IABSE-JSCE Joint Conference on Advances in Bridge Engineering-III, August 21-22, 2015, Dhaka, Bangladesh. ISBN: 978-984-33-9313-5
Amin, Okui, Bhuiyan, Ueda (eds.) www.iabse-bd.org

Numerical and experimental study on repaired steel beam using carbon fiber reinforced polymer

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ABSTRACT: Fiber Reinforced Polymers (FRPs) has been extensively used in structural rehabilitation. Previous experimental studies have confirmed the effectiveness of externally bonded FRPs to repair and strengthen steel structures. To obtain an accurate prediction, finite element model must capable to properly simulate interfacial stress and strains developed between FRP sheet and repaired structures. This paper presents study on both experimental and non-linier finite element analysis (FEA) of Square Hollow Section (cross-section 100 mm x 100 mm; thick 2.1 mm; length 2000 mm) beams having an artificial crack (width 3 mm, depth 25 mm) at mid-span on tension side and externally repaired with CFRP (SCH-41, thickness 1 mm). There were three beam specimens: one beam without repair (beam A); one beam reinforced with CFRP sheet 1000 mm length at bottom face (beam B); and the last one reinforced with CFRP sheet 1000 mm length at bottom face and both side faces of 25 mm depth (beam C). The steel beam has yield strength of 327 MPa and modulus of elasticity of 210 GPa, while the unidirectional CFRP sheet has tensile strength of 3800 MPa and modulus of elasticity of 230 GPa. Poisson's ratio of the steel beam and CFRP are 0.3 and 0.27, respectively. Epoxy adhesive was used to attach the CFRP sheet to steel beams, and all beam specimens were monotonically loaded until failure according to four-point bending test configuration. Collapse analysis of 3D finite element model was performed using ADINA, where CFRP sheet was defined as tension material and connect via gluemesh to steel beam. The test results showed that beam A failed due to local buckling and cracked opening at mid-span having bearing load capacity of 10.93 kN at mid-span displacement equaled to 14.13 mm. These results well agreed with the FEA though the bearing load capacity given by FEA is slightly higher, which is 12.37 kN at displacement equal to 12.98 mm. In the case of beam B, de-bonding failure was observed at maximum bearing load capacity of 13.17 kN. This increase due to CFRP reinforcement is not much and far less the load-bearing capacity of the steel beam without crack predicted by FEA (23.61 kN at displacement equals to 12.14 mm). Therefore, the beam C was intentionally reinforced at both ends to postpone de-bonding failure. The bending test showed this new beam C had bearing load capacity 20.87 kN at displacement equal to 13.95 mm. this study indicates that repair of steel beams with CFRP sheets is an effective means of increasing bearing load capacity of steel beams when de-bonding is successfully postponed.

1 INTRODUCTION

In aggressive environment such as along coastal line steel structures deteriorate due to corrosion which finally leads to reduction of its cross-sectional area, stiffness, and strength as well as its service life. Repair or strengthening is required to extend the service life of deteriorated buildings and this might be considered far less expensive than building renewal. Some examples of conventional method of steel structures repair are steel plate bonding, concrete jacketing, or sprayed concrete, etc. Repair of steel structures using additional steel plates (and welded) to the existing structures is not desirable in some situations as this introduce additional dead weight, interrupt building operation due to heavy equipments or even is prohibited as welding emits flame. Repair using fiber reinforced polymers (FRP), specifically carbon fiber reinforced polymers (CFRP) is more desirable as it does not require heavy equipment and is performed while the structure is in use.

A comprehensive review of state-of-the-art of FRP/ CFRP utilization for rehabilitation of steel structures was presented by Tawfik et al (2010) and Zhao et al (2005). CFRP basically is a composite material consisting of carbon fibers that provide strength, stiffness, and load carrying capacity, and a polymer matrix. Engineering properties of FRP depend on types and orientation of carbon fiber, type and percentage of resin material and curing condition. Shaat et al (2003) investigated of the strengthened and repaired of steel structures using FRP. They reported that FRP sheets are effective restored the load capacity of a damaged steel beam, resist higher loads, extend their fatigue life and reduce crack propagation, if adequate bond is provided and galvanic corrosion is prevented. In 2014, Wakabayashi et.al investigated repair method with CFRP to the corroded steel girder ends and they found that the necessary number of CFRP sheets should be determined from the thickness of CFRP sheet converted to steel similar to uniaxial compression test column.

