CYCLIC TEST FOR FRAMED STEEL PLATE WALLS WITH VARIOUS INFILL PLATE DETAILS

I. Choi[1] and H. Park[2]

[1] *Graduate Student, Dept. of Architecture., Seoul National University, San 56-1, Shinlim-Dong, Kwanak-Gu, Seoul 151-744, South Korea.*
[2] *Professor, Dept. of Architecture., Seoul National University, San 56-1, Shinlim-Dong, Kwanak-Gu, Seoul 151-744, South Korea.*
Email:inrak78@snu.ac.kr, parkhg@snu.ac.kr

ABSTRACT :

The framed steel plate wall has been used in U.S, Canada, and Japan, primarily as an earthquake resisting system. However, in order to design the steel plate wall system effectively, it was needed to verify the structural capacity of steel plate walls with various infill plate details that can be applied in practical affairs. In the present study, experimental studies were performed to investigate the variations of the structural capacity of steel plate walls with various infill plate details. The major parameters for the test specimens were the location and length of weld-connection, coupling wall, and the connection details between the moment frame and infill plates, such as, weld- and bolt-connections. A concentrically braced frame (CBF) and a moment-resisting frame (MRF) were also tested for comparison. Regardless of the infill plate details, the steel plate wall specimens exhibited excellent initial stiffness, strength, and energy dissipation capacity. However, the wall with bolt-connected infill plates showed slightly low deformation capacity. This result indicates that for architectural reasons and cost-saving, various wall details can be used in practice without a significant decrease in the structural capacity of the steel plate walls.

KEYWORDS: Steel Plate Shear Walls, Cyclic Tests, Connections, Energy Dissipation

1. INTRODUCTION

The framed steel plate wall that consists of frame members and infill plates is a structurally effective and economical lateral load resisting system. In early application of the steel plate wall, thick plates or plates with stiffeners were used to prevent the local buckling of the infill plates. Recently, for the purpose of cost-saving and enhanced constructability, the framed steel plate walls with unstiffened thin infill plates have been studied to utilize the diagonal tension-field action of the infill plate. Using the thin infill plates, the framed steel plate wall has advantages over the conventional reinforced concrete wall in several aspects. The overall weight of a structure can be significantly reduced by using the steel plate wall. Therefore, the size of the foundation and the earthquake load can be reduced. Furthermore, structures can be constructed faster with the use of the steel plate wall. Also, the usable floor area can be increased.

The framed steel plate walls without plate stiffeners have been studied by many researchers (Caccese et al. 1993; Driver et al. 1998; Elgaaly 1998; Lubell et al. 2000; Behbahanifard 2003; Park et al. 2007). The previous studies mostly focused on the structural capacity of the steel plate walls that had rigidly connected infill plates to the boundary frame. However, to expand the applicability of the steel plate wall, various details that can be frequently used in practice should be studied. Further, the plate aspect ratios (span-to-story height ratio of an infill plate) of the previous test specimens were relatively small, ranging from 1.2 to 1.8. However, the plate aspect ratio is generally greater than 2.0. Therefore, the steel plate walls having their plate aspect ratio with such practical ranges need to be tested.

To assure the force-transfer between the infill plate and the frame members, weld connection is generally required at all edges of the infill plate. However, the use of the full weld connection requires considerable labor-cost and may delay the construction of the steel plate wall system. Therefore, for fast construction and cost-saving, bolt connection may be required though it is not used for all steel plates in a building.

