Seismic Performance of Modular Structures with Novel Steel Frame: Light Gauge Slotted Steel Stud Walls

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Modular structures are premanufactured off-site and assembled on-site, leading to reduced on-site works and construction periods and improved quality compared to conventional prefabricated steel structures and reinforced concrete structures. Meanwhile, steel frame-light gauge slotted steel stud (LGSS) walls have been considered to be assembled into the modules due to their excellent thermal- and sound-insulation properties. These structural components will also contribute to the resistance of the modular system when subjected to lateral loads/actions, especially seismic actions. A finite element model was developed to investigate the seismic performance of high-rise modular steel structures with LGSS shear walls. Then, parametric studies were conducted to investigate the influences of the layouts of the LGSS shear walls and structure heights on the lateral stiffness, the deformation modes, and the stress states of the modular steel structures. Observed from the results of a ten-storey modular steel structure infilled with LGSS shear walls, the maximum storey drift reduced by 50.1–79.50% compared to the corresponding pure steel frame structure under the selected high-intensity seismic waves. The results indicate that the studied modular steel structure with LGSS shear walls possesses good seismic resistance and could be a practical and economical choice for future modular constructions.

1. Introduction

Modular constructions have gained increasing interest from designers, researchers, and engineers in recent decades [1–4]. Modules, the basic units of modular structures, are commonly designed to the demands, prefabricated in factories, transported to the desired construction sites, and then assembled into the designed structures, as shown in Figure 1. Compared to the conventional on-site construction techniques and other prefabricated construction technologies, modular structures possess a higher-level prefabrication rate (approximate 70%–95%) and are more convenient (fewer on-site works), time-saving, and environmental-friendly. Thus, they are ideal for buildings designed with repetitive architectural plans and structural layouts [1]. To date, extensive low-rise modular buildings have been constructed and the modular construction techniques are gradually applied for the constructions of mid-rise and high-rise buildings, such as the 60-storey Collins House in Melbourne (height: 184 m).

Since the modular components and the volumetric modules are designed and manufactured in standardised factories, they are commonly well guaranteed to withstand typical loads/actions, such as self-weights and various axial live loads. The key to ensuring structural integrity [5–14] becomes joints/connections, which attracts wide attention from engineers and researchers. Further, the performance of the assembled modular structures has also been mainly focused on the performance under seismic actions and some other transverse loads/actions [15–33]. Thus, extensive investigations have been reported, providing a clear understanding of the behaviour of modular
structures based on these three levels (i.e., single modular component and module level, the joint and connection level, and the assembled modular structural system level). Further, some of these studies [17, 29–32] indicate that the lateral stiffness can also be improved by employing some novel prefabricated shear wall members.

In this context, the seismic performance of multistorey modular structures with novel light gauge slotted steel stud (LGSS) walls, which have been employed in modular buildings in China due to their excellent thermal- and sound-insulation properties and mechanical properties [33, 34], was investigated systematically using the finite element methodology. This study started from the verification of the simplified models proposed for LGSS walls and joints between modules. Comparing the relevant experimental data obtained from literature and the FE results, a bar-spring model developed by Ye et al. [33] was finally adopted for the LGSS walls, and the hinge assumption [5] was then adopted for the joints. Based on these simplifications, an FE model was developed for analysing the behaviour of modular steel structures with LGSS walls under high-intensity seismic waves (two recorded seismic waves and one simulated wave were included). Parametric studies were then conducted to investigate the influences of the structure heights, the presence of the LGSS walls and the arrangement characteristics of the LGSS walls on the lateral stiffness, the internal force distributions, and the deformation modes of the modular steel structures.

2. Verification of the Simplified Models for LGSS Walls and Joints between Modules

Commonly, appropriate simplifications and assumptions can balance the efficiency and accuracy of the research of the modular structure [35, 36]. Thus, simplified models for LGSS walls and joints between modules were selected and verified in this section, which would be the basis for subsequent structural system-level research.

