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Experimental and numerical study on steel-concrete composite shear wall using light-weight concrete

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ABSTRACT

In the present study, the behaviour of concrete stiffened steel plate shear wall (CSPSW) using precast light weight-concrete panels is experimentally and numerically investigated. Three one-bay one-story CSPSW specimens were designed, fabricated, and tested. The steel materials and dimensions of all specimens were the same; however, their precast concrete panels were different. One of the specimens had a precast normal-weight concrete panel on one side of the infill steel plate; on the other hand, another specimen had a light-weight one. The third specimen had two light-weight concrete panels on both sides of the steel plate. The quasi-static cyclic test results indicate that CSPSW with a light-weight concrete panel is a reliable lateral load-resisting system for steel structures. In addition, the shear capacity of specimen with light-weight concrete was approximately similar to specimen with normal-weight one. Therefore, it can be inferred the new system is able to reduce the seismic mass and improve the behaviour of steel structures. In this study, the light-weight concrete panel was 36% lighter than normal-weight panel. Based on the test data, specimens could tolerate high inter-story drift between 5.04% and 6.24% until the shear capacity decline in 80% of the maximum shear load recoded in the test. It should be mentioned that the infill steel plate of CSPSW undergoes entirely inelastic deformation and dissipates significant seismic energy through large lateral displacement.

1. Introduction

Steel structures are numerously being manipulated in buildings which have different applications. Steel structures can have different lateral load-resisting systems such as the moment resisting frames or braced frames in order to resist earthquake or wind loads. However, in recent decades, the construction of steel plate shear wall (SPSW) has been widely used in steel structures designed for high seismic hazard area [1,2]. In SPSW, steel plates are installed in one or more bays of a structure and welded to surrounding steel frame—columns and beams. The advantages of SPSW are its simple and economical construction, significant initial stiffness, considerable shear capacity, proper ductility, and great energy dissipation [1–8].

By contrast, it would be unfair not to mention the fact that the main disadvantage of SPSW is the buckling of the infill steel plate in compression field which triggers significant reductions in lateral stiffness, shear capacity, and energy absorption of the system [9]. Steel stiffeners or a reinforced concrete panel can be manipulated in order to occlude the global and local elastic buckling of the infill steel plate. As a result, the stiffened system responses stable hysteretic behaviour with less pinched [9–12]. Furthermore, the stiffened shear wall supplies greater shear capacity and initial elastic stiffness.

Using the reinforced concrete panels attached to one side or both sides of the steel plates creates a composite shear wall [9]. The infill steel plates and the reinforced concrete panels are connected by shear studs or bolts in composite shear wall. The composite shear wall that contains an infill steel plate and a reinforced concrete panel on one side of the steel plate is denoted as concrete stiffened steel plate shear wall (CSPSW) in AISC Seismic Provisions [13]. In CSPSW, there is a gap between the reinforced concrete panel and the steel frame; hence, the concrete panel works merely as a stiffener [9,14]. The gap enhances significantly the performances of composite shear wall.

Although a vast variety of studies have been conducted on unstiffened and stiffened SPSW, a limited research has been fulfilled on CSPSW [13,15]. Astonishingly, the limited conducted investigations attests that the performance of steel frame is markedly improved by utilizing CSPSW. In addition, CSPSW is an applicable lateral load resisting system which provides a great ductility, high shear capacity, and considerable energy absorption [14–18]. The first experimental investigation conducted by Astaneh-Asl and Zhao illustrated that the innovative

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CSPSW can tolerate shear load up to the inter-story drift angle of 5%. Moreover, the gap between the concrete panel and steel boundary elements reduces the damage to the reinforced concrete panel. It should be mentioned that although the reinforced concrete panel prevents the elastic buckling of infill steel plate, the nonlinear local buckling of plate takes place in the areas between the shear connectors [14]. In a subsequent study accomplished by Arabzadeh et al., it is confirmed that CSPSW demand strong columns in multi-story because of bending force effect. If the system has strong columns, the seismic behaviour is improved in severe earthquake [17].

L. Guo et al. compared experimentally SPSW and CSPSW and it was figured out the shear capacity and energy absorption of CSPSW are obviously higher than SPSW. However, their experimental research was terminated due to failure of a column at approximately the drift of 2.5% [18]. It should be noted that based on the Rahai and Hatami research, shear connectors distances play a pivotal role in composite action of the system and the space between them should be determined in accordance with the thickness of steel plate. According to the study, beam-to-column connections and middle beams rigidity have a subtle impact upon the overall composite behaviour of CSPSW [19].

In accordance with a previous work of the authors [16], in CSPSW, a specific reinforced concrete panel thickness is required to improve the shear capacity of the system and a thickness greater than that does not have any effects on in-plane shear capacity of the wall. In other words, the greater thickness just increases the seismic mass of steel structures without improving lateral strength and stiffness. Based on a work of Dey and Bhowmick, CSPSW is a reliable lateral load resisting system and its response under time-history analysis is markedly excellent in terms of initial elastic stiffness, shear capacity, and ductility [20].

2. Purpose of the study

In CSPSW, the reinforced concrete panel merely works as a stiffener, owing to the gap between the reinforced concrete panel and the steel frame, and does not participate considerably in resisting in-plane shear loads [9]. In addition, as the reinforced concrete panel is attached to the infill steel plate by bolts, the weight of concrete panel imposes significant seismic mass to the steel frame. Hence, stronger columns and beam are demanded to satisfy seismic requirements. By contrast, manipulating a light-weight concrete panel can reduce the mass of steel structure considerably and satisfy seismic design effortlessly. Therefore, an experimental quasi-static cyclic study was conducted to perceive and compare the behaviour of systems with normal-weight and light-weight concrete panels. In addition, the seismic properties of those specimens are evaluated.

3. Experimental program

The experimental research was conducted at Road, Housing & Urban Development Research Center (BHRC), Tehran, Iran. In order to study the seismic behaviour of CSPSWs with reinforced light-weight concrete panel, three scaled specimens were tested. The main differences of specimens were their concrete panels. The first specimen had a normal-weight concrete panel on one side of the infill steel plate, but the second one had a light-weight concrete panel. The third specimen was constructed with two light-weight concrete panels on both sides of the infill steel plate.

3.1. Specimens and test set-up

In experimental investigations, three one-story one-bay CSPSWs with the scale of 1:4 are designed and fabricated. The steel parts of all the specimens were fabricated and welded at a factory and then at the laboratory the precast reinforced concrete panels were attached to the infill steel plates of specimens by high strength bolts.

The specimens were design in accordance with AISC341 [13]. It is assumed that the infill composite wall resists complete lateral load up to entire yield of the infill steel plate; therefore, beams and columns were design according to capacity design method.

