HYSTERETIC PERFORMANCE OF SHEAR PANEL DAMPERS OF ULTRA LOW-YIELD-STRENGTH STEEL FOR SEISMIC RESPONSE CONTROL OF BUILDINGS

Kiyoshi TANAKA[1] And Yasuhito SASAKI[2]

SUMMARY

Energy dissipating members play an important role in seismic response controlled structures. It is noticed that low-yield-strength steel has an appeal point as a material of hysteretic damper in Japan. In this study, hysteretic performances of the damper of shear panel type with ultra low-yield-strength ($100N/mm^2$) steel under static loading are verified through cyclic loading tests. As hysteretic performances of the damper, maximum strength, allowable deformation and hysteretic rule are discussed. In order to estimate these essential performances of the damper, a generalized width-thickness ratio as a function of yield-strain and yield-ratio is introduced. And estimation formulae for the performances that contain the generalized width-thickness ratio are empirically obtained from the test results. As the panel dampers in actual use have reinforcing rib-plates to prevent shear buckling, a modified width-thickness ratio is introduced to evaluate equivalent width-thickness ratio correspond to that for square-shaped panel dampers. A modified Skeleton-Shift-Model is applied as the hysteresis model. Hysteresis curves simulated by this model agree well with the test results.

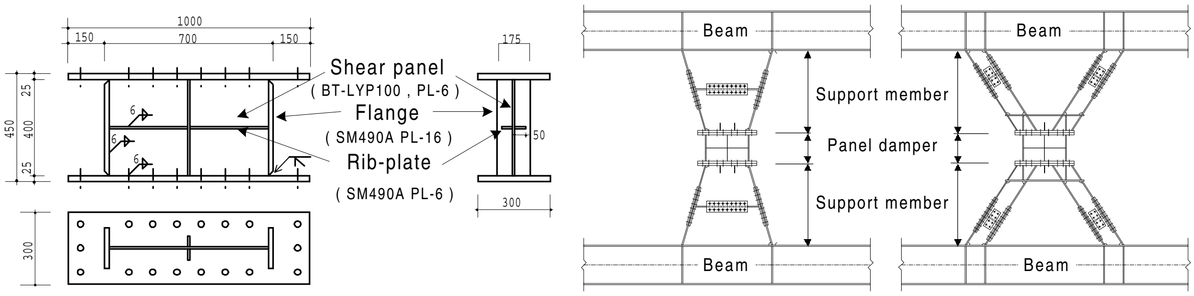
INTRODUCTION

Energy based seismic response control design approaches [Akiyama, 1985 et al.] have been widely applied to mainly high-rise steel buildings in Japan. In the design approaches, dampers that are installed in the building play an important role to dissipate energy during large earthquakes. In Japan, it is noticed that low-yield-strength (LYP) steel has an appeal point as a material of hysteretic damper. In this study, a shear panel damper with ultra low-yield-strength steel ($100N/mm^2$) is used as an energy-dissipating member. The panel damper has two distinguishing characteristics. One is the excellent ductile performance carried out by ultra low-yield-strength steel. Another is its adaptability to steel structures. Though the panel dampers have already been applied to many structures, information on accurate hysteretic performances of the dampers have not been obtained or utilized in seismic response control design. Several hysteretic behaviors of these panel dampers clarified in this paper are as follows: maximum shear strength, reliable energy dissipating capacity (allowable deformation) and hysteretic rule. In order to examine these performances of the damper, four series of static loading test programs on sixteen shear panel specimens were executed. On the other hand, a generalized width-thickness ratio, normalized by yield-strain and yield-ratio, is introduced to estimate the performances of the panel dampers. And also, since the panel dampers in actual use have reinforcing rib-plates, a modified width-thickness ratio is introduced to evaluate equivalent width-thickness ratio. A modified Skeleton-Shift-Model [Meng, Ohi and Takanashi, 1992] is applied as the hysteresis model.

[1] *Technology Research Institute, Fujita Corporation, Atsugi, Kanagawa, Japan Email: kiytanaka@fujita.co.jp*
[2] *Technology Research Institute, Fujita Corporation, Atsugi, Kanagawa, Japan Email: yasasaki@fujita.co.jp*

1. PRACTICAL APPLICATION

The shear panel dampers described in this paper have already been applied to about ten buildings for the purpose of response and damage control of main frames during large earthquakes [Tanaka and Torii et al., 1998b]. In case of tall steel buildings, seismic response controlled structures are more economical than ordinal seismic resistant structures. Figures 1 (a) and (b), (c) show an example of the panel damper and two typical arrangements of the damper, respectively. The panel damper would be set at nearly center height of the story and has support members to transfer shear force to main frames. The support members and beams should be rigid as far as possible, to obtain good response control ability.



