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AN EXPERIMENTAL STUDY ON STEEL - WOODEN HYBRID RESISTING SYSTEM

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SUMMARY

This paper presents experiments conducted on steel - wood hybrid resisting system that can be used in Japanese conventional wooden frameworks to resist lateral forces. The resisting system is composed of a steel plate with slits and a wooden frame. The experimental work consists of 16 small-scale specimens and 1 full-scale specimen subjected to repeated lateral loading. Four kinds of layout of slits, and without or with out-of-plane strengthening were chosen as experimental parameters. The test results showed great improvements in load-deformation relation, load-carrying capacity, and energy absorbing capacity of specimens when out-of plane strengthening elements were used.

INTRODUCTION

Performance evaluations on conventional wooden frameworks became necessary according to the amendments to the Building Standard Law of Japan and the constitution of laws on the qualities of housing in 2000. However, it is quite difficult to quantitatively evaluate the performance of the traditional shear walls in a wooden framework, such as those composed of wooden braces or structural plywood. And it is also difficult to expect ductile deformations of these shear walls.

In the proposed steel-wood hybrid resisting system, lateral forces are resisted by steel plate with slits. These numerous parallel slits afford the large deformation capacity of the steel plate that enables it to behave harmoniously with its surrounding wooden framework. Besides, these slits allow the steel wall to provide the strength and stiffness according to the structural demands by simply changing the layout of slits, including interval, length, and layer of slits. This feather enables performance-based designing of wooden structures. Furthermore, spindled hysteretic loops, easy construction by using nail connection, and stable quality can be obtained.

The main objective of this research is to investigate the structural performance of the proposed hybrid resisting system.

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STRENGTHS AND RIGIDITIES

The shear strength of a steel-wooden hybrid shear wall (see Figure 1) can be calculated on the basis of full plastic moments of the upper and the lower ends of the column parts and it can be expressed by the following equation (Hitaka [1], Li [2]).

$$Q_{wt} = \frac{nM_p}{l/2} = \frac{ntb^2}{2l} \cdot \sigma_y \tag{1}$$

where, M_p = full plastic moment of column part; n = number of column part; l = length of column part; b = width of column part; t = thickness of steel wall; and σ_y = yield stress of steel.

The rigidity can be calculated by Equation 2, based on the lateral force - deformation relations, where the flexural and shear deformations of column parts, and the shear deformations of wall parts (see Figure 1) are taken into consideration.

$$K = \frac{Q_w}{\Delta} = \frac{1}{\frac{\kappa h}{GBt} \cdot m + \frac{l^3}{Etb^2} \cdot \frac{m}{n}} \tag{2}$$

where, Q_w = lateral force; Δ = lateral displacement; E = Young's modulus of steel; G = shear modulus of steel; m = number of layer of column part; κ= sectional shape factor; and h = height of steel wall.

STRENGTHENING

The critical value of lateral force when a column shown in Figure 2 begins to buckle can be given by,

$$Q_{cr} = \frac{M_{cr}}{L/2} \tag{3}$$

where, M_{cr} = moment when lateral buckling occurs, and it can be calculated by Equation 4 (Architecture Institute of Japan [3], Timoshenko [4]).

$$M_{cr} = 2.5 \cdot \frac{\pi}{L} \cdot \sqrt{EI_\eta C} \tag{4}$$

where, I_η = moment of inertia of cross section about its weak axes; and C = torsion rigidity.

Lateral buckling will occur before the full plastic moments of column parts are reached in case the shear aspect ratio of a column part is large. In order to prevent lateral buckling, it is necessary to strengthening the column parts. In other words, the purpose of strengthening is to make the critical buckling force (Q_{cr}) become larger than its calculated shear strength (Q_{wt}) by Equation 1. And shortening the buckling length of the column parts can be an effective measure. When the out-of plane strengthening elements were installed at the upper and the lower ends of the column parts as shown in Figure 3, the buckling length becomes to L from l. Figure 3 shows how to determine the length L (=170mm) for a column part of l=250mm, b=25mm, t=1.15mm, and σ_y=304N/mm^2.

