby highlighting the dominant role of the story mass ratio.

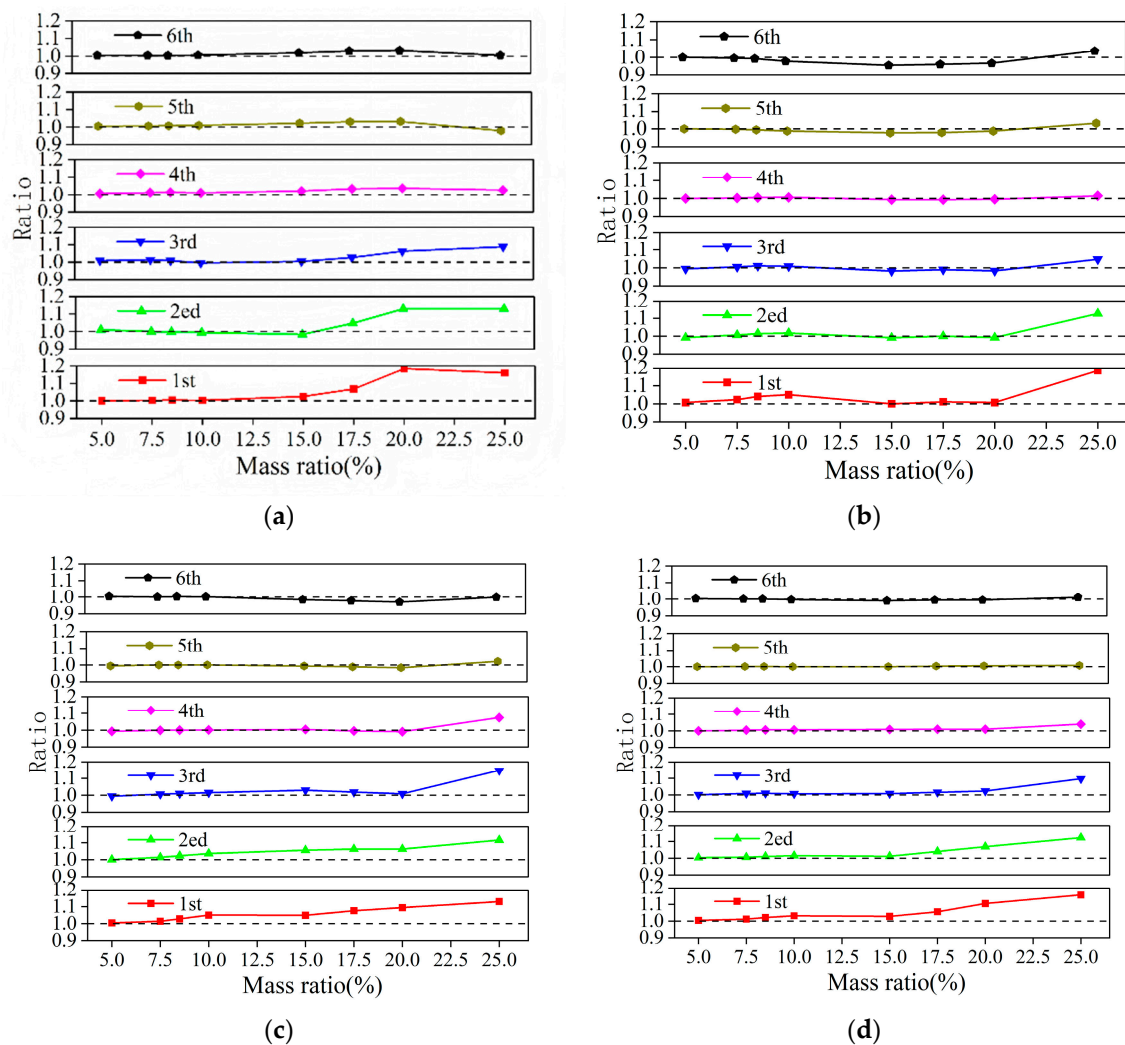


Figure 5. Variation curves of floor displacement ratio versus mass ratio for different ground motions: (a) floor displacement ratio under EI wave; (b) floor displacement ratio under Taft wave; (c) floor displacement ratio under artificial wave; (d) average floor displacement ratio.

