An Experimental Study on Full-scaled Steel - Wooden Hybrid Shear Walls

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SUMMARY:
Cyclic loading tests were performed on 12 Japanese conventional wooden frameworks (910mm in length and 2730mm in height). Five of them (PF specimens) were wooden frameworks, and the other 7 specimens (P805) were reinforced with a steel shear wall with slits and its out-of-plane stiffeners. The shear wall was set in the center of a wooden framework. The test results showed that PF specimens behaved in a ductile manner up to more than 6% drift without strength degradation. The wall strength ratios of all P800 specimens are larger than 3.79. P805 specimens showed stable behaviour when two pieces of steel plate and a connection member were used. The connecting member with cross-section of 45mm x 105mm can be thought more practical. It may be said that there was little influence on the maximum lateral force due to loading method, although deformation capacity of specimen under tie-rod loading method was somewhat smaller because of the earlier cracks in sill.

Keywords: Steel shear wall with slits, cyclic loading, stiffener, drift angle

1. GENERAL INSTRUCTIONS

Previous researches on small–scaled steel – wooden hybrid shear walls (910mm in length and 910mm in height) showed that seismic elements with out–of–plane strengthening had excellent structural performance subjected to repeated horizontal loads (Li, 2004, 2008, 2011). The shear strength and rigidity of a steel–wooden hybrid shear wall with slits can be calculated on the basis of full plastic moments at the upper and the lower ends of flexural columns (steel plate between slits), and they can be easily controlled by slit design that includes the distance between slits (b), the length of slit (l), and the layer number of slit (m) (Li, 2004). Besides, the stiffening wood plates for flexural columns were determined on the basis of making the critical buckling force of a flexural column became larger than its calculated shear strength, and the stiffening method was suggested (Li, 2004).

A series tests had been carried out, where the experimental parameters: slit design, strength of stiffening plates for flexural columns, connection methods of edge stiffener, number of cyclic load (1 or 3), workability of construction, and effects of seismic strengthening, were considered (Li, 2011). The test results showed that slit patterns of b=25mm, l=250mm or 350mm, end lap joint in the connections of edge stiffeners, one cycle of lateral loading, a smallish steel wall (with space of 2mm between the framework and the shear wall), and the values of M_{cr}/M about 1.0 were acceptable. Values of M_{cr}/M can be adjusted by changing the widths of stiffening plates, where, M_{cr} is the lateral torsion buckling moment of flexural columns between stiffening plates, and M is the maximum moment of the flexural columns between stiffening plates when full plastic moments occur at the top and the bottom of flexural columns.

In Japanese conventional wooden frameworks, wall elements are fixed inside the wooden frameworks so that the beautiful wood columns are visible (this kind of wall is called as Sinkabe in Japanese). In this study, 12 full–scaled Sinkabe specimens were tested under cyclic horizontal load to investigate their strengths and deformation capacities.

2. CYCLIC TESTS

2.1. Specimens

As experimental parameters, slit design, strength of stiffening plates for flexural columns, cross-section of connection member, loading method were considered. Table 2.1. shows the details of specimens. The length of frame is 910mm and the height is 2730mm. The size of steel shear wall is 2mm smaller than the inner dimensions of wooden frames in consideration of workability. The slit design included the combinations of b=25mm, l=250mm or 350mm, and m=2 (only in T0 specimen: m=4). The thickness of steel plate was 1.2mm or 1.6mm. Stiffening wood plates at the top and the bottom of flexural columns, and wood edge stiffeners (cross section: 45mm x 45mm) around the circumference of a steel plate were fixed to the steel plate by M6 bolts. End lap joint was used in the connections of edge stiffeners. The steel plate with stiffeners was fixed to a wooden framework by screws. The details of connections between columns and connection member are described in the notes of Table 2.1. Fig. 2.1. shows the details of specimen P805-25-350-2-T45. Table 2.2. shows the mechanical properties of 3 kinds of steel.