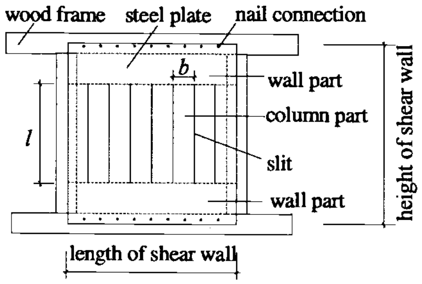


Figure 1 Technical terms

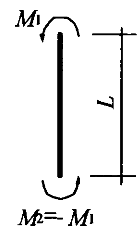


Figure 2 Column

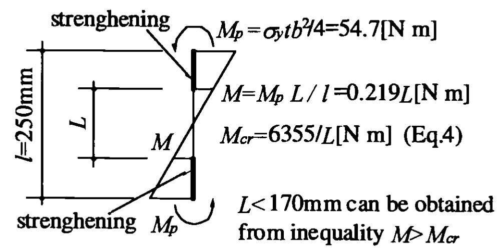


Figure 3 Strengthening

Equation 1 is on the premise that the column parts as well as the wall parts do not buckle laterally. If the whole wall is considered as a column and its buckling moment denoted by M_{crp}, strengthen should be made so that the moment M_{crp} becomes larger than the moment (denoted by M) at the top edge or the bottom edge of the wall. The moment M can be calculated by $M = M_p\ h/l$, where M_p is the full plastic moment of column parts. According to Equation 4, torsion rigidity of the wall parts should be increased $(M/M_{crp})^2$ times in order to prevent lateral buckling. It is found that 2 angles of 25 x 25 x 3 could be used to strengthen the out-of-plane rigidity of a wall part.

EXPERIMENTS

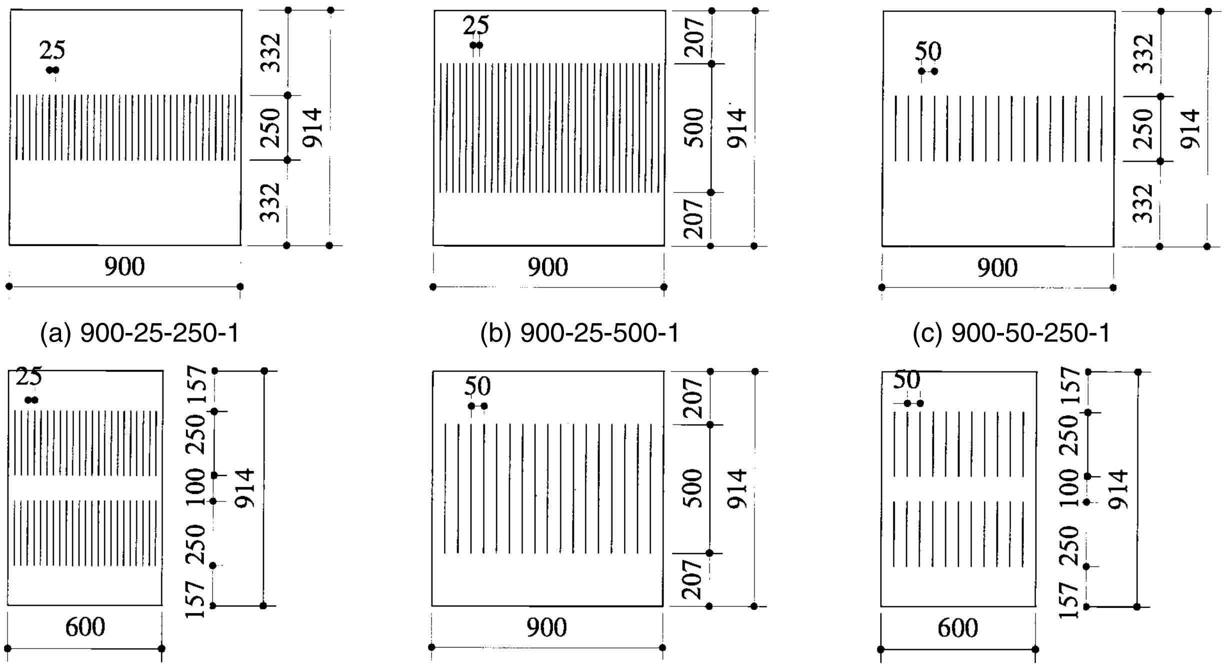
Experiments were conducted to investigate the structural performance of the steel - wood hybrid resisting system. The experimental work consists of 16 small-scale specimens and 1 full-scale specimen. Wooden elements (cross section: 105mm x 105mm) used in specimens were air-dried Japanese cedar. Four kinds of layout of slits (slit interval and slit length) shown in Figure 4 were considered in this experimental work. The slits were fabricated by laser-cut with a 0.5mm width. Steel plates of 1.2mm thickness (SPHC steel) were used in specimens, because a thin plate was considered suitable for a wooden framework. However, out-of-plane deformation develops easily in a thin plate, therefore, whether out-of-plane strengthening elements were used or not was chosen as another important experimental parameter. Table 1 shows the material properties of steel and Table 2 is the details of specimen.

