Structural damage evaluation of multi-story timber-steel hybrid structures through shake table tests

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Abstract

Timber-based hybrid structures provide a prospective solution for utilizing environmentally friendly timber material in the construction of mid-rise or high-rise structures. This study mainly focuses on structural damage evaluation for a type of timber-steel hybrid structures, which incorporate prefabricated light wood frame shear walls into steel moment-resisting frames (SMRFs). The structural damage of such a hybrid structure was evaluated through shake table tests on a four-story large-scale timber-steel hybrid structure. Four ground motion records (i.e., Wenchuan earthquake, Canterbury earthquake, El-Centro earthquake, and Kobe earthquake) were chosen for the tests, with the consideration of three different probability levels (i.e., minor, moderate and major earthquakes) for each record. During the shake table tests, the hybrid structure performed quite well with visual damage only to wood shear walls. No visual damage in SMRF and the frame-to-wall connections was observed. The correlation

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of visual damage to seismic intensity, modal-based damage index and inter-story drift was discussed. The reported work provided a basis of knowledge for performance-based seismic design (PBSD) for such timber-based hybrid structures.

Keywords: Timber hybrid structure; Shake table test; Damage assessment; Damage index; Inter-story drift.

Introduction

As a renewable structural material, wood has been commonly used in light wood-frame construction for low-rise residential buildings in North America. Recently, with the development of urbanization, many attempts have been made to explore the possibility of building mid-rise or even high-rise structures with wood. One of the attempts is to develop engineered wood products (e.g., cross laminated timber (CLT)) for achieving better material properties and structural performance (Sun et al. 2020, Li et al. 2020a). Another one is hybridization with other construction materials (e.g., concrete and steel). Timber-based hybrid structures can benefit from the advantage of each material if properly designed.

Extensive efforts have been made to explore the hybrid timber solution to use wood in the construction of mid-rise or even high-rise structures. For example, in Canada, Stiemer et al. (2012) proposed a timber-based hybrid structural system, in which CLT shear walls are incorporated into a steel moment-resisting frame (SMRF). Seismic assessment was conducted on such a structural system. The result indicated that the seismic vulnerability was reduced as the CLT shear wall was incorporated (Tesfamariam et al. 2014). Green and Karsh (2012) proposed a concept of finding the forest through the trees (FFTT), in which steel link beams are adopted to connect interior timber core shear walls to glulam frames or exterior timber walls. The FFTT system was expected to possibly reach a maximum building height of 30 stories, and Zhang et al. (2016) evaluated the seismic response of a 12-story FFTT system. Dagenais and Desjardins (2012) described the design process of a timber-reinforced concrete (RC) hybrid structure, which consists of a RC core and glulam frames, in Quebec, Canada. The study provided a demonstration of using wood in the construction of mid-rise buildings. In Japan, shake table tests were carried out to evaluate the engineering demand parameters (EDPs) of timber-RC core hybrid structures (Isoda et al. 2017). The experimental results demonstrated the effectiveness of the RC shear walls in terms of improving the seismic performance of such structures.

Timber-based hybrid structures are also being increasingly studied in China. He et al. (2014) proposed incorporating prefabricated light wood frame shear walls into a SMRF, forming a multi-story timber-steel hybrid structure. The wood shear walls serve as infill walls in the SMRF, aiming to form a cost-effective lateral load resisting system. With the aim to explore the lateral performance of the hybrid structure, reversed cyclic tests were performed on full-scale single-story timber-steel hybrid structures. The experimental results demonstrated that the lateral load-resisting capacity of the hybrid structure was improved as the wood shear walls were incorporated. Verified by the experimental results, numerical models were developed for such hybrid structures (Li et al. 2014). The models served as a robust tool for evaluating the seismic performance of such hybrid structures (Li et al. 2014, 2015, 2018a). Besides, further research efforts have been made in the field of steel-to-wood connections, in order to facilitate the on-site fabrication, increase the energy dissipation capacity and provide self-centering capability for such hybrid structures (Li et al. 2017, 2019, 2020b).

Compared to quasi-static cyclic tests, shake table tests can directly assess the dynamic behavior of structures. Recently, shake table tests were carried out on a four-story timber-steel hybrid structure, and the dynamic responses of the tested structure were reported by He et al. (2018). Although the hybrid structure exhibited excellent seismic performance during the tests, it is still necessary to evaluate the damage of the structure, to better understand the seismic performance and provide further insight into performance-based seismic design (PBSD) for such structures. In this regard, this study focuses on the structural damage evaluation of the timber-steel hybrid structures based on the outcomes of the shake table tests. The visual damage was described with correlation to various seismic hazard levels and was further quantified by a modal-based damage index and inter-story drifts.

