EXPERIMENTAL AND ANALYTICAL STUDY ON THE BEHAVIOR OF STEEL PLATE SHEAR WALLS WITH BOX-SHAPED COLUMNS UNDER CYCLIC LOADING

Nader Khajeh Ahmad Attari¹, Mohammadhossein Akhavan Sigariyazd²*, Reihane Tavakoli¹
¹ Structural Engineering Department, Road, Housing and Urban Development Research Center (BHRC), Hekmat Ave, Noori Highway, Tehran, Iran,
² Department of Civil Engineering, Sharif University of Technology, Azadi Avenue, Tehran, Iran.*Email: mhakhavan@mehr.sharif.edu

ABSTRACT

Steel plate shear walls are lateral load resisting systems consisting of vertical steel plate infills connected to the surrounding beams and columns. One of the parameters affecting the behavior of steel plate shear wall system under lateral load is characteristic of surrounding members. Since there are lots of experimental and analytical studies on steel plate shear walls with I-shaped surrounding members, this research is an experimental study carried out on a one-third scale steel plate shear wall system with box-shaped columns along with further analytical studies. The objectives were to calculate the stiffness, strength and energy dissipation capacity of the specimen and compare them with a very similar system constructed with I-shaped columns. Cyclic loading protocol of ATC-24 was used for test. Obtained experimental results showed a good conformity between box and I-shaped specimens. It is shown that the system can provide good initial stiffness and high ultimate capacity and remain intact under seismic effects. Some analytical studies on failure modes of system with box-shaped columns were also conducted using finite-element software confirming that the columns bottom connections and their flange buckling at that point are one of the most common modes of failure and a triangular reinforcing plate at that point can improve columns connection behavior effectively.

KEYWORDS

Steel plate shear wall, Box-shaped column, Cyclic behavior, Energy dissipation.

INTRODUCTION

According to the researches in recent decades, steel plate shear walls (SPWs) can be used as lateral load resisting system in high seismic hazard areas. The studies have revealed that this system has high initial stiffness, high elastic strength and behaves in a ductile manner (Driver et al. 1997; Caccese et al. 1993; Sabelli and Bruneau 2007) This system has been used in numerous buildings before advent of design requirements. The steel plate shear wall consists of steel plate surrounded by beams and columns; the most usual kind of this system is unstiffened walls recognized as special plate shear wall in AISC341 and ASCE7; the compression strength of these walls is very low so the shear buckling happens under low shear force and the lateral forces are carried by forming the tension field of the infill plate. Prior to key research in 1980s, the design limit state for SPW was considered to be out of plane buckling of the infill panel to prevent buckling; engineers designed SPW with heavily stiffened infill plates. At that time several analytical and experimental studies (Timler and Kulak 1983; Tomposch and Kulak 1987) showed that the post buckling strength and ductility of slender-web SPW can be substantial. This post buckling behavior is referred to as tension field action.

Thorburn et al. (1983) developed a simple analytical strip model to represent the tension field action of a thin steel wall under shear load. This mechanism can be described as follows: When lateral load is applied to a steel plate shear wall, it is assumed that the shear panel just experiences the shear deformation so the shear infill plate is subjected to essentially pure shear with principle stresses (compression and tension) oriented at a 45 angle to the direction of load; as the slenderness of the plate (depth to the thickness ratio and width to the thickness ratio) is high, the buckling strength of the plate is very low, furthermore the plate will not be straight or flat initially due to the fabrication and erection tolerances so the plate buckles at low level of the force then lateral loads are transferred through the plate by principal tension stresses. Timler and Kulak (1983) verified and refined the strip model.

The steel plate shear wall system which is designed properly and has specific details is so ductile and dissipate great amount of energy; the boundary element must be designed in such a way that allow formation of the tension field of the infill plate and its attainment to maximum capacity; so they play a key role in the accurate
performance of the system. The behavior of thin steel plate shear walls regarding frame members was studied by Alinia and Dastfan (2006); the results showed that the flexural stiffness of surrounding members has no significant effects, either on elastic shear buckling or on the post-buckling behavior of shear walls. The torsional rigidity has a significant effect only on the elastic buckling load, and the extensional stiffness slightly affects the post-buckling capacity.

In addition to gravity loads, the boundary columns on the sides of a steel plate shear wall resist the bulk of overturning moment and also provide an anchor point for tension field action. In structures with relatively large columns, these elements can also transfer a considerable amount of story shear. So the behavior of steel plate shear walls under the effects of lateral loads depends on the surrounding columns.

In this study two one-third scaled, single bay-single story SPW were studied. Specimens were both similar in web plate thickness and other specification but different in column type as boundary element; one with boxed-shaped columns and the other one with I-shaped columns. Both specimens are tested in laboratory under quasi-static loading.

