Experimental study on diagonally stiffened steel plate shear walls with central perforation

Erfan Alavi *, Fariborz Nateghi 1
Structural Engineering Research Center, International Institute of Earthquake Engineering and Seismology, IIEES, P.O. Box 19395-3913, Tehran, Iran

A R T I C L E I N F O

Article history:
Received 15 August 2012
Accepted 4 June 2013
Available online 5 July 2013

Keywords:
Steel plate shear wall
Diagonal stiffener
Perforated
Opening
Behavior factor
Ductility ratio
Strengthening

A B S T R A C T

One of the advantages of Steel Plate Shear Walls (SPSWs) is the easiness of openings application in infill plate. The openings are sometimes required for passing utilities, architectural purposes, and/or structural reasons. However, the recent researches on perforated steel plate shear walls have shown that the shear strength and stiffness of an un-stiffened steel shear wall decrease due to perforation of the infill plate. Hence, this paper presents a special combination of diagonal stiffeners with a central perforation. The seismic behavior of the new system is experimentally investigated and compared to the solid infill plate models. Experimental testing is performed on three ½ scaled single-story SPSW specimens under cyclic quasi-static loading. One of the specimens is un-stiffened and the two others are diagonally stiffened, which in one of them, a circular opening with the diameter of ⅓ depth of the panel is cutout from the wall center. It is observed that by means of the proposed stiffening method the shear strength of the perforated shear walls is achieved close to the un-stiffened wall with the solid panel, and the seismic behavior of the system is considerably improved. Test results show that the ductility ratio of the specially perforated specimen is about 14% greater than the un-stiffened specimen. A formula is developed and verified for the estimation of the shear strength of a perforated and diagonally stiffened SPSW. There are good agreements between the experimental outcomes and the theoretical predictions.

© 2013 Elsevier Ltd. All rights reserved.

1. Introduction

Upon the recent researches, Steel Plate Shear Wall (SPSW) is known as a reliable lateral load resisting system in the high seismic risk zones [1–4]. In addition, one of the advantages of steel shear walls is the easiness of application of openings in the infill plate, which sometimes are required for passing utilities, architectural purposes, and/or structural reasons. Nonetheless, the current building codes and structural designers are mostly conservative against using perforations in the shear walls, where they prescribe special details and restrictions whenever openings are required. Since, if a perforated shear wall is not designed properly, the seismic performance of the structure might be diminished. Besides, application of openings in the shear walls usually complicates the structural analysis. Moreover, the recent researches on perforated steel plate shear walls have verified that the shear strength and stiffness of an un-stiffened steel shear wall diminish due to perforation of the infill plate which may not be often desirable in the design. These contradictory requirements have provided research fields toward the goal of finding solutions for reducing the undesirable effects of openings on the structural and seismic properties of steel shear walls. A history of the main researches on the perforated SPSWs is briefly presented in the following:

The idea of using special openings in shear walls returns to Omori et al. [5] and Mutoh et al. [6] who proposed using slits in reinforced concrete shear walls in order to improve the seismic behavior of the RC shear walls. Hitaka and Matsu [7] studied the performances of slits in steel shear walls. They tested 42 steel plate walls of one-third scaled specimens under monotonic and cyclic lateral loading. All specimens behaved very ductile, although some degradation in the shear strengths of the walls happened after initiation of the out-of-plane buckling in the plates. They applied vertical edge stiffeners to restrain out-of-plane deformation of the plate, and found them effective.

Roberts and Sabouri-Chomi [8] carried out a series of cyclic quasi-static testing on 16 specimens which numbers of them had central circular openings. The panel depth (d) was taken 300 mm in the all of specimens and the values of the circular opening diameter (D) were selected as 0, 1/5, 1/3, and 1/2 of d. The connections of the peripheral frames were of hinge type. The loading on the specimens was applied diagonally. On the basis of the experimental results, they concluded an approximate strength and stiffness reduction factor as (1 − D/d) for a perforated panel with a circular single hole at its center in comparison to the solid panel. The pinching was observed in the cyclic loops of the tested specimens, especially, this phenomenon became increased in the envelope curves of the perforated specimens.
The effects of holes in the infill plate of un-stiffened SPSWs were also investigated by Vian and Bruneau [9]. They used low yield steel material for the infill plates and performed experimental testing on three \( \frac{1}{2} \) scaled single-story with single-bay specimens. One of the specimens was specially perforated with multiple holes laid out in the steel panel. In the second specimen, quarter-circles were cutout from the panel corners, and the corners were reinforced to transfer the panel forces to the adjacent frame. The third specimen was designed as a SPSW with the solid panel; it was tested as a reference sample. They reported that all the specimens resisted against imposed input history of increasing displacements to a minimum drift of 3%. The elastic stiffness and overall strength of the perforated panel were found decreased by 15% in comparison to the solid panel sample.