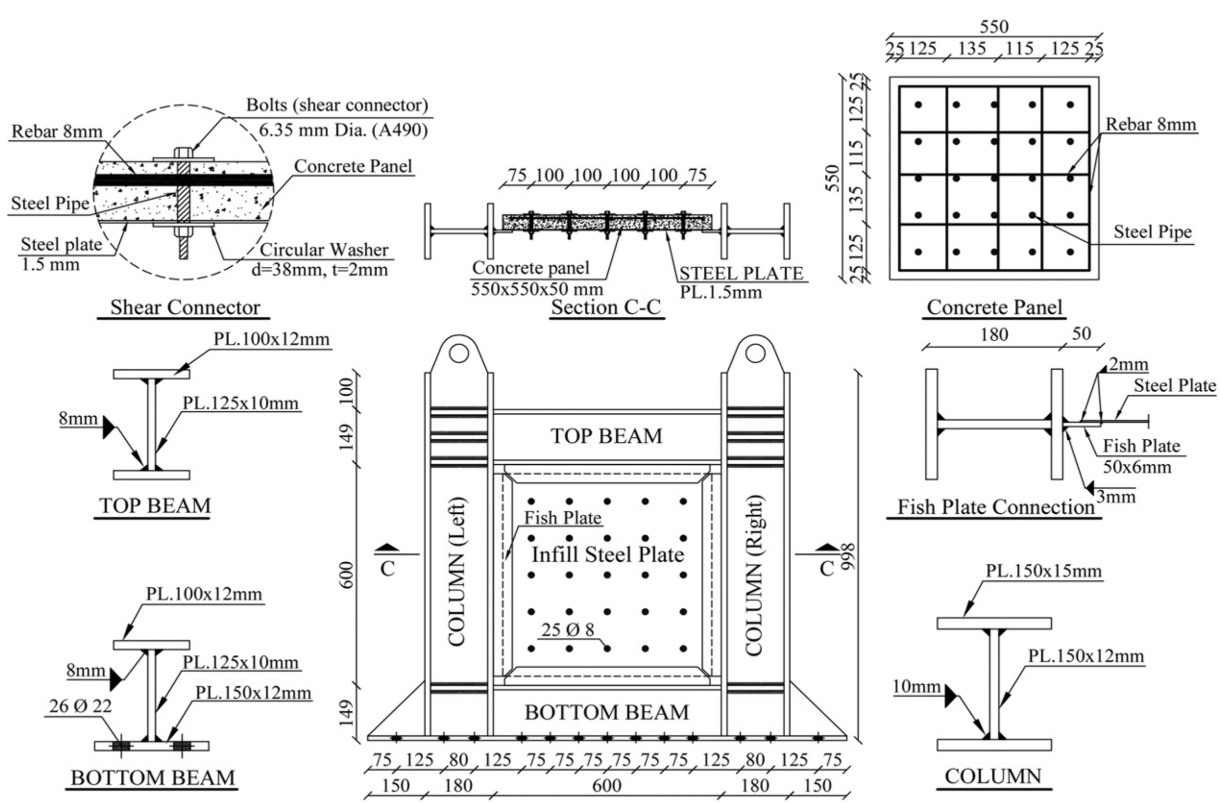

Fig. 1. Details of specimen CSPSW-L.

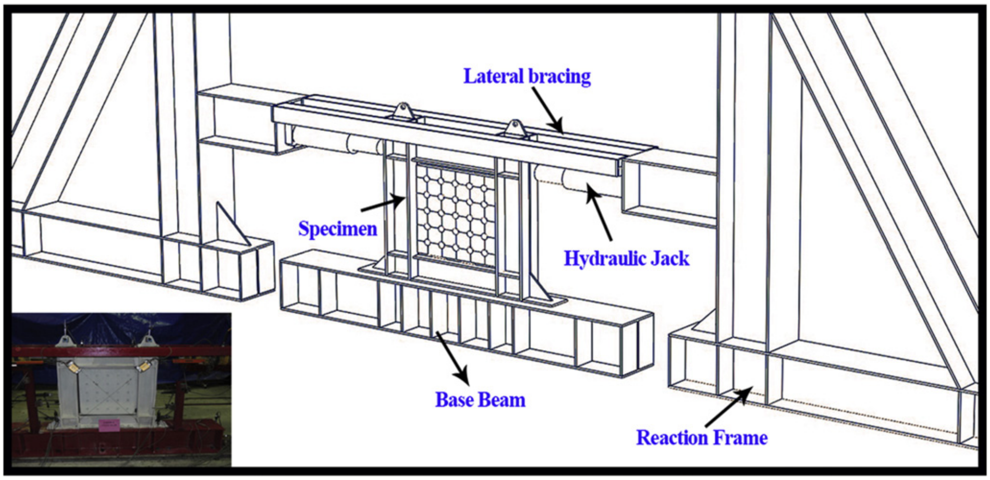

Fig. 2. Experimental test set-up of specimens.

The steel parts of three specimens were identical and they had approximately 1 m width and 1 m height. The width-to-height aspect ratio of 1:1 had been chosen to investigate normal shear wall in buildings which is pertinent to 4 m width and 4 m height of an archetypical building.

The infill steel plate thickness was 1.5 mm and the boundary elements—beams and columns—were built-up sections. In order to connect the infill steel plate to boundary elements, fish plates with the modified detail type B of a work of Schumakher et al. were used [21]. Fish plates with dimension of 50 × 6 mm were welded to surrounding steel frame. Beam-to-column connections were full moment connections by complete penetration groove welds.

The both normal-weight and light-weight concrete panels had dimension of 550 × 550 × 50 mm. The concrete panel was bolted by 25 high-strength (A490) bolts with 6.35 mm (1/4 In.) diameters to the infill steel plate to guarantee the composite behaviour. Center to center distances between the bolts were calculated 100 mm to prevent global and local elastic buckling of the infill steel plate [9]. In all three specimens, there were 25 mm gap between the boundary elements and the reinforced concrete panel. Fig. 1 shows a typical CSPSW specimen.

The arrangement of test set-up was based on the experimental requirements, so the main components were two significant rigid reaction frames, two lateral bracings, a strong base beam, hydraulic jacks and the specimen. The base beam was located between the bottom of the specimen and the strong floor. The specimen to the base beam and the base beam to strong floor were bolted by high strength bolts (A490). Lateral out-of-plane displacements of the specimen were restrained by lateral bracings which did not provide any mechanical connections in the direction of in-plane displacement. In other words, they were only lubricated contact faces. A 3D view of the experimental test set-up is illustrated in Fig. 2.

3.2. Material properties and loading history

The specimens had identical structural steel material properties. In order to determine the steel properties (stress-strain curve, the yield strength, and the ultimate strength), three tensile coupons were selected from each steel sheet and tested according to ASTM A370-05 [22]. Table 1 represent material properties of steel materials.

For concrete materials, compressive strengths and Young modulus were measured. Several cubic and cylindrical specimens were provided during the construction and they were tested at the age of 28 days and the test. The average compressive strength of normal-weight and light-weight concrete were 20.5 MPa and 22.8 MPa. In addition, the average Young modulus were 21,982 MPa and 17,198 MPa for normal-weight and light-weight concrete respectively. The density of normal-weight and light-weight were 2435 kg/m^3 and 1546 kg/m^3 as well.

Loading history was applied by two hydraulic jacks in accordance with ATC-24 [23]. Before the experiments, finite element models of specimens with the attained material properties were developed to predict the yield points of specimens which were approximately the same value observed during the tests. The cyclic displacement loading history of CSPSW-L is illustrated in Fig. 3.

Linear variable displacement transducers (LVDTs) and uniaxial strain gauges were placed on specimens to monitor the behaviour of the specimen during the lateral loading, as shown in Fig. 4. LVDTs measured the global and local displacement of key points of the specimens; furthermore, the out-of-plane displacements of the infill steel plate were captured by mounting them. In order to check the test-up and rigidity of base beam during the experiment, the displacements of the base beam were monitored by LVDTs as well. Strain guages were installed on critical points and propable plastic zones to acquire strains. The propable plastic zones were based on the finite element models of the specimens.

Table 1
Mechanical proprieties of the steel materials.

Type	Plate thicknesses (mm)	Modulus of elasticity (MPa)	Yielding stress (MPa)	Ultimate stress (MPa)	Elongation (%)
Infill steel plate	1.5	196e^3	200	308	42.0
Fish plate	6	208e^3	243	398	39.7
Boundary elements	10	201e^3	255	410	41.0
Boundary elements	12	204e^3	246	403	36.8
Boundary elements	15	203e^3	237	405	43.6
Reinforcement	8	209e^3	336	428	14.4

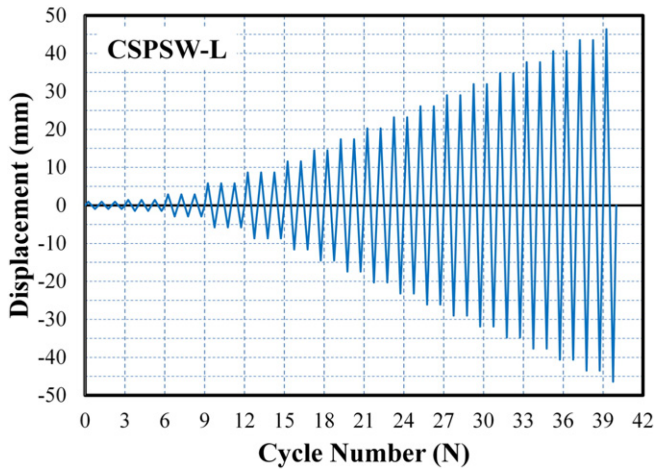

Fig. 3. Cyclic displacement loading history of CSPSW-L in accordance with ATC-24.