Brief Description of Shake Table Tests

Specimen Design

A four-story timber-steel hybrid structure was constructed and tested. The test specimen was assumed to be one representative bay of a prototype office building, as shown in Fig. 1. The location of the building was taken as Sichuan Province, a region of high seismicity in China. The length, width, and height of the prototype building were respectively 34.2 m, 12 m and 13.2 m, and the prototype had 6 bays in length and 3 bays in width. The dead loads were respectively taken as 1.9 kN/m^2 for the floors (including the office rooms and corridors) and 1.8 kN/m^2 for the roof. According to the provisions in Chinese Load Code for Design of Building Structures (MOHURD 2012), live loads of 2.0, 2.5 and 0.5 kN/m^2 were considered for the office room, the corridor and the roof, respectively. Besides, snow load was taken as 0.5 kN/m^2. The design procedure proposed by Li et al. (2018a) was adopted for the preliminary design of the prototype building.

Due to capacity limitation of the shake table, a length scale factor of 2/3 was considered for the test specimen. As shown in Fig. 1, the plan layout of the specimen was 3.75 $\times$8 m, and the total height was 8.8 m. Fig. 2 illustrates the basic unit of the test specimen. The SMRF was assembled with hot-rolled H-section members, which were made from mild carbon steel Q235B with a yield strength of 235 MPa, according to Chinese Code for Design of Steel Structures (MOHURD 2017). For beam-to-column connections of the SMRF, a stub beam was welded to the steel column, and the bolted spliced joint was adopted to connect the stub beam and the rest of beam, to avoid brittle failure of the connection. Besides, the panel zone of the steel column was welded with a steel reinforcing plate, in order to enhance the shear yield strength of the zone. The wood shear walls were prefabricated in a factory. 38mm ×89mm Spruce-Pine-Fir (SPF) No. 2 grade (NLGA 2014) dimension lumber was used as the studs for the wooden frame, and 12-mm thick oriented strand boards (OSBs) were used as the shear wall sheathings. The shear wall sheathing was connected to the wooden frame using nails of 3.3mm diameter and 82.5mm length, and the studs were spaced 406 mm and 305 mm apart for the wood shear walls in *x* and *y* directions, respectively. Previous study (Li et al. 2018, Ling al. 2019, Yang et al. 2020) showed that connections were of great importance for the performance of timber or timber-based hybrid structures. Considering the reliability of transferring shear force between the SMRF and wood shear walls, two kinds of frame-to-wall connections were adopted for assembling the wood shear walls, as shown in Fig. 2. It should be noted that the arrangement of frame-to-wall connections constrained the movement of the wood shear walls, and the constraint was encouraged for such hybrid structures. It was because the analytical model for the hybrid structure could be simplified if shear deformation dominated the deformation mode of the wood shear walls, which could help comprehend design and facilitate the implementation of such hybrid structures in practical application. Besides, the steel beams could be locally reinforced by the frame-to-wall connections. Further research is needed to quantify the reinforcement of steel beams contributed by frame-to-wall connections. Detailed configurations (i.e., nail spacing and sheathing side for infill light wood shear walls, and cross section for SMRF members) are given in Table 1. It should be mentioned that, in order to investigate the influence of stiffness of infill wood shear walls on the seismic behavior of such hybrid structures, the wood shear walls on the 1st and 2nd stories were strengthened by adding more nails during the tests.

According to Chinese Code for Seismic Design of Buildings (MOHURD 2016), additional seismic mass of 7.955 ton was attached on each floor, to simulate additional dead loads and 50% of the live loads for the floors. For the roof, 50% of snow load was considered instead of 50% of the live loads, and additional seismic mass of 5.115 ton was attached on the roof. It should be noted that, the additional seismic mass was calculated with the consideration of a mass scale factor of 0.222, which was calculated based on the length scale factor of 2/3.

