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Numerical investigation on the behavior of SHS steel frames strengthened using CFRP

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Abstract. Steel frames are widely used in steel structures. Exiting steel structures may be needed to strengthen for various reasons. Carbon Fiber Reinforced Polymers (CFRP) is one of the materials that are used to strengthen steel structures. Most studies on strengthening steel structures have been done on beams and steel columns. No independent study, to the researcher's knowledge, has studied the effect of CFRP strengthening on steel frames. This study explored the use of CFRP composite on retrofitting square hollow section (SHS) steel frames, using numerical investigations. Ten Finite Element (FE) models, which were strengthened with CFRP sheets, were analyzed under different coverage length, number of layers, and location of CFRP composite. One FE model without strengthening was analyzed as a control FE model to determine the increase of the ultimate load in the strengthened steel frames. ANSYS software was used to analyze the SHS steel frames. The results showed that the coverage length and the number of layers of CFRP composite have a significant effect on increasing the ultimate load of the SHS steel frames. The results also showed that the location of CFRP composite had no similar effect on increasing the ultimate load and the amount of mid span deflection of the SHS steel frames.

Keywords: SHS steel frame; CFRP; strengthening; ultimate load; numerical method

1. Introduction

Strengthening of hollow steel sections using CFRP sheets has attracted greater attention in recent times. Carbon fiber reinforced polymer is preferred to strengthen and retrofit hollow steel sections, due to its much higher elastic modulus and ability to be applied to any shape of structure. The use of CFRP is also a perfect solution in order to overcome the existing short comings and strengthen certain infrastructures such as bridges (ACI 440 2002). Over the past decades, some studies have been done on strengthening and retrofitting of steel columns (Teng and Hu 2007, Bambach *et al*. 2009, Haedir and Zhao 2011, Fanggi and Ozbakkaloglu 2015, Xie and Ozbakkaloglu 2015, Kim and Harries 2011, Keykha *et al*. 2015, Linghoff *et al*. 2009). Some other studies have been done on flexural strengthening, shear, tensile and torsional of steel beams (Deng *et al*. 2004, Youssef 2006, Islam and Young 2013, Al-Zubaidy *et al*. 2013, Abdollahi Chakand *et al*. 2013, Photiou *et al*. 2006).

Sundarraja and Prabhu (2011) strengthened the hollow steel beams, which were filled with concrete, using CFRP composite and then they tested. The results showed that the strengthened beams by full wrapping exhibited more enhancements in stiffness and moment carrying capacity. They also presented an economical method for strengthening of the hollow steel beams that were filled with concrete.

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In a similar study, Al Zand *et al*. (2015) strengthened the square CFST (concrete-filled steel tube) beams. The results showed that, for all strengthened CFST models using one layer of CFRP sheet, CFRP had no significant enhancement in the ultimate load values when wrapped along 50, 75 and 100% of the length of samples.

Steel members in compression strengthened using mortar-filled FRP tubes were tested by Feng *et al*. (2013). Feng *et al*. (2013) showed that after strengthening of steel members, non-dimensional slenderness of specimens reduced and buckling resistance increased. They obtained the ductility capacity by deformation. Also, they defined the ultimate deformation-to-the yielding deformation ratio as the ductility capacity. Their test results indicated that, the mortar-filled FRP tubes can enhance the load-bearing of steel members by 215% and the ductility capacity of steel members by 877%.

Keykha *et al*. (2016a) strengthened the slender square hollow section steel columns with different boundary conditions using CFRP. They offered a theoretical method to analyze these columns. The results showed that the CFRP composite did not have the same effect on columns with different conditions.

In another study, Keykha *et al*. (2016b) strengthened the SHS short steel columns with the use of adhesively bonded CFRP flexible sheets. They showed when the composite coverage percentage is less than 100% CFRP, it is not effective in the ultimate load capacity of the SHS short steel columns. The reason of no increase in the ultimate load capacity of these columns is that the failure mode of the columns happened in the outside location of the strengthened area.