The recent investigations on diagonally stiffened steel shear walls have shown that the diagonal stiffeners increase shear strength and improve cyclic behavior of a thin SPSW, authors [10,11]. On that account, it is deemed that a combination of special perforation with diagonal stiffeners might behave effectively. Hence, this study has focused on the rehabilitation of the shear strength and stiffness of perforated panels and even improvement of the non-linear behavior of perforated steel walls by means of the diagonally stiffening method. This paper presents the research results from experimental and numerical investigations on three \( \frac{1}{2} \) scaled single-story SPSWs. A set of comparative studies are also included to evaluate the seismic performance of a diagonally stiffened steel shear wall with a central opening in competition with the un-stiffened and diagonally stiffened solid-plate shear walls. Furthermore, a formula is derived and proposed for the estimation of shear strength of the new system.

2. SPSW specimens and test set-up

Laboratory study was conducted on \( \frac{1}{2} \) scaled single-story single-bay specimens of diagonally stiffened and un-stiffened steel shear walls at IIEES (Tehran-Iran). The laboratory is equipped with two reaction steel frames and a strong base, significantly stiffer than the specimens. One of the reaction frames was employed for installation of the specimens and applying the loads and the second frame for the lateral supporting of the specimens. Each test was performed under fully reversed cyclic quasi-static loading in compliance with ATC-24 [12] test protocol. The horizontal loads were applied on the specimens by means of a hydraulic jack with 1000 kN capacity. The gravity load of a magnitude for a typical building and corresponding to the dimensions of the specimens was applied to the specimens by a vertical jack. Total gravity load was taken as 160 kN; in which, 80 kN for each column of the specimen.

2.1. Steel shear wall specimens

Three \( \frac{1}{2} \) scaled one-story specimens with around 2 m width and 1.5 m height of SPSWs were designed and fabricated for the test program. The width-to-height aspect ratio of the specimens was selected as 1.33 to represent the moderate dimensions of a shear wall in the buildings. This aspect ratio was considered corresponding to the actual sizes of a shear wall with 4 m width and 3 m typical story height. The boundary elements were made of the standard profile HEB160, and the infill steel plate thickness taken 0.8 mm for SPSW(\( s_1 \), 2) and 1.0 mm for SPSW(\( s_4 \)). The boundary elements were such designed to meet the preliminary requirements of steel shear walls and AISC 341-05 [13] provisions. Full moment connections were provided at all beam-to-column joints by complete penetration groove welds and using the electrode type of E7018. At the top of each specimen, an additional HEB160 was placed on the frame beam and they were welded along with their flanges to each other to better anchor the internal panel forces and to contribute transferring loads of the horizontal jack to the specimen. This method was previously examined by Lubell [14] and resulted in good performance of the un-stiffened specimen.

In the specimen SPSW(\( s_1 \)), two-sided diagonal stiffeners plate of 40 mm × 4 mm were utilized in combination with the edge stiffeners (40 mm × 4 mm). The specimen SPSW(\( s_4 \)) was stiffened with two-sided diagonal and edge stiffeners plates (40 mm × 5 mm), and it was perforated in the centre with a circular opening. The diameter of hole was selected 400 mm corresponding to 1/3 of the wall depth. Observations on SPSW(\( s_1 \)) test regarding tears zone led to this size of the opening, besides, a hole with this relative dimension to the wall height could also satisfy the needs of the accessibility. The special hole was stiffened by means of a ring-shape stiffener plate of 90 mm × 5 mm. The diagonal stiffeners were placed between the edge and the ring-shape stiffeners and connected to them and to the infill plate by the fillet welds. Fig. 1 shows a framing view and geometrical details of this specimen. The third specimen SPSW2 was designed un-stiffened as a reference sample. Steel materials were chosen from the structural steel type. The mechanical properties of the steel materials are indicated in Table 1 were measured from the tension coupon tests in accordance with ASTM A370-05 [16].

The following steps were taken in the design of the diagonal stiffeners:

a) The stiffeners contribution in the seismic behavior of the shear walls was first evaluated through the pushover analysis of the stiffened systems, and the preliminary dimensions of the stiffeners were obtained accordingly.

b) Concerning the local buckling prevention, the width-to-thickness ratio (b/t) of the stiffeners was designed in compliance with the requirements of the compacted section requirements. In the specimen SPSW(\( s_4 \)), this ratio was taken under the plastic limit (b/t ~ 8.5) to improve performance of the stiffeners in the high inelastic zones.

c) The section and dimensions of the edge stiffeners were designed similar to the diagonal stiffeners, not less than them. The edge stiffeners were placed near to the midpoint of the connection of the fish-plates to each other in the corners, parallel to the diagonal stiffener in the same direction. They were also welded to the fish-plates.

d) An argon gas shield was used in the welding process of the stiffeners to the infill plate, because the infill plates were of the thin plates. This method of welding is known as MIG (metal inert gas) in the case of argon.

