Earthquakes and Structures, *Vol. 13, No. 5 (2017) 473-484*
DOI: https://doi.org/10.12989/eas.2017.13.5.473

Seismic upgrading of reinforced concrete frames with steel plate shear walls

Hasan H. Korkmaz* and Ali S. Ecemis[a]

Engineering Faculty, Civil Engineering Department, Selcuk University, Konya, Turkey

(Received February 8, 2016, Revised December 14, 2017, Accepted December 23, 2017)

Abstract. The objective of this paper is to report on a study of the use of unstiffened thin steel plate shear walls (SPSWs) for the seismic performance improvement of reinforced concrete frames with deficient lateral rigidity. The behaviour of reinforced concrete frames during seismic activities was rehabilitated with an alternative and occupant-friendly retrofitting scheme. The study involved tests of eight 1/3 scale, one bay, two storey test specimens under cyclic quasi-static lateral loadings. The first specimen, tested in previous test program, was a reference specimen, and in seven other specimens, steel infill plates were used to replace the conventional infill brick or the concrete panels. The identification of the load-deformation characteristics, the determination of the level of improvement in the overall strength, and the elastic post-buckling stiffness were the main issues investigated during the quasi-static test program. With the introduction of the SPSWs, it was observed that the strength, stiffness and energy absorption capacities were significantly improved. It was also observed that the experimental hysteresis curves were stable, and the composite systems showed excellent energy dissipation capacities due to the formation of a diagonal tension field action along with a diagonal compression buckling of the infill plates.

Keywords: steel plate; lateral rigidity; earthquake; rehabilitation; reinforced concrete frame

1. Introduction

A large portion of older existing reinforced concrete (RC) buildings in many countries around the world are seismically deficient. These buildings typically include structural members with non-seismic reinforcement details, strong beam-weak columns, or other non-ductile frame systems that are vulnerable during earthquakes (Korkmaz 2015, Ozturk 20015). Economical, fast and efficient retrofit methods are urgently needed to increase lateral stiffness, strength and ductility of these buildings. If the design details of an existing RC building do not meet the design requirements of the modern seismic codes, their deformation capacity or ductility is usually limited (Arslan and Korkmaz 2007). One of the major objectives of seismic retrofit and design is to increase the displacement capacity, preferably above an acceptable limit. For example, a lower limit for the interstory drift ratio was proposed by Sozen (1987) as 1%. This limit can be considered as an indication of the damage level in RC structures. Above a 1% interstory drift ratio, the structural and nonstructural damage can be significant.

Different methods and techniques have been developed and applied in recent decades to strengthen existing structures (Korkmaz *et al*. 2010). In practical rehabilitation applications, reinforced concrete infill walls and steel bracings have been frequently used mainly to increase the lateral stiffness of RC-framed structures. These retrofit systems not only increase the overall stiffness of the structures but also upgrade the lateral strength of the frame. On the other hand, in steel buildings, steel plate shear wall (SPSW) systems have been given more attention in the past few years as one of the major lateral-load resisting systems used around the world. A steel plate shear wall is a lateral force-resisting system that consists of thin steel panels added as infill to a building's structural frame (Tsai and Lin 2005). This type of system has been used in the design of new steel buildings as well for the retrofit of existing steel buildings (Zhao and Astaneh-Asl 2008).

SPSWs can effectively resist horizontal earthquake forces by allowing development of a diagonal tension field action after the infill plate buckles under large shear forces. Because of the yielding of the infill steel plates in both directions and associated hysteretic energy dissipation, this system can behave in a ductile manner (Qu *et al*. 2013). Due to their high strength and stiffness, steel plate shear walls have drawn the attention of many practicing engineers, and a number of studies have been carried out (Enami *et al*. 2013, Lubell *et al*. 2000, Seilie and Hooper 2005, Berman *et al*. 2008)

To take benefits of SPSWs, the steel plate needs to yield prior to the failure of the boundary beams or columns. Notwithstanding the advantages of the steel plate shear walls listed above, previous studies have presented some drawbacks of the SPSW system.

If the shear buckling occurs in the early stage, permanent out-of-plane deformation may occur and then affect the serviceability of the thin plate shear walls after a small or moderate earthquake (Chen and Jhang 2011). The shear buckling would also result in pinching in the hysteretic response (Chen and Jhang 2006).

The formation of the tension field action, which is a

*Corresponding author, Professor
E-mail: hasanhusnukorkmaz@gmail.com
[a]Ph.D.
E-mail: serdar@arfenmuhendislik.com

ISSN: 2092-7614 (Print), 2092-7622 (Online)

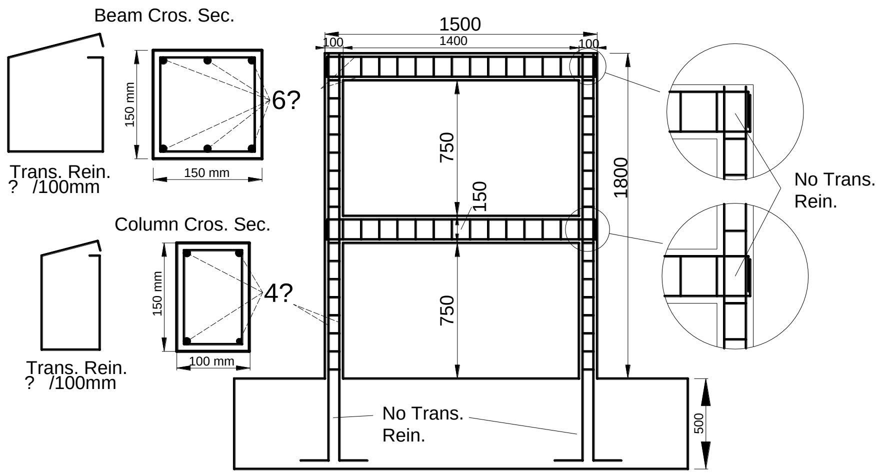


Fig. 1 Dimensions and reinforcement details of test specimens

fundamental requirement for the satisfactory nonlinear performance of SPSWs, would increase the demand on the boundary beams and columns and often lead to a premature failure in those boundary elements if they are not designed accordingly (Zhao and Astaneh-Asl 2008). To allow a complete plastic tension field action to develop in the steel infill plates in all stories, the boundary frame members must have a large inelastic deformation capacity. Otherwise, the system is expected to exhibit soft-story behavior (Choi and Park 2011) or other non-ductile failure modes.