In a notched beam model as commonly used to represent a corroded or deteriorated steel beams, stress concentrations at damage location result in local debonding of the CFRP sheet, followed by complete debonding failure of the sheet. Kim and Brunell (2011) showed that these failure modes are found to be independent of the level of initial damage (notch depth). In 2014 Hmidan et al performed finite element analysis to predict stress singularity in the crack-tip of wide-flange steel beam repaired with CFRP and they found that CFRP reinforcement was effective to reduce the stress intensity of the steel beams. In addition to steel yielding failure mode, Islam et al (2014) pointed out that there are three possible failure modes of FRP repaired steel beam are: adhesion or FRP debonding, interlaminar failure of FRP plate, and combination of adhesion and interlaminar failure of FRP plate. Yu et.al (2011) showed that de-bonding failure in FRP repaired steel beams can be postponed when strong bond between CFRP-steel interfaces is provided and this can be performed by reducing the laminate thickness, the laminate modulus and the adhesive thickness or by increasing the bond length.

This study is dealing with CFRP repaired steel beams having artificial crack at the tension side to simulate both strength and stiffness reduction. Full-scale experiments on CFRP strengthened beams under four-point bending was first carried out followed by finite element analysis developed using ADINA software. Besides load-bearing capacity increase, debonding failure was also discussed.

2 STEEL BEAMS AND CFRP MATERIALS

Square hollow section (SHS) steel beam having cross-section of 100 mm x 100 mm and 2.1 mm width was used in this study. Three steel beams which have an artificial crack in mid span at tension side were prepared. Detailed crack geometry and size of CFRP sheet of each beam are presented in Table 1. Beam A is the beam without repair, beam B is the repaired beam with CFRP sheet of 1000 mm length at bottom face, the last is beam C is the repaired beam with CFRP sheet of 1000 mm length at bottom face and both side faces of 25 mm depth. CFRP type SCH41 was selected for this study. SCH41 is unidirectional carbon composite having tensile strength of 3800 MPa and modulus of elasticity of 210 GPa according to technical specification published by the company. This CFRP sheet is wrapped on the SHS beams with epoxy adhesive.

Table 1. Beam specimens details

Beam	CFRP sheet (mm^2)	Crack width (mm)	Crack depth (mm)
A	NA	3	25
B	1000x100	4	25
C	1000x150	3	25

3 EXPERIMENT

Figure 1(a) shows the beams specimens after repaired with CFRP sheet (beam C). To ensure full utilization of the applied CFRP sheet, surface preparation of the SHS steel beam was cleaned using sand paper to enhance the formation of chemical bonds between CFRP and adhesive. The test was carried out one week after CFRP repair application.

All steel beams were monotonically loaded until failure according to four-point bending test configuration shown in Figure 1(b). At the load application points and supports, additional steel plates are placed as stiffener to ensure beam failure initiates at mid span. Length of the beam is 2000 mm and distance between the two supports is 1900 mm. Only at beam B, two strain gauges were attached to both side faces of steel beam at mid span at around 10 mm upper from the crack tips. Two displacement transducers (LVDT) were used to measure the beam displacement at mid span, while only one LVDT was used to measure beam displacement at each loading points. Beam displacement, magnitude of the applied load and strain measurement were continuously recorded and saved in a personal computer. Visual observation concerning beam failure mechanism was also performed during the test.

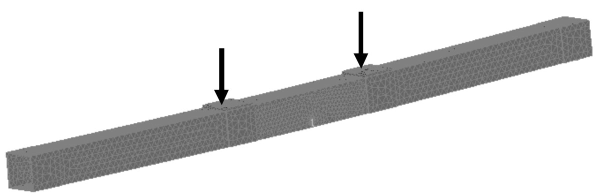
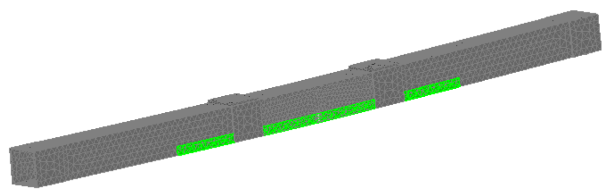
4 FINITE ELEMENT MODEL OF CFRP REPAIRED STEEL BEAM