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Recently, Keykha (2017) strengthened the slender and stocky SHS steel columns with the use of adhesively bonded CFRP flexible sheets under eccentric compression load, using numerical investigations. Due to the eccentricity of loads, these columns were located under the combination of pressure and bending. Finite Element Method (FEM) was employed for modeling. ANSYS was used to analyze the SHS steel columns. The results showed that the CFRP composite had no similar effect on the slender and stocky SHS steel columns under the eccentric compression load. The results also showed that the coverage length, the number of layers, and the location of CFRP composites were effective in increasing the ultimate load capacity of the SHS steel columns under the eccentric compression load.

Chen *et al*. (2015) investigated the flexural behavior of rectangular hollow section (RHS) steel beams with initial crack strengthened externally with CFRP plates. The results showed that yield loads of cracked beams could be enhanced with repairing. Meanwhile, the ultimate loads were increased to some extent. The effect of repair became significant with the increase of the initial crack depth. The failure patterns of the repaired specimens were similar to those of the control ones. It could be concluded that the patching repair could be used to restore the load bearing capacity of the deficient steel beams.

Awaludin and Sari (2015) carried out a numerical and experimental investigations on the nonlinear behavior of the SHS (cross-section 100 mm × 100 mm; thick 2.1 mm; length 2000 mm) steel beams. The SHS steel beam of investigated in this research were having an artificial crack (width 3 mm, depth 25 mm) at mid-span on tension side and externally repaired with CFRP sheet. Their studies showed that repair of the SHS steel beams having artificial crack with CFRP sheets is a suitable solution of increasing the ultimate load capacity of these beams. In beam B (the repaired beam with CFRP sheet of 1000 mm length at bottom face), CFRP sheet was not very effective on the ultimate load capacity, duo to the artificial crack was not completely covered with CFRP sheet.

From the past studies, it can be observed that some studies have done with the use of CFRP as a strengthening material for SHS steel members and also the presence of CFRP significantly enhance the behavior of the SHS steel members. It seems that there is a lack of understanding on the behavior of the SHS steel frames strengthened using CFRP sheets. Thus the main focus of the study is the numerical investigation on the behavior of the SHS steel frames strengthened using CFRP sheets. The coverage length, the number of layers, and the location of CFRP composite were varied to examine the ultimate load of the SHS steel frames.

2. Materials used

2.1 SHS steel tube

The square hollow steel tube having a dimension of 90 mm × 90 mm was used in this study. The thickness of the square hollow steel tube was 2.5 mm. The SHS steel tubes had a yield stress of 280 MPa, an ultimate stress of 375 MPa, and the modulus of elasticity about 200 GPa which were predicted from the experimental values (taken from research of Keykha *et al*. (2015)).

2.2 CFRP composite

In this study, SikaWrap-230C was used. It is a unidirectional carbon fibre reinforced polymer with a nominal modulus of elasticity of 238 GPa, a nominal tensile strength of 4300 MPa, and a nominal thickness of 0.131 mm that provided by the manufacturer.

2.3 Adhesive

The adhesive in this study was Sikadur-330. The Sikadur-330 have a nominal tensile strength of 30 MPa and a nominal modulus of elasticity of 4500 MPa. This type of adhesive is a two part systems: a hardener and a resin. In this type of adhesive mixing ratio is 1:4.

3. Numerical simulation

3.1 Method description

To model the steel frames, three dimensional (3D) simulation (Sundarraja and Prabhu 2011, Al Zand *et al*. 2015, Keykha *et al*. 2016a) using ANSYS software was performed. The SHS steel frames, CFRP sheets, and adhesive were simulated by using the 3D solid triangle elements (ten-nodes 187). Non-linear static analysis was carried out to simulate the failures. In this case, the load was applied incrementally until the plastic strain in an element reached to its ultimate strain (element is "killed"). Linear and non-linear properties of materials were defined based on the stress-strain curves obtained from the experimental values. The CFRP sheets material properties were defined as linear and orthotropic because CFRP materials have linear properties and they were unidirectional (Linghoff *et al*. 2009). The steel frames and adhesive were defined as the materials having non-linear properties. For meshing, the map meshing were used. Therefore, for analysis of the specimens from the solid element of 187 with the mesh size of 25 was used (Keykha *et al*. 2016a, b, Keykha 2017).