Fish plates with dimensions of 70 mm × 5 mm were used all around the panel for connection of the infill plates to the boundary members. A similar connection to the modified detail type B of the work of Schumaker et al. [15] was utilized. Accordingly, four holes with the dimensions of 25 mm × 25 mm were cut out in the corners of the fish plates to relieve the corners from the stress concentration. Fillet welds were used in connecting the fish plates to the boundary elements. The infill plate was connected to the fish plates by means of Argon welding since the thin plates were used for the infill plate.

2.2. Test Set-up

A test set-up was designed and arranged to meet the experimental requirements. For that mean, in addition to the analytical and numerical studies on the models, the existing experiences on the experimental works of the steel plate shear walls were taken into the design of the test set-up. A view of the test set-up is shown in Fig. 2. More details of the test set-up are described in following:

A strong plate girder was placed between the base plates of the specimens and the strong floor in order to provide appropriate connection between the two bases and to move the specimens up to a required elevation. The lateral supports were designed and installed at the top level of the specimens; two numbers of them were placed near to the two ends of the specimens and the third one adjacent to
connection of the vertical jack in the middle of the span. The lateral supports did not have any mechanical connection to the specimens due to not preventing from movement of the specimens inside the loaded plane. Thus, only lubricated contact surfaces were provided between the specimens and the lateral supports to keep the specimens in-plane against any lateral or rotational movements.

Numbers of Linear Variable Differential Transformers (LVDTs) were mounted to determine out-of-plane displacements at the joints and the connections to ensure that the specimen remains in-plane throughout the test. LVDTs were also used to measure the base plate movements and to control the fixity of the base connection. A number of strain-gauges were connected to the columns as well as beams and panel to register strains through an electronic data acquisition channel system. Locations of the strain-gauges were specified at probable plastic zones in the elements based on the preliminary numerical analysis results. Figs. 3 to 5 represent the fabricated and installed specimens prior to the testing.

3. Numerical analysis

Finite element models of the specimens were generated to predict behavior of the specimens in the testing. Other objective of the primary finite element study was to determine locations of the maximum stresses and strains in the elements for strain-gauges arrangement. The analytical results were also used for preliminarily design of the specimens and the test set-up. The modeling was performed using the general-purpose nonlinear finite element ANSYS software that its recent editions (Ver. 9 or upper) are suited for the solution of nonlinear engineering problems as SPSWs.

Multi-linear kinematic hardening plasticity model was assumed for the mild steel materials based on the coupon test data. Large displacement transient analyses were performed to incorporate the nonlinear buckling and post-buckling effects into the outputs. The implicit solution method based on Newmark’s algorithm was chosen. 4-node plastic shell type 181, which had six degrees of freedom at

| Table 1 |
| Mechanical proprieties of steel materials from the tension coupon tests. |

<table>
<thead>
<tr>
<th>Steel material</th>
<th>Elastic modulus (MPa)</th>
<th>Static yield (MPa)</th>
<th>Static ultimate (MPa)</th>
<th>Yield strain (%)</th>
<th>Hardening strain (%)</th>
<th>Ultimate strain (%)</th>
<th>Rupture strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HEB160 (SPSW(s1,s4))</td>
<td>2.06E+05</td>
<td>340</td>
<td>450</td>
<td>0.17</td>
<td>1.8</td>
<td>14.4</td>
<td>16.2</td>
</tr>
<tr>
<td>HEB160 (SPSW2)</td>
<td>2.07E+05</td>
<td>400</td>
<td>450</td>
<td>0.19</td>
<td>2.7</td>
<td>13.2</td>
<td>15.1</td>
</tr>
<tr>
<td>Plate (THK = 5 mm)</td>
<td>2.05E+05</td>
<td>340</td>
<td>470</td>
<td>0.17</td>
<td>3.06</td>
<td>20.5</td>
<td>22.3</td>
</tr>
<tr>
<td>Plate (THK = 4 mm)</td>
<td>2.05E+05</td>
<td>460</td>
<td>550</td>
<td>0.22</td>
<td>2.67</td>
<td>19.1</td>
<td>20.8</td>
</tr>
<tr>
<td>Plate (THK = 0.8 &amp; 1 mm)</td>
<td>2.04E+05</td>
<td>280</td>
<td>500</td>
<td>0.14</td>
<td>0.3</td>
<td>21.6</td>
<td>27.0</td>
</tr>
</tbody>
</table>

Fig. 1. Perforated specimen SPSW(s4) details.
each node and was developed for reduction of the convergence problems, was employed for meshing and modeling of the shear wall specimens. The numerical procedure had been previously verified considering the available experimental results by authors [10,11]. Also, the actual situations of the fabricated specimens including the initial imperfections were considered in the finite element analyses. Thereby, the numerical models were calibrated in accordance with the experimental specimens.

