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Experimental investigation of composite shear walls under shear loadings

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ABSTRACT

One of the efficient methods for improving the seismic behaviour of high-rise buildings is using Composite Steel Plate Shear Wall (CSPSW). In this paper, extensive experimental studies of one and three-story CSPSWs with the scale of 1:3 and 1:4, together with stress equations of each element are reported. The experimental results indicate that this system has reliable behaviour if the columns have high bending stiffness. Also bolts spacing to plate thickness ratio has direct relationship with system ductility. However, plate yield load has an inverse relationship with this ratio. In this system, plate stiffening requirement is obtained with minimum reinforcement for reinforced concrete, though for damage prevention high strength concrete is preferred. Also, the results show a good agreement for the recommended values of (b/t) by an AISC code for preventing plate buckling.

1. Introduction

Composite steel plate shear wall (CSPSW) is a lateral loading resistance system, which is adopted and used especially in high-rise buildings. This system is similar to stiffened steel plate shear wall (SSPSW); in which reinforced concrete replaces the steel stiffeners. The CSPSW is composed of a steel plate connected to a reinforced concrete (RC) panel on one side or both sides by mechanical shear connectors such as bolts. The main role of an RC panel is prevention of the early buckling of steel plate shear wall (SPSW). Integration of an RC panel with steel plate wall removes the weaknesses of each system in tension or compression, if used separately. Some advantages of using CSPSW in comparison with traditional RC shear wall (RCSW) and SPSW are [1]:

1. The CSPSW is thinner and lighter in comparison with the RCSW with the same shear capacity.
2. The CSPSW's weight reduction significantly reduces the loads applied to the columns and the foundation system.
3. The smaller footprint of CSPSW is very favourable from the architectural point of view, providing more useable floor space.
4. The initial imperfection of steel plate significantly reduces, because by using bolts, instead of steel plate, for stiffening the SPSW. In addition, its residual stress is less than SSPSW.
5. Its capacity and stiffness are more than those of SPSW with similar steel plate thickness, which are very useful for tall buildings.
6. CSPSW is preferred to an SPSW for resisting against moderate and more frequent earthquakes. That is because no buckling or cracking happens in its components, especially in the CSPSW with gap.

Substantial numerical and experimental researches have been conducted in recent decades for studying the seismic behaviour of an SPSW. Based on these studies, this type of shear wall is used in many buildings [1]. However, studies on CSPSW are limited. At first, Astaneh-asl et al. [2] introduced and tested two types of CSPSW under cyclic loading (innovative and traditional). In the innovative CSPSW, there was a gap between the reinforced concrete wall and the boundary frame, i.e. beams and columns. They showed that both specimens had ductile performance up to the angle drift of 0.05.

In the innovative system, damage to the concrete wall under relatively large cycles was much less than the damage to the concrete wall in a traditional system. Rahai and Hatami [3] performed analytical and experimental tests on the one-story CSPSW with different parameters. They concluded that increasing the shear stud spacing reduces the slope of load–displacement curve and improves ductility up to specific stud spacing, beyond which there is no change. In addition, middle beam rigidity and beam to column connections have no significant impact on the behaviour of CSPSW walls. The CAN/CSA-S16-01 code [4] approves this seismic resisting system and AISC-2005 code [5] provides the necessary instructions for the design and analysis of such walls.

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The main function of CSPSW is to resist horizontal story shear and overturning moments caused by lateral loads. However, there is no guideline for the design of CSPSW under the combination of shear and bending. More research is necessary for better understanding the local and global behaviours of CSPSW, and also its structural behaviour, when high strength RC panels are connected to infill steel plates for their lateral stiffening.

Furthermore, extensive experimental and analytical results have been reported on behaviour of double skin composite construction [6–8]. This product is called Bi-steel. The concept has been proved to be useful for places to have no limitation on using an RC heavily for the construction, or in the places, which internal hydrostatic pressure due to fresh concrete is the major concern.

In this paper, experimental studies on one- and three-stories CSPSW specimens are performed and some test results are discussed. The one-story specimens were subjected to pure shear, while the three-story specimens were under combination of shear and bending, due to shear in the uppermost-story. In addition, for some of the one-story tests, only the buckling load or the yielding load was computed. Initial stiffness, yield, ultimate displacement, ultimate displacement equivalent forces, total energy dissipation and design of CSPSW fundamental components were objectives of this study. Bolts, gap and the RC panel effects were considered as well.

2. Test programme

The test programme conducted consisted of two phases: eight and seven specimens were tested in the first (I) and second (II) phases, respectively. The specimens in phase-I included five-CSPSW specimens with pinned beam-column connections, one specimen with rigid moment frame, one specimen with SPSW and the last one with CSPSW with fixed beam–column connections. All of the specimens in phase-I were built with 1:4 scale, Fig. 1a. The specimens in phase-II included four 1:3 scale single-bay three-story and 1:4 scale single-bay one-story frames. All of the specimens in phase-II were CSPSW, Fig. 1. Before testing, all models were analysed by push-over analysis to evaluate yield-displacement and the gap size around the RC panel. The gap size was computed so that no interaction happens between the RC panel and the boundary frame.

3. Phase-I: description of the specimens and testing procedure

3.1. Specimens and tests setup

The phase-I five specimens were coded as "HC1"–"HC5" (HC is an abbreviation for Hinged-CSPSW). The primary test parameters in the hinged specimens were the number of bolts, number of RC panels (at one side or at both sides of the steel plate) and reinforcement ratio. The other three specimens with rigid beam-column connections were tagged as "S", "CS" and "F". (CS, S and F stand for CSPSW, SPSW and Frame, respectively). In order to compare their seismic behaviour and use of their results for the phase-II tests, one SPSW and steel frame without infill plate with the same size, were constructed and tested. The specimens and their properties are presented in Table 1. Furthermore, a schematic view of the experimental setup can be seen in Fig. 2. The main components of the tests setup included: reaction frame, H-shape beam, lateral bracing, actuators and specimen. In order to secure the specimen to the strong floor during the test, it was attached to a strong H-shape beam, which was bolted to the strong floor by high strength bolts.

The beam and column were connected together directly by the groove welding. The steel plate was welded continuously to the fish plate, which in turn was welded continuously to the boundary frame. Creating holes in the bolt places on the plate were done

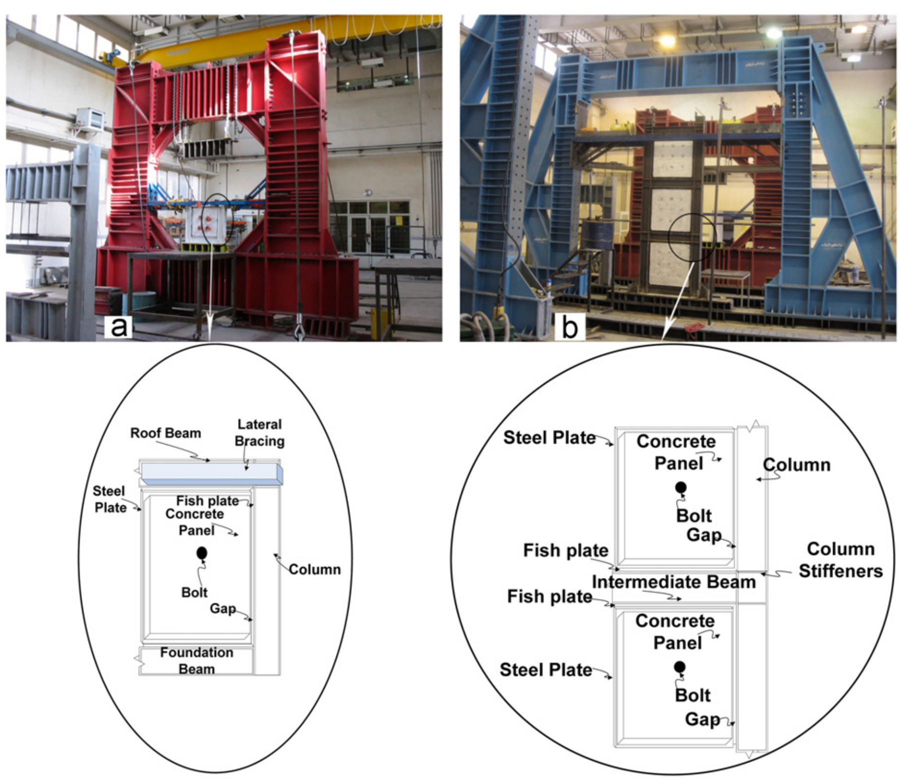

Fig. 1. (a) Setup experimental of one-story specimens and (b) setup experimental of three-story specimens.

