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SEISMIC RESPONSE OF HYBRID STEEL-CONCRETE MODULAR BUILDINGS

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Abstract

Modular construction is becoming more popular worldwide due to its various benefits. However, the unique structural characteristics of modular buildings may result in different seismic behaviour compared to conventional structures. This study assesses the seismic response of multi-storey modular buildings with focus on the demands in key collector components, particularly the module-lateral resisting system connections. A nine storey prototype building with reinforced concrete walls is used to examine the dynamic characteristics and seismic response of modular buildings. Firstly, a detailed nonlinear numerical model is created to represent the selected prototype building. Modal analysis and nonlinear response history analyses are then conducted to assess the seismic behaviour. Particular focus is given to the influence of the stiffness of connections and collector elements on the response. Whilst modular buildings with RC walls in the centre can typically be torsionally flexible, it is shown that the flexibility of the seismic collectors can lead to a significant change in behaviour in comparison with conventional buildings. It is also shown that common approaches adopted in codes can significantly underestimate the maximum acceleration response in such multi-storey modular buildings.

Keywords: Modular buildings, reinforced concrete walls, seismic behaviour, dynamic response, nonlinear analysis.

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1 INTRODUCTION

Modular construction involves fabrication of individual sections or modules of a building in a factory or off-site location, which are then transported to the building site for assembly. This approach has gained widespread attention in the construction industry due to its numerous advantages over traditional construction methods. Because of the unique prefabrication-assembly construction method, modular buildings have different structural characteristics compared to conventional buildings [1]. The specific structural characteristics of modular buildings also lead to different seismic response compared to conventional buildings.

The structural behaviour of modular buildings is nonetheless not yet well understood due to limited research. In particular, the demands in seismic collectors, particularly the connections between the modules and seismic force resisting elements, remain unclear. To this end, this study investigated the structural response of multi-storey modular buildings. For this purpose, the response of a representative nine-storey prototype modular building with RC walls is examined through modal analysis and nonlinear response history analysis. A parametric assessment is also conducted, with focus on evaluating the effect of the rigidity of seismic collectors.

2 PROTOTYPE BUILDING

A nine-storey prototype building with structural walls and modules, as illustrated in Figure 1, was designed and assessed. Two U-shaped RC walls with coupling beams were adopted in the centre, and the modules were set around RC walls. A gap size of 200 mm was selected based on the member sizes for the separation between the centrelines of adjacent columns and beams of different modules.

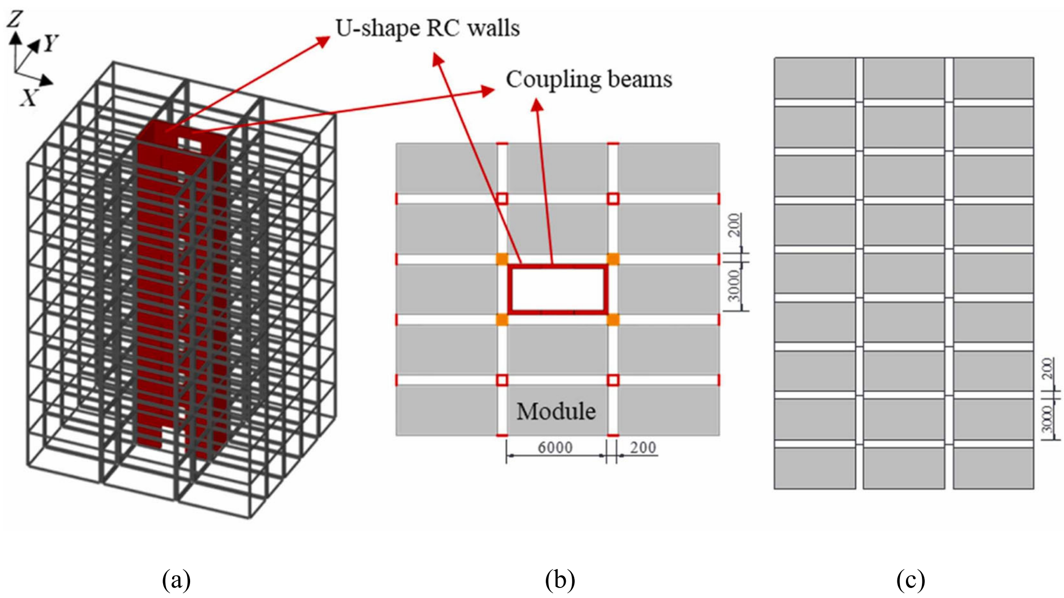


Figure 1: Prototype 9-storey modular building: (a) 3D view, (b) Plan view, (c) Elevation view (Units: mm)