In the light of above brief introduction, the research motivations of this study can be summarized as;

1) To increase the lateral stiffness, strength and ductility of RC-framed structures, economical, effective and feasible retrofit methods are required. The main goal of this research is to propose and develop an occupant friendly, rapidly deployable, economical and effective strengthening method for RC structures in seismic regions. A SPSW system can be a viable additional seismic force-resisting system for RC structures with low stiffness and strength (Korkmaz and Korkmaz 2015).

2) The use of SPSWs in steel buildings has been popular for reducing lateral displacements. On the other hand, the use of SPSWs within RC frame systems is a relatively new concept, and to the best of the authors' knowledge, few studies have been conducted on steel plate shear walls and RC-framed composite systems. Choi and Parks study in 2011 is appeared to be first experimental report on the use of SPSWs with RC frames. The main objective of this study is to investigate potential use of steel plate shear wall systems to increase the lateral stiffness and strength of existing RC frames. The authors' goal is to install SPSW systems on the external surface or façade of the structure so that the interior parts of the dwellings remain intact and the occupants do not need to vacate the building.

2. Experimental research

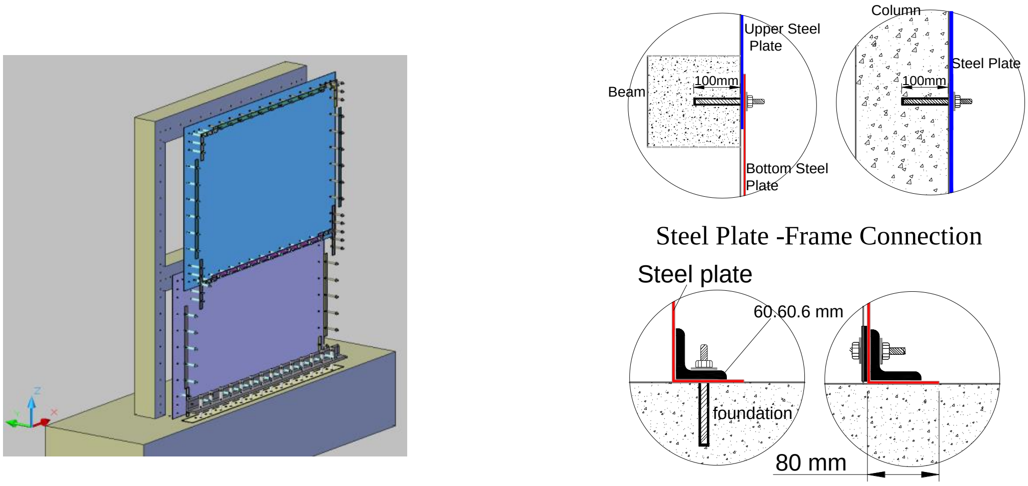
In the experimental part of the study, seven RC frame specimens were constructed and tested under lateral cyclic loading. The observed failure mechanisms, strength, stiffness degradation, energy absorption capacity, and characteristics of the measured hysteresis loops were described and discussed. The RC frames with low lateral resistance were rehabilitated with external thin steel plates. The thickness of the plates were 0.3 mm in 1/3 scale. The thickness of the plates was determined through preliminary finite element modelling analysis. The thicker plates result in higher shear forces in columns and premature failure before yielding of the steel plates was achieved.

2.1 Test specimens

The material properties and design of the specimens were based on practical engineering considerations and code requirements to represent older existing buildings. The reference model building, and hence laboratory specimens, represent buildings with low lateral stiffness a concrete compressive strength of approximately 16 MPa. The reinforcement detailing of the columns and beams are deficient and do not meet the seismic design requirements of the modern building codes. Thus, the specimens were constructed with commonly observed poor reinforcement details. Seven geometrically identical, one bay and two story test specimens were constructed. The test specimens represent 1/3 scale model of an ordinary moment frame with weak columns and strong beams (Korkmaz 2016).

Total height of the specimens was 1800 mm. The width of the test frame was 1400 mm between the centerlines of the columns. The columns and beams had 100×150 mm and 150×150 mm gross cross sectional dimensions, respectively. Fig. 1 shows the dimensions and design details of the specimens.

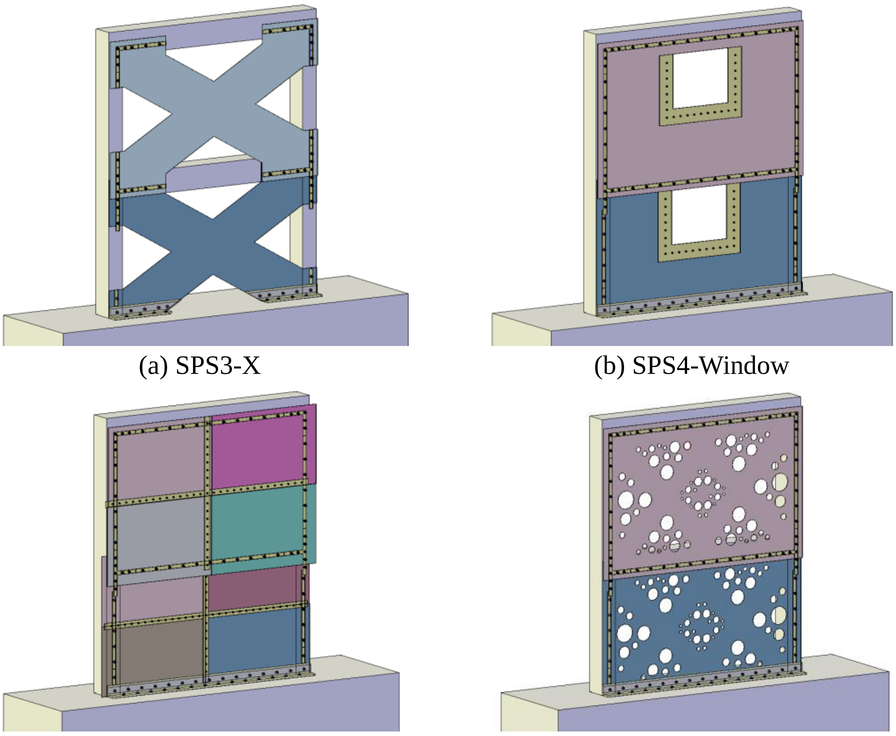
Four, 8 mm diameter deformed longitudinal bars were used in columns with a steel ratio of 1.34%. The ends of longitudinal column bars were bent 90^{o} inside the footing. Six 8 mm diameter deformed bars were used in the beams (reinforcement ratio=1.34%. The uniform vertical spacing of 6 mm diameter plain column ties was 100 mm. The 90 degree end hooks of the columns ties were anchored inside the cover concrete to represent a common seismically deficient reinforcement detailing practice. The thickness of the cover concrete was chosen as 8 mm.