(a) Panel damper (b) Pillar type (c) Bracing type

Figure 1: Example of practically applied panel damper and its arrangements in the frame

WIDTH-THICKNESS RATIOS

It is known that width-thickness ratio is a very important variable to prescribe elastic buckling strength of plate-girder and shear panel. In this study, two revised ratios will be used to estimate hysteretic performances of the panel damper in large plastic strain region.

Reduced width-thickness ratio

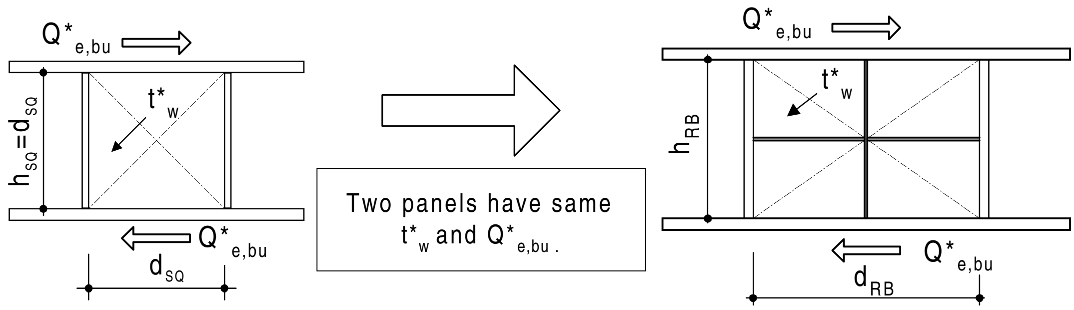
Normal width-thickness ratio (d/t_w) could be expressed by yield-strain (σ_y/E) in case of plastic buckling on H-shaped or Box-shaped steel members. Meanwhile, as the shear panel damper exhibits good hysteretic performance in a very large plastic strain region, it is more appropriate to select ultimate strength (σ_B) as the generalizing variable of the width-thickness ratio than yield strength. As a result, a width-thickness ratio by yield-strain and yield-ratio (σ_y/σ_B) is proposed as shown in Equation (1). This ratio is called "Reduced width-thickness ratio [$(d/t_w)_B$]", hereafter.

$$(d/t_w)_B = (d/t_w)\frac{\sqrt{\sigma_{wy}/E}}{\sqrt{\sigma_{wy}/\sigma_{wB}}} = (d/t_w)\cdot\sqrt{\sigma_{wB}/E} \quad (1)$$

Equivalent width-thickness ratio

Non-reinforced square-shaped shear panels (SQ-panel) shown in Figure 2(a) have a simple figure and also it is expected that its hysteretic performances have an evident correlation to normal width-thickness ratio. As will be discussed later, an evident correlation between hysteretic performances and width-thickness ratios is actually observed from test results on the SQ-panels. On the other hand, the panel damper in actual usage (RB-panel) has reinforcing rib-plates to prevent shear buckling of the panel, as shown in Figure 1 (a) and 2(b). When the SQ-panel and the RB-panel have almost the same hysteretic performances under the condition that both panel have same thickness and same material property, the width-thickness ratio of the RB-panel can be replaced with the ratio of the SQ-panel. "Equivalent width-thickness ratio [$(d/t_w)_{eq}$]" is proposed to realize this concept rationally. It is the practical definition of the equivalent width-thickness ratio of the RB–panel that when the elastic shear buckling strength ($Q^*_{e,bu}$) and panel thickness (t^*_w) of the SQ-panel and the RB-panel are the same, the equivalent width-thickness ratio of the RB–panel is equal to the normal width-thickness ratio of the SQ-panel. The elastic shear buckling strength of the RB-panel can be estimated by the energy equivalence method

[Timoshenko and Gere, 1961]. When the reduced width-thickness ratio of the RB-panel is to be obtained, the equivalent width-thickness ratio will be used in Equation (1), instead of normal width-thickness ratio.



(a) Non-reinforced square panel (SQ-panel) (b) Reinforced rectangular panel (RB-panel)

Figure 2: Concept of equivalent width-thickness ratio of reinforced rectangular panel