The prototype building was assumed to be in an area of moderate seismicity. The RC walls, described in Figure 2 and Table 1, were considered as the primary lateral force system and designed to ASCE 7-16 (2016) [2] with a response modification factor (R) of 4. The structural frames of the modules were designed to withstand gravity loading only considering the load combinations defined in ASCE. All modules were designed based on the same structural frames, with member sections as listed in Table 1. In this study, a typical inter-module connection with steel connecting plates and shear keys was adopted as shown in Figure 3 [3],[4],[5].

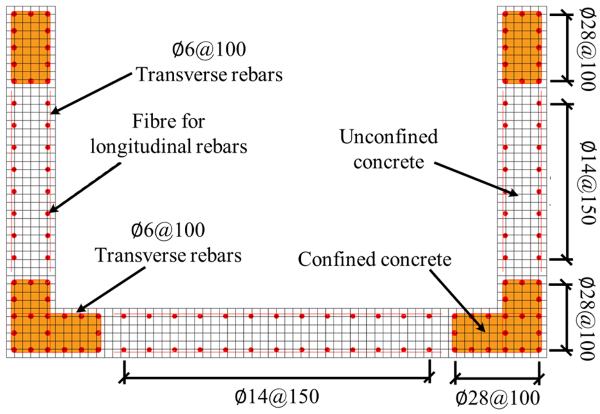


Figure 2: Details of U-shaped RC structural walls

U-shaped RC walls	Design base shear (kN)	1255
	Thickness (mm)	300
	Longitudinal reinforcement ratio	Flange: 2.5% Web: 2.0%
Modular frames	Column: 304.8×152.4×6.4 Ceiling beam in XZ plane: 254×76.2×3.2 Floor beam in XZ plane: 254×76.2×4.8 Ceiling beam in YZ plane: 88.9×50.8×3.2 Floor beam in YZ plane: 101.6×50.8×3.2	

Table 1: Properties of U-shaped RC walls and modular frames

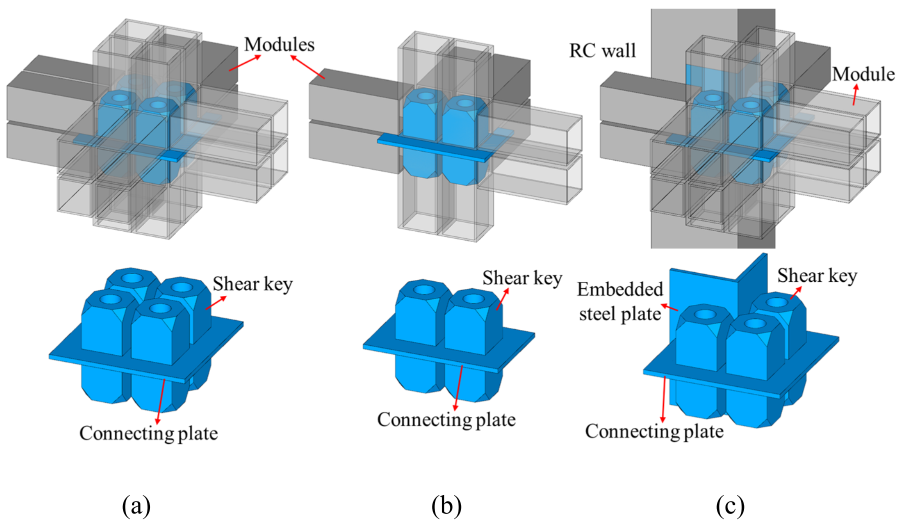


Figure 3: Connection arrangements: (a) internal inter-module connections, (b) external inter-module connections, and (c) connections between modules and RC walls