Data acquisition

During the shake table tests, the acceleration and displacement responses of the specimen were respectively measured by accelerometers and linear voltage displacement transducers (LVDTs) placed at each story. Besides, in order to obtain the lateral load distribution between the SMRF and wood shear walls, strain gauges were attached to the steel columns, by which the variation of internal forces in the SMRF members was measured. Detailed information for accelerometers, LVDTs and strain gages, as well as the calculation method for lateral load carried by the SMRF or wood shear walls, can be found in He et al. (2018).

Ground motions and testing program

Four ground motion records (i.e., Wenchuan earthquake, Canterbury earthquake, El-Centro earthquake and Kobe earthquake) were chosen as the seismic excitations for the shake table tests. The response spectra of unscaled ground motion records are shown in Fig. 3. Three seismic hazard levels were considered in the shake table tests: minor, moderate and major earthquakes. According to the Chinese Code for Seismic Design of Buildings (MOHURD 2016), the 50-year exceedance probabilities for minor, moderate and major earthquakes were respectively 63%, 10% and 2%. Considering the acceleration scale factor of 2.0, which was calculated based on the length scale factor of 2/3, the peak ground accelerations (PGAs) of the unidirectional ground motions were scaled to 0.14 g, 0.40 g, and 0.80 g for minor, moderate and major earthquakes respectively. Besides, white noise tests were performed to monitor the variation of modal parameters during the tests, for evaluating the structural damage induced by the seismic excitations. Detailed test schedule is listed in Table 2.

Results and Discussions

The primary objective of this paper is to evaluate the damage of the timber-steel hybrid structure. Therefore, the experimental results with respect to damage assessment are presented herein.

Correlation of Visual damage to Seismic Intensity

Although the hybrid structure exhibited excellent seismic behavior during the tests (He et al. 2018), earthquake-induced damage of the test specimen was observed through damage inspection. The visual damage of the test specimen during the tests is listed in Table 2. The visual damage was concentrated in the light wood shear walls, in the forms of OSB sheathing panel corner crushing (or falling off) and nailed connection failure. No visual damage to SMRF or frame-to-wall connections was found.

As shown in Table 2, no visual damage was found under seismic excitations with 0.14 g (corresponding to the hazard level of minor earthquakes). The corner crushing of one OSB sheathing panel on the 3rd story was observed after test 25 (shown in Fig. 4), in which the seismic excitation of scaled El-Centro earthquake with the PGA of 0.40 g (corresponding to the hazard level of moderate earthquake) was applied to the test specimen after strengthened. However, for the test specimen before strengthened, no visual damage was found after test 10, the seismic excitation of which was also the scaled El-Centro earthquake with the PGA of 0.40 g. It should be noted that the peak acceleration, inter-story drift and contribution of the infill walls on the 3rd story of test specimen after strengthened were respectively smaller than those before strengthened, as shown in Table 3. The contribution of the infill walls was quantified by the contribution factor, R, which was defined as:

$$R = 0.5 \times \left(\frac{Q_{\text{w, peak}+}}{Q_{\text{peak}+}} + \frac{Q_{\text{w, peak}-}}{Q_{\text{peak}-}} \right) \tag{1}$$

where $Q_{\text{w,peak+}}$ and $Q_{\text{peak+}}$ = positive peak shear forces in the light wood shear wall and the hybrid system, respectively; and $Q_{\text{w,peak-}}$ and $Q_{\text{peak-}}$ = negative peak shear forces in the light wood shear wall and the hybrid system, respectively. The comparison of response between the test specimen before and after strengthened indicated that the corner crushing of the OSB sheathing panel was probably due to the damage accumulated before test 25. The damage accumulation was detected through the changes in frequencies (Table 2), although no visual damage was observed through damage inspection. After the PGA reached 0.80 g (corresponding to the hazard level of major earthquakes), nailed connection failure was increasingly observed in the light wood shear walls on the 2nd and 3rd stories. Typical failure modes of nailed connections (i.e., flake debonding, local crushing, nail withdrawal and nail head pull-through) are shown in Fig. 5. Besides, corner falling off from the OSB sheathing panel was observed in light wood shear wall on the 1st story, as shown in Fig. 6. It should be mentioned that, the damage levels were different after tests 30, 32, 35 and 39, although the PGAs of the seismic inputs were all about 0.80 g. For example, after the test specimen was subjected to seismic excitation of scaled Wenchuan earthquake with the PGA of 0.80 g (i.e., test 31), no visual damage was further observed through damage inspection. However, after the seismic excitation of scaled El-Centro earthquake with the PGA of 0.80 g (i.e., test 35), nail withdrawal and local crushing of nailed connections were observed along the edges of the wood shear walls on the 2nd and 3rd stories. This was mainly caused by two reasons. One was the damage accumulation during the tests. The other one was the influence of frequency content of the seismic excitations on the dynamic response of the test specimen. To quantify the influence of frequency content, an acceleration amplification factor, β_e, is defined as