EXPERIMENTAL EXAMINATION

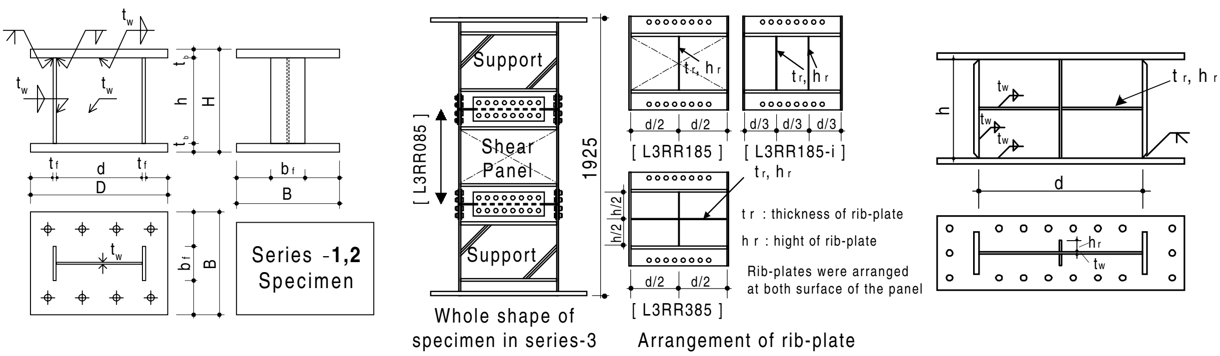
Four series of static loading test programs having sixteen shear panel specimens in all under cyclic loading were conducted to obtain estimation formulae for the performances and a hysteresis model of the panel dampers for the design purpose. Configurations of the panel specimen are listed in Table 1 and shown in Figure 3. Three types of loading apparatus applied in test programs are shown in Figure 4. Mechanical properties of used materials including ultra low-yield-strength steel are also listed in Table 1.

Shear Panel Specimen

Shear panel specimen commonly consists of a shear panel, two frame flanges and two end plates. Ultra low-yield-strength steel (LYP-100) is used for all shear panels and normal strength steels for flanges and end plates. As an exception, LYP-100 steel was used to the flanges of the specimen-L4RR385-4. There were two types of shear panel configuration: square-shaped and rectangular-shaped. And also, there were two types of reinforcing on the panel: non-reinforcing and rib-plate reinforcing. The normal or equivalent width-thickness ratio and the reduced width-thickness ratio of the each specimen are shown in Table 1. From the results of tensile test, the LYP-100 steel has a very low yield strength, a very small yield-ratio and a very large elongation property compared with the normal strength steel used in flanges, as shown in Table 1.

Table 1: Lists of shear panel specimens

Test Series	Specimen	Configuration									Property of used steel								Type of loading
		d (mm)	H (mm)	t_w (mm)	b_f (mm)	t_f (mm)	t_r (mm)	h_r (mm)	d/t_w [$(d/t_w)_{eq}$]	$(d/t_w)_B$	${}_w\sigma_y$ (tf/cm^2)	${}_w\sigma_B$ (tf/cm^2)	Y.R. (tf/cm^2)	elong. (%)	${}_f\sigma_y$ (tf/cm^2)	${}_f\sigma_B$ (tf/cm^2)	Y.R. (tf/cm^2)	elong. (%)	
1	L1SR020	167	164	8.6	100	8.5	—	—	19.3	0.67	0.84	2.49	0.34	39.0	3.59	5.34	0.67	29.8	A
	L1SR040	249	249	6.0	101	8.8	—	—	41.2	1.42	0.84	2.49	0.34	39.0	3.59	5.34	0.67	29.8	A
	L1SR050	249	248	4.9	100	8.8	—	—	50.7	1.74	0.84	2.49	0.34	39.0	3.59	5.34	0.67	29.8	A
	L1SR060	249	249	4.0	100	8.8	—	—	62.2	2.14	0.84	2.49	0.34	39.0	3.59	5.34	0.67	29.8	A
2	L2SR033	300	300	9.0	101	11.9	—	—	33.1	1.16	0.87	2.56	0.34	60.1	3.91	5.34	0.73	42.6	B
	L2SR050	300	299	6.2	101	12.0	—	—	48.0	1.73	0.86	2.74	0.31	56.5	3.91	5.34	0.73	42.6	B
3	L3RR085	668	384	5.1	175	16.0	—	—	[85.8]	2.88	0.77	2.36	0.33	63.5	3.41	5.36	0.64	26.3	C
	L3RR185	668	384	5.1	175	16.0	6.0	50.0	[63.7]	2.14	0.77	2.36	0.33	63.5	3.41	5.36	0.64	26.3	C
	L3RR285-1	668	384	5.1	175	16.0	6.0	50.0	[48.3]	1.62	0.77	2.36	0.33	63.5	3.41	5.36	0.64	26.3	C
	L3RR385	668	384	5.1	175	16.0	6.0	50.0	[43.0]	1.44	0.77	2.36	0.33	63.5	3.41	5.36	0.64	26.3	C
	L3RR285-2	668	384	5.1	175	16.0	6.0	50.0	[48.3]	1.62	0.77	2.36	0.33	63.5	3.41	5.36	0.64	26.3	C
	L3RR285-3	668	384	5.1	175	16.0	6.0	50.0	[48.3]	1.62	0.77	2.36	0.33	63.5	3.41	5.36	0.64	26.3	C
4	L4RR385-1	668	400	6.0	175	16.0	6.0	50.0	[37.2]	1.28	0.84	2.49	0.34	34.7	3.65	5.31	0.69	30.2	B
	L4RR385-2	668	400	6.0	175	16.0	6.0	50.0	[37.2]	1.28	0.84	2.49	0.34	34.7	3.65	5.31	0.69	30.2	B
	L4RR385-3	668	400	6.0	175	16.0	6.0	50.0	[37.2]	1.28	0.84	2.49	0.34	34.7	4.06	4.57	0.89	32.0	B
	L4RR385-4	668	400	6.0	175	16.0	6.0	50.0	[37.2]	1.28	0.84	2.49	0.34	34.7	0.91	2.51	0.36	46.9	B