3 MODELLING PROCEDURES

A detailed numerical model of the building was constructed using the finite element analysis software OpenSees [6]. The "Force-based beam-column element" with a fibre-based cross section was mainly used, in conjunction with the "Concrete01" and "Steel02" material constitutive models, for the walls and frames. In the RC walls, the "Concrete01" material model was used for both confined and unconfined areas, while the strength of concrete in the confined area was increased [7] compared with the unconfined area to simulate the enhancement of confined concrete. In the modular frames, beam elements were rigidly connected with column elements. Two diagonal rigid trusses were included to simulate the floor slabs. The seismic masses were modelled as concentrated masses at the corners, and corresponding gravity loads were applied at the positions of concentrated masses. Connections used in the prototype building, including the inter-module connections and the seismic collectors, were simulated using shell elements and spring elements. Figure 4 illustrates the detailed numerical model used in this study including the RC walls, the modular frames, and the connections.

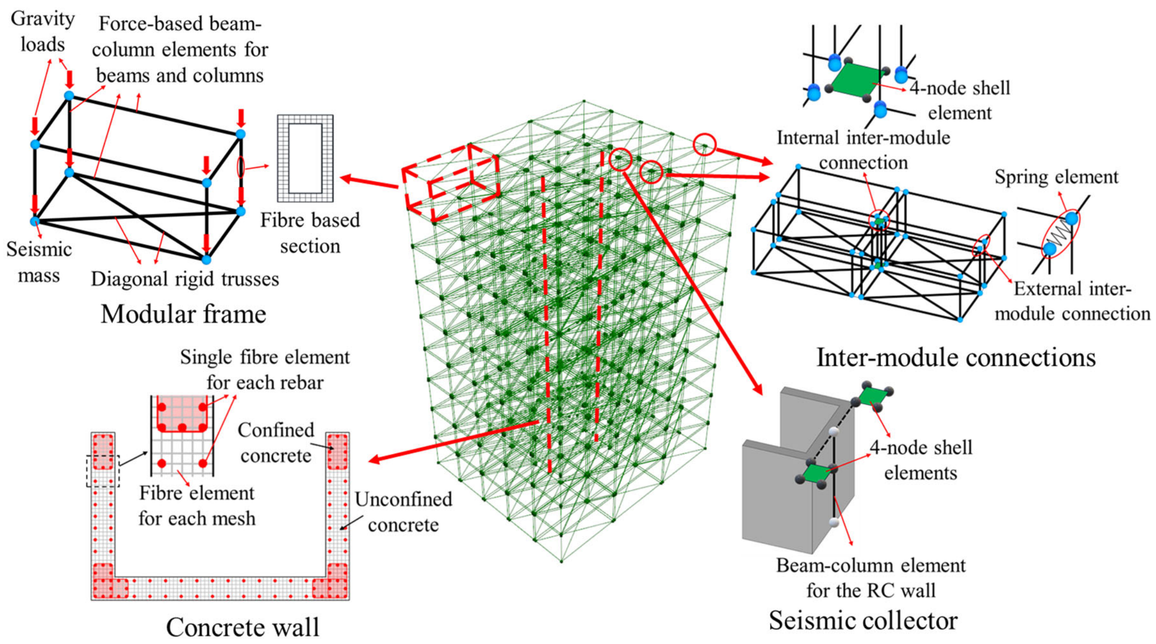
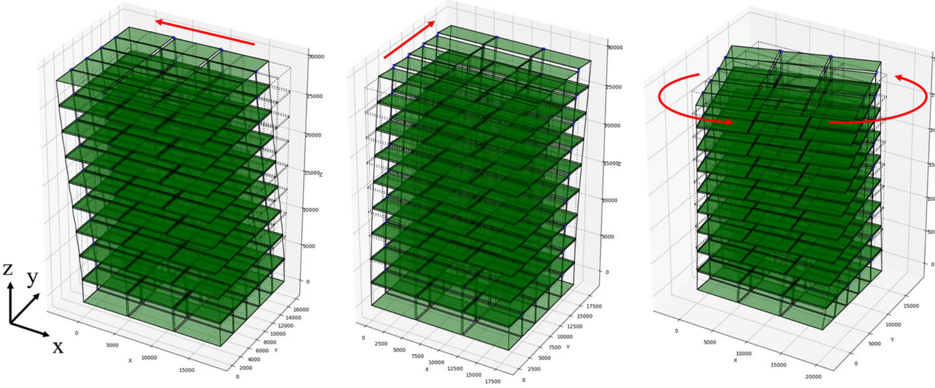


