



Flexural strengthening of reinforced concrete beams with NSM-CFRP bars using mechanical interlocking

Salam Al-Obaidi^{a,b}, Yasir M. Saeed^{c,*}, Franz N. Rad^d

^a Roads and Transportations Engineering Department, University of Al-Qadisiyah, Diwaniyah, 58001, Iraq

^b Department of Civil and Environmental Engineering, Politecnico di Milano, Milan, Italy

^c Department of Civil and Environmental Engineering, Portland State University, P. O. Box 751, Portland, OR, 97207, USA

^d Department of Civil and Environmental Engineering at Portland State University, Portland, OR, 97201, USA

ARTICLE INFO

Keywords:

Concrete beams
Near surface mounted
NSM-CFRP
Strengthening

ABSTRACT

Flexural strengthening of reinforced concrete (RC) beams using near-surface-mounted (NSM) technique has become an attractive alternative for rehabilitation using fiber reinforced polymer (FRP) materials. Previous studies have recommended using available anchoring techniques to overcome premature bonding failure. In this study, mechanical interlocking grooves were utilized to delay or prevent debonding failure. The first part of the study aimed to investigate the bond characteristics for NSM-CFRP bars by conducting several pullout tests on No. 6, No.10, and No.13 CFRP bars. Results indicate that mechanical interlocking grooves can significantly enhance the bonding capacity and prevent or delay premature bonding failure. In the second part, the proposed NSM CFRP strengthening technique was used to strengthen nine RC beams. In addition to longitudinal grooves, the proposed technique consisted of 6 mm wide lateral grooves (or mechanical interlocking) placed at 76 mm on center along the entire length of the strengthened beams. Steel reinforcement ratios of 0.7% and 0.4% were selected. All beam specimens were tested under four-point bending until failure. Results showed that strengthening was more effective for specimens with a lower steel reinforcement ratio. Percentages of enhancement in flexural strength were between 34–68% and 60–128% for specimens with 0.7% and 0.4% steel reinforcement ratios, respectively. Finally, a simple empirical model was created for the experimental results. Theoretical results showed reasonable agreement with the experimental results. However, the maximum load carrying capacity and flexural stiffness were overestimated for beams with a total reinforcement ratio (steel plus CFRP) larger than 1.1%.

1. Introduction

FRP materials have been used to strengthen many different types of structural components. FRP jackets, FRP strips, and FRP bars are the common types used to rehabilitate existing structural components. The main benefits of using FRP are that it is lightweight, high in strength, and easy to set-up. Among the many applications of using FRP, near surface mounted -fiber reinforced polymer (NSM-FRP) is a promising technique of strengthening concrete members [1–7]. The NSM-FRP technique consists of pressing an FRP bar into a cut that is pre-made in the concrete cover of the member to be strengthened. FRP bars are attached to the concrete by epoxy paste. Groove size, paste, concrete, bars properties are the main variables that control the efficiency of the NSM-FRP strengthened member [8]. Based on the available literature,

NSM-FRP technique significantly improves the overall behavior of flexural members [2–4,9]. De Lorenzis et al. [10] concluded that the application of NSM-FRP bars increased the flexural capacity of RC beams by 26%–44%. NSM-FRP technique has emerged as more promising as compared to externally bonded (EB) FRP strengthening techniques because NSM-FRP bars have stronger bond to concrete. Seo et al. [11] reported that NSM-FRP had excellent bond efficiency which was about 50% higher than EB-FRP. In addition, FRP materials in NSM-FRP technique are protected from environment, mechanical damages, and fire [12–16]. Astorga et al. [17] showed that the ultimate capacity of a flexural member was improved by 41% after being subjected to long-term testing. Sharaky et al. [18] tested beams strengthened with carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP). In these tests, the ultimate capacity of the strengthened

* Corresponding author.

E-mail addresses: salam.methem@qu.edu.iq, salammaytham.alobaidi@polimi.it (S. Al-Obaidi), yasir@pdx.edu (Y.M. Saeed), franz@pdx.edu (F.N. Rad).

<https://doi.org/10.1016/j.job.2020.101422>

Received 24 August 2019; Received in revised form 9 April 2020; Accepted 10 April 2020

Available online 18 April 2020

2352-7102/© 2020 Elsevier Ltd. All rights reserved.

beams increased by 150% and 170% compared to the control beams when they were strengthened by NSM-CFRP and NSM-GFRP, respectively.

Most of the tests reported in the literature had specimen cross-sections ranging from 180×250 mm to 200×300 mm [2,4]. This range provided adequate room to accommodate two CFRP bars of No. 10 or No. 13 as the maximum limits for the beams. The main steel reinforcement ranged from 0.4% to 1.6% [3,5]. These ratios covered the minimum and the maximum steel ratios that are normally present in most of the existing beams. The minimum steel ratio simulated beams with corroded reinforcement, and the medium or maximum steel ratio simulated components that needed upgrading to satisfy increased applied loads. The concrete compressive strength ranged from 14 MPa to 60 MPa [1,2,4].

The load-deflection response of previously tested NSM-CFRP RC beams is typically classified into three stages [3,5,6]. The first limit is the concrete cracking, where the strengthened and the un-strengthened beams attain this limit at the same load level. At this limit, the steel and the CFRP bars have not yet been effectively engaged in tacking the applied stresses. The second limit is when the steel reinforcement yields and the FRP bars become engaged. In this limit, the un-strengthened beams will stop taking additional load and behave in a ductile manner until failure, whereas, the strengthened beams continue to resist more load. The third limit is where the beam reaches its maximum capacity. After this point, three possible failure scenarios occur depending on the NSM parameter. The extreme scenario is the rupture of the FRP bars [19]. The rupture of the FRP would occur if the beam has low steel ratio and low FRP ratio. The second scenario is the debonding of the FRP bars [20]. This behavior occurs if the development length is not sufficient. The third scenario is the spalling of the bottom concrete layer that contains the FRP [3,4]. This phenomenon occurs if the steel and the FRP ratios are high, and FRP has adequate development length. Previous studies suggested using mechanical interlocking, or lateral grooves, to improve the bond performance of NSM-FRP bars [8]. Jung et al. [2] is one of the two studies available in the literature that used mechanical interlocking in flexural strengthening of RC beams with NSM-CFRP bars. Jung et al. [2] suggested using mechanical interlocking to overcome debonding and increase the overall flexural performance. The flexural ultimate capacity was increased by 15%. However, this conclusion was based on testing only one beam strengthened with a single NSM-CFRP bar with mechanical interlocking. Moreover, the mechanical interlocking extended over the entire width of the beam, which might not be necessary. Sharaky et al. [18] used mechanical interlocking with steel shear connectors to improve the bond performance. The authors reported that the mechanical interlocking with shear connectors increased the load carrying capacity by 33% compared to the strengthened beams without mechanical interlocking.

In summary, NSM-FRP technique is an effective method to strengthen reinforced concrete beams. The ultimate load capacity depends on the bond capacity strength between FRP, epoxy, and concrete. Most of the studies that used NSM-FRP to strengthen RC beams, but without mechanical interlocking grooves, reported that premature bond failure controlled the ultimate capacity. Although using mechanical interlocking grooves might be the solution to delay or prevent premature bonding failure, studies on the topic are limited to only a few sources [2,18]. Jung et al. [2] tested one RC beam with wide lateral grooves which may be viewed as uneconomical for practical applications. In addition to the lateral grooves, Sharaky et al. [18] used steel shear connectors, which may be prone to corrosion over time.

The current paper is the first to focus on the use of mechanical interlocking grooves in strengthening RC beams with different steel reinforcement ratios and by using three different sizes of NSM-CFRP bars. The main aim of this study is to implement the lateral grooves as mechanical interlocking when using NSM-CFRP technique. First, the characteristics of NSM-CFRP-to-concrete bonding behavior were determined by performing pullout tests on NSM-CFRP bars with and without

Table 1

Sieve analysis for sand and gravel.

Sand		Gravel	
Sieve size (mm)	%passing	Sieve size (mm)	%passing
4.75	97.81	25	100
2.0	84.67	12.7	48.34
0.85	72.99	9.5	26.16
0.425	49.64	4.75	0.66
0.15	5.11	–	–
0.075	0.73	–	–

Table 2

Concrete properties (average of testing 3 samples at 28 days).

Compressive strength (MPa)	Tensile strength (MPa)	Modulus of rupture (MPa)
26.0	2.35	3.8

mechanical interlocking. Three different CFRP sizes (No. 6, No.10, and No.13) were used in the investigation. Next, nine RC beams were tested to investigate the efficiency of using the mechanical interlocking in retrofitting existing RC beams strengthened with NSM-CFRP bars. The beam specimens were divided into two main groups; moderate steel reinforcement of 0.7% and low steel reinforcement of 0.4%. This research contributes to and provides more knowledge related to the use of NSM-CFRP in strengthening RC structures. The effects of reinforcement ratio and the size of CFRP bar were investigated. Recommendations were made based on the experimental results, predictions based on ACI 440, and the proposed empirical model.

2. Experimental program

2.1. Materials

The materials used in this project were concrete, steel, CFRP, and epoxy. All materials were tested according to American Society for Testing and Materials (ASTM) in the infrastructure Testing and Applied Research (ISTAR) Laboratory and in the South Green House Laboratory at Portland State University.

2.1.1. Concrete

A ready-mixed concrete was ordered from a local concrete supplier. Table 1 shows the sieve analysis of the fine and coarse aggregate used in the concrete mix. At an age of 28 days, concrete compressive strength, splitting tensile strength, and flexure strength were determined experimentally in the laboratory according to ASTM C39/39 M [21], ASTM C496 [22], and ASTM C78 [23], respectively. The values presented in Table 2 are the averages of testing three samples in each category of strength. Concrete compressive strengths of beam specimens (a total of nine beams) were determined by testing three concrete cylinders per beam on the day of test. Although the authors aimed to have the same concrete compressive strength for all beams, concrete strength for each beam was slightly different. This difference in strength most probably related to two main reasons. First, the operator of the concrete mixer mixed the concrete on site, so small changes in the mix proportions may have occurred. Secondly, the beams were not tested all on the same day. Therefore, depending on the age of the concrete, strength could have changed slightly. Finally, the small change in concrete strength is not significant and not uncommon given the fact that the beams were not tested on the same day and the variable nature of concrete material. The mean of the concrete strength values was 30.11 MPa and the sample variance was 1.86 MPa. Concrete compressive strength for each beam will be presented later in the results.

2.1.2. CFRP

The FRP materials are relatively weak in the transverse direction

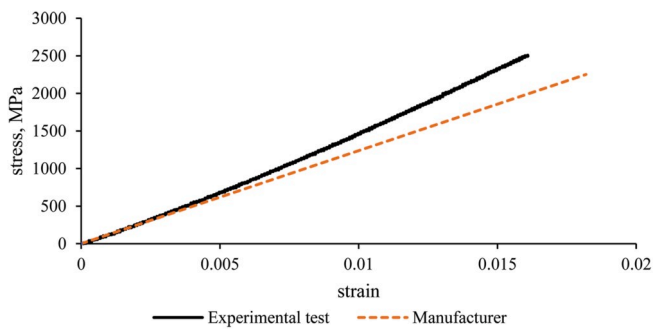


Fig. 1. Tensile properties of CFRP bars.

compared to the longitudinal direction. This inherent property of the composite materials causes premature failure due to the gripping action in tensile tests. Therefore, ASTM D7205/D7205 M [24] was adopted to perform the tensile tests. The experimental axial stress-strain behavior and modulus of elasticity were evaluated and compared with the manufacturer's data sheet [25]. The tensile properties of CFRP bars are shown in Fig. 1, and the manufacturer's data are provided in Table 3. The anchoring technique used here to test the CFRP bars was previously developed for prestressed concrete applications [26] and CFRP pullout testing [27].

The test (short-term static strength) was carried out in a 445 kN MTS tensile loading frame. Shown in Fig. 1, the strain in the "measured stress-strain relationship for CFRP bars" was measured by laser extensometer and strain gauges. The stress-strain relationship by the manufacturer was constructed based on values of tensile strength and the modulus of

elasticity provided by the manufacturer. Prior to rupture, breaking of the helical fibers was audible, after which the carbon fibers began to rupture. CFRP rupture happened at the mid-span of the bar's free length, as shown in Fig. 2. As shown in Fig. 1, the overall trend of the tensile stress-strain relationship was linear until failure. This trend was reported by the manufacture and exhibited by the physical test. These values will be used for the analysis and design of the NSM-CFRP specimens.

2.1.3. Epoxy

Sika AnchorFix-1 was used in this research as the NSM CFRP-to-concrete bonding agent. It is a fast curing anchoring adhesive consisting of two parts: adhesive and hardening agent. The two parts were sealed separately in one tube. A mixing nozzle and an adhesive gun were used to mix and apply the epoxy adhesive. The mechanical properties, as

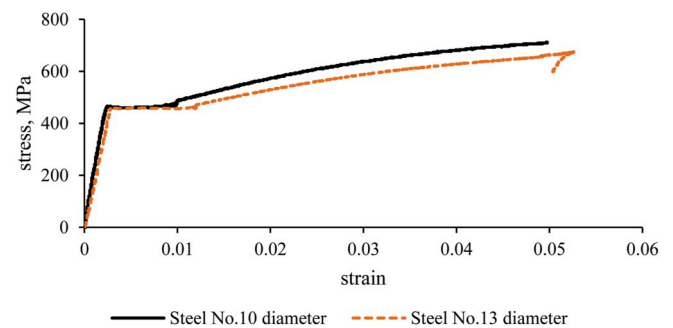


Fig. 3. Stress-strain relationship of steel bars.

Table 3

The manufacturer data sheet for CFRP bars.

Size	Nominal diameter, (mm)	Nominal area, mm ²	Guaranteed tensile strength, MPa	Max. tensile load, kN	Modulus of elasticity, (GPa)	Ultimate strain, %
No. 6	6	31.67	2241	71	124	1.81
No.10	10	71.26	2172	154	124	1.73
No. 13	13	126.7	2068	262	124	1.67



Fig. 2. Failure modes for No.6 CFRP bar.

Table 4

Epoxy properties based on the manufacturer data.

