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Paper No. 171. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

HYSTERETIC BEHAVIOR OF LOW YIELD STRENGTH STEEL PANEL SHEAR WALL -EXPERIMENTAL INVESTIGATION-

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ABSTRACT

Low-yield strength steel has a yield strength about one-third of that of conventional building structural steel (JIS SN400) and an excellent ductility, so that is an appropriate material for hysteretic dampers to control the seismic response of buildings. Low-yield strength steel plate shear walls (LYSWs) are hysteretic dampers of the shear panel type that are installed in structural frameworks. And the conventional design method of steel plate share walls is applied to LYSWs. This study experimentally investigated the hysteretic behavior of the LYSWs. The experimental work was conducted in two stages: shear loading experiment of the LYSWs and loading experiment of a three-story steel frame with the LYSWs. Experiment 1 verified the effectiveness of ribs installed to prevent the elastic buckling of the LYSWs and examined the basic hysteretic behavior of the LYSWs. Experiment 2 investigated the behavior of the LYSWs subjected to vertical load and cyclic shear load under conditions close to actual service conditions and verified the structural performance of the LYSWs as hysteretic dampers.

The experimental results obtained may be summarized as follows:
(1) LYSWs appropriately provided with ribs plastically buckled when subjected to the shear load, but exhibited stable hysteretic behavior with a high energy absorption capacity from low to high strain amplitudes.
(2) LYSWs installed in a steel frame subjected to the vertical load exhibited stable hysteretic behavior when subjected to the cyclic shear load.

From these results, it may be judged that the LYSWs are hysteretic dampers effective in reducing the seismic response of buildings.

KEYWORDS

Low yield strength steel; shear wall; hysteretic damper; plate buckling; plastic deformation capacity.

INTRODUCTION

Today, various systems are developed for controlling the vibration of buildings and reducing damage to the buildings in earthquakes. One of the earthquake vibration control systems is a damper that is installed in a structural framework to absorb earthquake energy. There are hysteretic dampers made of steel or lead, friction dampers and viscous dampers. The low-yield strength steel plate shear walls (LYSWs) addressed in this report are a kind of hysteretic damper that yields before such main structural members as columns and beams and absorbs the earthquake energy.

Low-yield strength steel has a yield strength about one-third of that of conventional building structural steel (JIS SN400) and an excellent ductility, so that it is an appropriate material of construction for dampers that control the seismic response of buildings. The LYSWs are large shear panel-type dampers installed in the frameworks of a building as shown in Fig. 1. And the conventional design method of steel plate share walls is applied to LYSWs (Takanashi et al., 1971; Okabe et al., 1977). Amount and arrangement of the LYSWs effective in reducing the seismic response of buildings are studied by some of the authors through their seismic response analysis of a high-rise building model (Torii et al., 1996).

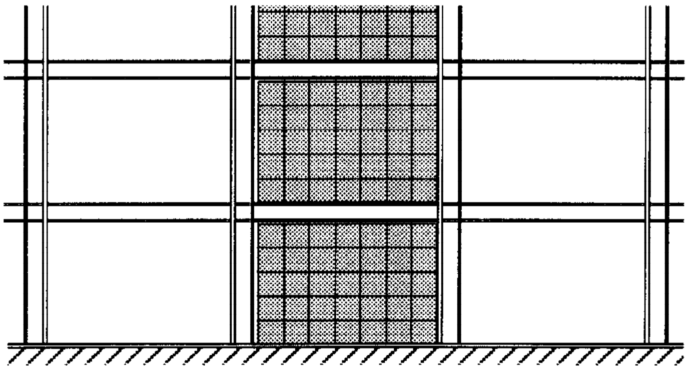

Fig. 1 Example of building frames with LYSWs

This report introduces the basic mechanical properties of the low-yield strength steel and presents the results of experiments conducted to clarify the elastic-plastic hysteretic behavior of the LYSWs. The experiments were conducted in the following two stages:

Experiment 1: When width-thickness ratio of the steel plate panel is large, the panel may buckle under a shear load and may be unable to exhibit stable hysteretic behavior. So, the LYSW panels must be ribbed to prevent elastic buckling. To verify the effectiveness of the ribs, the LYSW panels were experimented with under a shear load.

Experiment 2: To investigate the behavior of the LYSW panel installed as walls in the framework of a building, a three-story steel frame with the LYSWs was experimented with.