Table 1
Properties and dimensions of the specimens in phase-I.

Specimens	*HC*1	*HC*2	*HC*3	*HC*4	*HC*5	*CS*	*S*	*F*
Columns (mm)	IPE160	IPE160	IPE160	IPE160	IPE160	2IPE100+2Pl100 × 5	2IPE100+2Pl100 × 5	2IPE100+2Pl100 × 5
Foundation beam (mm)	IPE160	IPE160	IPE160	IPE160	IPE160	2IPE100	2IPE100	2IPE100
Roof beam (mm)	IPE160	IPE160	IPE160	IPE160	IPE160	2IPE100	2IPE100	2IPE100
Steel wall plate thickness (mm)	2	2	2	2	2	2	2	–
Fish plate (mm)	–	–	–	–	–	40 × 5	40 × 5	–
Number of bolts	4	4	4	5	1	4	–	–
Bolt diameter (mm)	6	6	6	6	6	20	–	–
Type of bolt (F_u (ton/cm^2))	12.9	12.9	12.9	12.9	12.9	12.9	–	–
Rebar diameter (mm)	3	3	3	3	3	3	–	–
Reinforcement ratio	1%	1%	1.5%	1%	1%	1%	–	–
Concrete thickness (mm)	30 (both sides)	30	30	30	30	30	–	–
Free space around concrete (gap) (mm)	20	20	20	20	20	11.25	–	–

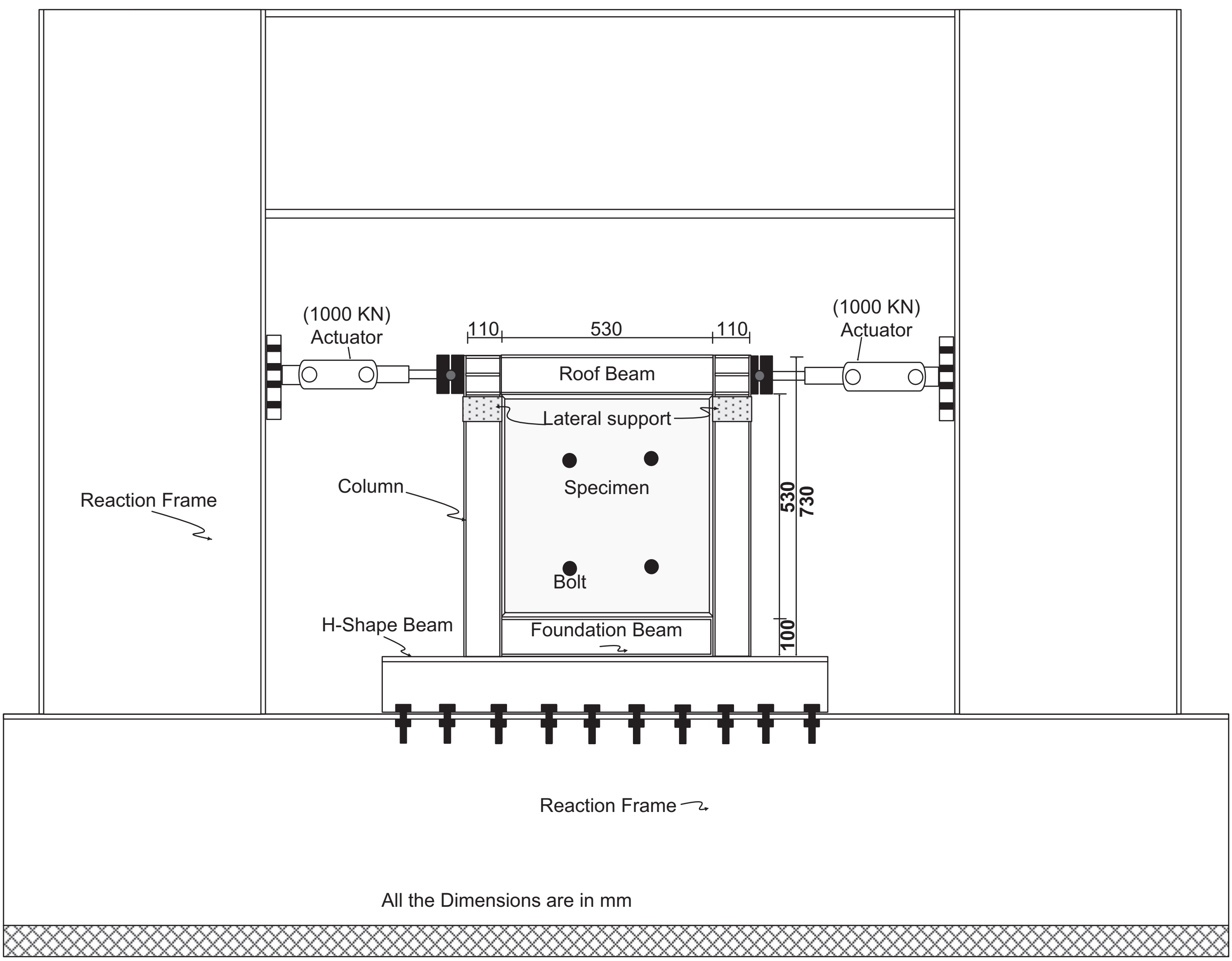

Fig. 2. General setup of the one-story tests.

by a drill. Bolts with enough length were used for all the tests. After the steel components were connected together, the bolts and reinforcing bars were put at their corresponding places. Then, the concrete panel, which was produced by fine aggregate material, was casted on the steel plate. The steel plate also plays the role of a formwork for the concrete wall in the time of concrete casting. There was a gap between the RC panel and the boundary frame in the CS specimen.

To connect the boundary frame to each other in the five-hinged specimens, half-flange of an IPE160 was cut-out at the end of the columns. Then, high strength bolts connected them to each other, Fig. 3. For avoidance of sudden tearing of the steel plate at the corners, small triangular steel plates were welded to it.

In the specimens with fixed-connection, cyclic deformations with increasing amplitude were applied to determine the cumulative energy dissipation. Determination of ductility, maximum

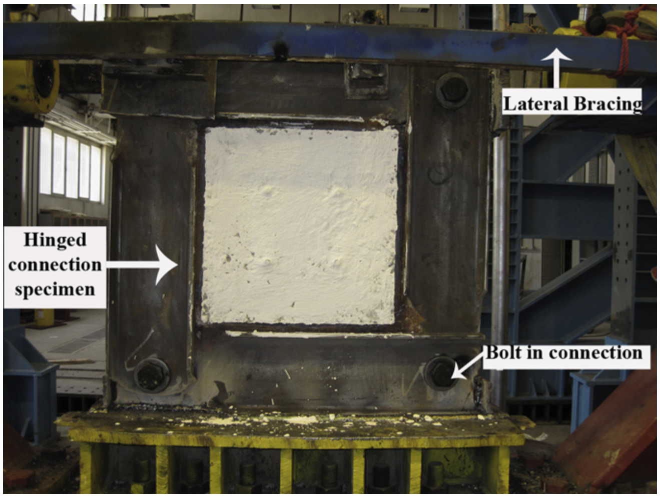

Fig. 3. Hinged-connection specimen with four bolts (*HC*2).

Table 2
Steel properties.

Section type	Yield stress F_y (MPa)	Ultimate strength F_u (MPa)
IPE160 column flange	361	510
IPE160 column web	344	481
IPE140 beam flange	352	501
IPE140 beam web	338	496
IPE100 beam flange	308	479
IPE100 beam web	285	446
Fish plate	297	406
Steel plate with 2 mm thickness	268	415
Steel plate with 3 mm thickness	281	416

strength and ultimate failure mode were also among the other objectives of this work. This also gives the general idea about the methodology and design of the phase-II tests. The main objective of testing the hinged-connection specimens was the determination of buckling and yield loads. Nevertheless, loading was continued up to the ultimate failure point.

3.2. Materials

The columns and beams were made up of ASTM A572/50 steel. The steel plates were ASTM A36 steel. The material properties of each steel member, used in the tests, are reported in Table 2. The bolts were of A490 high strength bolt type. The nominal ultimate tensile strength and the yield stress of the bolts were 12 and 10.8 ton/cm^2, respectively.

A series of standard 28-day concrete cylinders and cubes were made and tested to determine the concrete properties. Tension tests were performed to establish the monotonic stress–strain relationship of the vertical and horizontal reinforcements. The horizontal and vertical reinforcement ratios of the specimens were equal. The material properties of the concrete and the reinforcement bars used in the experimental tests are presented in Table 3.