The validated FE models also provided useful tools for evaluating the parameters which were not specifically investigated in the testing. The stress contours obtained from the push-over analyses are represented in Figs. 6 to 8 on the basis of Von-Mises yield criterion, for the final loading step at around 70 mm horizontal displacement. It can be seen that the stresses in some zones reached near to the ultimate strength of the steel materials, especially at the edges and the ends of the columns. From the contours, the probable tear locations in the tests are also investigated and compared to the experimental outcomes. Fig. 9 exhibits the strains distribution and concentration in the specimen SPSW(s4) at the last loading step. It can be foreseen that the tears most probably take place around the opening in the middle of the web plate. From the analyses, the pushover load-displacement curves are obtained and shown in Fig. 10. The maximum shear strength of the models is about 745 kN from the FE analyses. This value shows adequate margin to the horizontal jack capacity (1000 kN). In addition, the pushover curves imply that the specimens might tolerate around 5% story drift.

4. Cyclic Loading Program

As mentioned, one of the purposes of the numerical analysis of the specimens was to estimate the yield load and displacement of the specimens. These data were required to establish the loading program in compliance with ATC-24 test protocol. According to that protocol, the loading history consists of stepwise increasing deformation cycles, and a number of cycles are recommended to be applied in each loading step. Therefore, in the elastic region, the amplitudes of the initial two displacements were taken as $\frac{1}{3}$ and $\frac{2}{3}$ of the estimated yield displacement, and three cycles were specified to be performed in each one of these load steps. Considering that, 6 cycles were defined to be performed before the yield point of the specimen. And, at the yield displacement ($\delta_y$), the number of cycles was selected as three. After the yield point and for the next three load steps, the number of cycles was also taken as three. Then, for the next load steps up to the final, the number of cycles was reduced to two at each loading step. In the all successive load steps with a peak deformation greater than $\delta_y$, the increment in peak deformation was a constant. This increment was set equal or near to $\delta_y$. Fig. 11 represents the resulting values for the cyclic displacement-control loading history of the specimen SPSW(s4). It shows the interstory displacement magnitude on the vertical axis versus the cyclic numbers on the horizontal axis; the 70 mm target displacement corresponds to around 4.6% drift.
5. Test Results

The test results here consist of observations, hysteretic behaviors, envelope curves, strains, deformations, failures, etc. The outcomes are also extended by analyzing the experimental results. The tests and the main results are described and discussed hereinafter:

5.1. SPSW(s1) test

The response of the diagonally stiffened SPSW(s1) specimen was almost linear during the first 9 cycles of the testing up to around 0.4% drift. In cycle 13 the infill plate was deformed as shown in Fig. 12. At this stage, 0.86% drift, the inclined virtual tension fields are observable inside the four divided areas of the infill plate. It is also detected that the diagonal stiffeners reduced the buckling lengths of the plate by around half. In 1.5% drift (~3Δy), the local buckling happened in a pair of the stiffeners which caused around 10% drop in the shear resistance of the system in one-side of the cyclic curves. The width to thickness ratio (b/t) of the stiffeners was 10 in this specimen, upper than the plastic limit. By increasing the amount of lateral displacement (drift > 2.4%), a number of local tears were initiated adjacent to the diagonal stiffeners in the non-linear region. However, the tears did not have significant effects on the shear strength degradation. Finally, the test was terminated in cycle 28, Fig. 13. The tolerated drift by the specimen SPSW(s1) at the final loading step is 4.7%. And, the registered shear capacity of the shear wall is 712kN in the post-yield region.

5.2. SPSW(s4) test

In the design of SPSW(s4), a circular hole with ½ height of the wall was perforated in the wall center. The thickness of the infill plate was taken 1 mm in order to reduce the effects of welding on the connection zones of the stiffeners to the thin plate. The width to thickness ratio (b/t) of the stiffeners was 8, under the plastic limit. During the cyclic testing, the buckling of the plate occurred between the diagonal stiffeners at 0.56% drift and 353 kN base shear; Fig. 14 shows the wall at that status. By increasing the horizontal drift, the lateral load became larger where in 1.3% and 2.4% drifts the shear loads reached 595 kN and 700 kN, respectively. After that, some tears started happening around the opening in the infill plate, Fig. 15. Despite the tears, the lateral strength of the system was still rising where at 3.3% drift the horizontal loading reached 740 kN and remained approximately constant up to 3.8% drift. In the next cycles, some degradation occurred in the shear strength, and the testing was continued to 4.4% drift and stopped at cycle 28. Fig. 16 shows the specimen at the final loading step.

It can be highlighted that no fracture or local buckling was observed in the stiffeners up to the test end. The strain-gauges data indicated that the diagonal stiffeners yielded during the test, where their strains passed 0.44% (m/m). This amount of the strain was more than the yield limit (0.17%), as given in Table 1. The greatest strain was registered as 1.86% in the edge stiffeners which illuminated that the edge stiffeners also yielded.