$$\beta_e = a_e/\mathrm{PGA} \tag{2}$$

$$a_e = V_b/m_e \tag{3}$$

$$V_b = \sum_{i=1}^{n} m_i a_i \tag{4}$$

$$m_e = \frac{\left(\sum_{i=1}^{n} m_i \Delta_i\right)^2}{\sum_{i=1}^{n} m_i \Delta_i^2} \tag{5}$$

where a_e and m_e = equivalent acceleration and mass of the hybrid structure, respectively; V_b = base shear of the hybrid structure; m_i = lumped mass of the ith floor; and Δ_i = displacement of the ith floor. In this study, the mode shape data, instead of Δ_i, were used for the calculation of m_e for simplification. As shown in Fig. 7, the mode shapes of the hybrid structure were only slightly influenced during the tests and therefore, the mode shape data obtained from test 13 was adopted herein for the test specimen after strengthened. The β_e factors under unidirectional seismic excitations with the hazard level of major earthquakes are illustrated in Fig. 8. With a larger β_e factor, more nailed connection failures were observed through damage inspection. The results indicated that it is sometimes insufficient to adopt the single-parameter, PGA, to quantify the seismic intensity of ground motion records in seismic assessment or structural design for timber-steel hybrid structures. It is recommended that multi-parameters be considered for the determination of seismic intensity. For example, as one of the ground motion parameters, frequency content played a significant role in the evaluation on possible conditions for dynamic resonance, which enhanced dynamic response of the hybrid structure. Therefore, it is necessary to consider the frequency content in future research on the seismic performance of timber-steel hybrid structures. The duration of the ground motion should also be taken into account because the hybrid structure may suffer more damage during a long-duration ground motion than a short-duration one even if PGA and the frequency content of these two ground motions were identical.

Correlation of Visual Damage to Damage Index

During past decades, many models were developed to quantify the damage of structures or structural components (Gosain et al. 1977; Bannon and Veneziano 1982; Park and Ang 1985; Kraetzig et al. 1989; Dipasquale et al. 1990; Fajfar and Vidic 1994; Wong and Wang 2001). Change of natural frequencies has a close correlation with global damage of structures and therefore, the damage model based on change of natural frequencies was adopted to quantify the damage of the hybrid structure in this study. The modal-based damage index was calculated using the following formula (Dipasquale et al. 1990)

$$\delta_{\mathrm{f}} = 1 - \frac{\omega_{\mathrm{final}}^2}{\omega_{\mathrm{initial}}^2} \tag{6}$$

where ω_{intial} and ω_{final} = natural frequencies of the structure before and after earthquake respectively. Fig. 9 shows the damage indices of the test specimen during the test series. For the hybrid structure before and after strengthened, the natural frequencies before earthquake were respectively taken as the frequency measured from tests 1 and 13 (i.e., 3.875 Hz and 3.969 Hz). Therefore, the damage accumulation after tests 1 and 13 were respectively considered in damage indices presented in Fig. 9(a) and (b). As shown in Fig. 9(a), the damage to the test specimen before strengthened occurred while no visual damage was observed through damage inspection. For the test specimen after strengthened, the visual damage to the infill light wood shear wall was initially observed at the damage index of about 0.077 (shown in Fig. 9(b)). With the increase of damage index, more nailed connection failures were observed. The increase of damage index between two white noise tests corresponded to the damage occurred between these two white noise tests. For example, the increase of damage index between tests 34 and 36 corresponded to the damage occurred during the seismic excitations of scaled El-Centro earthquake with the PGA of 0.80 g (i.e., test 35). Therefore, the variation of damage index shown in Fig. 9(b) indicated the test specimen was more vulnerable to the seismic excitations of scaled El-Centro and Kobe earthquakes compared to the seismic excitations of scaled Wenchuan and Canterbury earthquakes. The result was consistent with the damage assessment based on β_e factor. It should be noted that, since no visual damage was found in the test specimen before strengthened, no restoring intervention was made to the test specimen during the process of strengthening (i.e., only additional nails were added to the light wood shear walls on the 1st and 2nd stories but the original part of the hybrid structure was not repaired). Therefore, the damage accumulation before strengthening intervention (shown in Fig. 9(a)) was not considered in the damage indices presented in Fig. 9(b), based on which the damage of the test specimen after strengthened was slightly underestimated.