3.3. Loading history

Before testing, the yield-displacement at the top of the specimens with rigid-connection was predicted by the finite element (FE) method (Table 4). Loading was applied from a 1000 kN actuator to the top of the specimen through rigid plates in the line of application, which passes through the roof beam centre. No vertical load was applied to the specimens. To prevent out-of-plane displacement, the specimens were braced laterally in the top (Fig. 3). The specimens with rigid-connections were tested using a cyclic quasi-static loading protocol according to an ATC-24 [9]. The test was terminated when the system could not sustain more load and failed. However, the hinged-connection specimens were tested using a monotonic loading in one direction. The loading was ramped linearly from a zero value to a collapse load.

A number of linear variable displacement transducers (LVDTs) and strain gauges were put on the specimens to measure global and local responses at the points of interest. During the tests, the slippage of H-shape beam was also monitored.

Table 3
Reinforced concrete properties.

Material		Property	Value (MPa)
Concrete	Phase-I	Cylinder compressive strength, f_c'	43
		Cube compressive strength, f_{cu}	47
	Phase-II	Cylinder compressive strength, f_c'	72.5
		Cube compressive strength, f_{cu}	79
Reinforcing bar	One-story	Yield stress, f_y	336
		Ultimate strength, f_u	492
		Young's modulus, E_S	20.3e4
	Three-story	Yield stress, f_y	365
		Ultimate strength, f_u	523
		Young's modulus, E_S	19.3e4

3.4. Test results

The specimens' height was short, so they were under pure shear approximately. The important factors including shear strength, yield and ultimate displacement, initial stiffness and ductility are given in Table 4. It is obvious that addition of steel plate to the rigid frame improves all of the mentioned seismic parameters. The stiffness of CSPSW was higher than that of SPSW, because of the RC panel, especially before the plate yielding. Also CSPSW' maximum shear strength and to some extent its ductility, were higher than those of the SPSW. As a result, the steel plate stiffened by the RC panel (CSPSW) showed by far a better behaviour in comparison to the steel plate lonely (SPSW). The applied loads versus the lateral deflection hysteresis for all the rigid-connection specimens in phase-I are shown in Fig. 4. During the cyclic loading, severe pinching occurred in the frame diagram (*F* specimen). Total energy dissipation of composite shear wall (CSW) was about 25% higher than that of the steel shear wall (Table 4). This large difference is due to the existence of the RC panel that prevents plate buckling. In general, appropriate spacing (small (b/t)) of the bolts provided conditions for steel plate yield in the CSW before its elastic buckling. In addition to plate yielding before buckling, CSW provided conditions (appropriate composition of the components) for column yield at the base before local buckling. The test results emphasise this fact that the ratio of column stiffness to plate stiffness is very important for favourable ductility. In cyclic loadings, two perpendicular waves were formed in the steel plate. Tearing of the plate initiated at an intersection of the waves. The waves formation on both sides and the tearing of the plate at some points near the fish plate and corners are illustrated in Fig. 5. Waves are formed between the bolts; while tearing of plate occurs near the bolts in CSW.

For the specimens with hinged-connections, local buckling behaviour of the steel plate resting on the tensionless rigid

Table 4
Behavioural characteristics of the tested specimens in Phase-I.

Specimen	Yield point			Maximum		Ductility	Total energy dissipation (kN m)	FE estimation
	P_y (kN)	δ_y (mm)	Initial stiffness (kN/mm)	$P_{\max}$ (kN)	$\delta_{\max}$ (mm)	$\mu=\delta_{\max}/\delta_y$		δ_y (mm)
CS	390	4.6	85	585	26	5.65	260	6.3
S	340	4.8	77.3	545	23.5	5.34	207	4.7
F	160	8.2	19.5	235	35	4.26	112	8.5

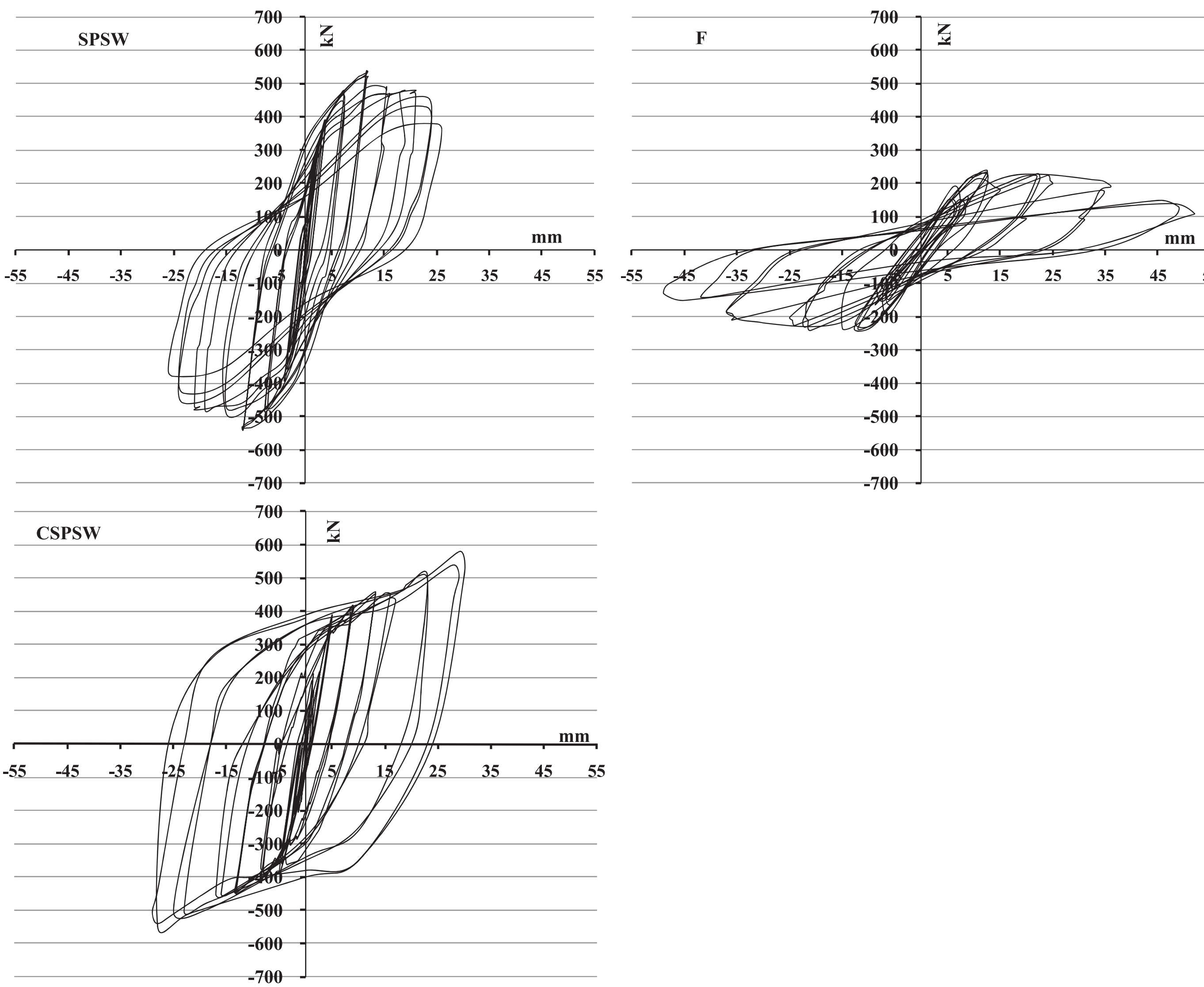

Fig. 4. Hysteretic shear behaviour in phase-I.

foundations (or an RC) is the same as in the unilateral buckling mode (Fig. 6). Under the effect of pure shear, unilateral buckling coefficient is obtained by using Eq. (1) [10–12]

$$\tau_{cr}=k_{cr}\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 \qquad (1)$$

In the specimens with four or more bolts (all the specimens, except HC5), yielding dominated buckling. In other words, as the bolt spacing decreases, out-of-plane buckling of the steel plate in the free side of steel plate decreases. However, there was no buckling in the side, where an RC panel is located. These conditions allow buckling in one side of the steel plate between the bolts, which makes buckling strength to be larger than an yield load and causes an yield to happen before buckling.