Table 2.1. Details of Specimens

No.	Specimen name	Steel wall				flexual column			Stiffening plate			Section of connection member (mm×mm)	refer-ence frame
		Steel	Think-ness (mm)	Width (mm)	Height (mm)	Width (mm)	Height (mm)	Height*4 (mm)	Thick-ness (mm)	Width (mm)	$\frac{Mcr}{M}$		
1	P805-25-350-4-T0	i	1.6	803	2623	25	350	267	18	60	1.16		8
2	P805-25-350-2-T105*1	i	1.6	803	1258	25	350	267	18	60	1.16	105×105	9
3	P805-25-250-2-T105(T)*1	ii	1.2	803	1258	25	250	169	15	60	1.13	105×105	10
4	P805-25-250-2-T105*1	iii	1.2	803	1258	25	250	169	15	60	1.13	105×105	9
5	P805-25-250-2-T45*1	ii	1.2	803	1288	25	250	169	15	60	1.13	45×105	11
6	P805-25-350-2-T45*2	iii	1.2	803	1288	25	350	240	18	65	0.89	45×105	12
7	P805-25-350-2-T45S*3	iii	1.2	803	1288	25	350	210	18	80	1.02	45×105	12
8	PFW-T0(T)*1								18	60			
9	PFW-T105*1								18	60		105×105	
10	PFW-T105(T)*1								18	60		105×105	
11	PFW-T45(T)*1								15	60		45×105	
12	PFW-T45S*3								18	80		45×105	

Notes:
*1 Connection member was set in 8mm notches of columns and was fixed by 2 screws (90mm in length) at each notch.
*2 Connection member was set in 8mm notches of columns and was fixed by L-shaped hardware and screws.
*3 Connection member was set in 15mm notches of columns and was fixed by L-shaped hardware and screws.
*4 the clear height of flexural column between stiffening plates

Table 2.2. Mechanical Properties of Steel

Steel	Young's Modulus (kN/mm^2)	Yield Stress (N/mm^2)	Tensile strength (N/mm^2)	Yield Ratio	Throttle (%)	Elongation (%)
i	189.2	248.3	346.9	0.72	36.9	41.5
ii	199.2	263.0	382.8	0.67	39.1	36.8
iii	183.4	294.6	382.5	0.69	23.4	37.9

2.2. Loading Set-up

Fig. 2.2. shows the loading set-up. Fig. 2.2.(a) is tie rod loading method and (b) is fixed end loading method. Specimens that were tested by tie rod loading were added a notation of (T) to the specimen name. Horizontal loads were applied by displacement–controlled procedure and repeated once at storey drift angle amplitudes of 1/600, 1/450, 1/300, 1/200, 1/150, 1/100, 1/75, 1/50, 1/30, 1/15 rad. (see Fig. 2.3.), after 1/15rad., monotonic loading was applied till about 1/13rad.