Correlation of Visual Damage to Inter-Story Drift

As an indicator of earthquake-induced structural or non-structural damage, inter-story drift can provide a direct and simple way for damage assessment or structural design. Therefore, it is a common practice to adopt inter-story drift as the performance criteria in PBSD, although it does not consider the fatigue-type damage of structures. An effort was also made to qualitatively correlate structural damage with inter-story drift in this study. Fig. 10(a) shows the peak inter-story drifts for the test specimen before and after strengthened, under the seismic excitations with the PGA of 0.40 g (corresponding to the hazard level of moderate earthquakes). For the test specimen after strengthened, the corner crushing of OSB sheathing panel was observed after test 25, during which the peak inter-story drift of the 3rd story was 0.263%. However, with a larger peak inter-story drift of 0.275% measured during test 10, no visual damage was observed through damage inspection. Such a difference was due to the damage accumulation between test 10 and test 25, as mentioned above. This indicated that, due to the lack of consideration of fatigue-type damage, it is sometimes inappropriate to adopt inter-story drift for the damage assessment or structural design for the hybrid structure. Fig. 10(b) shows the peak inter-story drifts for the test specimen after strengthened, under the seismic excitations with the PGA of 0.80 g (corresponding to the hazard level of major earthquake). After the peak inter-story drift of 2nd story reached 0.597% (during test 33), the failure of nailed connection was initially observed through damage inspection. Similarly, the failure of nailed connection was initially observed after the peak inter-story drift of the 3rd story reached 0.608% (during test 35). It should be mentioned that, in quasi-static cyclic tests of timber-steel hybrid structures that were carried out by He et al. (2014), the inter-story drift corresponding to the initial observation of the nailed connection failure was about 0.7%, which was slightly larger than those obtained from the shake table tests. Such a difference was probably because, compared to the quasi-static cyclic tests, more damage accumulated during the shake table tests before the initial observation of nailed connection failure.

Summary and Conclusions

In this study, the structural damage of the timber-steel hybrid structures was evaluated through shake table tests. The correlation of visual damage to seismic intensity, calculated damage index and inter-story drift was discussed. Conclusions are summarized as follows:

1. During the shake table tests, the hybrid structure performed excellently with no visual damage to SMRF or frame-to-wall connections observed. Only visual damage in wood shear walls was observed, and the visual damage was a consequence of the combination of deformation and fatigue-type damage. The damage in wood shear walls was initially observed under moderate earthquakes.
2. Not only PGA but also the frequency content of seismic excitations greatly influenced the dynamic response of the timber-steel hybrid structures. It is recommended that multi-parameters (e.g., PGA, frequency content and duration of seismic excitations) should be considered in the damage evaluation and structural design of the timber-steel hybrid structures.
3. The modal-based damage index was able to capture the damage of the hybrid structures, including the damage that was not visually observed through damage inspection.
4. The inter-story drifts corresponding to the initial observation of nailed connection failure was about 0.6%, which was slightly smaller than that obtained from quasi-static cyclic tests. This was because the fatigue-type damage was not considered in the inter-story drift.

This work presents the structural damage evaluation of the timber-steel hybrid structures, aiming to provide technical evidence to facilitate PBSD for such structures.

Data Availability Statement

All data, models, and code generated or used during the study appear in the submitted article.

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Table 1. Configuration of SMRF and light wood shear wall

Story No.	SMRF[a]			light wood shear wall[b]	
	Section of column	Section of steel beam in *x* direction	Section of steel beam in *y* direction	Nail spacing and sheathing pattern in *x* direction	Nail spacing and sheathing pattern in *y* direction
1	H-150×150×7×10	H-125×125×6.5×9	H-125×125×6.5×9	150(75[c]) /150, D	75/150, D
2	H-150×150×7×10	H-125×125×6.5×9	H-125×125×6.5×9	200(100[c])/200, D	75/150, D
3	H-150×150×7×10	H-125×125×6.5×9	H-125×125×6.5×9	150/300, D	100/200, D
4	H-125×125×6.5×9	H-125×125×6.5×9	H-100×100×6×8	125/250, S	150/300, D

[a]Hot-rolled H-section steel members; H-a × b × c × d indicates section with height a, width b, web thickness c, and flange thickness d; and all dimensions in millimeters.