As an economical connection method, partially connected infill plates can be also considered. The steel plate

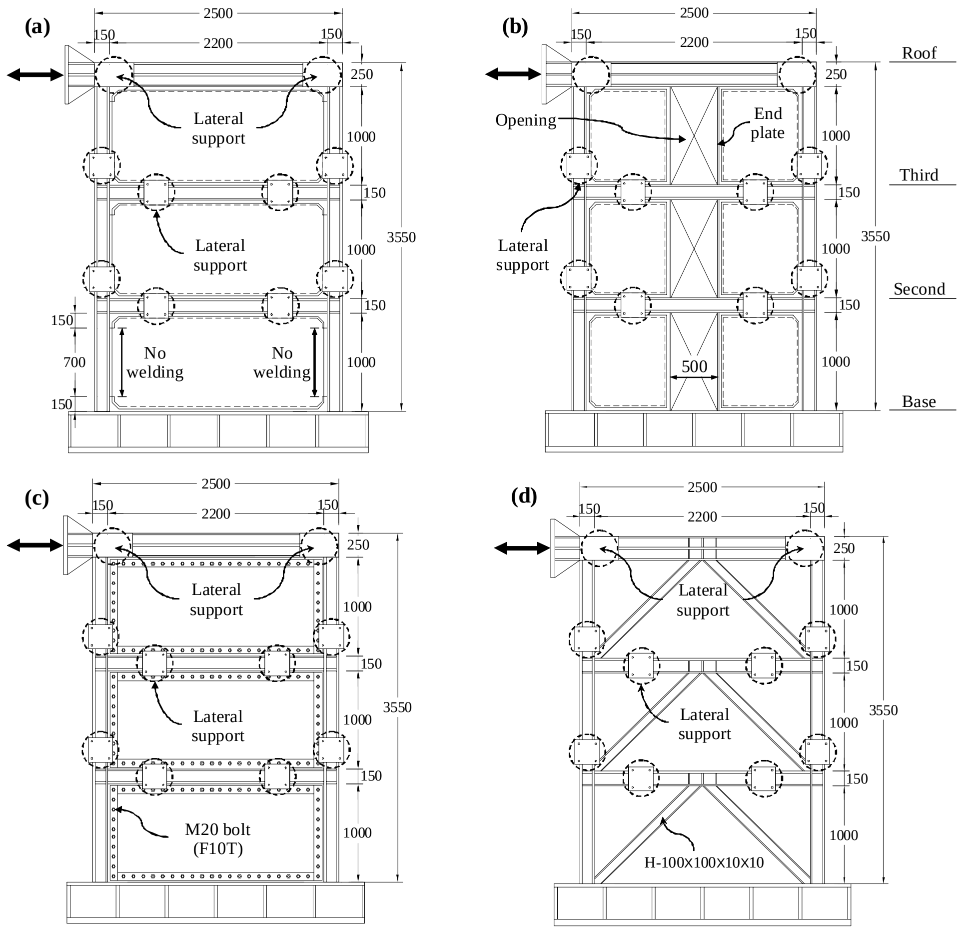

Figure 1. Dimensions of test specimens (mm): (a) FSPW4 (partially connected infill plate) (b) FSPW5 (coupled wall) (c) BSPW2 (bolt-connection) (d) CBF (concentrically braced frame)

walls with the partial connection details were analytically studied by Xue and Lu (1994). However, to confirm the result, experimental evidence is required. In typical buildings, frequently, windows and doors are placed in the infill plates. In case a large opening is required in the mid-span of the infill plate, the wall may be designed as a coupled wall. In such a case, the effects of the opening on the structural capacity of the coupled wall must be studied and economical methods for stiffening the wall openings must be developed.
In the present study, three-story steel plate walls were tested to investigate the structural capacity of steel plate walls with thin infill plates: 1) practical ranges of plate aspect ratios, 2) various infill plate details, such as, a wall with partially connected infill plats, a coupled wall with a wall opening in the mid-span, and walls with bolt-connected infill plates. Since the test specimens are reduced small-scale models of the prototype walls, it is difficult to quantitatively evaluate the structural performance of the full-scale steel plate walls from the test results. Therefore, for comparison with the test results of the reduced-scale steel plate walls, a concentrically braced frame (CBF) and a moment-resisting frame (MRF) were tested.

