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A seismic behavior and numerical model of narrow paneled cross-laminated timber building

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ABSTRACT

A national research project to investigate proper structural design methods for cross-laminated timber (CLT) buildings has been initiated by a subsidiary of the Ministry of Land, Infrastructure, Transport and Tourism of Japan since 2011. In the final stage of the project, shaking table tests were conducted for CLT buildings designed according to a proposed structural design procedure which confirmed damage limit state, safety limit state, allowable stress, and ductile factors, etc.

This paper presents results of shaking table testing for full-scaled CLT building and a design procedure. The three different systems examined are buildings composed of narrow panels, wide panels with an edge tensile connection, and wide panels with an edge tensile connection for each no-window shear part. The focus of this paper is the building of narrow panels. This building system is suitable for midrise CLT building with high ductility produced through rocking. The structure was shown to behave well during severe strong motion as specified in the Japanese building standard law and to have survived the 1995 Kobe earthquake despite the occurrence of a compressive rupture in shear walls which are support elements against the vertical load. Story shear capacity calculated from a numerical model and element tests (such as connections) were safely evaluated; but to evaluate the capacity correctly, further research is required in the element and system levels. Though a variety of undetermined issues and challenges remains, the Building Standard Law and Notification for three different CLT construction systems was enforced in April 2016 to ensure the construction of safe CLT buildings.

1. Introduction

Though Japanese domestic forests planted after 1945 have grown enough to be used as building components or elements of composite material, they have never been sufficiently used and maintained to plant a subsequent generation. So, it is a major concern that multiple functions of the forest, such as watershed and land conservation, have declined. To address this situation, “The Act for Promotion of Use of Wood in Public Buildings” was established and enacted in October 2010. Meanwhile, construction employing cross-laminated timber (CLT) panels was offered as a method for large scale and mid-rise building in domestic and foreign markets. According to Japanese building standards law, special permission using time history analysis is normally required to build a construction composed of new structural materials and members, such as CLT was not adequately covered. The use of time history analysis is widespread in Japan, but it is difficult to employ for mid-rise or low-rise residential buildings, mainly due to the structural design cost and length of design time. To save costs and avoid delays, a definition of the standard strength of materials and members and the development of a structural design procedure are required. In addition, typical structural details are required. The capacity spectrum method for seismic design, which is known as the “limit strength calculation method” in Japan is applicable for structural design. The structural specification of building method can be eliminated in this method, but a definition of standard strength and procedure for calculation, such as a damping factor, is required. A national project of research and development on CLT construction for solving the above problems was started in 2011.

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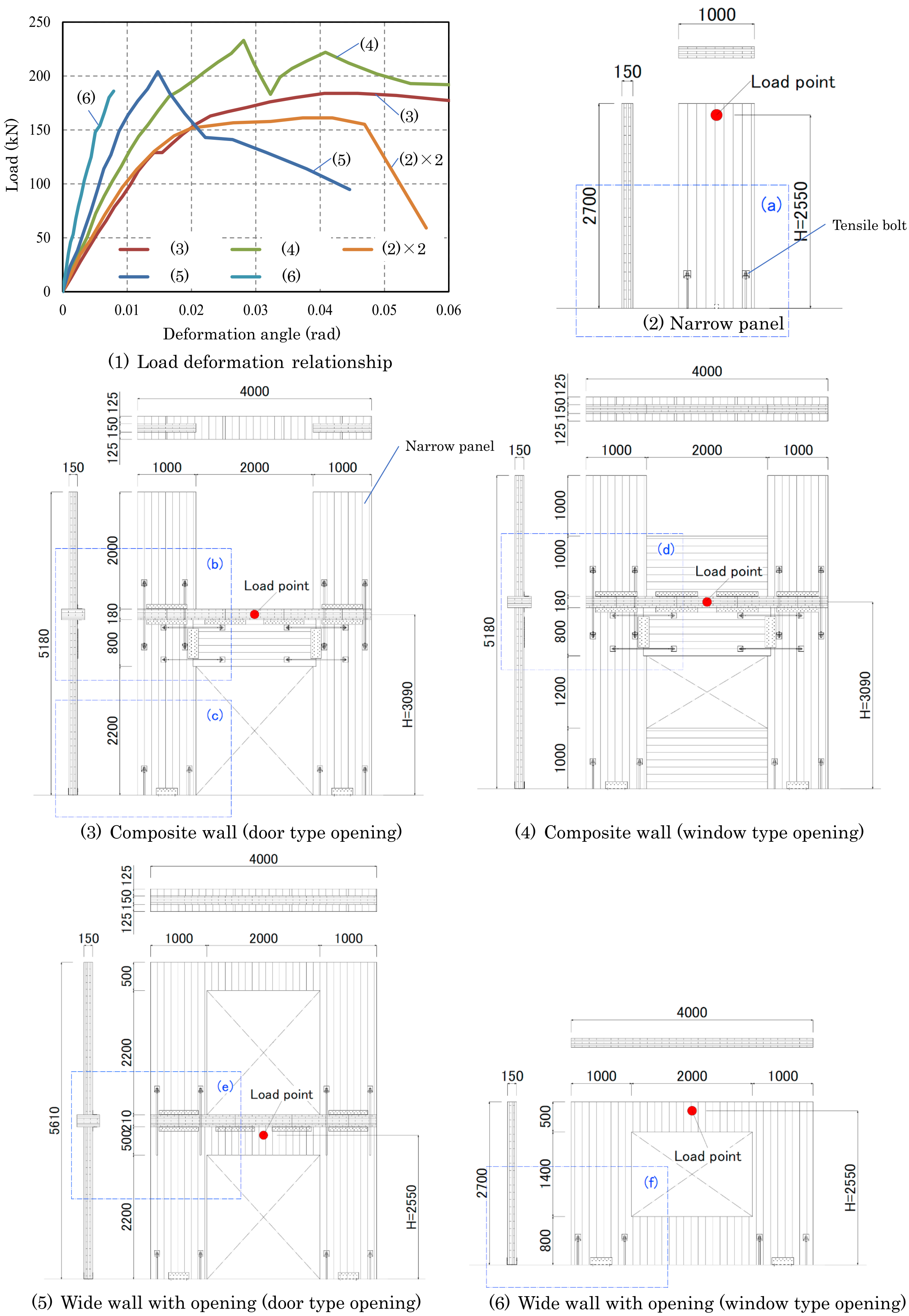
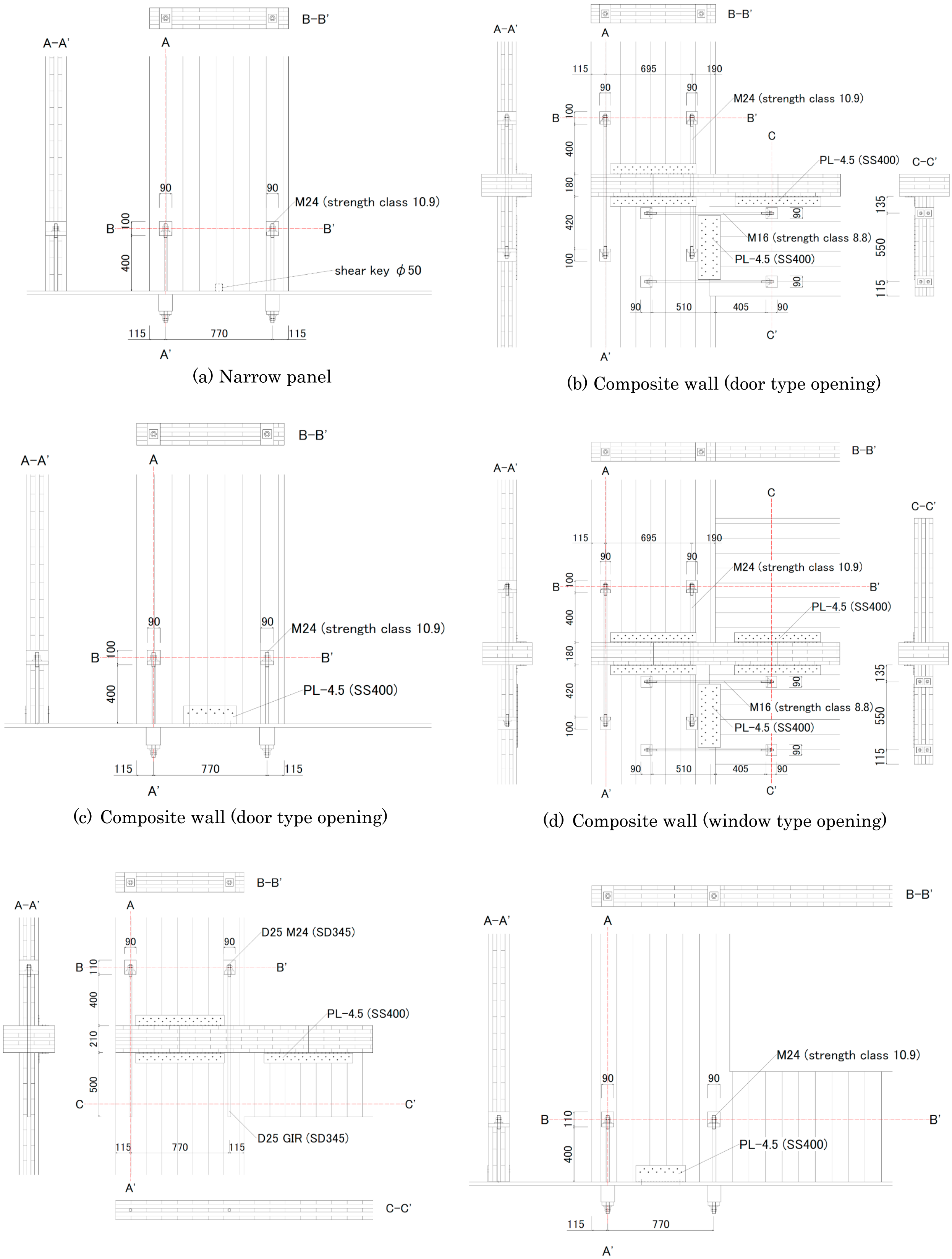


