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Seismic response of steel plate shear wall and timber frame hybrid structures

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Abstract

Steel-timber hybrid structures can bring the benefits of timber and steel together in an effective way. In this paper, the seismic response of steel plate shear wall and timber frame hybrid structures was evaluated using nonlinear static and dynamic analyses. Six-story prototype buildings located in Sichuan, China was designed first. The gravity load resisting system consists of cross-laminated timber floor slabs and timber frames. The lateral load resisting system is a steel plate shear wall system. A wall-to-floor connection between the gravity load resisting system and lateral load resisting system were also proposed. The numerical model of structures with the wall-to-floor connections were then developed within the OpenSees framework. Nonlinear hysteresis response of the steel plate shear wall and the timber joints were explicitly simulated. Analytical results revealed that the hybrid structure possessed an acceptable lateral stiffness and seismic energy dissipation capacity. Furthermore, satisfactory response was observed in the timber system of the hybrid structure.

Keywords: steel-timber hybrid structure, timber frame, steel plate shear wall, seismic design, nonlinear analysis

1. Introduction

In recent years, timber structures are gaining more and more attention due to the advantages of environmental friendliness, ease of assembly and good seismic performance (Cao et al. [1], Yang et al. [2]). A large number of studies have been conducted on timber structural systems. Hybrid timber structures, such as timber-concrete (Chen et al. [3]) and timber-steel hybrid structures (Li et al. [4], Li et al. [5]), have been proposed in addition to pure timber structures. Typically, structural members in a building system are required to provide stiffness, strength, and ductility to resist both gravity load and lateral loads. A new approach of separated system, which is characterized by two substructures of resistance to gravity and lateral force (named gravity load resisting system and lateral load resisting system respectively), was proposed as an alternative solution for steel-timber hybrid structures. With steel-timber hybrid structure, the benefits of timber and steel are brought together in the most effective way. The lateral load resisting system, independently made by steel, provides lateral stiffness for the overall structure with high ductility and energy dissipation capacity of steel. While the gravity load resisting system is a timber structure subjected only to gravity, which take the advantages of timber, such as low density and easy to erect.

Although there has been considerable research on the seismic performance of steel and timber hybrid structures, it is still of an urgent need to understand the behavior of the structural system composed of steel plate shear walls and timber frame gravity load resisting system. Particularly, seismic demands induced in the timber and steel components, as well as the interaction between the gravity and lateral load resisting system, should be investigated.

The purpose of this study is to explore the seismic performance of a new hybrid steel and timber structure via numerical simulations. The research particularly focuses on the seismic forces conducted on the structural members and connections. First, a proposed hybrid structure combining steel and timber is introduced in the study, followed by a detailed design of the prototype building. The investigation is then proceeded with two nonlinear analyses to assess the performance of the proposed steel-timber hybrid structure. Finally, the seismic behavior of the structure is evaluated using both static pushover and dynamic nonlinear response history analyses.

2. Proposed steel plate shear wall and timber frame hybrid structural system

In a steel plate shear wall and timber frame hybrid structure as shown in Figure 1, the lateral load resisting system is a steel frame-steel plate shear wall system distributed around the overall structure and a glued-laminated timber frame structure serves as the gravity resisting system. Steel plate shear walls are designed to yield in tension and buckle in compression in aim to dissipate seismic-induced energy as seismic fuses or sacrificial elements, while steel frames are expected to remain elastic. In addition, a special beam-to-column connection called a post-tensioned (PT) connection is introduced. In the connection, prestressed tendons are tensioned on the steel beam and then anchored at the columns to achieve the self-centering of the steel plate shear wall system and the whole structure. Gap opening occurs to the steel beam-to-column connections when the structure undergoes the earthquake, and gap tends to close under the tensile force caused by prestressed tendon. The gravity resisting system consisting of timber is designed to carry the gravity load. Compared to a structure with a floor system consisting of concrete, timber gravity resisting system can reduce the seismic weight of the structure and the seismic demand of the lateral load resisting system. The inertial force generated by seismic mass on the floor slab would be transferred to the lateral resisting system through the steel-timber wall-to-floor connections between the lateral resisting system and the gravity resisting system.