2. SPECIMENS AND TEST SETUP

The test specimens were one-third models of a three-story prototype wall with thin infill plates. The configuration of the test specimens is presented Figure 1. The steel plate wall specimens were designed with the ductile details—compact column sections, full-penetration weld connection at the beam-to-column joints, column strength and column stiffness for resisting the tension-field action of the infill plates.

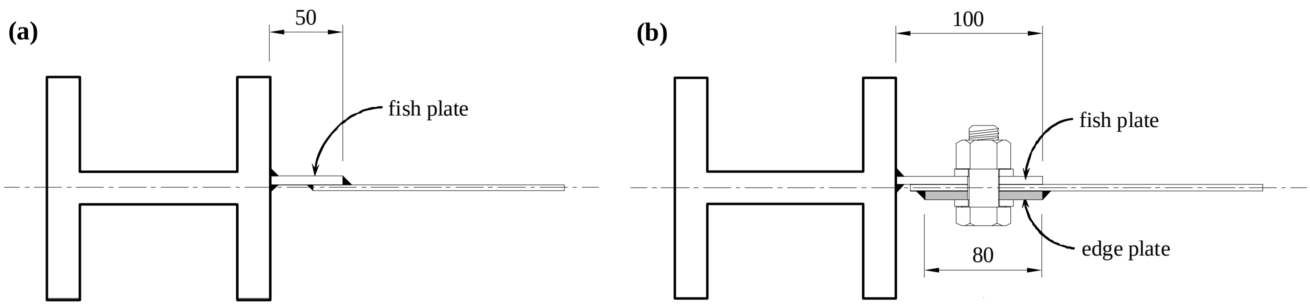

Figure 2. Detail of fish plate connecting infill plates and frame members: (a) weld-connection (b) bolt-connection

In typical buildings, unlike the specimens tested in previous studies, the span length is more than two times greater than the story height, and the depth of the column section is much less than the span length (the ratio of the span to column depth l/d_c = 20 ~ 30). Therefore, the reduced scale models used in this test were designed to have infill plates with a relatively large aspect ratio and columns with a small depth. The thickness of the infill plates in all the wall specimens was 4 mm (Korean Standard SS400, F_y = 240 MPa). The infill plate aspect ratio (l_p / h_p) was 2.2 (l_p = 2200 mm, h_p = 1000 mm). where l_p and h_p = net length and height of a infill plate, respectively.

The frame members were built-up sections made of SM490 steel (Korean Standard, F_y = 330 MPa). All columns were H-150×150×22×22 mm [built-up wide flange section, H-overall depth (d_c) × flange width (b_f) × web thickness (t_w) × flange thickness (t_f)]. The beams at the second and third stories were H-150×100×12×20. The top beam that was connected to the actuator was H-250×150×12×20. The flange and web elements of all beams and columns satisfied the requirements for the compact section according to the AISC Seismic Provisions (AISC, 2005). The columns were designed to have sufficient flexural and shear strengths for resisting the tension-field action of the infill plate.

Fully restrained moment connections were used at all beam-to-column joints. The beam flanges were rigidly connected to the columns by full penetration groove welding. The beam webs were connected to the column flange by two-side fillet welding. In the weld-connected specimens FSPW2, FSPW4, and FSPW5, using 50 mm wide and 6 mm thick fish plate [Figure 2(a)]. The fish plate was welded to the beams and columns by two-side fillet welding. In FSPW2, all of the four infill plate edges were weld-connected to the frame members. To reduce the length of the weld connection, the infill plates of FSPW4 were weld-connected only to the top and bottom beams [Figure 1(b)]. To prevent premature failure at the connection, the vertical edge of the infill plate was partially welded to the column flange by 150mm from the top and bottom flanges of the beams. The coupled wall FSPW5 consisted of two separated walls connected by a coupling beam [Figure 1(b)]. For cost-saving, 100 mm wide and 12 mm thick end plates were welded to the edge of the plate at the opening, instead of using additional columns.