Fig. 1. Static loading tests for shear walls [mm].



(a) Narrow panel

(b) Composite wall (door type opening)

(c) Composite wall (door type opening)

(d) Composite wall (window type opening)

(e) Wide wall with opening (door type opening)

(f) Wide wall with opening (window type opening)

Fig. 2. Connection details [mm].

Table 1
Maximum load, ductility factor and Ds.

	Qmax (kN/m)	Ductility factor	Ds
Shear wall with hanging and spandrel wall	117	5.23	0.33
Shear wall with hanging wall	93	4.72	0.35

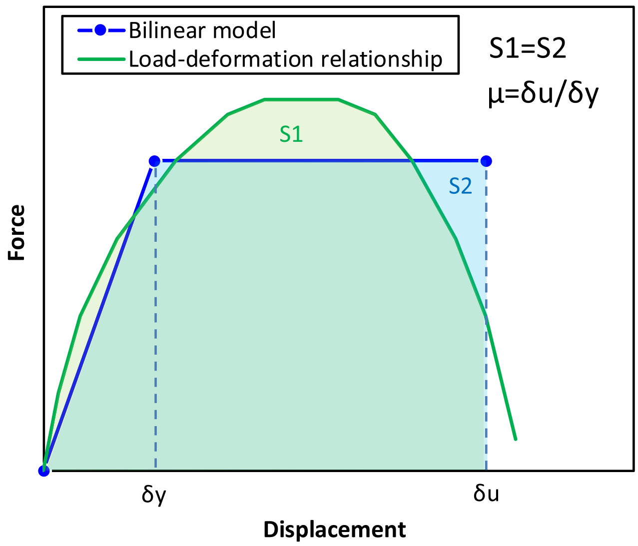


Fig. 3. Definition of ultimate and yield displacement.

Investigation into the behavior of CLT construction under seismic excitations has been limited so far. Ceccotti and Follesa reported shaking table tests on three different configurations of full-scale three-story CLT buildings in Tsukuba, Japan [1]. A seven-story CLT building also underwent shaking table tests in Miki, Japan [2]. Numerical research was performed by Ceccotti [3] and Hristvski et al. [4], as well as shaking table tests. Some monotonic and cyclic loading tests on CLT panels were conducted by Vessby et al. [5] and Popovski et al. [6]. According to past studies, CLT wall panels were found to behave as almost rigid bodies against lateral force and most of the shear wall deflections occurred as a result of yielding deformation in joints connecting walls to foundations or floors. As for seismic design in CLT construction, including nonlinear behavior, evaluation of ductility is one of the key issues. Pei et al. reported a series of nonlinear dynamic analyses [7], and results from these analyses were used to propose appropriate values for R-factors of CLT structures for the National Building Code of Canada (NBCC) [8] and preliminary R-factor for ASCE 7 [9] in the United States.

In this paper, we focus on seismic behavior and a performance evaluation procedure in CLT construction following a typical Japanese specification. One of the major different features of Japanese specification is the 1–2 m wide CLT shear walls (narrow walls) with high performance tensile bolts installed to provide a moment-resisting connection at the bottom and top of these walls. A shaking table test on a five-story CLT building was conducted to evaluate seismic characteristics of narrow-wall systems; parameters for the capacity spectrum method are shown, as well as the seismic design process of the target structure.

2. Prior research on CLT shear walls

Various wall types of CLT panels, such as narrow shear walls without openings and wide shear walls with openings, were tested during the first stage of the project [10]. Especially, two types of shear wall with openings were tested: first, a composite wall with an opening consisting of narrow, hanging and spandrel walls; second, a shear wall with an opening hollowed out from a wide CLT panel. This is because structural performances of these shear walls are thought to be significantly different. Examples of shear wall configurations considered in prior research employing quasi-static loading tests are shown in Figs. 1 and 2 shows connection details, with the test result of the relationship between applied load and inter-story drift at the bottom right. Here, inter-story drift is obtained from “H” in the figure and relative displacement between the base of the CLT and the height of “H.” There are five types of configurations, namely (2) simple narrow panel, composite wall with door type openings (3) and with window type openings (4), and wide wall with door type openings (5) and with window type openings (6). All of the CLT panels are 150 mm in thickness, and 5-layer 5-ply and tensile bolts are used to connect the bottom of the CLT panels with the foundation, the side of the CLT and the hanging wall.

The composite wall composed of a narrow shear wall has high ductility compared to the wide wall with an opening. It was shown in a previous study that panelized CLT walls possess a reasonable ductility and energy dissipation if the boundary condition is set up to allow rocking of the wall panel [11].

Table 1 shows the test results, including the maximum load and ductility factor, for shear panels composed of the narrow panels shown in Fig. 1. Ds refers to the reduction factor by the constant energy law in

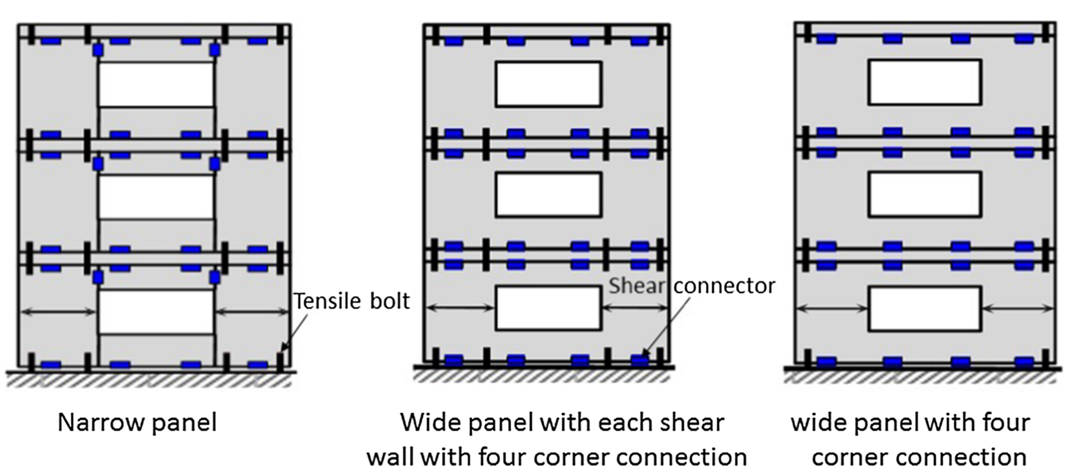


Fig. 4. Three different structural system.