PROPERTIES OF LOW-YIELD POINT STEEL

The stress-strain curve of low-yield strength steel is compared with that of conventional mild steel JIS SN400 (nominal tensile strength of 400 MPa) in Fig. 2. The low-yield strength steel has no clear-cut yield shelf, has a yield strength about one-third of that of SN400, a low yield ratio, and an excellent ductility. The chemical composition and mechanical properties of the low-yield strength steel used in the experiments are given in Tables 1 and 2, respectively. The low-yield strength steel contains very small amounts of carbon and alloying elements and is close to pure iron in chemical composition. Its real elastic limit is about 70% of its 0.2% offset yield strength.

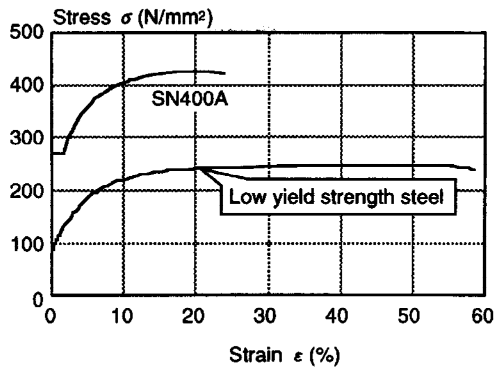

Fig. 2 Comparison of low-yield strength steel and mild steel in stress-strain curves

Table 1 Chemical composition of low-yield strength steel

Specimen	C	Si	Mn	P	S
Exp.1	0.0013	≦0.01	0.074	0.008	0.006
Exp.2	0.002	0.02	0.16	0.010	0.014

Table 2 Mechanical properties of low-yield strength steel

Specimen	t (mm)	Y.S. (MPa)	T.S. (MPa)	EL. (%)
Exp.1	4.41	87	246	59
Exp.2	4.60	103	243	64

BEHAVIOR OF LYSW SHEAR PANELS (EXPERIMENT 1)

Experimental Program

The specimens were low-yield strength steel plate panels (t=4.5mm) measuring 1700 by 1700 mm inside length and ribbed with the steel plates (t=6, JIS SN400). Each specimen was installed with high-strength bolts (JIS F10T, M22) in a high-stiffness loading frame connected with pin joints at the four corners as shown in Fig. 3. And it was statically and alternately loaded in tension and compression to develop cyclic shear stress. The experimental parameters were the rib arrangement, spacing, and height. Five specimens were prepared as shown in Fig. 4. Specimen 1 has ribs installed in a grid pattern on each side (this type is hereinafter referred to as the grid type). With the other specimens, the ribs are installed in one direction on each side in such a way that the ribs on one side run at right angles to those on the other side (this type is hereinafter referred to as the non-grid type). To investigate the effects of differences in the buckling behavior, the shear yield strength and elastic buckling strength of the panels were predicted, and the rib spacing and height of each specimen were determined accordingly. The experimental parameters are summarized in Table 3.