The steel beam is modeled as elasto-plastic material in both tension and compression, while CFRP SCH-41 sheet is modeled as tension layer. Material properties of SCH-41 based on technical specification published by the company are: yield stress 3800 MPa, modulus of elasticity 230 GPa, and Poisson's ratio 0.27. Coupon test results of two specimens taken from the steel beam after completing the bending test informed that average yield and ultimate tensile strength is 327 MPa and 383 MPa, respectively. Modulus of elasticity of the steel beam is taken as 200, and Poisson's ratio of the steel beam is assumed equal to 0.3. In this finite element model developed by ADINA software, steel beam and CFRP sheet are connected at the interface via gluemesh option to accommodate the presence of glue layer. Another possible option of this connectivity between steel beam and CFRP sheet at the interface is rigid link option. However, rigid link option would make the finite element model behave much stiffer than reality.


(a)

(b)

Figure 1. Repaired steel beam: (a) CFRP repaired beam; and (b) four-point bending test of the repaired steel beam


(a)

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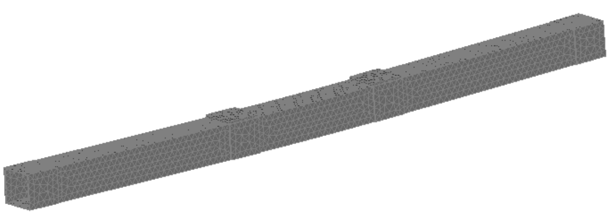

(c)

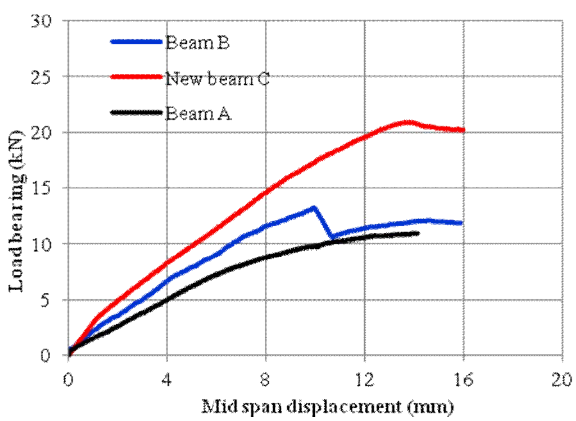
Figure 2. Finite element models of SHS steel beam: (a) cracked beam without CFRP CSH-41 sheet (beam A); (b) cracked beam with CFRP sheet repair (beam C); and (c) steel beam without crack

The beam specimens are simply supported at both ends and loaded at two point distance of 500 mm each other as shown in Figure 2(a). Load is applied step-by-step until collapse or failure indicated by large displacement corresponds to small load increase. Figure 2(a) and 2(b) show the finite element model of the beam A and C which includes crack at mid-span, CFRP sheet, and the steel beam where automatic mesh refinement option available in ADINA software was implemented. In this finite element analysis, model of the SHS steel beam without crack was also developed as shown in Figure 2(c) as basis for CFRP repair efficacy evaluation.