3.2 Validity of software results

It is necessary to validate the calculation of software. In this research, the software results have been validated and calibrated by the experimental results of Awaludin and Sari (2015). As mentioned in the introduction, Awaludin and Sari (2015) used the SHS steel with a dimension of 100 mm × 100 mm and thickness 2.1 mm. These dimension and thickness are close to the dimension and thickness (dimension 90 mm × 90 mm and thickness 2.5 mm) of the SHS steel were used in this study. In numerical method, the beams loaded as in experimental (as conducted by Awaludin and Sari 2015) and the same conditions were analyzed (as shown in Figs. 1-3). The results of analysis show that, there are a good agreement between experimental and numerical values, with comparing the ultimate load of beams as showed in Table 1.

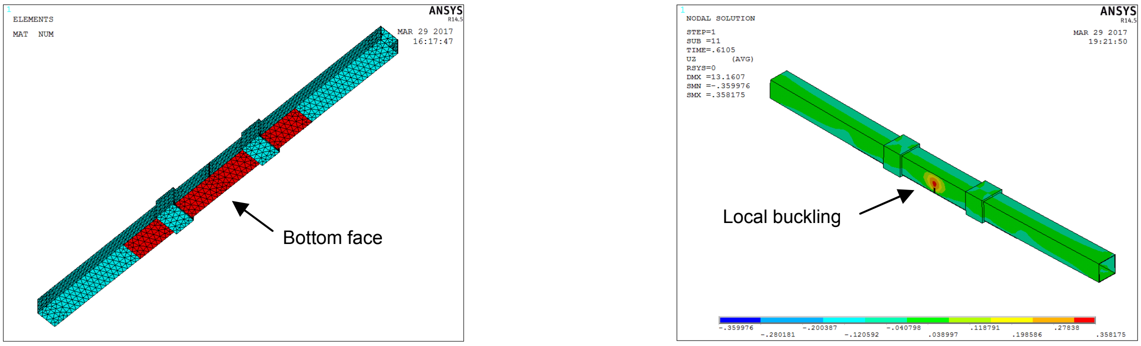
As shown in Fig. 1(a), in the beginning the applied load through a load cell is transmitted to loading beam and then by two blocks is transmitted to the SHS steel beam. In order to increase the accuracy of calculations, the simulation of the SHS steel beams was conducted according to laboratory conditions as shown in Fig. 3.

3.3 Description of FE models

The analyzed frames include one control FE model and ten FE models strengthened with one and two CFRP layers. The CFRP sheets were pasted on bottom and/or all four corner sides (beam-column connection) ten FE models from the SHS steel frames (Fig. 4). To determine the increase of the ultimate load in strengthened steel frames one FE model was considered (control FE model) without CFRP pasting. To identify the FE model easily, the SHS steel frames were designated by the names such as FC0, FC1-700, FC1-1000,

(a) (b)

Fig. 1 Deformed shape of beam A: (a) experimental (Awaludin and Sari 2015); (b) FE model



(a) (b)

Fig. 2 FE modeling of Beam B: (a) showing the CFRP location on SHS steel; (b) Deformed shape

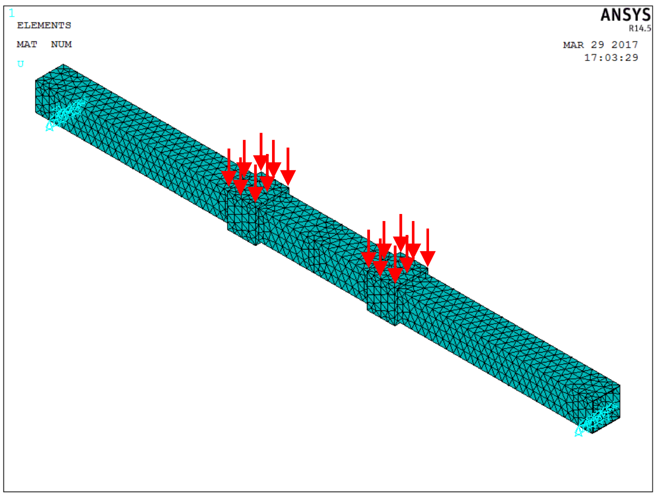


Fig. 3 Typical loading and boundary condition assigned to SHS steel beam in ANSYS