In BSPW1, the infill plates were bolt-connected to the frame members [Figure 1(c)]. Figure 2(b) shows the detail of the bolt connection. To reduce the number of bolts for fast fabrication, F10T M20 (Korean Standard, Bolt diameter = 20 mm), with a relatively large diameter compared to the size of the specimens was used, and the bolt connection was designed as the bearing type connection that had greater strength than that of the friction type connection. However, pretension was applied to the bolts to satisfy the serviceability condition under wind load by preventing premature slip of the bolt connection. To satisfy the minimum edge distance requirement of the bolt connection and to increase bearing strength of the bolt, 100 mm wide and 6 mm thick fish plates were used. For the same reason, 80 mm wide and 6 mm thick edge plate was welded to the infill plate [Figure 2(b)]. However, in actual infill plates, since the plate thickness is usually more than 6mm, the edge plate is expected to be unnecessary. The bolts were designed to resist the plastic tension-field action of the infill plates. However, to minimize bolt slip, in BSPW2, 100 mm bolt spacings (60 bolts per infill plate) were used.

The CBF and MRF consisted of beams and columns with the same sizes as those used for FSPW2. For

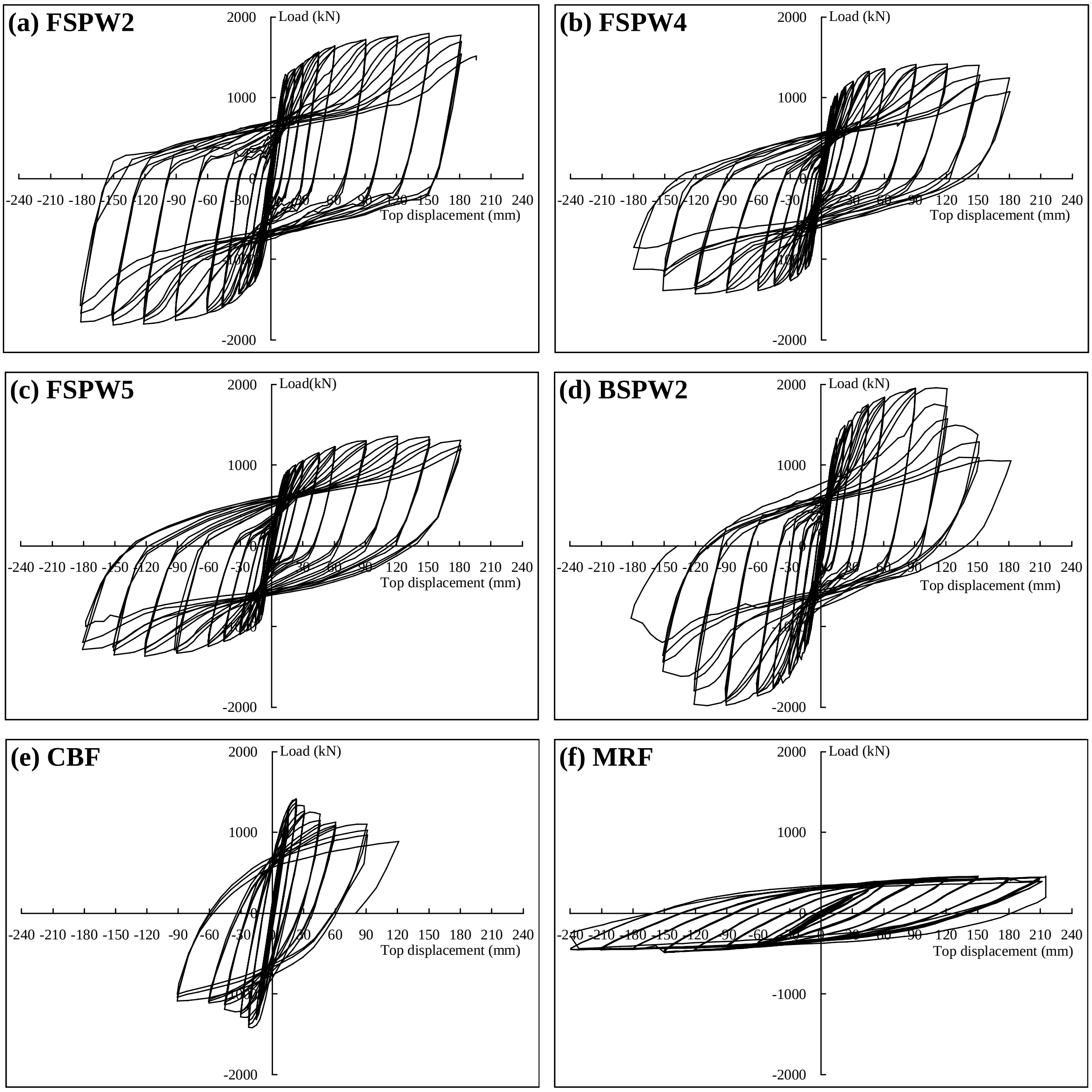