2.1. Simplification of the Light Gauge Steel Stud (LGSS) Walls

Light gauge steel stud (LGSS) walls, as shown in Figure 2, commonly have light self-weight and excellent thermal- and sound-insulation abilities. The LGSS walls comprise several light-gauge steel studs, tracks, insulation infills, and cover boards, which are convenient to be prefabricated in factories and suitable for forming the volumetric modules. The mechanical performance of the LGSS walls under lateral cyclic loading has been experimentally studied, such as those reported by Ye et al. [33] and Geng et al. [34]. The details of the test specimens in these studies are summarised in Table 1. An equivalent bracing model was also proposed by Ye et al. [33] to improve the efficiency of the analyses related to LGSS walls, as schematically shown in Figure 3 and calculated as follows.

The force of the inclined rod AD \( (F_t) \) can be obtained by the following equations:

\[
F_t = \frac{\Delta E \cdot A \cdot \cos \theta}{L_1},
\]

\[
L_1 = \sqrt{L^2 + H^2},
\]

\[
\cos \theta = \frac{L}{\sqrt{L^2 + H^2}},
\]

\[
F_t = \frac{\Delta E \cdot A \cdot L}{L^2 + H^2}.
\]

where \( H \) and \( L \) are the height and the width of the shear wall, respectively, \( L_1 \) is the length of AD, \( P \) is the lateral
force, and \( \theta \) is the angle between \( AD \) and \( BD \); \( \Delta \) is the corresponding lateral displacement, and \( E_bA_b \) is the axial rigidity of the brace.

Set the angles of the columns \( AB \) and \( CD \) as \( \theta_B \) and \( \theta_D \), and set the shear forces of the columns \( AB \) and \( CD \) as \( Q_{AB} \) and \( Q_{CD} \), respectively, which can be calculated by equations (5) and (6). According to the horizontal force balance at the bottom of the wall, the corresponding lateral displacement (\( \Delta \)) and the lateral stiffness (\( K \)) of the shear wall that consider the contribution of the wallboards can be calculated by equations (7)–(9).

\[
\theta_B = \theta_D = \frac{3 \Delta}{2H}
\]

\[
Q_{AB} = Q_{CD} = -\frac{3i \Delta}{H^2},
\]

\[
\theta_B = \theta_D = \frac{3 \Delta}{2H}
\]

\[
Q_{AB} = Q_{CD} = -\frac{3i \Delta}{H^2},
\]

Table 1: Details of the experimental studies conducted on the lateral resistance of LGSS walls.

| Specimen | Overall (length x height) | Hole on the wall (length x height) | C studs | Corner studs | Tracks | Origin |
|----------|---------------------------|-----------------------------------|---------|--------------|--------|--------|
| WA1      | 3600 x 3000               | —                                 | C89 x 50 x 13 | □89 x 100 x 0.9 | U91 x 50 | Ye et al. [33] |
| WA2      | 2100 x 3000               | —                                 | C140 x 50 x 13 | □140 x 140 x 1.5 | U142 x 50 | Ye et al. [33] |
| WA3      | 3600 x 3000               | —                                 | Double C89 x 50 x 13 | □89 x 100 x 0.9 | U91 x 50 | Ye et al. [33] |
| WB1      | 2100 x 3000               | —                                 | C140 x 50 x 13 | □140 x 140 x 1.5 | U142 x 50 | Ye et al. [33] |
| WB2      | 2100 x 3000               | —                                 | C140 x 50 x 13 | □140 x 140 x 1.5 | U142 x 50 | Ye et al. [33] |
| WB3      | 1500 x 3600               | —                                 | C140 x 50 x 13 | □140 x 140 x 1.5 | U142 x 50 | Ye et al. [33] |
| WC1      | 3600 x 3000               | 1500 x 1500                       | C140 x 50 x 13 | □140 x 140 x 1.5 | U142 x 50 | Ye et al. [33] |
| WC2      | 3600 x 3000               | 1500 x 2400                       | C140 x 50 x 13 | □140 x 140 x 1.5 | U142 x 50 | Ye et al. [33] |
| WC3      | 3000 x 3000               | 1500 x 1800                       | C150 x 40 x 15 | HW250 x 250 x 9 x 14 | HW250 x 250 x 175 x 7 x 11 | Geng et al. [34] |
| KJ-1     | 3000 x 3000               | 1500 x 1800                       | C150 x 40 x 15 | HW250 x 250 x 9 x 14 | HW250 x 250 x 175 x 7 x 11 | Geng et al. [34] |

Figure 2: Light gauge steel stud walls.