Figure 4: Detailed numerical model of the prototype building

4 MODAL ANALYSIS

Based on the modal analysis results, the first three modes are depicted in Figure 5. The first and second modes are translational in the x-direction and y-direction while the third mode is a torsional mode. The subsequent eight modes, Modes 3 through 10, are all torsional modes and have unique torsional shapes that exhibit torsion of different stories in different directions around the RC walls, as shown in Figure 6(a).

The proportion of the torsional modes in the first 10 vibration modes in the modal analysis indicates a lower rotational stiffness inherent in such modular building. As illustrated in Figure 6(b), the separation of floor slabs of different modules in modular buildings results in diminished torsional stiffness. Additionally, the RC walls, located at the centre of the prototype building, with a relatively small radius to stiffness centre, leads to a low global torsional stiffness. Although the reduction in torsional stiffness results in dominant lateral modes, torsion may still cause additional demands on both the modular frames and RC walls and may have a negative effect on the seismic response, necessitating the need for further assessment.

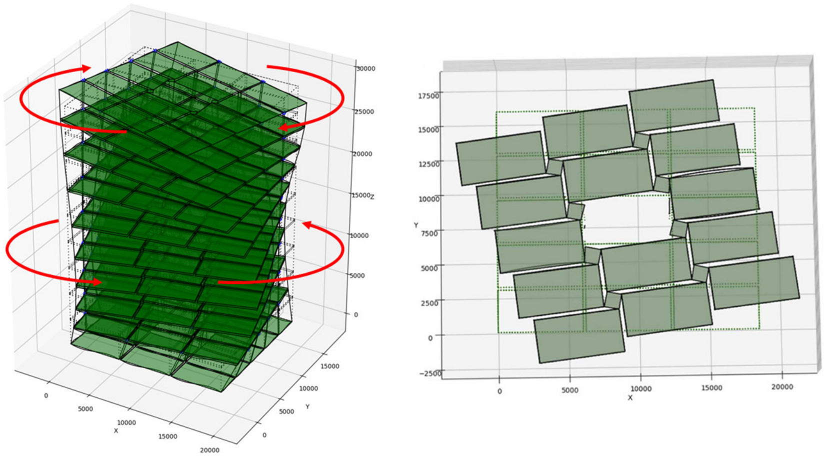


Mode 1: T = 1.011 s
Translation in x-direction

Mode 2: T = 0.797 s
Translation in y-direction

Mode 3: T = 0.629 s
Torsion

Figure 5: Modal shapes and periods of vibration for the first three modes



(a) (b)

Figure 6: (a) Higher torsional modes (b) Amplified relative movement of individual modules under torsion

5 NONLINEAR STRUCTURAL RESPONSE

A set of 22 ground motion (GM) records from FEMA P695 [8] was utilized to conduct nonlinear response history analysis. Two seismic intensity levels were considered in this study, corresponding to the design-based earthquake (DBE) and the maximum considered earthquake (MCE). The MCE-level corresponds to a 2 % exceedance probability in 50 years and the DBE-level ground motions were considered as two-thirds of the corresponding MCE ground level.

In modular buildings, the inertial seismic forces generated are transmitted through the horizontal inter-module connections and transferred to the lateral seismic force resisting system. Based on the nonlinear analysis results, the maximum force transferred through the collectors to the RC walls was extracted. This force, $F_{max,i}$, represents the extracted maximum force at the i^{th} level, and the corresponding maximum acceleration coefficient C_i is defined as:

$$C_i = \frac{F_{max,i}}{w_i} \quad (1)$$

where w_i is the mass at the i^{th} level. The maximum acceleration coefficients under the 22 GMs at both the DBE and MCE intensity levels are depicted in Figure 7.