The results show that the displacement ratio of each floor generally increases gradually with increasing μ . When $\mu < 0.10$, the displacement ratio of each floor is generally close to 1.0, indicating that the difference in displacement response between the coupled system and the system with only appended mass is small, and the dynamic participation of the boxes is limited. When $\mu \geq 0.10$, the displacement ratio of the lower floors begins to increase significantly, while the change in the upper floors remains small, indicating that as the mass ratio increases, both the inertial force of the boxes and the primary–secondary structure coupling effect are enhanced. When $\mu \geq 20\%$, the displacement ratio of the bottom floor increases further, while the relative difference in the upper floors remains limited.

Visualizing the data as shown in Figure 5, it can be seen that the sensitivity of the displacement ratio to μ exhibits a distinct layered characteristic along the height. Under most mass ratio cases, the displacement ratios of the first and second floors are generally greater than 1, and the amplification becomes more pronounced with increasing μ . The third and fourth floors also gradually increase with μ , but the magnitude is significantly smaller than that at the bottom. The displacement ratios of the upper floors are close to 1 or slightly reduced. The sensitivity of the displacement ratio is strongest in the lower floors and generally weakens with increasing height. Although there is some dispersion among different ground motions, and the dispersion gradually increases with μ , the overall

trend remains: as μ increases, the displacement ratio continues to rise in the lower floors, indicating that the influence of the coupling effect increases with the mass ratio.

For the investigated structural prototype, when $\mu < 0.10$, the appended-mass model produces displacement responses close to those of the coupled model. When $\mu \geq 0.10$, explicit coupling should be considered, particularly when evaluating the displacement and ductility demands of the lower stories.

The coupled/uncoupled ratios were adopted to isolate the influence of module–main-frame coupling under identical structural and seismic-input conditions. In addition, the absolute inter-story drift ratios obtained from the time-history analyses were checked against the elastic inter-story drift limit of 1/250 specified for multi-story steel structures. All analyzed cases remained within this limit. Therefore, the increase in the lower-floor displacement ratios represents a local redistribution and concentration of deformation demand rather than an exceedance of the code deformation limit.

3.2. Influence of Story Mass Ratio on Structural Dynamic Shear Response

The variation curves of the average story shear ratio (coupled/uncoupled) with the mass ratio under different story mass ratios are shown in Figure 6.

