Near-Surface-Mounted Fiber-Reinforced Polymer Reinforcements for Flexural Strengthening of Concrete Structures

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The use of fiber-reinforced polymer (FRP) materials for strengthening bridges and buildings has been used extensively in the last decade. FRP has been used in different configurations and techniques to use the material effectively and to ensure long service life of the selected system. One of these innovative strengthening techniques is the near-surface mounted (NSM) that consists of placing FRP reinforcing bars or strips into grooves precut into the concrete cover in the tension region of the strengthened concrete member. This method is relatively simple and considerably enhances the bond of the mounted FRP reinforcements, thereby using the material more effectively. This paper presents test results of reinforced concrete T-beams strengthened in flexure with different strengthening systems using FRP reinforcing bars and strips as NSM reinforcement and externally bonded FRP strips. The FRP reinforcements used in this investigation include carbon fiber-reinforced polymer (CFRP) reinforcing bars and strips and glass fiber-reinforced polymer (GFRP) thermoplastic strips. The behavior and effectiveness of the materials used for the various strengthening systems are compared. The structural performance and modes of failure of the tested beams are presented and discussed. Test results indicated that using NSM FRP reinforcing bars and strips is practical, significantly improves the stiffness, and increases the flexural capacity of reinforced concrete beams. The limitations of using NSM FRP reinforcing bars and strips are controlled by serviceability requirements in terms of overall deflections and crack widths rather than delamination, observed by many researchers, of externally bonded FRP reinforcement. Strengthening of reinforced concrete beams using NSM FRP strips provided higher strength capacity than externally bonded FRP strips using the same material with the same axial stiffness.

Keywords: bar; beam; carbon; concrete; fiber-reinforced polymer; glass; groove; strength; thermoplastic.

INTRODUCTION

Recently, fiber-reinforced polymer (FRP) reinforcements have been used extensively as an alternative reinforcement material to steel for new construction as well as for strengthening and repair of existing concrete structures. Externally bonded FRP sheets and strips are currently the most commonly used techniques for flexural and shear strengthening of concrete beams and slabs. Several researchers reported that the failure of members strengthened with externally bonded FRP sheets and strips could be brittle due to debonding and/or peeling of the FRP reinforcements, especially in the zones of high flexural and shear stresses (El-Hacha, Wight, and Green 2000). Externally bonded FRP reinforcements could be highly susceptible to damage from collision, fire and temperature, ultraviolet rays, and moisture absorption (ACI Committee 440 1996). In some cases, insufficient protection may reduce the service life of the structure. To minimize these problems, and to improve utilization of the FRP materials, near-surface-mounted (NSM) reinforcement was recently introduced as a promising technique for strengthening masonry walls and reinforced concrete members. Design guidelines for this technique are currently under consideration by ACI Committee 440 for the coming version of the "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (ACI 440.2R-02)." The NSM reinforcement technique consists of placing the FRP reinforcing bars or strips into grooves precut into the concrete cover in the tension region of the reinforced concrete member and bonded to the three sides of the groove using high-strength epoxy adhesive or cementitious grout.

The application of NSM FRP reinforcement does not require surface preparation work as in the case of externally bonded FRP reinforcement. In addition, the NSM FRP strengthening technique is also very efficient and practical for flexural strengthening of slabs and beams in the negative moment regions. Use of externally bonded FRP reinforcement in such cases could be subjected to mechanical and environmental damage and would require extensive protective cover that could interfere with the presence of floor finishes. Configuration of the FRP reinforcements used for the NSM technique is controlled by the depth of the concrete cover. After installation, the NSM FRP reinforcements are protected against mechanical damage, wear, impact, and vandalism from vehicles. The technique could also provide better fire resistance in the event of a fire; therefore, it could reduce the cost of fire protection measures.

RESEARCH SIGNIFICANCE

The use of NSM FRP reinforcement is currently emerging as a promising strengthening technique and a valid alternative to externally bonded FRP reinforcement for increasing the flexural strength of reinforced concrete members. The structural performance of reinforced concrete beams strengthened in flexure with NSM FRP reinforcement was examined and compared with beams strengthened with externally bonded FRP reinforcement. The behavior prior to and after cracking, ultimate carrying capacity, and modes of failure of all tested beams are discussed in this paper. The variables investigated were the type of fibers, including carbon fiber-reinforced polymer (CFRP) and glass fiber-reinforced polymer (GFRP) thermoplastic, and the configuration of the FRP reinforcement, including reinforcing bars and strips. The effectiveness of NSM FRP reinforcing bars and strips was examined and...
compared with externally bonded FRP strips using the same material and axial stiffness. The findings of this research provide data for the design guidelines currently under consideration by ACI Committee 440 for the NSM FRP strengthening technique.