Figure 3. Load-top displacement relationships of test specimens: (a) FSPW2 (b) FSPW4 (c) FSPW5 (d) BSPW2 (e) CBF (f) MRF

comparison with the test results of the steel plate walls, the braces (H-100×100×10×10) in the CBF were designed to have the same steel weight as that used for the infill plate of FSPW2 [Figure 1(d)]. The brace members satisfied the requirements for the slenderness and compactness of special CBFs, as specified by the AISC Seismic Provisions (AISC 2005).

Before testing the steel plate walls, a pushover analysis was performed for the specimens by using ABAQUS (HKS 2003). In the finite element analysis, material nonlinearity and geometrical nonlinearity were considered. From the analysis results, the yield displacement δ_y at the top of the specimens was estimated to be 15 mm on average. Based on the predicted yield displacement δ_y (15 mm), the target displacements for the cyclic loading were chosen as $\pm 0.2\delta_y$, $0.4\delta_y$, $0.6\delta_y$, $0.8\delta_y$, $1.0\delta_y$, $1.5\delta_y$, $2\delta_y$, $3\delta_y$, $4\delta_y$, $6\delta_y$, $8\delta_y$, $10\delta_y$, and $12\delta_y$. Cyclic loadings were repeated three times at each target displacement.

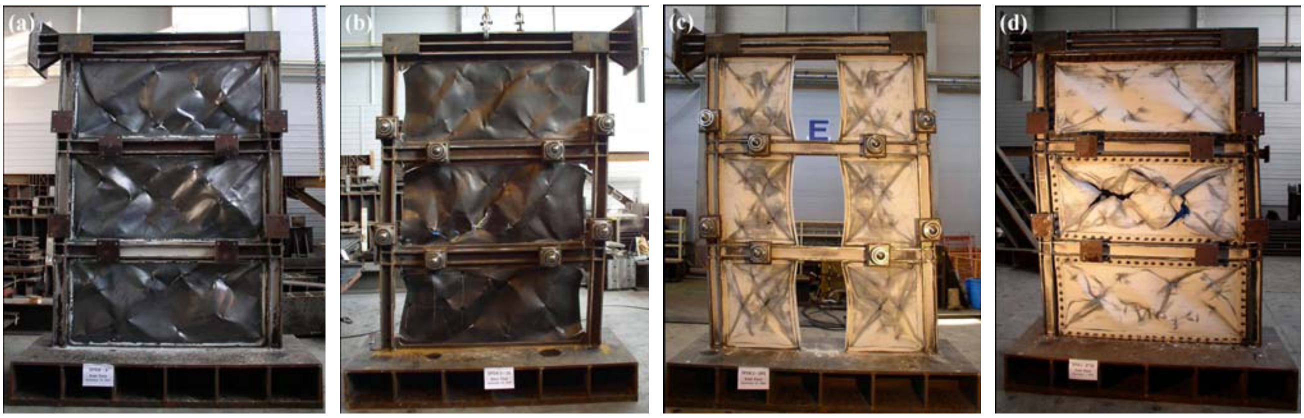

Figure 4. Deformed shapes of test specimens at the end of testing: (a) FSPW2 (b) FSPW4 (c) FSPW5 (d) BSPW2