Photo 1 shows 1 small-scale specimen and 1 full-scale specimen, which are set up in the loading frame. Both of them are with out-of-plane strengthening. Three steel plates (900 x 965) were lap jointed in the full-scale specimen. At the lap joint levels, two extra lateral wooden members were mortised to the wooden frame as corbels (see the right-side photo).

Lateral forces were applied by displacement - controlled procedure and repeated one time at drift angle amplitude of 1/600, 1/450, 1/300, 1/200, 1/150, 1/100, 1/75, 1/50, 1/30, 1/15 (radian) shown in Figure 5. The lateral displacements of the beam and the sill in a frame as well as the vertical displacements of column bases were measured by displacement transducers.

Table 1 Mechanical Properties of Steel

Young's Modulus (kN/mm^2)	Yield Stress (N/mm^2)	Tensile Strength (N/mm^2)	Yield Ratio	Throttle (%)	Elongation (%)
206.3	304.3	388.0	0.78	38.7	38.1



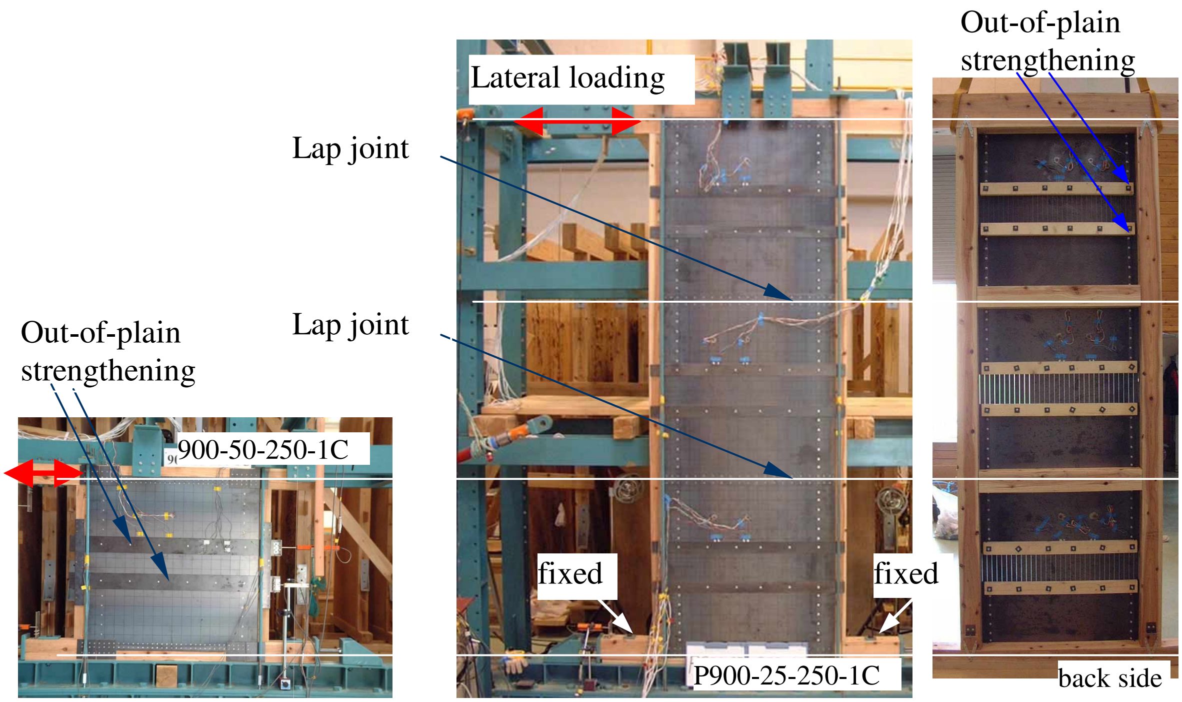
(a) 900-25-250-1 (b) 900-25-500-1 (c) 900-50-250-1