(a) Series 1,2: Non-reinforced square panel specimen (SQ-panel)

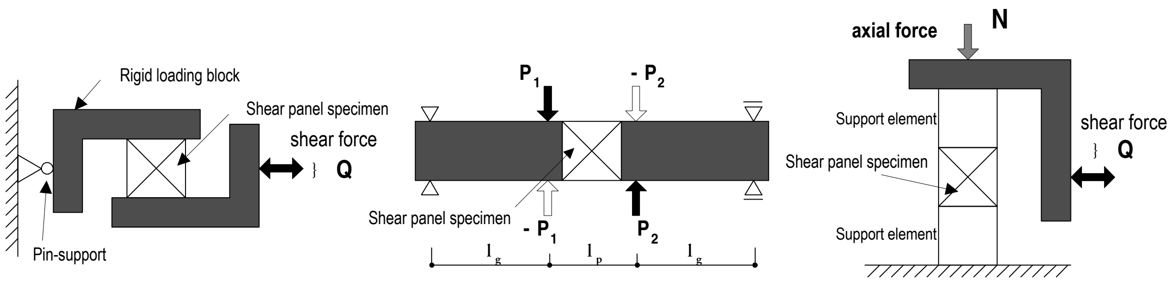
(b) Series 3: Reinforced rectangular panel specimen (RB-Panel)

(c) Series 4: Reinforced rectangular panel specimen. (RB-panel)

Figure 3: Configurations of shear panel specimen

1.1 Testing Apparatus

Three types of shear loading system were conducted as shown in Figure 4. All specimens except for one (L3RR285-2) were free from axial stress and axial constraint. In A and B-type loading system, shear panel specimens were set at rigid loading blocks using high-tension bolts. In C-type loading system, shear panel specimens had two supporting elastic elements and shear loads were applied at the ends of supporting element.



(a) A-type (b) B-type (c) C-type

Figure 4: Three types of testing apparatus

Test Results

As the test results, yielding stress, maximum strength, allowable deformation and failure mode of the specimens are listed in Table 2. Failure modes of the specimen are shown in Figure 5. Relationships of shear force versus drift angle on the specimens are shown in Figure 6, 7 and 8. Shear force is changed to average shearing stress (τ_w). Meanings of shearing stress used in this paper is explained in Table 2.

Table 2: Test Results of the Panel Damper Specimens

Test Series	Specimen	Test Results							
		Q_y (tf)	$\tau_{w,y}$ *1 (tf/cm^2)	Q_u (tf)	$\tau_{w,u}$ *1 (tf/cm^2)	${}_fQ_u$ *2 (tf)	τ_{pu} *3 (tf/cm^2)	${}_c\gamma_u$ (rad.)	Cracking Position
1	L1SR020	6.1	0.52	37.4	2.52	1.56	2.51	2.36	Welding
	L1SR040	5.9	0.53	28.8	1.85	1.11	1.84	0.69	Center
	L1SR050	5.4	0.57	19.4	1.53	1.13	1.50	0.28	Center
	L1SR060	4.3	0.60	15.5	1.53	1.11	1.45	0.23	Center
2	L2SR033	13.1	0.59	62.5	2.22	1.86	2.24	1.68	Welding
	L2SR050	7.5	0.58	34.7	1.78	1.89	1.75	0.56	Center
3	L3RR085	13.6	0.46	42.0	1.20	3.98	1.12	0.11	Center
	L3RR185	11.6	0.41	47.0	1.35	3.98	1.26	0.28	Center
	L3RR285-1	13.9	0.44	52.6	1.51	3.98	1.43	0.49	Center
	L3RR385	13.6	0.54	59.6	1.71	3.98	1.63	0.86	Center
	L3RR285-2	13.3	0.54	53.8	1.54	3.98	1.46	0.63	Center
	L3RR285-3	8.9	0.54	51.8	1.48	3.98	1.40	0.56	Center
4	L4RR385-1	12.5	0.59	78.1	1.90	1.02	1.92	0.85	Center
	L4RR385-2	14.0	0.55	77.7	1.89	4.55	1.83	0.95	Center
	L4RR385-3	14.0	0.55	78.8	1.92	4.09	1.87	0.80	Center
	L4RR385-4	13.0	0.61	77.1	1.88	4.09	1.82	1.15	Center