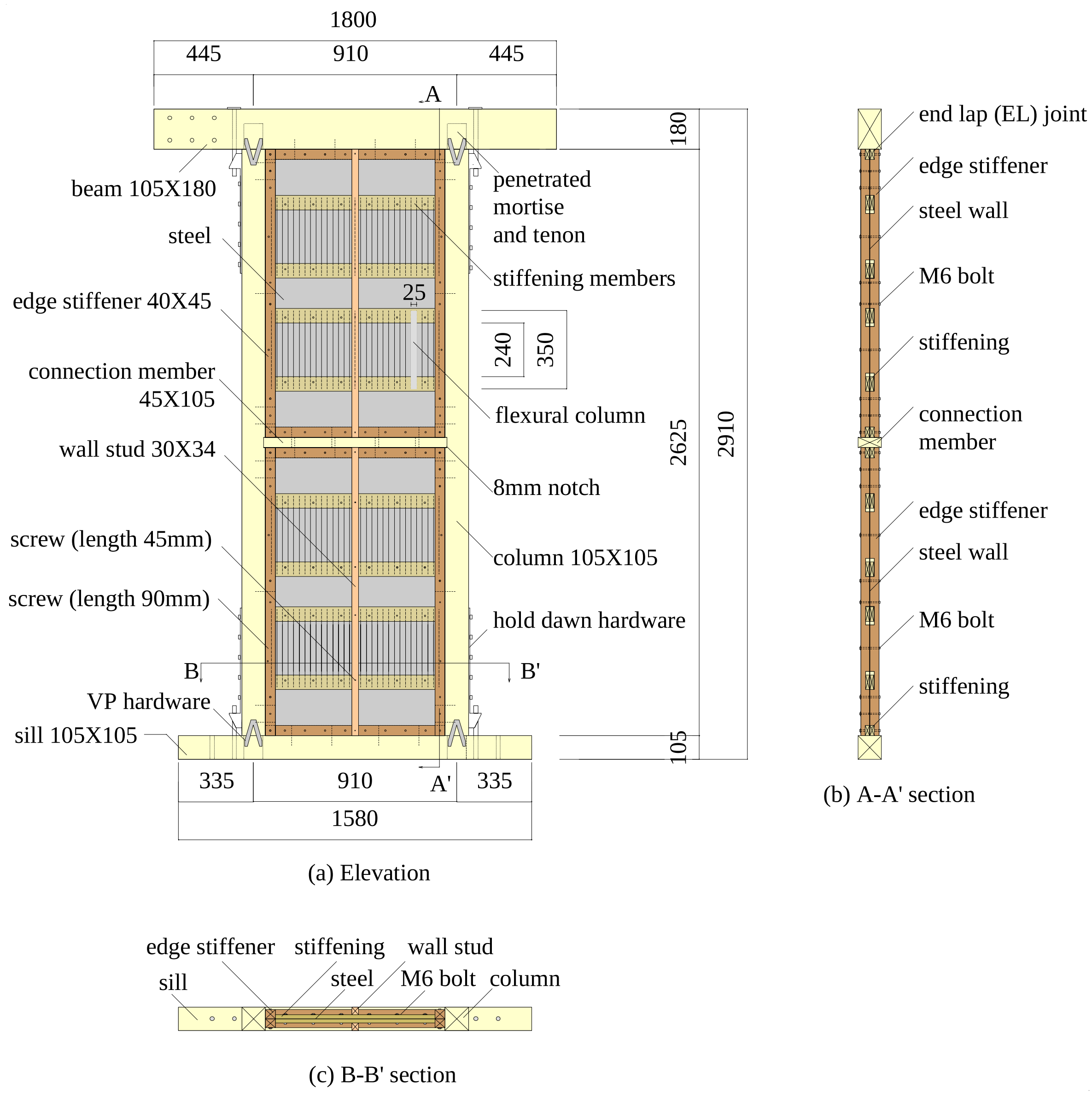


Figure 2.1. Details of specimen P805-25-350-2-T45

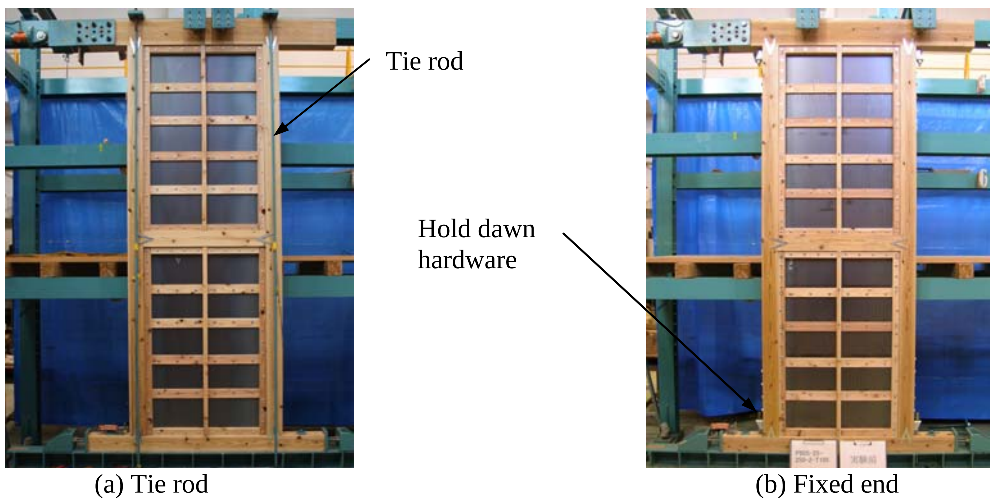


Figure 2.2. Loading method

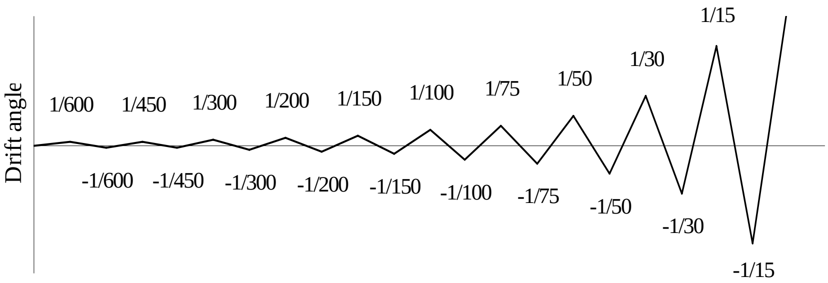


Figure 2.3. Loading program

3. TEST RESULTS AND DISCUSSIONS

3.1. Lateral Force – Drift Angle Relations

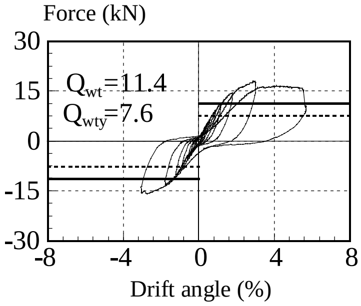
Lateral force – drift angle relations of specimens are shown in Fig. 3.1. The drift angle on the transverse axis is calculated by subtracting the rotation angle of wood column bases from the storey drift angle. In Figs. 3.1.(a)~(g), the horizontal solid line and the dashed line are the calculated ultimate shear strengths Q_{wt}, and the calculated yield shear forces Q_{wyt} (=2/3Q_{wt}), respectively (Li, 2004). In addition, lateral force – drift angle relations of P805 specimens is near in a spindle and has high energy absorption capacity. The maximum strength of a P805 specimen is larger than its calculated ultimate load–carrying capacity. Table 3.1. shows the strength ratios of test results till drift angle 1/15rad. to the calculations. From Table 3.1., it is clear that the ratios of strength of steel wall (P_{max}–P_f) (strength of its referencing framework, P_f, was subtracted) to its calculated load-carrying capacity (Q_{wt}) are 1.2~1.7. From Figs. 3.1.(h)~(l), the PF specimens behaved in a ductile manner up to more than 6% drift without strength degradation, and lateral force – drift angle relations are slip type . The maximum forces of PF specimens with edge stiffeners and stiffening plates were 3.0 ~ 8.5kN.

