Experimental Study on Diagonally Stiffened Steel Plate Shear Walls with Central Opening

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SUMMARY:
In the present study, a special combination of diagonal stiffeners with central perforation in steel plate shear walls is proposed, and seismic behavior of the new system is experimentally investigated and compared to the solid infill plate models. Experimental testing was conducted on three ½ scaled one-story steel plate shear wall specimens under cyclic quasi-static loadings. One of the specimens was un-stiffened and the two others were diagonally stiffened, which in one of them, a circular opening with diameter of ⅓ depth of the wall was cutout from the wall center, and all around of the opening was stiffened with a peripheral stiffener. Test results show that the perforated specimen tolerated comparable story drifts in competition with the two other shear walls with the solid infill plates. It is found that the specially perforated diagonally stiffened steel shear wall not only facilitates an access path through the wall without significant reduction effects on the shear strength of the wall, but also improves some of the seismic properties of un-stiffened steel shear walls, as the ductility ratio and the response modification factor.

Keywords: steel plate shear wall, diagonal stiffener, perforated, ductility ratio, response modification factor

1. INTRODUCTION

One of the attractions of steel plate shear walls, SPSWs, is the easiness of opening application in the infill plate; which are sometimes required for passing the utilities, architectural purposes, or structural reasons. On the other side, the current building codes and structural designers are mostly conservative against using perforations in shear walls, and prescribe special details and restrictions whenever openings are required; since, if they were not designed properly, seismic performance of the system might be diminished. Besides, existence of openings in the shear walls usually complicates the structural analysis. Furthermore, the recent researches on perforated steel plate shear walls have demonstrated that the shear strength and stiffness of an un-stiffened steel shear wall reduce due to perforation of the infill plate, which may not be often desirable in the design of steel shear walls. 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. In the following, the history of the main researches on perforated SPSWs is briefly reviewed.

The idea of using special openings in shear walls returns to Omori et al. (1966) and Mutoh et al. (1968), who proposed using slits in reinforced concrete shear walls in order to improve the seismic behavior of the RC shear walls . Hitaka and Matsui (2003) utilized this notion in steel shear walls and studied the performance of using slits in this system. They tested 42 steel plate walls of one-third scaled specimens under monotonic and cyclic lateral loads. All specimens behaved very ductile, and degradation of the walls' strengths happened after initiation of the out-of-plane buckling of the plates.

Roberts and Sabouri-Ghomi (1992) carried out a series of cyclic quasi-static tests on 16 specimens, which numbers of them had central circular openings. The specimens panel depth, d, was 300 mm for all specimens, and the values of the circular opening diameter, D, selected 0, 1/5, 1/3, and 1/2 of d. The connections of the peripheral frames were hinge type, and the loading on the specimens applied diagonally. On the basis of experimental results, they concluded the approximate strength and stiffness reduction factor, (1-D/d), for a perforated panel with a single hole at the center, in respect to the solid panels.

The effects of holes in the infill plate of un-stiffened SPSWs were also investigated by Vian and Bruneau (2009). They used low yield steel material for the infill plates, and did tests on three ½ 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 cut out from the panel corners, and reinforced to transfer the panel forces to the adjacent framing. The third one was a SPSW with solid panel, and was tested as a reference sample. They reported that the all specimens could resist the imposed input history of increasing displacements to a minimum drift of 3%, and the elastic stiffness and overall strength of the perforated panel reduced by 15% as compared with the solid panel specimen.

On the other hand, the authors' recent investigations on diagonally stiffened steel plate shear walls have shown that the diagonal stiffeners increase shear strength and improve cyclic behavior of a thin SPSW, Alavi-Nateghi (2009, 2010). Subsequently, the idea of combination of special perforation with diagonal stiffeners has formed on that base, and it is intended that by means of this method not only the reductions of shear strength and stiffness of the perforated panels to be retrieved but also non-linear behavior of the system to be even improved and controlled in the high non-linear deformations. Hence, this paper presents the research from experimental investigations on three ½ scaled single-story steel plate shear walls, and a set of comparative studies to evaluate seismic performances of the diagonally stiffened steel shear wall with a central opening in respect with the un-stiffened and diagonally stiffened solid plate shear walls.