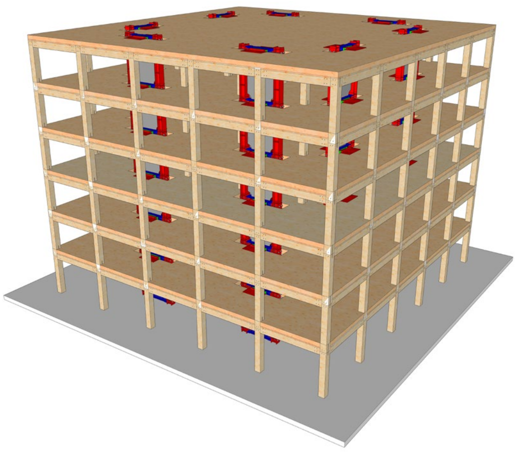

Figure 1: Proposed steel plate shear wall and timber frame hybrid structural system.

2.1. Steel-timber wall-to-floor connections

Due to the possible bending deformation of the steel plate shear wall under lateral force, it may lead to the deformation incompatibility of the structure, or more concretely, undesirable expansion effect, if the timber frame is connected to the steel plate shear wall at the beam-column joint. Therefore, the connection between the steel plate shear wall and the floor is a shear-resistant connector shear key, located at the middle position of the steel beam and the floor slab. The shear key is connected to the steel plate on the floor slab, and the steel plate is connected to the floor slab by using self-tapping screws (STS), which have recently become state of the art in mass timber structures for its large initial stiffness and load-carrying capacity (Dietsch et al. [6]).

3. Prototype building design

A steel plate shear wall and timber frame prototype hybrid building was first designed to determine the component sizes and connection details. Then, its seismic performance was assessed via both nonlinear static and dynamic analysis. The seismic response of the steel and timber structures were determined, and the interaction between steel and timber was investigated.

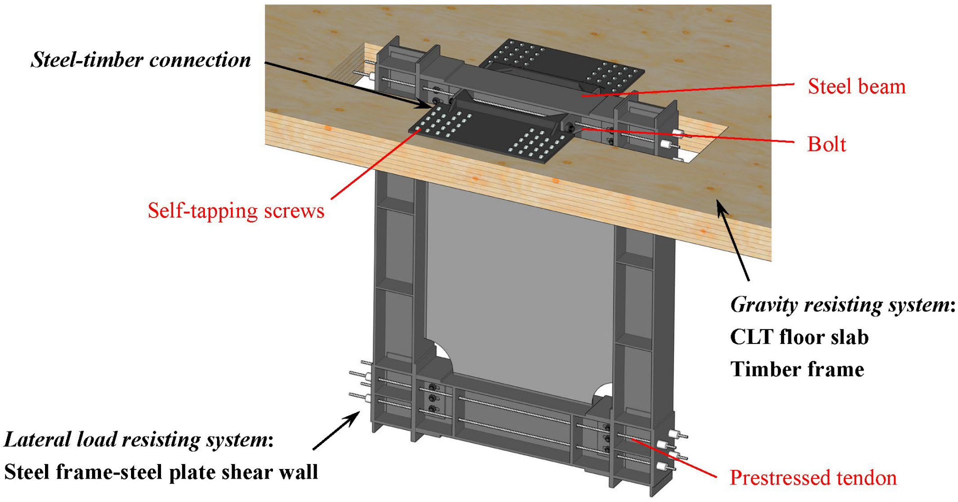

Figure 2: Proposed steel-timber wall-to-floor connection between steel plate shear walls and floor slabs.

3.1. Building selected

Figure 1 shows the 6-story hybrid building selected. The building is located in Dujiangyan, Sichuan, China, which is considered seismic-prone zone in China. According to the code for seismic design of buildings (GB 50011-2010), the fortification intensity is 8 degrees (design ground acceleration is 0.20 g, where g is the gravitational acceleration) and the design earthquake classification is Group 2. The field classification is Class 2 with a characteristic period of 0.40 s. The story height is 3.3 m, while the column spacing is 6.0 m in the two principal axes of the building, as shown in Figure 3.