Epoxy	Compressive Strength, (MPa)	Flexural Strength, (MPa)	Tensile Strength, (MPa)	Compressive E-Modulus, (MPa)	Tensile E-Modulus, (MPa)	Curing time
Sika	60	28	12	3500	4500	7 days

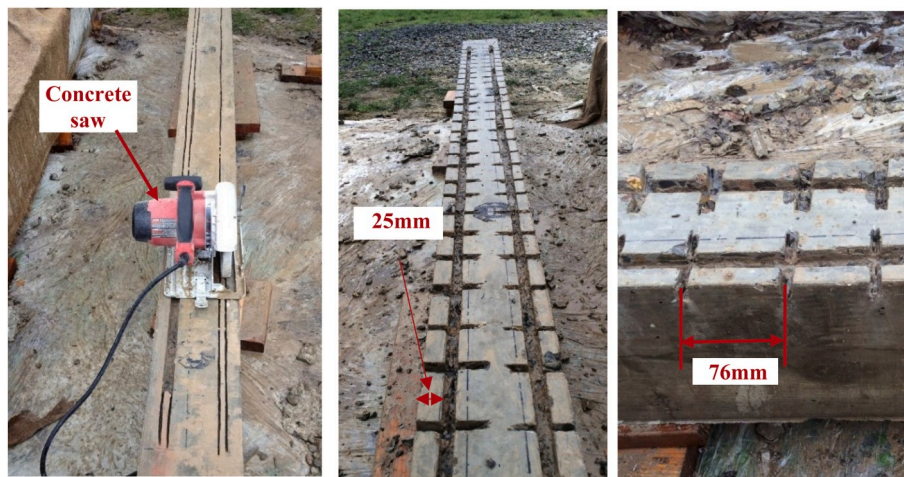


Fig. 4. Making grooves in the tension side of the strengthened beams.

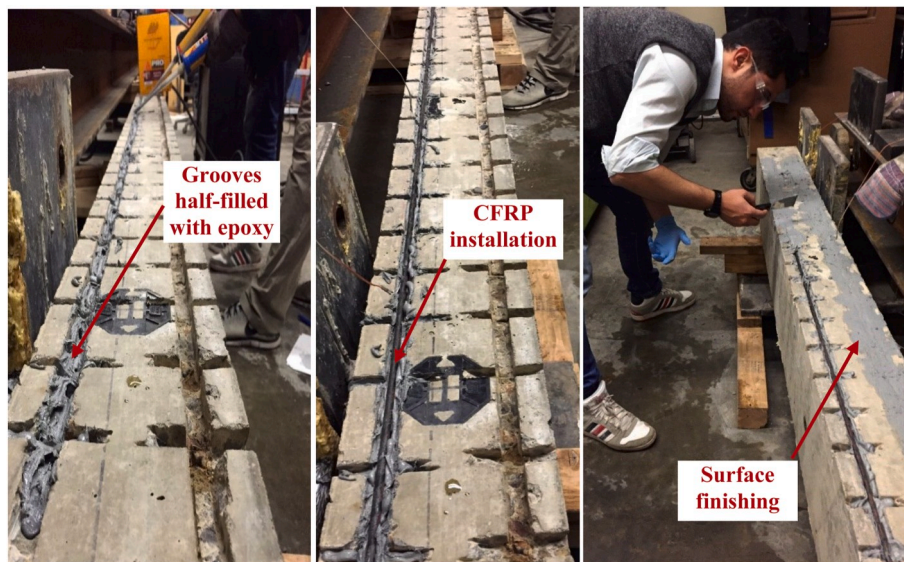


Fig. 5. Inserting CFRP bars.

provided by the manufacturer, are shown in Table 4.

2.1.4. Steel

No. 10 and No. 13 steel bars were used for flexural and shear reinforcement. The tensile strength and modulus of elasticity were obtained experimentally in the laboratory following ASTM A615 [28]. Three specimens from each size were tested using a 445 kN MTS machine, with strain gages attached at the middle section of the specimens. Fig. 3 shows the steel stress-strain relationships.

2.2. Preparation of the grooves

The grooves were made by a saw with diamond blade to form two parallel cuts along the length of the beams, followed by using a hammer and chisel manually to finalize the grooves. The groove's width, as well as the depth, was equal to two times bar diameter [20]; i.e., 12 mm for No. 6 CFRP bars, 20 mm for No. 10 CFRP bars, and 26 mm for No. 13 CFRP bars. The edge distance of the longitudinal grooves was 25 mm from the beam sides, and the spacing between the lateral grooves was 76 mm as shown in Fig. 4. The grooves were cleaned with an air compressor to remove the concrete dust before inserting the CFRP bars.

2.3. Inserting CFRP bars

To install the CFRP bars, a first layer of epoxy was applied to the grooves, with a thickness about half of the grooves' depth. Then, CFRP bar then was pressed in the groove followed by applying a second layer of adhesive to the top of the bar. The excessive adhesive was leveled and removed as shown in Fig. 5. The adhesive was left for at least seven days for curing at temperature of about 17.0 °C.

2.4. Pullout test specimens

Two concrete blocks were cast for conducting six CFRP pullout tests. The geometry and dimensions of the concrete blocks are shown in Fig. 6. The size of the CFRP bars and the groove characteristic for the pull-out test specimens were identical to the beam specimens. Three sizes of CFRP bars were used; No. 6, No.10, and No.13 bars. Two pullout tests were performed for each bar size. The details of the pull-out test specimens are presented in Table 5. Lateral grooves were used for all specimens except one pullout test, which was for a 6 mm bar. The width and length of the lateral grooves are shown in Fig. 7. The distance between the lateral grooves was 76 mm on center. The details for the lateral grooves were adopted from Sharaky et al. [29].

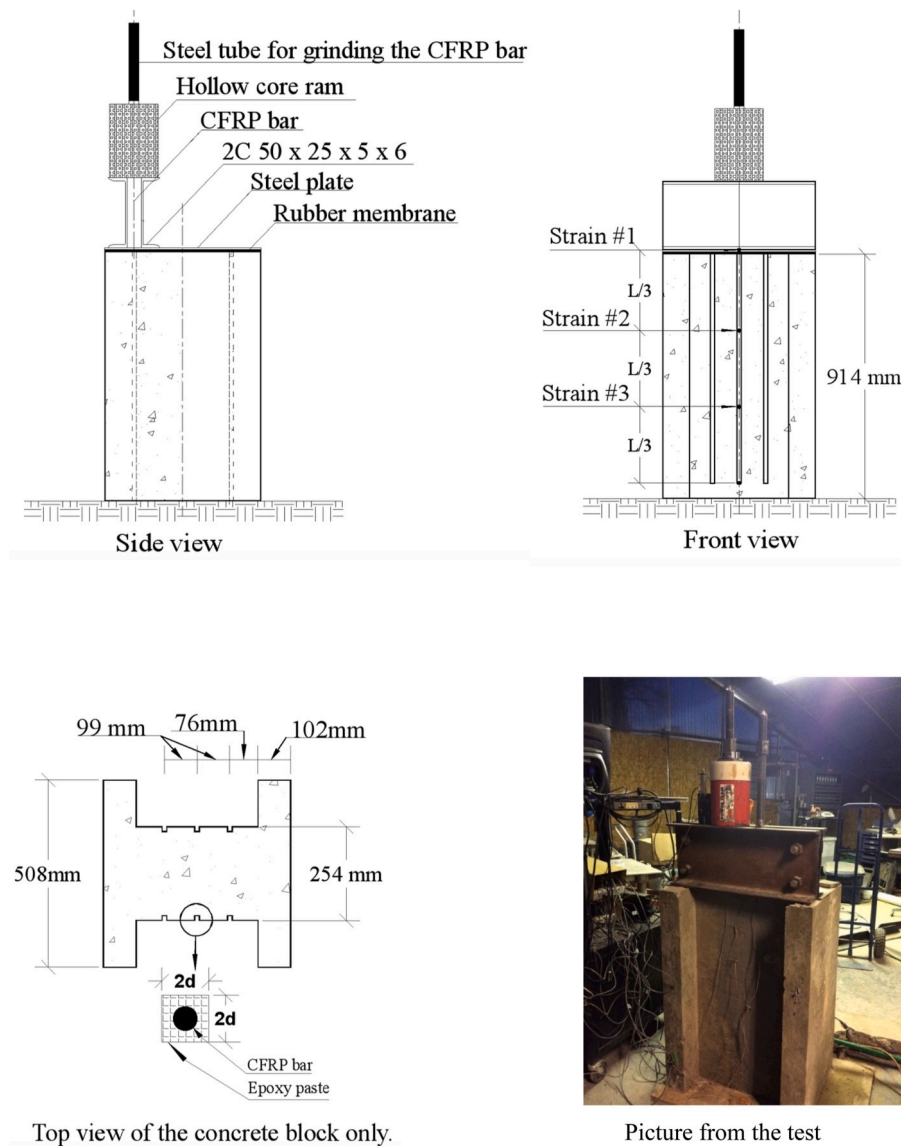


Fig. 6. Test set-up and dimension details for pull-out tests.

Table 5

Details of the pullout tests specimens.

Label	CFRP bar size, mm	Bond length, mm	Lateral grooves?	Max. force, kN	Average, kN	Failure mode
P-6-NO-1	6	360	NO	27	27	Debonding
P-6-YES-2	6	360	YES	63	63	CFRP rupture
P-10-YES-1	10	600	YES	95	99	Concrete crushing-bond failure
P-10-YES-2	10	600	YES	102		Concrete crushing-bond failure
P-13-YES-1	13	780	YES	138	137	Concrete crushing-bond failure
P-13-YES-2	13	780	YES	136		Concrete crushing-bond failure

The main objective of conducting pullout tests was to obtain a pullout resistance of at least 50% of CFRP maximum tensile capacity before failure. As suggested by De Lorenzis et al. [30], the grooves' width and depth were equal to two times the bar diameter. Soliman et al. [31] reported that the maximum strength of NSM CFRP bars was achieved when the bond lengths were $36d_b$ and $48d_b$ for 9.5 mm and 12.7 mm bars, respectively. However, the tensile strengths were 1546 MPa for 9.5 mm bars, and 1250 MPa for 12.7-mm bars. These values were less than the measured tensile capacity of the CFRP bars used for the current study. Based on the differences of the tensile strengths between CFRP bars used by Soliman et al. [31] and the ones used for this research, $60d_b$

was chosen as the bonding length for this study. The total bond length was 360 mm for the No.6 CFRP bars, 600 mm for No.10 CFRP bars, and 780 mm for No.13 CFRP bars.

2.5. Flexural beam tests

Two sets of reinforced concrete beams (denoted by Set A and Set B) were fabricated and tested. Nine beams were tested in flexure using a four-point load testing system. Table 6 presents the details of the beam specimens. The cross-sectional dimensions of the beams were 254 x 152 mm. The total length of beam specimens was 2.743 m, and the clear span

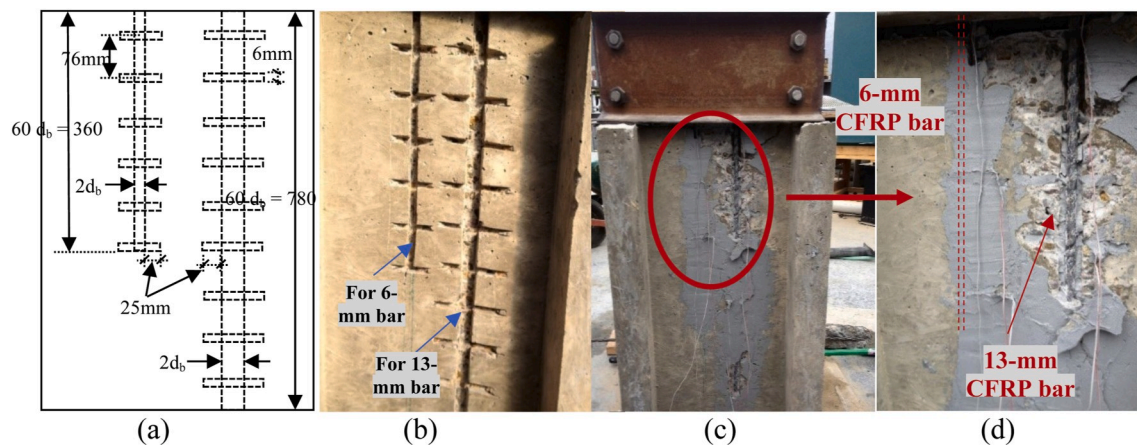


Fig. 7. Pullout test; (a) details of the lateral grooves, (b) picture of the grooves, (c) failure modes of No.6 and No.13 bars, and (d) closer look at the failure modes.

Table 6
Details of flexural beam specimens.

Label	Dimensions, mm	Concrete strength, MPa	Steel in tension	Steel in comp.	Steel reinf. ratio	NSM-CFRP bars	Total reinf. ratio
AC	152 x 254	29	(2) No.13	(2) No.10	0.7%	None	0.72%
AR2	152 x 254	29	(2) No.13	(2) No.10	0.7%	(2) No.6	0.91%
AR3	152 x 254	30	(2) No.13	(2) No.10	0.7%	(2) No. 10	1.12%
AR4	152 x 254	28	(2) No.13	(2) No.10	0.7%	(2) No. 13	1.45%
AR4-S	152 x 254	32	(2) No.13	(2) No.10	0.7%	(1) No. 13	1.09%
BC	152 x 254	31	(2) No.10	(2) No.10	0.4%	None	0.40%
BR2	152 x 254	30	(2) No.10	(2) No.10	0.4%	(2) No. 6	0.58%
BR3	152 x 254	32	(2) No.10	(2) No.10	0.4%	(2) No. 10	0.80%
BR4	152 x 254	30	(2) No.10	(2) No.10	0.4%	(2) No. 13	1.10%

was 2.439 m. (Fig. 8). Set A consisted of five beams, and Set B consisted of four beams. Each set had a different longitudinal steel reinforcement ratio in tension. Set A beams had two No.13 steel bars in tension and two No.10 steel bars in compression. No.10 steel stirrups 102 mm on center were used for shear reinforcement. The tensile reinforcement ratio of Set A beams was 0.7%, which was a moderate steel reinforcement ratio representing beams that may require upgrade to satisfy anticipated future loads. Specimen AC in set A represented the control (reference) beam, which had no CFRP bars (Fig. 8). Specimens AR2, AR3, and AR4 had the same steel reinforcement of beam AC but strengthened with two No. 6, two No. 10, and two No.13 NSM-CFRP bars, respectively. Specimen AR4-S was strengthened with one No.13 NSM-CFRP bar only.