Figure 3: Bar-spring simplified model for light gauge steel stud walls.

Figure 3: Bar-spring simplified model for light gauge steel stud walls.
\[ Q_{AB} + Q_{CD} + P - 2F_i \cos \theta = 0, \quad (7) \]

\[ \Delta = \frac{PH^2}{3I_c + \left(2H^2L^2E_bA_b/(L^2 + H^2)^{1.5}\right)} \quad (8) \]

\[ K = \frac{P}{\Delta} = \frac{6I_c}{H^2} + \frac{2L^2E_bA_b}{(L^2 + H^2)^{1.5}} \quad (9) \]

where \( I_c \) is the line rigidity of the side studs.

According to the Japanese design code \([37]\), the axial rigidity of the brace of the \( i \)-layer wallboard \( E_{bi}A_{bi} \) can be calculated by \((10)\). For multistorey structures, the lateral stiffness \( K \) can be calculated by \((11)\).

\[ E_{bi}A_{bi} = \frac{1}{2} \left(\frac{H^2 + L^2}{HL/G_i} + \frac{2H}{P_n} \right) \quad (10) \]

\[ K = \frac{P}{\Delta} = \frac{6I_c}{H^2} + \frac{2L^2 \sum E_{bi}A_{bi}}{(L^2 + H^2)^{1.5}} \quad (11) \]

where \( G_i \) and \( t_i \) are the shear modulus and the thickness of the \( i \)-layer wallboard; \( d \) is the deformation of the screw connection, \( P_n \) is the ultimate strength of the screw connection, and \( n_i \) is the number of the screws along the vertical edge on one side of the wall.

The comparisons between the experimental skeleton hysteresis curve (obtained from the experimental data in the reference literature \([33, 34]\)) and the predictions of the bar-spring simplified model are shown in Figure 4. It can be seen that the predictions well matched the test curves, which indicates that the equivalent bracing model is acceptable for predicting the lateral resisting performance of the LGSS walls.

2.2. Simplification of the Joints between Modules. For modular structures, joints and connections are essential for ensuring structural integrity. Meanwhile, many novel joints have also been developed \([5–14]\). However, the cover-plate bolted joints are still widely employed in current constructions, as shown in Figure 5. Commonly, the bolts can be simplified to be tie bars, and the steel plug can be considered using a hinge assumption or a fixed end assumption, as shown in Figure 6. Based on the experimental study for joints between modules presented by Chen et al. \([5]\), as shown in Figure 7, FE models with the hinge end and fixed end assumptions were developed, in which the corner studs and the beams were modelled using beam elements and the tie bars (i.e., the equivalent components for bolts as aforementioned) were modelled using truss elements. The strain-stress relationship model with the strain hardening effect was employed, as shown by equation \((12)\). The steel grade was Q345, whose yield stress was 345 MPa. Young’s modulus and Poisson’s ratio of steel were set to 200 GPa and 0.3, respectively, and the strain hardening modulus was set to 0.01 times Young’s modulus.

\[
\sigma = \begin{cases} 
E \varepsilon, & \varepsilon \leq \varepsilon_y, \\
 f_y + 0.01E(\varepsilon - \varepsilon_y), & \varepsilon > \varepsilon_y, 
\end{cases}
\quad (12)
\]

where \( \varepsilon \) and \( \sigma \) are the strain and the stress of steel, \( E \) is the elastic modulus of steel, and \( \varepsilon_y \) and \( f_y \) are the yield strain and the yield stress of steel, respectively.

Comparisons between the experimental results and the simulated results showed that the results of the FE model with hinge end assumption generally well matched the test results. Thus, the hinge model was finally adopted in this study to simplify the joints between modules when analysing the performance of multistorey modular structures hereafter.