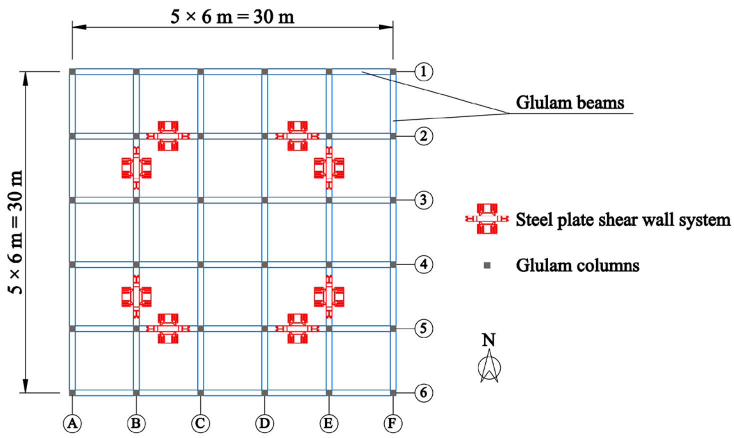

Figure 3: Plan view of the proposed hybrid steel-timber structural system.

3.2. Gravity loads and timber frame system design

The timber frame was designed to resist practical gravity loads, including the dead load, the live load, the snow load and the wind load. All the loads tableted in Table 1 were calculated per the current load code for the design of building structures (GB 50009-2012). The earthquake action on the prototype structures was automatically computed by SAP2000 by means of response spectrum analysis.

The gravity load resisting system consists of cross-laminated timber (CLT) panels resting on the glulam beams. The glulam beams are connected to glulam columns, which transfer gravity loads to the foundation. Grade TC17A glulam was used as the material for the glulam members. The glulam bolted connection with inserted steel plate was adopted as the beam-to-column joint of prototype building.

Q235B structural steel was adopted as the joint steel plates, while bolts with a grade of 8.8s were used as the fasteners.

Table 1: Design loads on the prototype structure.

Load type		Value
Dead load	Floor load	2.4 kN/m^2
	Roof load	2.5 kN/m^2
Live load	Floor live load	2.0 kN/m^2
	Roof live load	0.5 kN/m^2
Snow load		0.315 kN/m^2
Wind load	Reference wind pressure	0.3 kN/m^2
Earthquake action	Seismic fortification intensity	8
	Design ground acceleration (DGA)	0.20 g
	Design characteristic period of ground motion	0.4 s

In the 3D structural model established in SAP2000, the roof truss was designed as a simply supported beam. The timber columns were continuous and hinged at the foundation. The roof continuous beams was hinged at the column top. The timber beams from the other stories were all simply supported and semi-rigid connected to the column with a stiffness of 1.0×10^9 N·mm/rad. The cross sections of all the structural members were estimated in advance according to the vertical loads. The final cross sections of components were determined through successive iterations and optimization in the software. The cross sections of beams and columns used for the timber frame are 300 × 400 mm and 400 × 400 mm, respectively. Beams and columns are assumed to possess the same cross-section along with the building height. The bolt-gusset plate beam-to-column connection in timber frame was then designed based on the SAP2000 model analysis results.

3.3. Seismic loading and steel plate shear wall system design

Due to the gravity and lateral load separated resisting characteristic of this structural system, adding the entire seismic mass of the structure to the lateral load resisting system is feasible when designing the steel plate shear wall system. The mode-superposition response spectrum method was selected for design. The member sizes for the steel plate shear wall system were first selected before the modal analysis of the steel plate shear wall system with a structural damping ratio of 5.0%. The final selected member sizes for infill panel, beams and columns are shown in Table 2.

Table 2: Selected member sizes for steel plate shear wall system.