2. EXPERIMENTAL PROGRAM

Laboratory test program was conducted on ½ scaled one-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 used for installation of the specimens and applying the loads, and the second employed for the lateral supporting of the specimens. Each test was performed under fully reversed cyclic quasi-static loading in the elastic and inelastic response zones of the steel shear walls samples, in compliance with ATC-24 (1992) test protocol. The horizontal loads were applied on the specimens by means of a hydraulic jack with 1000 kN capacity. The gravity loads of a magnitude for a typical building correspond to dimensions of the specimens, and equivalent to a distributed load of 10kN/m^2, were applied through a distributing beam by a vertical jack. The amount of the gravity loads on each column was 80 kN.

2.1. Steel Shear Walls Specimens

Three ½ scaled one-story specimens with around 2m width and 1.5m height of SPSWs, correspond to the conventional dimensions of the shear walls (4m×3m) in the buildings, were assumed. The boundary elements were the standard profile HEB160, and the infill steel plate thickness was taken 0.8mm for SPSW(s1,2) and 1.0 mm SPSW(s4). SPSW(s1) was diagonally stiffened and SPSW(2) was an un-stiffened steel wall with the solid infill plates; SPSW(s4) was diagonally stiffened with a central opening with 400 mm diameter. The boundary elements were such designed to meet the preliminary requirements of steel plate shear walls and AISC 341-05 provisions. Full moment connections were provided at all beam-to-column joints by complete penetration groove welds. Fish plates with dimensions of 70mm×5mm were used all around the panel for connection of the infill plates to the boundary members, and four holes with the dimensions of 25mm×25mm were considered at the corners of the fish plates to relieve the corners from the stress concentration. For connecting the fish plates to the boundary elements fillet welds was used, and the connections of the infill plates to the fish plates made by Argon welding.

In the specimen SPSW(s1), two-sided diagonal stiffeners plate of 40mm×4 mm in combination with edge stiffeners 40mm×4 mm were used. The specimen SPSW(s4) was stiffened with two-sided diagonal and edge stiffeners plates of 40mm×5 mm, and perforated with a central and circular opening with the diameter of 400 mm , equal to ⅓ d ; the around of special hole was stiffened with a ring-type stiffener of 90 mm×5 mm; the diagonal stiffeners were placed between the edge and the ring stiffeners and connected to them and to the infill plate by means of the fillet welds; Fig. 2.1 shows a framing view and the geometrical details of this specimen. The steel materials were structural steel, and their mechanical properties were measured from the tension coupon tests in accordance with ASTM A370-05 provisions. Table 2.1 indicates the data were acquired from the material tests.

Table 2.1. Mechanical proprieties of steel materials from tension coupon tests

Steel Material	Elastic Modulus (MPa)	Static Yield (MPa)	Static Ultimate (MPa)	Yield Strain (%)	Hardening Strain (%)	Ultimate Strain (%)	Rupture Strain (%)
HEB160 (SPSW(s1,s4))	2.06E+05	340	450	0.17	1.8	14.4	16.2
HEB160 (SPSW2)	2.07E+05	400	450	0.19	2.7	13.2	15.1
Plate (THK.= 5 mm)	2.05E+05	340	470	0.17	3.06	20.5	22.3
Plate (THK.= 4 mm)	2.05E+05	460	550	0.22	2.67	19.1	20.8
Plate(THK.= 0.8 & 1 mm)	2.04E+05	280	500	0.14	0.3	21.6	27.0