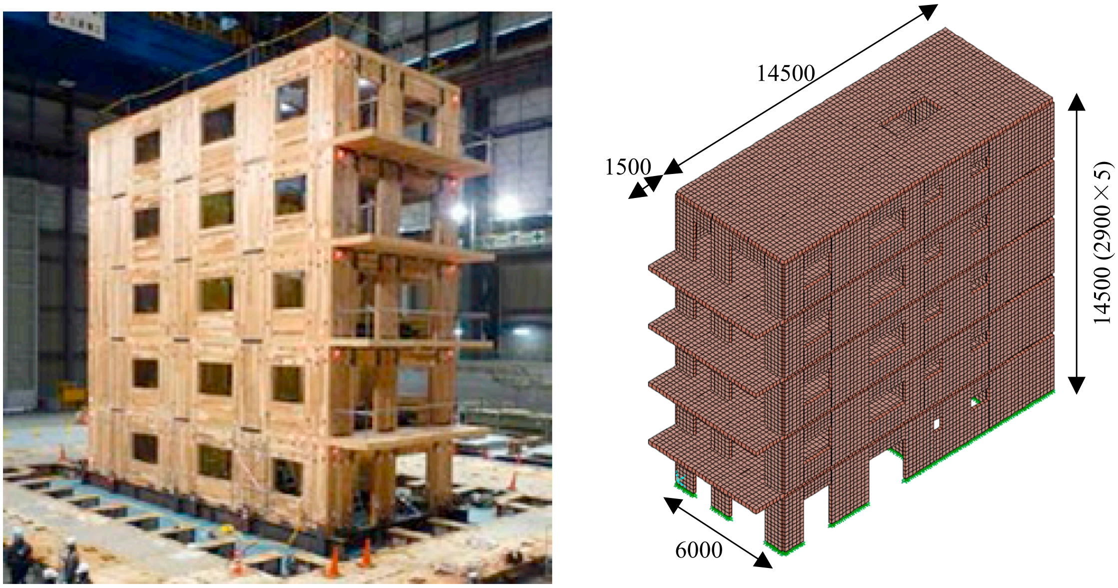


Fig. 5. Complete view of target structure.

Eq. (1). The definition of the ductility factor is shown in Fig. 3. The test results of narrow panels indicate high ductile performance.

$$Ds = \frac{1}{\sqrt{2\mu - 1}} \tag{1}$$

μ: Ductility factor (= ultimate displacement/yield displacement)
Definition of ultimate and yield displacement shown in Fig. 3

The ductility of configuration type (5)(6), a wide wall with openings, is low due to a brittle local failure in the corner of the wall panel with the large opening, but this brittle behavior was improved by the existence of a seamless floor or beam on top of the CLT shear walls as shown in the example of configuration type (5). Rocking behavior was not observed when the brittle failure occurred, and the side shear walls were separated. In the case of configuration type (5), a seamless floor prevents the side shear walls from being separated and rocking behavior of each shear wall occurring.

3. Classification of structural system

At the stage of making a structural design procedure for the project, two types of CLT building structure systems, one the assembly of narrow size panels and the other the use of large size panels with opening(s), are classified. The former is suitable for mid-rise buildings as it reveals ductile behavior, while the latter is better for low-rise buildings as it possesses high capacity even though it may be brittle. The wide shear wall systems are also divided into two systems. One is wide panel with each shear wall with four corner connection and the other is wide panel with four corner connection. When connectors are put at the four corners of each shear wall of one wide panel and cracks occur at each corner of the openings, this wide panel is divided into narrow panels and the seismic behavior of the structure corresponds to a building composed with narrow shear walls. The ductility of wide shear wall systems depends on flexural stiffness of floors or beams to restrict rocking behavior and aspect ratios of CLT panels. Future research is required to evaluate the correct response to lateral force. A summary of their structural systems is shown in Fig. 4.

When CLT building is composed with two types of wall system in one building, there is a possibility that the wide wall with an opening will be destroyed before the composite wall with an opening demonstrates their full performance. In other words, there is a possibility that the simple summation of wall strength is not satisfied. A mixed structure of both narrow and wide panels is prohibited in the specification route of Japanese building standard law (BSL).

4. Target structure

4.1. Summary of the structure

The target structure of this study is a five-story construction composed of narrow shear walls. Fig. 5 shows a complete view of the specimen. Fig. 6 shows a typical plan for this structure. Vertical walls are composed of 1000–2000 mm wide panels of grade Mx60 in the Japanese Agricultural Standard [15]. The average of the elastic modulus of lamina located in outer layers is equal to or more than 6000 N/mm^2, and the minimal elastic modulus is equal to or more than 5000 N/mm^2. They measure 3000 N/mm^2 and 2500 N/mm^2, respectively, and are located in inner layers. Japanese cedar is used for the lamina. For surface bonding of the lamina, water polymer isocyanate adhesive is used. Table 2 shows summary of the structure. Five-layered 150 mm thick panels are used for CLT panels for the walls, and seven-layered 210 mm thick panels are used for the floor. Fig. 7 illustrates the connection details, and Fig. 8 shows photographs of the connection. At the connections between walls, bolts and U-shaped metal connectors with holes for screws are used against tensile forces and L-shaped metal connectors with holes for screws are used against shear forces. All members and connections are designed in accordance with the minimum requirements of the BSL.

Initial gravity design was conducted to determine the thickness of the floor diaphragm and dimensions of the wall panels in accordance with specific load requirements of the BSL and allowable stress of the floor and the wall obtained from a static loading test. Seismic design was conducted in accordance with allowable stress and ultimate strength design. The details of the numerical model and design procedure are shown below.

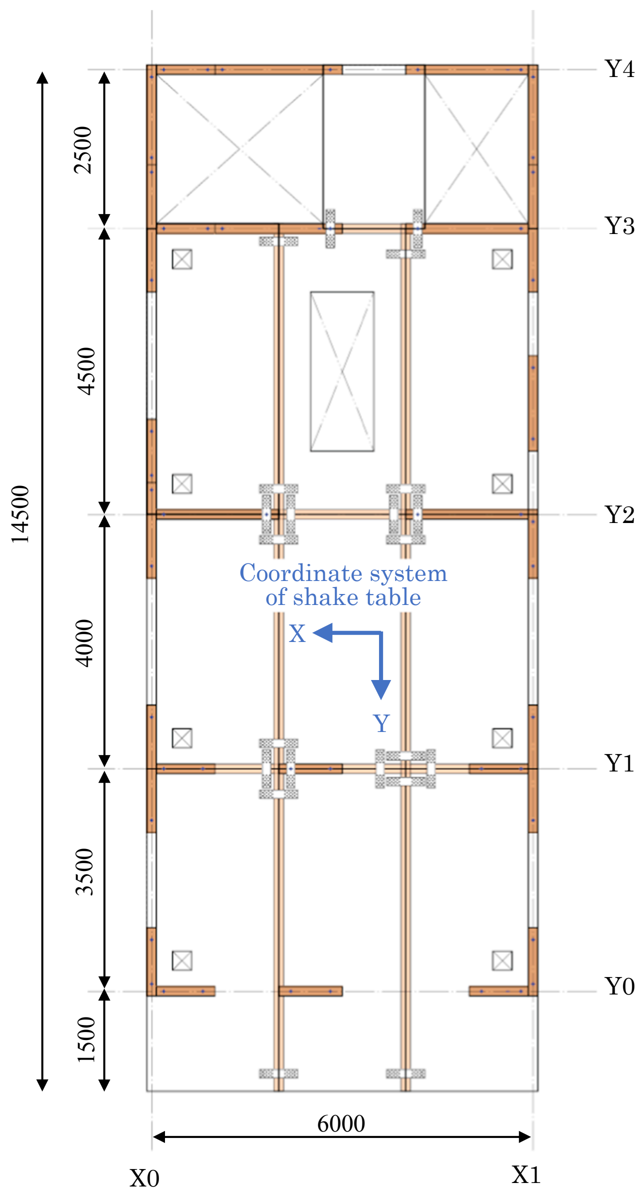


Fig. 6. Typical floor plan.