EXPERIMENTAL PROGRAM

Specimen specifications

The experimental program consists of a two one-third scaled, single bay-single story specimens with box-shaped and I-shaped columns called “SPW-1” and “SPW-2”, respectively. Specimens were designed according to high seismic design provisions of AISC Design Guide 20 requirements. For preliminary design it is assumed that web plate resists the entire shear of the system and the angle of the tension stress which varies between 30 to 60 degrees is assumed to be 45 degrees. By these assumptions the nominal stress can be calculated as:

$$V_n = 0.42f_y t_w L_{cf} \sin(2\alpha)$$  

(1)

Where $f_y$ is the infill panel yield stress, $t_w$ is the thickness of the infill plate, $L_{cf}$ is the clear distance between Vertical Boundary Elements (VBEs) flanges, $\alpha$ is the inclination angle of the tension field. The ultimate strength of a steel panel is fully developed only when the corresponding frame members are sufficiently stiff and strong to anchor the tension diagonals. For VBEs it has been recommended that moment of inertia should be calculated as:

$$I_c \geq 0.00307 t_w h^4 / L$$  

(2)

In which $h$ is height of panel. The same equation by swapping $h$ and $L$ is applicable to Horizontal Boundary Elements (HBEs). Regarding limitations on size of specimens and maximum allowable load, specimens are designed. To calculate the portion of the shear resisted by the web plate ABAQUS finite element (FE) software was used. From the analysis it was conducted that 35% of lateral load was resisted by the web plate. Details of designed specimens are shown in Figures 1-4.

As shown in Figure 3, built-up sections are used for beams and columns. In order to establish complete rigid connection, the connection of each column to bottom beam was developed utilizing complete penetration groove welds and fillet weld of web. For top beam connection, similar detail was used. In order to prevent the out of plane movement of top beam, two beams were applied as lateral support in top level of each specimen in both sides. Lateral support locations are shown in Figures 1,2.

Bottom beam flanges of specimens are connected to strong floor girder of the laboratory using M24 high strength bolts. The arrangements of these bolts are shown in Figure 4.
Figure 1 Specification of SPW-1 specimen (mm)

Figure 2 Specification of SPW-2 specimen (mm)

Figure 3 Beam and column sections (mm)
From construction point of view it is not easy to fit the web plate in the boundary frame. Therefore in steel plate shear walls, the connection of web plate to boundary frame is done using fish plates. In the above described specimens, fish plates with dimension of 40x4 mm were used. Noticing the low thickness of web plate, using bolts to connect the web plate to fish plates could result in high bearing stresses which could damage the plate. Hence it was decided to use welds to connect web to fish plates in all specimens.

**Material Properties**

The material of web plates and boundary frames were ST12 and ST37 steel, respectively, according to DIN standard. The strength of web plates was selected such that it was lower than the strength of boundary frames; which made the advantage of using smaller boundary elements. Mechanical properties of the steel plates used in the construction of the specimens were tested which are reported in Table 1. For each plate thickness, two samples were cut out from the plate, one in longitudinal direction of plate roll and one in transverse direction. The mechanical properties were determined by coupon test performed according to ASTM A370-12. In Table 1, ST12 and ST37 are comparable to A36 structural steel in the US.

<table>
<thead>
<tr>
<th>Type</th>
<th>Steel grade in Germany (DIN Standard)</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_u/f_y$</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web plate (Pl. 1.5 mm)</td>
<td>ST12</td>
<td>200</td>
<td>222</td>
<td>315</td>
<td>1.42</td>
<td>42</td>
</tr>
<tr>
<td>Column and Beam (Pl. 10 mm)</td>
<td>ST37</td>
<td>200</td>
<td>254</td>
<td>383</td>
<td>1.51</td>
<td>43</td>
</tr>
<tr>
<td>Column and beam (Pl. 15 mm)</td>
<td>ST37</td>
<td>200</td>
<td>280</td>
<td>423</td>
<td>1.51</td>
<td>43</td>
</tr>
<tr>
<td>Diagonal and fish plate (Pl. 4 mm)</td>
<td>ST37</td>
<td>200</td>
<td>313</td>
<td>450</td>
<td>1.44</td>
<td>38</td>
</tr>
</tbody>
</table>

**Test setup**

Figures 5,6 Show the fabricated and installed specimens, SPW-1 and SPW-2, prior to testing, respectively. The specimens were painted with whitewash. Water was added to hydrated lime until a workable mixture was formed. The whitewash was left overnight prior to testing. The whitewash provided a good visual indicator of the specimen behavior.
Loading and measurements

To simulate earthquake load and to investigate the cyclic behavior of the specimens, quasi-static loading was employed. Lateral loading was applied by two horizontal hydraulic jacks at the top beam level using ATC-24 protocol (1992). In this protocol it is required to determine the yield force and displacement. As was formerly described, ABAQUS software was used to simulate the test to predict these values. Monotonic analysis of specimens was applied to determine the yield displacements ($\Delta_{\text{yield}}$) of specimens which were measured as 8mm.