Table 1 Comparison of the ultimate load of beams in both laboratory and FE analysis

Model label	CFRP sheet (mm^2)	Experimental (kN) (Awaludin and Sari 2015)	FE analysis (kN) (Awaludin and Sari 2015)	FE analysis (kN) (This study)
SHS beam (without crack)	NA	---	23.61	23.21
Beam A	NA	10.93	12.37	11.01
Beam B	1000×100	13.17	---	14.12

FC1-1500, FC2-700, FC2-1000, FC2-1500, FC1-1000-180, FC2-1000-180, FC1-0-180 and FC2-0-180. For example, the FE model FC2-1000 indicates that it is strengthened by two layers and 1000 mm CFRP length on the bottom. The FE model FC1-1000-180 specifies that it is the SHS steel frame strengthened by one layer with 1000 mm CFRP length on the bottom and 180 mm CFRP length on the four corner sides, as illustrated in Fig. 4. Similarly, the FE model FC2-1000-180 specifies that it is the SHS steel frame strengthened by two layers with 1000 mm CFRP length on the bottom and 180 mm CFRP length on the four corner sides. Likewise, the FE model FC2-0-180 specifies that it is the SHS steel frame strengthened by two layers with 180 mm CFRP length on the four corner sides. The control FE model is named FC0 (the SHS steel frame without CFRP).

3.4 Model description

Nonlinear finite element models were prepared using ANSYS software to investigate the structural behavior of the SHS steel frames strengthened by CFRP sheets on bottom and/or four corner sides (at beam-column connections). All models were prepared as fixed supported frames with two pint loads applied at equal distance from each

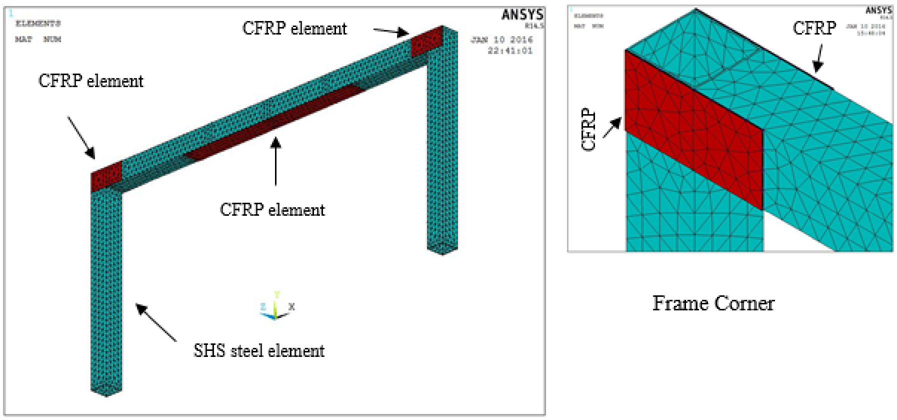


Fig. 4 FE modeling (FC1-1000-180)