4.2. Numerical model

Fig. 9 shows a numerical model of the target structure. Pushover analysis is conducted to evaluate seismic performance of the structure before the test and numerical model and input parameters are modified through the shaking table test. In this model, CLT panels were modelled as anisotropic shell elements. The structure model used spring elements of axial direction (i.e. tension and compression springs) and horizontal direction (i.e. shear springs) for the connection between CLT panels. The axial direction springs, which are 'wall-foundation' [AT-1], 'wall-wall' [AT-2] and 'lintel-wall' [AT-3] for tension springs, are jointed. The horizontal direction springs are 'wall-foundation' [AS-1], 'wall-floor, roof' [AS-2] and 'lintel-wall' [AS-3] for a shear spring of screws with steel plates. A compression spring was employed for the load of 'wall-foundation' [CC-1], 'the upper floor into the lower floor's 'wall-wall' [CC-2]. The horizontal direction springs are 'wall-foundation' [BS-1], 'wall-floor, roof' [BS-2] for a shear spring of screws with steel plates. The compression spring was intended to introduce the load of 'wall-foundation' [CC-1], 'the upper floor into the lower floor's 'wall-wall' [CC-2] located at every 100 mm. The axial direction is 'lamina parallel' [FT-1], and 'lamina orthogonal' [FT-2] for the tension spring. The horizontal direction springs are 'floor-floor' [FS] for a shear spring of screws with steel plates.

Table 2
Structure outline and connection details.

Number of story	5
Total floor area	471.0 m^2
Height	14.5 m
Floor size	6.0 m × 16.0 m
Wall panel	Thickness 150 mm 5 layered 5ply Mx60A class lamina
Floor panel	Thickness 210 mm 7 layered 7ply Mx60A class lamina
Tensile joint	Tensile bolt
Between wall panels (and foundation)	Vertical direction (Fig. 5) Bottom of 1st story: M24 (ABR490) Others: M24 (strength class 10.9) Horizontal direction (Fig. 5) M16 (strength class 8.8)
Shear joint between wall panels, Tensile joints between floor panels	Steel plate with screws (Fig. 5) Steel plate: thickness 4.5 mm (SS400) Screw: STS-65C (length 65 mm, diameter of shank 5.5 mm)
Shear joint between floor panels	Plywood spline joining (Fig. 5) Plywood: thickness 28 mm Screw: HBS8-140 (length 140 mm, diameter of shank 5.8 mm)

4.3. Characteristics of CLT panel

Elasticity and strength were defined based on test results [11]. Table 3 shows the structural performance of CLT panels. These tests were carried out under projects organized by MILIT and the Forestry Agency and summarized in a CLT manual published in 2016 [12]. First, direction was defined as shown in Fig. 10. The longitudinal and lateral directions of surface planks were defined as "x" and "y," respectively. The direction of the thickness of the CLT panel was defined as "z." Rotational directions around the x-axis and the y-axis were defined as "rx" and "ry," respectively. The layers where the longitudinal direction of the planks was set in the y-axis and the x-axis are called the "parallel layer" and the "cross layer," respectively, in the following sections.

4.4. Connections

Fig. 11(a) shows the load-displacement relationship of a U-shaped shear joint normally used between a wall and its base. A Lintel shear joint is used between the wing wall and the hanging wall, or the wing wall and the spandrel wall. The shear displacement in both instances is almost zero in the shaking table test, due to friction between CLT panel and steel frame caused by fluctuating axial force. The model of stiffness of these parts is modified in an analysis after the shaking table test. A compression test was also conducted to define the compression spring and load-displacement relationships, as shown in Fig. 11(b).

For the CLT building, the tensile joint is important for resisting the seismic force. Therefore, various types of tensile joints between a wall and a wall, or a wall and its base, are developed and tested; hold-down tensile joints with bolts, U-shaped tensile joints with wood screws,

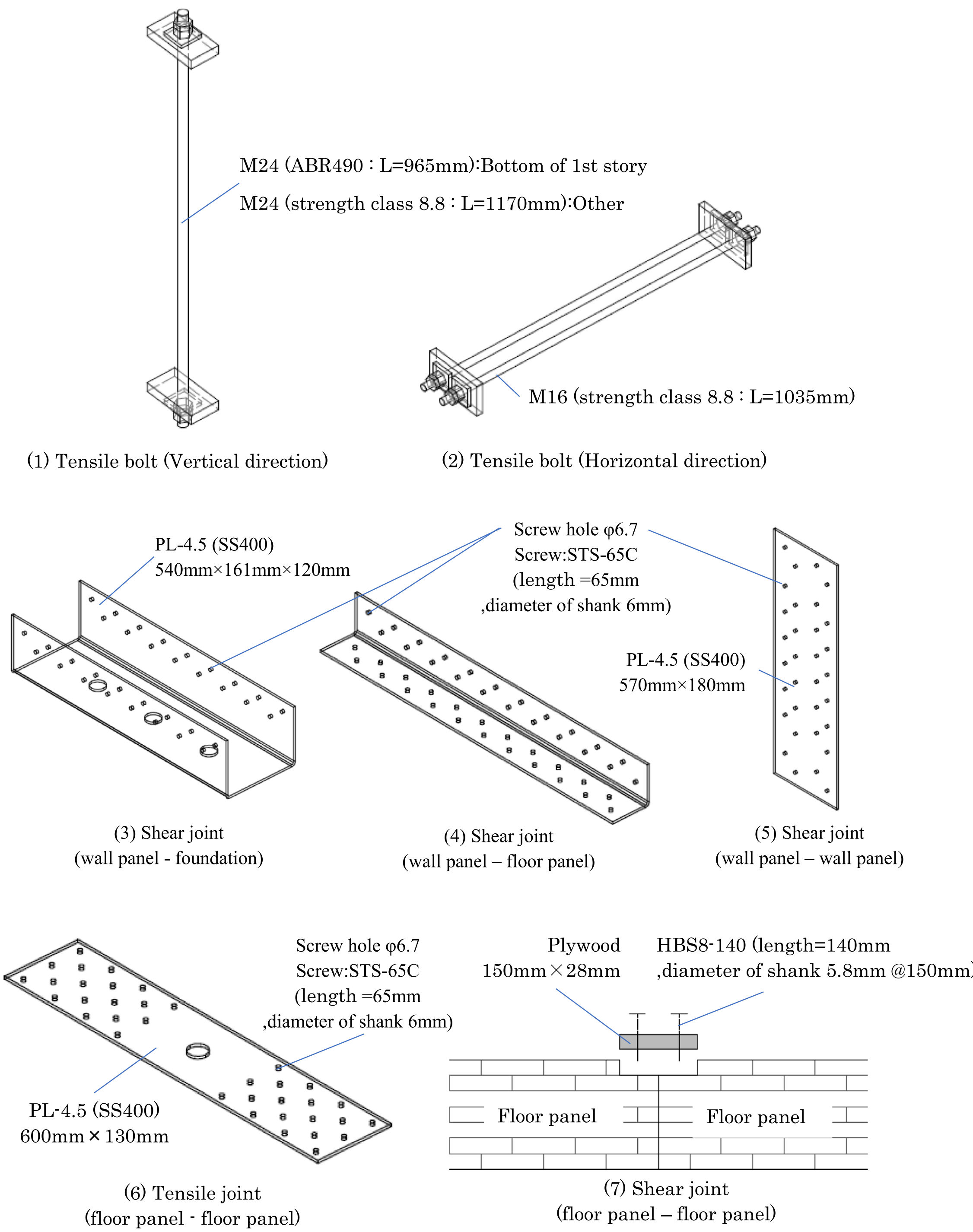
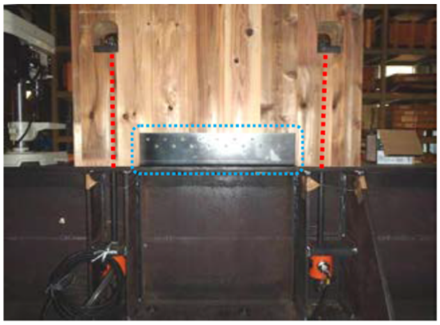