4. Experimental observations and results

In order to study the effect of light weight concrete panel, quasi-static cyclic test was conducted in the experimental program. The behaviour of the specimens was carefully captured during each cycle of loading until the end of the experiment. Hysteretic behaviour, failure modes, and the experimental observations were the main concentration of the program.

4.1. CSPSW-N test

Specimens SPSW-N had the normal weight concrete panel. The fabricated and installed specimen prior to the test is illustrated in Fig. 5. In the first six cycles, the behaviour of specimen was elastic and no elastic buckling of the infill steel plate was observed. It should be mentioned that the friction sound between the precast concrete panel and the infill steel plate could be barely heard during the cycles 4 up to 6. In cycle 7 when the lateral load was 385.43 KN, inelastic response of the infill steel plate initiated and the behaviour of specimens became nonlinear; however, the steel boundary elements were elastic. In 0.72% drift, the pure shear yield of the infill steel plate propagated and was noticeable.

In cycle 13, the inelastic local buckling between the bolts was captured; in addition, in this cycle, in 1.08% drift, shear yield of web plate of upper beam near Beam-to-Column connections was detected. In 1.8% drift, several hairy cracks were observed on the reinforced concrete panel and these cracks propagated during the next cycles. It should be mentioned that the hairy cracks disappeared, when the specimen was unloaded.

As the amount of lateral load was escalated, several X shaped tears appeared on the infill steel plate in 2.88% drift; on the contrary, the shear capacity of system did not decline. Furthermore, considerable inelastic local buckling of upper beam flange occurred in the loading group of 3.24% drift.

Ultimately, in cycle 38 with drift target of 5.04%, a bolt of base beam was suddenly fractured and the connection between column and base plate was fractured, as depicted in Fig. 6. The test was unfortunately terminated because of the unpredictable event. The final drift of the specimen CSPSW-N was 5.04% and the maximum attained shear load was 823.2 KN.

Fig. 7 shows the infill steel plate and the reinforced concrete panel of specimens SPSW-N at the ultimate point of the experiment. It can be clearly seen that the infill steel plate had undergone complete plastic response and several fractures which is desirable behaviour in a normal CSPSW.

4.2. CSPSW-L test

A light-weight concrete was used in specimen CSPSW-L on one side of the infill steel plate. In CSPSW-L, not only the dimensions, but also the materials of steel parts of the specimen were similar to specimen CSPSW-N. In the initial six cycles of loading, no distinguishable yield point on the specimen happened. Increasing of lateral load in cycle 7, the shear yield of the infill steel plate was taken place with the shear load of 367.39 KN. In this loading group, the infill steel plate between outer bolts and the steel boundary elements yielded, as illustrated in Fig. 8.

In the subsequent loading group with 0.77% drift, the pure shear yield spread out through the steel plate. In cycle 15 with lateral displacement of 8.7 mm, initial diagonal hairy cracks were observed on

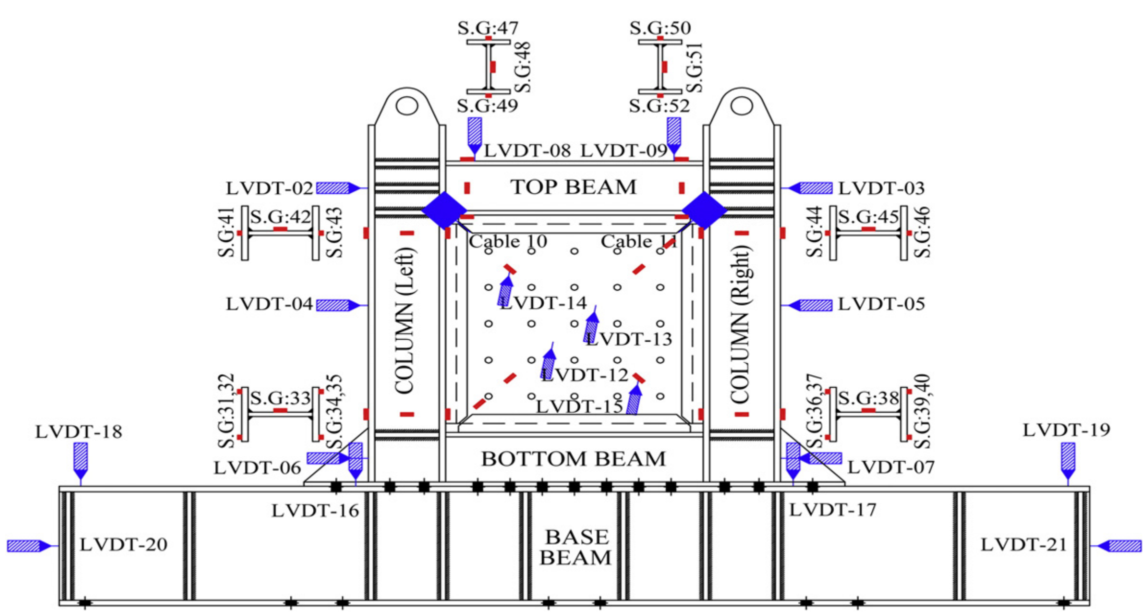

Fig. 4. A view of the installation of LVDTs and strain-gauges on the specimen CSPSW-L.

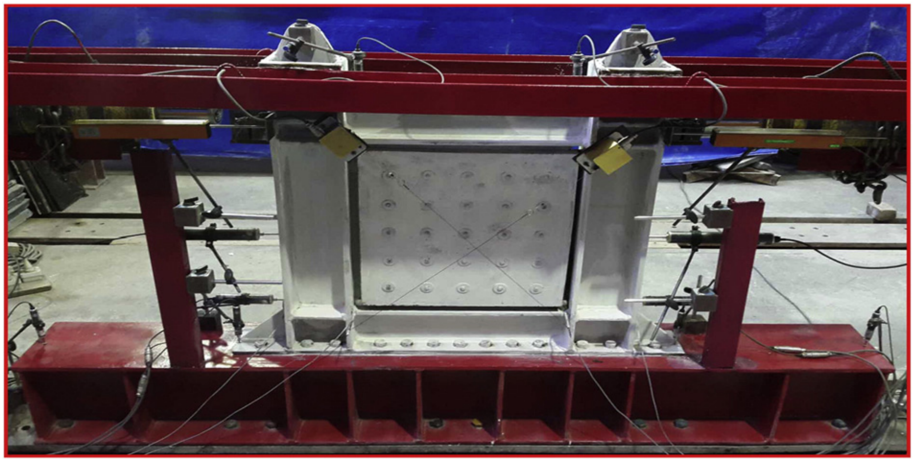

Fig. 5. Specimen CSPSW-N prior to the test.