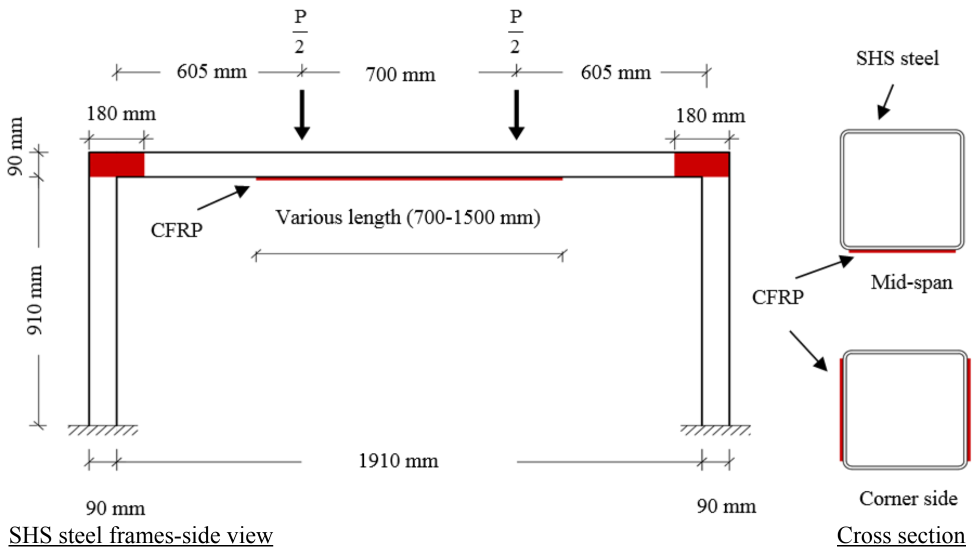


Fig. 5 Boundary conditions of a SHS steel frames strengthened with CFRP

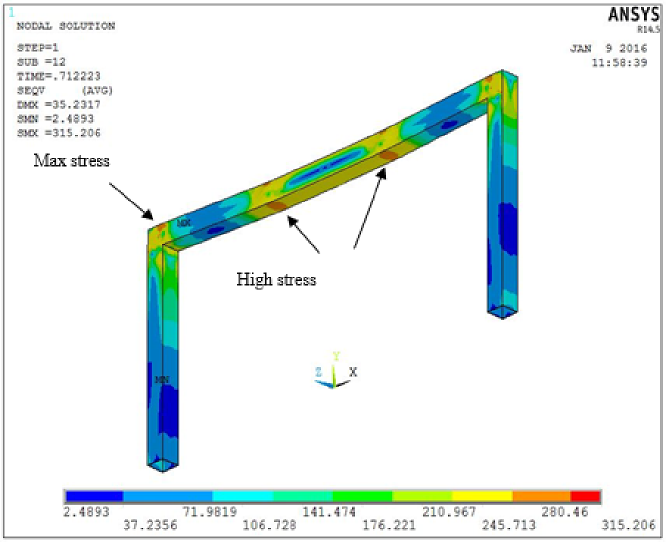


Fig. 6 Von Mises stress (FC0 FE model)

Table 2 FE model details and analysis results

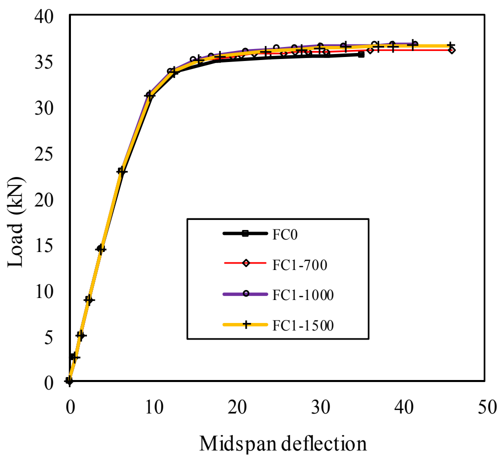
Designation of frames	Number of layers CFRP	Length of CFRP coverage (mm)		Ultimate load (kN)	% of increase in ultimate load
		In frame bottom	In frame corner		
FC0	0	0	0	35.611	0
FC1-700	1	700	0	36.153	1.52
FC2-700	2	700	0	36.171	1.57
FC1-1000	1	1000	0	36.787	3.30
FC2-1000	2	1000	0	37.696	5.85
FC1-1500	1	1500	0	36.703	3.07
FC2-1500	2	1500	0	37.694	5.85
FC1-1000-180	1	1000	180	40.489	13.70
FC2-1000-180	2	1000	180	42.777	20.12
FC1-0-180	1	0	180	38.756	8.83
FC2-0-180	2	0	180	40.678	14.23

column. Fig. 5 shows the condition of the SHS steel frames and the strengthening scenario adopted in this study. For example, Fig. 4 shows the 3D finite element model of the SHS steel frames prepared using ANSYS software (FC1-1000-180 FE model). The applied load was adopted in the finite element models to represent the applied pint loads. This load gradually increased until the strengthened SHS steel frames achieved their ultimate load capacity. In the beginning of modeling, the control FE model was analyzed. From this analysis was found the location of high tensile stress (Fig. 6). Due to the high tensile strength of CFRP composite, the SHS steel frames were strengthened in locations of high tensile stress, using CFRP (see Figs. 5 and 6).