It is evident that C_i exhibits its minimum value at the 7th storey and typically shows an increasing trend for other storeys in either direction, except for a decrease observed at the 2nd

storey. The pattern of the maximum acceleration coefficients has a different form than that dominated by the first-mode response, indicating the influence of the higher-mode effects on the maximum force in the seismic collectors.

GM ID	Year	Name	Recording station	Site class
1	1994	Northridge	Beverly Hills - Mulhol	D
2	1994	Northridge	Canyon Country-WLC	D
3	1999	Duzce, Turkey	Bolu	D
4	1999	Hector Mine	Hector	C
5	1979	Imperial Valley	Delta	D
6	1979	Imperial Valley	El Centro Array #11	D
7	1995	Kobe, Japan	Nishi-Akashi	C
8	1995	Kobe, Japan	Shin-Osaka	D
9	1999	Kocaeli, Turkey	Duzce	D
10	1999	Kocaeli, Turkey	Arcelik	C
11	1992	Landers	Yermo Fire Station	D
12	1992	Landers	Coolwater	D
13	1989	Loma Prieta	Capitola	D
14	1989	Loma Prieta	Gilroy Array #3	D
15	1990	Manjil, Iran	Abbar	C
16	1987	Superstition Hills	El Centro Imp. Co.	D
17	1987	Superstition Hills	Poe Road (temp)	D
18	1992	Cape Mendocino	Rio Dell Overpass	D
19	1999	Chi-Chi, Taiwan	CHY101	D
20	1999	Chi-Chi, Taiwan	TCU045	C
21	1971	San Fernando	LA - Hollywood Stor	D
22	1976	Friuli, Italy	Tolmezzo	C

Table 2: Details of selected GM set (FEMA, 2009)

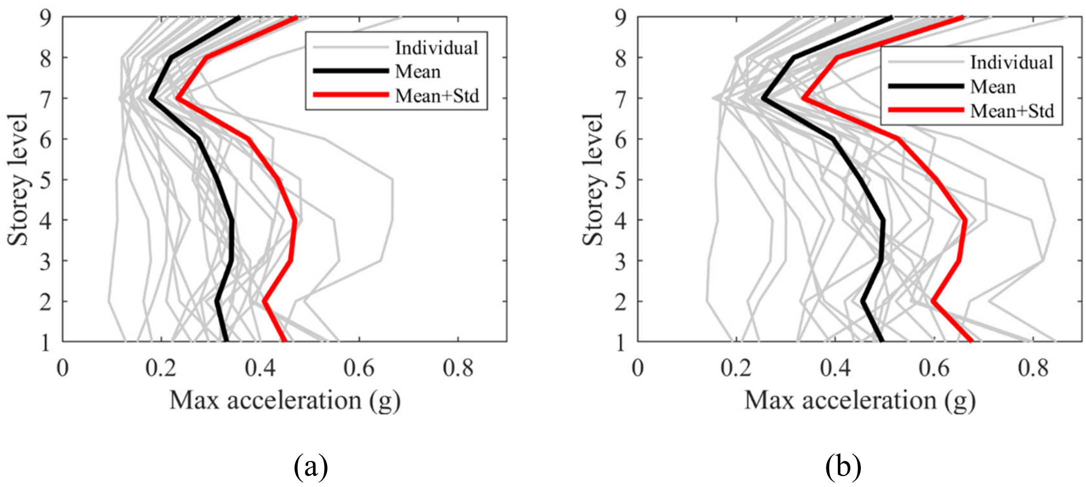


Figure 7: Maximum acceleration coefficients: (a) DBE intensity (b) MCE intensity

6 INFLUENCE OF CONNECTION STIFFNESS

A parametric assessment was conducted to examine the effect of the stiffness of seismic collectors on the maximum demands in these components, by varying the area of the corresponding truss element used in the simplified model from 1000 mm^2 to 8000 mm^2. A rigid connection was also used for comparison.

The maximum acceleration coefficients at all stories, under the 22 ground motions listed in Table 2, were extracted and averaged. Figure 8 presents the mean curves of maximum acceleration coefficients for all the studied models. As the seismic collector rigidity decreased, the maximum acceleration coefficients at stories below the 7th storey increased, with the extent of this increase being greater at lower stories.