Buckling load was obtained as 10 ton for the HC5 specimen, which led the k_{cr} to be equal to 10.5 for this specimen from Eq. (1). The yield load for the specimens HC1, HC2 and HC3 was obtained as 31, 21.5 and 23 ton, respectively. Unfortunately, in the specimen HC4, welding of steel plate to the boundary frame was torn before the plate buckling, and thus no yield load was recorded. The maximum recorded load belongs to HC1, in which the plate was stiffened by RC panels at both sides. It is worth noting that an extra reinforcement did not improve seismic behaviour of the specimen HC2 in comparison with the HC3. As

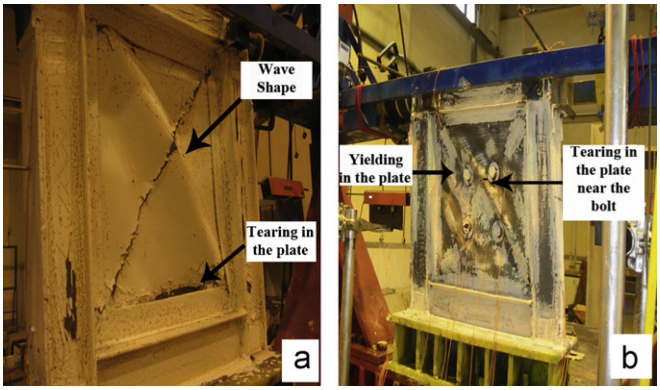

Fig. 5. (a) Waves shape and tearing of the plate in the *S* specimen and (b) yielding and tearing of the plate in the *CS* specimen.

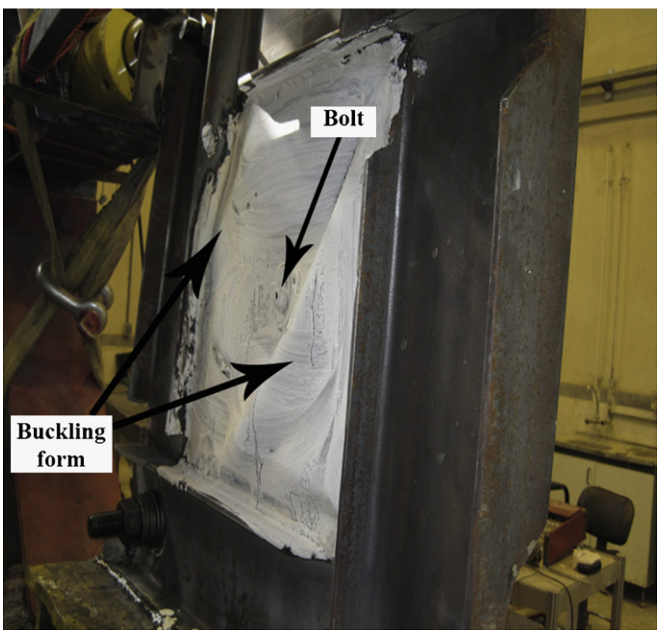

Fig. 6. Steel plate buckling in the *HC*-5 specimen with 1 bolt.

a result, the bolts prevent early elastic buckling of the steel plate. As the number of bolts increases, buckling load increases faster.

The AISC-2005 code recommends Eq. (2) for prevention of plate buckling prior to yielding under pure shear. This equation is indeed the equation of compact section in plate girder [5]

$$\frac{b}{t} \le 1.1\sqrt{\frac{k_V E}{F_y}} \tag{2}$$

where k_V is the shear buckling coefficient and can be computed from Eq. (3)

$$k_V = 5 + \frac{5}{(a/b)^2} \tag{3}$$

where a is the distance between the stiffeners in plate girder and also the distance between the bolts in CSPSW sub-panels.

For the specimens with hinged-connections, by assuming k_V=10; and b more than 19.2 cm, steel plate buckles before the yielding. The experiments results showed a good agreement with Eq. (2) presented in the AISC-2005 code.

4. Phase-II: description of the specimens and testing procedure

4.1. Specimens and tests setup

In phase-II, the three-story specimens with 1:3 scale were coded as “CS3-1”–“CS3-4”, and one-story specimens with 1:4 scales were coded “CS1-1”–“CS1-3”, wherein the first digit (3 or 1) shows the number of stories and the second digit is the index of specimen. A schematic diagram of the CS3 specimens setup is shown in Fig. 7. However, CS1s setup was used in the same way as the mentioned diagram in phase-I. The specifications of the test specimens in phase-II (one-story and three stories) are listed in Tables 5 and 6, respectively. All the specimens' performance were alike a dual system, say, moment resisting frame combined with CSPW. It means that beams and columns were rigidly connected to each other. In other words, moment resisting frame serves as a backup for CSPSW against the lateral load.

The variable parameters in CS1s were direction of reinforcement bar and number of RC panels in the sides of the steel plate. It is to be noted that diagonal cracks were observed in the RC panel connected to the CS specimen in phase-I. Therefore, as shown in Fig. 8, for comparison of seismic behaviour and prevention of damage to the RC panels, the reinforcement bars were used in 45° and 135° directions (perpendicular to the diagonal crack) in the CS1-3 specimen. After obtaining good results from the HC-1 test in phase-I, another specimen, i.e. CS1-2, with the RC panels on both sides of the steel plate was constructed and tested. The specifications of CS1s were similar to the CS in phase-I, except for type and diameter of the bolts.

The three-story CS3 specimens were constructed with three similar frames. A continuous weld connected all components of the experiment. The columns and roof beams consisted of 2IPE profile that welded together directly without any plate. However, according to the preliminary design, for the beams of the first and second floor plates were attached to IPE profiles. Beam to column connections were butt welded. In other words, the connections were rigid. To prevent local buckling of the columns' web and flange, small plates with appropriate dimension were welded to the column web along the beams' flange direction.

Nine bolts at regular spacing were used in each story for connecting the RC panels to the steel plate. Fish plate connection, which was used for the connection of infill plates to the inside flanges of boundary frame in the test specimens, was designed based on an infill plate shear capacity. It must transfer the entire shear force of an infill plate into the boundary frame. Initial imperfection and residual stress were excluded very well by discrete welding at different times. Furthermore, the existence of bolts at different points in the steel plate helped the removing of initial imperfections considerably.

The main damages to RC panels in the phase-I tests were crushing and cracking that occurred especially at the bolts locations. So, to avoid crushing and cracking of RC panels in the phase-II, high strength concrete was used. Nevertheless, reinforcement ratio was the same as in the tests phase-I, i.e. the vertical and horizontal small reinforcement ratios of 1% was used. Gap size around the RC panel was designed so that no interaction occurred between the RC panel and the boundary frame up to the failure point. Although, a slight damage occurred in the RC panels with reinforcement bars at 45° and 135°, but due to practical difficulty, traditional horizontal and vertical placements of reinforcement bars were practised for the CS3 specimens.

4.2. Materials

Mild steel, A36, was utilised for the infill panel and fish plate material in all specimens in the test programme based on the