As shown in Table 6, the second set (Set B) had a lower steel reinforcement ratio (0.4%). This steel ratio represented a poorly reinforced concrete beams or deficient beams due to corrosion [32]. Set B specimens consisted of two No.10 steel bars in tension and compression. The shear reinforcement was No.10 steel stirrups at 102 mm on center (Fig. 8). Specimen BC in set B represented the control (reference) beam, which had no CFRP bars. Specimens BR2, BR3, and BR4 had the same steel reinforcement as that of beam BC but strengthened with two No. 6, two No.10, and two No.13 NSM-CFRP bars, respectively. The details of the grooves, including the lateral grooves or the mechanical interlocking, are shown in Fig. 8.

2.6. Loading system, instrumentation, and procedure

2.6.1. Pullout tests

The modified pullout test, proposed by De Lorenzis et al. [33], was adopted in this study to investigate the bonding behavior of NSM-CFRP bars. The test set-up is shown in Fig. 6. The main difference between the modified pullout test and the conventional pullout test is that the reaction of the pull-out load does not apply a bearing load on the concrete next to CFRP bar. In other words, the test set-up is constructed such that there is no additional confinement pressure on the bar due to the bearing

pressure caused by the applied load [33]. A total of six NSM CFRP bars were tested (Table 5). The load was measured using a strain gauge-based load cell. To measure the tensile strains, each bar was instrumented with three strain gauges. Strain 1 was attached at the top of the bonding length. Strain 2 and Strain 3 were attached at every one-third of the bonded length as shown in Fig. 6. A loading rate of 0.0002 strain per minute was used following guidelines proposed by ASTM D7205/D7205 M [24].

2.6.2. Beam tests

The four-point loading system test setup is shown in Fig. 9. Strain gages were placed on both the flexural steel bars and the CFRP bars (Fig. 8b and c). Three strain gages were attached to the steel bars, placed in the pure bending moment region to capture the maximum flexural strain developed at the steel bars. Moreover, four strain gages were attached to the CFRP bars. As shown in Fig. 8 (c), two strain gauges were placed at the location where the point loads were applied, and two strain gauges were placed in the middle of the shear span (407 mm from the support). Steel and CFRP bar's surfaces were prepared to ensure proper bonding of the strain gages. Concrete strain gauge of 30 mm long was mounted on the top fiber of each beam at mid-span to record the maximum compression strain of the concrete. A linear variable differential transformer (LVDT) of 100 mm stroke was placed at the middle of the beam to record the deflection. The specimens were tested under a monotonically increasing load until failure, and the load was applied manually by a hydraulic jack. Before steel yielding, the loading rate was between 300 N/s and 500 N/s. It was decided to apply the load at a rate of 5 kN per step. At each step, crack widths and lengths were measured and labeled. After yielding, a displacement-controlled loading rate of 0.2 mm/s was used. The beam test setup is shown in Fig. 9.

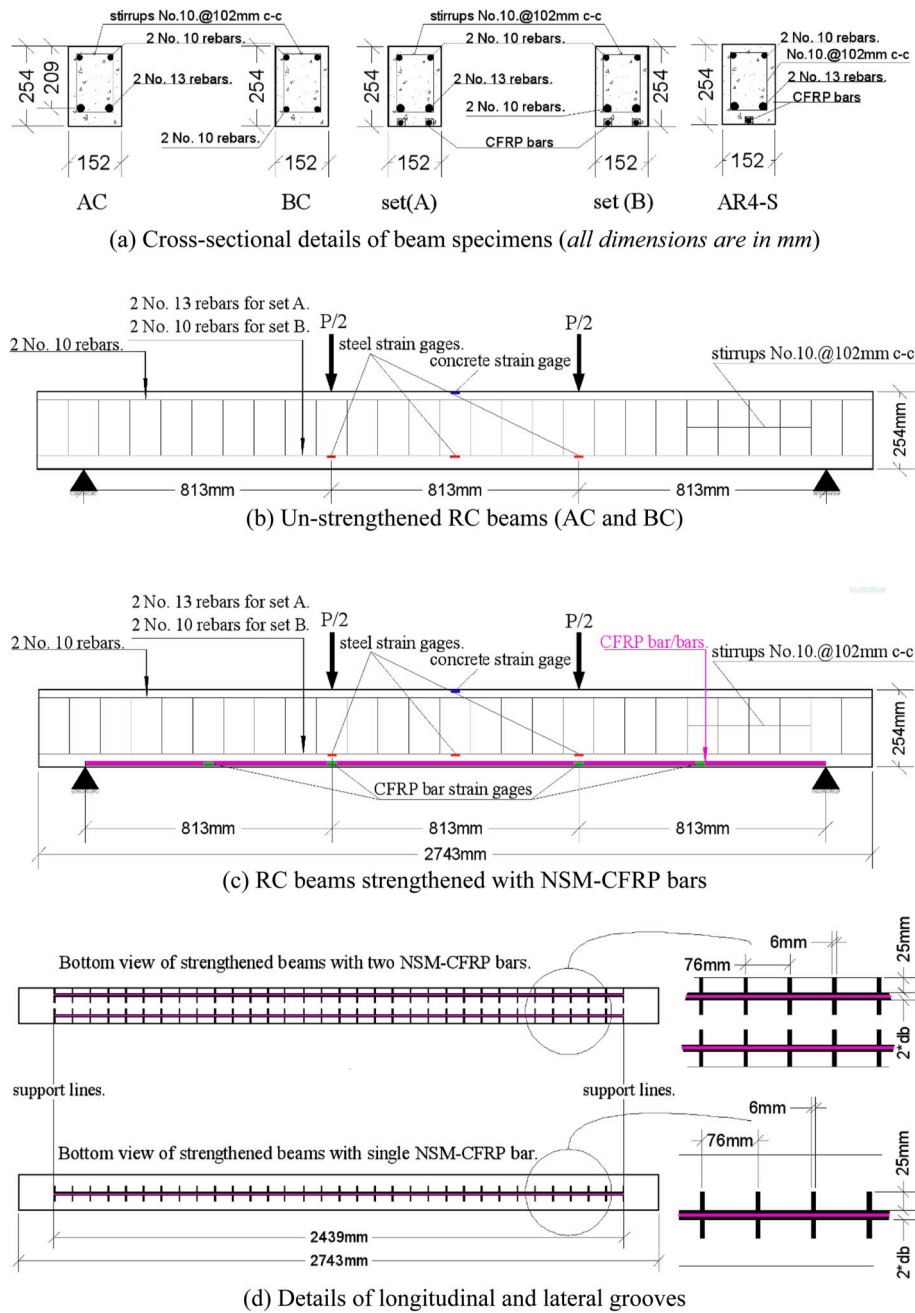


Fig. 8. Details of flexural beam specimens; set A and set B.

3. Experimental results and discussion

3.1. Pull-out results

The pullout test results are summarized in Table 5. For the first specimen (without mechanical interlocking), the maximum achieved force for No.6 NSM CFRP bar was only 38% of CFRP ultimate strength. The failure mode was debonding at the concrete-to-epoxy interface. CFRP bar and the epoxy were separated from the concrete as one piece. The same failure mode was reported by Sharaky et al. [8] and Novidis et al. [34]. Therefore, it was decided to enhance the bonding capacity in the subsequent specimens by using mechanical interlocking, or lateral grooves. Using the lateral grooves, the ratios of the failure load to the tensile strength of CFRP bars were 89% for No. 6 bars, 64% for No.10 bars, and 52% for No.13 bars (Tables 3 and 5). Although the maximum achieved load was 89% of the maximum CFRP tensile strength, the

failure mode of No. 6 NSM CFRP bar with lateral grooves was CFRP rupture. The failure modes for No. 10 and No. 13 bars were concrete crushing at the maximum stress region (the top of bonding region) followed by bonding failure (Fig. 7). Sharaky et al. [8] also tested pullout specimens with the same lateral grooves used in this study. However, they implemented $24d_b$ as the bond length for 8 mm GFRP bar. The failure mode was concrete crushing with epoxy splitting and longitudinal cracking of epoxy. Only 50% of the GFRP bar tensile capacity was obtained before the failure. The maximum achieved pullout forces in the current study were more than the forces obtained by Sharaky et al. [8]. It was concluded from the pullout tests that the lateral grooves enhanced the bond capacity by providing an interlocking bond between the epoxy and the concrete. This interlocking force increased the bond capacity between epoxy and concrete. Fig. 10 compares the load vs. CFRP stain relationships for No. 6 NSM CFRP bars with and without lateral grooves. In general, the strain values along the CFRP bar are maximum at the

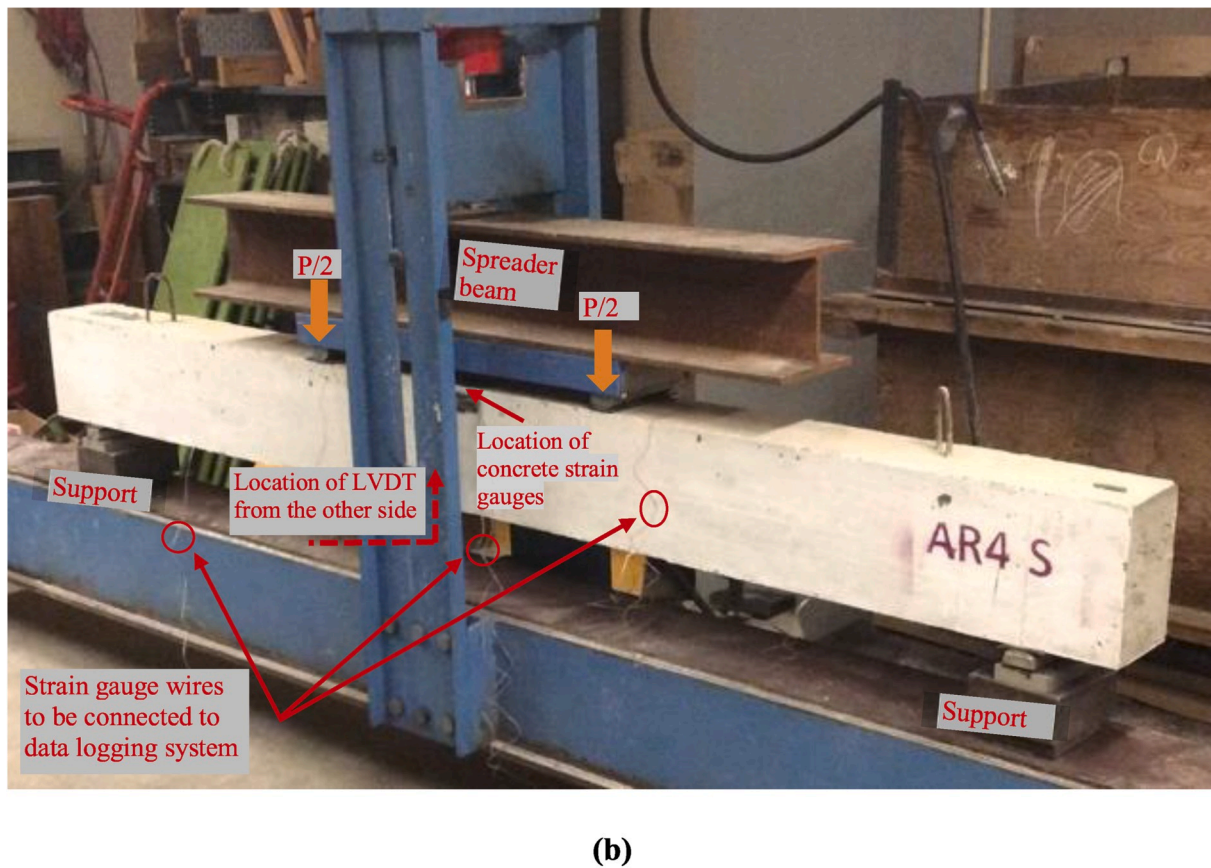
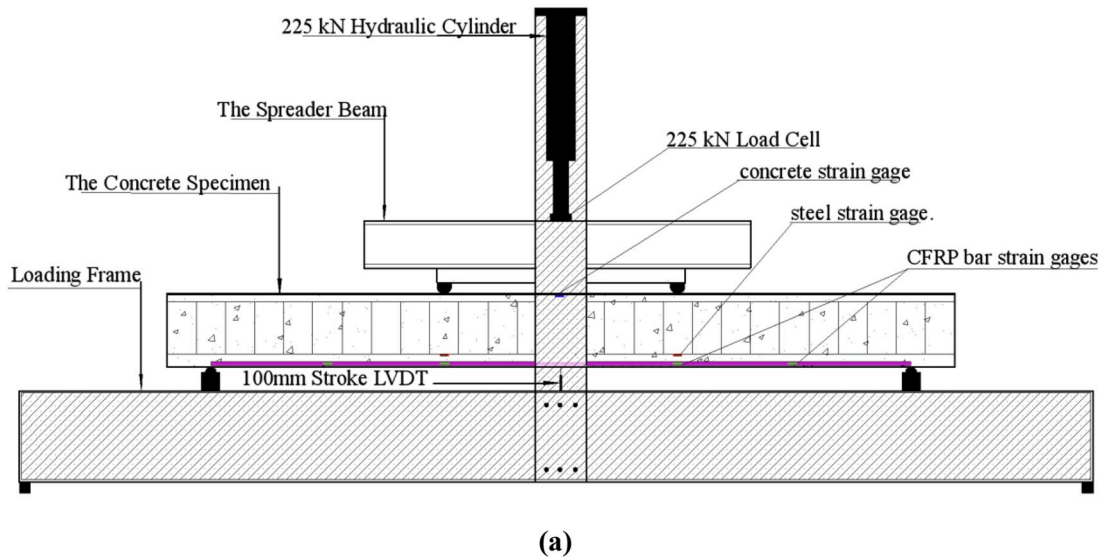


Fig. 9. Four-point load test setup; (a) schematic drawings, (b) picture.

loaded end and smaller at the other end. Another observation of the test results was that for almost all specimens, Strain 2 and Strain 3 did not start showing values immediately after applying the pullout load. The load had to transfer gradually from the CFRP bars to the epoxy, and to the concrete. It is worth mentioning that the straight vertical lines in Fig. 10 occurred when the strain gauges detached from CFRP bars. Therefore, after this point the strains were not recorded, and only the load values were considered.

Please note that the straight vertical lines in Fig. 10 occurred when

the strain gauges detached from CFRP bars, and therefore the strain readings were lost. The authors kept these lines in the figure to show the maximum pullout load.

It was observed that the failure for the No. 6 NSM-CFRP bar with lateral grooves did not involve any damage to the surface of the concrete before it ruptured. On the other hand, the failure of No.10 and No.13 bars caused serious damage to the adjacent concrete. The crushing of the concrete introduced the debonding failure. The failure mode for No.10 bars was similar to the mode of failure observed for the No.13 bars.