Fig. 3.2. shows the failure mode of P805-25-250-2-T45*1 and P805-25-350-2-T45*2 at drift angle –1/15rad. The details of notations *1 and *2 are described in the notes of Table 3.1. In Fig. 3.2.(a) of specimen P805-25-250-2-T45*1, the connection member was put out of the right column while screws were broken. The drift angle of a steel shear wall with stiffening members did not coincide with that of the whole framework. From Fig. 3.2.(b), cracks of column near the notch were observed but no other serious failure occurred. For specimens that connection member of 105 X 105 was used, no large damage was observed, either.

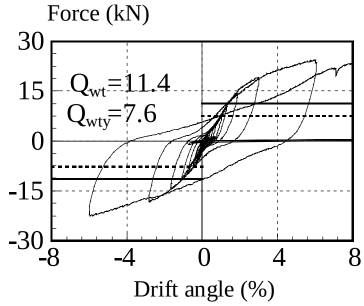
Figs. 3.3.~3.5. show the comparisons based on the skeleton curves of test results. Specimen names in Figs. 3.3.~3.5. are slit length, layer number of slit, thickness of connecting member, and loading method.

From Fig. 3.3., it can be seen that there was no out-of-plane buckling occurred till drift angle 1/15rad. in a T45 or a T105 specimen that two pieces of steel shear wall and a connection member were used, and the load–carrying capacity of T105 was higher than that of T0 or T45 specimen. The reason why load–carrying capacity of T105 was higher than T45 is because the steel used in T105 had higher yield stress. It can be said that there was almost no difference between a T45 and a T105 except the influence of steel strength. No obvious failure took place in both T45 and T105 specimens. Therefore, T45 is considered adoptable and practical because there is less damage in column due to notch and it is easy for construction when the steel shear wall is used as seismic strengthening for existing buildings.