Story	Infill panel thickness /mm	Beam section	Column section
6	1.8	W21*147	W12*230
5	1.9	W21*147	W12*230
4	1.9	W21*147	W12*230
3	1.9	W18*158	W12*230
2	2.0	W18*158	W12*230
1	2.0	W18*158	W12*230

4. Numerical model development

The three-dimensional fiber-based numerical model developed in the OpenSees program is shown in Figure 4a. The model simulates part of the structure in the east-west direction (Axes 1-3 in Figure 3) in a plane assuming that CLT panels can provide enough rigidity in the plane of floors and the roof and the seismic inertial forces evenly distribute between the steel plate shear walls. The timber columns and beams in the plane of the steel plate shear wall was only modelled, while the columns and beams aligned in Axes 1 and 3 were ignored. The lateral out-of-plane movement of the frame was restrained at the story levels.

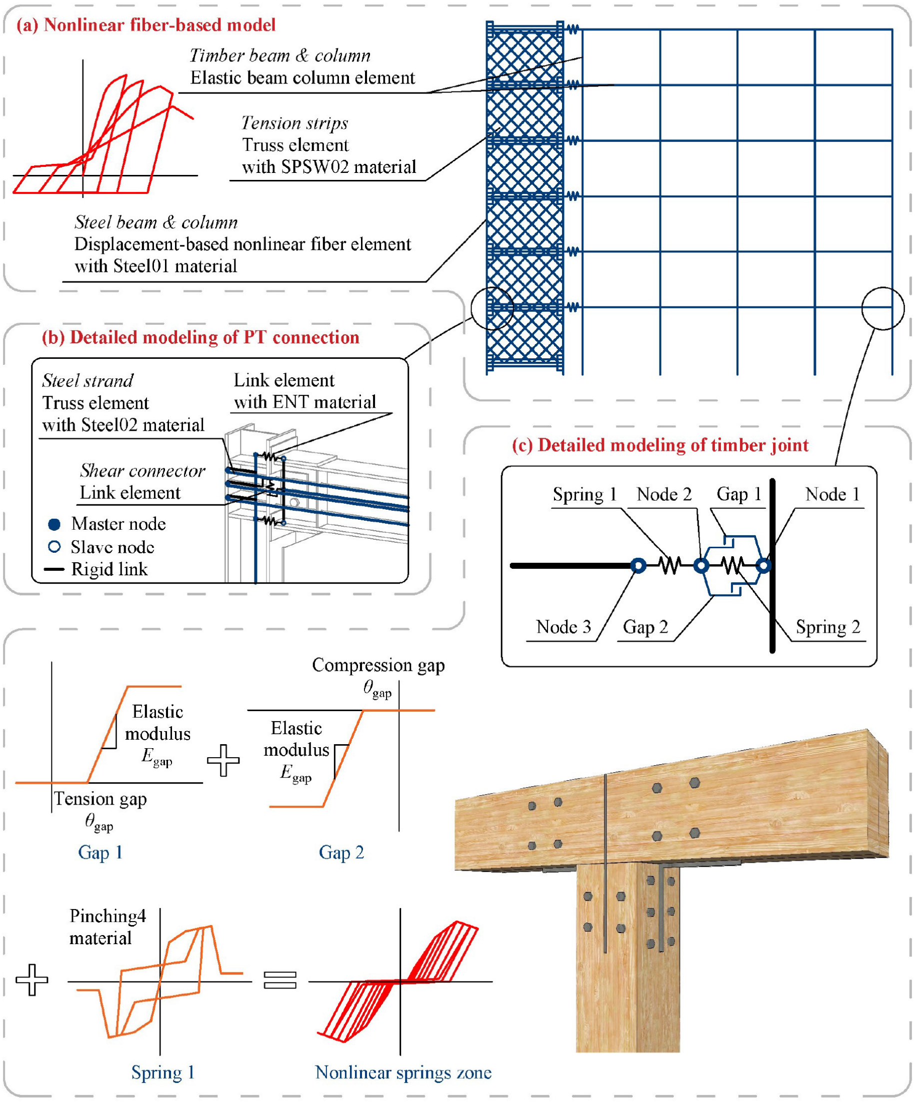

Figure 4: OpenSees model: (a) nonlinear fiber-based model; (b) detailed modeling of PT connection; (c) detailed modeling of timber joint.