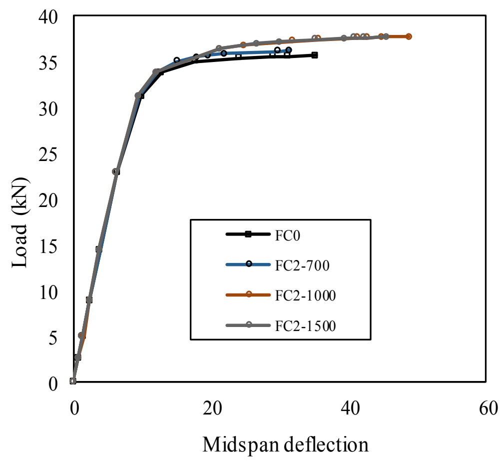
4. Results and discussions

4.1 Ultimate load results

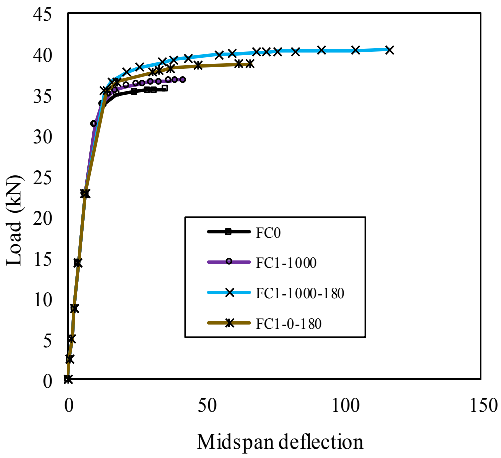
Table 2 shows the numerical analysis results of FE models with one and two layers of CFRP sheet. The coverage length of CFRP varies, based on the length of the frame. The center position of CFRP sheet is in the center of frame beam on the bottom or the four corner sides. The results showed that when the coverage length of CFRP is less than 700 mm (the distance between the applied loads), CFRP is not more effective in the ultimate load of the SHS steel frames strengthened with one and two CFRP layers than ones, duo to the area of high tension was not completely covered with CFRP sheet (see Fig. 6). The rate of the increase of the ultimate load in the SHS steel frames increases with an increase in the number of CFRP layers. When CFRP is located on the corner sides, CFRP is more effective in the ultimate load of the SHS steel frames. By combining CFRP wrapping on the corner and in length of the SHS steel frames, the ultimate load of the SHS steel frames can be increased. The maximum percentage of the increase in the ultimate load happened for the FE model FC2-1000-180 (20.12%). This FE model (FE model FC2-1000-180) was strengthened with two layers, and the coverage length CFRP sheet is 1000 and 180 mm on the