(d) 600-25-250-2 (e) 900-50-500-1 (f) 600-50-250-2

Figure 4 Layout of Slits

Table 2 Specimens

Specimen Name	Dimension of wooden frame	Dimension of Steel Plate With / without strengthening	Layout of Slits			Buckling Length
			Interval b (mm)	Length l (mm)	Layer m	L (mm)
frame1-vp1	910 x 910	Frame only	-	-	-	-
frame2-vp2			-	-	-	-
frame3-vp1			-	-	-	-
900-00-000-0N	910 x 910	900 x 914 without strengthening	no slits			-
900-25-250-1N			25	250	1	170
900-25-500-1N			25	500	1	140
900-50-250-1N			50	250	1	120
900-50-500-1N			50	500	1	170
900-25-500-1C	910 x 910	900 x 914 strengthening	25	500	1	170
900-50-250-1C			50	250	1	120
600-25-250-1C		600 x 914 strengthening	25	250	1	170
600-25-250-2C			25	250	2	170
600-25-500-1C			25	500	1	240
600-50-250-1C			50	250	1	120
600-50-250-2C			50	250	2	120
600-50-500-1C			50	500	1	170
P900-50-250-1C	910 x 2730	900 x 965 (3 plates) strengthening	25	250	1	170



(a) Small-scale specimen
frame: 910 x 910
1 steel plate: 900 x 914
slit interval: 50, length: 250

(b) Full-scale specimen
frame: 910 x 2730
3 steel plates: 900 x 965 /each
slit interval: 25, length: 250

Photo 1 Specimens

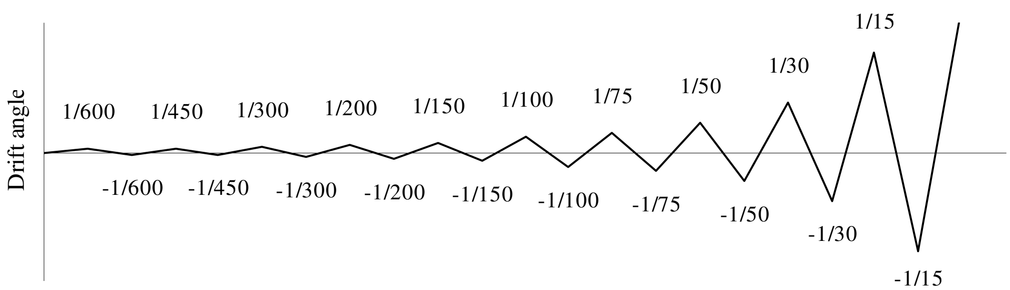


Figure 5 Loading program

TEST RESULTS

Lateral Force - Drift Angle Relations

Lateral force - drift angle relations of specimens are shown in Figure 6. The horizontal solid lines are the calculated ultimate shear strengths Q_{wt}, and the dotted lines the calculated yield shear strengths Q_{wty} which are 2/3 of strengths Q_{wt}. Specimen 900-50-250-1C showed unsymmetrical hysteretic features because lateral forces applied on it at minus drift angles could not exceed 25kN due to the capacity of the oil jack. As can be seen from Figure 6, the hysteretic loops of the specimens without out-of-plane strengthening were of somewhat slipped shapes, and the calculated ultimate shear strengths (Q_{wt}) could not be reached due to the earlier out-of-plane deformations during the loading tests. On the contrary, specimens with strengthening reached their shear strengths Q_{wt}. The hysteretic loops of the strengthened specimens except specimens 600-50-250-1C and 600-50-250-2C became much more spindled. The specimens 600-

50-250-1C and 600-50-250-2C showed a little bit slipped hysteretic loops because of the out-of-plane deformations in their whole steel plates.