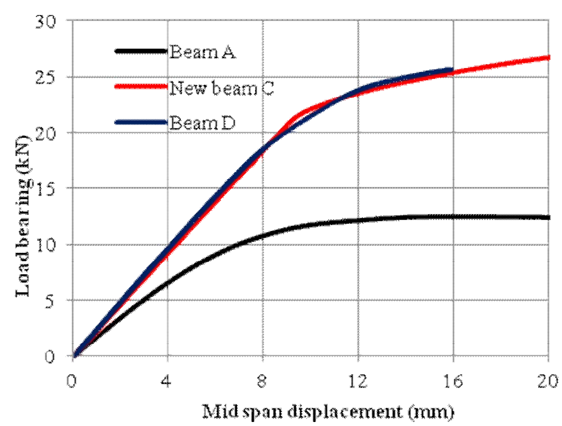
5 RESULTS AND DISCUSSIONS

Relation between load bearing and displacement at mid span of the steel beam A obtained from the experiment and the finite element analysis are presented in Figure 3. The curve given by FEA is slightly stiffer than the experimental curve. Experimental load bearing capacity of beam A is 10.93 kN at mid span displacement equals to 14.13 mm, while the load bearing capacity of beam A derived from FEA is 12.37 kN at mid span displacement of 12.98 mm. This difference is considered to be acceptable owing to know the imperfection or non-uniformity found during gluing process and simplification of beam section in the finite element model, which is exactly square hollow without bend at the corners. Visual observation during the test and deformed shape given by the FEA indicated that local buckling failure at top plate of the SHS beam was clearly observed in both experiment and finite element models. Opening of the crack and crack propagation however come first before this local buckling takes place as shown in Figure 4.

Load bearing - displacement curve of the steel beam B, the repaired beam with CFRP sheet of 1000 mm length at bottom face, is shown in Figure 3(a). During initial loading, this beam shows stiffer behavior than the beam A and reaches load bearing capacity of 13.17 kN at mid span displacement equals to 9.95 mm. After this load bearing capacity is reached, sudden and complete debonding failure occurred and leaved the cracked SHS steel beam supported the rest of the test. Sudden load decrease accompanied this complete debonding failure as indicated in the curve (Figure 3(a)). Owing to know that this load bearing capacity of the steel beam B is almost the same as the load bearing capacity of the beam A given by the finite element analysis, repair technique implemented in the beam B is not successful. Comparison of strain at steel beam at 10 mm upper from the crack tip between the measurement (SG-1 and SG-2) and finite element analysis is presented in Figure 5 where very good agreement was found in the initial loading.



(a)



(b)

Figure 3. Load bearing – displacement curves: (a) experiment; and (b) finite element analysis


(a)


(b)

Figure 4. Deformed shape of beam A: (a) experiment; and (b) finite element model

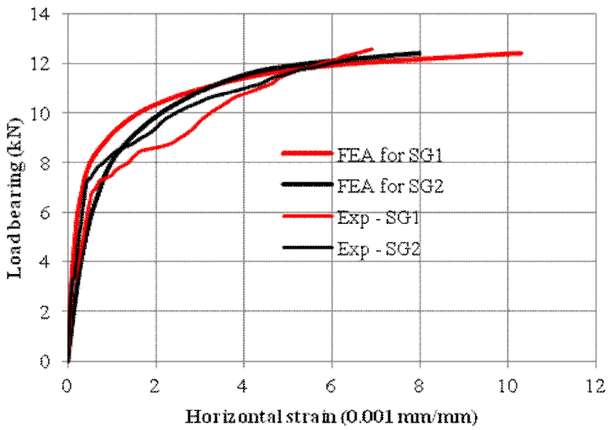


Figure 5. Strain measurement on the steel beam B at mid span at 10 mm upper from the crack tips

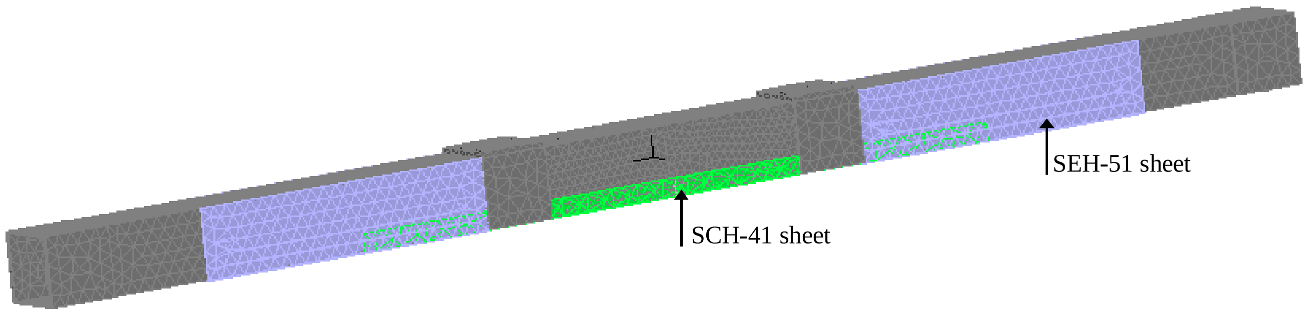
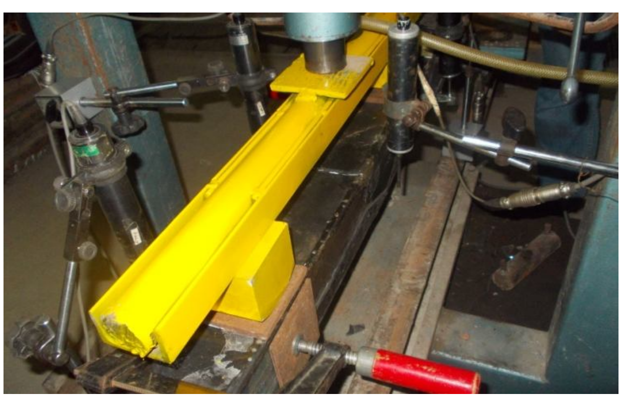