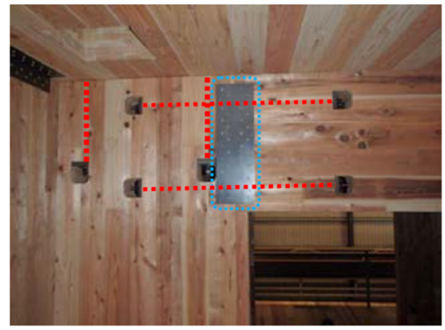
Fig. 7. Detail of connections.

tensile bolt types, glued in rods (GIR) and lug screw bolts. Based on a consideration of strength and ductility from test results, connections are chosen for the target structure. This project covers tension bolts. Fig. 11(c) shows the load-displacement relationship for a numerical spring model obtained from static tests [16].

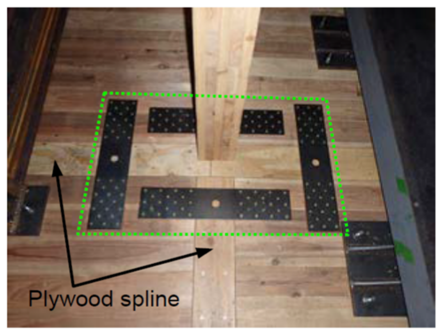
Currently, since we just have conducted CLT in-plane static loading tests and the in-plane shear performance of CLT floor with joints has not been confirmed then, the behavior of the CLT floor diaphragm is not well understood, meaning that the CLT floor diaphragm may be designed with excessive margins to ensure continuation of the shear load path. Screws of 150 mm in length are used for the spline connection for floor to floor, and a pitch of 150 mm was deemed sufficient to satisfy the resistant force obtained from push-over analysis.



a) Tensile and shear joints at the bottom of 1st story



b) Tensile and shear joints between wall panels



c) Tensile and shear joints between floor panels

: Tensile joint of wall
: Shear joint of wall
: Tensile joint of floor

Fig. 8. Photos of connections.

4.5. Allowable stress and ultimate strength design

Fig. 12 shows the result of pushover analysis, and Table 4 shows story shear in the x-direction in both design requirements and calculation results from pushover analysis. Requirements for the damage limit state and the safety limit state are 0.5% and 2.2% of the story drift in accordance with Japanese standard law. The damage limit is required to be within an allowable strength level in all components at 0.005% of the story drift, and the safety limit is required to be within an ultimate strength at 2.2% of the story drift, which is variable depending on the number of stories. The requirements of design shear resistance for each story are calculated from Eq. (2) [13].

$$Qa = 0.2 \times Ai \times \Sigma Wi$$
$$Qu = 1.0 \times Ai \times Ds \times \Sigma Wi \quad (2)$$

Ai: Lateral Force Distribution in the i-th story
Wi: Weight in the i-th story

In the structural design process, a Ds of 0.40 is used based on the composite wall tests as shown in Table 1. The target structure was designed so as to barely satisfy the minimum requirement in the x-direction.

5. Experimental program

During the test program, artificial and observed waves are used to evaluate seismic performance, and before and after each test, a white noise wave of 0.1 m/s of maximum acceleration was input to measure the natural frequency. For input waves of strong ground motion, an artificial wave that was produced to suit the response spectrum on "ground type 2" provided under the Building Standard Law of Japan and the strong ground motion recorded during the Kobe earthquake of 1995 (JMA Kobe) were used. BSL 20% as a moderate ground motion and BSL 100% as a severe ground motion were applied respectively in the x-direction and the y-direction, and JMA Kobe 100% was applied as the three-dimensional wave so that the component of larger acceleration (NS component) is applied to the long-span direction. Return periods for moderate and severe earthquakes are 50 and 200 years, respectively, in Japan. To simulate the mass condition corresponding to a five-story structure, the design weight of each floor was calculated in consideration of the dead load and live load. Steel weights were put on the floor panels so that the level of the experimental weight might reach the design seismic weight.

6. Discussion

6.1. Story drift and damage in tensile bolt

Table 5 shows the maximum story drift during BSL 20%, 100% and JMA Kobe 100% in a short span (x-direction). Torsional vibration was clearly observed, but the story drift obtained from the shaking during BSL 20% is substantially smaller than 0.5% of the limit state for damage, and the story drift during BSL 100% did not reach 2.2% of the safety limit state. During JMA Kobe, the maximum story drift exceeded the safety design limit, but the structure survived.

As vertical frames are composed of narrow CLT wall panels, damage at an early stage possibly occurs at wall-to-foundation or wall-to-wall