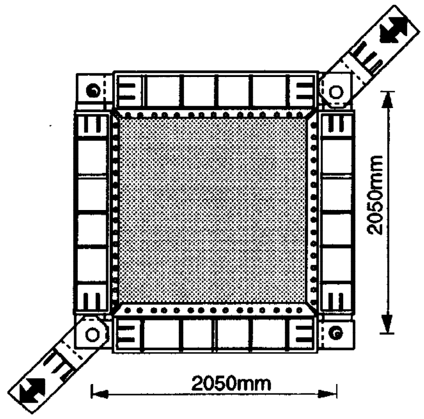

Fig. 3 Loading Frame

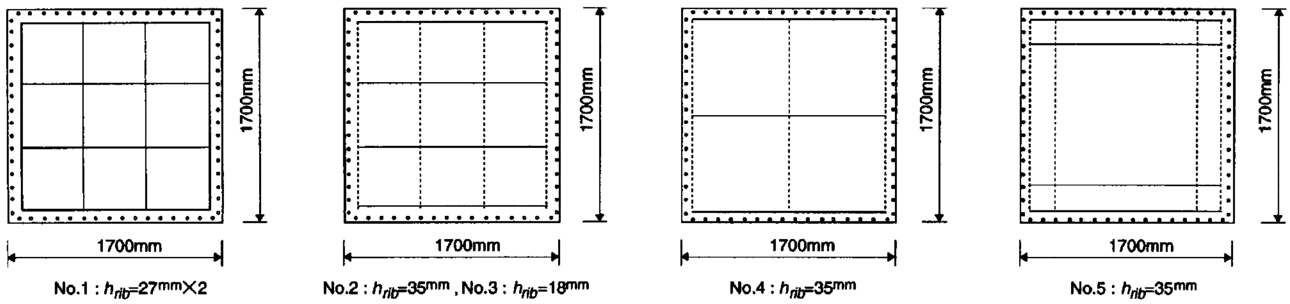

Fig. 4 Rib arrangement of specimens

Table 3 Predicted shear strength of specimens

Specimen	Rib arrangement	d/t	I (cm^4)	τ_y (MPa)	τ_{cr1} (MPa)	τ_{cr2} (MPa)	$\tau_{cr2'}$ (MPa)
No.1	GIRD	110	10	50	123	144	225
No.2	NON GRID	110	10	50	123	144	225
No.3	NON GRID	110	2	50	42	144	225
No.4	NON GRID	170	10	50	69	60	93
No.5	NON GRID	250	10	50	31	27	43

The symbol τ_y is the shear yield stress of the low-yield strength steel and τ_y equals to $\sigma_y/\sqrt{3}$ where σ_y is the 0.2% offset yield strength of the panel.

The symbol τ_{cr1} denotes the shear stress at which the entire panel undergoes elastic buckling (entire buckling). Its approximate value was calculated by the energy method by assuming that the panel was supported simply. That is, the buckling deformation ω of the panel was defined by the Fourier series of Eq. (1). The sum of the strain energy of the panel and ribs and of the potential energy of the shear force that is an external force was denoted by Π. The buckling stress of the panel was calculated by Eq. (2).

$$\omega = \sum_{m=1}^{M}\sum_{n=1}^{N} a_{mn} \sin\frac{m\pi x}{\mathrm{L}} \sin\frac{n\pi y}{\mathrm{H}} \tag{1}$$

$$\partial\Pi/\partial a_{mn} = 0 \tag{2}$$

where L and H are the side lengths of the panel.
The symbols τ_{cr2} and τ_{cr2}' denote the shear stress at which the panel enclosed with the ribs undergoes elastic buckling (local buckling) when the panel is assumed to be supported fixed and simply, respectively. They were calculated by Eq. (3).

$$\tau_{cr} = k\frac{\pi^2 E}{12(1-\nu)}\left(\frac{t}{d}\right)^2 \tag{3}$$

where k is 9.34 for τ_{cr2} (panel supported simply) and 14.58 for τ_{cr2}' (panel supported fixed); t is the panel thickness; and d is the rib spacing.

I is the geometrical moment of inertia of the ribs and is a parameter that affects the entire buckling strength of the panel. In experiment 1, the value of I was changed by changing the rib height of each specimen. The value of I was calculated for the grid type specimen by assuming a rectangular section combining the ribs on both sides and for the non-grid type specimens 2 to 5 by containing for the panel. As shown in Table 3, specimens 1 and 2 were designed for the shear yield to occur first, specimen 3 was designed for the entire buckling to occur first, specimen 4 was designed for the shear yield strength and local buckling load to become approximately equal to each other, and specimen 5 was designed for the local buckling to occur first.

Experimental Results

The hysteresis curves of specimens 1 to 5 are shown in Fig. 5. The load and deformation of each specimen are converted into the average shear stress τ and the shear strain γ, respectively. The specimens exhibited stable hysteresis loops of high energy absorption capacity and did not appreciably differ in the cumulative loop area.