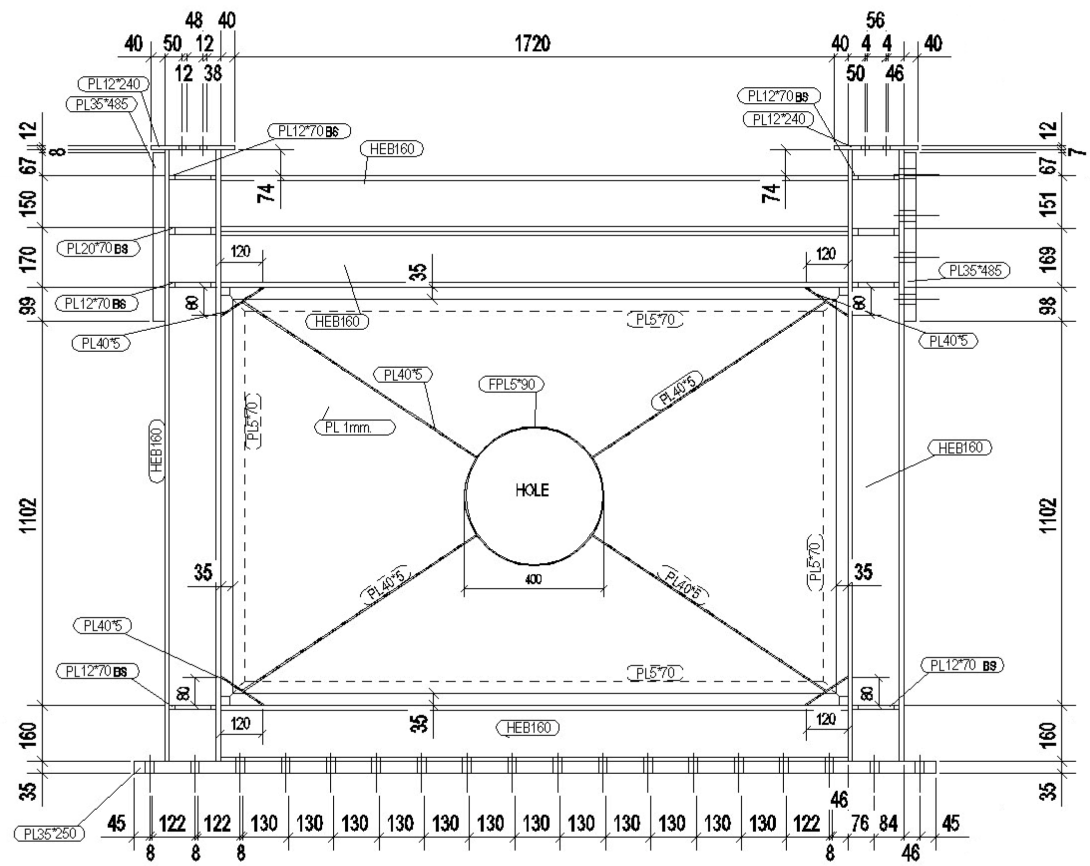

Figure 1. Special perforated specimen SPSW(s4) details

2.2. Test Set-up and Data Acquisition

A view of the arranged test set-up of the experiments is shown in Fig. 2.2. 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, considering available spaces and dimensions of the reaction frames, specimens, the horizontal and vertical jacks. The lateral supports were designed and installed at the top level of the specimens, two numbers of them were situated next to the two ends of the specimens and the third one adjacent to the connection of the vertical jack at the middle of the span, and perpendicular to the loaded plane, in order to preclude out-of-plane movement and distortion of the specimens during the tests. 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, and therefore, only lubricated contact surfaces were provided between the specimens and the lateral supports to keep the specimens in-plane, opposite to any lateral or rotational movements.

Numbers of linearly variable displacement 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 tests, and to measure the base plate movements to monitor the fixity of the base connection. Numbers of strain gauges at cross sections in the columns, beams, and panel were utilized to register the strains through an electronic data acquisition channels system by another mobilized computer. Locations of the strain were selected based on preliminary numerical analysis results, at the probable zones of maximum strains in the steel shear walls. Figures 2.3 to 2.5 show the photographs of the fabricated and installed SPSW(s1), SPSW2 and SPSW(s4) specimens, respectively, prior to the testing.

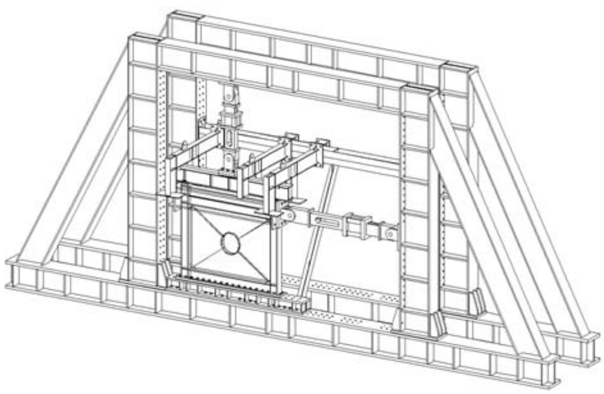

Figure 2. A view of the tests set-up

Figure 3. Diagonally stiffened specimen, SPSW(s1)