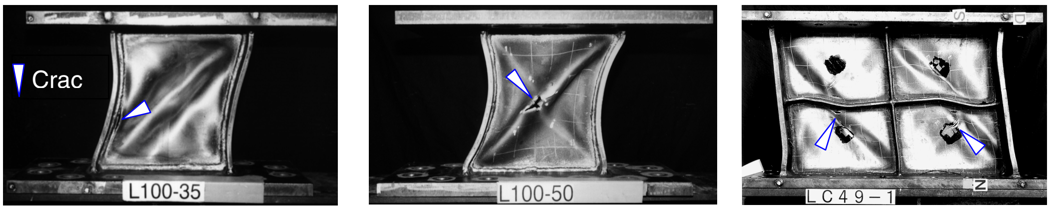
*1 ƒ Ñw,i = Qi/ ¡”_ŒŒ¡j
where t_w is thickness of the panel,
,Œ is distance between center of both flanges.

*2 Calcurated shear loading capacity of the flanges, assuming to have plastic hinges at both ends.

*3 ƒ Ñpu = (Qu-fQu)/ ¡”_Œo ¡j
where l_o is inside distance between both flanges.

Fracture mode

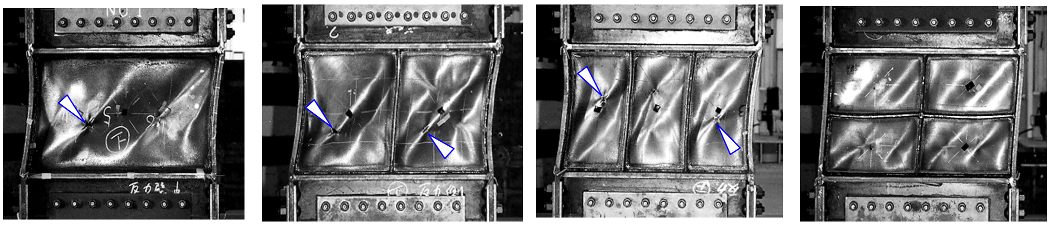
Typical fracture modes of the SQ-panels are shown in Figure 5(a) and (b). In case of smaller width-thickness ratio of 33, crack occurred at the side end of the panel in the vicinity of weld metal connecting the panel and the flange. On the other hand, in case of larger width-thickness ratio of 48, crack occurred at the center of the panel caused by cyclic and reversal plate-bending due to plate-buckling. In case of the RB-panels, crack could occur at the center of the panel or at the panel in the vicinity of welding metal connecting with the rib-plate as shown in Figure 5 (c) and (d) to (g). Furthermore, Figures 5 (d) to (g) show the effectiveness of the rib-plate reinforcing on the panels. Since restraint of rib-plates on the panel larger, deflection of the panel out of plane is smaller and energy dissipating capacity gets larger as a result.



(a) Specimen-L2SR033 ($d/t_w = 33$)

(b) Specimen-L2SR050 ($d/t_w = 48$)

(c) Specimen-L4RR385-1 { $(d/t_w)_{eq} = 37$ }



(d) Specimen-L3RR085 { $(d/t_w)_{eq} = 86$ }

(e) Specimen-L3RR185 { $(d/t_w)_{eq} = 64$ }

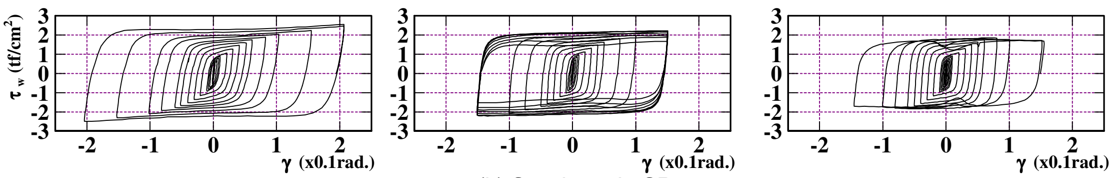
(f) Specimen-L3RR285-1 { $(d/t_w)_{eq} = 48$ }

(g) Specimen-L3RR385 { $(d/t_w)_{eq} = 43$ }

Figure 5: Typical fracture modes of the specimens

Shear force versus drift angle relation

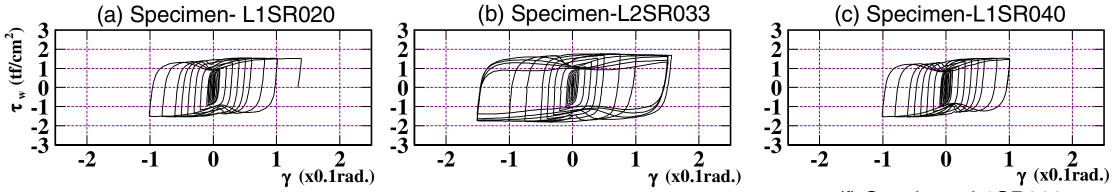
Figures 6 (a) to (f) show differences of dissipating energy among specimens with several width-thickness ratios in case of the SQ-panels. The smaller becomes the ratio of the specimen, the larger dissipating energy of the specimen. Especially, specimens with the ratio below 40 show very satisfactory energy dissipating performance. Figures 7 (a) to (f) show differences of dissipating energy among three details of reinforcing rib-plates on same non-reinforced panels. Figures 8 (a) and (b) show comparison of hysteretic loops between large constant deformation amplitude loading and earthquake-simulated deformation loading.