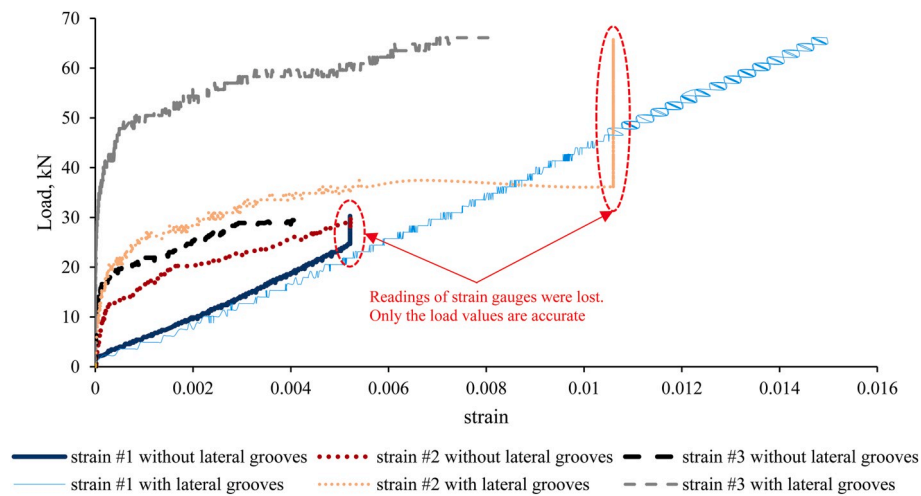


Fig. 10. Pull-out test results; lateral grooves vs. conventional NSM-CFRP bars.

Table 7

Set A experimental results.

Speci-men No.	Cracking load kN	Total reinforcement Ratio	Ultimate capacity (kN)	Increase in capacity beyond AC	Ultimate deflection (mm)	Ductility	Ultimate CFRP strain
AC	10.1	0.72%	76	—	49	3.75	—
AR2	13.3	0.91%	128	68%	38	2.5	0.011
AR3	15.3	1.12%	102	34%	25	1.6	0.006
AR4	22.2	1.45%	122	61%	20	1.0	0.003
AR4-S	17.8	1.09%	118	55%	28	1.7	0.0057

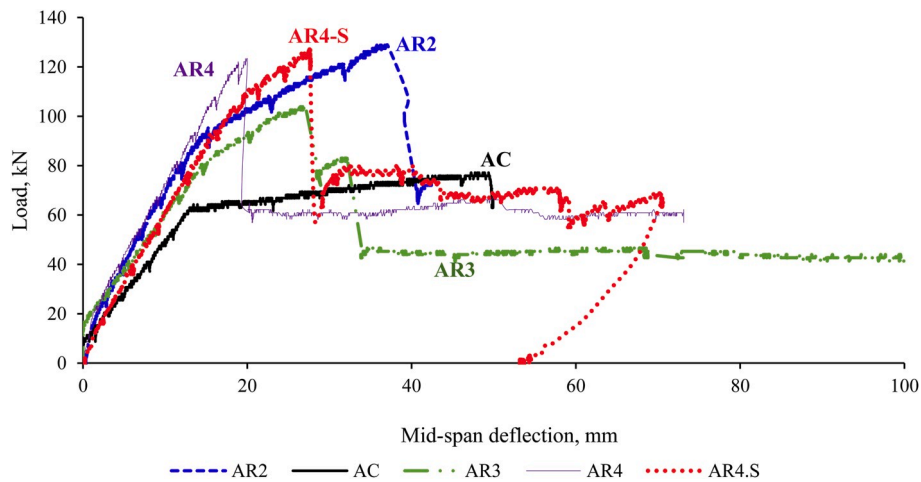


Fig. 11. Set A load-deflection curves.

Debonding failure reduced the pullout capacities of No.10 and No.13 bars to 64% and 52% of CFRP maximum tensile capacity, respectively (Tables 3 and 5). It appears from the results that the interlocking mechanism (lateral grooves) shifted the failure from concrete-to-epoxy interface to CFRP rupture for No. 6 bars, and concrete crushing for the No. 10 and No.13 bars.

3.2. Beam tests: set (A) specimens

Table 7 summarizes the test results of Set A beam specimens. The load-deflection curves of Set A specimens are shown in Fig. 11, and Fig. 12 shows the failure modes for all specimens. The first tested beam in set A as a control beam is specimen AC. The failure mode of this specimen was the crushing of concrete at about 76 kN. The second beam

tested was AR2. From the measured load-deflection response shown in Fig. 13, three distinct zones can be observed: the first zone is where the first crack of 0.05 mm occurred at about 13.34 kN and the beam lost some of its flexural stiffness. The second zone is where the steel and the CFRP bars behaved elastically; and the third zone is where the steel reinforcement yielded. In the last zone, more of flexural stiffness was lost and more cracks were initiated and widened at the locations of the lateral grooves. This observation was also reported by Al-Mahmoud et al. [5] and Szabo et al. [6] where the authors categorized the moment-deflection response into three zones: first zone is the elastic stage, second zone is concrete crack to steel yielding stage, and the third zone is steel yielding to failure stage. The load was then increased until concrete spalling occurred under one of the load points at about 128 kN. The increased load carrying capacity and initial stiffness over the control

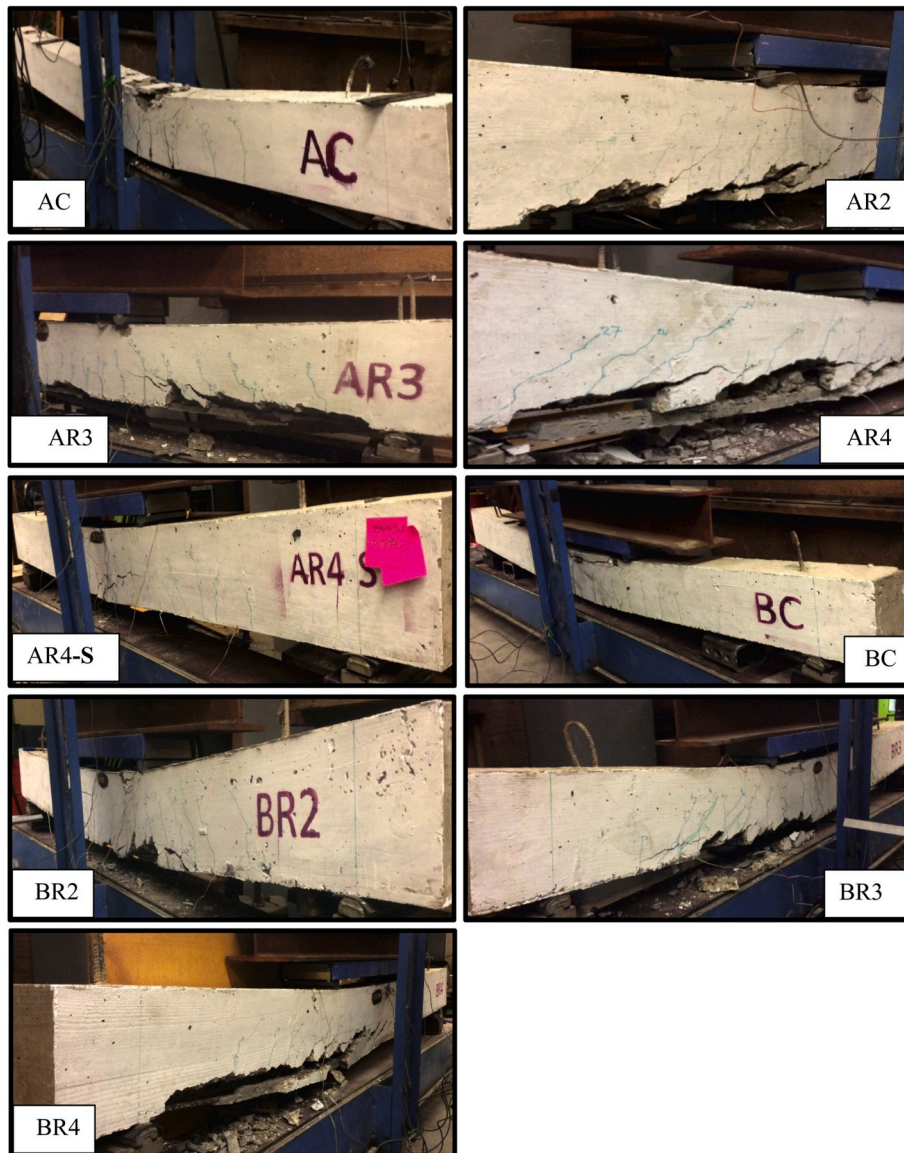


Fig. 12. Failure modes of flexural beam specimens.

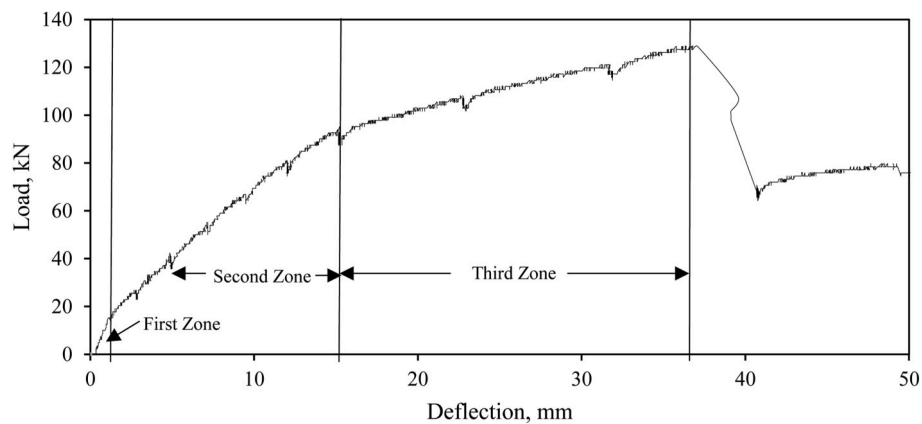


Fig. 13. Specimen AR2 load-deflection curve.

beam was about 68% and 40%, respectively. However, ductility decreases by 40%. Before deboning occurred, some diagonal cracking occurred at the maximum bending moment region. These cracks were

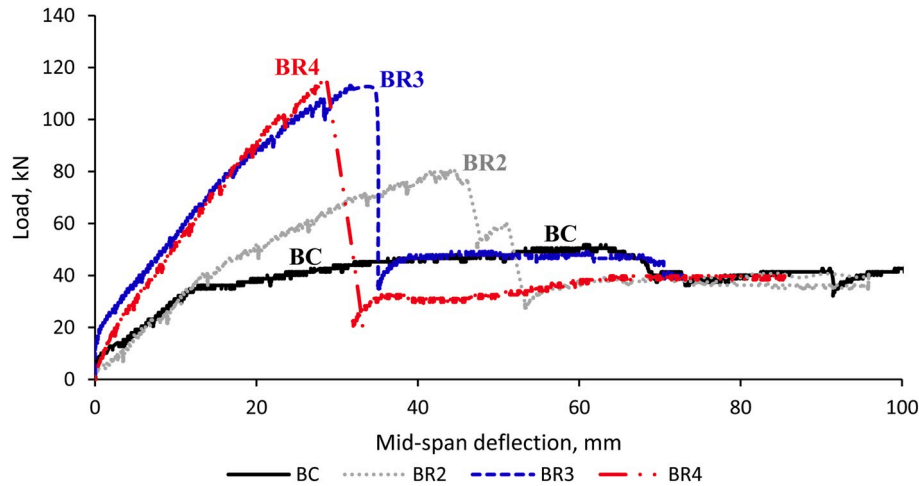
initiated near the middle and extended to the ends of the beam which is similar to the crack failure pattern reported in ACI 440.2R-17 [35].

The third beam tested in Set A was AR3. In general, the same three

Table 8

Set B experimental results.

Speci-men No.	Cracking Load kN	Total reinforcement Ratio	Ultimate capacity, (kN)	Increase in capacity beyond BC	Ultimate deflection (mm)	Ductility	Ultimate CFRP strain
BC	10.0	0.40%	50	–	64	5.0	–
BR2	11.1	0.58%	80	60%	44.7	3.5	0.014
BR3	13.3	0.80%	111	122%	35	2.3	0.0065
BR4	22.2	1.10%	114	128%	29	1.0	0.0044

**Fig. 14.** Set B load-deflection curves.

zones were exhibited in the load-deflection curve. The first zone was where the first cracking of 0.05 mm occurred at 15.3 kN. The second zone was at about 80 kN. The load was then increased until FRP debonding at one of the beam ends occurred at about 102 kN, and that was where the third zone of the curve was shaped. For AR3, the increased load carrying capacity was 34%, and the ductility decreased by 57% over the control beam. The expected capacity of this specimen was more than AR2. However, some bonding issues in one of the CFRP bars caused the first premature debonding failure, which resulted in reducing the load carrying capacity of the specimen. Also, very slight improvement in the initial flexural stiffness was observed for specimen AR3. The fourth beam tested in set A was AR4. For beam AR4, two distinct regions can be observed. The first region is where the first cracking of 0.04 mm occurred at about 22.24 kN. That is where the beam lost some of its flexural stiffness. The load was increased until FRP debonding failure occurred at one end of the beam at about 122 kN, and that is where the second region of the curve was exhibited by a significant drop in the load. The increased load-carrying capacity and initial stiffness over the control beam was 61% and 40%, respectively. However, the ductility decreased by 74% over the control beam. Therefore, this specimen had a brittle failure, which was an indication of over-reinforced concrete beam.

The fifth beam tested in set A was AR4-S. This specimen had the same properties as the AC beam but was additionally reinforced with a single No. 13 CFRP bar on the tension face. The purpose of testing this specimen was to minimize the stress at the bonding area by increasing the edge distance. The increased load carrying capacity was 55% over the control beam with very small improvement in the initial flexural stiffness (5% only). Before the failure, some diagonal cracking occurred at the maximum bending moment region. These cracks initiated in the middle and did not extend to the ends of the beam. The debonding was due to the over stress of the cover layer where concrete cracked and spalled, initiating the debonding. The failure mode can be characterized as a combination of concrete crushing in the compression zone and epoxy-to-concrete debonding in the mid-span region. Hassan and Rizkalla [36] stated that the edge distance should be four times the

Table 9

Maximum load carrying capacity; experimental vs. theoretical prediction.