Figure 4. Un-stiffened specimen, SPSW2

Figure 5. Special perforated specimen, SPSW(s4)

3. EXPERIMENTS & RESULTS

The tests results consisted of observations, the specimens' behaviours under the cyclic loads, hysteresis loops, strains, deformations, fractures, and etc.; in the following, the significant points are summarized and presented:

3.1. SPSW(s1) test

The response of the diagonally stiffened SPSW(s1) specimen during the first 9 cycles, up to 6 mm horizontal displacement corresponds to 0.4% drift, was almost linear. From cycle 10 some local buckling happened in the infill plate, and in 0.86% corresponds to 13mm horizontal displacement the infill plate was shaped as shown in Fig. 3.6, wherein the inclined virtual tension fields were also observable inside the four divided areas of the infill plate. In 1.5% drift, local buckling phenomenon started happening in one pair of the diagonal stiffeners, the width to thickness ratio (b/t) of them was 10. In the high non-linear region (> 2.4% drift), numbers of local tears started happening adjacent to the diagonal stiffeners connections to the web plate, but caused no significant loss of the shear strength. Finally, in cycle 28 (71.1 mm hor. dis.) the specimen was deformed as shown in Fig. 3.7; the tolerated drift by the specimen SPSW(s1) at the final step was 4.7%. The registered shear capacity of the shear wall was 712 kN in the post-yield region.

Figure 6. SPSW(s1) at 0.86% drift, in cycle 13

Figure 7. SPSW(s1) at 4.7% drift, in cycle 28

3.2. SPSW2 test

This specimen was an un-stiffened thin steel shear wall; its response up to around 0.45% drift was nearly elastic, the wave-shaped buckling phenomenon in the infill plate appeared in 0.8% drift. Fig. 3.8 shows the specimen at 2.5% drift (38 mm hor. dis.). The shear strength of the specimen reached 765 kN, in 60 mm horizontal displacement in cycle 26. Eventually, the specimen tolerated 4.6% drift (69.3 mm hor. dis.), and the test finished at cycle 28; Fig. 3.9 exhibits the specimen at its final situation. Numbers of local tears were initiated by low cycle fatigue in the non-linear region resulting from the cyclic kinking of the thin infill plate at the corner of the panel. Fractures in the groove welds of the fishplates at four corners were also detected, although they did not have noticeable effects on the specimen non-linear behaviour.

Figure 8. SPSW2 at 2.5% drift

Figure 9. SPSW2 at 4.56% drift, in cycle 28

3.2. SPSW(s4) test

Considering the test results on SPSW(s1), the most tears in the wall plate occurred around the centre of the wall in the high nonlinear stages, Fig.13, which implies that the best location for putting a hole in a diagonally stiffened shear wall could be at its centre; thus, in the design of SPSW(s4) a circular hole with ⅓ height of the wall was devised thereat. During the cyclic test on SPSW(s4), the initial local buckling of the infill plate observed in cycle 7, at about 5 mm horizontal displacement, and the global buckling of the plate between the diagonal stiffeners appeared at 0.56% drift (8.5mm hor. dis.) and 353 kN lateral load, Fig. 3.10, in cycle 13.

With increasing the horizontal drift the lateral load was also becoming larger, where in 1.3% and 2.4% drifts, the shear loads reached 595 kN and 700 kN, respectively. At this step, some tears started happening around the opening in the infill plate; however, in spite of the tears, the lateral strength of the system was still rising in the next steps, where at 3.3% drift the horizontal force extended to 740 kN and remained approximately constant up to 3.8% drift in cycle 25; loss in the shear strength happened in the next cycles, and the test continued up to 4.4% drift, corresponds to 67.4 mm hor. dis. and finished at cycle 28.

It is also noteworthy that though in the greater drifts than 2.4% the tears happened and spread in the infill plate around the hole, none fracture or local buckling happened in the stiffeners up to the test end; Fig. 3.11 shows the specimen at the final loading step. The strain gauges data indicated that the diagonal stiffeners yielded during the test and the strains in them passed 0.44%, which were more than the yield strain rate in Table 1, (0.17%). The maximum strain in the stiffeners was registered around 1.86% for the edge stiffeners.