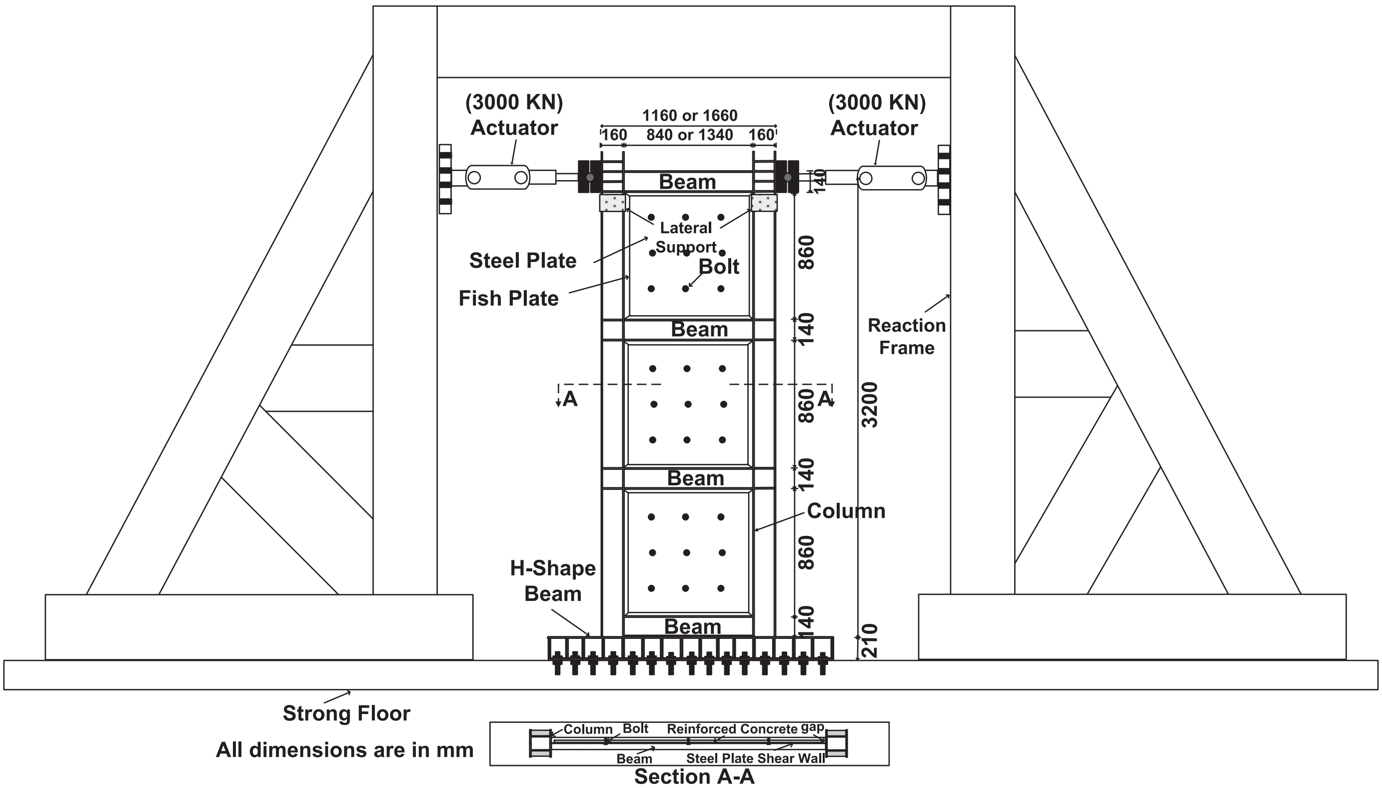

Fig. 7. A schematic diagram of the setup of three-story specimens.

Table 5
Properties and dimensions of the one-story specimens in phase-II.

Specimens	CS1-1	CS1-2	CS1-3
Columns (mm)	2IPE100+2Pl100 × 5	2IPE100+2Pl100 × 5	2IPE100+2Pl100 × 5
Foundation beam (mm)	2IPE100	2IPE100	2IPE100
Roof beam (mm)	2IPE100	2IPE100	2IPE100
Steel wall plate thickness (mm)	2	2	2
Fish plate (mm)	40 × 5	40 × 5	40 × 5
Number of bolts	4	4	4
Bolt diameter (mm)	6	6	6
Type of bolt	10.9	10.9	10.9
Rebar diameter (mm)	3	3	3
Reinforcement ratio	1%	1%	1% (45 and 135 deg.)
Concrete thickness (mm)	30	30 (both sides)	30
Free space around concrete (gap) (mm)	11.25	11.25	11.25

Table 6
Properties and dimensions of the third-story specimens in phase-II.

Specimens	CS3-1	CS3-2	CS3-3	CS3-4
Columns (mm)	2IPE160	2IPE160	2IPE160	2IPE160
Foundation beam (mm)	L=840, IPE140	L=840, IPE140	L=840, IPE140	L=1340, IPE140
First and Second beams story (mm)	IPE140+2PL100 × 8	IPE140+2PL100 × 8	IPE140+2PL100 × 8	IPE140+2PL100 × 8
Third-story beam (mm)	2IPE140	2IPE140	2IPE140	2IPE140
Steel wall plate thickness (mm)	2	2	3	2
Fish plate (mm)	60 × 5	60 × 5	60 × 5	60 × 5
Number of bolts	27	27	27	27
Bolt diameter (mm)	10	10	12.5	12.5
Type of bolt	8.8	8.8	8.8	8.8
Rebar diameter (mm)	8	8	8	8
Reinforcement ratio	1%	1%	1%	1%
Concrete thickness (mm)	40	40	40	40
Free space around concrete (gap) (mm)	30	-	30	40

ASTM standard. However, high strength steel A572/50 was used for the boundary frames, because strong column prevents system instability. The phase-II tests were performed similar to the phase-I tests to determine the specification of elements. The specimens' steel parts properties are given in Table 2. Specifications of the bolts (shear studs), which were made of high strength steel, A490, are $F_u=10$, $F_y=9$ and $F_u=8$, $F_y=6.4$ ton/cm^2 for the one-story and three-stories specimens, respectively. Neglecting the weight of RC panels, the bolts were subject to shear tension due to the steel plate's local inelastic buckling. Thus, the bolts should be designed so as to tolerate according to these forces. Since the bolts tension due to plate buckling is not known, they were designed so as to transfer total shear force between the reinforced concrete panel and the steel plate.

High strength concrete with a minimum thickness and reinforcement bar was used in the tests. Average properties of the RC components are presented in Table 3. One batch of the concrete was cast simultaneously in all panels for each specimen. Then, it was covered by wet hessian to avoid moisture loss up to the test day.

4.3. Loading history

Lateral loads were applied at roof beam level in a cyclic manner according to the guidelines proposed by an ATC-24 [9] in order to simulate a severe earthquake condition. The loading was similar to conditions for the specimens with rigid-connection in phase-I. In other words, yield-displacement in the top of the specimens was estimated by FE method and pus-hover analysis (Table 7). Then, the loading cycles were determined and applied. The specimens were loaded by a hydraulic jack at its most upper part. The top horizontal displacement of the specimens was taken as the displacement control parameter. Lateral supports were utilised at the level of top-story at both sides of the specimens to prevent the out-of-plane displacement. Vertical (gravitational) loads were not considered in the present study.

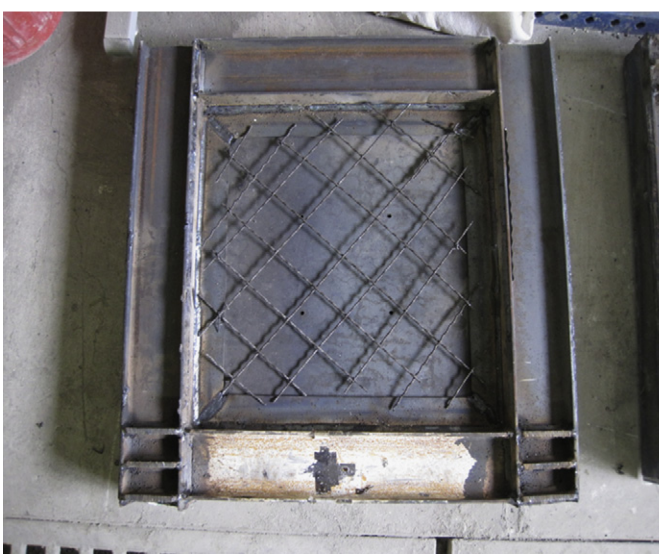

Fig. 8. Arranging of reinforced bars in the *SC1*-3 specimen.

4.4. Test results

Appropriate bolt spacing in the one-story specimens caused plate yield antecedent buckling (see the results for the CS specimen). This is an important factor in an energy dissipation, which applies less force to the boundary frame (i.e. column and beam). Besides, the RC panel prevents steel plate buckling at the side of an RC panel to reach minimum value (approximately zero). However, inelastic unilateral buckling formed at the free side of the specimens was covered at one side by the RC panel (CS1-1 and CS1-3). No considerable damage was observed in the RC panel, while it prevented buckling of the plate. That is because the gap between the panel and the boundary frame was designed properly so that no interaction occurred between the RC panel and the boundary frame. The use of normal reinforced concrete for the CS specimen in phase-I was accompanied by severe concrete cracking and crushing. However, specimens in phase-II with high strength concrete just developed fine diagonal cracks. In addition, even in the CS1-3 specimen, in whom the direction of reinforcement bars was perpendicular to the diagonal crack, no fine crack was observed. Fig. 9 shows the history of base shears versus the top horizontal displacements of the single-story specimens. In addition, the behavioural characteristics of the tested specimens (such as yield and maximum displacement, initial stiffness, ultimate base shear capacity and cumulative energy dissipation) are given in Table 7. As shown, initial stiffness, maximum base shear and total energy dissipation for the CS1-2 specimen were higher than those for the CS1-1 and CS1-3 specimens, while its ductility was less as compared with the mentioned specimens. As mentioned before, an RC panel prevents out-of-plane displacement of steel plate. In other words, steel plate action will be in-plane especially in the CS1-2 specimen. Steel plates in all the tested one-story specimens yielded and dissipated considerable energy during the cyclic loadings. The results indicate that there is no significant difference between the parameters CS1-1 and CS1-3 in this regard. It means that changing of the direction of reinforcement bars only reduced the degree of damage to the concrete. It is important to note that in one-story specimens, the bending effect was negligible because of short height. Also, in the one-story specimens, during the tests, some of bolts failed, because they were under shear and bending due to an inelastic buckling of plate (Fig. 10).