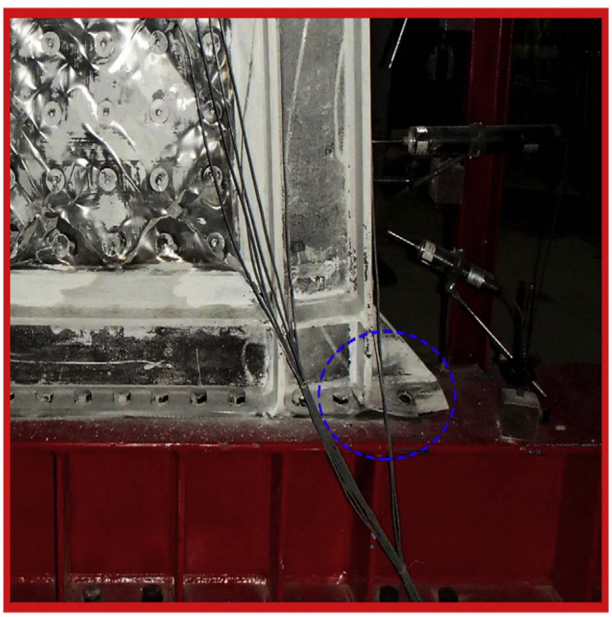

Fig. 6. The fracture of base beam bolt and the failure of connection between left column and base plate.

the light-weight concrete panel, as shown in Fig. 9b. In 1.55% drift, the shear load was 661.01 KN which caused the yield of web plate of upper beam close to beam-to-column connections.

In 2.71% drift, the fractures of the infill steel plate were detected in the four corners; in addition, many X shaped tears started to appear. Fig. 9a depicts the corner fracture of the infill steel plate. Although several more X shaped tears occurred on the infill steel plate, the shear capacity of the specimen CSPSW-L did not diminish and the maximum shear capacity of 809.48 KN was recorded in 4.64% drift.

In cycle 34 with 5.03% drift, the gap between the reinforced concrete panel and the right steel column were closed and concrete crushing of the panel took place. In the next loading group with 5.41% drift, the left beam-to-column connection commenced failing, so the shear capacity of the system reduced in the following cycles. Finally, the shear capacity of the system dropped below 80% of the maximum shear load recorded in the test. Fig. 10 demonstrates the infill steel plate and the light-weight concrete panel after the test. It can be obviously seen that the steel plate has experienced complete inelastic behaviour with many tears.

4.3. CSPSW-DL test

Specimen CSPSW-DL had two light-weight concrete panels on both sides of the infill steel plate. The dimension of precast light-weight

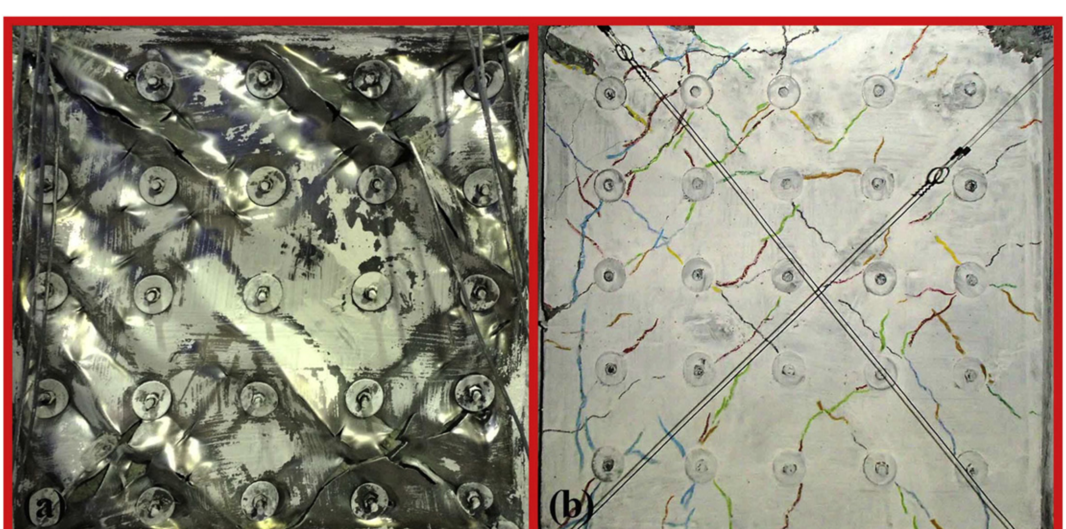

Fig. 7. Specimen CSPSW-N after the test: (a) the infill steel plate (b) the reinforced concrete panel.

Fig. 8. Specimen CSPSW-L: (a) after 6 cycles (b) after 9 cycles.

concrete panels was 550 × 550 × 50 mm. The obtained response of CSPSW-DL was approximately elastic in the first six cycles of loading. During cycle 7 the nonlinear response of the specimen CSPSW-DL was observed, when the lateral shear load was 393.9 KN. Furthermore, in this loading group, loud friction sound between the infill steel plate and the precast light-weight concrete panels was heard. In 0.96% drift, the web plate yield of upper beam near right beam-to-column connection occurred.

In 1.44% drift, the lateral load was 638.96 KN and the initial hairy cracks appeared on the front light-weight concrete panel. In the next loading group, in 1.92% drift, the web plate of bottom beam yielded. By increasing the lateral displacement during cycles 18, in 2.4% drift, the web plate yield of both columns was detected. In cycle 23 with 3.36% drift, the local inelastic buckling of the upper beam flange close to left beam-to-column connection could be clearly observed, as shown in Fig. 11.

The maximum shear load was captured 842.8 KN in cycle 26, in 4.32% drift. Afterwards, the shear capacity of the system initiated to decrease slightly. Finally, the shear capacity of specimen CSPSW-DL decreased below 80% of the maximum shear load due to the fracture of right beam-to-column connection in 6.24% drift, as shown in Fig. 12a. Fig. 12b depicts the infill steel plate after the test. It can be clearly seen that the infill steel plate has undergone sever inelastic deformation with so many fractures.

From cycle 30 (5.28% drift) up to the end of test (6.24% drift), concrete crushing took place in the corner of the light-weight concrete panels because of the buckling of the fish plates, as demonstrated in Fig. 12a. Fig. 13a and b illustrate the light-weight concrete panels after the test.

4.4. Hysteretic behaviour of specimens

"Shear load-lateral displacement" and "lateral stiffness degradation" curves of specimens are illustrated in Fig. 14. It can be obviously seen that all three specimens have illustrated significantly ductile behaviour and stable hysteretic post-yielding performance in the inelastic regions.

Although there are infinitesimal differences in the hysteretic behaviour of CSPSW-N and CSPSW-L, it can be inferred that utilizing precast light-weight concrete panel in CSPSW is a solution to reduce the seismic mass of steel structures. Moreover, although specimen CSPSW-DL had

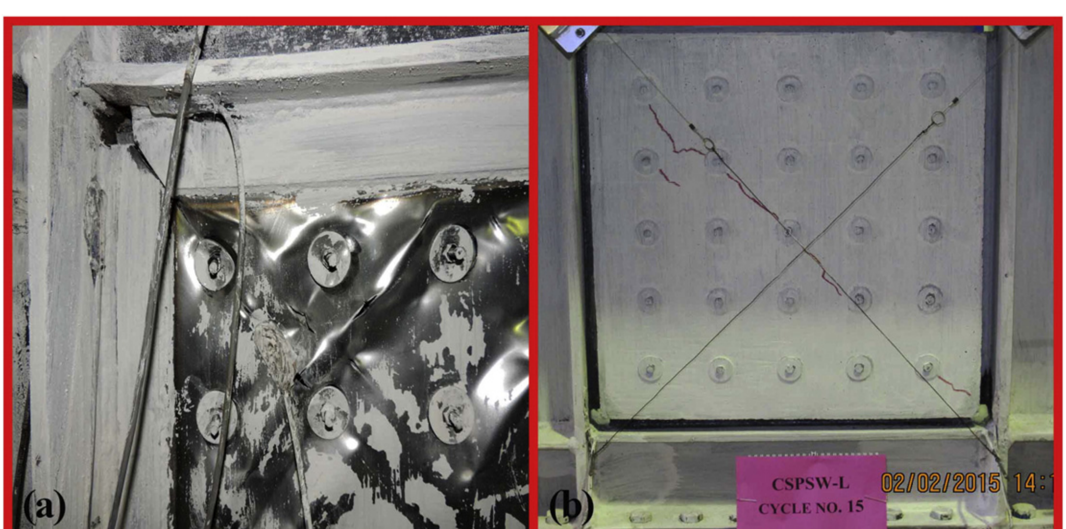

Fig. 9. Specimen CSPSW-L: (a) the fracture of the steel plate in the corner (b) initial diagonal hairy cracks.