The force response in the seismic collectors at the bottom floor was analysed in the frequency domain to explain that the reduction in stiffness of seismic collectors results in a greater maximum force in seismic collectors. This is particularly the case at the bottom floor, as shown in **Error! Reference source not found.** which compares the force response in the frequency domain between models with seismic collectors of 1000 mm^2 and 2000 mm^2 areas under four GMs. The results show that significant contributions from the components occur at a frequency of around 8 Hz for the model with seismic collectors of 1000 mm^2. On the other hand, significant contributions from components at a frequency of 12 Hz occur for the model with seismic collectors of 2000 mm^2. The acceleration level in the case of 1000 mm^2 seismic collectors at 8 Hz was larger than that for seismic collectors of 2000 mm^2 at 12 Hz, which explains the higher maximum acceleration coefficient in the seismic collector with the decrease in its stiffness.

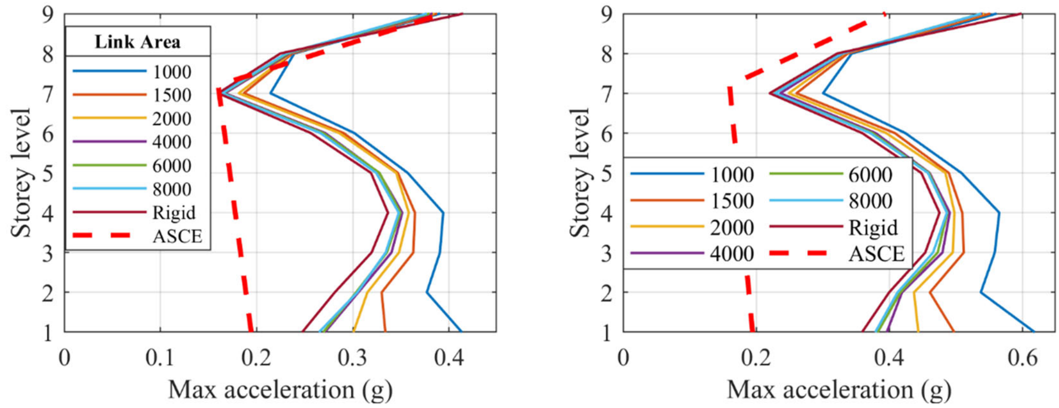


Figure 8: Mean maximum acceleration coefficients under the 22 GMs (a) DBE intensity (b) MCE intensity

It is also interesting to compare the results from the predictions from ASCE 7-16 [2] which includes a clause to determine the design force for seismic collectors and diaphragms. The design acceleration coefficient at the ground level, C_{p0}, is considered as the peak ground acceleration. To reflect the observation that the first-mode response predominantly contributes to floor accelerations at approximately 80 % of the structural height, the design acceleration coefficient at this level, C_{pi}, is calculated to consider the first-mode response as:

$$C_{pi} = 0.9\Gamma_{m1}\Omega_0 C_s \tag{2}$$

where C_s is the design base shear coefficient, Ω_0 is the overstrength factor, and Γ_{m1} is the modal contribution factor. For the design acceleration coefficient at the top of the structure, both the first mode and higher modes are considered and determined as:

$$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_0 C_s)^2 + (\Gamma_{m2} C_{s2})^2} \tag{3}$$

where the terms of $\Gamma_{m1}\Omega_0 C_s$ and $\Gamma_{m2} C_{s2}$ reflect the contribution of the first mode and higher modes, respectively.