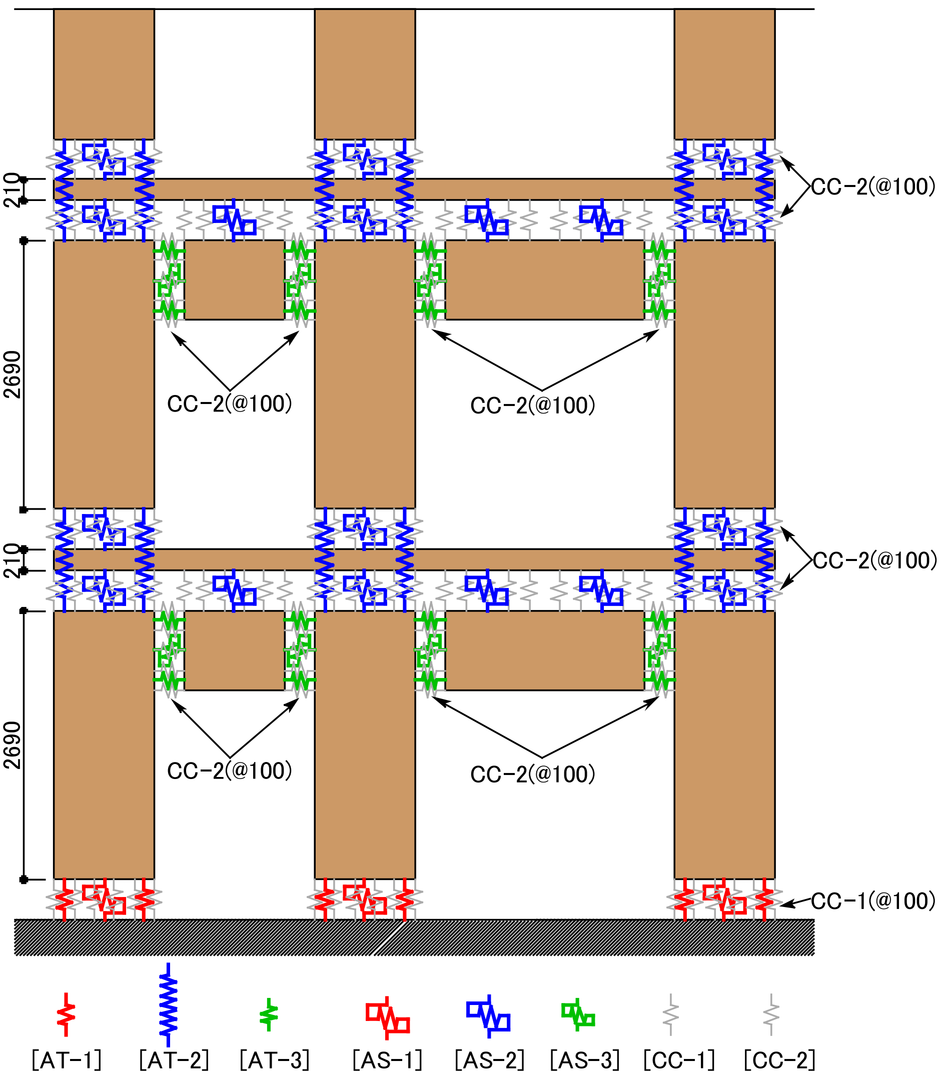


Fig. 9. Numerical model of target structure.

Table 3
CLT panel structural characteristics.

[mm]	[N/mm^2]			[N/mm^2]			
	Ex	Ey	Gxy	Erx	Ery	Gyz	Gzx
90	2000	4000	308	5778	222	123	154
150	1200	3000	231	4728	624	78	31
210	1286	2571	198	4041	866	99	53

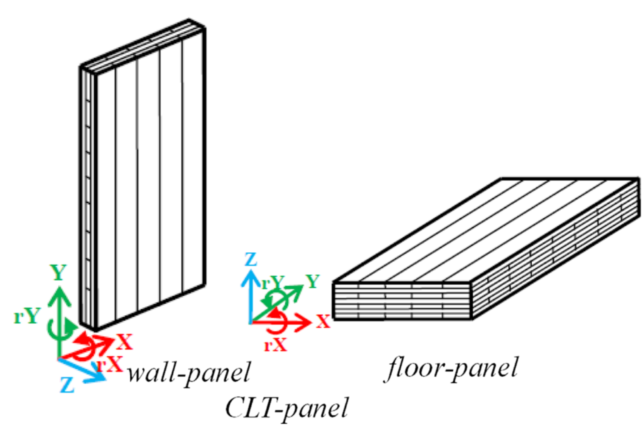


Fig. 10. Definition of direction of CLT panel.

tensile joints, and the structure was designed so that ductility of the building can be obtained by the yield of anchor bolts. During BSL 100% in a long-span direction, two tensile bolts between the CLT wall panel and the foundation reached the yield point, and the uplift of the CLT panel was 4 mm to 5 mm at the location of the yielded tensile bolts.

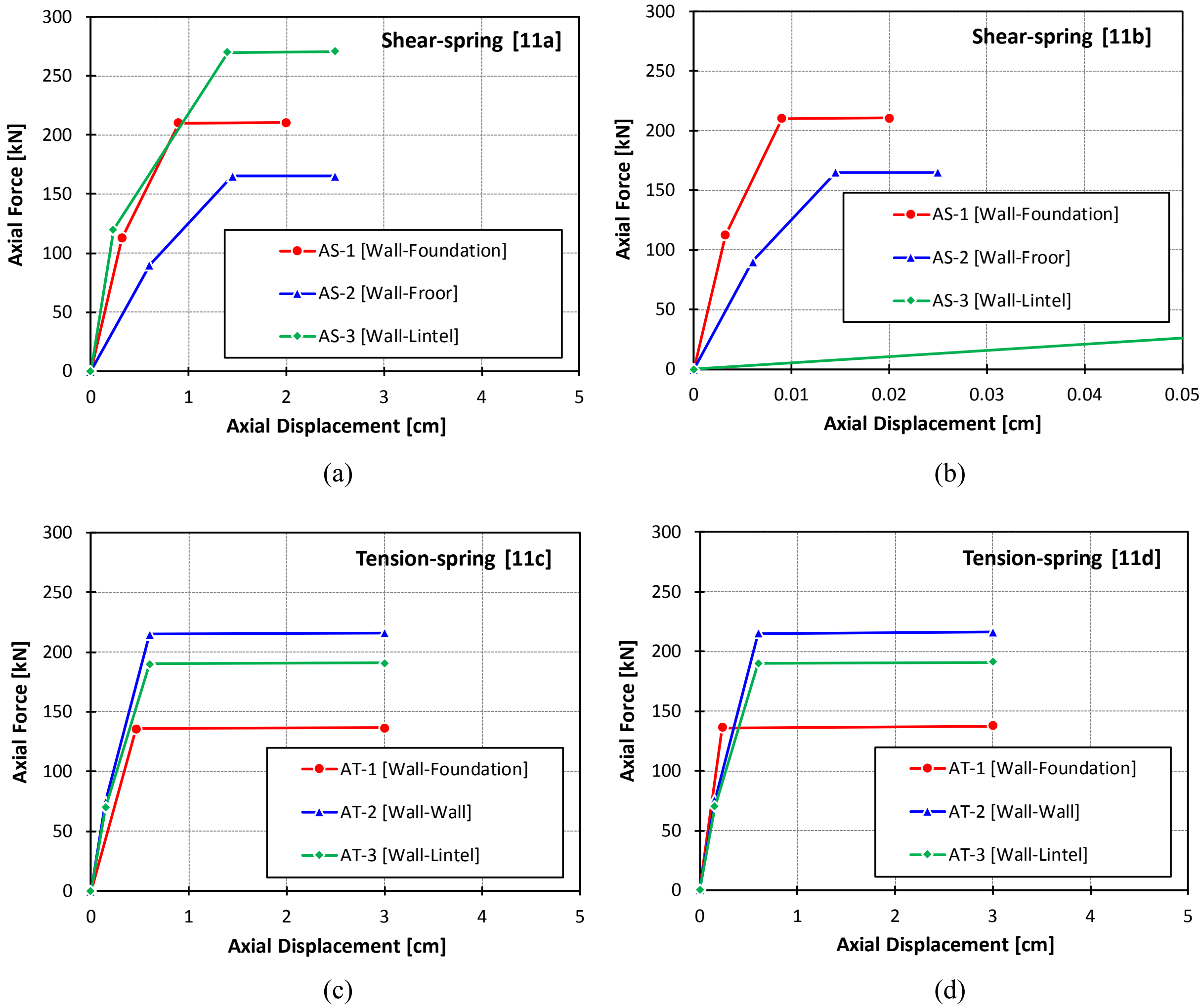


Fig. 11. Load-displacement relationship from static loading tests.

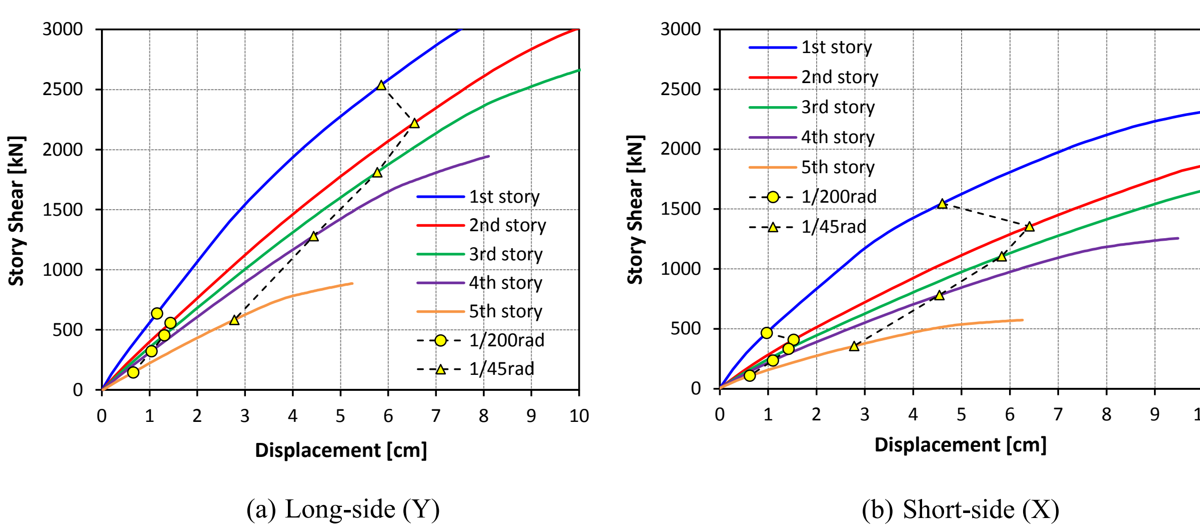


Fig. 12. Story shear versus story displacement relationship.