Steel Plate -Frame Connection

Steel Plate -Foundation Connection

Fig. 2 Specimen SPS2-Full with anchor details



(a) SPS3-X (b) SPS4-Window

(c) SPS5-Four Pieces (d) SPS6-Perforated

Fig. 3 Schematic illustrations of test specimens

The first specimen, RF1 was the reference bare RC frame with no steel plate shear wall installed (Korkmaz 2012). All other specimens included steel plate infill walls and RC frames with design and material properties virtually identical to those of RF1.

For the second specimen called SPS2-Full, two thin steel plates with a thickness of 0.3 mm were attached to the external face of the RC frame. The steel plates were anchored to both beams and columns using bolts at a uniform spacing of 100 mm. The spacing was reduced to 50 mm near the joint regions.

In order to obtain bonding between the old concrete and anchor bolts, the holes were cleaned with pressured air and filled with epoxy. The preferred epoxy has 4 minute hardening time. The anchor bolts are manufactured from solid circular threaded rods. In old buildings, if the surface of the RC members are not smooth, then, surface treatment may be required and also the length of the anchor bolts may be increased. A steel angle section was used to provide a relatively rigid connection (or fixity) between the bottom of the steel plate in the lower story and the foundation. The details of the external retrofitting scheme were illustrated in Fig. 2. This fixation scheme is possible when the external surface of the beams and columns are in the same plane.

The third specimen, which was named SPS3-X, included X-shaped steel plates (Fig. 3(a)). The objective for

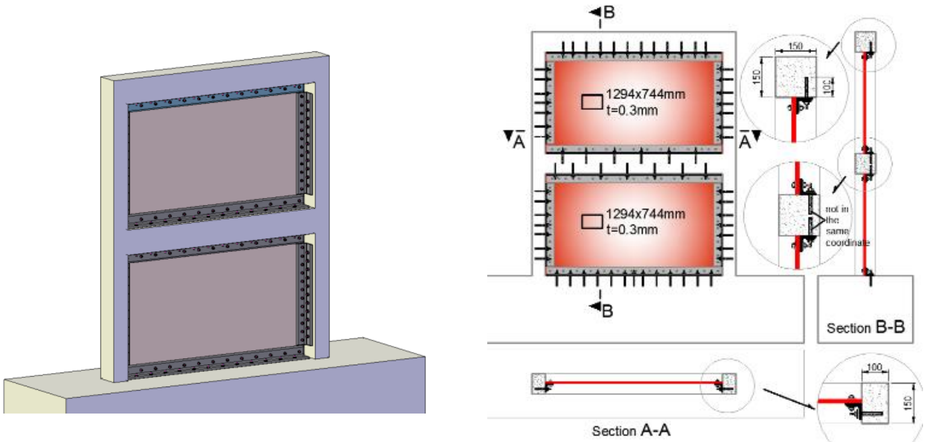


Fig. 4 Test specimen with internal steel plate

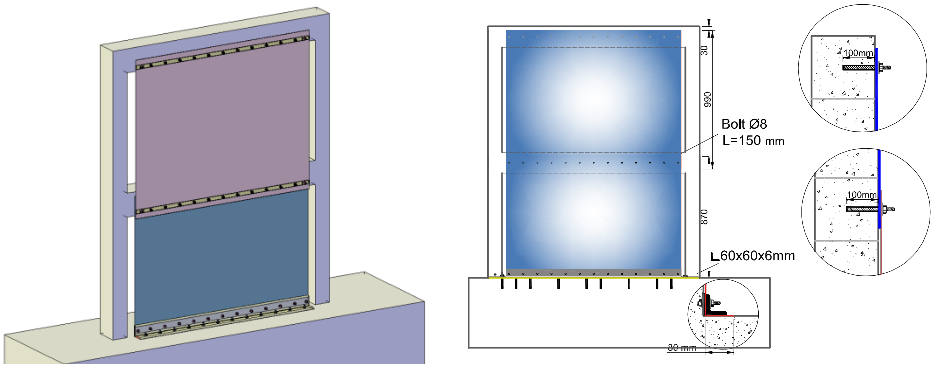


Fig. 5 Test specimen in which steel plate were anchoraged to the beams only

using X-shaped steel plates was to reduce the amount of steel material, the number of attachment points and, consequently reduce the labor costs. Each X-bracing was 300 mm wide and 0.3 mm thick.

The test specimen represents an external frame in the model building. Windows are frequently located in the exterior walls of typical buildings. To investigate the effect of wall opening on the structural response, the fourth specimen (SPS4-Window) contained an opening at mid-length of the steel plate near the top (Fig. 3(b)). The height and width of each window opening was 400 mm and 350 mm, respectively. To prevent a local failure on the perimeter of the window opening due to potential stress concentration, a 100 mm wide and 0.6 mm thick plate was bolted around the windows.