BACKGROUND

Published literature on the use of NSM FRP for structural strengthening is very limited when compared with that of externally bonded FRP laminates. Even though the use of NSM FRP reinforcement for strengthening is relatively recent, NSM steel bars were used in Europe for strengthening of reinforced concrete structures. Asplund (1949) carried out tests on concrete beams strengthened with NSM steel bars grouted into diamond-sawed grooves filled with cement mortar and compared their behavior with that of conventional concrete beams reinforced with steel bars. Identical behavior for both sets of specimens was observed. The same technique was used in strengthening a reinforced concrete bridge deck in Sweden that experienced excessive settlement of the negative moment reinforcement during construction, so that the negative moment capacity needed to be increased (Asplund 1949). The advantage of using FRP instead of steel is primarily due to its corrosion resistance, which is particularly important in this case due to the location of the reinforcing bars or strips being very close to the surface that could be exposed to aggressive environmental attacks (De Lorenzis and Nanni 2002). Alkhadrji et al. (1999) conducted in-place tests on reinforced concrete bridge decks strengthened with NSM sandblasted CFRP rods. Test results showed an increase in the moment capacity of 27% compared with the unreinforced deck.

Research in Germany indicated that the bond characteristics of NSM CFRP strips are superior to externally bonded CFRP strips (Blaschko and Zilch 1999). NSM CFRP sandblasted rods and deformed GFRP rods increased the flexural strength of simply supported reinforced concrete T-beams by 30 and 26%, respectively (De Lorenzis, Nanni and La Tegola 2000). Increasing the amount of NSM reinforcement did not produce significant gain in the capacity. Arduni, Gottardo, and DeRiva (2001) found that the use of high-strength mortar with compensated shrinkage or epoxy putty guarantee full use of the NSM FRP strengthening system. The ultimate load-carrying capacity of beams strengthened with rectangular NSM CFRP rods using epoxy adhesive and cement grout as bonding agent has increased by 77 and 56%, respectively (Tiljsten and Carolin 2001). Using high-strength rectangular NSM FRP rods and high modulus rectangular NSM FRP rods increased the ultimate load capacity by 108 and 93%, respectively (Carolin, Hordin, and Tiljsten 2001).

The performance of various NSM FRP reinforcing bars and strips, as well as externally bonded FRP sheets on small-scale concrete beams and slabs, was investigated by Hassan (2002), including cost analysis for each of the FRP-strengthening techniques. Test results showed that using NSM CFRP reinforcing bars increased the strength by 36%. Using NSM CFRP strips increased the strength by 43% in comparison with an increase of only 11% using the axial stiffness used as externally bonded strips due to peeling failure of the strips. Hassan (2002) reported that the efficiency of using FRP reinforcing bars as NSM reinforcement is controlled by the bond characteristics of the reinforcing bars in addition to the bond between the epoxy adhesive material and the surrounding concrete in the groove. Such behavior has been confirmed and reported recently by other researchers (De Lorenzis and Nanni 2002). The maximum tensile strain in the CFRP and GFRP bars used as NSM reinforcement did not exceed 33 and 60% of the rupture strain of the bars at failure, respectively (De Lorenzis and Nanni 2002). Hassan (2002) reported that such a limiting value is highly dependent on the configuration and the ratio of the steel reinforcement inside the concrete beam as well as on the stress level at the concrete-epoxy interface. The author found that the maximum measured tensile strain in the CFRP bars at failure is in the range of 40 to 45% of the rupture strain of reinforcing bars, and that the rupture of CFRP reinforcing bars is not likely to occur regardless of the embedment or bond length or the type of epoxy adhesive used.

Recently, prestressed NSM CFRP rectangular rods have been used as a bonded post-tensioned system to strengthen concrete beams (Nordin, Tiljsten, and Carolin 2001). The initial strain was approximately 0.002, which is about 12% of the ultimate strain. The beam strengthened with prestressed NSM CFRP rods showed a 100 and 37% increase in the cracking and yielding load, respectively, compared with the beam strengthened with nonprestressed NSM CFRP rods. The ultimate loads and failure modes were the same with or without prestress by rupture of the NSM FRP rods; however, beams prestressed with NSM CFRP rods had smaller deflections at failure.

EXPERIMENTAL PROGRAM

Test specimens and setup

A total of eight, simply supported, 2.7 m (9 ft) long, concrete T-beams were constructed and tested under a monotonically increasing concentrated load applied at midspan of the beam. The test setup of a T-beam specimen is shown in Fig. 1. The load was applied using a closed-loop controller-testing machine operating using stroke-control mode at a loading rate of 1.07 mm/min (0.042 in./min). One beam was tested as a control specimen whereas the other seven beams were strengthened using different FRP reinforcements including CFRP reinforcing bars and strips as well as GFRP thermoplastic strips.