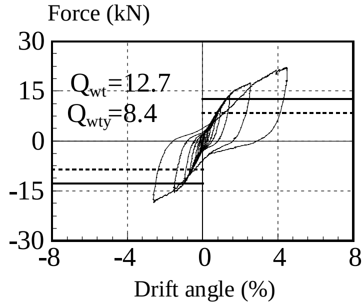
From Fig. 3.4., it can be seen that there was no difference at the initial stage between 350-2-T45 and 350-2-T45S. 350-2-T45S reached its highest resistance at drift angle 1/22.5rad. The out–of–plane deformation of flexural columns of 350-2-T45 occurred earlier because the value of its M_{cr}/M was a



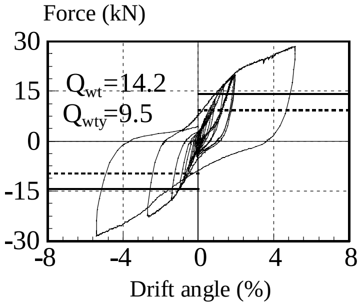
(a) P805-25-350-2-T0



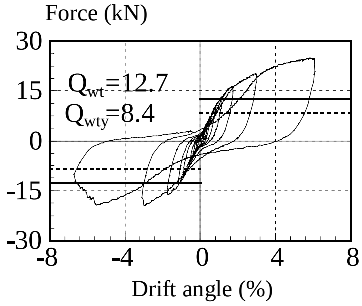
(b) P805-25-350-2-T105



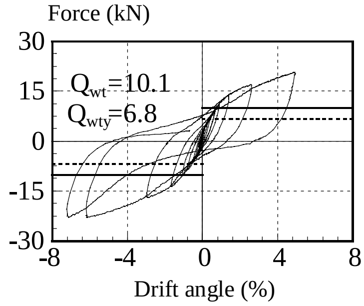
(c) P805-25-250-2-T105(T)



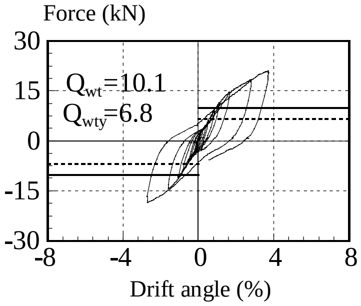
(d) P805-25-250-2-T105



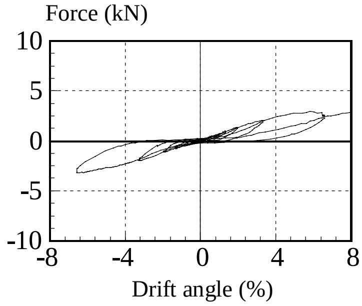
(e) P805-25-250-2-T45



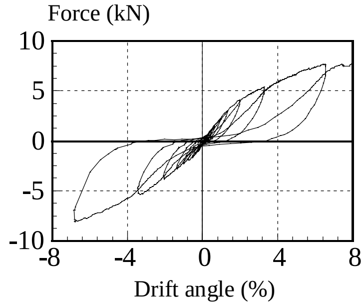
(f) P805-25-350-2-T45



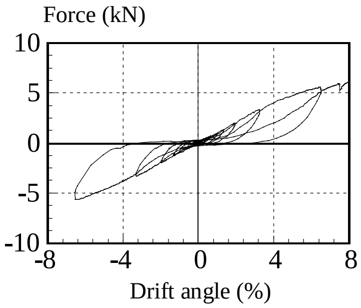
(g) P805-25-350-2-T45S



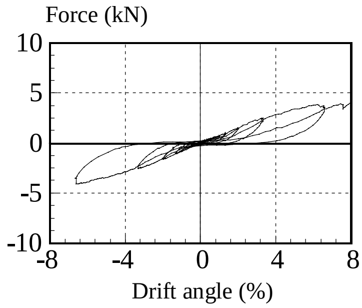
(h) PFW-T0(T)



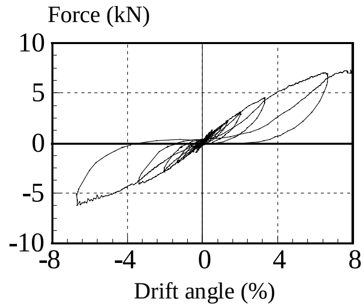
(i) PFW-T105



(j) PFW-T105(T)



(k) PFW-T45(T)



(l) PFW-T45S

Figure 3.1. Test results

(a) P805-25-250-2-T45

(b) P805-2-350-2-T45

Figure 3.2. Failures of Connections at Drift Angle -1/15rad.