3. SPECIMENS AND TEST SETUP

3.1. Load–Displacement Relationship

Figure 3 shows the load-top displacement relationships of the test specimens. As shown in Figure 3, the steel plate wall specimens showed a large initial stiffness and load-carrying capacity. FSPW2, which had a relatively large aspect ratio (l_p / h_p = 2.2), sufficient column strength for resisting the tension-field action, and compact column sections, exhibited the greatest load-carrying capacity and deformation capacity [Figure 3(a)].

The initial stiffness and deformation capacity of FSPW4 with partially connected infill plates were similar to those of FSPW2 [Figure 3(b)]. However, the load carrying capacity of FSPW4 was slightly less than that of FSPW2. This result indicates that for cost-saving and fast construction, the partial weld connection of the infill plates can be used without a great loss of the structural capacity of the steel plate walls. The coupled wall specimen FSPW5 exhibited less initial stiffness and strength than the solid wall specimen FSPW2 did [Figure 3(c)]. However, FSPW5 exhibited excellent deformation capacity that was equivalent to that of FSPW2. This result indicates that the coupled wall reinforced only with the end plates at the wall opening can exhibit an excellent structural capacity.

Figure 3(d) shows the test results of the BSPW2 with bolt-connected infill plates. The initial stiffness of BSPW2 was similar to that of FPSW2 with weld-connected infill plates. The maximum strength was slightly greater than that of FSPW2. This is because 100 mm wide fish plates and 80 mm wide edge plates used for the bolt-connections increase the strength of the infill plates. However, the BSPW2 specimen exhibited less deformation capacity than FSPW2 did.

Figures 3(e) and (f) show the test results of the CBF and MRF. The CBF showed considerably low deformation capacity because of the early buckling of the braces in compression. The MRF exhibited large deformation capacity. However, more importantly, since its initial stiffness and strength were relatively low, the ductility of the MRF ($\delta_{\max}$ / δ_y = 3.58) was significantly less than that of FSPW2 ($\delta_{\max}$ / δ_y = 11.70).

3.2. Failure Mechanism

In FSPW2, during early loading, tension-field action developed due to the local buckling of the infill plate. Subsequently, the shear yielding of the infill plates propagated along the wall height [Figure 4(a)]. After the yielding of all the infill plates, plastic hinges were developed at the beam ends and the first story column base by the moment frame action. FSPW2 exhibited excellent ductility without any sudden decrease in strength up to 5.3% drift (top displacement = 180 mm). Unlike other specimens, no fracture occurred in the boundary frame members. Ultimately, the load-carrying capacity of FSPW2 was decreased by the severe tearing of the infill plates. The tearing of the infill plates occurred at their centers where the two orthogonal tension-fields repeatedly intersected each other under reversed cyclic loading.

In specimen FSPW4, at 1.8% drift ratio (top displacement = 60 mm), fracture initiated in the weld connection at