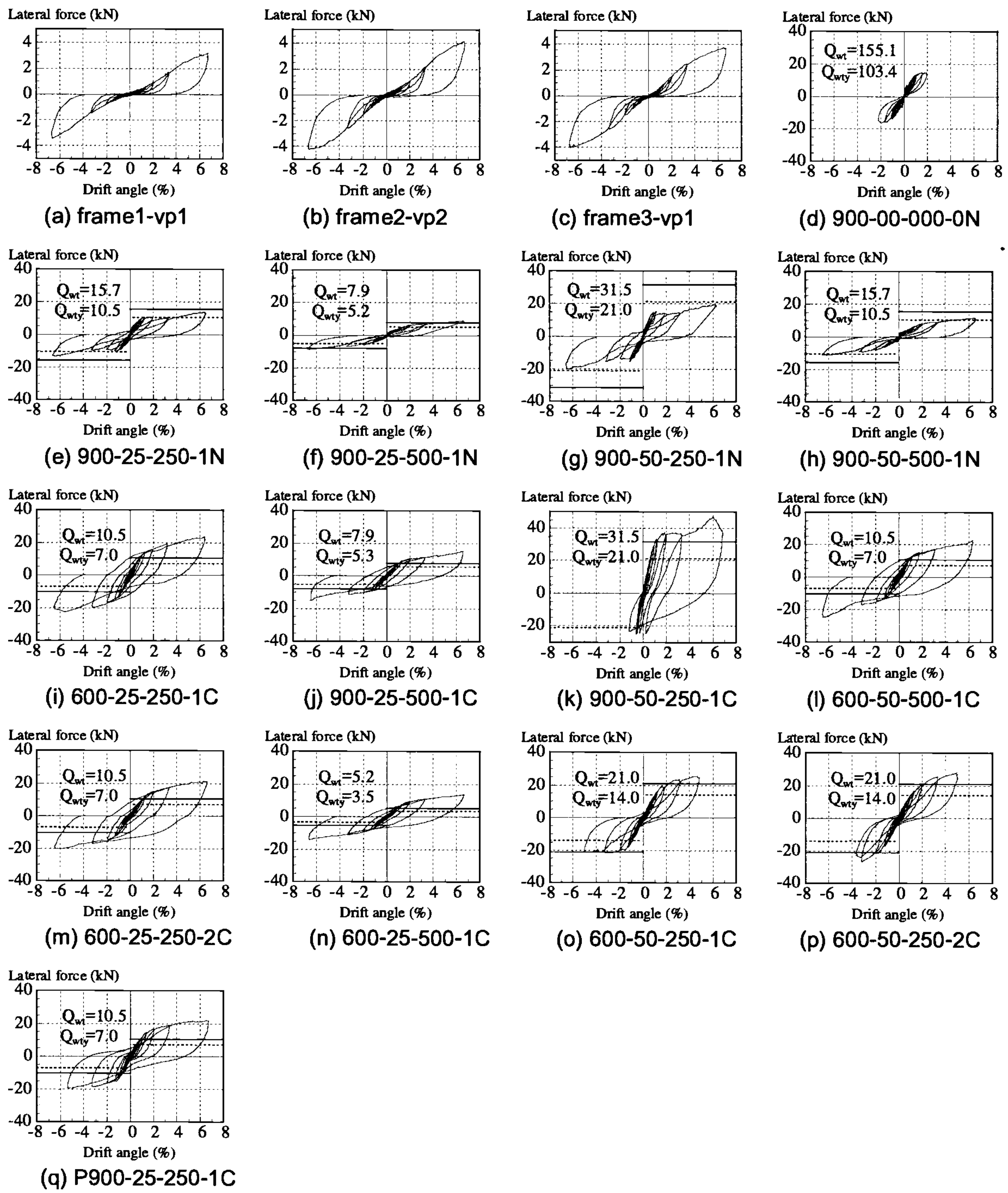
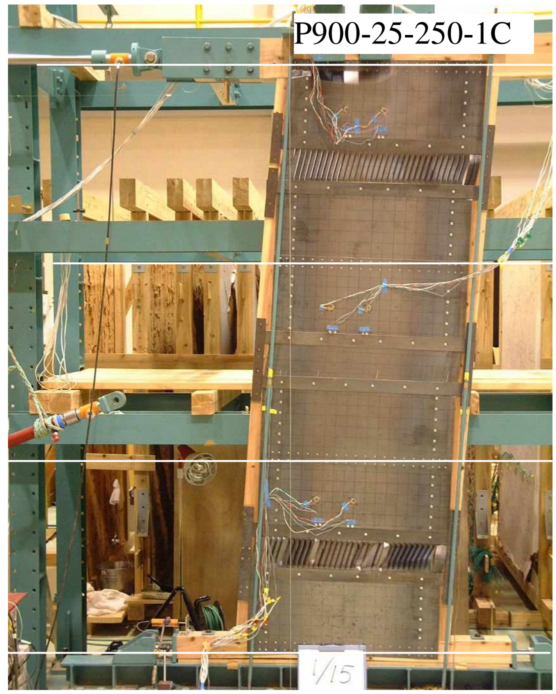


