CYCLIC TESTS ON STEEL – WOODEN HYBIRD RESISTING SYSTEM

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ABSTRACT :

This paper presents experimental study on steel–wood hybrid resisting system that can be used in Japanese conventional wooden frameworks to resist lateral forces. As aseismic element, a thin steel plate with slits was adopted and it was fixed inside a wooden frame. The experimental work consists of 4 small–scale steel–wood hybrid specimens and 3 small–scale wooden frames. All specimens were subjected to repeated horizontal load. Four kinds of layout of slits were chosen as experimental parameter. Out of plane strengthening elements were used in these 4 steel–wood hybrid specimens. The test results showed that these steel–wood hybrid specimens behaved in a ductile manner up to 6~10% of storey drift angle without strength degradation, and their wall strength ratios are larger than 5.0. The maximum horizontal loads of these specimens were about twice of their calculated ultimate strengths.

KEYWORDS : cyclic test, steel, shear wall, wooden framework, stiffener

1. INTRODUCTION

In previous researches on steel–wood hybrid resisting shear walls, where steel plates with slits were fixed on one side of wooden frames (Li, 2004) showed that aseismic elements with out of plane strengthening had excellent structural performance subjected to repeated horizontal load. In Japanese conventional wooden frameworks, however, columns are usually not sealed by walls. In other words, wall elements are fixed inside the wooden frameworks so that the beautiful wood columns are visible (This kind of wall is called as SHIN KABE in Japanese). In this study, SHIN KABE specimens were tested under cyclic horizontal load to investigate their strengths and deformation capacities.

The shear strength Q_{wt} of a steel–wood hybrid shear wall can be calculated on the basis of full plastic moments of the upper and the lower ends of flexural columns (flexural column means the steel plate segment between two slits) and the equation is as follows (Li, 2004).

$$Q_{wt} = \frac{nM_p}{l/2} = \frac{ntb^2}{2l} \cdot \sigma_y \tag{1.1}$$

where, M_p = full plastic moment of a flexural column; n = number of flexural column; l = length of slit; b = width of flexural column or interval of slit; t = thickness of steel plate; and σ_y = yield stress of steel.

The formula for rigidity K of a steel plate with slits was conduced where the flexural deformation of flexural columns and shear deformation of steel plate were considered, and the equation is as follows (Li, 2004).

$$K = \frac{Q_w}{\Delta} = \frac{1}{\dfrac{\kappa h}{GBt} \cdot m + \dfrac{l^3}{Etb^3} \cdot \dfrac{m}{n}} \tag{1.2}$$

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

Besides, the strengthening wood plates were determined on the basis of making the critical buckling force of flexural column became larger than its calculated shear strength, and the strengthening method was suggested by Li (2004).