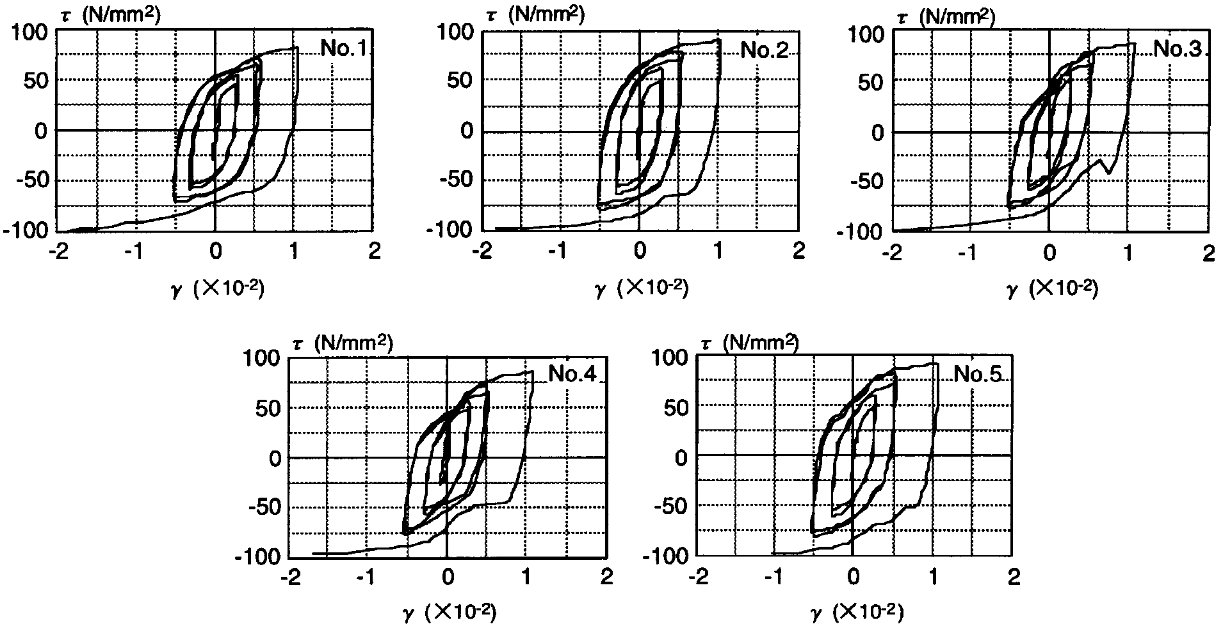

Fig. 5 Share stress and share strain relationship of specimens

The plate buckling of the specimens is shown in Table 4. The plate buckling load is the average shear stress τ of the specimen when out-of-plane deflection is visually observed in the panel. The elastic limit of the low-yield strength steel used in the specimens is about 60 MPa and is about 70% of the 0.2% offset yield strength of 87 MPa. Given this material property, the specimens were judged to have substantially undergone plastic buckling, except for the specimen 5 that local buckling first.

Table 4 Summary of buckling deflection

Specimen	Initial load of buckling deflection		Deformation mode at end of loading	
	Cycle	τ (MPa)	Local buckling	Entire buckling
No.1	-2 (Compression side)	47	○	—
No.2		52	○	○
No.3		37	—	○
No.4	+2 (Tension side)	39	○	○
No.5		41	○	○

With the grid type specimen 1, the ribs effectively restrained the panel and maintained the local buckling mode until the end of the experiment when $\gamma = 1/15$ was imposed. The non-grid type specimens 2 to 5 ultimately developed the out-of-plane deformation of the ribs and exhibited the entire buckling mode. The entire buckling was clearly exhibited only by the specimen 3 of reduced rib stiffness, however. Specimens 2, 4, and 5 gradually changed from the local buckling mode to the entire buckling mode in the final cycle where the cumulative deformation was large. With the non-grid type specimens, the neutral axis of the ribs is considered to have moved to reduce stiffness after the yielding of the panel. If the stiffness of the ribs is evaluated without considering the panel, therefore, the non-grid type is believed to have the same panel restraint effect as the grid type. When specimens 2, 4, and 5 with panel width-to-thickness ratios (*d/t*) of 110, 170, and 250, respectively, were compared in terms of the effect of rib spacing on buckling load, specimen 2 was higher in buckling load than specimens 4 and 5. There were found no clear differences in the buckling load between specimens 4 and 5.

The plastically buckled specimens exhibited slip-like hysteretic behavior on the inversion of buckling deflection in large-amplitude cycles. Specimen 3 exhibited reduction in strength in the final cycle. After the slip-like hysteretic behavior, specimen 3 immediately formed a tension field and exhibited excellent plastic deformation performance as the buckling strength increased. When the hysteresis loops of specimens 2 and 4 are compared, specimen 4 with a larger *d/t* shows the slip-like characteristic relatively early after the imposition of the 1/200 shear strain.