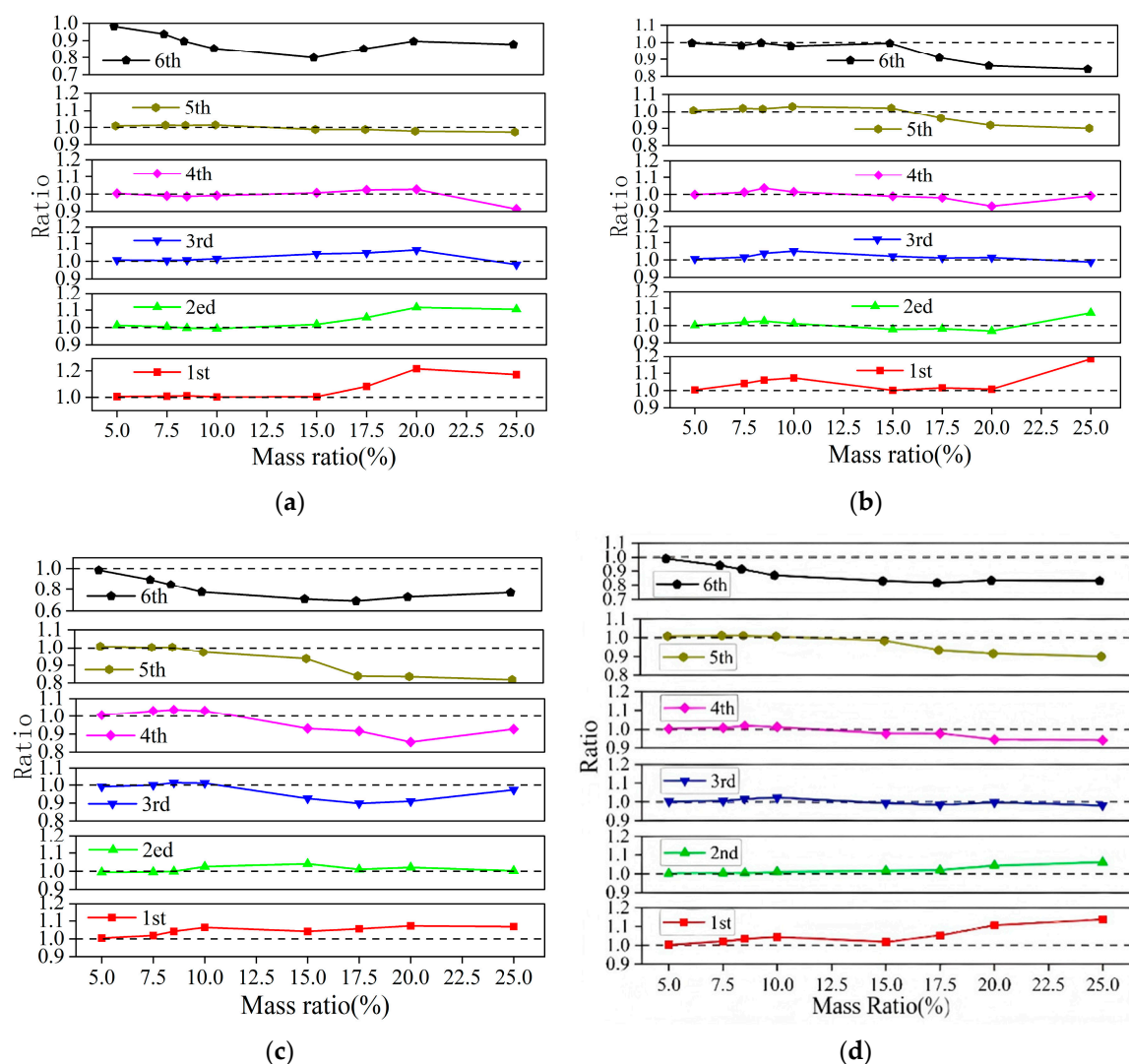


Figure 6. Variation curves of Story shear ratio versus mass ratio for different ground motions: (a) story shear ratio under EI wave; (b) story shear ratio under Taft wave; (c) story shear ratio under artificial wave; (d) average story shear ratio.

As the story mass ratio increases, the story shear ratio of the coupled system relative to the uncoupled system exhibits a systematic redistribution along the height:

Small mass ratio (e.g., $\mu < 0.10$): The ratio is close to 1, the coupling effect is weak, and the boxes have a limited influence on the main-structure story shear.

$\mu \geq 0.10$: The shear difference at the top floor exceeds 10%, while the difference at the bottom floor is about 5%; the reduction effect in the upper floors is greater than the amplification in the lower floors.

$\mu \geq 0.15$: The average ratio of the first and second floors is greater than 1, and that of the fourth to sixth floors is less than 1; the shear redistribution tends to stabilize.

$\mu \geq 0.20$: The difference at the first floor also exceeds 10%; the coupling effect cannot be neglected in the design of bottom-story shear.

Overall, the bottom-story shear increases, the upper-story shear decreases, there exists a transition floor in the middle, and the difference expands with increasing μ .

Although there is some dispersion among different ground motions, the overall trend is stable:

Under high mass ratios: The first-story ratios of the EI wave and Taft wave are higher than that of the artificial wave, while the artificial wave exhibits a more pronounced reduction in top-story shear.

