Bending Stiffness of the Floor of the Assembled-Type Light Steel-Modular House

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Abstract
The assembled-type light steel (ATLS) modular house is an emerging type of eco-friendly modular building and has been widely used for buildings with repetitive units. This paper focused on the bending stiffness of the bottom frame of ATLS modular house. The bending stiffness of the full-scale ATLS modular house was tested. It was indicated that the stiffness of the cold-formed thin-walled beam of the bottom floor was weak and the vertical deformation of the beam was obvious. Furthermore, the finite-element (FE) model was developed based on the general purpose finite-element software ABAQUS. The results of the FE model were consistent approximately with the test results in terms of the load–displacement curves at the mid-span of the bottom frame beam, and center of the bottom floor. Based on the verified FE model, the influence of the calcium silicate board, opening of corner fitting, stiffness of bolt connection, and the constraint of corner fitting on the bending stiffness of the bottom frame beam was revealed. The opening of the corner fitting has little effect on the bending stiffness of the floor. The existence of the calcium silicate board can increase the bending stiffness of the floor by 23%. The increase of stiffness of the high-strength bolt connection has less influence on the bending stiffness of the floor. The bending stiffness of the floor decreases by 14.7% when constraining the inside edge of the corner fitting compared to other boundary conditions. The presented experimental and numerical studies will be useful to promote the design of ATLS modular house.

Keywords Assembled-type light steel (ATLS) · Modular house · Bending stiffness · Non-symmetric cold-formed cross-section · Finite-element analysis

1 Introduction
With growing environmental impact, increasing cost, and shortage of skilled labor for the traditional on-site construction [1–4], there is an increasing trend across the construction industry to focus on modular construction. It has been verified that modular construction can be built more effectively than traditional constructions [5–9]. Modular steel construction is prefabricated in the factory and subsequently installed on site. The promising construction method has been increasingly adopted by engineers because of its superiority with regard to the less environmental impact, reduced construction time, less energy consumption, and lightweight for transportation. The typical module units used in modular construction are illustrated in Fig. 1 [4, 10]. In recent years, modular steel construction has been gradually adopted in many areas, such as hotels, apartments, lodging houses, hospitals, offices, and other similar buildings with repetitive units. In addition, modular steel construction can be put into use as soon as possible and it was reported that the Huoshenshan Emergency Hospital was built in 10 days in Wuhan, China, in the fight against COVID-19 [11].
Modular steel constructions can be divided into integral modular houses and assembled-type light steel (ATLS) modular houses [11]. The ATLS modular house, as shown in Fig. 2, is a kind of modular construction with a light steel structure as its framework and a sandwich board as its enclosure. Its framework generally consists of a top assembly frame, a bottom assembly frame, columns, and corner fittings. ATLS modular houses are economical and efficient due to its lightweight, high speed of construction, and greater flexibility in manufacture. Compared to the cold-formed steel buildings, ATLS modular houses have advantages of construction efficiency and removable performance [12–14]. In contrast to traditional modular structure buildings, the frame and enclosure of the ATLS modular house can be utilized repeatedly [15].

Lately, many scholars have focused on modular steel constructions. Many innovations have been reported for the inter-module connection of modular construction. Wang et al. [16] proposed a fully bolted column–column–beam joint for panelised steel-modular structure. The optimal performance of the proposed joint was obtained by pseudo-static tests on four different full-scale joint specimens. Wang et al. [17] proposed the novel coupled modular steel structure that could avoid the structural redundancy. An innovative inter-module connection that exhibited excellent seismic performance was proposed. Zhai et al. [18] investigated two different ways of strengthening the connection to prevent premature fracture of beam-column welds in the module. Both diagonal braces and flange reinforcement could dramatically improve stiffness and bearing capacity of the connection. Lyu et al. [19] analyzed the bending behavior of the proposed splice connection by experiments and numerical simulation. The results indicated adequate reserve of bending resistance for the proposed splice connection under large deformation scenario.

At present, the lateral performance of modular steel construction has been widely researched. Papargyriou et al. [13] studied the lateral load-resisting capacity, deformation capacity, ductility, and energy dissipation of a cold-formed steel strap-braced stud wall system under lateral loading, and the design recommendations were derived for performance-based design. Xu et al. [20] investigated the lateral response of modular steel sub-frames with laminated double beam. Finite-element analysis was conducted based on a series of full-scale experimental tests. Peng et al. [21] proposed a novel design of a corner-supported steel–concrete composite module that was suitable for multi-story modular buildings. The elastic stiffness and load bearing capacity of the proposed composite module under lateral load were 21% and 33% higher than that of the typical steel module, respectively. Wang et al. [11] conducted experiment of a two-story ATLS modular house. The results showed that the ATLS modular house has a suitable degree of ductility and energy consumption, and the simplified method of connections was then proposed. However, most current studies focused on the mechanical properties of the connections and the lateral performance of ATLS module house. Limited experimental studies have been performed on the overall vertical bending behavior of the module unit.