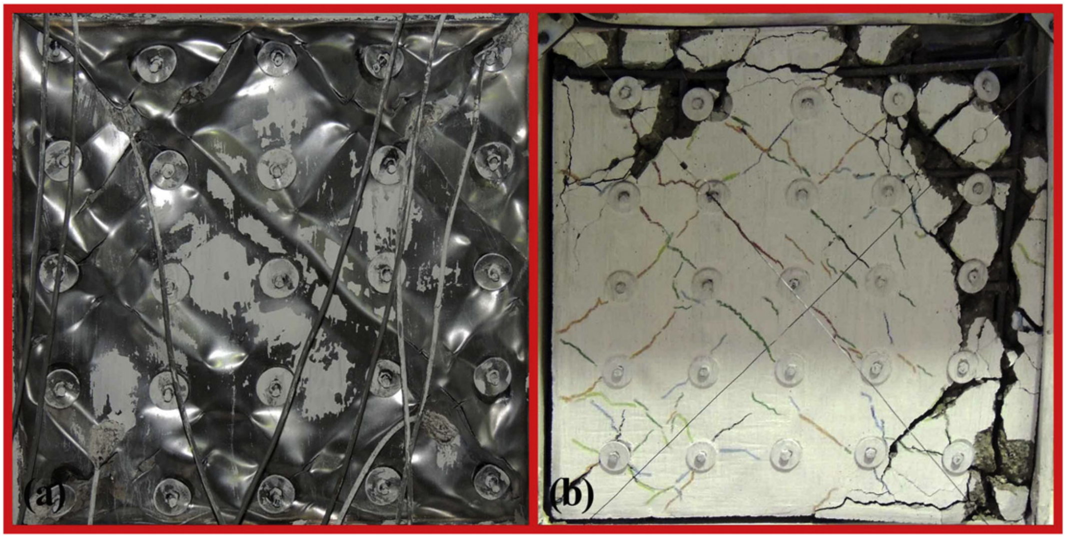

Fig. 10. Specimen CSPSW-L after the test: (a) the infill steel plate (b) the reinforced concrete panel.

two light-weight concrete panels on both sides of the infill steel plate, its behaviour is approximately similar to other specimens. Hence, when there is a gap between the reinforced concrete panel and the steel boundary elements, a stiff panel on one side is adequate.

Based on the Fig. 14d, the lateral stiffness degradation of specimens indicates that CSPSW specimens experience a sudden loss of shear stiffness, as the infill steel plate yields. The loss of shear stiffness of three specimens is approximately similar. In comparison with CSPSW-N and CSPSW-L, specimen CSPSW-DL provides greater lateral stiffness after shear yield of infill steel plate. However, lateral stiffness of this specimen declines to zero in 4.32% drift which is smaller than two other specimens. It should be noted that after 1.8% drift, the lateral stiffness of CSPSW-N is akin to CSPSW-L and they reach zero in 4.64% drift.

The proposed hysteretic model is depicted in Fig. 15. In accordance with the Fig. 15b, under minor lateral loading, CSPSW response is elastic until "point B″ and, in this phase, unless the lateral load of CSPSW is removed, the shear wall returns to its initial step which is "point A."

Increasing lateral load, beyond the "point B," the nonlinear behaviour of infill steel plate initiates and the stiffness of the system declines, as illustrated in Fig. 15b. In this phase, the shear wall reaches "point D" by unloading, in which there is merely plastic deformation on the infill steel plate. Since there is no elastic buckling of the infill steel plate, in the reverse cyclic reloading, CSPSW reach "point E" and after that steel plate undergoes inelastic yield in the opposite side.

While the shear load goes beyond shear capacity of the infill composite wall, the nonlinear response of steel frame commences that is "point C." As shown in Fig. 15c, the stiffness of composite shear wall decreases for the second time. At this stage, if the CSPSW is unloaded, it will reach "point I″ in which there are inelastic deformations of infill wall and steel frame. Under reloading in the reverse direction, the shear wall reaches "point J" where the lateral stiffness changes and after that reach "point F″ where the plastic response of the infill steel plate and frame starts in the opposite side. Finally, the cycles are repeated until the shear wall reaches its ultimate shear capacity, "point N." Beyond that sudden collapse will occur.

4.5. Energy dissipation of the system

One of the influential and paramount characteristics of a shear wall designed for high seismic loads is its energy dissipation capacity.

Fig. 11. The inelastic buckling of the upper beam flange of specimen CSPSW-DL in 3.36% drift: (a) front view (b) back view.

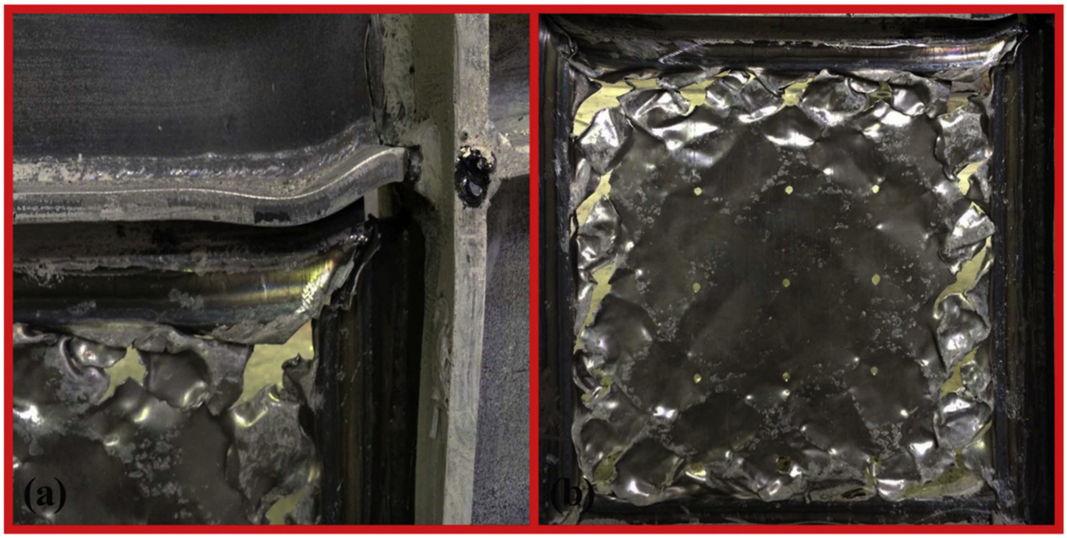

Fig. 12. Specimen CSPSW-DL (a) the fracture of right beam-to-column connection (b) back view.

Therefore, the accumulative dissipated energies of the specimens is evaluated and compared with each other. The dissipated energy by the system is calculated as the area surrounded in each loop, as shown in Fig. 16a, and the accumulative dissipated energy is the summation of dissipated energy in loops. The accumulative dissipated energies of the specimens CSPSW-N, CSPSW-L, and CSPSW-DL are 837.14 KN·m, 1018.49 KN·m, and 1069.46 KN·m respectively. Based on the obtained results, the cumulative energy absorption of specimen SPSW-DL is slightly higher than two other specimens.

Fig. 16b depicts the dissipated energies of the specimens during the cyclic test. In accordance with the graphs, the dissipated energies of three shear walls are the same up to the cycle 15. Beyond that cycle, the energy absorption of CSPSW-DL increases significantly. According to Fig. 16b, the energy dissipation of CSPSW-N and CSPSW-L are almost equivalent trough the lateral loading. The main reason is that the reinforced light-weight concrete panel of CSPSW-L can preclude global and local elastic buckling of the infill steel plate.

4.6. Structural properties

The structural responses of the specimens are investigated and compared to grasp behaviour of CSPSW specimens. Fig. 17 represents the comparison between the envelop curves of experimental results of three CSPSW specimens.