Specimen	Ultimate Loads, (kN)			Ratio	
	(1) ACI Prediction	(2) Experimental	(3) Theoretical	(1)/(2)	(3)/(2)
AC	62.8	76	70.0	0.83	0.92
AR2	96.2	128	117	0.75	0.91
AR3	120.0	102	142	1.17	1.40
AR4	126.7	122	166	1.00	1.40
AR4-S	118.3	118	135	1.00	1.10
BC	36.6	50	44	0.75	0.90
BR2	80.0	80	102	1.00	1.27
BR3	105.4	111	120	0.90	1.08
BR4	120.0	114	149	1.10	1.31

diameter of the bar, which was achieved by using one CFRP bar in specimen AR4-S. However, comparing the results of this specimen to specimen AR2, which had two No. 6 CFRP bars, specimen AR2 had about the same strength but better performance in terms of ductility. On the other hand, AR4-S performed better than AR4 in terms of strength and ductility.

3.3. Beam tests: set (B) specimens

Table 8 summarizes the test results of Set B beam specimens. Fig. 14 shows the load deflection curves for all beams in Set B. The first beam tested in Set B was the control beam BC. The first observed cracking was 0.2 mm at 13.34 kN. The load was then increased until the concrete crushing occurred at about 50 kN. The second beam tested in Set B was BR2. Three distinct regions (zones) were observed from the load-deflection curve of BR2 in Fig. 13. The first region was where the first crack of 0.05 mm occurred at about 13.34 kN, where the beam lost some of its flexural stiffness. The second region was where the steel and the CFRP bars behave elastically. The third region was where the steel

Table 10
Percentage of enhancement.

Beam	Initial stiffness, k kN/mm	$k/k_{control}$	Max. load, P_u (kN)	$P_u/P_{u,control}$	Max. deflection, Δ (mm)	$\Delta/\Delta_{control}$	Ductility, μ	$\mu/\mu_{control}$
AC	4.3	1.00	76	1.00	50	1.00	3.8	1.00
AR2	6.0	1.40	128	1.68	37	0.74	2.5	0.60
AR3	4.5	1.05	102	1.34	27	0.54	1.6	0.43
AR4	6.0	1.40	122	1.61	20	0.40	1.0	0.26
AR4-S	4.5	1.05	118	1.55	28	0.56	1.7	0.45
BC	2.5	1.00	50	1.00	64	1.00	5.0	1.00
BR2	2.6	1.04	80	1.60	45	0.70	3.5	0.70
BR3	3.6	1.44	111	2.22	35	0.55	2.3	0.46
BR4	3.6	1.44	114	2.28	29	0.45	1.0	0.20

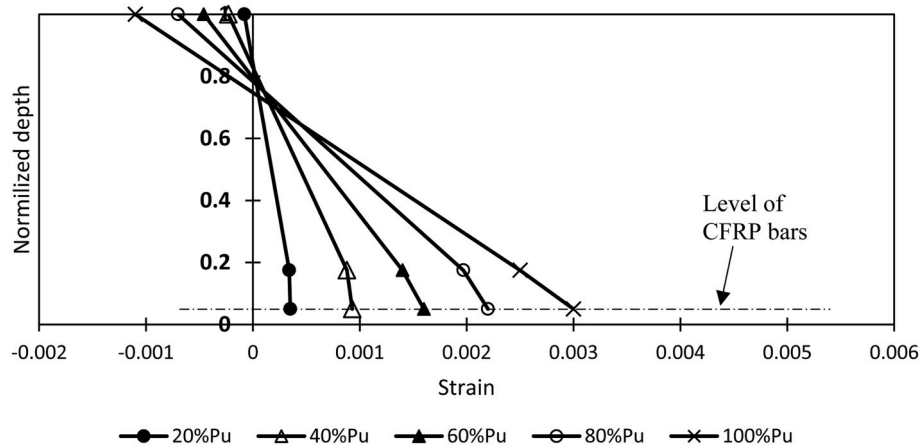


Fig. 15. Mid-span strain distribution of specimen AR4.

reinforcement yielded. In this last stage, more of the flexural stiffness was lost, and more cracks were initiated and widened at the positions of the lateral grooves. As the load increased, the cracks increased in length and width. The load was then increased until concrete cover spalled under one of the load points at about 80 kN (Table 9). Table 10 shows the percentage of enhancement in load, values of maximum deflection and ductility for all specimens. Ductility for each specimen was calculated by dividing the ultimate deflection by the deflection at the yielding of flexural steel reinforcement. As shown in Table 10, the increased load carrying capacity for this beam (BR2) was 60% over the control beam. The initial stiffness increased by only 4%, and the ductility decreased by 30% over the control beam.

The third beam tested in Set B was BR3. From the load deflection response, three distinct regions were observed: the first region was when the first crack of 0.05 mm occurred at a load of about 13.34 kN, when the

beam lost some of its flexural stiffness; the second region was where the steel and the CFRP bars behave elastically; and the third region was where the steel reinforcement yielded. In this last stage, more of flexural stiffness was lost, and more cracks were initiated and widened at the positions of the lateral grooves. As the load increased, the width of cracks increased. The load was then increased until FRP debonding failure occurred in the mid-span region at about 111 kN. The increased load carrying capacity and initial flexural stiffness over the control beam was 122% and 44%, respectively. However, the ductility of the beam decreased by 54% compared to the control beam. Before debonding occurred, some diagonal cracking occurred in the shear span. These cracks initiated at the positions of the lateral grooves and extended to the middle sections of the beam. The debonding was due to the over stress of the beam's tensile layer where the edge concrete cracked and spalled, initiating the debonding. The fourth beam tested in Set B was

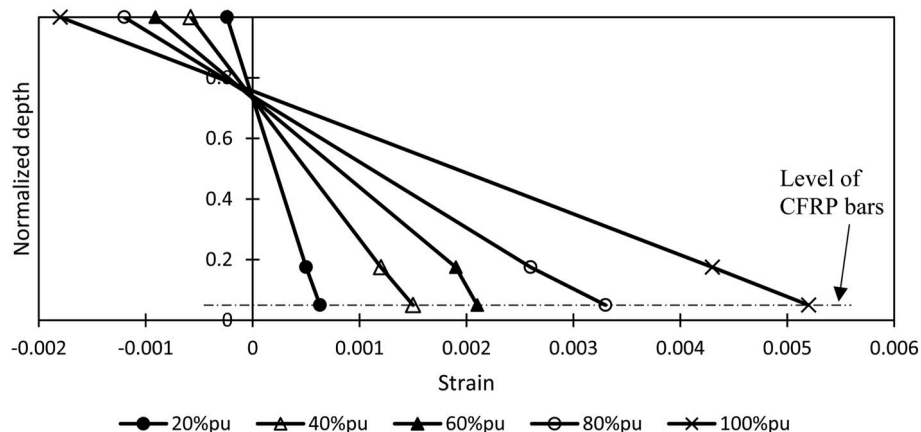


Fig. 16. Mid-span strain distribution of specimen AR4-S.

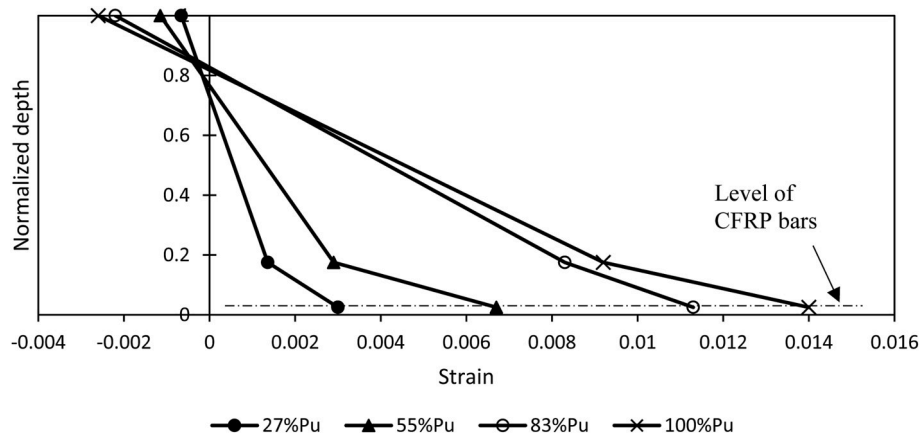


Fig. 17. Mid-span strain distribution of specimen BR2.

BR4. The load deflection response showed two distinct regions. The first region was when the first cracking of 0.04 mm occurred at 22.24 kN, when the beam lost some of its flexural stiffness. The load was then increased until FRP debonding failure at one of the beam ends occurred at about 114 kN, and that was where the second region of the curve was shaped. The increased load carrying capacity and stiffness over the control beam was 128% and 44%, respectively. However, the ductility decreased by 80% over the control beam.

3.4. Comparisons and discussions

From Figs. 11 and 14, it is apparent that adding CFRP bars increased the flexural load carrying capacity and the initial stiffness of the strengthened beams. Another observation was that as the total reinforcement ratio increased, the ductility decreased, as also observed by Barros et al. [37]. Generally, NSM-CFRP technique increases the load carrying capacity but decreases the ductility of the strengthened beams [12,37]. As it was found by El-Hacha et al. [9], the deflections of the NSM-CFRP beams were smaller than that of the un-strengthened beams at all load levels. From CFRP, steel, and concrete strain gauges, the strain profile at mid-span was constructed for three selected specimens; AR4, AR4-S, and BR2, as shown in Figs 15, 16 and 17 respectively. The y-axis of the graphs represents the depth ratio (normalized depth) of the cross section, hence “1” means top of the beam (1 h) that shows the concrete strain, and “0.2” means 0.2 h above the bottom representing the steel location, and approximately “0.05” means 0.05 h above the bottom representing the CFRP bar location. For BR2 (Fig. 17), where No. 6 CFRP bar was used, the level of CFRP was 0.025 h. The term “Pu” represents the failure load. It can be observed that the strains in AR4 (Fig. 15) and AR4-S’s sections (Fig. 16) are reasonably linear up to the failure. The same observation was reported by Hosen et al. [38]. The strain at the steel level barely exceeds the yield strain for AR4 and was two times the yield strain for AR4-S. These low strain values point to the brittle behavior of the specimens. However, specimen BR2 (Fig. 17) exhibited non-linear proportion in the strain profile. The strain of the steel was 0.0092, which was much larger than the yield strain. The CFRP strain was 0.014 which was close to the rupture value. These values point to the ductile behavior of specimen BR2 compared to the other specimens. Sharaky et al. [18] used mechanical interlocking with steel shear connectors to delay or prevent debonding failure and to increase the efficiency of NSM-CFRP technique. Sharaky et al. suggested to design NSM-CFRP strengthened beams so that the tensile steel yielded and the CFRP achieved high strain before failure. Designing strengthened beams like specimen BR2 in the current study would achieve both of the above criteria. The improvement in load carrying capacity for BR2 was 60%, and the loss was 30% in terms of ductility. Among all strengthened beams, the decrease in ductility was the least for specimen BR2

compared to the control beams. Table 10 compares the percentages of enhancement for all beam specimens tested in the current study.

As shown in Table 7, comparing specimen AR2 (two No. 6 CFRP bars and a total reinforcement ratio of 0.91) with specimen AR4-S (one No. 10 CFRP bar and a total reinforcement ratio of 1.09), it was observed that specimen AR2 had a higher load capacity and ductility than those of specimen AR4-S. That can be attributed to the fact that tensile stresses in specimen AR4-S are concentrated around the single bar of CFRP, whereas, for specimen AR2 the stresses are shared by the two CFRP bars. As a result, in specimen AR4-S the concrete layer around the CFRP bar peeled off before the CFRP bar reached its capacity resulting in debonding failure. Therefore, using smaller size of CFRP bars rather than a large size with same cross-section area resulted in a better behavior in terms of ductility and overall capacity [39]. By comparing Set A to Set B results, it is observed that adding CFRP bars to Set B specimens worked more efficiently than adding the CFRP bar sizes to Set A specimen. Soliman et al. [3] reported that 0.4% is the optimum steel reinforcement ratio to increase the capacity by adding CFRP bars. Tables 7 and 8 reveal that the optimum total reinforcement ratio was within the range of 0.58 to 0.91%. A reinforcement ratio larger than 0.91% resulted in premature debonding CFRP failure with sharp drop in load capacity. On the other hand, a reinforcement ratio smaller than 0.58% would result in smaller added load capacity. As the reinforcement ratio ranged between 0.58% to 0.91%, the increased capacity ranged from 60% to 128%, respectively. Sharaky et al. [18] observed that the increased load carrying capacity for strengthened specimens without mechanical interlocking and shear connectors ranged between 50% to 70% over the control specimens. On the other hand, the improvement in load carrying capacity ranged from 85% to 122% for specimens strengthened with NSM-CFRP and mechanical interlocking.

The ductility of NSM-CFRP strengthened beams is another important property that requires review and a better understanding. As an example, the load carrying capacity of AR2 was improved by 68%, but the ductility was reduced by 40% compared to the control beam (Table 10). The total reinforcement ratio of specimen AR2 was 0.91%. As another example, the load carrying capacity of BR3 was improved by 122%, but the ductility was reduced by 54% compared to the control beam. The total reinforcement ratio of specimen BR3 was 0.80%. It was reported previously that the ductility of beam specimens strengthened with NSM-CFRP bars with lateral grooves and shear connectors was 13% higher than that of beam specimens strengthened with conventional NSM-CFRP bars [18]. Therefore, it appears necessary and prudent to evaluate the system before applying the strengthening scheme. By knowing the type of reinforcement of the section before adding the CFRP bars, the desired ultimate capacity and the ductility can be reached by selecting a proper total reinforcement ratio. Debonding failure must be accounted for in the strengthening design.

Table 11
Debonding CFRP strains.