5.3. SPSW2 test

This specimen was an un-stiffened thin steel shear wall. The responses were nearly elastic up to around 0.4% drift. The wave-shaped buckling phenomenon appeared in the infill plate at around 0.8% drift.
Fig. 7 shows the specimen at 2.5% drift. The shear strength of the specimen reached 765 kN in 60 mm horizontal displacement in cycle 26. Eventually, the specimen tolerated 4.6% drift and the test stopped in cycle 28; Fig. 18 exhibits the specimen at its final status. Numbers of local tears and fractures occurred in the groove welds of the fishplates in the four corners.

Maximum Stresses at the edges and the end of columns

Fig. 8. SPSW2, Von-Mises stress (Pa) at final load step.
5.4. Frame behavior

Well performances of the surrounding frames are observed during the tests in the all specimens. A number of the strain-gauges were connected to the probable plastic zones of the frame elements, as can be seen in Figs. 3–5. Observations and the strain-gauges outputs indicated that the columns yielded at their two ends and behaved like a sway mechanism. It is noteworthy that the occurrence of the plastic hinges in the columns of steel shear walls is mostly classified as a ductile mode, as reported by Astaneh-Asl [3]. Situation of the plastic deformations in the columns can be seen in Figs. 13, 16, 18. While, the beams remained in the elastic ranges during the tests and no plastic hinges formed in them. In addition, the beam-to-column connections sufficiently responded to the cyclic loading and no fracture took place at the connections.

5.5. Hysteretic behavior

Important information is usually gleaned from the hysteresis loops of the structural systems. In that regard, the load–displacement curves of the specimens obtained from the tests are presented in Fig. 19. From these curves, it is observed that all of the specimens have stable hysteretic behavior in the inelastic regions. In spite of the perforation, particularly, the pinching phenomenon did not happen in the envelope curves of the specimen SPSW(s4). This implies that the combination of the diagonal stiffeners and the central opening result in good performance of the system under the cyclic loading. The hysteresis loops also show that no significant loss of the strength and stiffness occurred in the responses of the combined system in the large drifts.

5.6. Dissipated energy

One of the main properties of a lateral resistant system subjected to large cyclic loading is its ability to dissipate the energy. Thus, the dissipated energies by the specimens are computed and compared to each other. The cumulative dissipated energies by the specimens are evaluated from the summation of the areas surrounded in the loops. Fig. 20 exhibits the graphs of the dissipated energies in the cycles during the tests. From comparison of the results, it is found that the absorbed energies by SPSW(s1) are more than the two others in the all phases of the testing. In addition, the dissipated energies by SPSW(2) and SPSW(s4) are appropriately close. Absorption of the furthest amount of the cyclic energy by the SPSW(s1) implied that the diagonal stiffeners were able to increase the areas under the passed loops, and consequently, the dissipation of the input energy was increased. Moreover, the closeness of the dissipated energies by SPSW(2) and SPSW(s4) signified that the combined system was able to sustain the well performance of the shear wall despite the perforation.
5.7. Structural responses

The effects of the perforation and the diagonal stiffening are also investigated on the structural properties of the shear walls. Concerning the seismic displacements, the experimental data indicated that all specimens could tolerate the high story drifts between 4% to 5% and endured 28 cyclic loops. In particular, the specimen SPSW(s4) with circular perforation was successfully tested to a maximum interstory drift of 4.4% and a base shear of 740 kN. This shear strength is close enough to the value obtained for the un-stiffened solid type SPSW2 as 765 kN, where only 4% difference is seen between the two base shears. Furthermore, the lateral elastic stiffness of the specimens resulted in about 99.2 kN/mm, 76.4 kN/mm, and 95.0 kN/mm for SPSW(s1, 2, and s4), respectively. Comparison of these values shows...
that the elastic stiffness of the perforated and diagonally stiffened panel is nearly 24% greater than the un-stiffened solid type. As a result, the reduction in the lateral stiffness of a perforated steel shear wall is rehabilitated or even improved by means of the diagonal stiffening method.

5.8. Seismic factors

From the experimental results, the seismic factors such as the ductility ratio, behavior factor (R), and the overstrength factor (Ω) are evaluated for the steel shear wall systems. These factors are usually used in the elastic force-based analysis method according to the seismic design codes. There are several methods for the evaluation of the seismic factors of a structural system. In this study, the proposed method by Uange et al. [17] is used for the evaluation of the factors. In this method, the structural system is assumed as an idealized bilinear relation regarding the base shear versus the roof displacement. The R is described as a product of numbers of factors as expressed by Eq. (1), for LRFD method. The generic envelope curves and the force-displacement components associated with the definition of R are illustrated in Fig. 21.

\[
R = \frac{V_r}{V_s} = \frac{V_r}{V_y} \times \frac{V_y}{V_s} = R_u \Omega
\]

where, \( R_u \) is the period-dependent ductility factor, and \( \Omega \) is equal to the maximum base shear (\( V_r \)) divided by the design base shear (\( V_s \)); \( V_y \) corresponds with the first yield of the structural elements that

---

Fig. 16. SPSW(s4) at 4.4% drift, in cycle 28.