Figure 6 Lateral force - drift angle relations

Photo 2 shows the specimens at drift angle 1/15rad. In specimen 900-50-250-1C, not only buckling of column parts but also buckling of wall parts can be observed. Almost no buckling in the middle steel plate can be observed in specimen P900-25-250-1C, compared with obvious out-of-plane deformations occurred in the top and the bottom steel plates. Shear forces could not be transferred smoothly among the three steel plates and bending effects became large in the full-scale specimen.

(a) Small-scale specimen



(b) Full-scale specimen

Photo 2 Specimens at drift angle 1/15 radian

Comparisons between Tests and Calculations

Table 3 shows the force at drift angle 1/150rad. (denoted by $P_{1/150}$), the yield strength (P_y) in the elastic-plastic model obtained from test result, the ultimate strength (P_u) in the elastic-plastic model, the maximum force (P_{max}), and the calculated yield and ultimate shear strengths (Q_{wty} and Q_{wt}). The yield strengths (P_y) of strengthened specimens except 600-25-250-1C and 600-25-250-2C exceed the calculated shear strengths (Q_{wt}). On the contrary, the yield strengths (P_y) of the specimens without out-of-plane strengthening are smaller than their calculated shear strengths (Q_{wt}).

Table 3 also shows the comparisons between the calculated rigidities K_c by Equation 2 and the experimental values K_e. An experimental rigidity of a specimen without out-of-plane strengthening is defined as the ratio of the force at drift angle 1/150 rad. ($P_{1/150}$) to the lateral displacement ($\delta_{1/150}$), which is $K_e = P_{1/150}/\ \delta_{1/150}$; while for a specimen with out-of-plane strengthening, an experimental rigidity is defined as the minimum of $P_{1/150}/\ \delta_{1/150}$ and $Q_{wty}/\ \delta_{wty}$, where δ_{wty} is the displacement at Q_{wty}. The ratios of K_e to K_c scatter from 0.11 to 1.27 (see Figure 7), which is probably caused by the initial lateral deformation during construction.

Wall strength ratios

A technical term called wall strength ratio is used to evaluate the strength of a shear wall used in wooden frameworks, which is an important term especially for those wooden structures designed according to specifications. The wall strength ratio of a shear wall can be calculated from the following equation (Kenchiku Gijyutsu [5]).

$$\text{wall strength ratio} = \frac{\min\{P_{1/150}, P_y, \frac{2}{3}P_{\max}, 0.2P_u/D_s\}}{1.96L}\alpha \tag{5}$$

where, D_s = structural characteristics factor; L = length of shear wall [m] (in this research, L=0.91m); α = reduction factor due to construction and permanence (in this research, α=1.0); and the digitals 1.96 = horizontal strength [kN/m] when the wall strength ratio equals to 1.0.