The ultimate inter-story drift is calculated as a drift in which the shear capacity of CSPSW specimen decreases below 80% of the maximum shear load recorded during the test. In accordance with cyclic tests' data (as shown in Fig. 17), all three specimens were capable of tolerating considerable inter-story drift between 5.04% and 6.24%. Specimen CSPSW-N with the normal weight concrete panel had maximum shear load capacity of 823.2 KN, on the other hand, the maximum shear load recorded for CSPSW-L with light-weight concrete panel was 809.5 KN. The difference between these shear load capacities was negligible; hence, it can be conclude that using light-weight concrete in CSPSW can reduce the mass without decreasing shear capacity of the system. The light-weight concrete panel was 36% lighter than normal-weight one.

In addition, structural properties of specimen CSPSW-DL with two light-weight concrete panels were almost similar to CSPSW-N and CSPSW-L. Thus it can be deduced that when concrete panels are utilized to prevent global and local elastic buckling of the infill steel plate, a stiff reinforced concrete panel on one side of steel plate is sufficient. Paramount structural properties of all three specimens are presented in Table 2.

Fig. 13. Specimen CSPSW-DL after the test: (a) front light-weight panel (b) back light-weight panel.

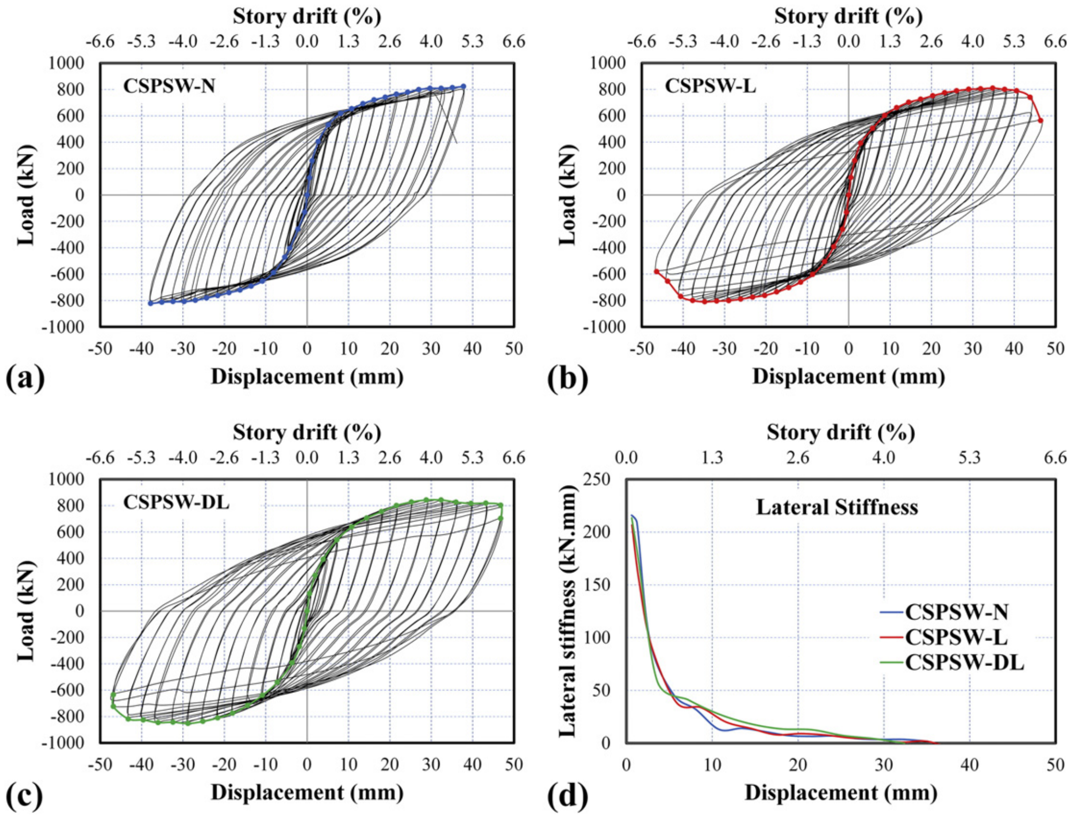

Fig. 14. (a) Hysteresis curves of CSPSW-N (b) hysteresis curves of CSPSW-L (c) hysteresis curves of CSPSW-DL (d) curves of lateral stiffness degeneration.

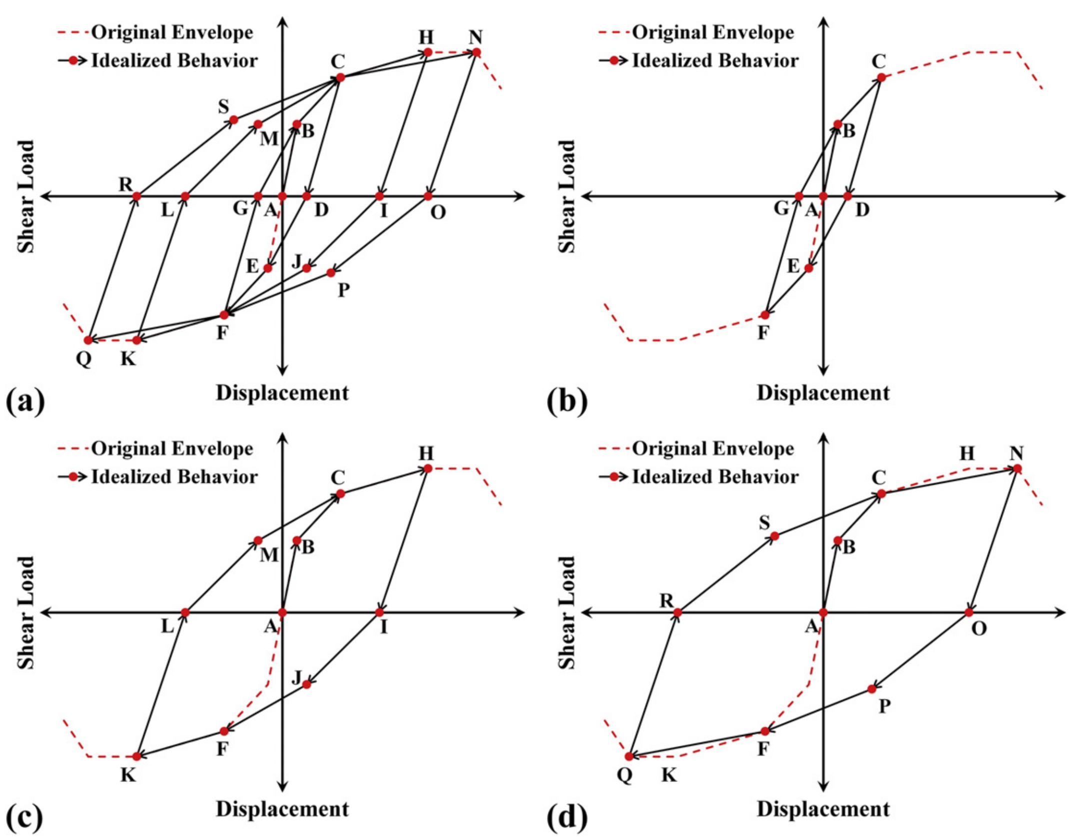

Fig. 15. (a) Proposed hysteretic behaviour of CSPSW (b) proposed hysteretic behaviour of CSPSW during the nonlinear response of infill steel plate (c) Proposed hysteretic behaviour of CSPSW during the nonlinear response of steel frame (d) Proposed hysteretic behaviour of CSPSW during the ultimate shear capacity.