Fig. 17. SPSW at 2.5% drift.

Fig. 18. SPSW at 4.56% drift, in cycle 28.

Fig. 19. Hysteresis curves of the specimens, from the tests.
causes softness in the real envelope curve of the system, and $\Delta_s$ is the displacement at $V_s$; $\Delta_y$ is the displacement at $V_y$; $V_e$ is the ultimate elastic base shear, and $\Delta_e$ is the displacement at $V_e$; $\Delta_{\text{max}}$ is the maximum horizontal displacement.

The definitions proposed by Newmark and Hall [18] are elaborated for the evaluation of $R_\mu$ of the specimens since the tests are performed on the single-story specimens. Therefore, $R_\mu$ is assumed to be equal to $\sqrt{2\mu-1}$ for $T<1$, and to $\mu$ for $T\geq 1$ s. The ductility ratio of the specially perforated specimen is evaluated $\mu = 8.7$ that is about 14% greater than the un-stiffened solid type value as $\mu = 7.6$. In the assessment of the effective ductility ratios, the shear strength degradation is taken into the account to be not less than 15% of the maximum base shear in the final loading.

The components of $R$ are determined for the specimens and presented in Table 2. Evaluation of $\Delta_s$ is the first important data in drawing of the bi-linear curve. Often, the envelope curves from testing do not clearly show the first yield point of a system. Hence, parallel to investigation on the envelope curve form, the strain-gauge outputs and the numerical results are studied for finding the first yield points. Fig. 22 shows the skeleton curves of the specimens including the bilinear idealized curves. Comparison of the envelope curves of SPSW(s4) and SPSW2 show the closeness between them. This can mean that the diagonal stiffeners led to preclude the reduction in the strengths as well as in the seismic behavior of the perforated shear wall. In addition, it is observed that the diagonal stiffeners effectively reduced the yield displacement of the system which resulted in increase of the ductility ratio of the stiffened shear walls. The yield points of the systems are indicated in Table 2.

The last column of Table 2 represents the $R$ factor values which are computed from the minimum of $\{R_\mu_1, R_\mu_2\}$ that is multiplied by $\Omega$. The values indicate that the $R$ factors for the diagonally stiffened SPSW without opening and with the central opening are 9.5 and 9.9, respectively. While, the $R$ for the un-stiffened shear wall is around 8.4. The overstrength factor $(\Omega)$ of the specimens are obtained between 2.24 and 2.44. The response values shows that the behavior factor of the specially perforated shear wall is about 18% greater than the $R$ factor of the un-stiffened SPSW.

### 6. Implications to design

The current building codes and a number of the previous researches on the perforated shear walls have often required structural designers to consider the diminishing effects of openings on the shear strength as well as on the non-linear behavior of the shear walls. However, this study showed that by means of the proposed stiffening method not only the shear strength and the dissipation of energy of the perforated shear walls could be adjusted and maintained close to the un-stiffened wall with the solid panel, but also the behavior factor of the system was considerably improved.

In addition, to extend the design procedure, a formula is developed for the prediction of the shear strength of a diagonally stiffened steel plate shear wall with a central perforation, as stated by Eq. (2). The total shear strength ($V$) is evaluated based on the contribution

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Elastic disp., $\Delta_s$(mm)</th>
<th>Yield disp., $\Delta_y$(mm)</th>
<th>Ductility ratio, $\mu = \Delta_{\text{max}}/\Delta_y$</th>
<th>Over-strength factor, $\Omega_2$</th>
<th>Ductility factor, $R_\mu_1$ $(T&lt;1 \text{ s})$</th>
<th>Ductility factor, $R_\mu_2$ $(T\geq 1 \text{ s})$</th>
<th>Response factor, $R$ $(\text{min.})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPSW(s1)</td>
<td>3.3</td>
<td>7.6</td>
<td>9.3</td>
<td>2.27</td>
<td>4.20</td>
<td>9.3</td>
<td>9.5</td>
</tr>
<tr>
<td>SPSW2</td>
<td>4.5</td>
<td>10.1</td>
<td>7.6</td>
<td>2.24</td>
<td>3.77</td>
<td>7.6</td>
<td>8.4</td>
</tr>
<tr>
<td>SPSW(s4)</td>
<td>3.6</td>
<td>8.9</td>
<td>8.7</td>
<td>2.44</td>
<td>4.05</td>
<td>8.7</td>
<td>9.9</td>
</tr>
</tbody>
</table>