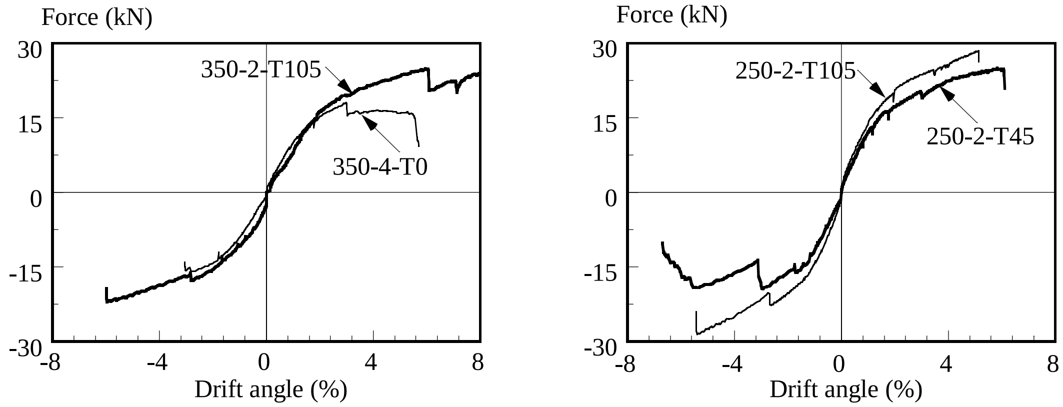


Figure 3.3. Effects of cross-section of connection member

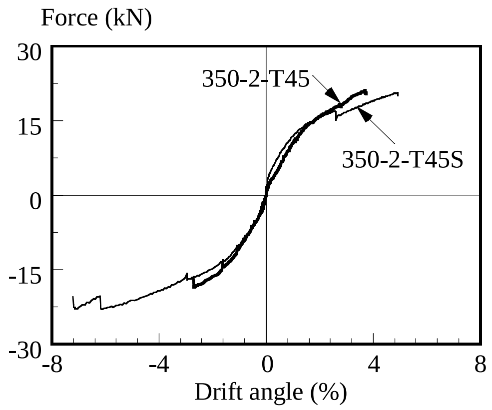


Figure 3.4. Effects of M_{cr}/M

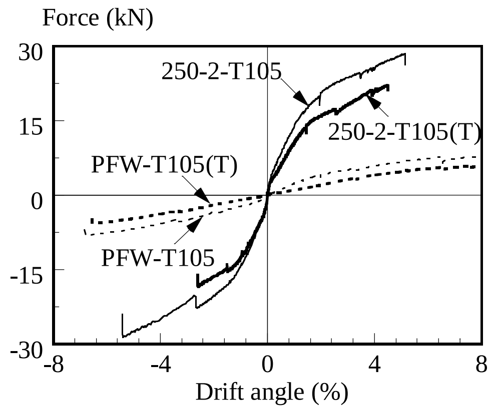


Figure 3.5. Effects of loading method

little smaller. The earlier out–of–plane deformation led to a drop in load–carrying capacity of flexural columns. On the other hand, as for specimen P805-25-350-2-T45S which did not have deformation of flexural columns much more, the wooden sill has failed first. In order to prevent damage of wooden sill occurs earlier, it is necessary to make the value of M_{cr}/M lower and let the flexural columns of steel deform earlier than the failure of wooden sill.

Fig. 3.5. shows the influence of loading method. By the comparison of tie rod loading to fixed end loading of wooden frame specimens in Fig. 3.5., there was a difference of about 2 kN in the load–carrying capacity. The load–carrying capacity of 250-2-T105 was almost the same as 250-2-T105(T) to drift angle 1/30rad, but 5kN higher than 250-2-T105(T) at drift angle 1/15rad. It is thought that this is because the yield stress of steel of 250-2-T105 was high.