CYCLIC HORIZONTAL LOADING OF 3-STORY STEEL FRAME WITH LYSWS (EXPERIMENT 2)

Experimental Program

To investigate the behavior of an intermediate portion of a high-rise building constructed with the LYSWs, a three-story steel frame was fabricated as a 1/2-scale model of the portion, and attention was focused on the story 2 of the three-story steel frame. The overall shape and member size of the three-story steel frame are shown in Fig. 6. The columns and beams are designed to maintain elasticity to a drift angle of 1/200 and to form no plastic hinges at a drift angle of 1/100. The LYSWs were 4.5 mm in plate thickness, and their *d/t* ratio was put at 110 so that their hysteresis loop would not develop the slip-like hysteretic behavior up to a drift angle of 1/200 according to the results of experiment 1. The ribs were made of JIS SS400 and arranged non-grid type, and their height was doubled to 70 mm to ensure the out-of-plane stiffness of the panel according to the results of experiment 1. The ribs were discontinuously fillet welded to the panels, and the panels were jointed frictionally to the frame by high-strength bolts (JIS F10T, M20).

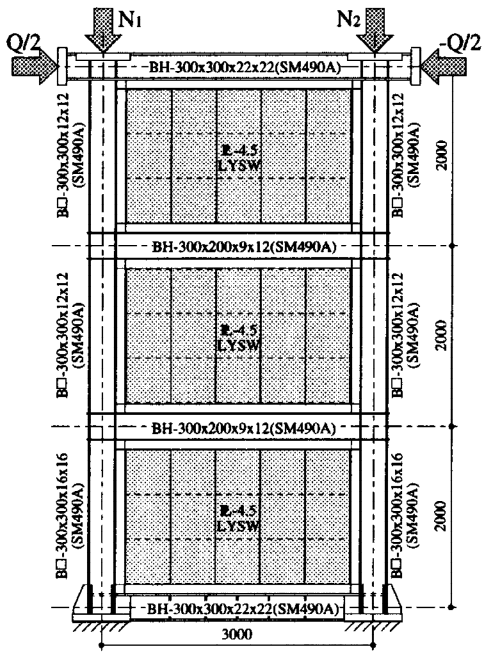

Fig. 6 Three-story frame with LYSWs

The loads were applied to the frame at the four positions indicated by the arrows in Fig. 6. The horizontal load Q was equally applied to the heads of the right and left columns in a completely reversed, static, and alternating pattern. The specimen was loaded cyclically according to the history shown Fig. 7 in which the ordinate indicates the drift angle of story 2. The history was composed of stages with three seismic intensity levels. The vertical loads N_1 and N_2 applied to the heads of the right and left columns were 1960 kN in total and were individually controlled to $N_1 = 980 + 2/5Q$ and $N_2 = 980 - 2/5Q$, in order to investigate the effect of the additional axial load resulting from the horizontal load Q at the bases of the right and left columns.

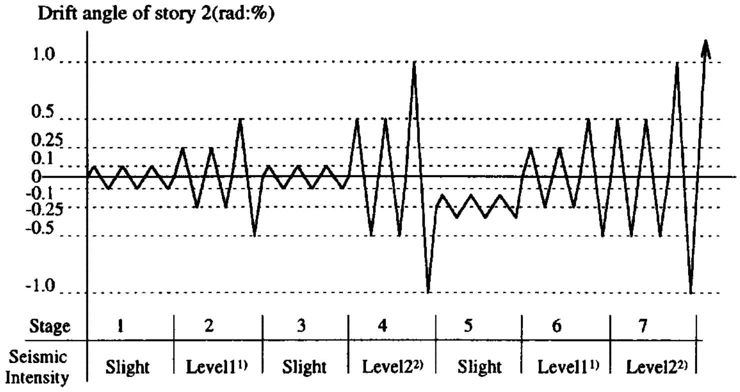

Fig. 7 Loading pattern of three-story frame with LYSWs.
(1) Small and medium earthquake motions to be encountered several times during service life of building; 2) Large earthquake motions rarely encountered during service life of building)

Experimental Results

The relations between the shear load Q and drift angle R of story 2 in loading stages 1 to 4 and 5 to 7 are shown in Fig. 8. The Q-R relations of story 1 are shown in Fig. 9 for reference in observing the behavior of the frame under a high variable vertical load. Visually, the panels plastically buckle at the first drift angle of 1/400 in stage 2, but this effect is not evident in the hysteresis loops. Up to stage 4, the hysteresis curves gradually increase in strength with strain hardening and draw stable spindle-shaped loops. After stage 5, the hysteresis loops do not become completely spindle-shaped under the influence of the large buckling deflection caused at the 1/100 drift angle in stage 4. Story 1 has higher frame stiffness than story 2 and thus a smaller drift angle than story 2, but exhibits approximately the same hysteresis loops as story 2. The ribs are not deflected out of the panel plane in the final cycle and are confirmed to have sufficient stiffness in the non-grid type arrangement.