The three-story tests hysteresis loops, which present applied shear force versus lateral displacement in the level of roof beam, are shown in Fig. 11. Also, their primary parameters of them are given in Table 7. These curves indicate a good energy dissipation and rather high ductility, since they are S-shape curves. Among the

Table 7
Behavioural characteristics of the tested specimens in phase-II.

Specimen		Yield point			Maximum		Ductility	Total energy dissipation (kN m)	FE estimation
		P_y (kN)	δ_y (mm)	Initial stiffness (kN/mm)	P_{max} (kN)	δ_{max} (mm)	$\mu=\delta_{max}/\delta_y$		δ_y (mm)
One-story	CS1-1	380	4.9	80.9	595	27	5.5	250	5.6
	CS1-2	420	4.4	102.4	630	22	5	295	7.1
	CS1-3	375	4.6	85.2	600	24	5.2	270	5.2
Three-story	CS3-1	18.2	5.8	18.1	483	106	5.8	732	21.3
	CS3-2	15.5	4.94	26.5	575	76.3	4.9	630	17.6
	CS3-3	16.7	5.32	21.9	557	90	5.4	960	20.1
	CS3-4	24.4	7.77	21.1	672	133	5.5	1263	31.2

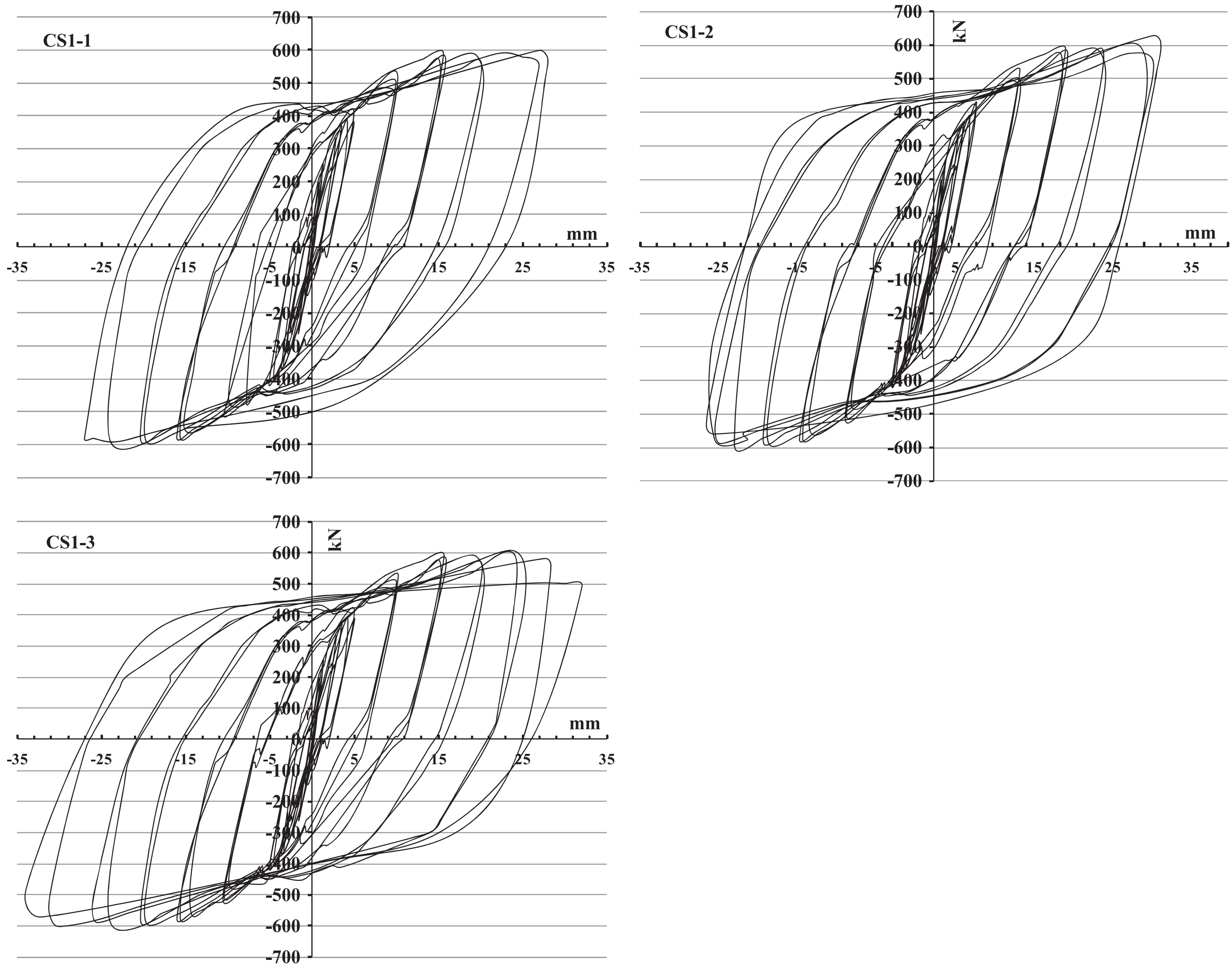

Fig. 9. Lateral load vs. displacement (hysteresis loop) of the one-story specimens.

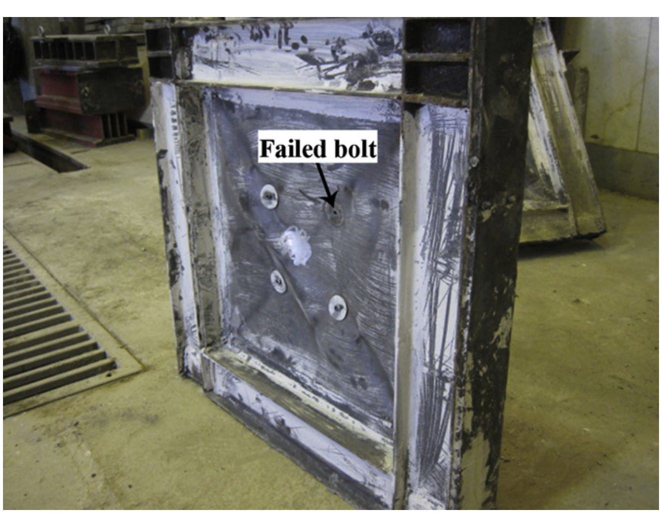

Fig. 10. Failure bolt in the *SC*1-1 specimen.

mentioned specimens, seismic behaviour of the CS3-3 specimen was better than the others, because of the suitable composition of its components. This favourable behaviour was attributed to appropriate composition of its constitutive elements. As a result, the specimens with weak columns did not show desirable behaviour seismic, especially when it accompanied pinching phenomena.

The ductility and energy dissipation of SC3-2 specimen were less than those of the SC3-1 specimen (about 20% and 16%, respectively). This is mainly due to contribution of the RC panel in the specimen's stiffness, which induces its force to the columns and eventually to the buckling of the columns in the SC3-2 specimen. Maximum shear force of the CS3-2 specimen was 20% more than that of CS3-1, because of less damage to the high strength reinforced concrete and also its contribution in load bearing. In the SC3-4 specimen testing, with the length to height of plate ratio in each panel equal to 1.5, the steel plate was more subjected to shear rather than bending, because of the bending stress distribution. This influences waves formation as shown in Fig. 12. The RC panel connected to the steel plate by bolts (shear connectors) produced lateral stiffness that delayed buckling. It sounds that minimum concrete thickness and the ratio of reinforcement bars are enough. As can be seen in Fig. 13, the RC panels in the specimens remained intact at the end of the tests, except for some fine diagonal cracks. Nevertheless, except for the SC3-2 that had no gap around the RC panel, significant out-of-plane of the RC panels was visible in the others at the end of the test, so that they were kept in place only by bolts.