3. FE Model Developed for the Modular Structures

3.1. FE Methodology. Based on the simplifications mentioned above, a FE model was then developed using ABAQUS for multistorey modular steel structures with LGSS walls to investigate their performance under seismic loading. The strain-stress relationship model with the consideration of the strain hardening effect was also used for the steel frames of the modules, the same as the material model used for simulating LGSS walls. Considering the efficiency of the FE simulation processes, all the structural members of the module frame were modelled using beam elements. The LGSS walls were modelled with the validated bar-spring model and the joints were assumed to be hinge ends. The other nonstructural components were ignored in the FE model as they commonly have little effect on the mechanical behaviour of the studied modular structures.

To achieve a better understanding of the seismic performance of the studied modular structures, three earthquake waves were selected as the seismic test loads, including two recorded earthquake waves and one simulated earthquake wave, that is, the earthquake waves recorded at Loleta Fire Station of Cape Mendocino earthquake in 1992 and at North Palm Springs Fire Station #36 of the Landers earthquake in 1992 and a 40-second simulated earthquake wave generated using the SIMQKE_GR programme. The amplitudes of the selected seismic waves were adjusted before applied, according to the relevant provisions stated in GB50011-2010 \([38]\), as shown in Figure 8. The maximum acceleration-time histories of frequent earthquakes and rare earthquakes are 70 gal and 400 gal, respectively.

3.2. Benchmark Model

3.2.1. Details of the Benchmark Model. A 10-storey modular structure model was established as a benchmark model in this study. Based on the volumetric modules developed by Metalurgical Corporation of China (MCC), the dimension was set to 14.4 m, 3 m, and 3 m in length, width, and height of each module, respectively, as shown in Figure 9. The upper and lower beams were manufactured using H-shape wide flange steel HW150 × 150 × 7 × 10 mm and narrow flange steel HN300 × 150 × 6.5 × 9 mm, and the columns are made from square hollow sections with a section width of 200 mm and a...
Figure 4: Continued.
The grade of these structural steel members was Q345, whose nominal yield stress was 345 MPa. Young’s modulus and Poisson’s ratio of steel were set to 200 GPa and 0.3, respectively. The benchmark model was assembled with eight modules on each floor and 80 modules in total for the 10-storey modular structure.

The details of the LGSS walls used in the model are shown in Figure 10. The filler LGSS walls herein were assembled by several perforated C studs (C140 × 70 × 15 × 1 mm), two U tracks as tie beams (U140 × 90 × 1 mm), one-layer plasterboard, and the thermal-insulation infills. To improve the lateral stiffness of the LGSS walls, strengthened LGSS walls were also used.

Figure 4: Comparison of the experimental [33, 34] and the simulated (using the bar-spring model) skeleton curves of the LGSS walls. (a) WA1. (b) WA2. (c) WA3. (d) WB1. (e) WB2. (f) WB3. (g) WC1. (h) WC2. (i) WC3. (j) KJ-1. (k) KJ-2.
in this study by replacing the slotted C-shaped side studs with square hollow tubes (□140×140×5 mm) and increasing the thicknesses of the inner C studs and the U tracks from 1 mm to 2 mm, as shown in Figure 10(b). Using the simplification method in Section 2.1, the strengthened LGSS walls can be simplified to a bar-spring system with an elastic lateral stiffness ($K_e$) of 8499.99 N/mm and a shear capacity ($F_p$) of 291.17 kN.

For the benchmark model, the strengthened LGSS walls were employed as shear walls as shown in Figure 11. The filler LGSS walls were ignored because their contribution to stiffness and capacity of structures is very slight. The constant and live loads in common conditions, which are 3.0 kN/m² and 0.5 kN/m² in total for the roof and 2.0 kN/m² and 2.0 kN/m² for the floors, respectively, were applied to the benchmark modular structure. The self-weight of the LGSS walls could be modelled as linear load with a value of 2.7 kN/m, applied on the frame beams.