The bottom tension reinforcement consisted of two No. 13 deformed steel bars of nominal diameter 12.7 mm (1/2 in.) running along the full length of the beams and two No. 16 deformed steel bars of nominal diameter 15.9 mm (5/8 in.) terminated with a 90-degree bent at 100 mm (4 in.) away from the midspan section on both sides, as shown in Fig. 1. This arrangement of the bottom reinforcement was selected to ensure that the flexural failure of the strengthened beam will always occur at the midspan section and to simulate field conditions where the bottom steel reinforcement is
corroded or damaged. The top compression reinforcement consisted of two No. 13 deformed steel bars of nominal diameter 12.7 mm (1/2 in.). The beams were designed to avoid compression failure due to concrete crushing and shear failure before failure of the strengthening system. Shear reinforcement consisted of double-legged steel stirrups No. 13 deformed steel bar of nominal diameter 12.7 mm (1/2 in.) uniformly spaced at 100 mm (4 in.) center to center. The top flange was reinforced with a welded wire fabric mesh, 51 x 51 MW5.6 x MW5.6, with a nominal diameter of 2.67 mm, a wire spacing of 51 mm center to center in both the longitudinal and transverse direction, and a reinforcement area of 110 mm²/m. Reinforcement details of a T-beam specimen are shown in Fig. 1. The specified yield strength and modulus of elasticity of the tension and compression reinforcements were 400 MPa (58 ksi) and 200 GPa (29,000 ksi), respectively. The concrete was designed for a nominal compressive strength of 45 MPa (6.5 ksi) at 28 days.

Test matrix

One beam was tested without strengthening (B0) and served as the control specimen for comparison purposes. Four beams (B1, B2, B3, and B4) were strengthened with different NSM FRP systems using CFRP reinforcing bars, two types of CFRP strips, and thermoplastic GFRP strips. Three beams (B2a, B2b, and B4a) were strengthened with different externally bonded CFRP and GFRP strips.

Beam B1 was strengthened with one 9.5 mm (3/8 in.) diameter NSM CFRP reinforcing bar (Hughes Brothers 2002). Beam B2 was strengthened with two Type 1 2 x 16 mm (0.078 x 0.63 in.) NSM CFRP strips (Hughes Brothers 2002). Beam B3 was strengthened with two Type 2 1.2 x 25 mm (0.05 x 1.0 in.) NSM CFRP strips (Structural Composites, Inc. 2002). Beam B4 was strengthened with five 2 x 20 mm (0.078 x 0.78 in.) NSM GFRP thermoplastic strips (Dow Plastics Chemical 2000).

Beams B2a and B2b were each strengthened with two Type 1 2 x 16 mm (0.078 x 0.63 in.) externally bonded CFRP strips (Hughes Brothers 2002). Beam B2b was severely damaged before strengthening. Beam B4a was strengthened with five 2 x 20 mm (0.078 x 0.78 in.) externally bonded GFRP thermoplastic strips (Dow Plastics Chemical 2000).

The embedment length of all the NSM FRP reinforcing bars and strips and the length of the externally bonded FRP strips were kept constant in all beams as 2400 mm (7 ft, 10-1/2 in.). The same axial stiffness \((EA)_{FRP}\) for all FRP reinforcement used in this study was kept constant, hence, according to the classical beam theory, the load-deflection behavior of all strengthened beams is anticipated to be identical, where \(E\) and \(A\) are the modulus of elasticity and cross-sectional area of the FRP reinforcement, respectively. A summary of these beams is given in Table 1.

Instrumentation

The beams were instrumented, as shown in Fig. 1, to measure applied load, deflection, and strain in the concrete and in the FRP reinforcement during testing. The concrete strains in the compression zone at the top surface of the beams were measured using two displacement transducers (DT) placed at 100 mm (4 in.) from the midspan on both sides of the beam. In addition, three DT gages were installed in the midspan zone along the front face on one side of the beam centerline to measure the strain distribution in concrete over the depth of the beam. The DT gages were placed at 65, 150, and 275 mm (2.6, 5.9, and 10.8 in.) from the bottom surface of the concrete beam. The strains in the NSM FRP reinforcing bars and strips and the externally bonded FRP strips at three different locations were monitored during testing using three electrical resistance 120 ohms strain gages. The strain gages were installed on all FRP reinforcing bars or strips at midspan, at 400 and 800 mm (15.75 and 31.5 in.) from the midspan on one side of the FRP reinforcing bars or strips. The deflections at midspan, at the supports, and at 400 and 800 mm from midspan on both sides of the beam were measured using linear variable displacement transducers (LVDTs). One LVDT was placed at each location except at midspan where two LVDTs were used and averaged. The slip at the free ends of the NSM FRP reinforcing bars and strips was measured using two LVDTs. The data were automatically collected. The location of cracks and their propagation was clearly marked on both sides of the beams. Crack widths were also measured at every 4.5 kN (1.0 kips).