Table 4
Design requirement and capacity of target structure (X-direction).

Story	Wi [kN]	ΣWi [kN]	αi	Ai	Qad	Qud	Before test		After test	
							Qa	Qu	Qa	Qu
5	311.0	311.0	0.11	2.10	131	261	100	355	229	612
4	623.1	934.0	0.33	1.53	286	572	219	779	502	1344
3	623.1	1557.1	0.55	1.30	405	810	310	1102	711	1902
2	623.1	2180.1	0.77	1.14	497	994	380	1352	872	2333
1	660.6	2840.8	1.00	1.00	568	1136	434	1544	996	2663

Qad: Requirement in allowable story shear force/Qud: Requirement in ultimate story shear force.
Qa: Capacity in allowable story shear/Qu: Capacity in ultimate story shear.

Table 5
Maximum story drift (%) of X direction.

	Y0	Y1	Y2	Y3	Y4
(1) BSL 20%					
5F	0.029	0.026	0.030	0.017	0.010
4F	0.060	0.060	0.060	0.027	0.017
3F	0.105	0.093	0.108	0.049	0.023
2F	0.129	0.118	0.122	0.057	0.022
1F	0.113	0.089	0.062	0.036	0.027
(2) BSL 100%					
5F	0.286	0.228	0.303	0.172	0.100
4F	0.531	0.490	0.583	0.293	0.145
3F	0.831	0.714	0.772	0.397	0.159
2F	1.041	0.879	0.869	0.421	0.166
1F	0.959	0.741	0.528	0.266	0.131
(3) JMA Kobe					
5F	0.621	0.366	0.538	0.379	0.300
4F	1.041	1.000	1.152	0.693	0.403
3F	1.979	1.690	1.810	1.017	0.486
2F	3.700	3.145	2.572	1.255	0.507
1F	2.921	2.203	2.007	0.800	0.421

Fig. 13. Damage observed during JMA Kobe.

However, other damage during BSL 100% was not so severe, including slight deformation of steel plates, gaps between wall panels, and embedment of CLT panels at the corner of openings. During JMA Kobe 100%, all tensile bolts between CLT wall panels and foundations reached the yield point and the uplift of CLT panels reached a maximum of 30 mm. Compressive rupture at the corners of CLT wall panels also occurred, as shown in Fig. 13. There is a small opening for tensile bolts near the corner of CLT panels, and it seems to cause compressive rupture.

6.2. Validation of design

Fig. 14 shows a comparison between our test results and an analytical study with regard to the relationship between story shear and story displacement. Analytical studies were conducted before and after the test. The elastic modulus of CLT and the stiffness of the shear connection are doubled due to the quality of CLT and the velocity effect in the study after the test. Shear springs between a wall and a wall, and a wall and its base, are multiplied by 100 as mentioned above. These modifications can be explained by the test, but there is still a slight difference between the test result and the modified calculation. The difference in y-direction is small compared to that of the x-direction. Further analysis is required in future research of performance-based structural design. In Table 4, the right column also shows story shear resistance calculated from the modified parameters in the numerical model. The real capacity of the structure is almost 50% bigger than the calculation result obtained before the test.

6.3. Damping ratio

One of the key parameters of the Japanese limit strength calculation method [14] is the damping ratio of a structure. The required strength can be reduced by the following equation.

$$Fh = \frac{1.5}{1 + 10h} \tag{3}$$

Fh: Reduction factor by damping
h: Equivalent damping of a structure

Fig. 15 shows the equivalent damping ratio obtained from test results, h_{eq} in each half cycle of A-⊿relations and a displacement ratio that is the ratio of displacement amplitude in each half cycle to the maximum value of ⊿. The equivalent damping ratio is obtained from the following equation and Fig. 16

$$h = \frac{1}{2\pi}\cdot\frac{E_{Diss}}{E_{sto}} \tag{4}$$

E_{Diss}: Dissipated Energy in a half cycle
E_{sto}: Stored Energy

The damping ratio was 10–15%, and Fig. 16 shows the Sa-Sd spectrum and capacity curve of a single degree of the freedom system. The meeting point between spectrum and capacity is the maximum response of the target structure. This response corresponds to test results.

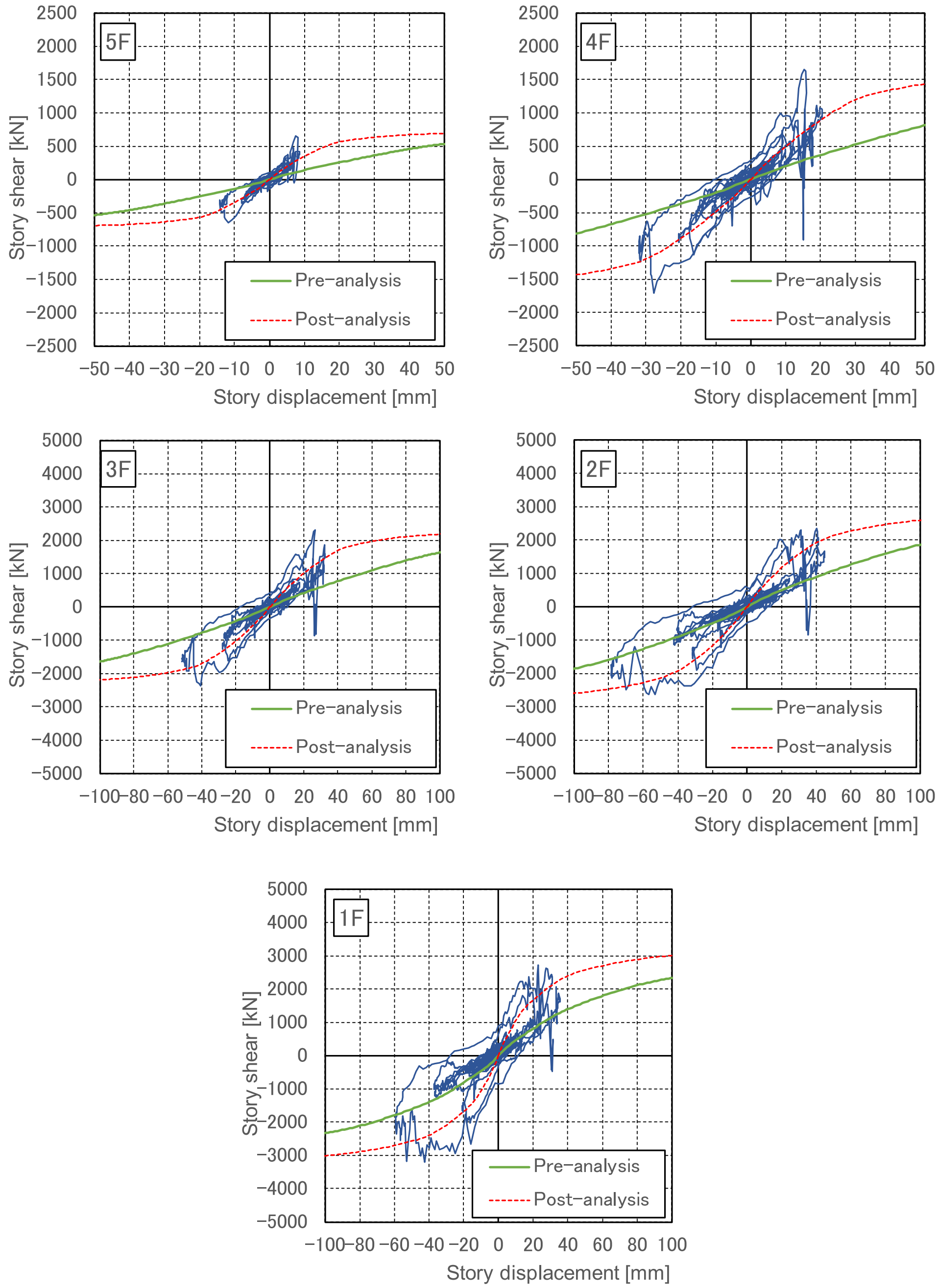


Fig. 14. Comparison between test and analytical results.