Displacement control lateral load was applied to specimens according to the following procedure:

- The first nine cycles consisted of fully reversing displacements of 25%, 50% and 75% of $\Delta_{\text{yield}}$, three of each.
- The next three cycles consists of displacements equal to $\Delta_{\text{yield}}$.

Loading is followed by three cycles of 200%, 300%, 400% of $\Delta_{\text{yield}}$ and so on until the maximum stoke of hydraulic jacks are achieved. The loading protocol is shown in Figure 7.

The loading history was interrupted with small cycles after each three cycles of loading with displacement equal to or larger than $\Delta_{\text{yield}}$ based on ATC-24 recommendation. These small cycles were force control, using a peak value of 0.75 yield force, which is the force that causes yielding to occur.

Also it is worth mentioning that because the displacement at the top of the specimens was monitored as the control point and there were some slipping and rotation at base connection, it was not possible to precisely achieve the target drifts, particularly at low drift levels. The actual drifts attained were always lower than the target drifts. All the drift values reported in the next sections are actual drift values.

During the loading, the value of applied load and the amount of displacement at the top of the specimens were continuously measured using load cells and LVDTs. Besides, LVDTs and strain gauges were used to measure the amount of deformation and strain at other points of the specimens where all of the measurements were recorded by a data logger. The positions of the LVDTs and strain gauges are shown in Figure 8.
EXPERIMENTAL RESULTS

Difference in mechanical characteristics and properties including stiffness, energy dissipation, ultimate strength and failure mode distinguish the seismic behavior of different configuration of steel plate shear walls. The test procedures and results are described and discussed in the following sections.

Behavior of Specimens during loading

Specimen SPW-1

In the first 6 cycles of loading (up to drift 0.6%) there were no significant yielding lines on the infill plate, however web plate buckling was observed in the early cycles of loading. Web plate yielding occurred during cycle 7 when drift reached 7.12mm (0.64%). In this cycle the stain gauge’s stress on the plate exceeded the yielding limits of the plate. In cycle 13 with 16mm (1.44%) drift several bangs were heard due to the deformation of the infill plate, post buckling waves were clearly observed and the first tear was detected in the top left corner of the infill plate (Figure 9(a)). In cycle 14 with the same lateral load, the second and third tears were observed at two other corners of the web plate which grew larger by increasing the loading amplitude in the subsequence cycles. In cycle 17 with 24mm (2.16%) drift a tiny crack in the right column bottom groove weld was detected and at the same time buckling of the column flange happened. By continuing the loading sequences the connection of the other column also cracked. In cycles 19 and 20, groove weld distortion as well as beam column connection distortion progressed and the tears increased thoroughly and finally in cycle 21 the loading was terminated (Figure 9(b)).

Total number of cycles applied to the specimen was 21. The ultimate story displacement was 50 mm and the ultimate load carried by the specimen was 820 KN at 2.2% drift.

Specimen SPW-2

In was observed that the infill web plate underwent elastic buckling during the first cycle of loading when the drift was 2.55 mm (0.23%). The loading continued until in cycle 10 when the drift reached 9.31 mm (0.84%), the web plate experienced permanent out of plane deformation (Figure 10(a)) while yielding occurred at the bottom of the outer flange of columns according to data recorded by data logger. As it was expected, the boundary
element yielding happened after complete yielding of the web plate. By continuing and increasing the applied load, the web and flange of the bottom beam were cut out right under the columns during cycle 14 when the drift was 20.03 mm (1.82%). The loading was terminated in cycle 18 at the drift of 31.5 mm (2.86%). Figure 10(b) shows the specimen at the end of loading.

![Figure 10](image)

The maximum shear capacity of the specimen was observed to have reached 714 kN during cycle 13 at 20.36 mm (1.85%) drift.

**Hysteresis behavior and structural properties of the specimens**

Hysteresis curves of tested specimens are shown in Figure 11. The quantitative comparisons are proposed in the following sections.

![Hysteresis Curve](image)
Because the ability of dissipating input energy is one of the main characteristics of a reliable structural lateral resistance system, the amount of cumulative dissipated energy is calculated and for different drift levels. As mentioned earlier, since the actual drift applied by the electrohydraulic jacks are different than the target drifts, it does not make sense to calculate and compare the cumulative dissipated energies by the specimens at each individual cycle of hysteresis, so these amounts are shown and compared at different drift levels. The absorbed energy by SPW-1 at 1 and 2 percent drift is 18.3 and 38.2 kN.m respectively while the absorbed energy by SPW-2 at 1 and 2 percent drift is 14.3 and 30.1 kN.m respectively. It shows that the amount of energy absorbed by SPW-1 is 27% more than SPW-2 at 2 percent drift which shows better behavior of this specimen due to more rigidity of boundary elements.