Fiber-reinforced polymer strengthening systems

Four products, provided by three manufacturers (Manf.-1 to Manf.-3) were investigated. Product-1 = Aslan 200 CFRP reinforcing bars (Manf.-1 = Hughes Brothers 2002); Product-2 = Aslan 500 CFRP strips (Manf.-1 = Hughes Brothers 2002); Product-3 = unidirectional pultruded laminate

![Diagram](image-url)
the FRP reinforcing bars or strips. Each groove was filled with concrete and consisted of a two-component adhesive with a modulus of elasticity of 1200 MPa (174 ksi), an ultimate tensile strength of 48 MPa (6.96 ksi), a compressive yield strength of 71.7 MPa (10.4 ksi), and a compressive modulus of 3378 MPa (490 ksi). Another type of high-strength epoxy adhesive was used to bond the CFRP and GFRP strips to the concrete and consisted of a two-component adhesive with a mixture ratio of 2:1 by volume (Structural Components, Inc. 2002). As reported by the manufacturer, the adhesive has a modulus of elasticity of 3500 MPa (507.6 ksi), an ultimate tensile strength of 70 MPa (10.2 ksi), and a compressive strength of 82.7 MPa (12 ksi).

Installation of NSM FRP reinforcements

Installation of the NSM FRP reinforcing bars and strips begins by making a series of grooves with specified dimensions cut into the concrete cover in the longitudinal direction at the tension side of the beam specimens. A special concrete saw with a diamond blade was used to cut the grooves with the dimensions shown in Fig. 2. The grooves were cleaned from any dust and air-brushing pressure was used to remove debris and fine particles to ensure proper bonding between the epoxy adhesive and the concrete.

The adhesive was applied into the groove before inserting the FRP reinforcing bars or strips. Each groove was filled completely with the appropriate epoxy adhesive paste using a manual epoxy gun to provide the necessary bond with the surrounding concrete. Then the FRP reinforcements (reinforcing bars and strips) were inserted inside the grooves ensuring that they were completely covered with epoxy and lightly pressed to displace the bonding agent. This action forces the epoxy paste to flow around the FRP reinforcement and completely fill the space between the FRP and the sides of the groove. The groove was then filled with more paste if needed and the surface was leveled. The excess adhesive was removed with a spatula. The surface was smooth finished to achieve uniform distribution. The same procedures in terms of cutting the groove, injecting the epoxy, and placing the FRP reinforcing bars and strips were applied. The epoxy adhesive was allowed to fully cure at room temperature for at least 1 week before testing of the beams.

Dimensions of grooves

The grooves cut at the bottom surface of the concrete beams had different cross sections depending on the type of FRP reinforcements used as shown in Fig. 2. For the 9.5 mm (3/8 in.) NSM CFRP reinforcing bar, the groove was approximately 18 mm (0.708 in.) wide and 30 mm (1.18 in.) deep cut at the middle of the bottom width of the beam specimen. For the Type 1 2 x 16 mm (0.078 x 0.63 in.) NSM CFRP strip, two grooves, 75 mm (3.0 in.) apart, were cut at the bottom width of the beam specimen; each groove was approximately 6.4 mm (0.25 in.) wide and 19 mm (3/4 in.) deep. For the Type 2 1.2 x 25 mm (0.05 x 1.0 in.) NSM CFRP strip, two grooves, 75 mm (3.0 in.) apart, were cut at the bottom width of the beam specimen; each groove was approximately 6.4 mm (0.25 in.) wide and 25 mm (1.0 in.) deep. For the 2 x 20 mm (0.078 x 0.78 in.) NSM GFRP strip, three grooves, 38 mm (1.5 in.) apart, were cut at the bottom width of the beam specimen; each groove was approximately 6.4 mm (0.25 in.) wide and 25 mm (1.0 in.) deep. One GFRP strip was inserted into the middle groove and two strips bonded together side by side were placed in each of the outer two grooves.

Installation of externally bonded FRP reinforcements

To ensure a good, strong bond, the bottom surface of the concrete beams was prepared by grinding until the coarse aggregates were exposed, then cleaned by washing and air-brushing to remove dust or debris and fine particles. Care was taken to ensure that the resulting concrete surface after grinding was uniform. Following cleaning, a uniform 2 mm (0.078 in.) thin layer of the two-part epoxy-based adhesive was applied by palette knife to the bottom surface of the concrete beam. The FRP strips were placed in position on the concrete surface and pressed onto the epoxy by hand. To ensure a good bond with concrete, a uniform pressure was applied along the entire length of the strips. A U-shaped wrap CFRP sheet with 100 mm (4.0 in.) width was placed...
around the web of the concrete beams at both ends of the externally bonded FRP flexural reinforcements, with the direction of the fibers perpendicular to the longitudinal axis of the member, to improve the anchorage of the FRP strengthening system (Fig. 3). The externally bonded CFRP strips (Type 1) were placed at the bottom surface of the concrete beams (B2a and B2b) with spacing in between equal to twice the width of the strip (Fig. 3). The externally bonded GFRP strips were placed side by side at the bottom surface of the concrete beam (B4a) leaving 25 mm (1.0 in.) from each side of the beam (Fig. 3).