In this study, the full-size ATLS modular house was tested to study the bending stiffness of the ATLS modular
house under vertical load. The deformation pattern and stress distribution were discussed. The finite-element model was developed and verified accordingly based on the finite-element software ABAQUS. The high-strength bolts between column and corner fitting were simulated by spring element. Then, four influence factors on the bending stiffness of the floor were analyzed and discussed in detail. This paper provided useful information regarding the bending stiffness of ALTS modular house, serving as a valuable reference for design of ATLS modular house.

2 Experimental Program

2.1 Test Specimen and Material Property

A full-scale ATLS modular house with dimensions of 6055 mm × 3000 mm × 2790 mm (length × width × height) was selected as a prototype. The bottom frame of the ATLS modular house was tested under vertical multi-stage loading. The specimen composed of a top frame, a bottom frame, and four columns, as shown in Fig. 3(a). To apply the floor load conveniently, a 15 mm-thick calcium silicate board was tied on the bottom frame. The top frame was consisted of four beams, four corner fittings, and some stringers. Every two adjacent beams were welded to a corner fitting. The structure of the bottom frame was similar to that of the top frame. Furthermore, the top frame, the bottom frame, and the columns were connected by high-strength bolts to form a complete structure, as shown in Fig. 3(b). Non-symmetric cold-formed cross-sections were used for both the beams and the columns. Figure 4(a) and (b) illustrates the section profiles of the beams and columns, respectively.

All the nominal dimensions and the measured dimensions of the components are presented in Tables 1 and 2. The middle of the bottom frame beam of the ATLS modular house has two openings with a distance of 1200 mm, which are convenient for the forklift to hoist. The corners of the openings were reinforced by steel angles and plates, as shown in Fig. 4(c). The specific sizes of the components are shown in Fig. 4(c). The longitudinal beams of the bottom frame were designated as LA and LB, and the columns were denoted as ZA, ZB, ZC, and ZD (counterclockwise), respectively, as shown in Fig. 4(d).

Before the loading test, material property test was performed to determine the material properties. Beams, columns, corner fittings, and stringers were manufactured from Q345 steel with a nominal yield strength (f_y) of 345 MPa. Tensile coupons were tested to determine the Young’s modulus (E), ultimate tensile strength (f_u), yield strength (f_y), and elongation (Elo). The average data are listed in Table 3. The ratio between the yield strength and ultimate tensile strength (f_y/f_u) should be no more than 0.85 and the elongation (Elo) should be no less than 20% according to the Tensile Test of Metallic Material (GB/T 228.1–2010) [22]. Additionally, the Young’s modulus of the calcium silicate board was 8874 MPa and Poisson’s ratio was 0.24, which were provided by the manufacturer.

2.2 Test Setup and Instrumentation

The four corner fittings of the specimen were directly placed on a 0.5 m tall rigid support, as shown in Fig. 3(a). As illustrated in Fig. 5, displacement transducers and strain rosettes were used to record the displacements and stress changes of the bottom frame, respectively. Displacement transducers at the supports were denoted as ZA1, ZB1, ZC1, and ZD1. Displacement transducers at the supports were denoted as ZA1–LA5 in the LA and LB1–LB3 in the LB of the specimen. Strain rosettes placed at the longitudinal beam were named as D1–LA5 in the LA and D1–LB3 in the LB of the specimen. Strain rosettes placed at the longitudinal beam were named as a capital letter and a number. For example, “D1” represents the monitoring point at the middle span of LA for recording the strain, where “D” represents the longitudinal local of LA and “1” represents the position of the monitoring point.

![Fig. 3 Assembled-type light steel-modular house](image-url)
Fig. 4 Details of the components

Table 1 Geometric size of the beams (units: mm)

| Type     | L         | t  | H     | A     | B   | C   | D   | E   | G   |
|----------|-----------|----|-------|-------|-----|-----|-----|-----|-----|
| Nominal  | 5635.0    | 3.00 | 121.0 | 90.2  | 8.20| 55.0| 20.0| 50.0| 92.0|
| LA       | 5633.0    | 2.88 | 120.5 | 89.7  | 8.15| 54.5| 19.8| 49.5| 91.8|
| LB       | 5632.0    | 2.87 | 120.5 | 89.5  | 8.17| 54.7| 19.7| 49.7| 91.7|

*L denotes the length of the bottom frame beam; t denotes the thickness of the bottom frame beam.