2. CYCLIC TEST

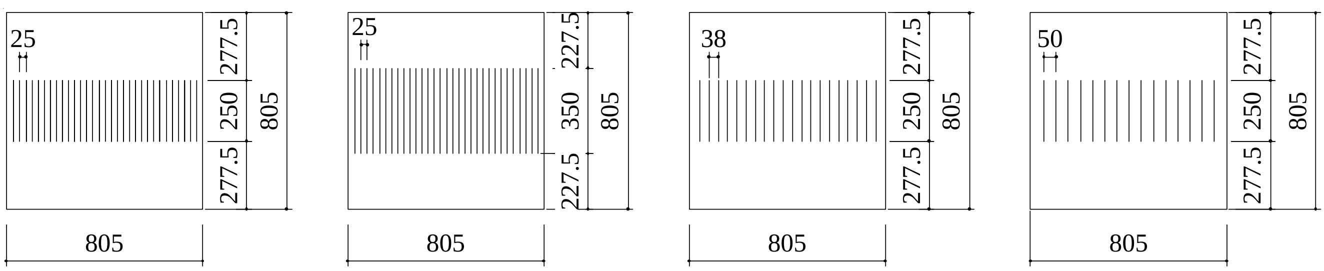
2.1 Specimen

Experiments were conducted to investigate the structural performance of the steel–wood hybrid resisting system. The experimental work consists of 7 small–scale specimens. 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) were considered in this experimental work based on the experimental results of previous research (LI, 2004). The layout of slits is shown in Figure 1. The slits were fabricated by laser-cut with a 0.5mm width. Steel plates of 1.2mm thickness (SPHC steel) were used in steel–wood hybrid specimens, because thin plate was considered suitable for a wooden framework. At the upper and the lower ends of flexural columns, out of plane strengthening elements (wooden plates of 12mm or 25mm thickness) were used, and the whole steel plate was stiffened by wood edge stiffeners with cross section of 45mm x 45mm or 50mm x 50mm. The wood edge stiffeners were around the circumference of a steel plate. Out of plane strengthening elements and edge stiffeners were fixed to the steel plate by M6 bolts to prevent the steel plate from out of plane buckling during early loading stage. The steel plate with stiffeners was fixed to a wooden framework by 32 screws. The details of specimen 805-25-250-1 are shown in Figure 2. Table 1 shows the mechanical properties of steel and Table 2 is the details of specimens. The width of strengthening plate is determined according to the suggested method described by Li (2004).

2.2 Test Set-up

Photo 1 shows a specimen set up in the loading frame. Each specimen was tied to the loading frame using four M22 high–strength bolts and the horizontal displacement of the sill was restrained by apparatuses from two sides. The wood beam was prevented from moving out of plane by two apparatuses fixed on the loading frame. The horizontal load was applied to the specimen through a hydraulic jack indicated in Photo 1. The horizontal load was measured by a load cell connected to the hydraulic jack.

Horizontal loads were applied by displacement–controlled procedure and repeated once at storey 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 3, after these loading cycles, horizontal load was applied monotonically until about 10% of storey drift angle was achieved. The horizontal displacements of the beam and the sill in a frame as well as the vertical displacements of column bases were measured by displacement transducers. The storey drift angle is the relative displacement between beam and sill divided by the clear distance between beam and sill. The vertical displacements of column bases are used to calculate the rotation angle of wood column base.



(a) 805-25-250-1 (b) 805-25-350-1 (c) 805-38-250-1 (d) 805-50-250-1

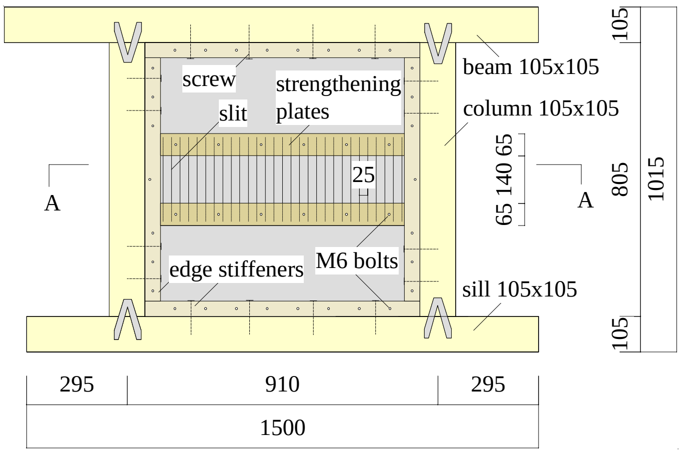
Figure 1 Layout of slits

Table 2.1 Mechanical properties of steel

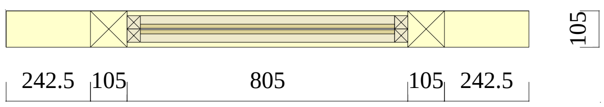
Young's Modulus (kN/mm^2)	Yield Stress (N/mm^2)	Tensile Strength (N/mm^2)	Yield Ratio	Throttle (%)	Elongation (%)
202.6	215.9	323.4	0.67	41.6	45.2

Table 2.2 Specimens

Specimen name	Steel plate						Cross section of edge stiffener (mm^2)	Strengthening plate	
	Width (mm)	Height (mm)	Thickness (mm)	Interval of slits (mm)	Slit length (mm)	Layer number of slits		Width (mm)	Thickness (mm)
F-W45-N	-	-	-	-	-	-	45 mm square	-	-
F-W45-H	-	-	-	-	-	-	45 mm square	80	25
F-W50-H	-	-	-	-	-	-	50 mm square	85	25
805-25-250-1	805	805	1.15	25	250	1	45 mm square	65	12
805-25-350-1	805	805	1.15	25	350	1	45 mm square	100	12
805-38-250-1	805	805	1.15	37	250	1	45 mm square	80	25
805-50-250-1	805	805	1.15	50	250	1	50 mm square	85	25