7. Conclusions

Element tests and connection tests for various types of shear wall were conducted to define a numerical model and its parameters, and a shaking table test was conducted to confirm the model and its parameters and to evaluate structural performance. A target structure composed of narrow shear walls and high ductile tensile bolts was designed so as to satisfy the minimum requirement of Japanese building standard law. The structure was shown to behave well during severe strong motion as specified in Japanese building standard law and survive the 1995 Kobe earthquake despite the occurrence of a compressive rupture in the shear wall as well as a support element against the vertical load. Story shear capacity calculated from a numerical model and element tests (such as connections) were safely evaluated; but to evaluate the capacity correctly further research is required at the element and system levels. Though a variety of further research is valuable, the Building Standard Law and Notification for three different CLT construction systems was enforced in April 2016 to ensure the construction of safe CLT buildings.

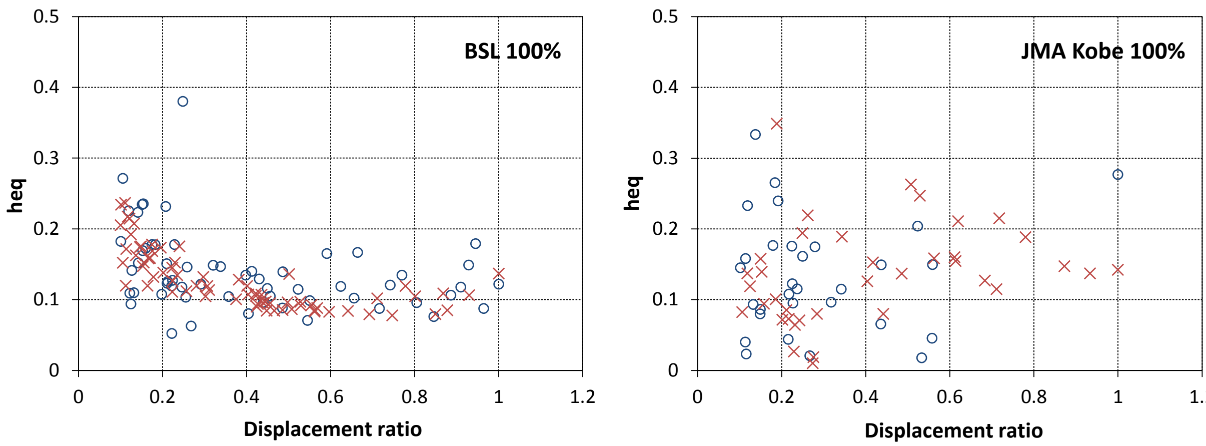


Fig. 15. Equivalent damping ratio from the shaking table test.

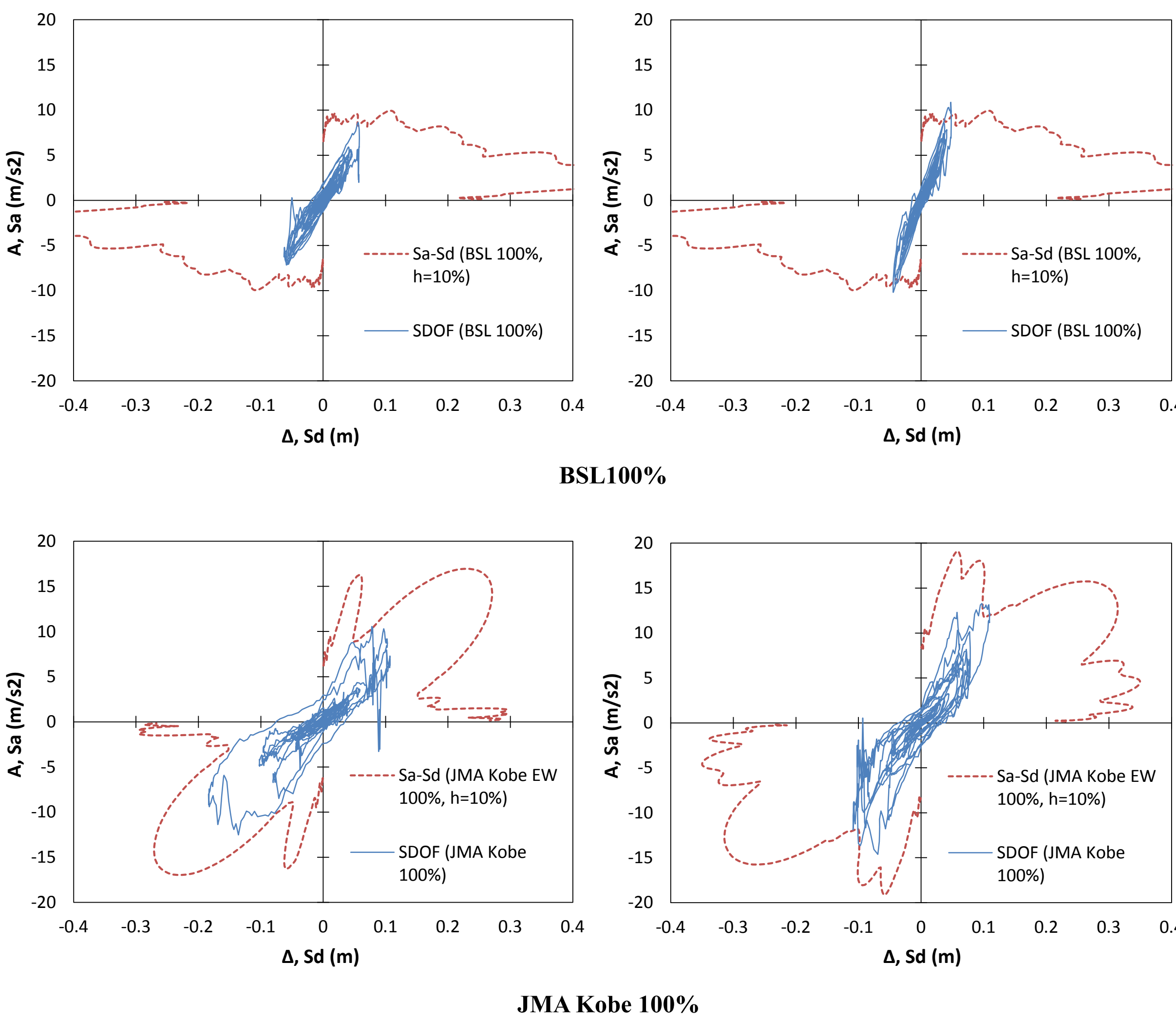


Fig. 16. Relationship between the Sa-Sd spectrum and capacity curve of the SDOF system.

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Appendix A. Supplementary material

Supplementary data to this article can be found online at https://doi.org/10.1016/j.engstruct.2018.09.054.

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