**TEST RESULTS AND DISCUSSION**

A summary of significant test results describing the flexural behavior of all tested beams is presented in Table 3. The concrete compressive strength when the beams were tested was determined according to ASTM C 39-01, using three standard concrete cylinders and ranged between 48 MPa (6962 psi) for Beams B0, B1, B3, and B4, and 57 MPa (8267 psi) for Beams B2, B2a, B2b, and B4a. Beam B0 was tested without strengthening and used as a control specimen for comparison purposes to evaluate the improvement in flexural strength provided by the various NSM and externally bonded FRP reinforcements. The unstrengthened control beam failed by crushing of the concrete after yielding of the steel tension reinforcement.

**Effectiveness of NSM CFRP reinforcing bars and strips**

The load-midspan deflection behavior of the strengthened beams with NSM CFRP reinforcing bars (Beam B1) and strips (Beams B2 and B3) in comparison with the unstrengthened control beam (B0) is shown in Fig. 4. Prior to cracking, the load-deflection behavior for all strengthened beams was similar to that of the unstrengthened beam. This behavior indicates that using NSM FRP reinforcements did not contribute to increasing the stiffness and strength in the elastic range. After cracking, however, the flexural stiffness and strength of the strengthened beams with NSM FRP reinforcements were significantly improved compared with the

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**Table 3—Summary of significant test results**

<table>
<thead>
<tr>
<th>Strengthening systems</th>
<th>Beam no.</th>
<th>$P_{cr}$, kN (kips)</th>
<th>$\Delta_{u}$, mm (in.)</th>
<th>$P_{y}$, kN (kips)</th>
<th>$\Delta_{u}$, mm (in.)</th>
<th>$P_{u}$, kN (kips)</th>
<th>$\Delta_{u}$, mm (in.)</th>
<th>$e_{tu}$, %</th>
<th>Failure mode</th>
<th>Percent increase in $P_{u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near-surface-mounted FRP reinforcement</td>
<td>B0</td>
<td>21.98 (4.94)</td>
<td>1.35 (0.053)</td>
<td>38.11 (8.57)</td>
<td>8.88 (0.35)</td>
<td>55.4 (12.5)</td>
<td>64.4 (2.54)</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>B1</td>
<td>24.7 (5.55)</td>
<td>1.27 (0.05)</td>
<td>47.94 (10.78)</td>
<td>4.85 (0.191)</td>
<td>93.8 (21.0)</td>
<td>29.2 (1.15)</td>
<td>0.88</td>
<td>Debonding of NSM CFRP reinforcing bar (epoxy split failure)</td>
<td>69.3</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>22.24 (5.9)</td>
<td>1.08 (0.043)</td>
<td>48.62 (10.93)</td>
<td>5.61 (0.221)</td>
<td>99.3 (22.3)</td>
<td>30.5 (1.20)</td>
<td>1.34</td>
<td>Rupture of NSM CFRP strips</td>
<td>79.2</td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td>30.11 (6.77)</td>
<td>1.702 (0.067)</td>
<td>49.16 (11.05)</td>
<td>5.25 (0.207)</td>
<td>110.2 (24.7)</td>
<td>50.8 (2.00)</td>
<td>1.38</td>
<td>Rupture of NSM CFRP strips</td>
<td>98.9</td>
</tr>
<tr>
<td></td>
<td>B4</td>
<td>24.46 (5.50)</td>
<td>1.6 (0.063)</td>
<td>48.17 (10.83)</td>
<td>5.67 (0.223)</td>
<td>102.7 (23.1)</td>
<td>44.3 (1.75)</td>
<td>1.35</td>
<td>Debonding of NSM CFRP strips (concrete split failure)</td>
<td>85.4</td>
</tr>
<tr>
<td>Externally bonded FRP reinforcement</td>
<td>B2a</td>
<td>22.46 (5.05)</td>
<td>1.22 (0.048)</td>
<td>44.88 (10.09)</td>
<td>4.42 (0.174)</td>
<td>64.6 (14.5)</td>
<td>43.7 (1.71)</td>
<td>0.48</td>
<td>Debonding of externally bonded CFRP strips</td>
<td>16.6</td>
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<tr>
<td></td>
<td>B2b</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>64.3 (14.5)</td>
<td>21.7 (0.85)</td>
<td>0.44</td>
<td>Debonding of externally bonded CFRP strips</td>
<td>16.1</td>
</tr>
<tr>
<td></td>
<td>B4a</td>
<td>29.13 (6.55)</td>
<td>0.95 (0.037)</td>
<td>48.16 (10.82)</td>
<td>4.39 (0.173)</td>
<td>71.1 (15.9)</td>
<td>22.2 (0.87)</td>
<td>0.62</td>
<td>Debonding of externally bonded GFRP strips</td>
<td>28.3</td>
</tr>
</tbody>
</table>