Table 2 Geometric size of the columns (units: mm)

| Type | h       | t  | H₁   | H₂   | D₁   | D₂   | D₃   | D₄   |
|------|---------|----|------|------|------|------|------|------|
| Nominal | 2450.0 | 4.00 | 210.0 | 152.0 | 25.0 | 30.0 | 30.0 | 25.0 |
| ZA    | 2448.5  | 4.03 | 208.1 | 151.8 | 25.7 | 30.1 | 30.8 | 26.1 |
| ZB    | 2449.2  | 4.06 | 209.6 | 151.5 | 25.7 | 30.3 | 30.7 | 25.7 |
| ZC    | 2448.3  | 4.05 | 208.2 | 151.6 | 25.5 | 31.1 | 30.4 | 26.7 |
| ZD    | 2449.5  | 3.96 | 206.1 | 151.5 | 25.7 | 30.3 | 30.1 | 26.2 |

*h denotes the height of the column; t denotes the thickness of the column.

Table 3 Material properties of steel specimens

| Type          | E (GPa) | fₘ (MPa) | fₜ (MPa) | fₘ/fₜ | Elos(%) |
|---------------|---------|----------|----------|-------|--------|
| Beam, column  | 208     | 365      | 518      | 0.70  | 32.5   |
| Corner fitting| 208     | 360      | 510      | 0.71  | 34.0   |
| Bolt          | 206     | 640      | 800      | 0.80  | i      |
2.3 Loading Protocol

The most unfavorable load condition of the floor was about 4.954 kN/m², considering the most unfavorable combination of dead load and live load (i.e., 1.2 × dead load + 1.4 × live load) in accordance with the Load Code for the Design of Building Structures (GB 5009-2012) [23]. Then, the floorboard was divided into 50 zones averagely to apply the floor load uniformly, as shown in Fig. 6(a). Sandbags and steel plates were carried on the floorboard as the vertical load. Sandbags and steel plates were laid alternatively, and sandbags were put on the first layer. Totally, six layers of the sandbag and steel plate were laid to apply the floor load, as shown in Fig. 6(b). Every step of the load on the
floor was imposed and sustained for 5 min. Preloading was the same as the first step of the nominal test. As illustrated in Fig. 6(c), the loading protocol was diagrammed.

### 3 Test Results and Discussion

#### 3.1 Experimental Phenomena

The vertical deformation of the bottom frame developed gradually with the increasing of the load. The gap between the column and the bottom frame beam can be observed when the floor load reached 1.152 kN/m². When the floor load reached 2.806 kN/m², the gap was expanded evidently, as shown in Fig. 7(a). At the end of the loading procedure, the maximum value of gap measured was 7.5 mm. When the floor load reached 4.454 kN/m², tiny upward deformation occurred on the upper flange of the bottom beam at the opening, as illustrated in Fig. 7(b). It suggested that local buckling occurred at the opening. Then, the speed of deformation of the bottom frame beam became faster and faster by increasing the vertical load.

The loading was terminated when vertical deformation at the middle span of the bottom frame beam reached 90.37 mm. Although the elastic deformation recovered quickly, the residual deformation was obvious in the bottom frame beam, while the sandbags and steel plates were removed. In the later stage of the test, shape change of the bolt was hardly observed, but the bolt hole damaged to some extent, as illustrated in Fig. 7(c). Figure 7(d) illustrates the global flexural deformation of the bottom frame beam.

#### 3.2 Deformation Pattern

The displacement–location curves of the bottom frame beam during the test are illustrated in Fig. 8. Before the floor load reached 4.454 kN/m², deformation curve of the bottom frame beam on the LA was similar to the bottom frame beam on the LB. With the floor load growing up, the opening section of the bottom frame beam on the LA exhibited local buckling, resulting in that the deformation speed of the bottom frame beam on the LA became quicker than on the LB.
Figure 9 shows the load–displacement curve at the middle span of the bottom frame beam. The specimen was in the elastic stage when the floor load was less than 2.00 kN/m². When the floor load reached 1.08 kN/m², the deformation of the bottom frame beam was 12.10 mm, which was approximately 1/500 of the span of the bottom frame beam. When the floor load reached 2.04 kN/m², the deformation of the bottom frame beam was 24.44 mm, which was approximately 1/250 of the span of the bottom frame beam, meeting the requirement of Standard for