### 3.2.2. Results of the Benchmark Model

The seismic performance of the benchmark model and the reference model (steel frame) under the three selected earthquake waves was evaluated. The earthquake waves were applied along with the X-axis, shown in Figure 11(a), which is the axis of the modular structure with a weaker lateral stiffness. The results of the storey drifts ($\theta$) of the benchmark model and the reference model under frequent and rare earthquake conditions are shown in Figure 12 and summarised in Table 2. It can be seen that the maximum storey drifts ($\theta_{max}$) were $3.39 \times 10^{-4}$, $5.43 \times 10^{-4}$, and $3.25 \times 10^{-4}$ rad under the frequent earthquake conditions (Cape Mendocino wave, Landers wave, and the simulated wave with an adjusted amplitude of 70 gal). And the maximum storey drifts increased to $19.02 \times 10^{-4}$, $32.15 \times 10^{-4}$, and $17.61 \times 10^{-4}$ rad under the rare earthquake conditions (the recorded and simulated waves with an adjusted amplitude of 400 gal). In other words, the maximum storey drifts were approximate 1/1842 and 1/311 under the frequent and rare earthquake conditions, while the limits are 1/300 and 1/50, respectively, following the relevant provisions of the Chinese Code T/CECS 507: 2018 [39].

The maximum lateral displacements ($u_{max}$) of all the frame columns were 8.46 mm, 13.98 mm and 7.25 mm under the Cape Mendocino wave, Landers wave, and the simulated wave with a wave amplitude of 70 gal (frequent earthquake condition), respectively. With the increasing amplitude to 400 gal, $u_{max}$ values were increased to 46.73 mm (+452%), 78.44 mm (+461%), and 46.55 mm (+542%), correspondingly. Meanwhile, the torsional deformations of the frame columns were also considered. The maximum torsional displacements ($u_{zmax}$) were 0.72 mm, 0.84 mm, and 0.69 mm for the frequent earthquake conditions and 1.53 mm, 2.30 mm, and 1.34 mm for the rare earthquake conditions.

The maximum stress states are extracted and presented in Figure 13. It can be seen that the maximum stresses of the
Figure 7: Simplification for joints [5]. (a) Simplification for joints between two modules. (b) Simplification for joints between four modules.
structural members were 144.78 MPa, 151.80 MPa, and 143.56 MPa for the three waves under frequent earthquake conditions and 201.59 MPa, 240.07 MPa, and 189.39 MPa under rare earthquake conditions, which means that all the structural members of the benchmark modular structure remained elastic during the applied seismic loads.

Compared with the reference model, for the three waves under frequent earthquake conditions, the maximum storey drifts of the benchmark model reduced by $-72.88\%$, $-58.61\%$, and $-79.50\%$, and the average maximum stress reduced by 30.77%, 28.10%, and 32.55%, respectively; under the rare earthquake conditions, the maximum storey drifts of the benchmark model reduced by $-68.29\%$, $-50.15\%$, and $-77.99\%$, and the average maximum stress reduced by 28.45%, 19.47%, and 40.75%, respectively.

Generally, the results show that the 10-storey benchmark modular steel structure model with strengthened LGSS walls possesses excellent lateral stiffness.

### 4. Influences of Wall Layouts and Structure Heights on the Seismic Performance

Based on the developed FE model, parametric studies were conducted to evaluate the influences of wall layouts and structure heights (number of floors) on the seismic performance of modular structures.
Figure 10: Details of the LGSS walls considered in the FE model. (a) LGSS filler walls. (b) LGSS shear walls. (c) Details of the slotted studs. (d) Cross section of the LGSS shear walls.
4.1. Influences of Wall Layouts

4.1.1. Presence of the LGSS Shear Walls. To evaluate the contribution of the strengthened LGSS walls to the seismic performance of the benchmark modular structure model, a reference steel frame model without walls was also analysed. The storey drifts ($\theta$) are also summarised in Table 2 and plotted in Figure 12. A significant increase can be observed when comparing the storey drifts ($\theta$) obtained from the reference steel frame model with the benchmark modular structure model with strengthened LGSS walls. Under rare earthquake conditions, the maximum storey drifts ($\theta_{\text{max}}$) increased by 215.4%, 100.6%, and 354.3%, with the LGSS shear walls replaced by the filler walls. Also, the most harmful earthquake wave for the reference steel frame model changed to be the simulated earthquake wave.