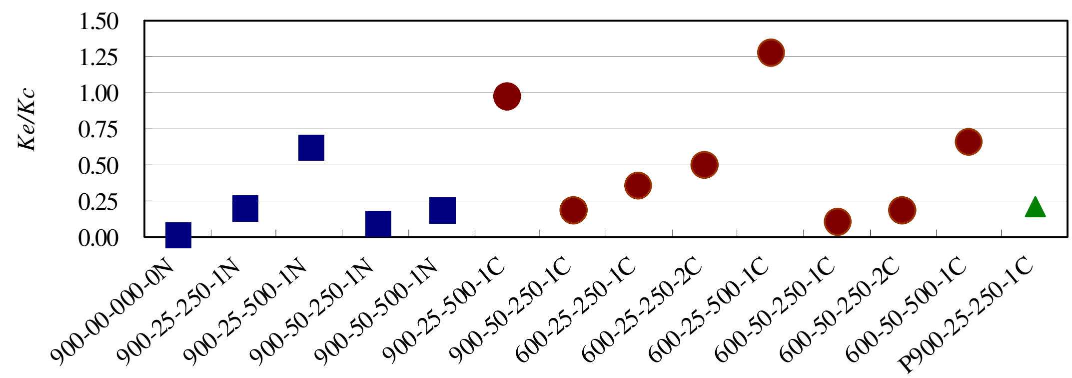


Figure 7 Ratio of rigidity

Table 3 Test results

Specimen name	$P_{1/150}$ (kN)	P_y (kN)	P_u (kN)	D_s	P_{max} (kN)	Q_{wty} (kN)	Q_{wt} (kN)	K_e (kN/mm)	K_c (kN/mm)	K_e/K_c	Wall strength ratio
frame1-vp1	0.4	2.9	3.4	1.00	3.2	-	-	-	-	-	0.22
frame2-vp2	0.6	3.7	4.5	1.00	4.1	-	-	-	-	-	0.31
frame3-vp1	0.7	2.3	3.3	0.68	3.7	-	-	-	-	-	0.36
900-00-000-0N	11.8	7.8	14.0	0.48	15.0	103.4	155.1	2.2	76.2	0.03	3.28
900-25-250-1N	8.6	9.7	11.4	0.31	13.6	10.5	15.7	1.6	7.8	0.21	4.07
900-25-500-1N	3.7	5.4	7.9	0.46	9.0	5.2	7.9	0.7	1.1	0.64	1.93
900-50-250-1N	13.6	14.4	16.1	0.29	19.4	21.0	31.5	2.5	23.9	0.11	6.32
900-50-500-1N	4.5	7.4	10.5	0.47	11.8	10.5	15.7	0.8	4.1	0.20	2.51
900-25-500-1C	6.1	9.3	12.5	0.41	15.1	5.2	7.9	1.0	1.1	0.98	3.41
900-50-250-1C	25.2	32.9	39.5	0.33	46.5	21.0	31.5	4.4	23.9	0.18	13.29
600-25-250-1C	10.0	13.5	20.0	0.40	23.5	7.0	10.5	1.9	5.2	0.36	5.59
600-25-250-2C	8.0	13.1	18.8	0.44	21.1	7.0	10.5	1.4	2.7	0.49	4.47
600-25-500-1C	4.9	8.2	11.9	0.44	13.8	3.5	5.2	0.9	0.7	1.27	2.73
600-50-250-1C	11.4	16.4	23.1	0.47	25.6	14.0	21.0	1.7	15.9	0.11	5.57
600-50-250-2C	11.7	16.0	24.7	0.47	27.8	14.0	21.0	1.7	9.4	0.18	5.85
600-50-500-1C	9.7	13.8	18.8	0.41	22.7	7.0	10.5	1.8	2.7	0.66	5.18
P900-25-250-1C	9.0	13.8	19.8	0.39	21.8	7.0	10.5	0.5	2.6	0.20	5.03

Denotations: $P_{1/150}$: the force at drift angle 1/150rad.; P_y: yield strength in the elastic-plastic model; P_u: ultimate strength in the elastic-plastic model; D_s: structural characteristics factor; P_{max}: the maximum force; Q_{wty}: the calculated yield shear strength; Q_w: the calculated ultimate shear strengths; K_e: the experimental rigidity; K_c: the calculated rigidity

Table 3 shows the values of wall strength ratio obtained from test results. The smeared cells in Table 3 represent the terms that determined the wall strength ratios. As shown in Table 3, the shear wall strength ratios of the specimens with out-of-plane strengthening increased. The full-scale specimen has a value of 5.0, which is the biggest value described in the Building Standard Law of Japan. Shear walls with slit interval 50mm and slit length 250mm, whether they were out-of-plane strengthened or not, can provide wall strength ratio bigger than 5.0. Shear strengths of these walls, however, are quite large so that it is easy to cause the wooden frames or connections to fail. On the contrary, shear walls with slit interval 25mm and slit length 250mm, have wall strength ratio bigger than 4.0, and they worked well with the surrounding wooden frames.

Energy Absorption

Figure 8 shows the energy absorption from the beginning of a loading test to drift angle 1/150rad., 1/50rad., and to the end of the test, respectively. Commonly, energy absorption increased when out-of-plane strengthening was used. The full-scale specimen P900-25-250-1C is of large energy absorption at each state, which is almost 3 times of that of a small-scale specimen whose slit layout is the same as that of the full-scale specimen.