(a) Gap between column and corner fitting
(b) Local buckling of upper flange at the opening
(c) Deformation of bolt hole
(d) Flexural deformation of the bottom frame beam

Fig. 7 Experimental phenomena during the test

Fig. 8 Deformation pattern of the bottom frame beam

which was approximately 1/500 of the span of the bottom frame beam. When the floor load reached 2.04 kN/m², the deformation of the bottom frame beam was 24.44 mm, which was approximately 1/250 of the span of the bottom frame beam, meeting the requirement of Standard for
Design of Steel Structures (GB 50017-2017) [24] and Technical Code of Cold-formed Thin-wall Structures (GB 50018-2002) [25]. Figure 9 also indicates that the difference between the two curves was inconspicuous when the floor load was less than 4.454 kN/m$^2$. After the floor load reached 4.454 kN/m$^2$, the deformation speed of section LA3 was faster than that of LB2, causing the local buckling of the section with opening, i.e., section LA2. With the increasing of the floor load, the stiffness of the whole frame decreased gradually and the deformation of the bottom frame beams increased significantly.

### 3.3 Stress Distribution

Figure 10 shows the load–stress curves. Figure 10(a) indicates the stress on the upper flange (D1 and I1) of the bottom frame beam was generally less than the stress on the lower flange (D4 and I3). The lower flange started to yield when the floor load was approximately 3.452 kN/m$^2$, while the upper flange started to yield until the load reached 4.254 kN/m$^2$. It was obvious that the lower flange yielded prior to the upper flange. The stress magnitude of the web was close to the upper flange. The load-stress curves at the opening of the bottom frame beam are shown in Fig. 10(b). It can be seen from Fig. 10(b) that the stress at the lower flange (C3 and E3) was less than the upper flange (C1 and E1), because the reinforcing steel plate was placed closely to the lower flange at the opening. Also, Fig. 10(b) illustrates that the stress of point (E2) which was close to the upper flange at the opening differed from the stress of the lower flange. Compared to the stress condition of point (E2), the difference between the stress of the point (C2) and the upper flange is significant, indicating that the influence of the opening on the stress condition should not be ignored.

Furthermore, Fig. 10(c) and (d) illustrates the load–stress curves at the end of the bottom frame beams. The stress distribution among the curves is dispersive to some extent. The reason is that the components of the specimen are made of cold-formed thin-walled steel with smaller stiffness, leading to that the influence of the machining error is nonnegligible. The stress of all the key points did not reach the yield stress and the components remained in elastic stage under the basic load combination.

### 4 Finite-Element Analysis

#### 4.1 Development of the FE Model

It is already a common scientific research method to use a reliable FE model to analyze the performance of steel structure [26, 27]. In this paper, the commercial software package ABAQUS (version 6.14) was used in simulation of the test. The full-scale FE model of the specimen was established to simulate the bending test of ATLS modular house. There was tiny difference between the nominal geometric dimension and the measured dimension. For convenience, the nominal geometric dimensions of the components were applied in the FE model. The accuracy of the FE model was verified against the test results.

Figure 11 illustrates the FE model developed in this study. The main components in the test were modeled, including the steel column, steel beam, corner fitting, stringers, and calcium silicate board. The stringers were merged with the frame beams. The material properties in the FE model were the same as those in the test. The bilinear elastoplastic constitutive model was adopted for steel, as shown in Fig. 12. $E_s$ is the Young’s modulus of the steel. $\sigma_y$ and $\epsilon_y$ are the yield stress and strain of the steel, respectively. $E_t$ is tangent modulus of steel, which is 1% of $E_s$. $f_u$ is the ultimate stress of the steel, and $\epsilon_u$ is the ultimate strain. The calcium silicate board was used to apply the floor load and was assumed to be elastic during the test. Therefore, linear elastic constitutive model was adopted for the calcium silicate board in the FE model. The shell element with a reduced integration point (S4R) in the ABAQUS element library was employed to simulate all of the components [28]. The bolts between the column and the corner fitting were replaced by springs [29] with a stiffness of $1.5 \times 10^5$ kN/mm according Eq. (1) in the Y direction [30, 31]

$$k_b = 1.6 \frac{A_s}{L_b},$$

(1)
(a) Load-stress curves at middle span

(b) Load-stress curves at the opening

(c) Load-stress curves at the end of the beam on the LA

(d) Load-stress curves at the end of the beam on the LB

Fig. 10 Load–stress curves of the bottom frame beam

Fig. 11 FE model
where, $L_b$ is the bolt elongation length, $A_s$ is cross-sectional area of the screw, and $k_b$ is the spring stiffness of the bolt.