The lateral displacement-time history curves are plotted in Figure 14. It can be seen that the amplitude of the lateral
Figure 12: Story drifts of the modular structure models. (a) Story drifts under Cape Mendocino earthquake wave. (b) Story drifts under Landers earthquake wave. (c) Story drifts under simulated earthquake wave.
deformations increased and the vibrations were more vital for the modular without the LGSS shear walls. Moreover, the maximum torsional displacements ($\mu_{\text{max}}$) and the maximum stress of the reference model were also raised, as summarised in Table 3. The increases of these critical responses indicated that the seismic performance of the studied modular structure could be improved by employing the LGSS shear walls.

### 4.1.2. Layouts of the LGSS Shear Walls.

Commonly, to meet the requirements of practical use, some of the walls have to be designed with holes for doors and windows or be removed for achieving open space. Thus, the layouts of the shear walls need to be considered. Three layout plans, named Model PA, Model PB, and Model PC, were modelled in this section, as shown in Figure 11.

The storey drifts ($\theta$) of these models are summarised in Table 2 and plotted in Figure 12. It can be seen that the modelled modular structure using the three layout plans still possessed good lateral resistance compared to the benchmark model (328.1 MPa, 328.8 MPa, and 347.1 MPa under rare earthquake conditions, as shown in Table 3). Compared with the reference model (298.4 MPa), the maximum stress increased by 9.95%, 10.19%, and 16.32%, respectively. Generally, based on the key indexes, that is, the maximum storey drifts ($\theta_{\text{max}}$) and the maximum displacements ($u_{\text{max}}$), the third layout plan (model PC) was recommended. However, based on the stress states, the first and the second plans are still worth being considered.

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### Table 2: Story-drifts of the modular structures with different layouts ($\times 10^{-4}$ rad).

| Intensity | Story | Height (mm) | Benchmark model | Reference model | PA model | PB model | PC model |
|-----------|-------|-------------|-----------------|-----------------|----------|----------|----------|
|           |       | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 |
| Frequent conditions (70 gal) | 1 | 3245 | 19.02 | 31.73 | 16.81 | 59.98 | 63.59 | 80.00 | 25.16 | 28.99 | 21.57 | 14.21 | 39.42 | 26.38 | 20.08 |
| | 2 | 6490 | 18.64 | 32.15 | 15.90 | 56.21 | 64.49 | 78.75 | 25.77 | 28.55 | 20.86 | 13.60 | 38.66 | 30.69 | 26.38 |
| | 3 | 9735 | 18.16 | 31.20 | 15.59 | 48.35 | 57.48 | 67.62 | 25.73 | 27.75 | 20.47 | 13.21 | 35.99 | 29.36 | 18.90 |
| | 4 | 12980 | 17.38 | 29.80 | 16.94 | 49.46 | 53.43 | 61.70 | 24.97 | 26.47 | 19.75 | 13.09 | 32.37 | 28.63 | 19.32 |
| | 5 | 16225 | 16.48 | 27.68 | 17.61 | 52.98 | 51.09 | 54.24 | 23.75 | 24.36 | 18.33 | 13.30 | 28.84 | 26.47 | 19.85 |
| | 6 | 19470 | 15.22 | 25.07 | 17.57 | 50.82 | 46.73 | 56.49 | 21.82 | 21.58 | 16.31 | 12.88 | 25.13 | 23.72 | 19.32 |
| | 7 | 22715 | 13.49 | 21.86 | 16.61 | 50.46 | 45.75 | 57.65 | 19.16 | 18.20 | 14.58 | 11.74 | 21.00 | 20.31 | 16.92 |
| | 8 | 25960 | 11.27 | 18.13 | 16.45 | 47.57 | 42.77 | 54.24 | 15.97 | 14.68 | 12.17 | 9.91 | 16.66 | 16.45 | 13.09 |
| | 9 | 29205 | 8.68 | 13.98 | 11.33 | 38.14 | 29.28 | 34.78 | 11.93 | 10.98 | 9.23 | 7.54 | 12.35 | 12.29 | 9.23 |
| | 10 | 32205 | 5.97 | 9.4 | 7.98 | 19.57 | 14.36 | 17.31 | 6.90 | 8.28 | 6.37 | 4.39 | 7.88 | 6.98 | 5.84 |

Note: The three earthquake waves (i.e., the recorded Cape Mendocino wave, Landers wave, and the simulated wave) are abbreviated to W1, W2, and W3 in the table, respectively. And the maximum values are marked in bold.