Label	Ultimate strains based on			Debonding CFRP strains based on:			
	Experiment (measured)	ACI analysis (section 4.1)	Theoretical analysis (section 4.2.2)	ACI [35]	Barros et al. [37]	Hassan and Rizkalla [20]	Present study (Eq. (15))
AC	—	—	—	—	—	—	—
AR2	0.011	0.009	0.012	0.0120	0.012	0.019	0.016
AR3	0.006	0.007	0.008	0.0116	0.012	0.011	0.010
AR4	0.003	0.005	0.006	0.0111	0.011	0.007	0.006
AR4-S	0.005	0.007	0.008	0.0111	0.011	0.009	0.009
BC	—	—	—	—	—	—	—
BR2	0.014	0.012	0.014	0.0120	0.015	0.019	0.016
BR3	0.006	0.009	0.009	0.0116	0.014	0.013	0.010
BR4	0.004	0.006	0.006	0.0111	0.013	0.007	0.006

From Tables 7 and 8, it can be observed that the ductility and the ultimate CFRP strains in set B were slightly larger than the corresponding ones in set A. On the other hand, the effect of strengthening set A beams and set B beams on the initial flexural stiffness and ultimate deflection was almost the same. To summarize the difference between set A and set B beams, Table 10 compares the initial stiffness, maximum load, ductility, and maximum deflection for all specimens in set A and set B. The percentages of enhancement for all previous factors are also presented in the table. Since set A specimens have larger steel reinforcement ratios, the percentage of enhancement in load carrying capacity for set B specimens was greater than that of set A specimens. However, the difference in steel reinforcement ratios between set A and set B specimens did not significantly affect the percentages of enhancement in terms of initial stiffness, maximum deflection, and ductility. Soliman et al. [3] also reported that 0.4% (similar to the steel reinforcement ratio of set B specimens) was the optimum steel reinforcement ratio to increase the flexural load carrying capacity of the strengthen RC beams.

4. Theoretical analysis

4.1. ACI prediction

The guidelines proposed by ACI 318-14 [40] and ACI 440.2R-17 [35] were used to predict the flexural strength of rectangular reinforced concrete beams strengthened with NSM CFRP bars. ACI 440.2R-17 [35] assumes that (1) bonding between CFRP bars, epoxy, and concrete is perfect, (2) plane sections before bending remain plane after bending, (3) maximum strain in concrete is 0.003, (4) CFRP bars have linear-elastic tensile behavior until failure, and (5) strain compatibility and equilibrium condition of the beam cross section must be satisfied. This document [35] introduces the following equations to consider the environmental reduction factor and the bonding coefficient;

$$f_{fu} = C_E f_{fu}^* \text{ ACI 440.2R - 17 (9.4a)} \quad (1)$$

$$\epsilon_{fu} = C_E \epsilon_{fu}^* \text{ ACI 440.2R - 17 (9.4b)} \quad (2)$$

$$\epsilon_{fd} = k_e \epsilon_{fu} \text{ ACI 440.2R - 17 (10.1.1)} \quad (3)$$

where f_{fu} is the design rupture stress of FRP reinforcement (MPa), f_{fu}^* is the ultimate rupture stress of FRP reinforcement (MPa), ϵ_{fu} is the design rupture strain of FRP reinforcement, ϵ_{fu}^* is the ultimate rupture strain of FRP reinforcement, ϵ_{fd} is the debonding strain of FRP reinforcement, C_E is the environmental reduction factor (ACI 440.2R-17; Table 9.4), and k_e is the bonding coefficient and is recommended to be taken as 0.7 [35]. Based on ACI 440.2R-17, the environmental reduction factor for interior exposure conditions can be taken as 0.95, and that is what was used in this study. An iterative solution was guided by assuming a value for “c”,

the distance from the neutral axis to the top fiber of concrete. This assumption was checked by using the equilibrium condition as explained in equations (4) and (5) below. Equation (4) is used when concrete crushing controls the design, and equation (5) is used when FRP rupture controls the design;

$$\epsilon_{fe} = \epsilon_{cu} \left(\frac{d_f - c}{c} \right) \text{ ACI 440.2R - 17 (10.2.5)} \quad (4)$$

$$\epsilon_{fe} = \epsilon_{bi} + \epsilon_{fd} \text{ ACI 440.2R - 17 (10.3.1.2a)} \quad (5)$$

Where ϵ_{fe} is the effective strain of FRP reinforcement attained at failure, ϵ_{cu} is the ultimate axial strain of concrete, d_f is the effective depth of FRP flexural reinforcement (mm), c is the distance from the extreme compression fiber to the neutral axis (mm), ϵ_{bi} is the initial strain in prestressing steel reinforcement, and ϵ_{fd} is as defined earlier. Based on the theory of strain compatibility, the strain at the steel and the concrete can be determined from the following relationships:

$$\epsilon_s = (\epsilon_{fe} + \epsilon_{bi}) \left(\frac{d - c}{d_f - c} \right) \text{ ACI 440.2R - 17 (10.2.10a)} \quad (6)$$

$$f_{fe} = E_f \epsilon_{fe} \text{ ACI 440.2R - 17 (10.2.6)} \quad (7)$$

$$f_s = E_s \epsilon_s \leq f_y \text{ ACI 440.2R - 17 (10.2.10b)} \quad (8)$$

$$c = \frac{A_s f_s + A_f f_{fe}}{\alpha_1 f_c' \beta_1 b} \text{ ACI 440.2R - 17 (10.2.10c)} \quad (9)$$

where ϵ_s is the strain in steel reinforcement, d is the effective depth of steel flexural reinforcement (mm), f_{fe} is the effective stress in the FRP at the failure (MPa), E_f is the tensile modulus of elasticity of FRP (MPa), f_s is the stress in steel reinforcement (MPa), E_s is the tensile modulus of elasticity of steel reinforcement (MPa), f_y is the specified yield stress for steel reinforcement (MPa), A_s is the area of steel reinforcement (mm²), A_f is the area of FRP (mm²), α_1 is a multiplier on f_c' to determine intensity of an equivalent stress distribution of concrete, f_c' is the specified compressive strength of concrete (MPa), β_1 is the ratio of depth of equivalent rectangular stress block to depth of the neutral axis, and b is the width of compression face of the concrete member (mm). After the strain compatibility and equilibrium conditions were satisfied, the following equation may be used to obtain the flexural moment capacity (M_n) of the concrete section;

$$M_n = A_s f_s \left(d - \frac{\beta_1 c}{2} \right) + \psi_f A_f f_{fe} \left(d_f - \frac{\beta_1 c}{2} \right) \text{ ACI 440.2R - 17 (10.2.10d)} \quad (10)$$

Where ψ_f is a reduction factor of 0.85 statistically suggested to account for the uncertainty of CFRP strength. The procedure described above was followed to predict the flexural capacity of all tested specimens. The results of the analysis are reported in Table 9.

4.2. Theoretical prediction

4.2.1. CFRP predicted strain

One of the main limits controlling the theoretical prediction of the flexural behavior of NSM-CFRP concrete beams is the debonding strain of CFRP bars. Some of the previous studies focused on predicting the debonding strain of NSM-FRP reinforcement. Barros et al. [37] stated that as the internal steel reinforcement increases, the ultimate strain in the NSM-CFRP reinforcement decreases. Barros et al. [37] proposed the following formula to predict the ultimate strain in the NSM-CFRP reinforcement.

$$\varepsilon_{fd} = (-32.648 * \rho_{l,eq} + 0.9606) * \varepsilon_{fu} \quad (11)$$

where $\rho_{l,eq}$ can be defined as the following:

$$\rho_{l,eq} = A_{st} / (b_w * d_s) + (A_f * E_f / E_s) \vee (b_w * d_f) \quad (12)$$

where ε_{fu} is the design rupture strain of CFRP reinforcement, A_s is the area of steel reinforcement (mm^2), A_f is the area of CFRP (mm^2), E_f and E_s is the tensile modulus of elasticity of FRP and steel respectively (MPa), d_s and d_f is the effective depth of steel and CFRP reinforcement respectively (mm), and b_w is the width of the beam. By applying Eq. (11) and Eq. (12) to the specimens of the current study, the ultimate strains for each beam specimen were listed in Table 11.

Hassan and Rizkalla [20] followed ACI guidelines to calculate the development length of NSM-CFRP bars, which was used to predict the debonding stress for NSM-CFRP bars. Two scenarios were assumed: concrete-to-epoxy de-bonding (peeling off) and FRP-to-epoxy de-bonding stresses. In addition, the authors proposed reduction factors to account for the spacing and edge distance of the NSM-FRP bars.

$$\sigma_{con-epoxy} = \frac{G_1 * d^{*f}_{FRP}}{\mu * 4 * L_d} \quad (13)$$

$$\sigma_{FRP-epoxy} = \frac{G_2 * d^{*f}_{FRP}}{\mu * 4 * L_d} \quad (14)$$

where $\sigma_{con-epoxy}$ and $\sigma_{FRP-epoxy}$ are the tensile stresses at the concrete-epoxy and FRP-epoxy interfaces, respectively, f_{FRP} is the tensile stress delivered to NSM-FRP bar before de-bonding, L_d is the bond length needed to develop f_{FRP} , d is the diameter of FRP bar, and μ is the friction coefficient between epoxy and bar and assumed to be 0.66 for FRP bars with rough surface. G_1 and G_2 are numerical coefficients related to the geometry of the grooves. These coefficients were determined based on a finite element analysis and can be extracted from design charts provided by the authors [20].

The theory advanced by Hassan and Rizkalla [20] was applied to the beam specimens of the present study to determine FRP debonding strains. $\sigma_{con-epoxy}$ and $\sigma_{FRP-epoxy}$ may be assumed to be equal to the tensile stresses of concrete (2.35 MPa) and epoxy (12.0 MPa), respectively. Since CFRP bars used in this study were sand coated, the value of μ was taken as 0.66. The values of G_1 and G_2 were extracted from the design charts [20]. Finally, f_{FRP} can be calculated by using Eq. (13) and Eq. (14). Since linear elastic stress-strain relationship was assumed for CFRP bars, debonding strains were determined by multiplying the values of f_{FRP} by the modulus of elasticity. Based on the obtained results, Eq. (13) resulted in smaller strain values for all specimens, indicating that the failure occurred at the concrete-to-epoxy interface. The debonding strains based on Eq. (13) for all beam specimens of this study are tabulated in Table 11.

In the current study, two reduction factors were considered to determine CFRP debonding strains. The first reduction factor was empirical based on the results of pullout tests on NSM-CFRP bars with lateral grooves. Based on the pullout tests, the empirical reduction factors for determining the debonding strains of NSM-CFRP bars with lateral grooves was 0.89 for No. 6 CFRP bars, 0.64 for No. 10 CFRP bars,

Table 12

Proposed empirical reduction factors and debonding strains.

Beam specimen	R _{F1}	R _{F2}	ε_{fu}	ε_{fd}
AC	—	—	—	—
AR2	0.89	1.00	0.0181	0.016
AR3	0.64	0.85	0.0175	0.010
AR4	0.52	0.72	0.0167	0.006
AR4-S	0.52	1.00	0.0167	0.009
BC	—	—	—	—
BR2	0.89	1.00	0.0181	0.016
BR3	0.64	0.85	0.0175	0.010
BR4	0.52	0.72	0.0167	0.006

and 0.52 for No. 13 CFRP bars. The second reduction factor (adopted from Hassan and Rizkalla [20]) was related to the groove geometry, edge distance, and spacing between CFRP bars since these parameters were not considered in the pullout tests conducted in this study. The spacing between NSM-CFRP bars for all strengthened beam specimens used in this study satisfied the requirements and did not require a reduction factor. However, the edge distances for some of NSM-CFRP bars (specimens AR3, BR3, AR4, and BR4) were not adequate based on the requirement of Hassan and Riskalla [20], and therefore reduction factors were considered. The edge distance reduction factor was 0.85 for AR3 and BR3 and was 0.72 for AR4 and BR4. Therefore, the maximum CFRP debonding strains proposed in this study can be calculated using the following equation:

$$\varepsilon_{fd} = R_{F1} * R_{F2} * \varepsilon_{fu} \quad (15)$$

where ε_{fd} is the maximum debonding CFRP strain, R_{F1} is the empirical reduction factor based on the pullout tests, R_{F2} is the spacing and edge distance reduction factor adopted from Hassan and Rizkalla [20], and ε_{fu} is the guaranteed tensile strength of CFRP bars provided by the manufacturer. The reduction factors and strain results using Eq. (15) are presented in Table 12. Table 11 compares the measured CFRP strains (experimental results) to the predicted strains resulted from ACI analysis (section 4.1) and the theoretical analysis of this study (section 4.2.2). In addition, the debonding strains proposed by ACI [35], Barros et al. [37], Hassan and Rizkalla [20], and the present study are tabulated in Table 11. It should be noted that any realistic empirical model must be developed based on statistically meaningful data. However, to the best of authors knowledge, no data from the literature was found to be included in the analysis. Therefore, the empirical factors proposed here are limited to the pullout tests that were conducted in this study as the groove characteristics and the geometry of the mechanical interlocking used in the pullout tests are similar to those used in the strengthened beams.