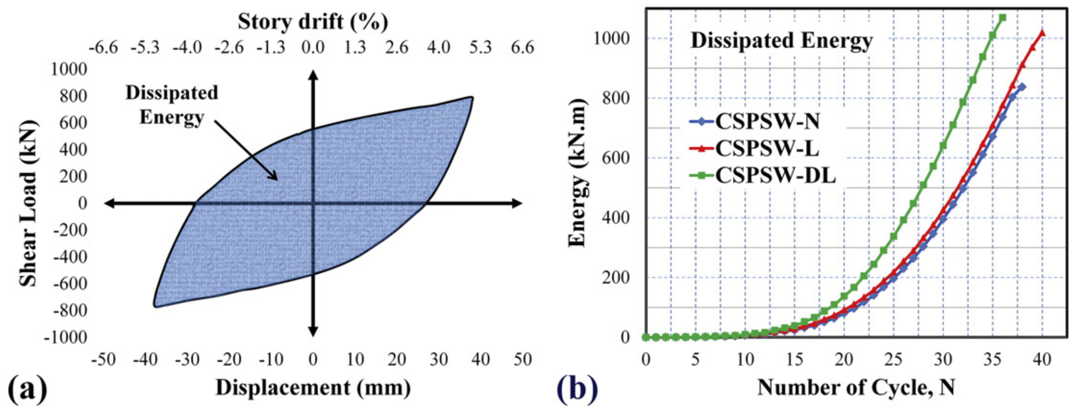

Fig. 16. (a) Calculation of energy dissipation in each cycle. (b) the dissipated energies of the specimens.

4.7. Strain analysis

According to finite element simulations developed before the tests, strain gauges were mounted on probable plastic locations. During the experiment the strains of those probable zones were monitored. According to collected strain data, the failure modes and behaviour of specimen CSPSW-N and CSPSW-L were nearly similar.

The specimens CSPSW-N and CSPSW-L were approximately elastic in the first six cycles. After that, the obtained outputs from strain gauges installed on the infill steel plates of the both specimens depict that the distinguishable initiation of inelastic behaviour was in cycle 7, as shown in Fig. 18. Based on the figure, the infill steel plates initiated inelastic responses as the strain value had surpassed the yield limit in cycle 7. It can be clearly seen that the infill steel plate of CSPSW undergoes plastic deformation at the expected level and dissipate markedly the seismic energy.

In specimen CSPSW-DL, no strain gauge was installed on the infill steel plate because both light-weight concrete panels were precast and the friction between the steel plate and concrete panel would destroy the strain gauges. By contrast, the strain gauges mounted on the steel boundary elements showed an acceptable behaviour.

5. Development of Finite Element Modelling (FEM)

Finite element models of the specimens were developed before the tests manipulating the commercially available finite element software ABAQUS [24]. After the experimental investigations, the models were modified according to results and observations collected during the tests.

Due to complexity of achieving numerical convergence and steel-concrete interaction, Dynamic/Explicit was utilized to conduct nonlinear push-over analysis. 4-node shell element (S4R) was chosen for the infill steel plate and 8-node solid element (C3D8R) was selected for steel boundary elements and the concrete panel. In addition, 2-node three-dimensional beam element (B31) and 2-node three-dimensional truss element (T3D2) were utilized for shear connectors and reinforcement respectively.

As tensile coupon tests were accomplished for the steel materials and the compressive tests were done for concrete materials, those attained results were used in finite element model. More details about the material nonlinearity, steel-concrete interaction, shear connectors, loading procedure, and boundary conditions can be found in the previous work of the authors [16].

As depicted in Fig. 19, the obtained finite element results were compared with the envelope curves of test results. In addition, the behaviour of specimens observed during the tests was compared to finite element models. Fig. 20 illustrates the inelastic local buckling of the infill steel plate of specimen CSPSW-L. It should be noted that although the global and local elastic buckling of the infill steel plate was occluded by the reinforced concrete panel in the test, the inelastic local buckling was captured in the places between the bolts. This phenomenon was observed in the finite element model as well.

In accordance with finite element models, numerical study can reasonably predict the overall behaviour of CSPSW. Although there are some differences between numerical and experimental results, the numerical results of the initial elastic lateral stiffness, shear capacity are mostly in a good agreement.

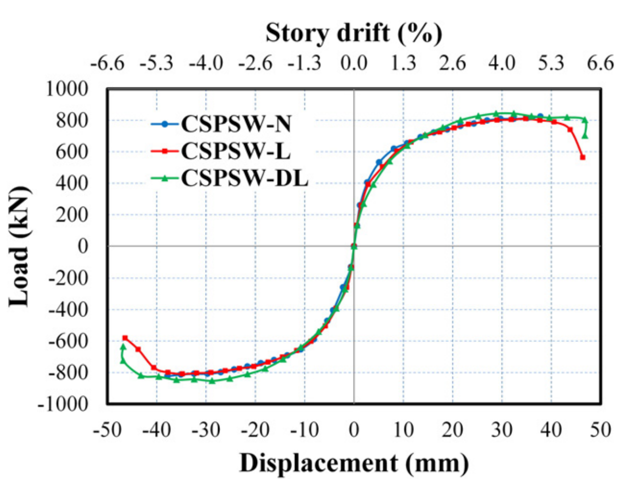

Fig. 17. Envelop curves of three CSPSW specimens.

6. Conclusion

In this paper, experimental and numerical studies on CSPSW using light-weight concrete panel were conducted to investigate the impact of light-weight concrete panel on seismic performance of the system. In this regard, three one-story one-bay CSPSW specimens were designed, fabricated, and tested. Steel parts of all three specimens were

Table 2
Structural properties based on the cyclic tests.

Specimen	P_y (kN)	Δ_y (mm)	P_M (kN)	Δ_M (mm)	P_u (kN)	Δ_u (mm)	θ_u (%)
CSPSW-N	385.4	2.7	823.2	37.8	823.2	37.8	5.04
CSPSW-L	367.3	2.9	809.5	34.8	572.3	46.4	6.19
CSPSW-DL	393.9	3.6	826.1	36.0	627.1	46.8	6.24

Note: $\mathbf{P_y}$: the yield lateral load, $\mathbf{\Delta_y}$: the displacement at the yield point, $\mathbf{P_M}$: the maximum tolerated shear load, $\mathbf{\Delta_M}$: The displacement at the maximum load, $\mathbf{P_u}$: the ultimate shear load, $\mathbf{\Delta_u}$: the displacement at the ultimate point, $\mathbf{\theta_u}$: the ultimate inter-story drift.