[b]Numbers indicate nail spacing along panel edges and along intermediate supports, e.g., 75/150 indicates that sheathing panels are attached to wood frame members with nails spaced at 75 mm on center along panel edges and spaced at 150 mm on center along intermediate supports; S (D) indicates wood frame sheathed with single-sided (double-sided) OSB panels.

[c]Numbers in brackets indicate the nails spacing on center along panel edges after strengthened.

Table 2. Seismic inputs, frequencies, observations and strengthening intervention during testing

Test	Seismic input			First frequency (Hz)	
	Record	PGA(g) in *x* direction	PGA(g) in *y* direction	In *x* direction	In *y* direction
1	White noise	0.07	-	3.875	-
2	White noise	-	0.07		3.875
3	Wenchuan	0.14	-		-
4	Canterbury	0.14	-		-
5	El-Centro	0.14	-		-
6	Kobe	0.14	-		-
7	White noise	0.07	-	3.875	-
8	Wenchuan	0.40	-		-
9	Canterbury	0.40	-		-
10	El-Centro	0.40	-		-
11	Kobe	0.40	-		-
12	White noise	0.07	-	3.719	-
Strengthening intervention: On the 1st and 2nd stories, more nails were added on center along panel edges of the light wood shear walls in *x* direction.					
13	White noise	0.07	-	3.969	-
14	White noise	-	0.07	-	3.875
15	Wenchuan	0.14	-	-	-
16	Canterbury	0.14	-	-	-
17	El-Centro	0.14	-	-	-
18	El-Centro	0.14	0.0595	-	-
19	Kobe	0.14	-	-	-
20	Kobe	0.14	0.0595	-	-
21	White noise	0.07	-	3.922	-
22	White noise	-	0.07	-	3.875
23	Wenchuan	0.40	-	-	-
24	Canterbury	0.40	-	-	-
25	El-Centro	0.40	-	-	-
Observed damage: On the 3rd story, the corner of one OSB sheathing panel was crushed, as shown in Fig. 4.					
26	El-Centro	0.40	0.17	-	-
27	Kobe	0.40	-	-	-
28	Kobe	0.40	0.17	-	-
29	White noise	0.07	-	3.813	-
30	White noise	-	0.07	-	3.875
31	Wenchuan	0.80	-	-	-
32	White noise	0.07	-	3.813	-
33	Canterbury	0.80	-	-	-
Observed damage[a]: On the 2nd story, the damage of nailed connections along the edges of a light wood shear wall was observed in the form of flake debonding.					
34	White noise	0.07	-	3.813	-
35	El-Centro	0.80	-	-	-
Observed damage[a]: On the 2nd and 3rd stories, the damage of nailed connections along the edges of light wood shear walls was observed in the forms of local crushing and nail withdrawal.					
36	White noise	0.07	-	3.719	-
37	Kobe	0.80	-	-	-
Observed damage[a]: On the 1st story, the corner of one OSB sheathing panel was crushed, as shown in Fig. 6. On the 2nd story, the damage of nailed connections around the corner of light wood shear walls was observed in the forms of nail head embedment and nail withdrawal.					
38	White noise	0.07	-	3.625	-
39[b]	Kobe	0.75	-	-	-
Observed damage[a]: On the 2nd and 3rd stories, the damage of nailed connections along the edge of light wood shear walls was observed in the forms of local crushing, nail head embedment and nail withdrawal.					
40	White noise	0.07	-	3.531	-
41	White noise	-	0.07	-	3.828

[a]Typical failure modes of nailed connections (i.e., flake debonding, local crushing, nail withdrawal and nail head embedment) are shown in Fig. 5.
[b]The PGA of the Kobe earthquake in test 39 was scaled down to 0.75 g because the overturning moment of the test specimen exceeded the limitation of the shake table during test 37.

Table 3. Response of 3rd story of the test specimen before and after strengthened

Test specimen	Seismic excitation	Story No.	Peak acceleration(g)	Peak inter-story drift	Contribution factor
Before strengthened	El-Centro with PGA of 0.4g in *x* direction (i.e., test 10)	3	0.6960	0.275%	0.6648
After strengthened	El-Centro with PGA of 0.4g in *x* direction (i.e., test 25)	3	0.6668	0.263%	0.6483