Figure 10. SPSW(s4) at 0.56% drift, in cycle 13

Figure 11. SPSW(s4) at 4.4% drift, in cycle 28

4. COMPARISON OF RESULTS & DISCUSSIONS

For assessment of the effectiveness of the new system, the tests results are extended and analyzed. In that regard, some of the main structural and seismic properties of the examined shear walls are obtained and compared in the following sections.

4.1. Hysteretic behaviours of the specimens

The load-displacement envelope curves of the specimens are represented in Figures 4-12 to 4-14. It can be inferred that all of the specimens have had stable hysteretic behaviour in the inelastic regions. In particular, the hysteresis curves of the specimen SPSW(s4) had no significant pinching in spite of the perforation, which leads to this point that the combination of the diagonal stiffeners and the central opening has resulted in good performance of the specimen under cyclic loads without significant loss of the strength and stiffness up to the high story drifts (4.4%).

4.2. Dissipation of energy

One of the important properties of a lateral resistant structural system, subjected to large cyclic loading, is its ability to dissipate hysteretic energy. For a cycle of load reversal, the hysteresis energy is taken the area enclosed by the loop of the force-displacement curve, and under the repeated cycles of loading, the energy dissipated in each cycle is summed to calculate the total energy dissipated. The cumulative energies dissipated by the specimens have been evaluated from the hysteresis loops, and the graphs showing the dissipated energies in the cyclic movements of the specimens during the tests are displayed in Fig. 4-15. From comparison of the graphs, it is brought into the results that the absorbed energies by SPSW(s1) has been more than the two others in nearly all phases of the tests; in addition, the dissipated energies in SPSW(s4) and SPSW2 tests were close to each other. Reasons of the furthest absorption of the cyclic energy by the specimen SPSW(s1) can be interpreted from the more spindle shape, fatter, of the hysteresis loops of this specimen than the other ones. This result can also mean that the diagonal stiffeners have been able to better control the lateral stiffness of the system from the degradation in the non-linear regions and in the load reversals, which have caused increase of the areas under the passed loops, and consequently, the total amount of the dissipated energies. Besides, the closeness of the absorbed energies by the un-stiffened specimen and the perforated stiffened specimen indicates that by the diagonal stiffening of the shear panels with central opening, the performance of the shear wall is remained unchanged with regard to the absorption of the input energies.

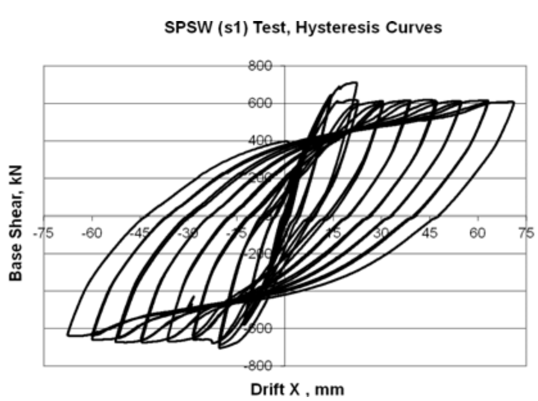

Figure 12. Envelope curves of SPSW(s1)

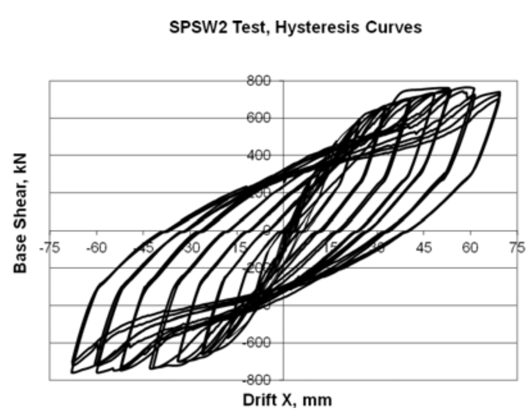

Figure 13. Envelope curves of SPSW2

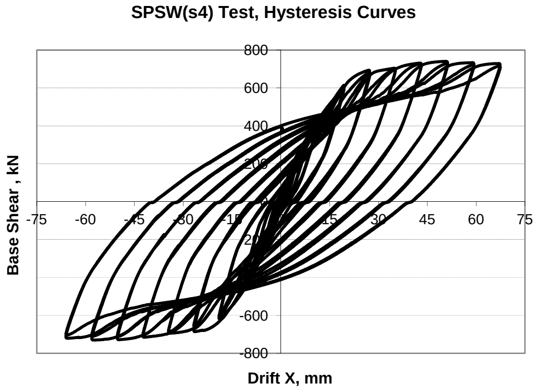

Figure 14. Envelope curves of SPSW(s4)