4.1. Detailed modeling of steel plate shear wall

The base of the steel columns in the model were pinned. The elastic no tension element available in the OpenSees library was employed to simulate the gap opening mechanism of the self-centering beam-to-column connections. The Giuffre-Megnegtto-pinto material model integrated in Steel02 material model was used to simulate the prestressed tendon.

The steel plate shear wall in the OpenSees model was simulated by a strip model. The strip model comprises a series of tension strips with equal width, which are pin-connected to the surrounding boundary elements. To accurately capture the infill panel behavior, a minimum of 10 strips per panel is required. In addition, unstiffened infill panels were simulated as a series of truss elements aligned in the direction of the tension field in OpenSees. The SPSW02 uniaxial material model available in the OpenSees library was selected to define the inelastic behavior of these truss elements. The accuracy of the strip model in representing the nonlinear behavior of SPSWs has been well verified by Elgaaly [7], and Berman and Bruneau [8].

In the case of multistory SPSWs, the strip model may have staggered node points at the HBE in neighboring stories due to the different tension field angles and number of strips from one story level to another. Thus, a programme that can automatically generate strip models named SPSWGenerator was proposed, with which only necessary parameters are needed to simulate multistory steel-plate shear walls with steel frames.

The configuration of the PT connection is shown in Figure 4b. PT high-strength tendons are used to connect the beam and the column. Reinforcing plates are placed on the beam flanges to limit the yielding and local buckling. In addition, one shear tab plate is welded in the flange of the column. It is bolted to the steel beam to transmit shear forces.

4.2. Detailed modeling of timber frame

Elastic beam-column elements were used to simulate the beams and columns of the timber frame system. The rotational stiffness of the beam-to-column connections was modeled as the nonlinear springs zone. The nonlinear springs zone was modeled by series of zero-length elements with two nodes sharing the same location coordinate, as shown in Figure 4c. Three zero-length elements named Gap 1, Gap 2, and Spring 2 were adopted in this model between Node 1 and Node 2 to realize the sliding behavior of the bolted connections in the initial stage. Elastic perfectly-plastic gap uniaxial material model was adopted for Gap 1 and Gap 2. The corresponding constitutive diagram is presented in Figure 4c. It was found from previous experiments (Shu et al. [9], He et al. [10]) that the initial slip angle of the beam-to-column bolted connection could be generally set as 0.01 rad when the bolt hole was 2 mm larger than the bolt diameter. The tension gap and compression gap, denoted as θ_{gap}, represented the initial slip of the connections and were both set as 0.01 rad. The connection performance under forward loading was assumed to be the same as that under reversed loading. The elastic modulus E_{gap} and yield strength in Figure 4c were set to a large value to ensure that no deformation of the gap material would occur after the initial slip of the connections. The rotational stiffness of Spring 2 was set to a small value to improve the convergence of the model. Spring 1 that connected Node 2 and Node 3 was defined as a zero-length element with "Pinching4" uniaxial material. This command can be used to reflect the "pinched" behavior and the cyclic degradation of strength and stiffness in the load-deformation response of the bolted connections.

The modeling method of column-to-base connection was quite similar to that of beam-to-column connection. Nonlinear springs zone was also adopted for the modeling of column-to-base connection (Figure 4c). However, for the column-to-base connection, the initial slip angle was smaller than that of the beam-to-column connection as there were more bolts in the column-base connection. Therefore, the tension and compression gap for column-to-base connection was set as 0.004 rad considering the number of bolts. The key parameters of the Pinching4 material model for the beam-to-column connections and column-to-base connections referred from Tao [11].

5. Response evaluation

The pushover analysis was conducted to evaluate the lateral performance of the hybrid structure. Furthermore, the nonlinear dynamic analyses were carried out to assess the seismic demands induced in the structure and its components and evaluate the interaction between the gravity and lateral load resisting system.