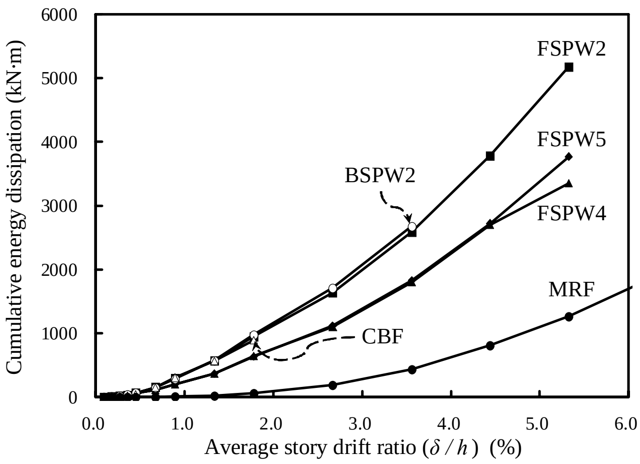

Figure 5. Cumulative energy dissipation of test specimens

the corner of the second story infill plate where a part of the infill plate edge was welded to the column flange. At 4.4% drift ratio (top displacement = 150 mm), the fracture propagated to the beam flange as the out-of-plane displacement of the infill plates increased due to the local buckling [Figure 4(b)]. At 5.3% drift ratio (top displacement = 180 mm), the fracture completely penetrated the cross-section of the second story beam. To achieve better structural performance by preventing premature fracture failure of the beam-to-column connection, vertical end plates may be required at the free edges of the infill plates.

The coupled wall specimen FSPW5 exhibited stable hysteretic behavior up to 5.3% drift ratio (top displacement = 180 mm). At 5.3% drift ratio, fracture occurred in the bottom flange at the end of the second story beam and in the column base. At 0.7% drift ratio (top displacement = 22.5mm), the end plates installed at the boundary of wall opening began to deflect toward the boundary columns by the tension-field action of the infill plates [Figure 4(c)]. At the end of the test, the maximum in-plane deformation of the end plate in the first story was 42 mm.

In BSPW2, bolt-connected wall, tearing of the infill plates initiated in the first and second stories at 2.7% drift ratio (top displacement = 90 mm). At 5.3% drift ratio (top displacement = 180 mm), the second story infill plate completely fractured along its diagonal direction [Figure 4(d)], and the fracture penetrated the flange of the second story column. In the bolt-connected wall, tearing of the infill plates was aggravated by the out-of-plane displacement caused by the slip of the bolt-connection. For this reason, deformation capacity of BSPW2 was 67% of that of FSPW2 with weld-connected infill plates.

In the CBF, at 0.9% drift (top displacement = 30 mm), in-plane buckling occurred at the center of the compression braces on the first and second stories. As the bending deformation of the braces due to the buckling increased, the second floor beam visibly deflected downward at 1.8% drift (top displacement = 60 mm).

In the MRF, plastic hinges were developed at the ends of beams and at the column base. At 5.3% drift (top displacement = 180 mm), a fracture occurred at the tension flange of the third story column that was connected to the top beam. At 7.1% drift (top displacement = 240 mm), a fracture occurred at the beam-to-column joint in the second story.

3.3. Energy Dissipation Capacity

Figure 5 shows the variations in the cumulative energy dissipation capacity of the specimens according to their story drifts. The energy dissipation capacity of FSPW2 was greater than the energy dissipation capacity of FSPW4 and FSPW5 with partially weld-connected infill plates and coupled wall. At 3.6% drift, the energy dissipation capacities on the first, second, and third stories of FSPW2 were in the ratio 1:1.09:0.80. This result indicates that the plastic deformation was uniformly distributed along the wall height. Before the buckling of the brace occurred at 0.9% drift ratio (top displacement = 30 mm), the energy dissipation capacity of the CBF was similar to that of FSPW2. However, the total cumulative energy dissipation of FSPW2 was 5.8 times that of the CBF.