Initial stiffness of specimens are 89.9 and 96.3 kN/mm for SPW-1 and SPW-2, respectively which shows specimens are very similar regarding stiffness.

Comparative study of the results about strength of specimens shows 15% increase in maximum base shear capacity by using box-shaped columns as boundary elements.

**Finite element modeling**

**Model Description**

Finite element models of tested specimens were developed for comparison with experimental results. Frame members were modeled explicitly as built-up sections of plate elements to capture plastic hinging and local buckling observed in the specimens during testing. Due to the narrow width, FE modeling of the fish plates which were used to connect the infill plate to the boundary elements was ignored; instead a direct connection was used to model this connection, an approximation whose effects on analysis results were found to be negligible (Driver 1997). Since the bottom beam connection to rigid floor beam was established using some bolts and it was not practically possible to connect them entirely, there are some points to be considered for base connection; at bolt locations, springs are used with stiffness equal to AE/L of each bolt in which A, E and L represent area, modules of elasticity and grip of each bolt, respectively. At other points of bottom beam and rigid floor beam connection, springs with theoretically infinite stiffness in compression and zero stiffness in tension are defined and used.

Both the infill plate and boundary elements were modeled using the four-noded S4R element, a general purpose shell with reduced integration to avoid shear locking.

Nested surface model provided within ABAQUS known as kinematic hardening was used to represent the stress-strain behavior of infill panel and the frame member materials for plastic behavior. Kinematic hardening considers shifting of the yield surface without expansion during plastic straining and is proper for pushover analysis. The kinematic hardening component of the material model is considered using true stress (Cauchy stress) and logarithmic strain, $\sigma_{true}$ and $\varepsilon_{true}$, respectively.
**Loading and imperfection modeling**

To implement lateral load and to investigate the behavior of the specimens, in the lab, lateral loading was applied by horizontal hydraulic jack at the top beam level using ATC-24 protocol. For FE modeling, load was applied to the models through monotonic top beam displacement.

The initial shape of each specimen infill panel was not recorded prior to testing, although in general, small out of plane deformations (deviation from perfect flatness) due to fabrication process, welding distortion and assemblage were present which led to buckling of the infill plate. These imperfections need to be considered in FE analysis of the specimens. To account for the initial imperfections, an eigenvalue buckling analysis was performed to determine the first infill panel buckling mode prior to pushover analysis of each specimen model. The first panel buckling mode multiplied by a small displacement amplitude (10 mm) was applied as the initial conditions of the three specimens for pushover analysis. The magnitude of the imperfection was found to have a negligible effect as buckling of the plate occurred as soon as the system pushed.

**Comparison of Results**

Figure 12 shows the deformation shape of SPW-1 model a lateral displacement corresponding to 3% interstory drift and contours representing the magnitude of the von Mises stress in MPa. For SPW-2 is also similar deformation shape.

The stress concentration at the beam-column connection, column bottom connection and bottom beam web at column connection zone is consistent with those observed during testing (by flaking of white wash as a qualitative measure of yielding).

Presented in Figure 13 is a comparison between the analytically predicted results for the SPW-1 and SPW-2 and the test result observed in the lab. As it can be seen good agreement between analytical and experimental results are held.

![Figure 12 Deformed shape and stress distribution of SPW-1 FE models at 3% drift](image)

![Figure 13 Comparison of the analytical and experimental results: (a) SPW-1; (b) SPW-2](image)
CONCLUSIONS

This study presented the procedure and results from an experimental and analytical investigation on seismic behavior of steel plate shear walls. Two 1/3-scaled one story single bay specimens denoted by SPW-1 and SPW-2 with box-shaped and I-shaped vertical boundary elements, respectively, were designed and built for the testing program. Through a parametric study the characteristics, advantages and drawbacks of system with box-shaped columns compared to system with I-shaped columns.

The hysteretic behavior of the specimens showed that the envelope curves of the SPW-1 has formed a more spindle shape and is richer than SPW-2 as it was verified by comparing energy dissipation of these two specimens in the past section. The results of comparative studies indicated that box-shaped columns improve some of the structural properties of steel plate shear walls. For example, the energy dissipation of SPW-1 at 2% drift is 27% greater than SPW-1; moreover maximum base shear capacity of SPW-1 is 15% higher.

Finite element study of specimens using ABAQUS software was also conducted and good agreement between analytical and experimental results for both models was observed.

According to these findings, it is concluded that using box-shaped columns as vertical boundary elements in steel plate shear wall system can improve its characteristics as these elements have a good torsional rigidity.

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REFERENCES