Trend along height: The story shear ratio gradually decreases with increasing height, and the difference between the coupled and uncoupled systems becomes greater as the mass ratio increases.

In the present model, the first-story shear corresponds to the structural base shear. Therefore, the first-story shear ratio shown in Figure 6 also represents the relative variation in base shear between the coupled and uncoupled models. The amplification observed in the lower stories indicates a concentration of seismic demand. However, response amplification alone is insufficient to identify a soft or weak story, which requires separate checks of story stiffness and lateral load-carrying capacity.

For the investigated structural prototype, the influence of coupling on story shear is limited when $\mu < 0.10$. As the story mass ratio increases beyond this range, explicit coupled modeling or an appropriate correction should be considered, with particular attention to the increased shear demand of the lower stories. The reduction in upper-story shear observed in the present model should be further verified before being used for member design or optimization.

3.3. Influence of Story Mass Ratio on Floor Acceleration Response Spectra

In this study, the floor acceleration response spectra were calculated from the absolute floor acceleration time histories using a damping ratio of 0.02, and the envelope of the spectra under the three ground motions was adopted as the representative floor response spectrum. As illustrated by the representative complete spectra in Figure 7, two characteristic peaks are clearly separated by a distinct spectral trough. The first peak was identified as the first local maximum in the short-period range and is mainly associated with the combined effects of higher-order structural modes and the high-frequency components of the input ground motions. The second peak was identified as the local maximum closest to the first-mode period of the corresponding structural model. For the coupled and uncoupled models, the two peaks were identified independently using the same physical criteria, and their corresponding periods were marked in Figure 7.

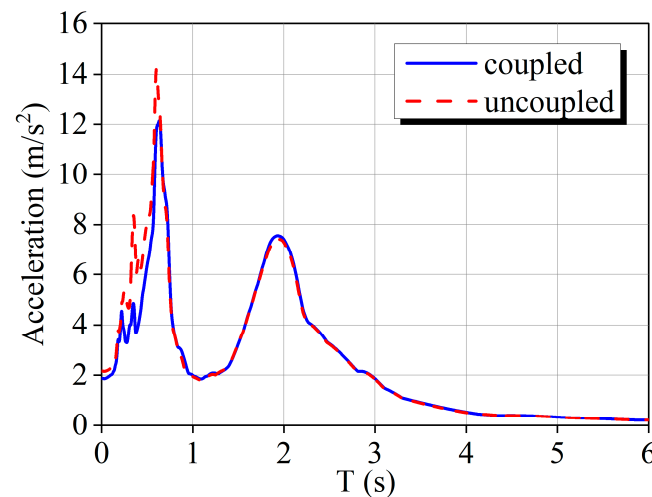


Figure 7. Comparison of the fourth-floor acceleration response spectra of the coupled and uncoupled models for $\mu = 17.5\%$.

The spectral ordinates of the two characteristic peaks were statistically compared under different story mass ratios. Following the criterion adopted in Reference [28], a difference of 5% between the coupled and appended-mass floor response spectra was used as a comparative indicator to quantify the response discrepancy between the two modeling approaches. Because the two models have identical structural parameters, mass distributions, boundary conditions, and seismic inputs, the observed spectral difference mainly reflects the influence of module–main-frame coupling. When the difference is less than 5%, the discrepancy between the two models is considered relatively small within the scope of this study; when the difference exceeds 5%, the coupling-induced response variation becomes noticeable and explicit coupled modeling may provide a more accurate representation of the module–main-frame interaction. The corresponding spectral ordinates and relative differences are summarized in Tables 5–12.