(a) Elevation



(b) A-A cross section

Figure 2 Details of specimen 805-25-250-1

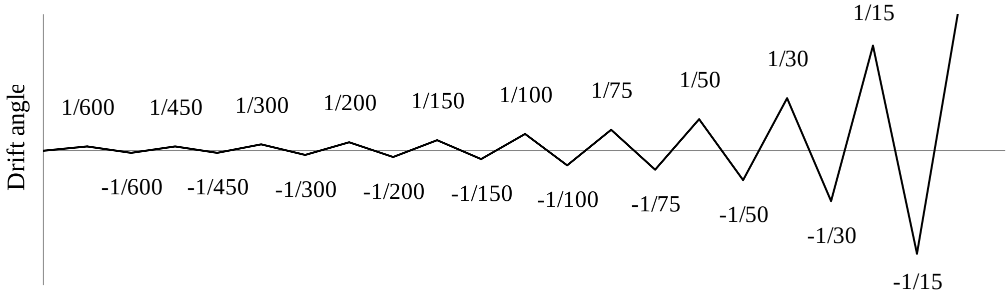


Figure 3 Loading program

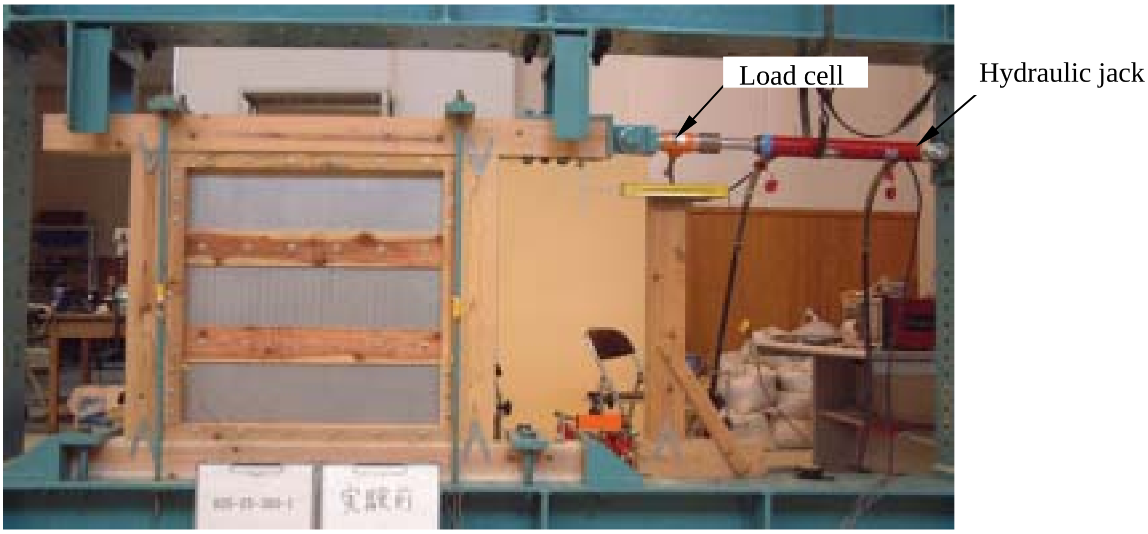


Photo 1 Test set-up

3. TEST RESULTS AND DISCUSSIONS

3.1 Lateral Force – Drift Angle Relations

Lateral force – drift angle relations of specimens are shown in Figure 4. The drift angle on the transverse axis is calculated by subtracting the rotation angle of wood column bases from the storey drift angle. Figure 4(c2) is the previous research result of a wood frame. The maximum force of a pure wood frame is only about 4kN. When edge stiffeners and strengthening plates were fixed in a wood frame, the maximum force increased to more than 9.6kN. This is because of the bearing between wood frame and edge stiffeners, as well as between edge stiffeners and strengthening plates.

Specimen 805-25-250-1 was loaded only till 6.7% of drift angle because of the capacity of hydraulic jack. For the other three specimens, a hydraulic jack of large capacity was used.