(a) Specimen- L1SR020

(b) Specimen-L2SR033

(c) Specimen-L1SR040

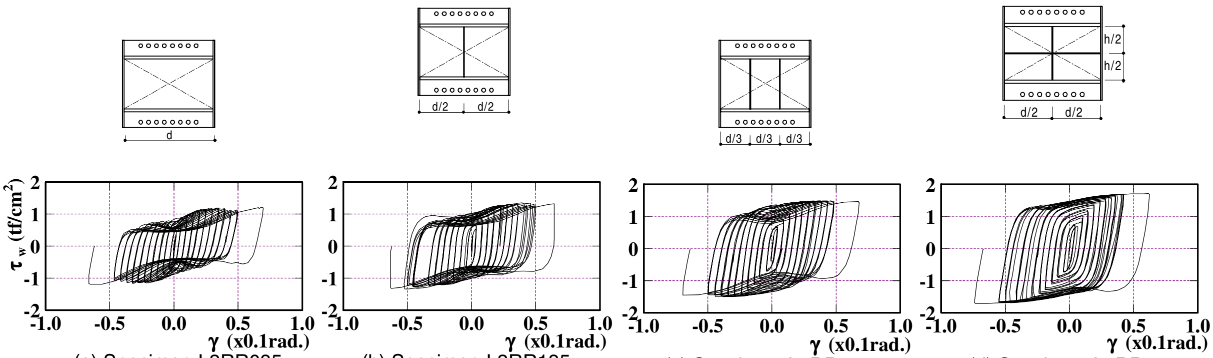


(d) Specimen-L1SR050

(e) Specimen-L2SR050

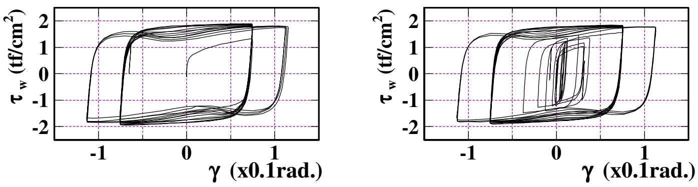
(f) Specimen-L1SR060

Figure 6: Comparison of shear force versus deformation relationships on SQ-panels (S-1, S-2)



(a) Specimen-L3RR085 (b) Specimen-L3RR185 (c) Specimen-L3RR285-1 (d) Specimen-L3RR385

Figure 7: Comparison of shear force versus deformation relationships on RB-panels (S-3)

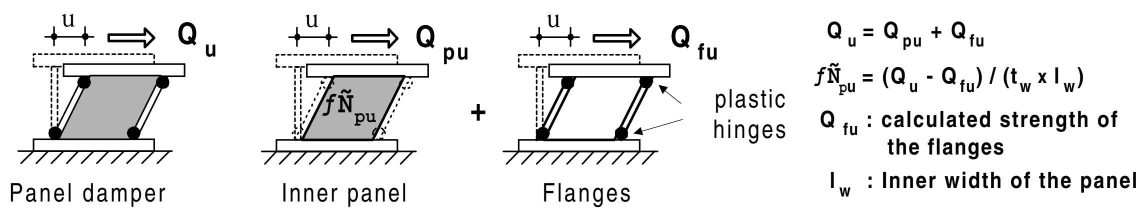


(a) Specimen-L4RR385-1 <Constant amplitude cyclic loading > (b) Specimen-L4RR385-2 < Earthquake-simulated loading >

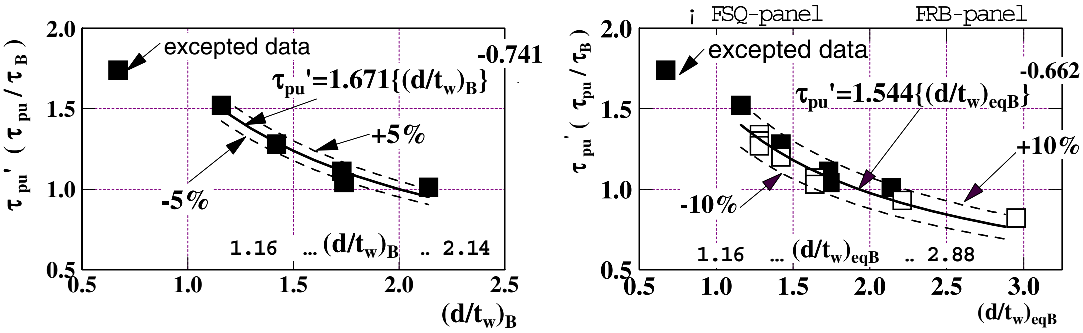
Figure 8: Comparison of shear force versus deformation relationships of the same two RB-panels under constant amplitude cyclic loading and earthquake-simulated loading (S-4)