![Fig. 20. The cumulative dissipated energies in the cyclic tests.](image1.png)

![Fig. 21. Generic base shear versus roof displacement envelope curves.](image2.png)

![Fig. 22. Envelope and bilinear curves of the specimens.](image3.png)

![Table 2](image4.png)
of the peripheral frame.

\[ V = \left( 1 - \frac{D}{d} \right) V_p + V_{st} + V_{sc} + V_f \]  

(2)

where,

- \( V_p \) is shear force, taken by the steel infill plate;
- \( V_{st} \) is shear force, taken by the diagonal tensile stiffener;
- \( V_{sc} \) is shear force, taken by the diagonal compressive stiffener;
- \( V_f \) is shear force, taken by the frame.

The reduction factor of \( 1 - D/d \) is applied to \( V_p \) according to the work of Roberts and Sabourin [8]. The value of \( D \) in the equations is equal to zero for a system without any opening. The frame capacity \( (V_f) \) is related to type of beam-to-column connections and the frame behavior. Thus, value of \( V_f \) is taken as zero if beam-to-column connections are of the simple type. And, in the rigid type, the governing failure mechanism of frame is considered in the estimation of \( V_f \). As described in Section 5.4, plastic hinges formed at the two ends of the columns, therefore, this mechanism is here assumed for the rigid frames.

By substitution of the shear forces components in Eq. (2), the summarized form of that equation is expressed as Eq. (3):

\[ V = \left( 1 - \frac{D}{d} \right) \left( \frac{1}{2} \sigma_{yw} bt \sin 2\alpha \right) + A_t (\sigma_{st} + \sigma_{sc}) \cos \theta_d + \frac{4M_{pc}}{d} \]  

(3)

where, \( D, b, t \), and \( M_{pc} \) are the hole diameter, panel depth, panel width, infill plate thickness, and the column plastic moment, respectively. \( A_t \) is the cross sectional area of tensile or compressive diagonal stiffener, \( \theta_d \) is the inclination of the diagonal stiffeners with respect to the horizontal axis. \( \alpha \) is the angle of tensile strips with the column, modified by Timler and Kulak [19]. This definition for \( \alpha \) is used here, because the tension field development and the buckling phenomenon of infill plates are observed in the testing. Detection of the inclinations implies that the angle of tension fields in the diagonally stiffened walls is not noticeably changed in comparison to the un-stiffened panel. Expression \( \sigma_{st} \) is the tensile axial stress, and \( \sigma_{sc} \) denotes the compressive axial stress in diagonal stiffeners. The supplementary equations are given in the Appendix A.

### 6.1. Verification of the theoretical method

The shear strengths obtained from the theoretical and the experimental methods are presented in Table 3. For the specimen SPSW(s4), the ratio of \( D/d \) is equal to 1/3 and the shear strength from the theoretical approach is resulted 736.2 kN, while from the cyclic test the maximum shear force is 740 kN. The last column of Table 3 displays the proportion of the shear strengths values. With reference to that column, it can be brought into the results that the variations between the two approaches are more than 4% for all the shear walls. These results indicate that the theoretical method has good accuracy for the estimation of the shear capacities of the SPSWs.

For calibration of the evaluated seismic factors, a comparison is made between the current test results and the available values in ASCE 7-10 [20] code. In this code, the response and the overstrength factors are recommended as \( R = 8 \) and \( \Omega = 2.5 \) for the un-stiffened SPSWs with the special rigid frames, for use in LRFD method. From the testing on that type of steel shear wall system (SPSWS), these factors are obtained as \( R = 8.4 \) and \( \Omega = 2.24 \), which are close to that code data.

### 7. Conclusions

This paper presented the experimental study on a multipurpose perforation in the steel shear wall system to provide an access through the shear wall without significant diminishing effects either on the strengths and stiffness or on the non-linear behavior of SPSWs. In that regard, three half-scaled single-story steel plate shear walls with different systems were designed and successfully tested. Numerical nonlinear analyses were first performed on the finite element models of the specimens to predict the behavior of the specimen during the test. The numerical results provided initial indications of the well performance of the specially perforated system in comparison to the other two systems.

The cyclic tests showed that the specimens had had stable hysteresis loops and behaved as appropriate energy dissipative systems. In particular, no pinching in the envelope curves of the specially perforated specimen SPSW(s4) occurred despite the perforation. On that account, the combination of the diagonal stiffeners with the perforation can be considered as an effective and applicable modification on the perforated shear panels.

From the comparative studies, it is also observed that the structural properties of the specially perforated and diagonally stiffened shear

### Table 3

Theoretical and experimental shear strengths of the shear wall specimens.