### 4.2. Influence of Structure Heights (Number of Floors).

Two more modular structure models with six floors and twelve floors were analysed to investigate the influence of the building height on the dynamic performance of modular steel structures with LGSS shear walls subjected to seismic loads. These two models were named P6 and P12, respectively, as shown in Figure 11, in accordance with the numbers of the floors. The storey drifts ($\theta$) of these models are summarised in Table 4 and plotted in Figure 12. The maximum lateral displacements ($u_{\text{max}}$), the maximum torsional displacements ($\mu_{\text{max}}$) and the maximum stress of the assembled structural members were presented in Table 3. With the increasing structural heights from 19,225 mm to 38,695 mm (the number of floors increased from six to twelve), the maximum storey drifts, the maximum lateral displacements, and the maximum stress of the structural members under Landers earthquake wave increased by 23.9%, 170.6%, and 123.0%, respectively. It can be seen that these key response indexes were all tended to increase with the increasing structural heights. However, some intermediate floors' storey drifts ($\theta$) may decrease for higher modular buildings.
Figure 13: Stress states of the benchmark model under seismic loads. (a) Cape Mendocino wave (amplitude: 70 gal). (b) Cape Mendocino wave (amplitude: 400 gal). (c) Landers wave (amplitude: 70 gal). (d) Landers wave (amplitude: 400 gal). (e) Simulated wave (amplitude: 70 gal). (f) Simulated wave (amplitude: 400 gal).

Figure 14: Lateral displacement-time history curves (dark blue: benchmark model, light blue: reference model). (a) Cape Mendocino wave (amplitude: 70 gal). (b) Landers wave (amplitude: 70 gal). (c) Simulated wave (amplitude: 70 gal). (d) Cape Mendocino wave (amplitude: 400 gal). (e) Landers wave (amplitude: 400 gal). (f) Simulated wave (amplitude: 400 gal).
Table 3: Maximum lateral and torsional displacements (in mm) and maximum stress (in MPa) of the studied modular structure models.

| Intensity            | Indices | Benchmark model | Reference model | PA model | PB model | PC model | P6 model | P12 model |
|----------------------|---------|-----------------|-----------------|----------|----------|----------|----------|-----------|
|                      | u\_max | W1   | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 |
| Frequent conditions  |         | W1   | 8.46| 13.98| 7.25| 23.78| 24.7 | 26.44| 14.92| 15.19| 9.16| 7.95| 17.06| 13.3 | 7.37| 9.78| 11.19| 3.47| 5.97| 4.01|
|                      |         | W2   | 0.72| 0.84 | 0.69 | 1.55 | 1.55 | 1.55 | 1.65 | 1.62 | 1.46 | 1.27 | 1.38 | 1.38 | 0.78 | 0.85 | 0.82 | 0.43 | 0.39 | 0.39 |
|                      |         | W3   | 144.8| 151.8 | 143.6 | 208.5 | 210.0 | 212.2 | 223.9 | 229.9 | 226.7 | 225.9 | 238.0 | 226.6 | 228.6 | 239.4 | 229.8 | 103.7| 104.3| 104.4|
| Rare conditions      |         | W1   | 46.73| 78.44 | 46.55 | 133.97| 138.78| 149.18| 85.49| 86.43| 57.69| 43.69| 95.25| 85.53| 40.27| 54.95| 50.52| 17.87| 31.98| 20.97|
|                      |         | W2   | 1.53 | 2.3 | 1.34 | 1.6 | 1.59 | 1.6 | 6.35 | 7.52 | 3.97 | 3.57 | 4.59 | 6.12 | 2.19 | 2.5 | 2.71 | 0.94 | 0.8 | 0.71 | 0.34 |
|                      |         | W3   | 201.6| 240.1 | 189.4 | 281.7 | 298.4 | 319.1 | 287.3 | 328.1 | 307.5 | 235.3 | 328.8 | 266.6 | 308.2 | 347.1 | 302.3 | 114.0| 125.4| 109.5|