Figure 6. Finite element model of new beam C

To prevent complete debonding failure as found in the steel beam B, changes were made in the steel beam C as follows. Glass fiber reinforced polymers (GFRP) type of SEH-51 sheet is glued over the existing SCH-41 sheet at both ends as shown in Figure 6. Length of this SHE-51 sheet is 500 mm and glued at bottom and both side faces of the SHS steel beam. Technical specification issued by the company informed that this SEH-51 sheet has yield strength of 3240 MPa and modulus of elasticity of 72.4 GPa. This new steel beam C has load bearing capacity of 20.87 kN at the displacement equals to 13.95 mm as shown in Figure 3(a). Load bearing capacity of this new beam C is slightly lower than the load bearing capacity given by the finite element analysis for the SHS steel beam without crack, which is 23.61 kN at mid span displacement of 12.14 mm (Figure 3(b)). This indicates that repair technique adopted in the new beam C is successful. Figure 3(b) summarizes the analytical load bearing displacement curves obtained from the finite element analysis where the load bearing – displacement curve of the new beam C is about the same as the curve of the steel beam without crack.


(a)

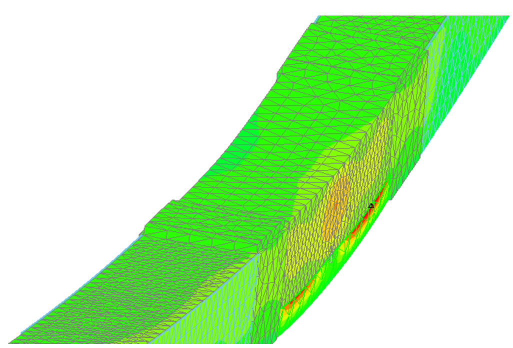

(b)

Figure 7. Closed looks of failure of the new beam C: (a) experiment; and (b) finite element model

Failure mode of the new beam C is shown in Figure 7 where separation of CFRP sheet around the crack and adhesion failure at SEH51 and steel beam interface occurred just before the beam reached its maximum load bearing. Local buckling at the top plate of the SHS beams was also observed following this CFRP separation but it was occurred at much smaller area compare to local buckling failure in the beam A and B.

6 CONCLUSIONS

This study carried out load baring evaluation of square hollow section steel beam having artificial crack repaired with CFRP sheet under four-point bending test configuration. The experimental results was compared to the numerical results based on finite element model developed in ADINA software where the steel beam is modeled as elasto-plastic material and CFRP sheet is modeled as tension material. Gluemesh are used at the steel beam and CFRP interface. Analytical load bearing – displacement curve behaves slightly stiffer than the experimental curve though good agreement of load bearing capacity is found between experiment and analysis. Provided that debonding of CFRP sheet is well prevented, the repaired beam could reach load bearing capacity nearly the same as the load bearing capacity of the steel beam without crack given by finite element analysis.

ACKNOWLEDGEMENT

The authors sincerely gratitude to PT. FyFe Fibrwrap Indonesia for providing CFRP used in this research and sharing the CFRP engineering properties. The first author also with to thank PT. FyFe Fibrwrap Indonesia for supporting his travel to Dhaka to attend IABSE-JSCE joint conference in 2015.

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