<table>
<thead>
<tr>
<th>SPSW specimen</th>
<th>Diagonal stiffeners</th>
<th>Theoretical values</th>
<th>Experimental values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \frac{ts}{bs} )</td>
<td>( \frac{\sigma_{yw}}{\tau_{cr}} )</td>
<td>( \alpha )</td>
</tr>
<tr>
<td>SPSW(s1)</td>
<td>4</td>
<td>10</td>
<td>1.88</td>
</tr>
<tr>
<td>SPSW(s4)</td>
<td>5</td>
<td>8</td>
<td>2.93</td>
</tr>
<tr>
<td>SPSW2</td>
<td>0(^a)</td>
<td>0.58</td>
<td>43.2</td>
</tr>
</tbody>
</table>

\(^a\) Unstiffened.

\(^b\) For un-stiffened specimen SPSW2 \( K = 5.35 + 4/\phi^2, \phi \geq 1 \) \( K = 4 + 5.35/\phi^2, \phi \leq 1 \) are used in the calculation of \( \tau_{cr} \) [22].
wall are close to the un-stiffened solid type or even improved in some areas. For instance, the difference between the base shear strengths of the two systems was not more than 4%, and the elastic stiffness of the perforated shear wall was nearly 24% greater than the un-stiffened solid type.

From the evaluation of the seismic factors, it is derived that the ductility ratio (μ) of the specially perforated specimen is μ = 8.7 that is about 14% greater than the un-stiffened solid type value as μ = 7.6. Furthermore, for the seismic design (LRFD method), the behavior factors are proposed for the un-stiffened and the diagonally stiffened steel shear walls as R = 8 and R = 9, respectively.

In view of these findings, it is concluded that the special perforated and diagonally stiffened shear wall is an effective lateral load resisting system. In addition, the diagonal stiffening is an appropriate strengthening method for the shear walls with a central opening. Besides, a good agreement is observed between the theoretical predictions and the experimental outcomes.

Acknowledgements

This work was supported by the International Institute of Earthquake Engineering and Seismology, IIEES. The support and assistance of the structural laboratory specialists are acknowledged and appreciated.

Appendix A

The supplementary equations used in Eq. (3) are as follows [10,21–23]:

\[
\tau_{cr} = \frac{k \cdot \pi^2 \cdot D}{b \cdot t} = \frac{k \cdot \pi^2 \cdot E}{12(1-\nu^2)} \left( \frac{t}{t} \right)^2 \leq \tau_y = \frac{\sigma_y}{\sqrt{3}} \quad (A-1)
\]

\[
K = 11.9 + 10.1/\phi + 10.9/\phi^2 : \phi = \frac{d}{b} \quad (A-2)
\]

\[
\sigma_{sc} = \sigma_l \left[ 1 - (1 + \nu) \sin^2 \left( \theta_d + \theta \right) \right] + [(1 + \nu) \tau_{cr} \sin 2\theta_d] \leq \sigma_{crs} \quad (A-3)
\]

\[
\sigma_{st} = \sigma_l \left[ 1 - (1 + \nu) \sin^2 \left( \theta_d - \theta \right) \right] + [(1 + \nu) \tau_{cr} \sin 2\theta_d] \leq \sigma_{ys} \quad (A-4)
\]

\[
\sigma_{crs} = \sigma_{ys} \quad \text{for} \; \sqrt{2} \geq \lambda \quad (A-5)
\]

\[
\sigma_{crs} = \sigma_{ys} \left( 1 - 0.53(\lambda - 0.45)^{1.36} \right) \quad \text{for} \; 0.45 \leq \lambda < \sqrt{2} \quad (A-6)
\]

\[
\sigma_{crs} = \sigma_{ys} \quad \text{for} \; \lambda < 0.45 \quad (A-7)
\]

\[
\lambda = \left( \frac{b_s}{t_s} \right) \sqrt{12(1-\nu^2)} \left( \frac{\sigma_{ys}}{E} \right) \left( \pi^2 k_5 \right) \quad (A-8)
\]

\[
k_5 = \left( \frac{b_s}{t_s} \right)^2 + 0.425 \quad (A-9)
\]

\[
\sigma_l = -\frac{3}{2} \tau_{cr} \sin \theta + \sqrt{\left( \frac{q^2}{4} \sin^2 \theta - 3 \right) \tau^2_{cr}} \quad (A-10)
\]

Where, \( \tau_{cr} \) is the shear buckling stress of diagonally stiffened plate; \( E, \nu, \) and \( \sigma_{crs} \) are the elastic modulus, Poisson’s ratio, and the yield stress of infill steel plate, respectively. \( \sigma_{ys} \) is the yield stress, and \( \sigma_{crs} \) is the compressive buckling stress capacity of the stiffeners. \( \sigma_l \) is the tension field stress in the infill plate. \( b_s, t_s, \) and \( l \) are width, thickness, and effective length of the diagonal stiffeners, respectively.

References