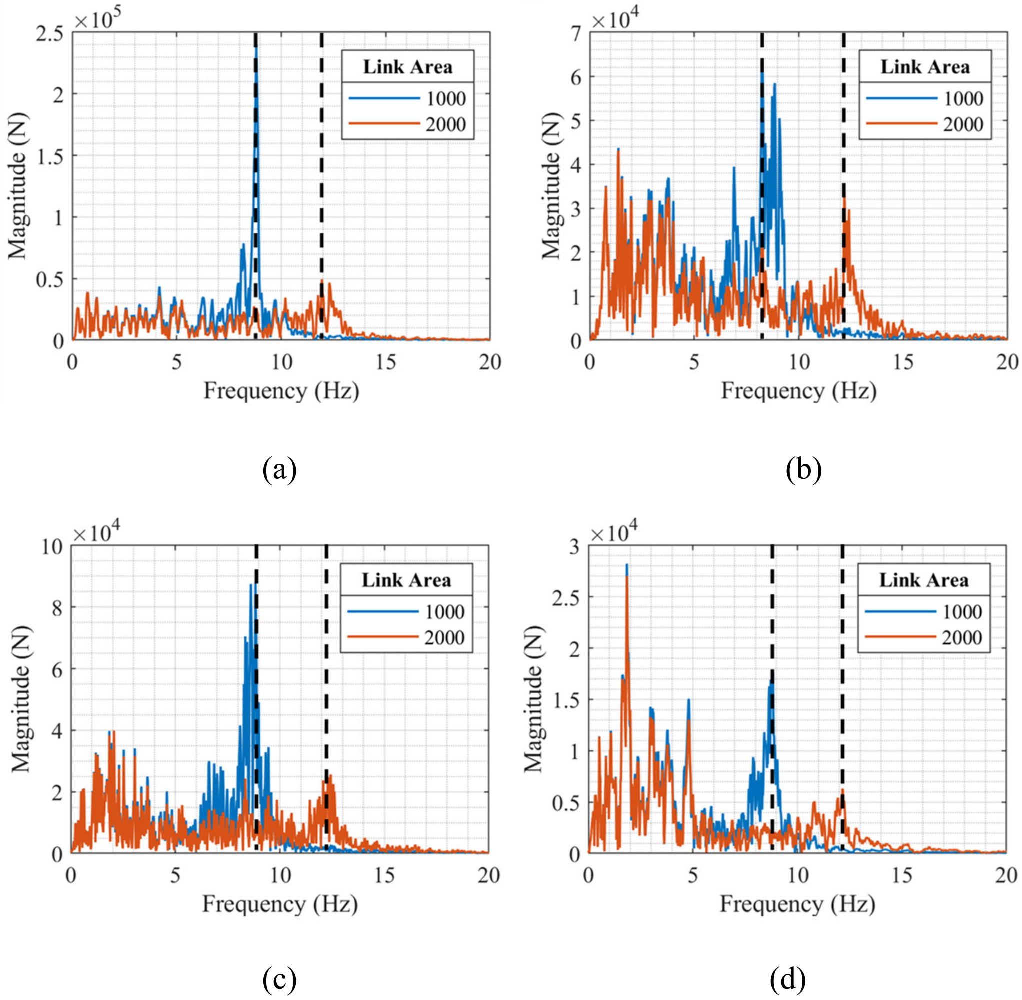


Figure 9: Comparison of force response in frequency domain (a) GM 3 (b) GM 5 (c) GM 13 (d) GM 14

The design acceleration coefficients were also determined using the method in ASCE 7-16 [2] as shown using the red dashed lines in Figure 8. For the DBE-level earthquakes, the calculated design acceleration coefficients matched well for stories above 80 % of the structural height but underestimated the maximum acceleration coefficients considerably for stories below 80 % of the structural height. For the response under MCE-level earthquakes, the calculated design acceleration coefficients underestimated the response at all stories since the method in is mainly associated with the DBE-level events.

7 CONCLUSIONS

The study investigated the dynamic characteristics and seismic response of modular buildings with reinforced concrete (RC) walls, using a nine-storey prototype building. A detailed nonlinear numerical model was created in OpenSees, and subsequently employed for conducting modal and modal and nonlinear response history analyses. A parametric assessment was conducted to further investigate the effect of the rigidity of seismic collectors and the results were compared with the approach used in ASCE 7-16.

Compared to conventional multi-storey buildings, it was shown that the lower torsional stiffness of the diaphragm, due to the discrete inter-connected modules, results in different dynamic characteristics for modular buildings with central RC walls. Moreover, the reduction in the stiffness of the seismic collectors resulted in an increase in the maximum force demands in the collectors. The effect of the seismic collector stiffness is determined by the period of the resulting mode that contributes most to the force response in the seismic collectors. Finally, it was shown that, for the DBE-level events, ASCE 7-16 can predict the maximum acceleration coefficients at stories above 80 % of the height but significantly underestimates the results for stories below 80 % of the structural height.

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