*Where $P_{cr}$ = cracking load; $\Delta_{u}$ = midspan deflection at cracking; $P_{y}$ = yield load; $\Delta_{u}$ = midspan deflection at yielding; $P_{u}$ = ultimate load failure; $\Delta_{u}$ = midspan deflection at failure; and $e_{tu}$ = maximum tensile strain in FRP reinforcing bar or strip at failure.*

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**Fig. 3—Externally bonded FRP strengthening systems.**

**Fig. 4—Load-midspan deflection of beams strengthened with NSM CFRP reinforcing bar and strips.**
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Fig. 5—Debonding failure of Beam B1 strengthened with NSM CFRP reinforcing bar.

Fig. 6—Rupture failure of Beam B2 strengthened with two NSM CFRP strips Type 1.

unstrengthened beam. After cracking, a nonlinear behavior was observed up to failure.

In general, the behavior of the strengthened beams indicated a significant increase in the stiffness and strength in comparison with the unstrengthened beam. Using the same axial stiffness of CFRP reinforcement, an increase in the ultimate strength of 69, 79, and 99% were measured for Beams B1, B2, and B3, respectively. The significant increase in ultimate load-carrying capacity of Beam B3 strengthened with NSM CFRP strips (Type 2) compared with Beam B2 strengthened with NSM CFRP strips (Type 1) is due to the high ultimate tensile strength of the material used in this case as well as to the thinner Type 2 versus Type 1 strips, which reduce the risk of delamination. Beam B1 with NSM CFRP reinforcing bars showed a smaller increase in strength due to early debonding of the CFRP reinforcing bars at failure and the smaller bonding surface of the NSM CFRP reinforcing bars with respect to the NSM CFRP strips (Types 1 and 2).

Using NSM FRP reinforcement resulted in a significant reduction of the deflection and crack widths and delayed formation of new cracks in the strengthened beams. The formation of cracks followed a typical crack pattern of flexural members. The first flexural crack occurred at the midspan of the beam directly under the location of the applied load. Under further increase in the load, the cracks became wider and new flexural cracks started to initiate. Many uniformly distributed cracks of small widths were observed along the full length of the strengthened beams that were symmetric about their midspan, whereas fewer cracks of greater width were observed in the unstrengthened control beam. This behavior indicates that complete bond of the FRP materials, even after yielding of the steel reinforcement, controlled crack widths and their distribution along the span. As the load approached failure stage, the flexural stiffness decreased further until the ultimate load was reached and failure occurred.

Failure of Beam B1 was due to splitting of the epoxy cover in the groove followed by complete debonding of the reinforcing bar at the CFRP-epoxy interface and cracking of the concrete surrounding the epoxy as shown in Fig. 5. This type of failure is categorized as epoxy split failure. Initiation of the crack in the epoxy was accompanied by a distinct noise followed by progressive cracking of the epoxy paste. Longitudinal splitting cracks, which developed in the epoxy cover, led to the loss of bond of the NSM CFRP reinforcing bars. After debonding, the load dropped to a load level equivalent to the measured yielding load and the deflection kept on increasing until failure occurred due to crushing of the concrete in the compression zone—then the test was stopped as shown in Fig. 4. The debonding initiated at the concrete section where 39% of the bottom flexural reinforcement was terminated (Fig. 1). Splitting of the epoxy is the result of high tensile stresses at the CFRP reinforcing bar-epoxy interface. It has been reported that to reduce the induced tensile stresses at the FRP-epoxy interface, the thickness of the epoxy cover must be increased and/or high tensile adhesive must be used to shift the location of failure to the concrete-epoxy interface (Hassan 2002). Increasing the thickness of the adhesive will reduce the shear deformation within the adhesive layer and therefore results in a significant increase in debonding loads. De Lorenzis and Nanni (2002) reported that increasing the groove size and the cover thickness leads to higher bond strength when failure is controlled by splitting of the epoxy cover.

The failure of Beams B2 and B3 was due to rupture of the NSM CFRP strips at midspan as shown in Fig. 6 and 7, respectively. After rupture of the NSM CFRP strips, the load dropped to a load level equivalent to the yielding load of the cross section and the beam behaved as conventional concrete beams reinforced with steel bars as shown in Fig. 4.

The load-tensile strain behavior of the NSM CFRP reinforcing bars and strips is linear up to cracking of the concrete as shown in Fig. 8. At the onset of cracking, a significant increase in the measured tensile strain was observed for all tested beams measured by the strain gage attached to the NSM CFRP reinforcing bars or strips. At failure, the maximum measured tensile strain in the NSM CFRP reinforcing bars prior to debonding was 0.88%, which is 77% of the rupture strain of the CFRP reinforcing bar.
maximum measured tensile strain in Types 1 and 2 CFRP strips at failure for Beams B2 and B3 were 1.34 and 1.38%, respectively, as shown in Fig. 8, indicating full use of the tensile strength of the two types of CFRP strips used.