It can be assumed that a full-scale building frame will have a typical bay or opening approximately 5000 mm long and 3000 mm tall. A typical commercial steel plate with 2500 mm×1300 mm dimensions is not large enough to cover or enclose one bay or opening. Therefore, for a full-scale frame, the designer may need to use four or more steel plates to cover one frame opening in one story. Consequently, in the fifth test specimen (SPS5-Four pieces) four steel plates were connected to enclose external surface of one frame opening. The steel plate pieces were attached to each other with 6 mm diameter bolts at a uniform 50 mm interval as shown in Fig. 3(c).

In the sixth specimen, the steel plates were perforated to create several different size circles. This created an aesthetically pleasing appearance while increasing the lateral strength and stiffness of the frame. One major reason for using the perforated plates was to improve the appearance of the external façade of the structure. It was likely that the circular holes on the perforated plates would decrease the plate's strength as compared to solid steel plate, i.e., the second specimen or SPS2-Full. In the specimen with perforated plates, there is a possibility for the plates to fail before the columns fail. This specimen was named SPS6-Perforated and is schematically illustrated in Fig. 3(d).

In the seventh specimen (SPS7-Inner plate) the steel plates were installed inside the frame instead of the outer face of the frame or façade. The plates were restrained along each of the four edges to create a tension field action inside the plate and to transfer forces to the surrounding beams and columns. The bottom story plate was also anchored to the foundation as in other specimens. Along the edges of the plates, 60 mm×60 mm×6 mm steel angle profiles were used to connect the steel plates to the surrounding RC frame. Steel anchor bolts passing through the steel angles were installed on the RC members (Fig. 4). The anchoring holes for the dowels were 50 mm deep and 8 mm in diameter. In the corners or near the beam-column joints, four holes were drilled at 50 mm intervals. The uniform spacing of the other holes was 100 mm.

For the last specimen, the steel plates were attached only to the beams. The tension field forces were transferred to beams only (Fig. 5). By doing this, the shear failures of

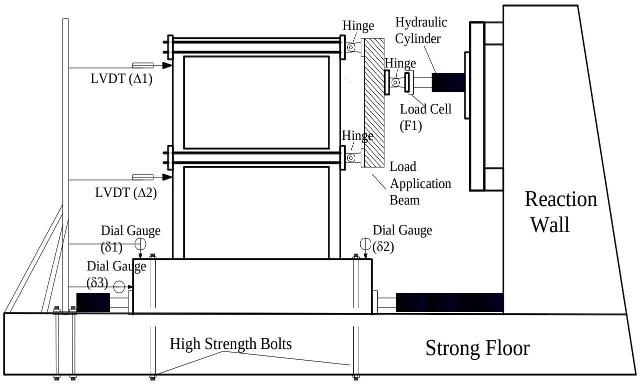


Fig. 6 Representation of the instrumentation scheme

columns were aimed to be prevented.

In this study, the thickness of all steel plates was chosen as 0.3 mm. This relatively low thickness was chosen to allow for shear buckling at large displacements and to reduce the force demand on the adjacent members, resulting in more efficient retrofit design.

2.2 Test setup

A horizontal, reversed cyclic loading was applied to the specimens to represent seismic loads. A hydraulic jack with a 1000 kN capacity was attached horizontally to a rigid reaction wall. One third and two thirds of the total horizontal load were applied to the first and top story of the test specimens, respectively (Fig. 6). A rigid steel beam was used to distribute the loads accordingly. At the load application points at each story level, hinges were provided to allow for rotational movement of the beam-column joints. Test specimens were strongly fixed to the rigid floor of the laboratory using steel rods.

A steel instrumentation frame was used to measure global displacements of the specimens. Dial gauges and LVDTs were installed between the instrumentation frame and specimens. Although the specimens were fixed to the floor of the laboratory, movement of the specimen foundation relative to the strong floor was also monitored using dial gauges. A data acquisition system was used to record the applied load and displacements during the experiments.

2.3 Test program

To simulate cyclic earthquake loading, the specimens were subjected to a cyclic load history with gradually increasing loads or displacement amplitudes. The loading history started with load controlled cycles and continued until the specimen reached the 90% of pre-determined yield capacity. In the load controlled cycles, the load increment in each step was 10 kN. Following the yield point, displacement controlled cycles were applied. The displacement increment between successive cycles was 5 mm. After reaching 30mm top displacement, the 10 mm increments were used. The tests were terminated if a major failure was observed in the frame. The maximum displacement capacity of the loading setup was 70 mm.