4.2.2. Flexural analysis

Recent studies [41] have employed ACI guidelines [35] to calculate the cracking moment, yield moment and the ultimate moment of the beams. The mid-span deflection was calculated as a function of the bending moment (cracking, yielding, and ultimate) and the effective moment of inertia. Although debonding was observed in the strengthened beams based on the experimental tests, Abdallah et al. [41] did not consider debonding of CFRP as one of the failure modes in their proposed model. Sharaky et al. [42] considered the debonding strain of NSM-FRP bars, but their strengthening technique did not include lateral grooves. In the present study, CFRP debonding strain was one of the limits considered in the empirical model. The flexural analysis was based on developing a moment-curvature relationship for a reinforced concrete beam. First, some assumptions were made, such as (1) plain sections remain plain after bending, (2) perfect bond between the materials is assumed, (3) stress-strain relationship of steel rebars is as described in Fig. 3, (4) stress-strain relationship of CFRP bars is as described in Fig. 1, and (5) stress-strain relationship of concrete material

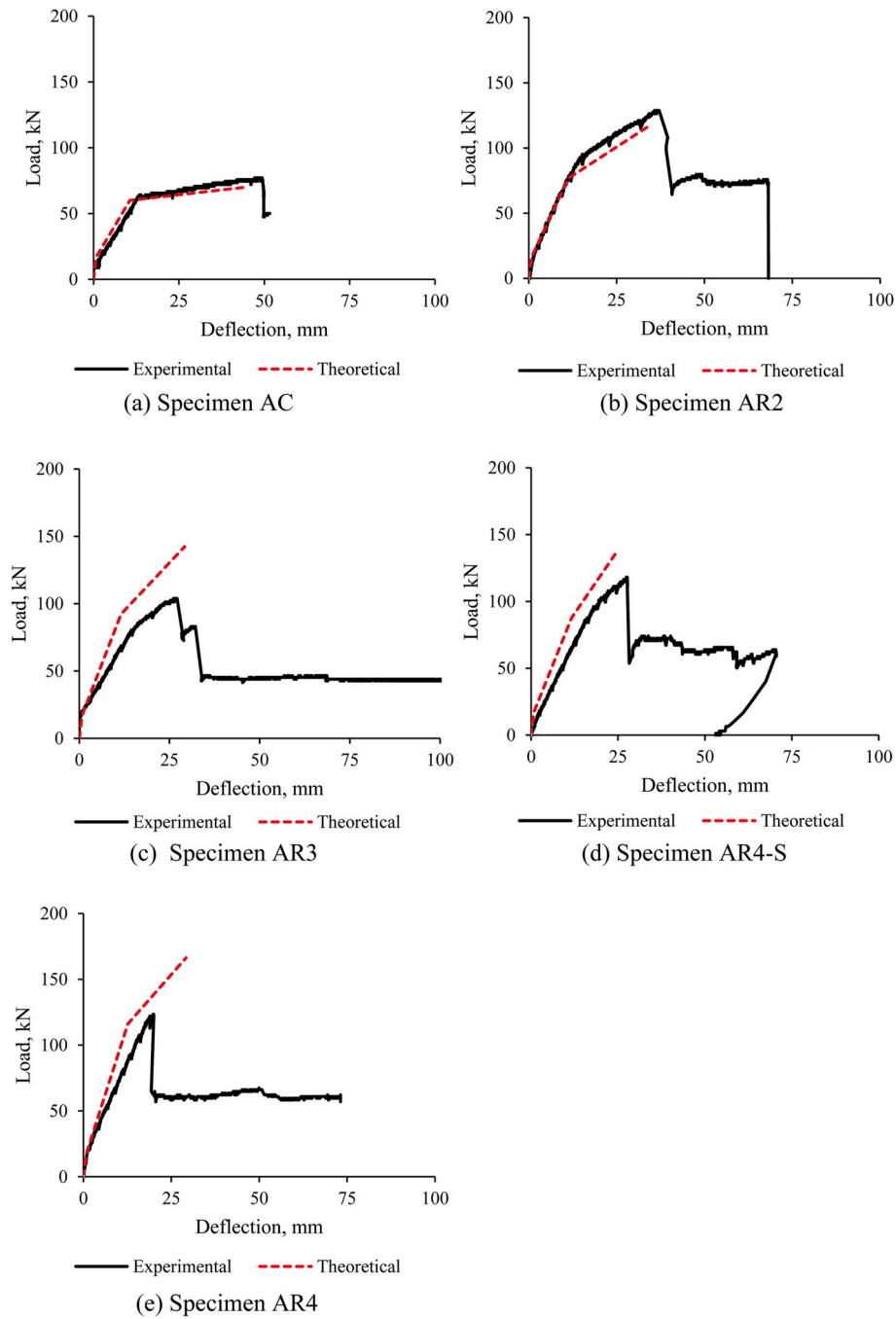


Fig. 18. Theoretical vs. experimental load-deflection curves – Set A.

is as described in equations (16) and (17). Stress-strain relationships for steel and CFRP materials were determined experimentally in the laboratory. However, the concrete stress-strain relationship was adopted from Kent and Park [43]. The relationship is defined by equations (16) and (17) below;

$$f_c = f'_c \left[\frac{2\varepsilon_c}{\varepsilon_{co}} - \left(\frac{\varepsilon_c}{\varepsilon_{co}} \right)^2 \right] \text{ when } \varepsilon_c \leq \varepsilon_{co} \quad (16)$$

$$f_c = f'_c [1 - Z(\varepsilon_c - \varepsilon_{co})] \text{ when } \varepsilon_c \geq \varepsilon_{co} \quad (17)$$

where;

$$Z = \frac{0.5}{\varepsilon_{50u} - \varepsilon_{co}} \quad (18a)$$

$$\varepsilon_{50u} = \frac{3 + 0.29 f'_c}{145 f'_c - 1000} \quad (18b)$$

where f_c is the compressive strength of concrete (MPa), f'_c is the specified compressive strength of concrete (MPa), ε_c is the axial strain of concrete, ε_{co} is the concrete strain at maximum compressive strength (assumed to be 0.002), and ε_{50u} is the strain at 50% of the maximum concrete strength.

In order to compute the load deflection response, the moment curvature diagram was constructed first. Three main points were defined to construct the moment-curvature diagram. The first point in the moment-curvature relationship was at the cracking moment. Concrete was assumed to be cracked when the tensile stress at the bottom fiber exceeded the cracking strength of concrete defined by ACI 318 [40]

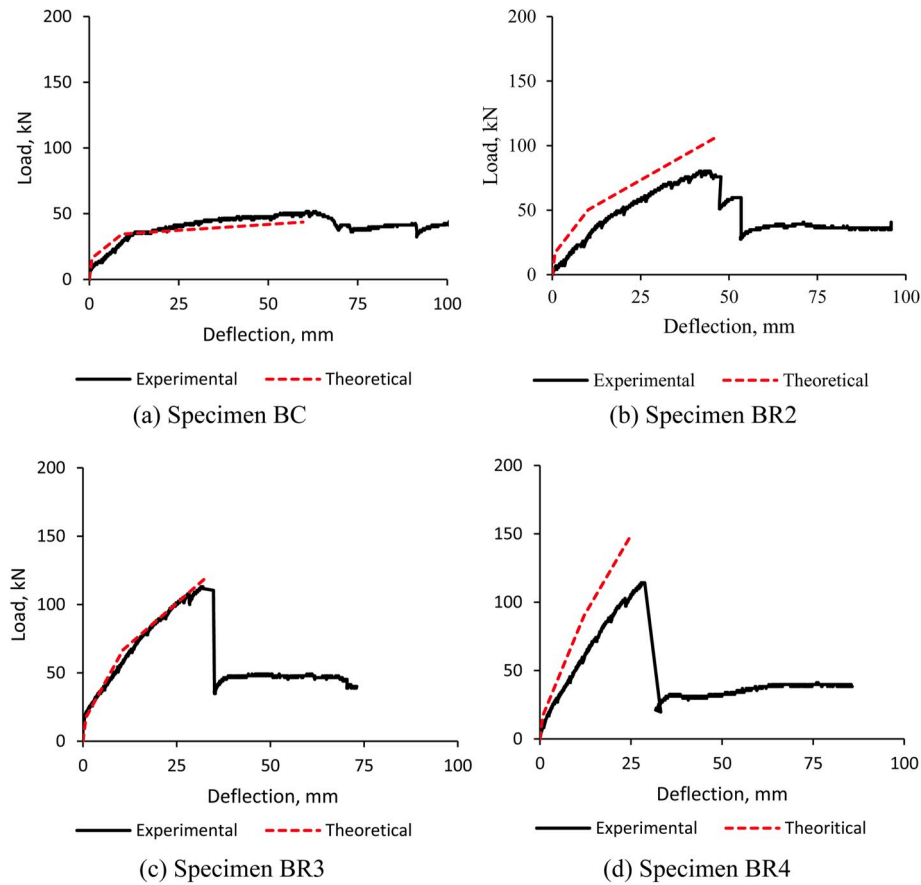


Fig. 19. Theoretical vs. experimental load-deflection curves – Set B.

($0.62\sqrt{f'_c}$). The second point on the curve was the moment corresponding to steel yielding. Based on the experimental tests performed on the steel reinforcement bars, steel yield strain was 0.0025. Finally, the third point representing the ultimate theoretical moment capacity was defined when the concrete strain (????) reached 0.003, or the strain at CFRP bars reached the maximum debonding strain (Eq. (15) and Table 12).

At each increment of concrete strain, the compression force of the concrete was determined by integration the area under the concrete stress-strain curve. Equilibrium equation for the internal forces of the cross section and strain compatibility equations were implemented to find the depth of the uncracked section (c) at the assumed concrete strain. Finally, the strains in steel and CFRP bars were determined based on the strain compatibility of the cross section. The strain of CFRP was then checked against the maximum de-bonding strain (Eq. (15)), and the strain in steel was checked against the assumed steel strain level (elastic, yielding or hardening). Finally, if all assumptions were correct, the capacity of the section at the assumed concrete strain was determined. The moment was calculated by multiplying the forces of each material by their respective lever arms. The curvature was calculated by dividing the concrete strain by the distance from the neutral axis to the concrete top fiber. Load-deflection curves were then derived from load-curvature curves by using the common integration method. A flowchart explaining the calculation steps is provided in the appendix.

4.3. Discussion of theoretical results

Table 9 shows the ultimate loads for each specimen based on the provisions of ACI codes [35,40], experimental values, and the theoretical values as explained in section 4.2. The ratios of ACI predictions over experimental, and theoretical prediction over experimental results are

also shown. It can be observed that both ACI and the proposed theoretical predictions were in a reasonable agreement with the experimental results. However, the ultimate loads according to the theoretical analysis (section 4.2) seemed to be generally higher than the experimental results for specimens with high CFRP reinforcement ratios. This could be attributed to the fact that epoxy deformation and CFRP slipage were not considered in the theoretical analysis. CFRP bars were assumed to be fully bonded to the concrete layer, which resulted in the higher predicted load values. To make the prediction of the theoretical analysis more accurate, CFRP ultimate strains were limited to the debonding strain proposed in section 4.2.1. In general, the method of ACI prediction (section 4.1) resulted in lower load values than the proposed theoretical analysis mainly because it reduced the values of CFRP ultimate strains by environmental reduction factor (C_E) and bonding coefficient (k_e). In addition, ACI method reduced FRP tensile force by 15% ($\psi_f = 0.85$) when calculating the resistant moment capacity of the strengthened beams, as explained in equation (10). Sharaky et al. [44] reported that the predicted maximum load carrying capacity was 37% higher than the experimental value for a beam strengthened with two 8 mm NSM carbon FRP bars, and it was 22% higher than the experimental results for a beam strengthened with two 8 mm NSM glass FRP bars. The total reinforcement ratio of the strengthened beams was 0.83%. In the current study, the theoretical results showed good agreement with the experimental results for specimens with a total reinforcement ratio less than 1.1% (Table 9). However, for specimens with a total reinforcement ratio greater than 1.1%, the predicted load carrying capacity was at least 31% higher than the experimental results. Therefore, probably a reduction factor (ψ_f), like what ACI uses to reduce FRP strength, should be applied to have a better estimation for strengthened beams with a total reinforcement ratio higher than 1.1%.

Fig. 18 and Fig. 19 compare the theoretical and experimental load-

deflection curves for all specimens. The overall behavior of the theoretical load-deflection curves was comparable to the experimental results. However, as stated before, the ultimate load carrying capacities were over-estimated for specimens with a total reinforcement ratio greater than 1.1%. Moreover, the predicted load-deflection curves have higher flexural stiffnesses mainly because CFRP bars were assumed to be fully bonded. Thus, no CFRP slippage or epoxy deformation were accounted for in the theoretical analysis. CFRP slippage reduces the measured beam flexural stiffness and resulted in larger deflection than the predicted deflection under the same applied load.

5. Conclusions

An experimental study was carried out to investigate the efficiency of strengthening rectangular reinforced concrete beams with NSM-CFRP bars. The bond of the NSM-CFRP bars was strengthened with lateral grooves (or mechanical interlocking). First, pullout tests were conducted on three different sizes of NSM-CFRP bars. Then, a total of nine RC beams were tested (seven beams were strengthened with three different sizes of NSM-CFRP bars, and two beams were the control specimens) in flexure under four-point loading. The test results were analyzed and compared to the analytical calculations. Based on the results obtained in this study, the following conclusions can be drawn:

1. Based on the results of the pullout tests, the lateral grooves enhanced the bond capacity of NSM-CFRP bars and delay the premature debonding failure. The efficiency of NSM-CFRP bars were 89%, 64%, and 52% for No. 6, No. 10, and No. 13 CFRP bars, respectively. As a result, the debonding strains were 0.016, 0.011, and 0.0086 for No. 6, No. 10, and No. 13 CFRP bars, respectively.
2. NSM CFRP application with mechanical interlocking (lateral grooves) was effective in increasing the flexural load carrying capacity. However, the ductility was reduced for the strengthened beams, especially for beams with a total reinforcement ratio greater than 1.1%.
3. The failure mode of strengthened specimen was mainly related to CFRP debonding failure and peeling off the concrete cover. The mechanical interlocking delayed the premature bonding failure. The measured CFRP strains at failure ranged from 0.3% to 1.4% depending on the CFRP size and total reinforcement ratios.
4. Strengthening was more effective for specimens that had lower steel reinforcement ratios. Strengthening the control beam in Set A (steel reinforcement ratio = 0.7%) improved the flexural load carrying capacity by a maximum of 68%, whereas strengthening the control beam in Set B (steel reinforcement ratio = 0.4%) improved the flexural capacity by a maximum of 128%.
5. Based on the results of this study, using smaller size of CFRP bars rather than larger size with same cross-sectional area resulted in a better performance in terms of ductility and overall flexural response.
6. The theoretical analysis had a reasonable agreement with experimental and ACI prediction in term of maximum moment capacity and failure mode.
7. The theoretical load-deflection curves were generally stiffer than those obtained from the experimental results. It is recommended to reduce the load-carrying capacity by 15% ($\psi_f = 0.85$) for strengthened beams with a total reinforcement greater than 1.1% to match the experimental results.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

CRediT authorship contribution statement

Salam Al-Obaidi: Conceptualization, Methodology, Software, Formal analysis, Investigation, Writing - original draft. **Yasir M. Saeed:** Conceptualization, Methodology, Formal analysis, Investigation, Writing - review & editing. **Franz N. Rad:** Conceptualization, Validation, Writing - review & editing, Supervision.

Acknowledgments

The authors express gratitude to the Higher Committee for Education Development in Iraq (HCED) for financially supporting the research project, the Department of Civil & Environmental Engineering at Portland State University for providing laboratory and technical support, and friends and colleagues who helped with experimental work.

Appendix A

Fig. A1 shows the calculations explained in section 4.2.