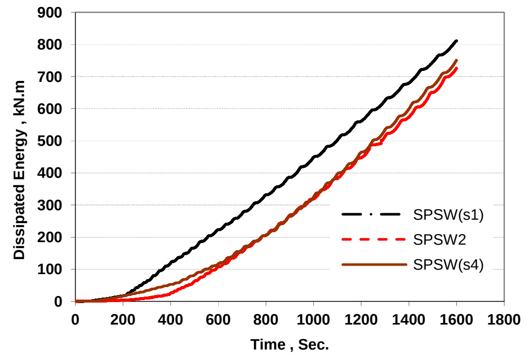

Figure 15. Cumulative Dissipated Energies by the specimens in the cyclic tests

4.3. Structural properties

The elastic lateral stiffness, the peak shear strength, ductility ratio, overstrength and response modification factors of the specimens are obtained and indicated in Table 4.2. Specimen SPSW(S4), with centeral circular perforation, was successfully tested to a maximum interstory drift of 4.4% , 8.7δy, and maximum base shear of 740 kN, the specimen strength was approximate to the value obtained for the un-stiffened solid type, SPSW2, as 765 kN , where utmost only 4% different is found between the base shear strenghts. The lateral elastic stiffness of the perforated and diagonally stiffned panel is resulted nearly 24% higher than the un-stiffened solid type. Furthermore, the ductilty ratio, μ, of the specially perforated specimen is evaluated 8.7 that is about 14% greater than the un-stiffened solid type value, 7.6. The all specimens have tolerated the high story drifts, 4% to 5%, and the 28 cycles. In evaluation 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 high non linear region. Furthermore, the response modification factor, R, and the overstrength factor, $\Omega_{\circ}$ in accordance with the method introduced by Uange et al. (1991) are computed. In this method, R is estimated from an idealized bilinear curve for the system that is expressed by Eqn. 4.1; the generic envelope curves and the forces-displacement components associated with the definition of *R* are illustrated in Fig. 4.16.

$$R = \frac{V_e}{V_s} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R_{\mu}\Omega_{\circ} \qquad (4.1)$$

Where, R_{μ} is the period-dependent ductility factor, and $\Omega_{\circ}$ is the overstrength factor and equal to the maximum base shear (V_y) divided by the design base shear (V_s), V_s corresponds to the first yield of the structural elements that causes softness in the real envelope curve of the system. V_e is the ultimate elastic base shear, Δ_e is the displacement at V_e , and $\Delta_{\max}$ is the maximum horizontal displacement. To ensure about the elastic point determination at the actual curves, in parallel with investigating on the curves forms, further controls have been made by means of the strain-gauges outputs and the numerical models. The strain-gauges were installed in the possible locations of the plastic zones based on the numerical studies. The ductility factor, R_{μ} , is a measure of the global nonlinear response of a framing system, and depends mainly on the ductility ratio μ , fundamental period (T), soil type, and the damping ratios. In this study, the definitions were proposed by Newmark and Hall (1982) are elaborated for evaluation of R_{μ} of the specimens, since the tests were performed on the single-story specimens; hence, R_μ is assumed equal to $\sqrt{2\mu-1}$ for T < 1, and μ for T ≥ 1 sec. The determined values for R components of the specimens are indicated in Table 4.2. The last column in Table 4.2 represents R values, are obtained from min. $\{R_{\mu 1}, R_{\mu 2}\}$ multiplied by $\Omega_\circ$.