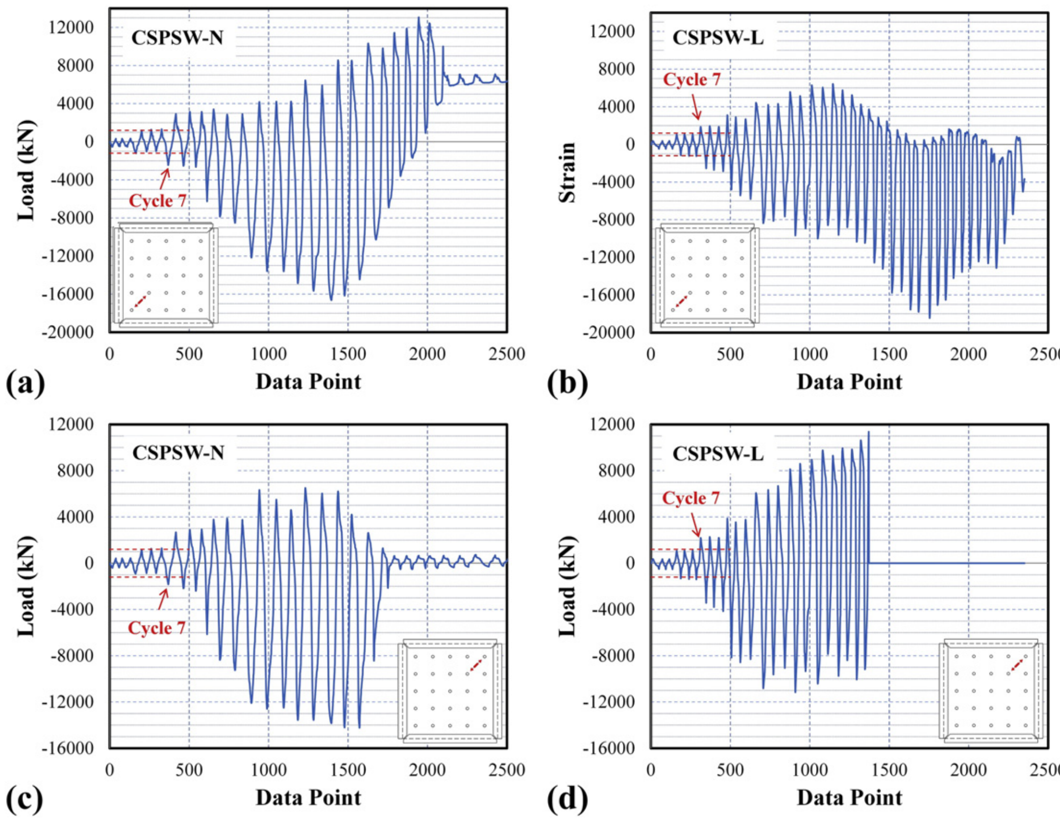

Fig. 18. Strain-gauge outputs of the infill steel plate ($\varepsilon.10^{-6}$).

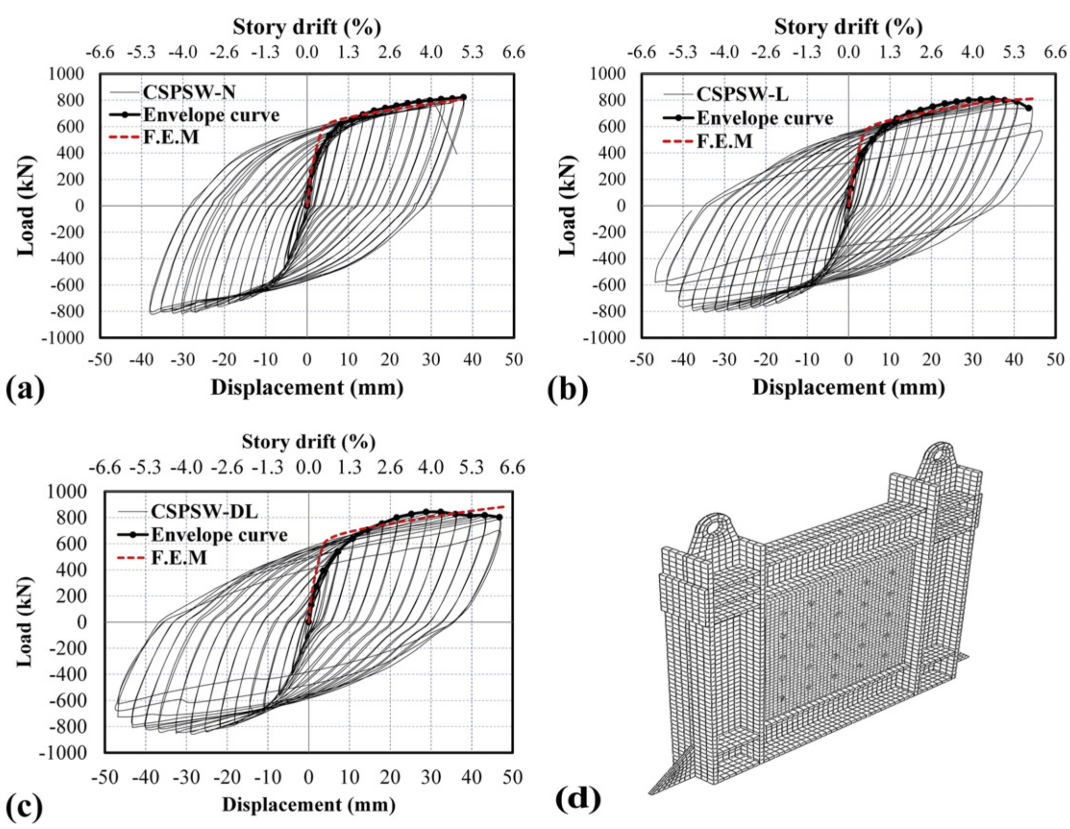

Fig. 19. Finite element results (a) specimen CSPSW-N (b) specimen CSPSW-L (c) specimen CSPSW-DL (d) finite element model of CSPSW-L.

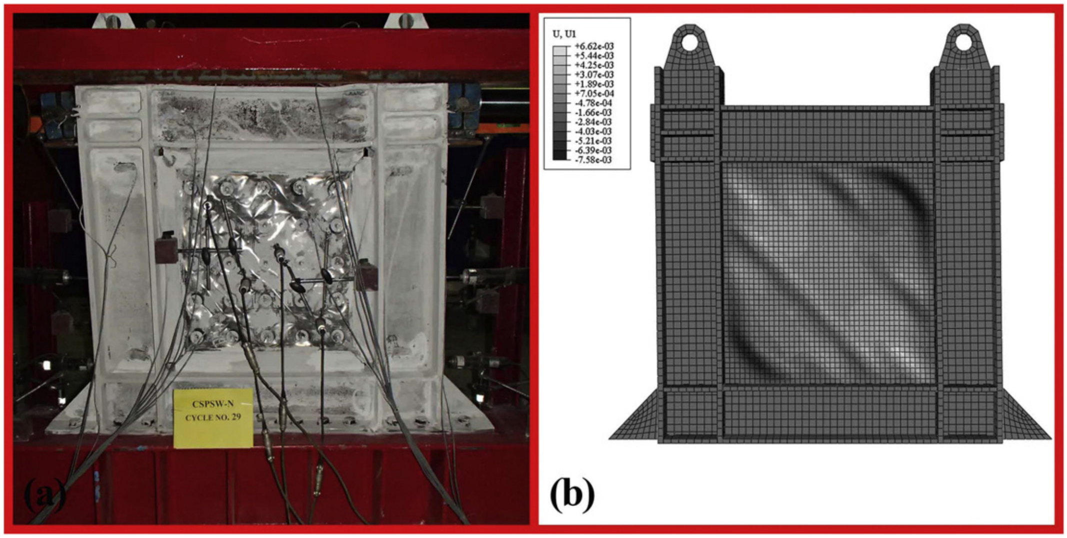

Fig. 20. Local inelastic buckling of the infill steel plate of specimen CSPSW-L (a) experiment (b) finite element model.

similar, but their reinforced concrete panels were different. Finite element models were developed before the tests to grasp not only the overall behaviour, but also the yield point of specimens. After the tests they were modified based on the experimental results and observations.

- In accordance with cyclic test observations and results, specimens had stable hysteretic behaviour and adequate energy dissipation. It should be mention that no pinching in the hysteretic curves of specimens was observed while undergoing large lateral displacement.
- CSPSW provides high initial elastic stiffness owing to the present of concrete panel; furthermore, if the system is designed properly, the infill steel plate can undergo inelastic deformation and dissipate considerable seismic energy in steel structures.
- It should be noted that in CSPSW specimen, the infill steel plate yielded entirely through the lateral loading; by contrast, this phenomena happen rarely in SPSW.
- All three specimens—CSPSW-N, CSPSW-L, and CSPSW-DL—showed very high cyclic ductility, which tolerated inter-story drift of 5.04%, 5.80%, and 6.24% respectively.
- The finite element models of three specimens modified after the test indicate there is a proper agreement in numerical results and experimental data. It is attested that the finite element method can reasonably predict the behaviour of CSPSW with light-weight concrete panels.
- Although further investigations are essentially demanded on CSPSW utilizing reinforced light-weight concrete panel, in accordance with the conducted study, light-weight concrete panels can be used as a stiffener instead of normal-weight concrete panel. In addition, this shear wall can be presumed a proper lateral load-resisting system which reduces a considerable seismic mass and improve the behaviour of steel structures. It should be noted that light-weight concrete was 36% lighter than normal-weight concrete.

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