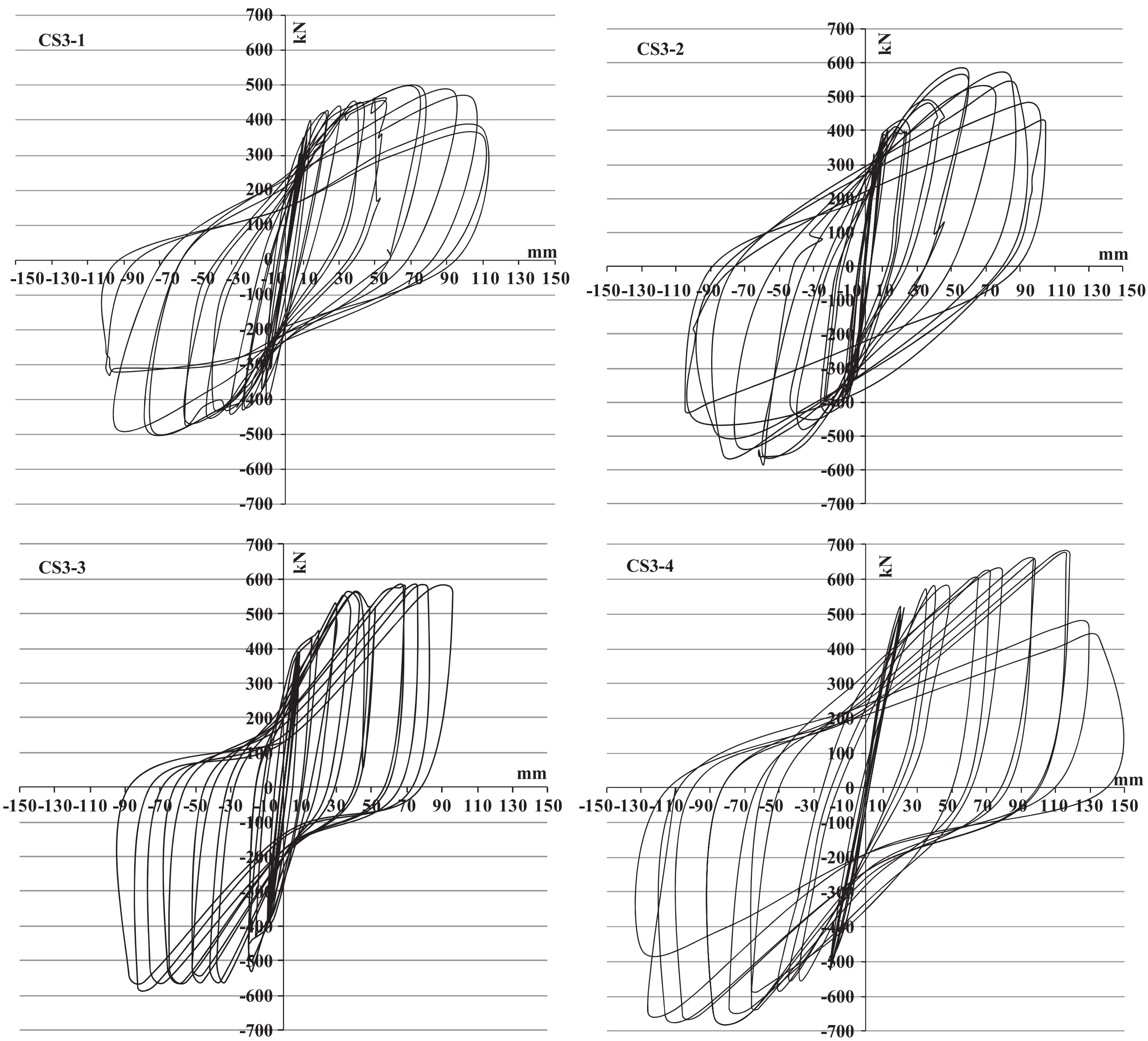

Fig. 11. Lateral load vs. displacement (hysteresis loop) of the three-story specimens.

Fatigue in the base columns of the three-story specimens caused failure, but this failure was brittle in specimens, CS3-2 and CS3-4. However, ductile failure mode occurred in the other two specimens (CS3-1 and CS3-3). In other words, in the SC3-2 and SC3-4 specimens, the yielding of steel plate occurred after the yielding of the columns. The steel plate behaviour depends on the ratio of bolt spacing to plate thickness, b/t. In these tests, bending effect due to shear force in the top-story was the dominated shear, particularly in the lowest story specimens, except for the SC3-4 specimen. Therefore, proper choice of b/t ratio can prevent buckling before yielding, as in the case with the SC3-3 specimen. In other specimens, post-buckling behaviour of the steel plate was due to the tension-field action between the bolts, Fig. 14. At the end of tests, due to proper design of the gap size around the RC panel, no contact was observed between the boundary frame and the RC panels. Out-of-plane displacement of the steel plate at the region near the bolts was negligible. It means that the bolts fulfilled their main role. A slight slide was observed in some bolts, especially in the corner ones, though none of them failed during the tests. The middle bolts were subjected to shear, since no waves were formed in the region near the bolts. Other bolts were under both shear and bending. In general, favourable strength of the bolts transferred forces (shear and axial) into the RC panels.

It is to be noted that, all beams were yielded before an ultimate failure in all specimens. After formation of plate waves or yielding of plate, beams web in all three floors started to yield. Uniform distribution of force caused beam web to yield uniformly along its length.

5. Member design

5.1. Steel plate

In CSPSW, the total capacity of the wall can be used as against lateral load and there will be a uniform stress distribution

Fig. 12. Waves shaped in the first-story in the *CS*3-4 specimen.

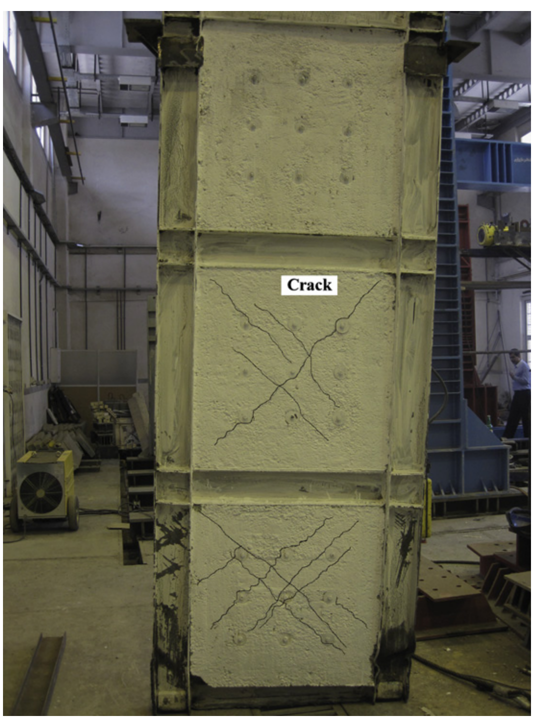

Fig. 13. Diagonal micro-cracks in the concrete for the *CS*3-2 specimen at the end of the test.

between the boundary frame and the steel plate when the bolt spacing is arranged such that buckling of plate occurs prior to its yielding (even, they have a better distribution in comparison with an SSPSW). The steel plate is subjected to stresses shown in Fig. 15. The interaction between shear and axial stresses for unilateral buckling can be computed using Von-Mises equation [13] and the AISC code predicted by Eq. (2) for pure shear.

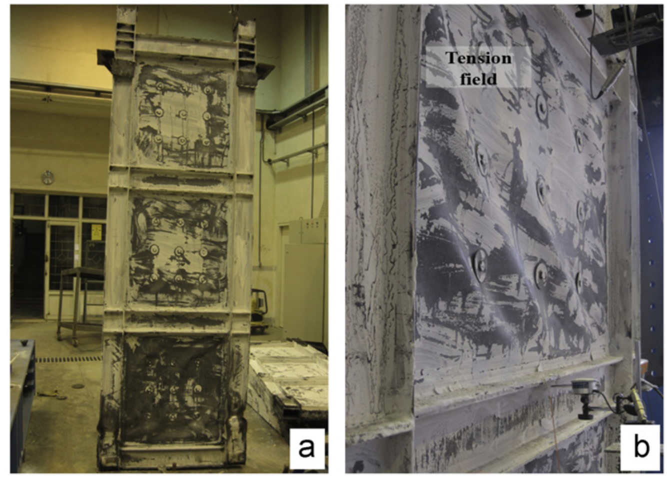

Fig. 14. (a) Yielding steel plate panels in the *CS*3-3 specimen and (b) plate tension-field action between the bolts in the second-story at the *CS*3-4 specimen.

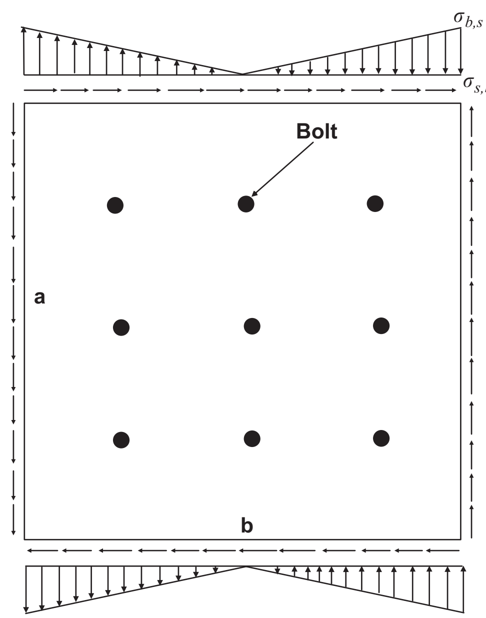

Fig. 15. Applied stresses to the steel plate.