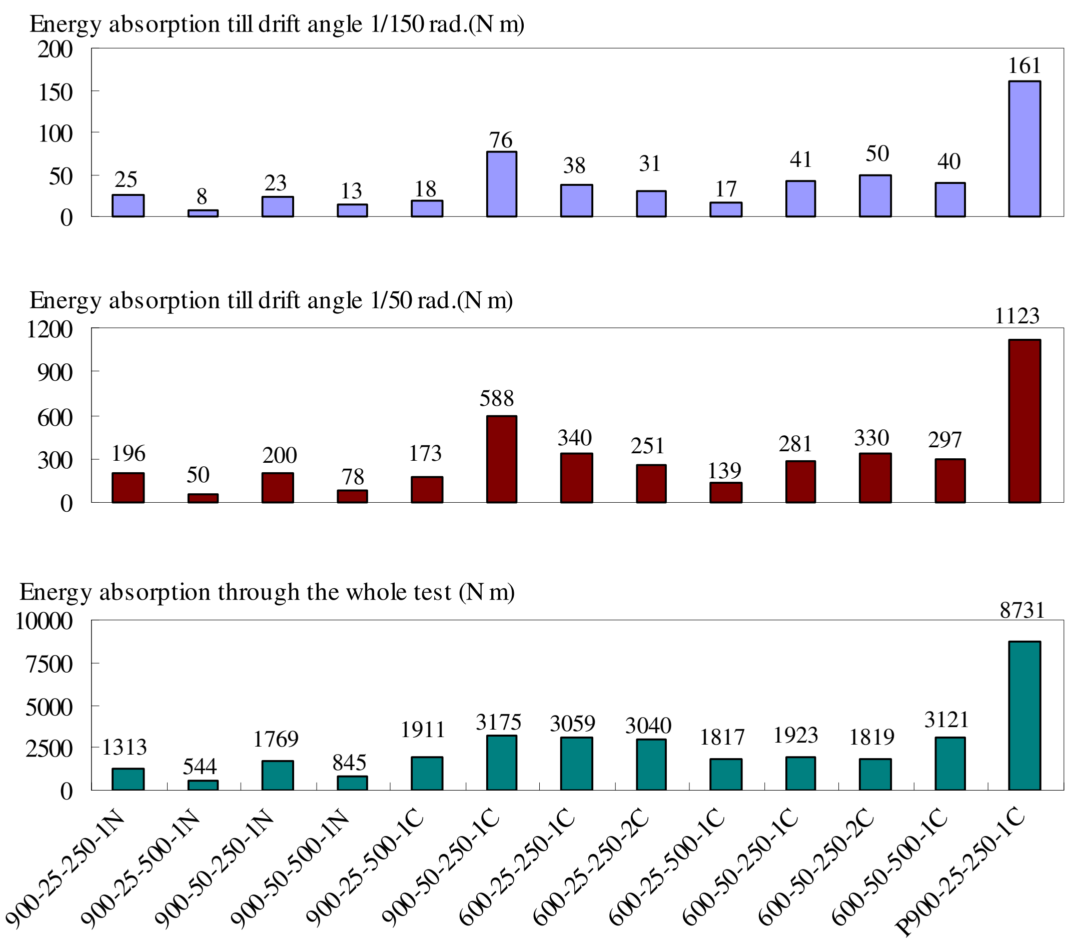


Figure 8 Energy absorption

CONCLUSIONS

The test results of this research indicated that the suitable layout of slits for a wooden framework could be *b*=25mm and *l*=250mm or 500mm because of their stable behavior and good structural performance. The specimens with out-of-plane strengthening reached their calculated shear and their hysteretic loops were spindled. Besides, the load-carrying capacities, energy absorbing capacities and wall strength ratios of the specimens with out-of-plane strengthening were increased as compared to the specimens without out-of-plane strengthening. On the contrary, the rigidity evaluation equation could not estimate the rigidities of steel-wood hybrid specimens precisely.

The full-scale specimen showed good structural performance. In order to utilize the proposed hybrid resisting system to wooden structures, more experiments on full-scale specimens should be conducted.

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