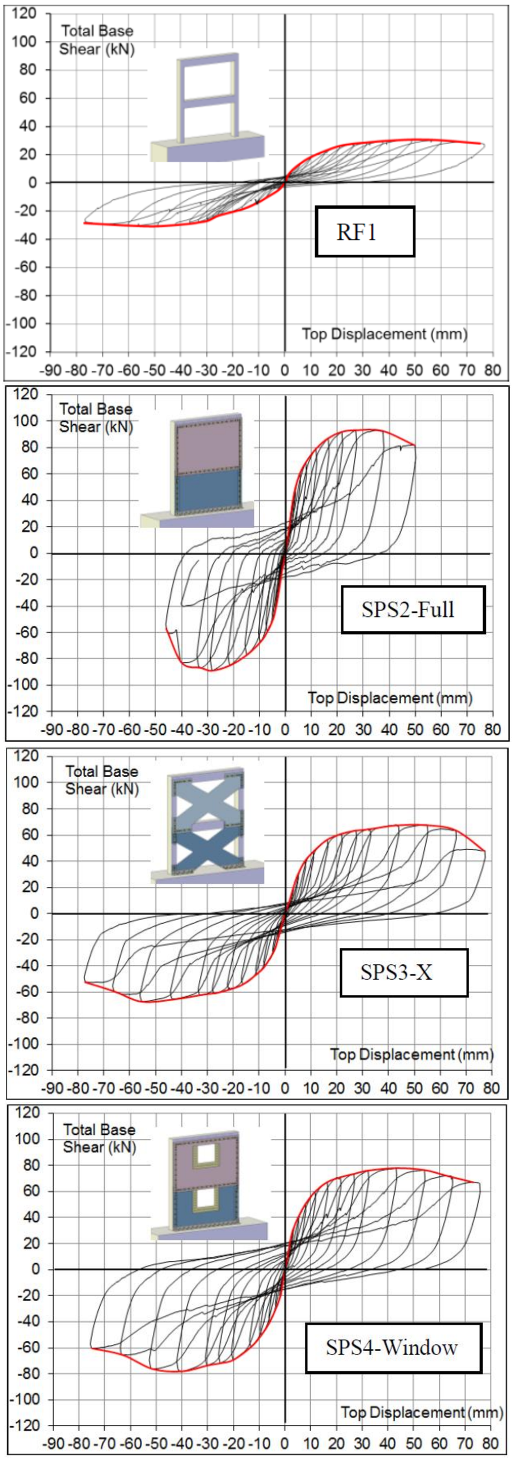


Fig. 7 Measured total lateral load versus displacements (reference specimen and first 3 specimen)

3. Test results

The measured total lateral load or base shear versus top displacement relations for each test specimen were shown in Fig. 7 and Fig. 8. The envelope for the hysteretic response of each specimen is also shown in the Figures (in red). The stiffness, strength, deformation capacity and overall observed behavior of each specimen were discussed

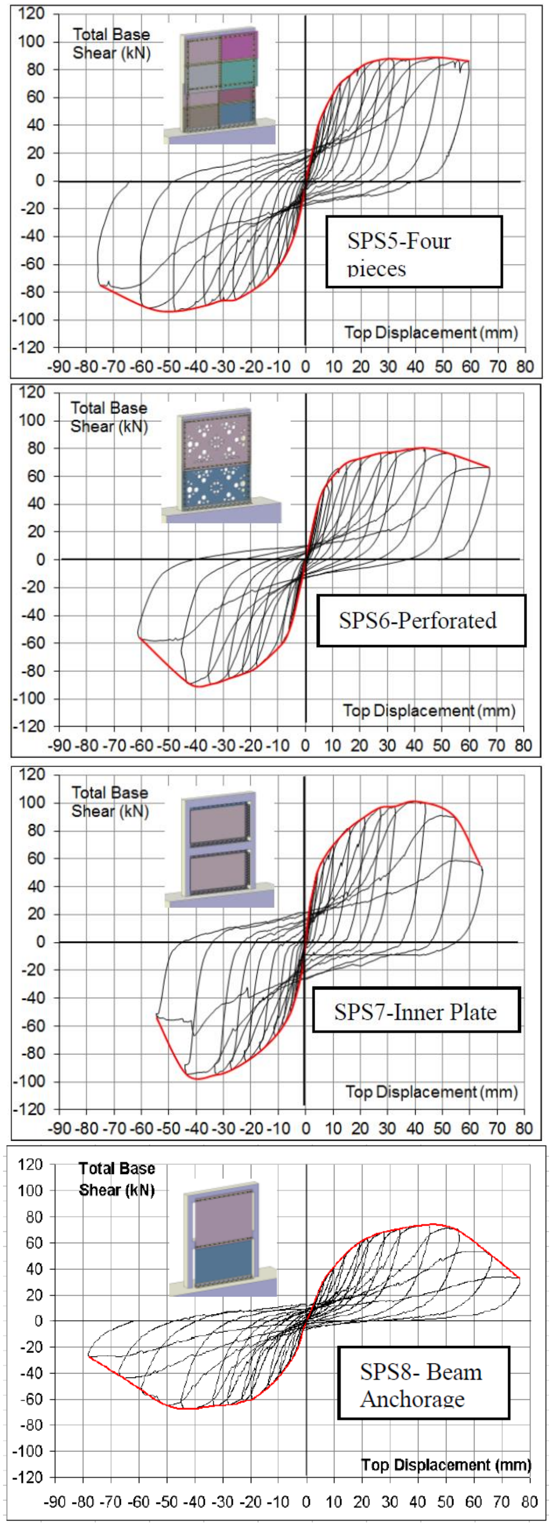


Fig. 8 Measured total lateral load versus displacements (last 4 specimen)

below.

The first specimen, RF1 was tested to observe the behavior of bare RC frame (Korkmaz 2016). The goal was to compare the measured response of RF1 with those of the retrofitted specimens. This specimen resisted a maximum lateral load of 29 kN and 37 kN in the forward (positive) and backward (negative) directions, respectively. The

Fig. 9 Overall appearence of specimen RF1 at the end of the test

Fig. 10 Specimen RF1 and observed cracks and damage at the end of the test

corresponding top displacements were 44 and 41 mm for the forward backward cycles, respectively. The relative story drifts corresponding to the ultimate load levels, were approximately 2% and 3% for the first and second stories, respectively. The larger drift measured in the first story indicates that the damage was concentrated more in the first story. In fact, the reference specimen (RF1) lost a significant part of its lateral load carrying capacity and failed after formation of plastic hinges in the first story columns. The final failure was dominated by combined flexural and shear damage both in columns and beam-column joints (Fig. 9).

During the final loading cycles, a sudden drop (up to approximately 31% reduction) in the load carrying capacity was observed in both directions. During these cycles, i.e., the cycles before the last cycle, the top displacement of the specimen was 67 mm. The corresponding maximum relative story drifts during this cycle were 2.8% and 5.4% in the second and first stories, respectively. Figs. 9-10 shows the specimen at the end of the test, including the crack pattern on each side of frame. As discussed above, the cracking and damage was mostly concentrated in the first story while less damage and cracks were observed in the second story.