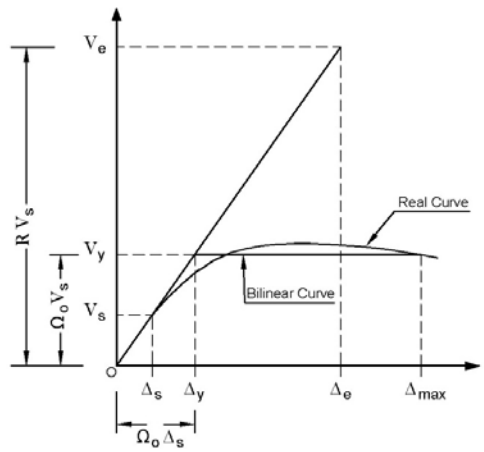

Figure 4-16. Generic base shear versus roof displacement envelope curves

Table 4.2. Structural values for each specimen from the cyclic tests

Specimens	Elastic Stiffness, (kN/mm)	Peak Base Shear, (kN)	Ductility Ratio, μ=Δmax /Δy	Over-strength Factor, $\Omega_\circ$	Ductility Factor, $R_{\mu 1}$ (T<1 sec)	Ductility Factor, $R_{\mu 2}$ (T≥ 1 sec)	Response Factor, R (min.)
SPSW(s1)	99.2	712	9.3	2.27	4.20	9.3	9.5
SPSW2	76.4	765	7.6	2.24	3.77	7.6	8.4
SPSW(s4)	95.0	740	8.7	2.40	4.05	8.7	9.7

The values indicate that R for the diagonally stiffened SPSW without opening and with the central opening, in the LRFD method, are 9.5 and 9.7, respectively; while, R for the un-stiffened shear wall is obtained 8.4. This shows that the response factor of the specially perforated shear wall is about 15% greater than the un-stiffened wall, while, the overstrength factors of the specimens are about 2.2 and 2.4, respectively. In ASCE 7-10 code, R factors of the steel shear walls are specified for the dual special frames systems, 8; and $\Omega_\circ$= 2.5. From comparative study between the test results and the code data, it can be inferred that the tests outputs have been near to the code values for the un-stiffened SPSWs with the special rigid frames, R=8.4 and $\Omega_\circ$=2.24 against R=8 and $\Omega_\circ$=2.5, respectively. However, seemingly, no difference is set forth between the un-stiffened and stiffened shear walls in that code; despite, this series of the tests on the diagonally stiffened and un-stiffened steel shear walls leads to this point that the response factors differ in the two systems. Hence, the code values can be conservative if they are assumed for the stiffened steel shear walls.

5. CONCLUSIONS

This paper presented the experimental study on a multipurpose perforation in the steel shear wall system to not only provide possibility of having the accesses through the walls, but also to not have significant deductive effects either on the elastic or on the inelastic behaviors of the steel shear walls. In that regard, application of the perforation on the diagonally stiffened steel plate shear walls was investigated; to demonstrate the performance of the special perforated system, three half-scaled one-story steel plate shear walls with different systems were developed; SPSW(s1) was diagonally stiffened without perforation, SPSW2 was un-stiffened with solid infill plate, and SPSW(s4) was diagonally stiffened and perforated with a circular hole cutout from the centre of the wall.

From the comparative study, it is found that despite the perforation, the structural responses of the perforated diagonally stiffened shear wall have been near to the un-stiffened solid type or even improved in some areas. For example, the pinching phenomenon has not happened in none of the hysteresis loops of the specimens, the dissipated energy by the special perforated system in the cyclic tests is near to the un-stiffened system with the solid infill plate, variations between the base shear strengths of the systems are not more than 4%, the tolerated interstory drifts by the special perforated specimen has been 4.4% that is in a good range in comparison with the other specimens results and the existing data from the tests on a steel shear wall system, the ductility ratio (μ) is obtained 8.7 for this system that is about 14% greater than the un-stiffened value, the greatest ductility ratio is approximated 9.3 for the diagonally stiffened model with the solid infill plate, the elastic stiffness of the special perforated diagonally stiffened system has been near to the diagonally stiffened specimen with the solid infill plate that is nearly 24% greater than the un-stiffened type, and etc.

In view of these findings, it can be concluded that the diagonally stiffened steel shear wall with a central circular opening with the diameter of about one-third of the wall height, has behaved sufficiently effective as a seismic resistant system, without considerable reduction effects on the shear strength; rather, the combination of the central hole and the diagonal stiffeners improves some of the un-stiffened shear walls properties as the ductility ratio and the response modification factor around 15%. Based on the tests outcomes, the response modification factors for the un-stiffened shear wall and the diagonally stiffened shear walls are proposed R=8 and R=9, respectively, for the LRFD method.

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