MAXIMUM SHEAR STRESS OF THE PANEL

In case of the panels of smaller width-thickness ratio, the maximum shear strength obviously exceeded theoretical ultimate shear strength ($\tau_B=\sigma_B/\sqrt{3}$, σ_B :ultimate strength), as shown in Table 2. Meanwhile, maximum strength of the panel is necessary to design of the support members and its joints to the panels. Therefore, the precise estimation formula for the maximum strength should be obtained from the test results. Figure 9(a) shows share of the maximum strength of the panel damper on two types of element: the panel and the flanges. Figures 9(b), (c) show relationships between the maximum shear stresses and reduced width-thickness ratios of the specimens in case of the SQ-panels and all of the panels, respectively. They can be fitted to single curve. Solid lines are proposed formulae for the SQ-panels and all of the panels. The maximum strength of the panel damper can be estimated as shown in Figure 9(a). But, when the ratio is smaller than about 2.0, τ_{pu}/τ_B becomes much larger than 1.0, up to 1.7. The reason for this phenomena is not known yet.



(a) Share of maximum strength of panel damper

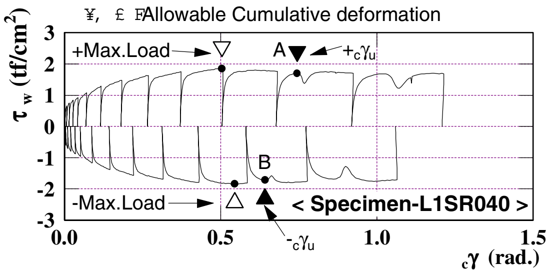


(b) Non-reinforced squre panel (SQ-panel) (c) All panel specimens

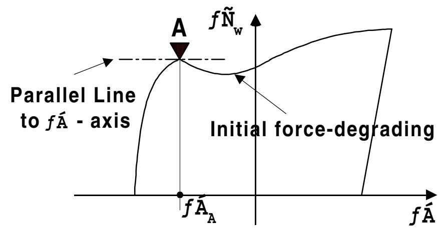
Figure 9: Estimation formulae for maximum shear stress of the panel

ALLOWABLE CUMULATIVE DEFORMATION

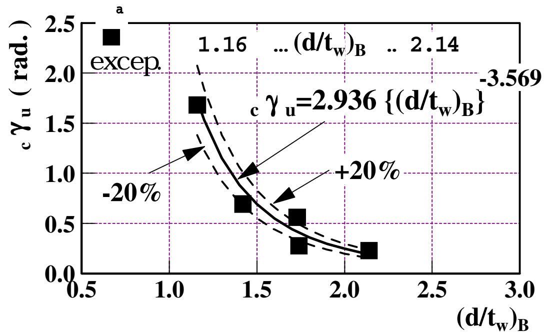
Allowable energy dissipating capacity of the panel damper is expressed by the cumulative shear drift angle. This amount is also necessary to evaluate safety of the panel dampers and building. In this study, allowable cumulative deformation is defined as shown in Figure 10(a) and (b). Figure 10(a) shows an example of shear force versus cumulative shear deformation relationship. In this figure, two abscissas of the point A and B present the allowable cumulative deformations. Figure 10(b) shows the schematic concept of the point A in Figure 10(a). Point A is a critical point where force degrading caused by distinct plastic shear-buckling begins. Figure 10(c), (d) show relationships between the allowable cumulative deformations and the reduced width-thickness ratios of the specimens in case of the SQ-panels and all of the panels, respectively. They also correlate closely with each other. And solid lines are proposed formulae for the SQ-panels and all of the panels.



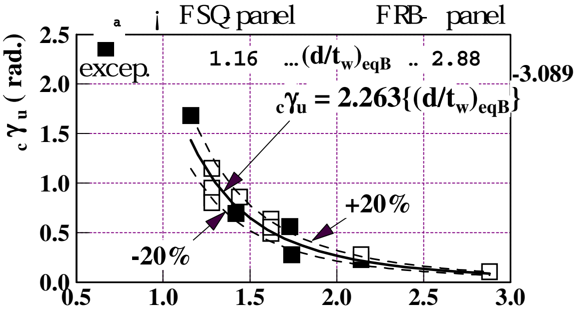
(a) Shear force versus cumulative deformation relation



(b) Allowable deformation point



(c) Non-reinforced squre panel (SQ-panel)