The hinge constraint of the ATLS modular house was simulated by the inside edge constraint of the corner fitting. Four corner fittings in the frame were tied to the beam via surface-to-surface tie constraint. Hard contact for normal contact and frictionless contact for tangential contact were applied to the end plate of the column and corner fitting. The load was applied on the floor of calcium silicate plate directly by uniform load. Giving consideration to accuracy and efficiency of the FE model, the analysis of mesh convergence was conducted. Finally, the mesh size for the corner fitting was 15 mm and the other components was 50 mm.

### 4.2 Validation of the FE Model

The load–displacement curves of the FE model were validated against the test results at the mid-span of the beam, the 1/4 section of the beam, and the center of the bottom frame, respectively. Only one simulated value of the mid-span and 1/4 section of the beam were selected considering symmetry of the specimen. Figures 13, 14 and 15 illustrate a comparison of load–displacement curves of the results between the test and the FE model. As illustrated in Fig. 13, the simulated value was slightly smaller than the experimental value in the linear stage. When the floor load reached 4.02 kN/m², the simulated and experimental values tended to coincide. As illustrated in Fig. 14, the simulated value was close to the experimental value at the
linear stage. When the load reached about 3.84 kN/m², the difference between simulated and experimental values was minimal. The stringer center in the middle of the bottom frame was selected as the bottom frame center and the test value was in good agreement with the simulated value in the whole loading process, as shown in Fig. 15. Therefore, it was reasonable to use the spring to simulate the bolt and the assumed spring stiffness was acceptable. In a word, the FE model developed in this paper was reliable for the floor stiffness analysis of the ATLS modular house.

4.3 Analysis of Influencing Factors

In this section, based on the verified FE model, the influences of the calcium silicate board floor, the opening of corner fitting, the stiffness of bolt connection, and the constraint of corner fitting on the floor stiffness were studied.

4.3.1 Influence of the Calcium Silicate Board

Since the calcium silicate board was connected to the bottom frame beams and stringers by self-tapping nails in practical project, the calcium silicate board contributed partly to the vertical bending capacity of the bottom frame. The FE model without calcium silicate board was developed, as shown in Fig. 16. The vertical load was applied on the stringers directly. Figure 17 illustrates the load–displacement curves at the mid-span of the bottom frame beam with and without calcium silicate board. As illustrated in Fig. 17, both curves presented an upward trend with a decreasing slope and the displacement difference between them increased gradually. When the load standard combination value reached 2.5 kN/m² and 4.0 kN/m², compared with the simulation value of the model with calcium silicate board, the ratios between $W_1$ and $W_2$ of the simulation value of the mid-span displacement without calcium silicate board were 1.23 and 1.33, respectively, as shown in Table 4. The bending stiffness of the bottom frame increased by at least 23%. It suggested that it was necessary to take the calcium silicate board into account in the floor stiffness analysis of ATLS modular house.

Table 4 Comparison of displacement of the model with and without calcium silicate board under load standard combinations (units: mm)

| Load (kN/m²) | $W_1$  | $W_2$  | $W_1/W_2$ |
|-------------|--------|--------|------------|
| 2.5 kN/m²   | 35.95  | 29.30  | 1.23       |
| 4.0 kN/m²   | 73.84  | 55.53  | 1.33       |

$W_1$ denotes the displacement at the mid-span of bottom frame beam without calcium silicate board; $W_2$ denotes the displacement at the mid-span of bottom frame beam with calcium silicate board.
4.3.2 Influence of the Opening of the Corner Fitting

There were two 50 mm x 75 mm openings on the two outer surfaces of the corner fitting, which were convenient for hoisting during the construction stage. As illustrated in Fig. 3(b), the thickness of the side faceplate of the corner fitting was 4 mm. To study the influence of the opening of the corner fitting on the floor stiffness, two models were developed by ABAQUS, which were the models with and without the opening in the corner fitting. Figure 18 illustrates the simulation values of the non-opening and opening models. It was found that the opening of the corner fitting affected rarely on the mid-span load–displacement of the bottom frame beam. Therefore, the opening of corner fitting influences little on the bending stiffness of the floor.