5.1. Pushover analysis

A pushover analysis was performed by gradually increasing the roof displacement to 5.0% of the total height of the structure, corresponding to 2.5 times the design inter-story drift ratio (the design inter-story drift ratio is 2.0%). The fundamental period of the structure is 1.18 s, and the load pattern is based on the first mode of the structure (the timber frame gravity load resisting system). Figure 5 shows the results of the pushover analysis, and it is worth noting that the displacement-related data shown in the results are all from the timber frame. Figure 5a illustrates the relationship between the inter-story drift of each floor and the roof drift ratio of the structure (i.e., the ratio of the roof displacement to the total height of the structure). It can be seen from the figure that when the roof drift ratio was less than 0.3%, the inter-story drift ratios have neglectable differences between each floor, which shows that the structure was in a uniform deformation shape. As the displacement increased, the inter-story drift ratios of the second and third floors increased more rapidly than other floors. The second floor was the first to reach the design inter-story drift angle of 2.0%, meanwhile the roof drift ratio is 1.39%.

Figure 5b shows the relationship between the base shear and the roof drift ratio of the structure. The whole structure started to yield when the roof drift ratio was 0.36%. As the roof drift ratio reached 1.39%, the base shear of the structure was 1237 kN, and the structure was in the post-yield state according to the pushover curve.

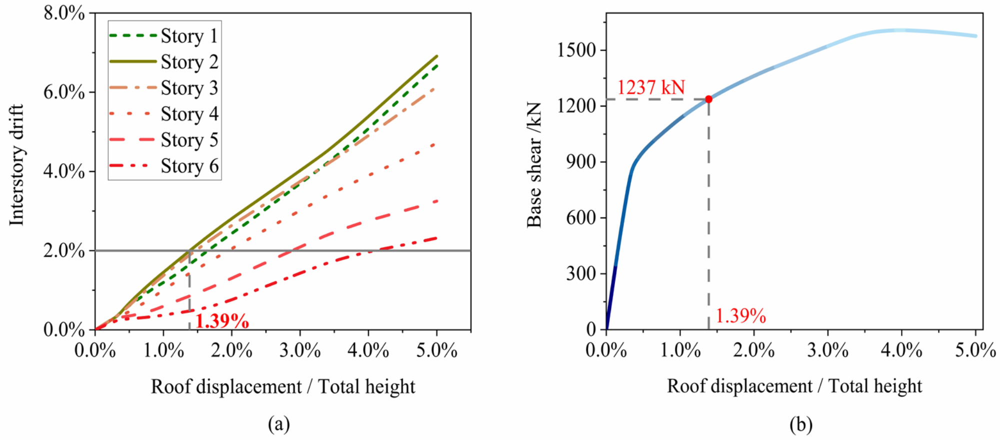

Figure 5: Pushover analysis results: (a) Interstory drifts; and (b) Pushover curve.

The variation of timber beam-to-column joint rotation angles from each story is shown in Figure 6. The rotation angles of the beam-to-column joints at the fifth and sixth storys were almost zero at a base shear force of 1217 kN. No joint existing in the timber frame exhibited more rotation angle than the tension and compression gap for beam-to-column joints θ_{Gap} which is 0.01 rad, when the interstory drift ratio was less than 2.0%. Meanwhile, the stiffness of beam-to-column joints and timber frames was close to zero, demonstrating that the timber frame did not provide stiffness to the whole structure. The timber beam-to-column joint successfully prevented the frame action in the timber system under the applied lateral loads, resulting in controlled seismic demands in timber components. In contrast, the steel plate shear wall lateral load resisting system resisted the induced seismic loads. The designed timber joint can, therefore, be used appropriately to allow the timber frame to sway when the lateral load resisting system deflects during a seismic event without inducing an unintended moment on the timber frame.