Therefore, width to thickness ratio of the plate between the bolts (b/t) can be determined for a particular loading. It is worth noting that the effect of bending is lower in the upper stories. Similar to the above-mentioned stresses, axial stresses resulting from the weight of the RC panel are imposed to the plate through the bolts in vertical direction, which is negligible as compared to other stresses.

5.2. Beam

Except the stresses produced by the gravity load of floors, the other stresses applied to the fixed beams due to the performance of seismic system against lateral load are shown in Fig. 16. The weight of CSPSW, W_C at each story level is applied to the bottom beam. The stresses of two adjacent stories cancel out each other at the intermediate beam, except for (W_C). The floor beam is subjected to high shear and bending stresses as well as stress resulting from (W_C), while the last floor beam due to the lack of bending is only under shear stress. In order to improve the

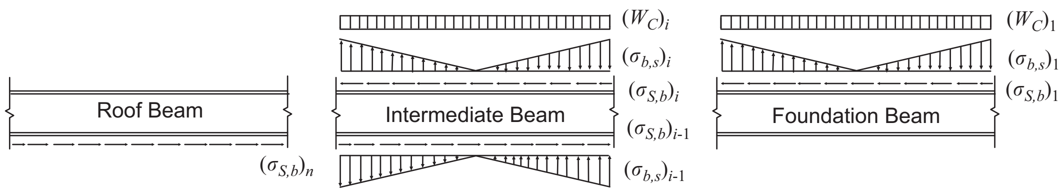

Fig. 16. Applied stresses to the story beams.

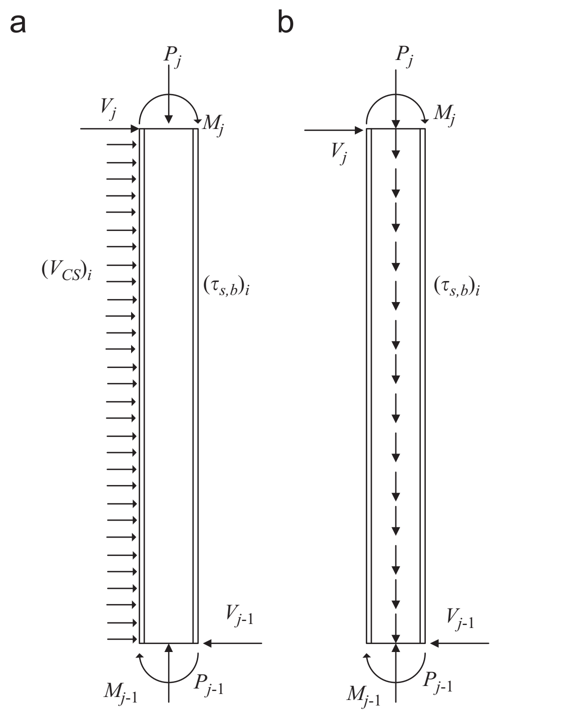

Fig. 17. Applied stresses to the columns of the specimens: (a) without gap and (b) with gap.

seismic behaviour of the system, whole steel plate of the panel should yield before the formation of plastic hinge in beams. Accordingly beams are simultaneously under axial force and bending moment. Eq. (4) proposed by AISC-LRFD-1999 [14] can be used to control the strength of beams and columns:

$$\frac{P_u}{P_n}+\frac{8M_u}{9M_n}\leq 1 \tag{4}$$

where all of its parameters are computed from Eq. (5) to Eq. (8)

$$P_u=((\sigma_{S,\,b})_{i-1}t_{i-1}-(\sigma_{S,\,b})_i t_i)L \tag{5}$$

$$M_u=\frac{((\sigma_{b,\,S})_{i-1}t_{i-1}-(\sigma_{b,\,S})_i t_i)L^2}{8}+\frac{(W_C)_i L^2}{12} \tag{6}$$

$$P_n=F_y A \tag{7}$$

$$M_n=F_y Z \tag{8}$$

where L is the beam length, Z is the plastic section modulus, A is cross section area of the beam and i is the numerator of the story.

The testing results and observation of specimens showed that all of the beams were intact up to the end of the tests. It means that beams could transfer all forces safely into the columns and the mentioned equations verify our tests result as well.

Table 8
Evaluation of strength in the first-story columns.

Specimen	First-story column (kN m)					
	M_n	M_n	P_u	M_u	$\frac{P_u}{P_n}+\frac{8M_u}{9M_n}$	Result
1	86.7	86.7	913	0.0	0.67	O.K
2	86.7	86.7	1450	88.55	1.96	N.G.
3	86.7	86.7	1352	0.0	0.986	O.K
4	86.7	86.7	1449	0.0	1.06	N.G.

5.3. Column

By proper design of CSPSW, i.e. precedence of buckling by plate yield, columns are under gravity loads, shear stresses due to the steel plates and vertical loads due to the shear wall without gap (Fig. 17). Moreover the gravity load in the columns of bottom stories resulting from the weight of CSPSW is noticeable. In order to the transfer produced stresses in the shear wall, the columns should be capable to withstand these stresses before the formation of plastic hinge

$$P_u=\sum_{i=1}^{n}((\tau_{S,\,b})_i t_i)h+\sum_{i=1}^{n-1}\frac{(W_C)_i L}{2} \tag{9}$$

$$M_u=\frac{(V_{CS})_i (h)_i^2}{12} \tag{10}$$

In accordance with an ACI-318-05 [15] and assuming the uniform distribution and transfer of total shear force from the RC panel into the columns (Fig. 17b), (V_{CS}) can be computed from the following equation for the specimens without gap

$$(V_{CS})_i=\frac{\sqrt{(f'_C)_i}}{6(h)_i}(t_c)_i L_i+\left(\frac{A_V F_y d}{Sh}\right)_i \tag{11}$$

where h is the story height, t_C is the concrete thickness, $d=0.8L$, A_V is the cross section area, S is the distance between horizontal shear bars and n is equal to the number of stories above the considered story plus one.

As an example, the strengths of the first-story of experimental specimens that contain the most critical column are evaluated in Table 8. Assuming that bolts are spaced in a manner that the total capacity of steel plate can be used, the stresses and consequent axial forces and also the moment imposed to the column are computed. In a CSPSW with correctly designed plate and bolts spacing, the smallest force will be applied (transmitted) to the columns and beams. Even, no bending moment occurs in the column with gap. In comparison with non-stiffened SPSW, the imposed forces to the boundary frame, particularly to the columns decrease considerably.

In the CS3-1 specimen, the column was designed to be strong. Therefore, it remained intact until yielding of the panel. Although according to Von-Misses equation, bolt spacing was designed in a way that the plate was buckled before yielding. In the case of taking proper bolt spacing, the column will have the capability to withstand all the stresses. In the CS3-2 specimen, due to the contribution of high strength concrete to carrying of lateral load and to improve wall stiffness, stronger columns are required to carry these forces. According to the experimental observations, the base of column at the first-story yielded at the initial stages of loading. The CS3-3 specimen was designed properly. It means that the bolt spacing was appropriate and also the columns had favourable performance during the force transferring process. Also, according to the experimental observations, the columns were weak in the CS3-4 specimen.

6. Conclusion

In this research, a comprehensive experimental study of composite steel plate shear wall (CSPSW) was carried out and results are presented. Hinged and rigidly connected specimens with 1:4 and 1:3 scales were subjected to cyclic loading and push-over. The main parameters in the testes were gap around the RC panel, direction of reinforcement, number of bolts and RC panels, steel plate thickness and the specimen length to the width ratio. Based on the obtained results, the following observations and conclusions were drawn:

- If the number of bolts increases, CSPSW buckling load acceleration will increase.
- Using RC planes at both sides improves the main properties of specimens like energy dissipation and strength, while decreasing its ductility.
- Bolts should be designed for combined shear and bending forces, except for the bolts at the centre of the panel, to which designing of the bolt for shear alone is enough.
- Using high strength concrete reduces damage to the RC panel significantly in comparison with the normal concrete. However, it has no serious effect on the system's strength.
- For lateral stiffening of steel plate, minimum reinforced concrete thickness and reinforcement bars are adequate.
- If centre-to-centre bolt spacing decreases, steel plate capacity will increase, while system ductility will decrease.
- In multi-story specimens, the bending effect due to shear dominates the shear force. In other words, base columns are subject to considerable bending stress and it may result in plate buckling or yielding.
- Appropriate combination of system elements shows favourable seismic behaviour, especially with strong columns. It means that columns must tolerate the entire force induced by infill steel plates as well as the gravity loads.

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