In Figure 4(d), (e), (f), and (g), the horizontal solid lines are the calculated ultimate shear strengths Q_{wt}, and the

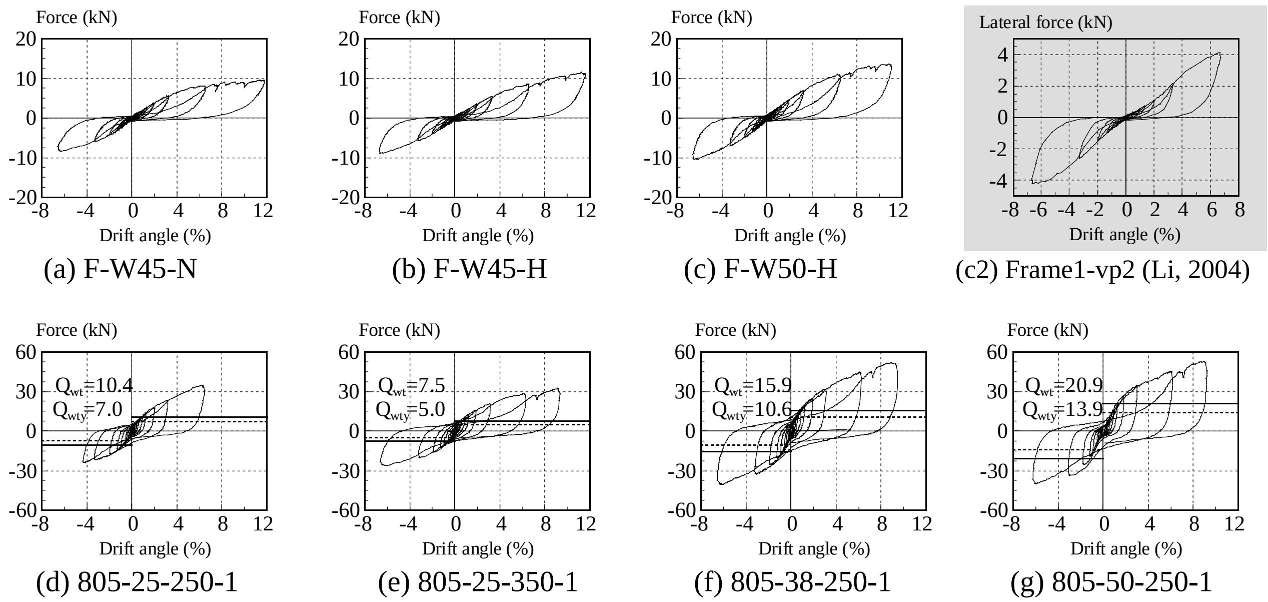


Figure 4 Test results

dotted lines the calculated yield shear strengths Q_{wty} which are 2/3 of strengths Q_{wt}. The maximum strengths of all steel–wood hybrid specimens are about twice of the calculated ultimate load–carrying capacities.

As can be seen from Figure 4, the hysteresis loops of the steel–wood hybrid specimens were close to spindle-shaped. Though out of plane stiffening was used, out of plane deformation of steel plate occurred in these specimens as shown in Photo 2. Deformations are concentrated in flexural columns in specimen 805-25-250-1, however, in the other three specimens, plate buckling occurred rather than the flexural deformation of flexural columns.

3.2 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.

$$\text{wall strength ratio} = \frac{\min\left\{P_{1/150}, P_y, \tfrac{2}{3}P_{\max}, 0.2P_u / D_s\right\}}{1.96L}\alpha \qquad (3.1)$$

where, $P_{1/150}$ = 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 (see Table 3); 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 wall strength ratio equals to 1.0. The elastic-plastic model of a specimen was determined depending on equivalent energy absorption of the envelope curve of force–displacement relation of the specimen.

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 wall strength ratios of the hybrid specimens are larger than 5.0, which means that the lateral loading resistance is larger than 10kN for a shear wall of 1 meter width. Wall strength ratio of 805-50-250-1 is smaller than that of 805-38-250-1, which is because the out of plane deformation occurred in specimen 805-50-250-1 at very small horizontal deformation.

3.3 Energy Absorption

Figure 5 shows the energy absorption from the beginning of a loading test to drift angle 1/150rad., 1/50rad., 1/15rad., and to the end of the test, denoted as ENERGY(1/150), ENERGY(1/50), ENERGY(1/15), and ENERGY(whole), respectively. Theoretically, the ascending order of energy absorption at each state should be 805-25-350-1, 805-25-250-1, 805-38-250-1, and 805-50-250-1. ENERGY(whole) of 805-25-250-1 if smaller than that of 805-25-350-1 because horizontal load was discontinued after 1/15 drift angle. Energy absorption of 805-50-250-1 at each state is smaller than that of 805-38-250-1, which is because out of plane deformation occurred early in specimen 805-50-250-1 and the lateral force – drift angle curve is somewhat slip–shaped.