(d) All panel specimens

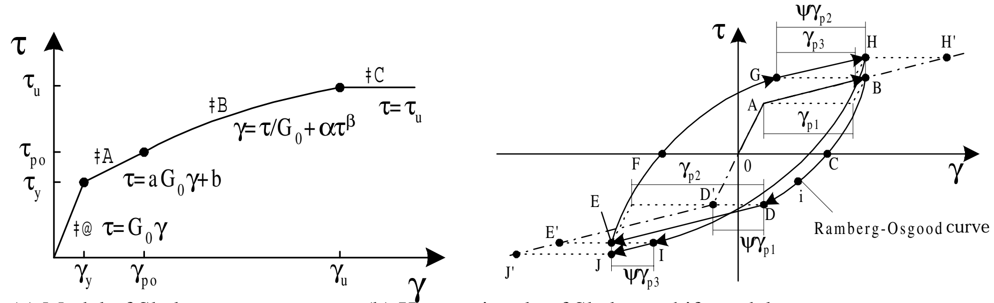
F... e cumulative shear-drift-angle of the panel damper

HYSTERESIS MODEL

"Skeleton Shift Model (SS-model) [Meng, Ohi and Takanashi, 1992]" has been proposed to express strain hardening or stress degrading phenomena of steel members. In this proposed model, an original form of the skeleton curve is composed of tri-linear curve and hysteresis loops are expressed by RO model [Ramberg and Osgood, 1943]. A special feature of the SS-model is that skeleton curve moves depending on experienced plastic strains. In the previous studies, it was shown that the skeleton curves of the panels using same steel had a common envelope regardless of the width-thickness ratio [Tanaka and Sasaki, 1998a]. The skeleton curves were obtained from hysteresis loops of the specimens, applying Kato and Akiyama's method [Kato and Akiyama, 1969]. A model of the skeleton curve and the hysteresis rule of the SS-model used in this study are shown in Figure 11(a) and (b). There are some variables to determine as for the type of steel used in the shear panel. For the ultra low-yield-strength steel, variables were extracted from the test results and are listed in Table 3. Comparisons between the experimental hysteresis and the estimated hysteresis by SS-model are shown in Figure 12. Skeleton Shift Model correlated well with the hysteresis loops of the specimens.

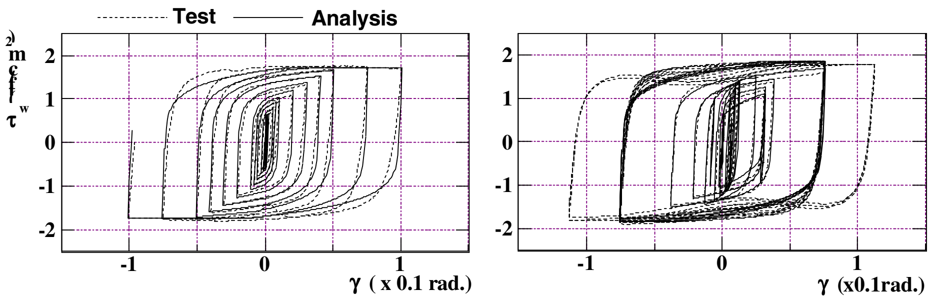
Table 3: Parameters applied to the SS-model for 100 N/mm^2 low-yield-strength steel

Parameter	a	b	ƒ ¿	ƒ À	ƒ Ñ$_y$(tf/cm^2)	ƒ Ñ$_{p0}$(tf/cm^2)	ƒ Á$_y$(rad)	ƒ Á$_{p0}$(rad)	ƒ µ	r
Selected value	0.0491	0.475	0.0200	3.98	0.50	0.67	0.00062	0.0048	0.80	10



(a) Model of Skeleton curve (b) Hysteresis rule of Skeleton shift model

Figure 11: Skeleton curve and hysteresis rule of the Skeleton Shift Model



(a) Specimen-L2SR050 (b) Specimen-L4RR385-2

Figure 12: Comparison on test and analysis (Skeleton Shift Model) in hysteresis loop

CONCLUSIONS

Hysteretic performances of the panel dampers with ultra low-yield-strength steel were estimated through proposed estimation method. Concluding remarks are obtained as follows:

(1) In case of width-thickness ratio of less than 40, the panel dampers exhibited very excellent hysteretic performances that are suitable for a hysteretic damper.

(2) Two proposed width-thickness ratios: reduced and equivalent width-thickness ratio, are very useful to estimate both hysteretic performances of the panel damper and effects of the panel-reinforcement on the performances. Empirical equations to estimate maximum strength and allowable deformation of the panel dampers are proposed for the seismic response control design purpose.

(3) Skeleton-Shift Model is suitable for simulating hysteresis of the panel damper. For actual application, a set of values of parameters are extracted for the panel dampers with the ultra low-yield-strength steel, regardless of the width-thickness ratios. Simulated hysteresis by the Skeleton-Shift Model agreed well with the test results.

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