The energy dissipation capacity of FSPW4 with partially weld-connected infill plates was less than that of FSPW2 with rigidly weld-connected infill plates. This result is because the effective area of the infill plate that

participated in the tension-field action decreased due to the partial weld connection. Similarly, the energy dissipation capacity of the coupled wall specimen FSPW5 was less than that of FSPW2, because the effective area of the infill plate was decreased by the wall opening. At the drift of 5.3% ratio (top displacement = 180 mm), the ratios of the cumulative energy dissipation of FSPW4 and FSPW5 to that of FSPW2 were 0.65 and 0.73, respectively.
The energy dissipation capacity of the BSPW2, that had bolt-connected infill plates, was similar to that of FSPW2 up to their maximum story drift, 3.6% drift (top displacement = 120 mm). The ratios of cumulative energy dissipation of the bolt-connected wall BSPW2 to that of the weld-connected wall FSPW2 was 0.52, which indicates that the energy dissipation capacity of the bolt-connected wall is only a half of the capacity of the weld-connected wall.

4. CONCLUSION

Experimental study was performed to investigate the structural capacity of framed steel plate walls with thin infill plates. The principal parameters of this test were the infill plate details, such as, partially connected infill plates, coupled wall, and bolt-connected infill plates. The test results showed that, regardless of the connection details, steel plate walls showed excellent initial stiffness, strength, and energy dissipation capacity even though local buckling of the thin plates occurred at the early loading stage. Particularly, unlike conventional braced frames and reinforced concrete walls, the thin plate walls showed good deformation capacity and energy dissipation capacity. The findings obtained in the present study are summarized as follows.

1) The displacement ductility and energy dissipation of the steel plate wall FSPW2 with fully weld-connected infill plates were 2.8 times and 5.8 times those of the concentrically braced frame (CBF), and 3.3 times and 2.8 times those of the moment-resisting frame (MRF).
2) The steel plate wall with partially weld-connected infill plates exhibited an excellent deformation capacity equivalent to that of the solid wall with fully connected infill plates. Therefore, the partially connected infill plates can be applicable to ordinary buildings that typically have a large span to story height ratio.
3) The coupled wall that was stiffened only with the end plates at the wall opening exhibited an excellent deformation capacity, equivalent to the deformation capacity of the solid wall.
4) The wall with bolt-connected infill plates exhibited large initial stiffness and load-carrying capacity, comparable to the wall with weld-connected infill plates. However, at the ultimate state, due to slip of the bolt connection, the deformation capacity of the bolt-connected wall was two-third that of the weld-connected wall.

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REFERENCES

American Institute of Steel Construction (AISC). (2005). Seismic provisions for structural steel buildings, Chicago.
Behbahanifard, M. R. (2003). Cyclic behavior of unstiffened steel plate shear walls. PhD Dissertation, Dept. of Civil Engineering, Univ. of Alberta, Edmonton, Alberta, Canada.
Caccese, V., Elgaaly, M., and Chen, R. (1993). Experimental study of thin steel-plate shear walls under cyclic load. *Journal of Structural Engineering* **119:2**, 573-587.

Driver, R. G., Kulak, G. L., Kennedy, D. J. L., and Elwi, A. E. (1998). Cyclic test of a four-storey steel plate shear wall. *Journal of Structural Engineering* **124:2**, 111-120.
Elgaaly, M. (1998). Thin steel plate shear walls behavior and analysis. *Thin-Walled Structures* **32**, 151-180.
Hibbitt, Karlsson, and Sorenson Inc. (HKS). (2003). ABAQUS standard, Version 6.4, HKS, Pawtucket, RI.
Lubell, A. S., Prion, H. G. L., Ventura, C. E., and Rezai, M. (2000). Unstiffened steel plate shear wall performance under cyclic loading. *Journal of Structural Engineering* **126:4**, 453-460.
Park, H. G., Kwack, J. H., Jeon, S. W., Kim, W. K., and Choi, I. R. (2007). Framed steel plate wall behavior under cyclic lateral loading. *Journal of Structural Engineering* **133:3**, 378-388.
Xue, M. and Lu, L. W. (1994). Interaction of infilled steel shear wall panels with surrounding frame members. *Proc., Structural Stability Research Council Annual technical Session*, SSRC, Bethlehem, PA, 339-354.