3.2. Wall Strength Ratios

A technical term called wall strength ratio is used to evaluate the strength of a shear wall in wooden framework, 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 by Eq.(1).

$$\text{wall strength ratio} = \frac{\min\left\{P_{1/150} \text{ or } P_{1/120}\,,\ P_y,\ \tfrac{2}{3}P_{\max},\ 0.2P_u / D_s\right\}}{1.96L}\alpha \tag{3.1}$$

where, $P_{1/150}$ = the force at drift angle 1/150rad. of tie rad loading; $P_{1/120}$ = the force at drift angle

1/120rad. of fixed end loading; P_y = yield strength in elastic–plastic model based on test result till 1/15rad.; P_u = ultimate strength in elastic - plastic model; D_s = structural characteristics factor; P_{max} = maximum force between drift angles 0 and 1/15rad.; 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 1.96kN/m=horizontal strength 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 between drift angle –1/15 and 1/15 rad.

Table 3.1 shows the values of wall strength ratio obtained from test results till drift angle 1/15rad. As shown in Table 3.1., the wall strength ratios of P805 specimens were between 3.79 and 5.23 and the effects of the values of M_{cr}/M (0.89~1.16) were not as significant as that observed in small–scaled specimens (Li, 2010). Besides, the effects of steel strength and cross–section of connection member are also not significant. Wall strength ratio, load–carrying capacity, and deformation capacity of specimen tested under fixed end loading tends to be somewhat higher than that under tie rod loading.

Table 3.1. Test Results

Specimen name	Q_{wty} (kN)	Q_{wt} (kN)	$P_{1/150}$ (kN)	$P_{1/120}$ (kN)	P_y (kN)	P_u (kN)	D_s	P_{max} (kN)	P_a (kN)	Strength ratio	$\frac{P_y}{Q_{wty}}$	$\frac{P_u}{Q_{wt}}$	$\frac{P_{max}-P_f}{Q_{wt}}$
P805-25-350-2-T0	7.6	11.4	--	8.1	10.6	16.1	0.4	18.1	7.3	4.10	1.4	1.4	1.3
P805-25-350-2-T105	7.6	11.4	--	7.9	15.6	22.2	0.5	24.9	7.9	4.43	2.1	1.9	1.5
P805-25-250-2-T105(T)	8.4	12.7	7.6	--	13.8	19.4	0.5	22.1	7.1	4.00	1.6	1.5	1.3
P805-25-250-2-T105	9.5	14.2	--	9.6	16.4	24.4	0.5	28.4	9.3	5.23	1.7	1.7	1.5
P805-25-250-2-T45	8.4	12.7	--	9.2	14.8	21.9	0.5	25.0	9.2	5.14	1.8	1.7	1.7
P805-25-350-2-T45	6.8	10.1	--	7.8	12.6	19.5	0.5	23.0	7.7	4.34	1.9	1.9	1.4
P805-25-350-2-T45S	6.8	10.1	--	7.8	12.4	18.8	0.6	21.1	6.8	3.79	1.8	1.8	1.2
PFW-T0(T)			0.5	--	1.9	2.7	0.7	3.0	0.5	0.27			
PFW-T105			--	2.0	4.0	6.3	0.5	7.7	2.0	1.14			
PFW-T105(T)			0.7	--	4.2	5.2	0.8	5.6	0.7	0.39			
PFW-T45(T)			0.5	--	3.0	4.4	1.0	3.9	0.5	0.29			
PFW-T45S			--	1.3	4.8	7.5	0.9	8.5	1.3	0.70			

4. CONCLUSIONS

The test results showed that PF specimens behaved in a ductile manner up to more than 6% drift without strength degradation. The load–carrying capacities of P805 specimens were 1.2~1.7 times of their calculated ultimate strengths. The wall strength ratio of P800 specimens is larger than 3.79. P805 specimens showed stable behaviour when two pieces of steel plate and a connection member were used. The connecting member with cross–section of 45mm x 105mm can be thought more practical, which can be easily adopted as seismic strengthening in existing buildings. It may be said that the influence on load–carrying capacity due to loading method is not quite large, although wall strength ratio and deformation capacity of specimen under tie rod loading was somewhat smaller because of the earlier cracks in wooden sill.

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