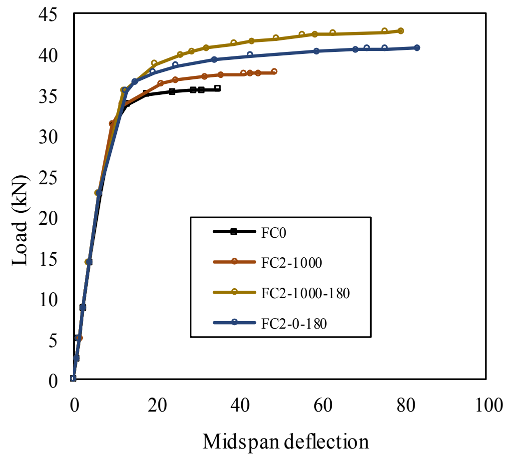
(a) FC0, FC1-700, FC1-1000 and FC1-1500



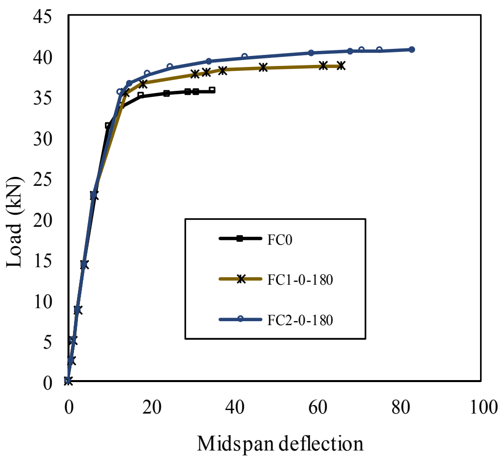
(b) FC0, FC2-700, FC2-1000 and FC2-1500



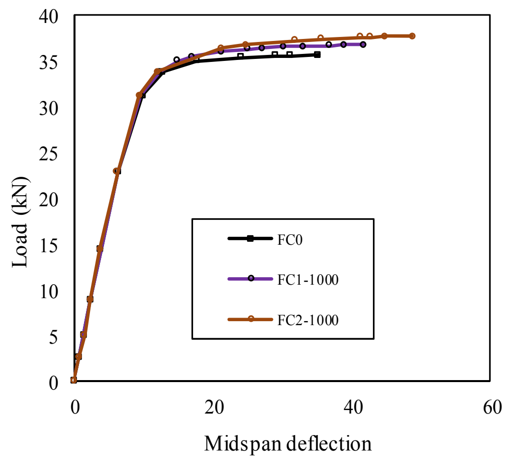
(c) FC0, FC1-1000, FC1-1000-180 and FC1-0-180



(d) FC0, FC2-1000, FC2-1000-180 and FC2-0-180



(e) FC0, FC1-0-180 and FC2-0-180



(f) FC0, FC1-1000 and FC2-1000

Fig. 7 Comparison of load–deflection behavior of non-strengthened and strengthened frames

bottom and the corner sides of the SHS steel frames, respectively.

As shown in Table 2, generally, in the SHS steel frames of thin walled under contracted loads (such as the SHS steel frames of this study) duo to the buckling failure is occurred at the top flange, the CFRP sheet is not very effective in the ultimate load capacity, while the CFRP sheet is located at the bottom face of the SHS steel frames.

4.2 Axial load-displacement behavior

To investigate of axial load–displacement behavior of the SHS steel frames, the results were also presented in Figs. 7(a)-(f). At the initial stage, the control and CFRP reinforced frames exhibited linear elastic behavior followed by inelastic response when the load is increased. Furthermore, it was observed that the reinforced frames