**NSM versus externally bonded CFRP strips**

Beam B2a, strengthened with externally bonded CFRP strips (Type 1) exhibited similar behavior to that of the unstrengthened control beam up to cracking. This behavior indicates that using externally bonded FRP reinforcements did not contribute significantly to the stiffness and strength in the elastic range. After cracking, the load-deflection response of the beam strengthened with externally bonded CFRP strips followed the same behavior of beams strengthened with NSM CFRP strips up to yielding of the flexural reinforcement as shown in Fig. 9. After yielding of the internal steel reinforcement, and under further increase of the applied load, the cracks continued to widen and failure occurred due to debonding of the externally bonded strips as shown in Fig. 10.

Using the same axial stiffness \( (EA)_{FRP} \) of the CFRP strips used as NSM for Beam B2, externally bonded CFRP strips increased the strength by only 16.6% compared with the unstrengthened beam due to debonding failure of the externally bonded strips from the concrete surface. However, as shown in Fig. 9, the NSM CFRP strips (Type 1) increased the strength by 79%. Therefore, the strength increase using the same CFRP strips as NSM was approximately 4.8 times that obtained using externally bonded strips. Another beam, B3a, strengthened with two externally bonded CFRP strips (Type 2), was tested and achieved a 25% increase in strength and failed by debonding of the CFRP strips from the concrete at maximum measured tensile strain in the strips of 0.42% (El-Hacha et al. 2004). Thus, the NSM strengthening technique using CFRP strips is more effective than the externally bonded one.

The load-tensile strain behavior of the CFRP strips was similar for Beams B2 and B2a until debonding occurred for the externally bonded CFRP strips as shown in Fig. 11. At the onset of delamination, the maximum measured tensile strain...
The failure of Beam B2b was similar to Beam B2a due to stiffness and increasing the load-carrying capacity of the severely damaged beam and at the same time to approach the ultimate capacity of undamaged, strengthened Beam B2a. The behavior of retrofitted Beam B2b was similar to that of undamaged, strengthened Beam B2a. The externally bonded CFRP strips were capable of restoring the original stiffness and increasing the load-carrying capacity of the severely damaged beam and at the same time to approach the ultimate capacity of undamaged, strengthened Beam B2a. The failure of Beam B2b was similar to Beam B2a due to debonding of the CFRP strips from the concrete substrate. The maximum measured tensile strain in the CFRP strips was 0.44%, as shown in Fig. 11, indicating that only 39% of the rupture strain reported for the CFRP strips was used. This measured strain is 83% of the strain limitations recommended by ACI 440.2R (2002) to prevent debonding of the CFRP strips at the ultimate-limit state.

**Effect of material type of fiber**

Beam B4, strengthened with GFRP thermoplastic strips as NSF reinforcement, exhibited significant enhancement in strength and stiffness in comparison with the unstrengthened beam as shown in Fig. 12. An increase in the ultimate strength of 85% was observed. The failure of Beam B4, due to cracking of the concrete surrounding the epoxy in the groove, occurred at the concrete-epoxy interface known as "concrete split failure" as shown in Fig. 13. Debonding started to occur at the location where 39% of the bottom steel reinforcement was terminated as shown in Fig. 1. This failure is the result of the high shear stress concentration in this zone as discussed previously. Debonding of the concrete and the split failure occurred when the tensile stresses at the concrete-epoxy adhesive interface reached the tensile strength of concrete. This failure mode is greatly influenced by the groove dimensions as well as the mechanical characteristics of the materials (De Lorenzis and Nanni 2002; Hassan 2002). Deboning extended as a horizontal splitting crack along the concrete cover toward the ends of the beam. Under further increase of the applied load, the horizontal split crack became wider and extended into the end of the NSF GFRP strips causing severe cracking in the concrete cover. At the onset of failure, the load dropped to the yielding load level of the beam cross-section and the deflection kept on increasing until failure occurred due to crushing of the concrete in the compression zone. After complete failure, the concrete cover to the internal steel separated from the steel. It was observed that the NSF GFRP strips had adhered well to the concrete of the strengthened beam. The amount of concrete adhered to the NSF strips varied considerably. In examining the concrete surface of the failed member, aggregate pullout was noted without any sign of damage of the NSF GFRP strips.
It should be noted that using the same axial stiffness for strengthening Beam B4 with five NSM GFRP thermoplastic strips exhibited similar load-deflection behavior as Beam B2 strengthened with two Type 1 NSM CFRP strips up to failure of Beam B2 as shown in Fig. 12.