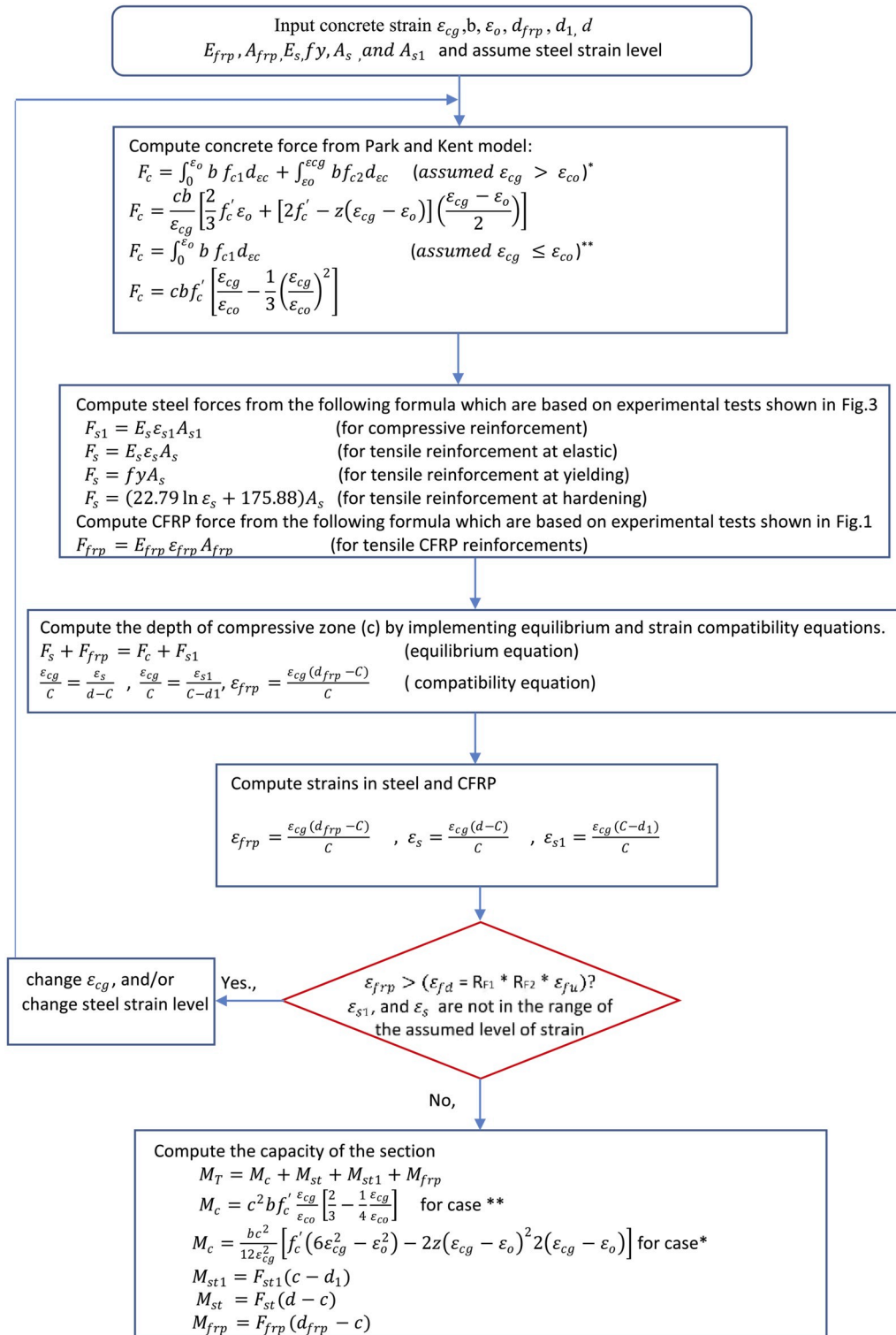


Fig. A.1. Flowchart explaining procedure of calculations explained in section 4.2.

References

- [1] B. Täljsten, A. Carolin, H. Nordin, Concrete structures strengthened with near surface mounted reinforcement of CFRP, *Adv. Struct. Eng.* 6 (2003) 201–213, <https://doi.org/10.1260/13694330322419223>.
- [2] W.T. Jung, Y.H. Park, J.S. Park, J.Y. Kang, Y.J. You, Experimental investigation on flexural behavior of RC beams strengthened by NSM CFRP reinforcements, in: 7th Int. Symp. Fiber-Reinforced Polym. Reinf. Concr. Struct., 2005, pp. 795–806.
- [3] S.M. Soliman, E. El-Salakawy, B. Benmokrane, Flexural behaviour of concrete beams strengthened with near surface mounted fibre reinforced polymer bars, *Can. J. Civ. Eng.* 37 (2010) 1371–1382, <https://doi.org/10.1139/L10-077>.

- [4] F. Micelli, L. De Lorenzis, Near-surface mounted flexural strengthening of reinforced concrete beams with low concrete strength, *Proc. Inst. Civ. Eng. Constr. Mater.* 166 (2013) 295–303, <https://doi.org/10.1680/coma.11.00056>.
- [5] F. Al-Mahmoud, A. Castel, R. François, C. Tourneur, Strengthening of RC members with near-surface mounted CFRP rods, *Compos. Struct.* 91 (2009) 138–147, <https://doi.org/10.1016/j.compstruct.2009.04.040>.
- [6] Z.K. Szabó, G.L. Balázs, Near surface mounted FRP reinforcement for strengthening of concrete structures, *Period. Polytech. Civ. Eng.* 51 (2007) 33–38, <https://doi.org/10.3311/pp.ci.2007-1.05>.
- [7] M. Mahal, B. Täljsten, T. Blanksvärd, Experimental performance of RC beams strengthened with FRP materials under monotonic and fatigue loads, *Construct. Build. Mater.* 122 (2016) 126–139, <https://doi.org/10.1016/j.conbuildmat.2016.06.060>.
- [8] I.A. Sharaky, L. Torres, M. Baena, C. Miàs, An experimental study of different factors affecting the bond of NSM FRP bars in concrete, *Compos. Struct.* 99 (2013) 350–365, <https://doi.org/10.1016/j.compstruct.2012.12.014>.
- [9] R. El-hacha, S.H. Rizkalla, Near-surface-mounted fiber-reinforced polymer reinforcements for flexural strengthening of concrete structures, *ACI Struct. J.* 101 (2004), <https://doi.org/10.14359/13394>.
- [10] L. De Lorenzis, A. Nanni, Strengthening of reinforced concrete structures with near surface mounted FRP rods previous work on NSM rods, in: *Int. Meet. Compos. Mater.*, 2000, pp. 1–8. Milan, Italy, <http://www.concrete.org/Publications/InternationalConcreteAbstractsPortal.aspx?m=details&i=11534>.
- [11] S.Y. Seo, L. Feo, D. Hui, Bond strength of near surface-mounted FRP plate for retrofit of concrete structures, *Compos. Struct.* 95 (2013) 719–727, <https://doi.org/10.1016/j.compstruct.2012.08.038>.
- [12] G. Wu, Z.Q. Dong, Z.S. Wu, L.W. Zhang, Performance and parametric analysis of flexural strengthening for RC beams with NSM-CFRP bars, *J. Compos. Construct.* 18 (2014), [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000451](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000451).
- [13] A. Palmieri, S. Matthys, L. Taerwe, Experimental investigation on fire endurance of insulated concrete beams strengthened with near surface mounted FRP bar reinforcement, *Compos. B Eng.* 43 (2012) 885–895, <https://doi.org/10.1016/j.compositesb.2011.11.061>.
- [14] S.B. Singh, A.L. Reddy, C.P. Khatri, Experimental and parametric investigation of response of NSM CFRP-strengthened RC beams, *J. Compos. Construct.* 18 (2014), [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000411](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000411).
- [15] R. El-hacha, S.H. Rizkalla, R. Kotynia, Modelling of reinforced concrete flexural members strengthened with near-surface mounted FRP reinforcement, *Proc. 7th Int. Symp. Fiber Reinf. Polym. Reinf. Concr. Struct. (FRPRCS-7)*, New Orleans, SP-230 (2005) 1681–1700.
- [16] A. Siddika, M.A. Al Mamun, R. Alyousef, Y.H. Mugahed Amran, Strengthening of reinforced concrete beams by using fiber-reinforced polymer composites: a review, *J. Build. Eng.* 25 (2019), <https://doi.org/10.1016/j.jobbe.2019.100798>.
- [17] A. Astorga, H. Santa Maria, M. Lopez, Behavior of a concrete bridge cantilevered slab reinforced using NSM CFRP strips, *Construct. Build. Mater.* 40 (2013) 461–472, <https://doi.org/10.1016/j.conbuildmat.2012.09.095>.
- [18] I.A. Sharaky, M. Baena, C. Barris, H.E.M. Sallam, L. Torres, Effect of axial stiffness of NSM FRP reinforcement and concrete cover confinement on flexural behaviour of strengthened RC beams: experimental and numerical study, *Eng. Struct.* 173 (2018) 987–1001, <https://doi.org/10.1016/j.engstruct.2018.07.062>.
- [19] W.C. Tang, R.V. Balendran, A. Nadeem, H.Y. Leung, Flexural strengthening of reinforced lightweight polystyrene aggregate concrete beams with near-surface mounted GFRP bars, *Build. Environ.* 41 (2006) 1381–1393, <https://doi.org/10.1016/j.buildenv.2005.05.029>.
- [20] T.K. Hassan, S.H. Rizkalla, Bond mechanism of near-surface-mounted fiber-reinforced polymer bars for flexural strengthening of concrete structures, *ACI Struct. J.* 101 (2004) 830–839, <https://doi.org/10.14359/13458>.
- [21] ASTM C39/C39M-18, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, 2018. West Conshohocken, PA.
- [22] ASTM C496/C496M-17, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens, 2017. West Conshohocken, PA.
- [23] ASTM C78/C78M -18, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading), 2018. West Conshohocken, PA.
- [24] ASTM D7205/D7205M - 06(2016), Standard Test Method for Tensile Properties of Fiber Reinforced Polymer Matrix Composite Bars, 2016. West Conshohocken, PA.
- [25] S.M. Al-Obaidi, Behavior of Reinforced Concrete Beams Retrofitted in Flexure Using CFRP-NSM Technique, Portland State University, 2015.
- [26] Y.M. Saeed, F.N. Rad, Experimental investigation of CFRP prestressed concrete beams, *ACI Spec. Publ.* 331 (2019) 101–121.
- [27] S.M. Al-Obaidi, Y.M. Saeed, F.N. Rad, Pullout tests on near-surface-mounted CFRP rods with and without lateral grooves, *Int. J. Eng. Technol.* 7 (2018) 72–78, <https://doi.org/10.14419/ijet.v7i4.20.25855>.
- [28] ASTM A615/A615M - 04, Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement, 2004. West Conshohocken, PA.
- [29] I.A. Sharaky, L. Torres, M. Baena, I. Vilanova, Effect of different material and construction details on the bond behaviour of NSM FRP bars in concrete, *Construct. Build. Mater.* 38 (2013) 890–902, <https://doi.org/10.1016/j.conbuildmat.2012.09.015>.
- [30] L. De Lorenzis, A. Rizzo, A. La Tegola, A modified pull-out test for bond of near-surface mounted FRP rods in concrete, *Compos. B Eng.* 33 (2002) 589–603, [https://doi.org/10.1016/S1359-8368\(02\)00052-5](https://doi.org/10.1016/S1359-8368(02)00052-5).
- [31] S.M. Soliman, E. El-Salakawy, B. Benmokrane, Bond performance of near-surface-mounted FRP bars, *J. Compos. Construct.* 15 (2011) 103–111, <https://doi.org/10.1061/ASCECC.1943-5614.0000150>.
- [32] A. Kreit, F. Al-Mahmoud, A. Castel, R. François, Repairing corroded RC beam with Near-Surface Mounted CFRP rods, *Mater. Struct. Constr.* 44 (2011) 1205–1217, <https://doi.org/10.1617/s11527-010-9693-6>.
- [33] L. De Lorenzis, A. Rizzo, A. La Tegola, A modified pull-out test for bond of near-surface mounted FRP rods in concrete, *Compos. B Eng.* 33 (2002) 589–603, [https://doi.org/10.1016/S1359-8368\(02\)00052-5](https://doi.org/10.1016/S1359-8368(02)00052-5).
- [34] D.G. Novidis, S.J. Pantazopoulou, Bond tests of short NSM-FRP and steel bar anchorages, *J. Compos. Construct.* 12 (2008) 323–333, <https://doi.org/10.1061/ASCE1090-02682008123323>.
- [35] ACI Committee 440 (ACI 440.2R-17), Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, 2017. Farmington Hills, MI.
- [36] T. Hassan, S. Rizkalla, Investigation of bond in concrete structures strengthened with near surface mounted carbon fiber reinforced polymer strips, *J. Compos. Construct.* 7 (2003) 248–257, [https://doi.org/10.1061/\(ASCE\)1090-0268\(2003\)7:3\(248\)](https://doi.org/10.1061/(ASCE)1090-0268(2003)7:3(248)).
- [37] J.A.O. Barros, S.J.E. Dias, J.L.T. Lima, Efficacy of CFRP-based techniques for the flexural and shear strengthening of concrete beams, *Cement Concr. Compos.* 29 (2007) 203–217, <https://doi.org/10.1016/j.cemconcomp.2006.09.001>.
- [38] M. Hosen, M. Jumaat, U. Alengaram, A. Islam, H. bin Hashim, Near surface mounted composites for flexural strengthening of reinforced concrete beams, *Polymers (Basel)* 8 (2016) 67, <https://doi.org/10.3390/polym8030067>.
- [39] I.A. Sharaky, L. Torres, J. Comas, C. Barris, Flexural response of reinforced concrete (RC) beams strengthened with near surface mounted (NSM) fibre reinforced polymer (FRP) bars, *Compos. Struct.* 109 (2014) 8–22, <https://doi.org/10.1016/j.compstruct.2013.10.051>.
- [40] ACI Committee 318 (ACI 318-14), Building Code Requirements for Structural Concrete and Commentary, 2014. Farmington Hills, MI.
- [41] M. Abdallah, F. Al Mahmoud, R. Boissière, A. Khelil, J. Mercier, Experimental study on strengthening of RC beams with Side Near Surface Mounted technique-CFRP bars, *Compos. Struct.* 234 (2020) 111716, <https://doi.org/10.1016/j.compstruct.2019.111716>.
- [42] I.A. Sharaky, S.A.I. Selmy, M.M. El-Attar, H.E.M. Sallam, The influence of interaction between NSM and internal reinforcements on the structural behavior of upgrading RC beams, *Compos. Struct.* 234 (2020) 111751, <https://doi.org/10.1016/j.compstruct.2019.111751>.
- [43] D.C. Kent, R. Park, Flexural members with confined concrete, *J. Struct. Div.* 97 (1971) 1969–1990.
- [44] I.A. Sharaky, L. Torres, H.E.M. Sallam, Experimental and analytical investigation into the flexural performance of RC beams with partially and fully bonded NSM FRP bars/strips, *Compos. Struct.* 122 (2015) 113–126, <https://doi.org/10.1016/j.compstruct.2014.11.057>.