Table 5. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.05$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	12.813	2.657	12.792	2.643	−0.163	−0.527
2	13.242	4.467	13.117	4.514	−0.939	1.039
3	11.431	6.829	11.543	6.889	0.979	0.886
4	11.042	8.689	11.016	8.770	−0.236	0.941
5	8.568	9.913	8.473	10.026	−1.105	1.137
6	14.760	10.500	14.723	10.633	−0.254	1.265

Table 6. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.075$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	14.669	2.997	14.304	3.008	−2.489	0.351
2	17.226	5.087	17.440	5.176	1.243	1.731
3	13.821	7.791	14.010	7.910	1.362	1.533
4	12.965	9.892	12.694	10.038	−2.093	1.478
5	10.884	11.252	11.022	11.426	1.268	1.549
6	18.565	11.894	18.767	12.088	1.089	1.633

Table 7. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.085$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	14.701	3.041	13.855	3.062	−5.755	0.701
2	17.650	5.057	17.836	5.158	1.051	1.998
3	14.258	7.754	14.592	7.890	2.344	1.750
4	13.075	9.844	12.763	10.010	−2.388	1.694
5	11.188	11.185	11.349	11.378	1.439	1.728
6	19.049	11.814	19.360	12.021	1.632	1.749

Table 8. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.10$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	12.938	2.866	10.377	2.900	−19.791	1.175
2	15.322	4.413	15.410	4.466	0.572	1.200
3	12.511	6.754	12.762	6.867	2.009	1.682
4	11.576	8.631	9.792	8.753	−15.414	1.410
5	9.848	9.843	9.872	9.974	0.246	1.323
6	16.654	10.409	16.769	10.538	0.693	1.244

Table 9. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.15$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	12.258	3.047	7.990	3.126	−34.817	2.596
2	15.159	4.285	13.486	4.350	−11.040	1.523
3	12.815	6.598	11.549	6.767	−9.883	2.571
4	12.029	8.536	3.848	8.727	−68.007	2.246
5	9.920	9.837	9.063	10.032	−8.632	1.982
6	16.901	10.459	15.234	10.641	−9.862	1.740

Table 10. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.175$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	13.485	3.377	8.767	3.384	−34.988	0.209
2	16.691	4.816	14.636	4.885	−12.312	1.439
3	14.401	7.405	12.172	7.556	−15.477	2.033
4	13.601	9.548	4.619	9.774	−66.041	2.361
5	11.156	11.009	9.975	11.259	−10.581	2.272
6	18.975	11.681	16.339	11.945	−13.891	2.263

Table 11. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.20$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	11.166	3.171	7.955	3.028	−28.757	−4.525
2	14.094	4.377	11.096	4.438	−21.273	1.381
3	12.088	6.646	9.709	6.753	−19.674	1.616
4	11.709	8.544	5.032	8.739	−57.019	2.288
5	9.540	9.829	7.794	10.077	−18.307	2.520
6	15.966	10.438	12.560	10.695	−21.331	2.458

Table 12. Statistical table of peak acceleration response spectrum values for mass ratio $\mu = 0.25$.

Floor	Uncoupled System		Coupled System		Difference (%)	
	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value	First Peak Value	Second Peak Value
1	10.724	3.466	10.375	3.127	−3.252	−9.769
2	15.685	5.144	11.692	5.395	−25.457	4.867
3	13.264	7.690	10.461	8.116	−21.136	5.538
4	12.260	9.660	6.732	10.227	−45.086	5.874
5	10.993	10.909	8.054	11.575	−26.737	6.107
6	17.785	11.462	13.503	12.176	−24.078	6.229

Small mass ratio ($\mu \leq 0.075$): The differences for both types of characteristic peaks (short-period first peak and first-mode period second peak) are less than 5%. The coupling effect is negligible, and the classical spectrum can be used.

$\mu = 0.085$: The difference for the short-period first peak on the first floor exceeds 5% (the coupled spectrum gives a smaller value), while the differences for the other floors and for the second peak are all less than 5%. For short-period-sensitive equipment on that floor, a local check is required; for long-period equipment, the classical spectrum can still be used.

$\mu = 0.10$: The difference in the short-period first peak exceeds 5% on multiple floors. The coupled system significantly reduces the first peak in the high-frequency range, and the coupling effect cannot be neglected; a coupled spectrum is required.