Other than the reference specimen RF1, all specimens were strengthened by the SPSWs. The damaged and deformed test specimens at the conclusion of the experiments were shown in Figs. 11-17. The photos on the left hand side show the frontal view or the outer façade of the test frames.

As can be observed from Figs. 11 through 17, the steel plates typically buckled and inclined (approximately 45^{o}

Fig. 11 View of specimen SPS2-Full at the end of the test

Fig. 12 View of specimen SPS3-X at the end of the test

Fig. 13 View of specimen SPS4-Window at the end of the test

with the horizontal) tension field action was developed during the load reversals. No serious separation was observed between the RC frame members and the SPSW's in tests. This indicated that the detailing and the construction of the steel dowels were adequate. An approximately three-fold increase was observed in the lateral load carrying capacity of the strengthened frames as compared to the bare RC frame, RF1. This additional lateral load had to be transferred to the columns and, subsequently, to the foundation through the tension field forces of the steel plates. All tests were concluded when shear failure was observed in the first story columns. The diagonal or tension field forces in the steel plates were transferred to the columns as shear forces and, in turn, column failures were observed. As can be seen in Figs. 7-8, the proposed retrofit schemes improved the cyclic response of the specimens and

Fig. 14 View of specimen SPS5-Four Pieces at the end of the test

Fig. 15 View of specimenSPS6-Perforated at the end of the test

Fig. 16 View of specimen SPS7-Inner Plate at the end of the test

resulted in stable hysteretic loops.

In the specimens SPS3-X and SPS4-Window, the steel plates were fractured and torn around the sharp corners of the openings. Similar plate tearing was also observed around the circular perforations of the specimen SPS6-Perforated near the end of the experiment.

To evaluate the efficiency of the applied strengthening methods, the strength and stiffness characteristics of the test specimens were discussed with the help of response envelope curves (red lines in Figs. 7-8). The maximum strength values in each hysteretic loop were connected to draw the response envelope curves. These curves were shown in Fig. 18 to compare the strength and stiffness characteristics and overall response of the specimens.

The absolute maximum lateral loads measured during the forward and backward loading cycles as well as the corresponding top beam displacements were provided in Table 1. The ratios of the ultimate strength capacities of the specimens relative to that of the reference frame RF1 were also provided. To determine quantify the yield load and the displacement, both the forward and backward half-cycles were idealised as bi-linear curves. An iterative procedure