The load-tensile strain behavior of the NSM CFRP strips (Types 1 and 2) was similar to the NSM GFRP thermoplastic strips as shown in Fig. 14. As presented previously, failure of the NSM CFRP strips (Types 1 and 2) occurred by rupture of the strips when the strain in the strips reached the ultimate tensile strain capacity reported by the manufacturers (1.12% for Type 1 CFRP and 1.34% for Type 2 CFRP). These ultimate strain capacities are significantly less than the 2.2% ultimate tensile strain capacity of the GFRP thermoplastic strips as reported by the manufacturer. Because the GFRP thermoplastic strips possess large ultimate strain capacity and because the thickness of the epoxy in the outer grooves was almost half that of the epoxy in the middle groove, failure was dominated by the higher shear stresses at the concrete-epoxy interface. Therefore, increasing the thickness of the epoxy (that is, increasing the groove size) will reduce the shear stresses at the concrete-epoxy interface and could result in an increase in debonding load. This has been confirmed by De Lorenzis and Nanni (2002) and Hassan (2002).

**NSM versus externally bonded GFRP strips**

The ultimate load-carrying capacity of Beam B4a strengthened using externally bonded thermoplastic GFRP strips increased by 28%. Beam B4a showed similar behavior compared with Beam B4 up to a load level of 66 kN at which debonding of the externally bonded GFRP strips occurred as shown in Fig. 15. Debonding of the externally bonded GFRP strips involved separation of the strips from the concrete substrate in the form of adhesive shear failure at the concrete-adhesive interface (interfacial failure) as shown in Fig. 16. At failure, the externally bonded GFRP strips slipped instantaneously from the end anchorages. A more ductile behavior was observed in Beam B4 compared with Beam B4a.

The maximum measured tensile strain at failure of the NSM GFRP strips and externally bonded GFRP strips were approximately 1.35 and 0.62%, respectively, as shown in Fig. 17. The measured strain is 61 and 28% of the rupture strain of the GFRP strips. At the onset of delamination of the externally bonded GFRP strips, the maximum measured tensile strain of the GFRP strips was 50% less than the maximum limit specified by ACI 440.2R (2002) to prevent debonding of the FRP at ultimate. The results indicate that the ACI 440.2R equation for strain limitation to avoid debonding should be reexamined.

**CONCLUSIONS**

The effectiveness of using near-surface-mounted FRP reinforcing bars or strips for strengthening concrete beams...
has been illustrated. The following observations and conclusions can be drawn from the experimental results:

1. Strengthening with NSM FRP reinforcing bars or strips improved the load deflection response of the reinforced concrete beams. The use of NSM FRP reinforcements enhanced the flexural stiffness and significantly increased the ultimate load-carrying capacity of the strengthened concrete beams. The difference in the behavior prior to cracking was insignificant. After cracking, the behavior of the strengthened beams significantly improved. The NSM FRP reinforcing bars or strips limited the deflections and crack widths. At any load level, the deflections of the strengthened beams were significantly less than that of the unstrengthened beam;

2. The ultimate strength of the strengthened beams with NSM CFRP strips was governed by the tensile rupture strength of the CFRP strips. A full composite action between the NSM CFRP strips and concrete was achieved;

3. FRP-epoxy-split failure was the dominant mode of failure for the beam strengthened with NSM CFRP reinforcing bars as a result of high tensile stresses at the CFRP reinforcing bar-epoxy interface;

4. Concrete split failure was the governing mode of failure for the beam strengthened with NSM GFRP thermoset strips;

5. Failure of beams strengthened with externally bonded CFRP or thermoset GFRP strips was due to debonding between the strips and the concrete. The debonding failure of the externally bonded FRP strips was brittle and occurred at a load level significantly lower than the ultimate load measured for beams strengthened with NSM CFRP reinforcing bars or strips and NSM GFRP thermoset strips;

6. Strengthening a concrete beam with NSM CFRP reinforcing bars provided a considerably less increase of the load-carrying capacity compared with similar beams strengthened with NSM CFRP strips with the same axial stiffness due to possible early debonding failure that occurred at the CFRP reinforcing bar-epoxy interface and to the smaller bonding surface of the NSM CFRP reinforcing bars with respect to the NSM CFRP strips;

7. No slip was observed for the different NSM FRP reinforcing bars and strips strengthening techniques up to ultimate load-carrying capacity;

8. The strength of the reinforced concrete beam strengthened with the NSM technique provided a significant increase of the overall ductility of the member when compared with externally bonded FRP strips; and

9. Using the same axial stiffness of FRP to strengthen reinforced concrete beams, the beams strengthened with NSM FRP reinforcement achieved higher ultimate load than beams strengthened with externally bonded FRP reinforcement. This is due to the high utilization of the tensile strength of the FRP reinforcement. The NSM FRP strips have double the bond area compared with an externally bonded FRP strips. It should be noted that the thickness could affect the debonding phenomena.

In summary, the NSM FRP strengthening technique could be considered as a valid alternative to an externally bonded FRP strengthening system and an attractive, efficient method for enhancing the stiffness and increasing the flexural strength of deficiently reinforced concrete members.

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