$\mu = 0.15$ – 0.20 : The reduction in the short-period first peak exceeds 5% on all floors, forming a systematic overall suppression pattern. The difference for the second peak is still less than 5%, indicating limited influence on the first mode.

$\mu \geq 0.25$: The difference for the second peak first exceeds 5% on some floors. The coupling effect begins to influence the dominant frequency band of the first mode, and a coupled model should be used for a full spectral domain evaluation.

Overall frequency-band selectivity: The short-period range is the coupling-sensitive band. As the mass ratio increases, it sequentially experiences the stages: negligible $\rightarrow$ local check $\rightarrow$ non-negligible on multiple floors $\rightarrow$ systematic reduction on all floors. The second peak exceeds the threshold only under very high mass ratios. In summary, the influence of the story mass ratio μ on the floor acceleration response spectrum exhibits frequency-band selectivity and piecewise characteristics:

Short-period range (first peak): For $\mu \leq 0.075$, the effect is negligible, and the classical spectrum can be used. For $\mu \geq 0.085$, a local check for short-period equipment on specific floors is required. For $\mu \geq 0.10$, the effect enters the engineering non-negligible range. For $\mu \geq 0.15$, a systematic overall reduction on all floors occurs (systematic suppression).

Second-peak range (first mode): The difference exceeds the 5% threshold only when $\mu \geq 0.25$, and a coupled model should be used for a full spectral domain evaluation.

3.4. Physical Interpretation of the Coupled Response

As shown in Table 2, for the same story mass ratio, the first three natural periods of the coupled and appended-mass models differ only slightly, and the dominant modal sequence remains unchanged, with the first three modes governed by X-direction translation, Y-direction translation, and torsion, respectively. This suggests that the explicit module–main-frame coupling does not significantly modify the global low-order modal characteristics of the main structure.

In the coupled system, higher-order modes exhibit increased local deformation within the embedded modules, which may reduce the contribution of higher-order global modes of the main frame and relatively enhance the participation of low-order modes.

Since low-order modes primarily govern global lateral deformation and story shear distribution, their relatively higher contribution is consistent with the observed increase in lower-story response and shear demand. The reduced contribution of higher-order modes may also be associated with the attenuation of high-frequency components in floor acceleration response spectra, which provides a possible explanation for the observed suppression of short-period spectral peaks.

Overall, the observed differences in displacement, shear, and spectral response may be related to the redistribution of modal contributions induced by module–frame interaction.

4. Conclusions

This study systematically investigates the influence of story mass ratio on the seismic response of embedded modular buildings by comparing coupled and appended-mass finite element models. The main conclusions are summarized as follows:

1. The story mass ratio is an important parameter influencing the dynamic coupling effect within the investigated prototype. For the selected seismic inputs, cases with relatively lower mass ratios exhibit similar responses between the coupled and appended-mass models, while the influence of module–main-frame interaction becomes progressively more significant as μ increases. This coupling effect leads to amplified lower-story displacement and shear responses, as well as reduced upper-story responses and changes in short-period spectral peaks.
2. The influence of coupling exhibits clear height-dependent and frequency-dependent characteristics under the investigated conditions. Lower floors are more sensitive in terms of displacement and shear responses, while the short-period range of floor acceleration spectra is affected more significantly than the first-mode range.
3. Within the investigated prototype, connection condition, and seismic input range, the appended-mass simplification provides similar response trends for relatively small mass ratios. For higher mass ratios, explicit coupled modeling is recommended to better evaluate the influence of module–main-frame interaction, particularly for lower-story demands and acceleration-sensitive components.

Future work should focus on incorporating additional ground motions and site conditions, investigating the effects of structural height, module stiffness, and connection stiffness, and considering material and geometric nonlinearities under stronger seismic excitations. Experimental validation or benchmark comparisons are also recommended to further improve the reliability and applicability of the coupled modeling approach.

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