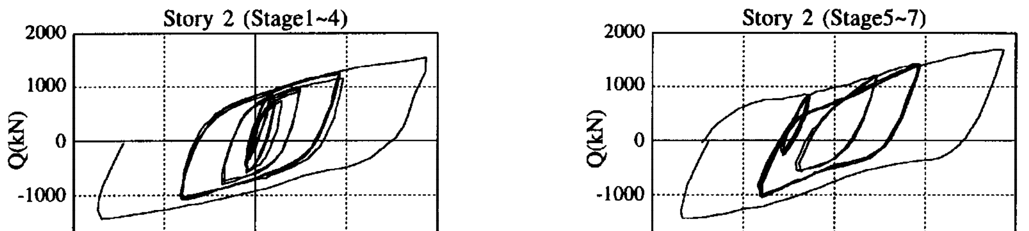

Fig. 8 Q-R relationship of story 2

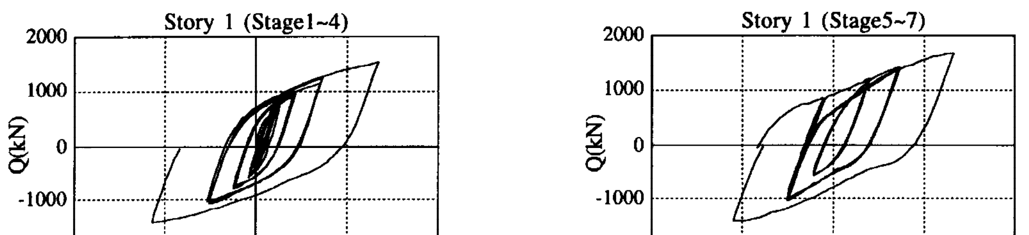

Fig. 9 Q-R relationship of story 1

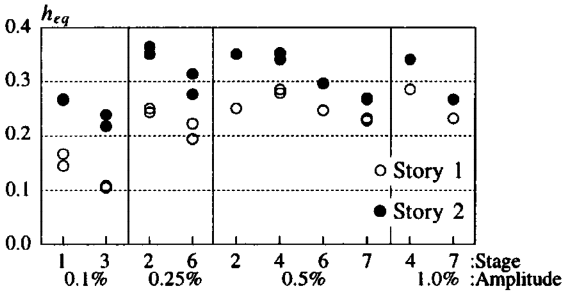

Fig. 10 Equivalent viscous damping ratio h_{eq}

The equivalent viscous damping ratio h_{eq} of the three-story steel frame is shown by cycle and stage in Fig. 10. When h_{eq} is compared at the same amplitude, h_{eq} decrease in the stages where the slip-like behavior appeared in the hysteresis loops. When the drift angle of story 2 is 1/400 or more, h_{eq} is above 0.3 in stage 2 and stage 4 and slightly below 0.3 in stage 6 and stage 7. These values mean that the LYSWs have enough damping performance. Story 1 is higher in frame stiffness than story 2 and is thus smaller in h_{eq}, but is similar to story 2 in the relative change in h_{eq} under cyclic loading.

These results show that the LYSWs have sufficient damping performance when cyclically loaded in a rigid frame subjected to a high variable vertical load.

CONCLUSIONS

Experimentation of the LYSWs produced the following findings:

Low-yield strength steel plate shear panels ribbed against elastic buckling plasticize earlier than shear panels made of mild steel like JIS SN400, form a diagonal tension field immediately after plastic buckling, and exhibit stable hysteretic behavior.

The energy absorption capacity of the LYSWs does not greatly vary with how they are reinforced with ribs in this study. Each type of the LYSWs has an excellent plastic deformation capacity. When the ribs are arranged in the non-grid type, their stiffness must be calculated by considering the movement of the neutral axis after the yield of the panel.

When the LYSWs are installed in a rigid frame and horizontally displaced under a high variable vertical load, they have damping performance similar to that of single shear panels and exhibit stable spindle-shaped hysteresis loops to a large-deformation region.

According to the above findings, the LYSWs that plasticize early and retain a stable energy absorption capacity to a large deformation region under high variable vertical loading are believed to be hysteretic dampers effective in reducing the seismic response of buildings where they are installed.

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