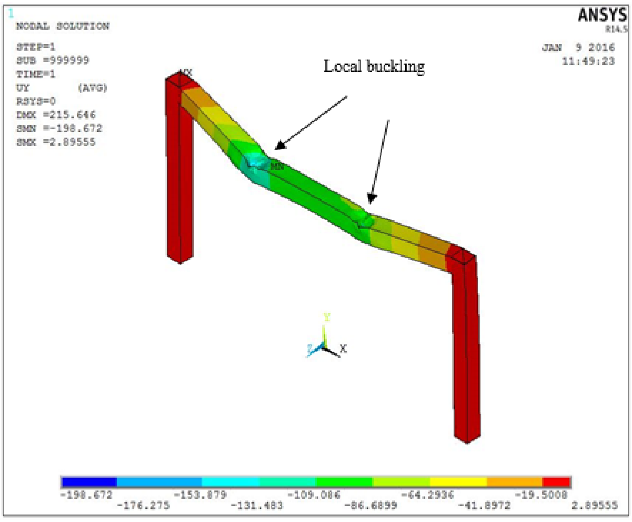


Fig. 8 Failure modes of FC0 FE model

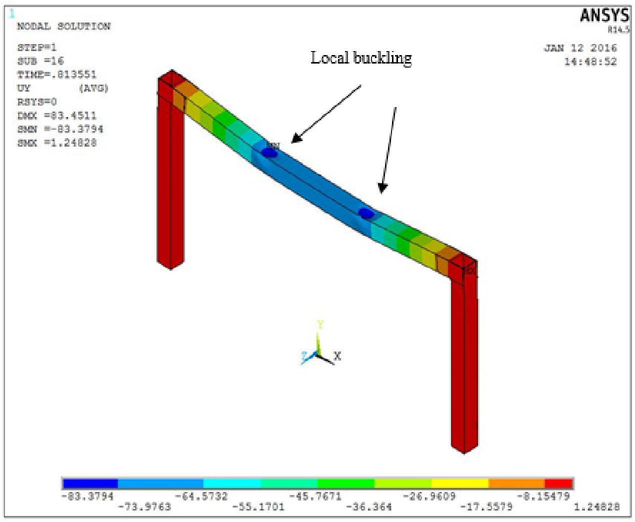


Fig. 9 Failure modes of FC2-1000-180 FE model

sustained higher ultimate load, compared to control frame. Fig. 7(d) shows that the greatest increase in the ultimate load carrying capacity of the SHS steel frames is for the FE model FC2-1000-180.

4.3 Ductility

The ductility capacity can be determined in different ways, such as by rotation, curvature and energy, displacement and so on. Feng *et al*. (2013) obtained the ductility capacity by deformation. They defined the ultimate deformation-to-the yielding deformation ratio as the ductility capacity. Fig. 7(c) shows that, the strengthening of the SHS steel frames using CFRP sheets improves the ductility capacity. The highest ductility capacity was observed in the FE model FC1-1000-180.

4.4 Failure modes

All FE models were subjected to two pint loads on the top of frame beams until failure. Two local buckling in the bottom applied loads were noted in all FE models (as shown in Figs. 8 and 9) at the ultimate load (the ultimate load of the SHS steel frames is shown in Table 2). The failure mode of all samples was the same. For example, failure modes of the FE models of FC0 and FC2-1000-180 are shown in Figs. 8 and 9.

5. Conclusions

In this research, CFRP layers were pasted with different coverage length on the bottom and/or the four corner sides of the SHS steel frames to enhance the structural performance. Based on the obtained results, the failure modes, the ultimate load carrying capacity, and the role of CFRP fabrics on the SHS steel frames were discussed. Based on eleven analyzed FE models that ten FE models were strengthened with different coverage length and with one and two CFRP layers, the following conclusions can be drawn:

- For the SHS steel frames, the ultimate load carrying capacity increased if CFRP composite was located in area of maximum tensile stress. The maximum tensile stress area in the studied frames located in the four corner sides (see Fig. 6).
- The number of layers and the coverage length of CFRP influences on rate of the ultimate load carrying capacity of the SHS steel frames. This coverage length depends on the length of the tension zone. In this study, the coverage length on bottom frames was from 700 to 1000 mm. The results showed that the most appropriate coverage length was 1000 mm.
- CFRP composite location has a positive effect on increasing the ultimate load carrying capacity of the SHS steel frames. In this research, when CFRP composite was located on the corner sides of the SHS steel frames, CFRP was more effective on rate of the ultimate load carrying capacity of the SHS steel frames. (Compared the FE model FC1-1000 with the FE model FC1-0-180, and the FE model FC2-1000 with the FE model FC2-0-180).
- The maximum ultimate load of the SHS steel frames happened in the FE model FC2-1000-180. The maximum percentage of the increase in the ultimate load for this FE model was about 20%.
- The results showed that the CFRP strengthening significantly increases the ductility capacity for all the SHS steel frames. Among all FE models, the highest ductility capacity was observed for the FE model FC1-1000-180. In this FE model, percent of the increase in the ductility capacity was about 826% (see Fig. 7(c)).
- Generally, in the SHS steel frames of thin walled under contracted loads, duo to the buckling failure is occurred at the top flange, the CFRP sheet is not very effective in the ultimate load capacity while the CFRP sheet is located at the bottom face of the SHS steel frames.

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