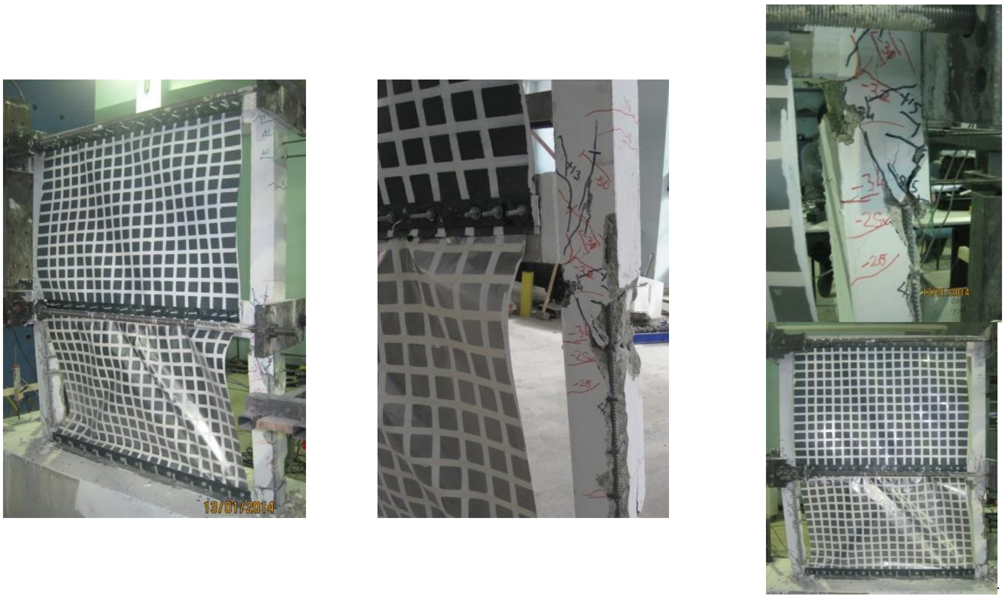


Fig. 17 View of specimen SPS8-Only Beam Anchorage at the end of the tests

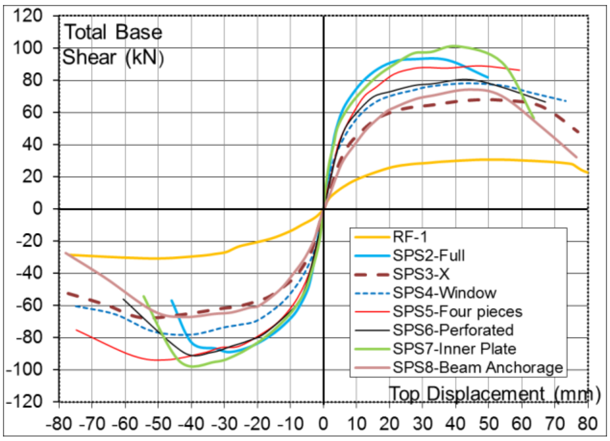


Fig. 18 Measured lateral load-displacement envelope curves for all specimens

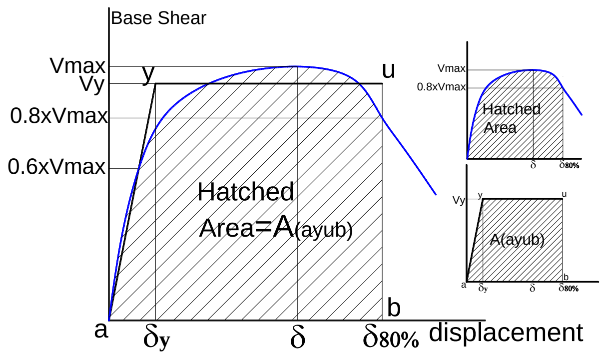


Fig. 19 Bi-linear approximation of the envelope curves

based on an equal area concept was used as shown in Fig. 19. The maximum base shear carried by the system was designated as "V_{max}" (column #1 in the table) and corresponding top displacement was δ (column #2). The elastic portion of the bi-linear curve was assumed to pass through the point at "$0.6\times V_{max}$". The ultimate displacement ($\delta_{80\%}$-column #3) point was defined as the displacement level corresponding to a 20% decrease in the ultimate lateral load capacity ($0.8\times V_{max}$). The horizontal portion of the curve was assumed to end at a point of $\delta_{80\%}$. The area under the experimental curve was calculated up to the ultimate point ($\delta_{80\%}$). The yield load (F_y-column # 4) of the bi-linear curve (horizontal portion) was leveled to result in approximately equal gained and lost areas with respect to the original curve up to the ultimate point. The displacement value at the intersection of the linear and horizontal portion was the yield displacement, δ_y (column #5) (Dimova and Negro 2005). The ratio between the estimated values of $\delta_{80\%}$ and δ_y was defined as the displacement ductility ($\lambda_d=\delta_{80\%}/\delta_y$) (column #6). Ductility is an important concept as far as earthquake behaviour is

Table 1 Comparison of the calculated values

	Forward (+) Half Cycles						
	V_{max} (kN)	δ (mm)	$\delta_{80\%}$ (mm)	F_y (kN)	δ_y (mm)	$\frac{\delta_{80\%}}{\delta_y}$	$\frac{V_{max}}{RF\ V_{max}}$
	1	2	3	4	5	6	7
RF1	30.7	44.2	69.7	28.4	18.6	3.74	1
SPS2-Full	93.1	27.1	58.0	85.8	7.4	7.85	3.1
SPS3-X	67.5	54.3	72.7	63.2	12.4	5.86	2.3
SPS4-Window	78.0	43.2	82.6	70.6	7.7	10.69	2.6
SPS5-Four Pieces	89.0	47.7	59.3	85.2	11.7	5.07	3.0
SPS6-Perforated	80.6	43.1	70.2	73.7	9.5	7.42	2.7
SPS7-Inner Plate	101.1	40.8	56.8	92.8	10.4	5.48	3.4
SPS8-Beam Anch.	74.3	43.9	60.6	68.8	17.2	3.52	2.5

concerned. Generally, ductility is defined as the ability of the structure to sustain deformations beyond the elastic limit without significant strength degradation (Akin 2011). The computed displacement ductility values were listed in Table 1. Finally in column #7, the ratio of total base shear of each specimen (V_{max}) to the referans specimens base shear

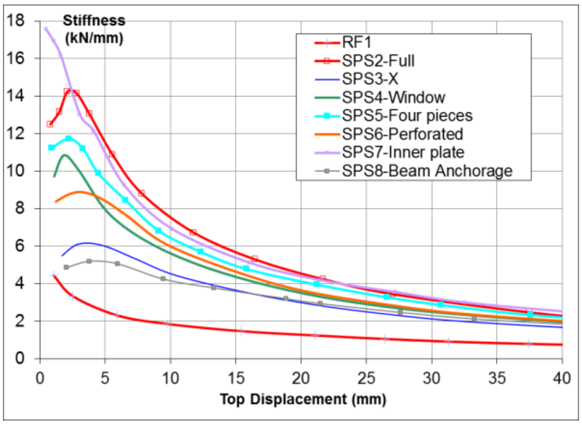


Fig. 20 Stiffness values based on the secant stiffness of the cycles of each displacement increment

capacity (V_{max}_RF) is given.

With the inclusion of the SPSWs on the reinforced concrete frame, the ultimate load carrying capacity of the system was significantly increased. The highest improvement was achieved in specimen **SPS7-Inner plate** (3.4 times in the forward half-cycles) and in specimen **SPS2-Full** (3.1 times in the forward half-cycles). From Fig. 17 and Table 1, it can be observed that the strength of specimen **SPS2-Full** was close to that of specimen **SPS5-Four pieces** because both specimens completely enclosed the external façade of the specimens. The number of steel plate segments did not affect the capacity and behaviour of the composite system. It does not make any difference if one complete steel plate or four or more attached plates were used. On the other hand, specimens with a window opening or perforations displayed relatively low ultimate strengths with respect to the other strengthened specimens. The capacity increase was limited to an increase on the order of 2.6-2.7 times with respect to the reference bare frame. The lowest capacity gain was achieved in the specimen with X-type tension bracings. The increase in the ultimate capacity was 2.3 times. If the steel plates were attached only to the beams, than the lateral load carrying capacity increase remained limited to 2.5 times.

The ultimate displacement values were obtained at a point where the maximum load carried decreased by 20%. At this load level, the displacement values of the first group specimens (**SPS2-Full**, **SPS5-Four pieces**, and **SPS7-Inner plate**) were similar to each other. The second group specimens with openings or X-type steel plates displayed relatively high ultimate displacement values. The greatest displacement capacity was achieved in the specimen with the window opening. The reason of this behavior can be explained as; the capacity increase was high in the first group and therefore relatively high tension action forces were transferred to the columns and earlier or primitive failures limit the displacement capacities.

The lateral stiffness of the structural system can also be used to evaluate its seismic performance. The stiffness properties of the tested frames were evaluated using their secant stiffness, defined at the peak of each cycle, at each deformation amplitude. The lateral stiffness degradation of the specimens as a result of the cyclic loading was compared. The stiffness values of the specimens were estimated from the forward hysteresis curves and were illustrated in Fig. 20 with respect to the top horizontal displacement. From this curve, the decay in the stiffness can be clearly observed and compared.