(a) 805-25-250-1 (b) 805-25-350-1 (c) 805-38-250-1 (d) 805-50-250-1

Photo 2 Specimens at drift angle of 1/15 or 1/13 radian

Table 3 Test results

Specimen name	Q_{wty} (kN)	Q_{wt} (kN)	$P_{1/150}$ (kN)	P_y (kN)	P_u (kN)	D_s	$P_u \times 0.2/D_s$ (kN)	P_{max} (kN)	$P_{max} \times 2/3$ (kN)	Wall strength ratio
F-W45-N	-	-	1.7	6.2	8.7	0.47	3.7	9.6	6.4	0.92
F-W45-H	-	-	1.6	6.4	9.9	0.62	3.2	11.6	7.7	0.89
F-W50-H	-	-	2.2	8.1	12.2	0.61	4.0	13.6	9.0	1.22
805-25-250-1	7.0	10.4	10.3	17.7	30.6	0.61	10.1	34.4	22.9	5.66
805-25-350-1	5.0	7.5	9.5	19.2	27.7	0.50	11.0	32.2	21.5	5.32
805-38-250-1	10.6	15.9	15.6	27.8	44.0	0.48	18.5	51.8	34.5	8.74
805-50-250-1	13.9	20.9	13.8	29.9	44.4	0.47	19.1	52.6	35.1	7.75

Denotations: Q_{wty}: the calculated yield shear strength; Q_w: the calculated ultimate shear strength; $P_{1/150}$: the force at drift angle 1/150; 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.

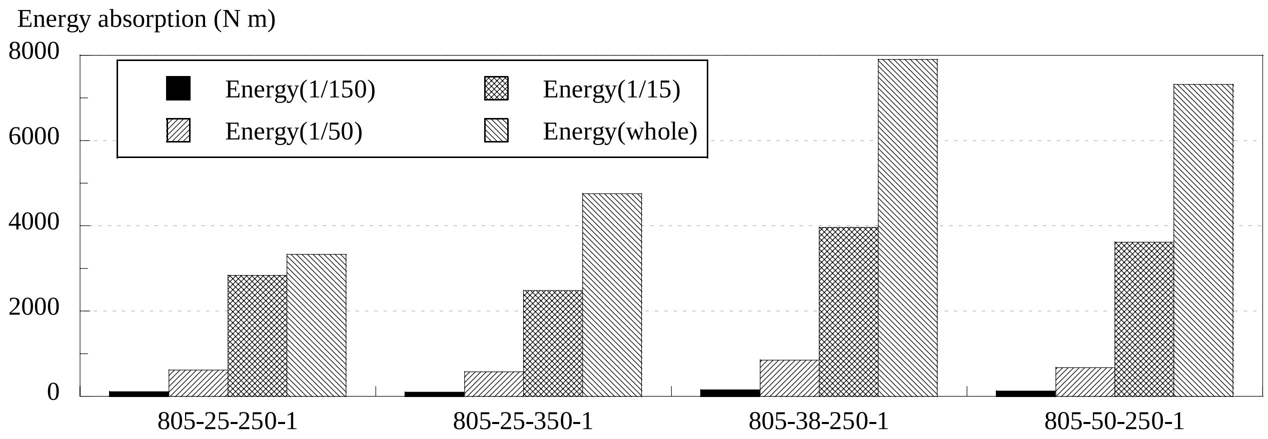


Figure 5 Energy absorption

4. CONCLUSIONS

The test results showed that all steel–wood hybrid specimens behaved in a ductile manner up to 6~10% drift without strength degradation. The maximum lateral loads of these specimens were about twice of their calculated ultimate strengths. The wall strength ratios of these hybrid specimens are larger than 5.0. For specimens of high load–carrying capacity, such as 805-50-250-1, out of plane deformation is difficult to be prevented even though out of plane stiffening is adopted. The suitable layout of slits for a wooden framework could be 25mm of slit interval and 250mm of slit length because of its stable behavior and good structural performance.

REFERENCES

Li., L (2004), An experimental study on steel – wooden hybrid resisting system, *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 2811