4.3.3 Influence of the Stiffness of the Bolt Connection

The spring was used as the connection between the column and the corner fitting in this paper. The stiffness of the spring in Y direction was assumed to be a reasonable value to simulate the tensile stiffness of the bolt connection based on the verified FE model. The stiffness of the spring in Y direction was assumed to be $K_1 = 1.5 \times 10^3$ kN/mm, $K_2 = 1.5 \times 10^5$ kN/mm, $K_3 = 1.5 \times 10^7$ kN/mm, and $K_4 = \text{rigid connection}$, respectively, to study the influence of the stiffness of the bolt connection. $K_2$ is the stiffness calculated by Eq. (1). Figure 19 indicates the effect of the stiffness of the spring. Comparison of displacement of models with different spring stiffness under load standard combinations is listed in Table 5. As illustrated in Fig. 19 and Table 5, the curves were gradually parallel with a tiny difference when the stiffness was greater than or equal to $1.5 \times 10^5$ kN/mm, while the difference cannot be ignored when the stiffness was less than $1.5 \times 10^5$ kN/mm. It can be predicted that the bending stiffness of the bottom frame increases rarely as the stiffness of the bolt connection increases.

4.3.4 Influence of the Constraint of the Corner Fitting

It was necessary to explore the influence of different constraints of the corner fitting on the floor stiffness. Different boundary conditions of the corner fitting were considered including constraint of the whole plate of the corner fitting, constraint of the outer edge of the corner fitting, and constraint of the inside edge of the corner fitting.

Fig. 19 Effect of the stiffness of the spring

Table 5 Comparison of displacement of models with different spring stiffness under load standard combinations (units: mm)

| Load (kN/m²) | $K_1$ | $K_2$ | $K_3$ | $K_4$ | $K_1/K_2$ | $K_2/K_2$ | $K_3/K_2$ | $K_4/K_2$ |
|-------------|------|------|------|------|----------|----------|----------|----------|
| 2.5         | 31.20| 29.30| 28.30| 26.43| 1.06     | 0.97     | 0.90     |          |
| 4.0         | 64.06| 55.53| 54.32| 49.68| 1.15     | 0.98     | 0.89     |          |

$K_1$, $K_2$, $K_3$, and $K_4$ denote the displacement at the mid-span of the bottom frame beam when the stiffness of the spring equal to $1.5 \times 10^3$ kN/mm, $1.5 \times 10^5$ kN/mm, $1.5 \times 10^7$ kN/mm, and rigid connection, respectively.

Fig. 20 Details of constraint
an FE model was developed and validated against the test results. Furthermore, four factors influencing the bending stiffness of the floor were studied. The following conclusion can be drawn based on the experimental and numerical study:

1. The stiffness of bottom frame beam is weak, and the vertical deformation of the beam was obvious in the test. The reinforcement of the section with opening resulted in the upper flange buckled later during the loading.

2. The FE model can be used to simulate the bending stiffness test reliably. The bolts between the column and the corner fitting were replaced by spring elements.

3. The existence of the calcium silicate board can increase the bending stiffness of the floor by 23%. The opening on the corner fitting has less effect on the bending stiffness of the floor. The increase of stiffness of the bolt connection between the column and the corner fitting has less influence on the bending stiffness of the floor when high-strength bolts were utilized.

4. Different constraint conditions of the corner fitting have an obvious influence on the stiffness of the floor. The effects of constraint of the whole plate of the corner fitting and constraint of outer edge of the corner fitting were the same. Compared to the other boundary conditions, the bending stiffness of the floor decreases by 14.7% when constraining the inside edge of the corner fitting.

### 5 Conclusion

In this paper, an experimental investigation of the floor stiffness of a full-scale ATLS modular house was presented. Experimental phenomena were observed. Deformation pattern and stress distribution were discussed. Then, a finite element model was developed and validated against the test results. Furthermore, four factors influencing the bending stiffness of the floor were studied. The following conclusion can be drawn based on the experimental and numerical study:

1. The stiffness of bottom frame beam is weak, and the vertical deformation of the beam was obvious in the test. The reinforcement of the section with opening resulted in the upper flange buckled later during the loading.

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### Declarations

**Conflict of Interest** The authors have no conflicts of interest to declare that are relevant to the content of this article.

**Ethical Approval** The manuscript has been prepared by the contribution of all authors, it is the original authors’ work, it has not been published before, it has been solely submitted to this journal, and if accepted, it will not be submitted to any other journal in any language. This article does not contain any studies with human participants or animals performed by any of the authors.
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