Applied SPSWs significantly improved the lateral stiffness of the test specimens. The greatest initial stiffness value was achieved in the specimen **SPS7-Inner plate**. This specimen exhibited high initial stiffness, and during the following cycles, the stiffness of the frame followed a decreasing trend. On the other hand, the other specimens with external SPSWs had stiffness values in the second or third cycles that were higher than the stiffness values in the first cycle. This is because the stretching of the plates could not be performed properly or could not be conserved during the drilling and fixing operations that occurred during the installation of the external plates. On the other hand, the dimensions of the inner plate were smaller than the frame's inner dimensions. During the fixation of the inner plate, the workers had to apply an extra tensional effect on the plate to fit it to the frame opening.

The stiffness performance of specimen **SPS8-Beam Anchorage**, was found to be worst among all the strengthened specimens.

The overall stiffness values of the specimens lacking an opening were very similar to each other. The specimens with window openings and perforations displayed relatively low stiffness values with respect to the full steel plate specimens, as expected. The stiffness of all the strengthened specimens gradually decreased without any abrupt changes. Prior to failure, the stiffness of the composite specimens were of the same order of magnitude as the other specimens.

4. General behavior of test specimens

During testing of composite systems, with SPSW and RC frame, stable hysteresis curves were obtained. Well-defined elastoplastic load deformation envelopes were found in every test. A significant number of plastic folds formed and were distributed over the entirety of the panels. All the specimens failed after a significant inelastic tension field action occurred with a large storey drift deformation. Local buckling and tension field actions in the steel plates were developed in all stories as shown in Fig. 11 through Fig. 17. The damage in the first storey is more pronounced than in the second storey. A high initial stiffness was evident in every test. Small fractures were found at the panel corners with window openings and X-shaped steel plates at the conclusion of the tests.

The lateral load carried by the systems was increased considerably during the tests. These additional lateral forces were transferred to the columns by tensional forces in the panels. The anchorage of the tension field force of the steel plates caused the formation of vertical concrete cracks, and additional shear forces in the columns resulted in shear failures; subsequently, the load-carrying capacity of the system decreased abruptly. The out-of-plane deformations of the plates due to significant amounts of buckling was observed.

5. Conclusions

This paper presents the results of an experimental study that attempted to determine the performance and stiffness improvements of deficient reinforced concrete frames. The main aim of this study is that, the applied strengthening must be performed externally to minimize the disturbance to the owners of the structure. The external application also decreases the cost involved in strengthening the structure while simultaneously minimising the construction. The application of thin, unstiffened steel plate shear walls have been used in steel structures to increase the lateral rigidity of the frame. The application of SPSWs to reinforced concrete systems are limited and more experimental data are needed. Based on the limited number of tests in this study, we can summarise our conclusions.

In the composite structures that endowed the steel plate shear wall and reinforced the concrete bare frame, a high degree of rigidity was secured, and an effective energy absorption mechanism was formed. The behaviour was significantly improved relative to the reference specimen. The ultimate base shears in the frames increased considerably. Although the behaviour of specimen SPS3-X was inferior relative to the other composite specimens, the ultimate base shear was increased by 230% relative to the capacity of the reference bare frame. The maximum improvement was obtained in specimens with SPSWs affixed to the interior (an increase of 340%). If the steel plate has a window opening or perforations, the maximum lateral load carried by the system remained limited (having an increase of 260%). On the other hand, steel plates with openings displayed higher ultimate displacement capacities. The specimen with a window opening survived the highest lateral displacement of the specimens. The perforations in the sixth specimen decreased the strength capacity but increased the ultimate displacement capacity. The perforations also allow for an aesthetically acceptable appearance. Generally, the appearance of the structure after the strengthening application is ignored. The strengthening and disturbance of architectural formations can create problems. The external strengthening and architectural remedies may become a popular direction for future studies.

The steel plate shear wall systems also increased the lateral stiffness of the frames. The greatest improvement was achieved in the specimen with inner SPSWs. The fixation of the plates is very important for the initial stiffness performance.

In composite systems using steel plate shear walls and reinforced concrete frames that had elastic-plastic restoring force characteristics, stable hysteresis curves were obtained, and an effective energy absorption mechanism was achieved.

A slippage or debonding of the anchorages was not observed in any test. On the other hand, the anchorage of the tension field force of the steel plates caused the formation of shear cracks that may result in an unexpected column failure. As a result, to benefit from the tensional capacity of steel plates, the columns must have enough shear capacity to resist the forces transmitted from the anchorage rods. The application of SPSWs must be limited to structures with an average concrete quality (20 MPa concrete compressive strength). The success of the anchorage rods and the shear capacity of the columns mainly depends on the concrete strength.

This method generally does not necessitate relocating the occupants; hence, it is appropriate for residential buildings. On the other hand, the external face of the structure must not include overhangs.

In this proposed strengthening method, to enable the infill plates to form the tension field actions, the boundary frame has to be strong and stiff enough to anchor the steel plates. Although steel plate shear walls strengthen the reinforced concrete frames, the strength demand for the columns, beam-column joints are significiantly increased. The failure of boundary frame should not occur before the formation of the tension field action. During the preliminary analytical modelling of the frame, if the shear resistance of the columns, beams or joints are found to be inadequate to carry additional shear forces due to tension field actions, then, additonal strengthening of RC members may be needed. RC jacketing of columns can be applied before the steel plate shear wall fixations. Also, if it will be adequate, steel jacketing of columns or beams with L type steel profiles and steel strips can be more easier way of strengthening. Besides the columns or beams, strengthening of available footing of the building may be required to transfer additional loads to the ground. At this point, preliminary analytical analysis is required to decide the additional strengthening.

The experimental components of this study have shown that composite systems with steel plate shear walls and reinforced concrete frames exhibit many desirable characteristics for structures in areas of high seismic risk. Further studies on this subject are required.

Acknowledgments

The authors wish to thank Dr. Yunus Dere, Dr. Serra Z. Korkmaz and Dr. Halil Sezen, from Ohio State University-USA, for their help during the preparation of the paper. The experiments in this study were supported by Selcuk University Scientific Research Office (BAP) with the support numbers: SU-BAP-11101012 (reference bare frame) and SU-BAP-15201025.

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