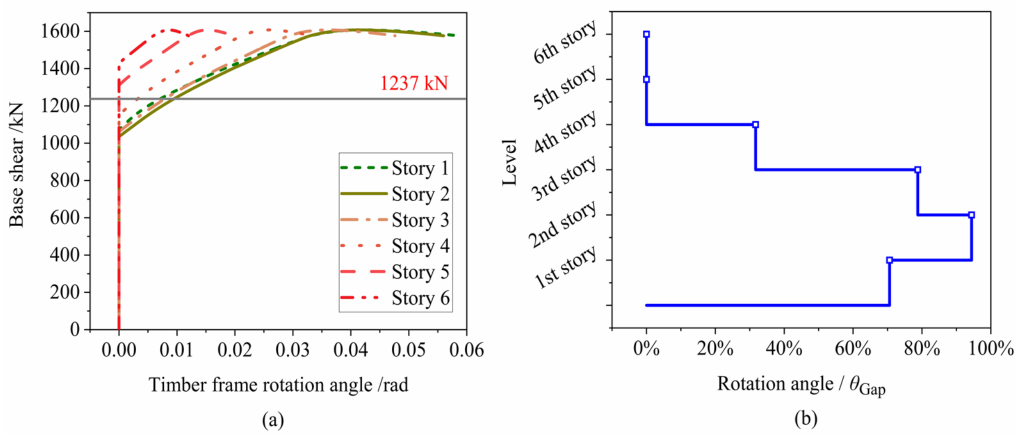

Figure 6: Variation of timber beam-to-column joint rotation angles (θ_{Gap} is the tension and compression zero-stiffness gap for beam-to-column joint).

5.2. Nonlinear response history analysis

Nonlinear response history (NLRH) analysis was performed to evaluate the seismic-induced demands in the proposed hybrid system. The time history response of the inter-story drift and story shear was evaluated, and the peak value of the inter-story drift and floor acceleration was also investigated.

A group of 22 far-field seismic events in FEMA P695 was considered as input for the nonlinear dynamic analysis. These seismic events were obtained from the sites classified as D and C in the United States, and Site Class II which is the assumed classification of the prototype building in China corresponds to the Site Classes C and D in the United States.

Figure 7 shows the time history response of the hybrid system under SFERN/PEL090 motion record. All the measured forces were normalized to their respective capacity. Story shear was normalized to design story shear. As shown in Figure 7a, the maximum inter-story drift took place in Story 1 at 4.0 s.

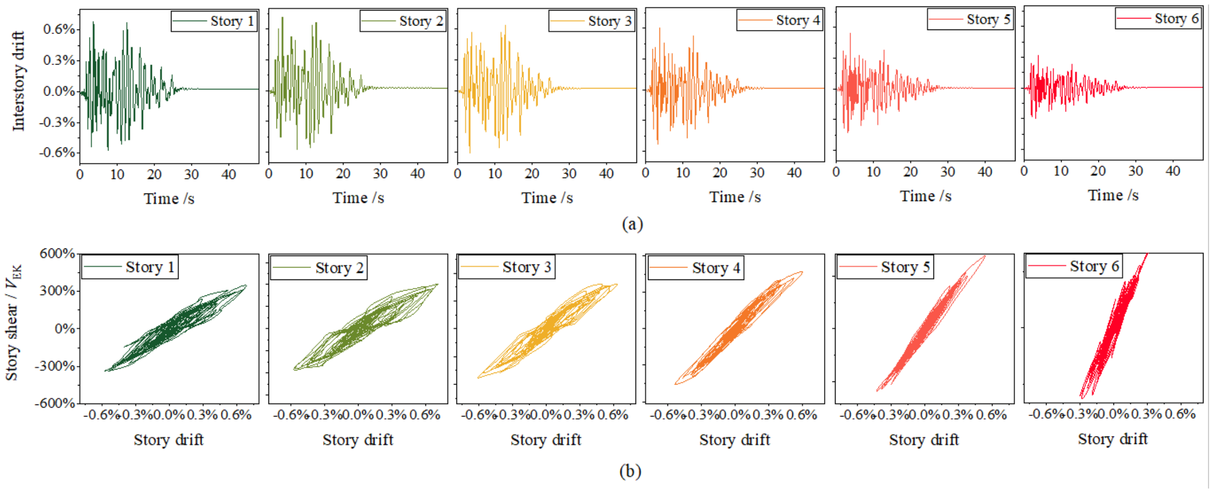

Figure 7: Time history of: (a) Interstory drift; (b) Normalized story shear of the prototype building under SFERN/PEL090 record.