Note: The three earthquake waves (i.e., the recorded Cape Mendocino wave, Landers wave, and the simulated wave) are abbreviated to W1, W2, and W3 in the table, respectively.
5. Conclusion

This paper numerically investigated the seismic performance of modular steel structures with LGSS walls. A finite element model was developed and parametric studies were then conducted to investigate the influences of the layouts of the LGSS shear walls and structure heights on the lateral stiffness, the deformation modes, and the stress states of the modular steel structures. Based on the results, the following conclusions can be drawn:

1. The simplified bar-spring model proposed for LGSS walls and the hinge joint assumption were verified using existing experimental studies. The comparison of the experimental and the numerical results showed that the simplified model could characterise the lateral resisting behaviour of LGSS shear walls, and the hinge assumption could balance both accuracy and efficiency.

2. A FE model was developed to investigate modular steel structures’ performance under seismic loads. Compared with the steel frame model, the maximum storey drifts, the maximum lateral displacements, the maximum torsional displacements and the maximum stress of the modular steel structure model decreased notably, indicating that the benchmark modular steel structure possessed good lateral resistance.

3. Parametric studies were then conducted, in which the LGSS shear wall layouts and the structural heights (number of floors) were included. The results showed that these three studied models still possessed good lateral stiffness. The maximum torsional displacement tended to rise when increasing the structural heights.

Data Availability

All the data generated or analysed during this study are included in this published article and also, the datasets analysed to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest regarding the publication of this study.

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Table 4: Story drifts of the modular structures with different structure heights (×10^-4 rad).

| Intensity | Story Height (mm) | Benchmark model | Reference model | P6 model | P12 model |
|-----------|------------------|-----------------|-----------------|---------|----------|
|           | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 | W1 | W2 | W3 |
| **W1**    | 3245 | 3.39 | 4.96 | 3.07 | **12.50** | **13.12** | **15.85** | **2.44** | **3.80** | **2.57** | **3.30** | **4.73** | **3.97** |
| **W2**    | 6490 | 3.20 | 5.37 | **3.25** | 9.61 | 11.06 | 13.68 | 2.06 | 3.78 | 2.34 | 2.94 | 4.59 | 3.76 |
| **W3**    | 9735 | 3.16 | **5.43** | 3.03 | 8.33 | 9.91 | **12.01** | 1.85 | 3.49 | 2.31 | 2.77 | 4.64 | 3.76 |
| **W4**    | 12980 | 3.06 | 5.31 | 2.71 | 8.63 | 9.45 | **10.93** | 1.54 | 2.89 | 2.00 | 2.78 | 4.73 | 3.65 |
| **W5**    | 16225 | 2.92 | 5.01 | 2.53 | 9.34 | 9.01 | 9.61 | 1.06 | 2.07 | 1.36 | 2.90 | **4.77** | 3.77 |
| **W6**    | 19470 | (19225*) | 2.70 | 4.54 | 2.32 | 10.20 | 8.13 | 9.94 | 1.86 | 2.07 | 1.99 | 3.01 | 4.68 | 3.81 |
| **W7**    | 22715 | 2.40 | 3.94 | 2.17 | 10.61 | 8.03 | 10.12 | 3.06 | 4.43 | 3.69 |
| **W8**    | 25960 | 2.00 | 3.26 | 1.95 | 9.30 | 7.19 | 8.75 | 3.01 | 4.05 | 3.43 |
| **W9**    | 29205 | 1.58 | 2.55 | 1.61 | 6.64 | 5.09 | 6.35 | 2.82 | 3.57 | 3.06 |
| **W10**   | 32205 | (32450*) | 1.90 | 2.53 | 1.88 | 3.55 | 2.64 | 2.83 | 2.48 | 3.02 | 2.61 |
| **W11**   | 35695 | | | | | | | 2.05 | 2.37 | 2.16 |
| **W12**   | 38695 | | | | | | | 1.63 | 2.77 | 2.11 |

Note. The three earthquake waves (i.e., the recorded Cape Mendocino wave, Landers wave, and the simulated wave) are abbreviated to W1, W2, and W3 in the table, respectively. And the maximum values are marked in bold.
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