Figure 7b demonstrates the normalized story shear of the prototype building under SFERN/PEL090 record. A large overstrength was observed in all the stories under the selected ground motion record.

Figure 8a shows the inter-story drift profile along the building height and the average and maximum value for each story obtained under 44 motion records. The largest inter-story drift demand equals to 1.06% occurred in the second story. The mean value of the peak inter-story drift in Stories 1 to 6 is 0.67%, 0.77%, 0.72%, 0.57%, 0.43% and 0.29%. The results obtained from NLRH show that the inelastic lateral deformation of the steel plate shear wall is concentrated in Story 2, as observed in the pushover analysis. Figure 8b shows the floor acceleration profile along the building height under each ground motion record and the average and maximum value for each story. The largest floor acceleration response equals to 1.22g occurred in the second story. The mean value of the peak floor acceleration in Stories 1 to 6 is 0.52g, 0.57g, 0.57g, 0.54g, 0.58g and 0.69g.

6. Conclusion

This research introduces a hybrid steel-timber structure as a viable option for resisting seismic forces and gravity loads. A prototype building was selected and then designed, while a new kind of steel-timber connection was also proposed. To examine the seismic performance of the proposed hybrid system and the interaction between the timber and steel systems, a methodology for modeling the structure was developed via the OpenSees program and nonlinear static and dynamic analyses were subsequently conducted. The main findings and conclusions are summarized as follows:

1) The steel plate shear wall system resists lateral seismic loads, while the timber frame structure carries only applied gravity loads.

2) Shear resistance in the steel plate shear wall system acts as ductility sources and seismic energy dissipator under seismic loads.

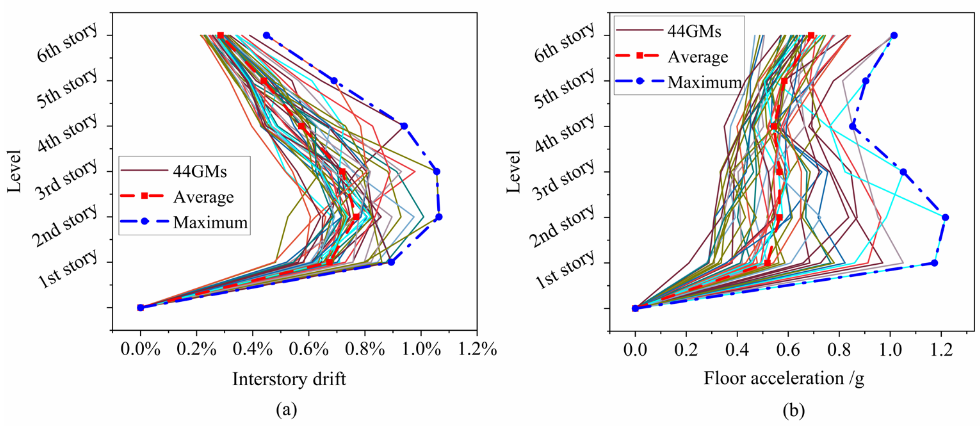

Figure 8: Dynamic response of the prototype building under SFERN/PEL090 record: (a) Inter-story drift profile; (b) Floor acceleration profile.

3) The proposed steel-timber connection efficiently transfers lateral load from the seismic mass of the timber frame to the lateral load resisting system, demonstrating the feasibility of a separated gravity and lateral load resisting system for the hybrid steel-timber structure.

4) The timber beam-to-column joint isolates the timber frame from lateral load resistance, preventing unintended seismic-induced bending moments. No brittle failure was observed according to the proposed detail for the timber beam-to-column joint.

Future work will focus on refining the steel-timber connections and timber beam-to-column joints. Research on experimental validation is necessary to provide further insights into the mechanical performance of this hybrid steel-timber structure.

Acknowledgements

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