CHARACTERIZATION OF FRP RODS AS NEAR-SURFACE MOUNTED REINFORCEMENT

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ABSTRACT: The use of near surface mounted (NSM) fiber-reinforced polymer (FRP) rods is a promising technology for increasing flexural and shear strength of deficient reinforced concrete (RC) members. The advantages of NSM FRP rods compared with externally bonded FRP laminates are the possibility of anchoring the reinforcement into adjacent RC members and the minimal surface preparation work and installation time necessary (Nanni et al. 1999).

INTRODUCTION

The use of near surface mounted (NSM) fiber-reinforced polymer (FRP) rods is a promising technology for increasing flexural and shear strength of deficient reinforced concrete (RC) members. The advantages of NSM FRP rods compared with externally bonded FRP laminates are the possibility of anchoring the reinforcement into adjacent RC members and the minimal surface preparation work and installation time necessary (Nanni et al. 1999).

The method used in applying the rods is as follows: a groove is cut in the desired direction into the concrete surface, the groove is filled halfway with epoxy paste, then the FRP rod is placed in the groove and lightly pressed. This forces the paste to flow around the rod and fill completely between the rod and the sides of the groove. Finally, the groove is filled with more paste and the surface is leveled.

As this technology emerges, the structural behavior of RC elements strengthened with NSM FRP rods needs to be fully characterized. Given the variability of material properties and groove geometry, this requires that, preliminarily, the tensile properties of the FRP rods and the mechanics of load transfer between NSM FRP rods and concrete be investigated.

The FRP rods used in this study were commercially available carbon FRP (CFRP) deformed rods manufactured through the hybrid pultrusion process. Tensile and bond testing of the rods for application as NSM reinforcement were carried out. Tensile testing was conducted in accordance with a protocol that is a standard test method in Japan (JSCE 1997). The adopted bond test method was already used in previous research on the bond of FRP sheets to concrete (Miller 1999). Results of three tensile tests and five bond tests are presented. In addition, the paper presents results of experimental tests on the use of NSM FRP rods to increase the shear capacity of RC beams. Three full-size beams, one control beam and two beams strengthened with NSM FRP rods, were tested. Finally, test results are compared with the predictions of a simple design approach.

Even though limited in scope, the protocol presented herein (characterization at material, subsystem, and structural level and design approach) is proposed to be followed for other FRP material systems and applications of the NSM reinforcement technology. A wider investigation on NSM rods for structural strengthening has been carried out (De Lorenzis 2000). It involved tensile and bond characterization and flexural and shear strengthening of RC beams. Bond characterization and flexural testing were conducted using various types of FRP rods.

TENSILE TEST

Specimens

The mechanical properties of FRP rods are influenced by different factors, such as fiber type and content, resin type, and manufacturing method. An understanding of the tensile properties of the rods is necessary for their application as NSM reinforcement. Specimens of CFRP No. 3 deformed rods [nominal diameter 3/8 in. (9.5 mm)] (Marshall Industries Composites Inc. 1998) were tested to determine their tensile properties. Testing was conducted in accordance with a protocol that is a standard test method in Japan (JSCE 1997) and is currently under consideration to become a standard in North America (Benmokrane et al. 1998).

An essential requirement for conducting tensile tests of FRP rods is a suitable method to grip the specimens without causing slippage or premature local failure during the test (Yan et al. 1999). The conventional friction grips used for steel rebars are not suitable for tensile testing of FRP rods. The reason for this is that FRP rods are weak in the transverse direction; the transverse compressive strength is controlled by the resin properties and is usually less than 10% of the longitudinal tensile strength, which is controlled by the fibers. The stress concentration due to gripping can easily crush the specimen and result in premature failure. To prevent these problems, a grouted anchor consisting of a steel pipe filled with expansive grout was used (Dye et al. 1998). The internal pressure due to expansion of the grout prevents the rod from slipping out of the pipe when direct tension is applied. This type of anchorage device distributes the gripping force to a much larger area, thereby reducing the chance of premature failure. The dimensions of the pipes were: length = 18 in. (457 mm); outside diameter = 1.66 in. (42 mm); and thickness = 0.14 in. (3.5 mm). The grout was mixed according to the manufacturer’s instructions with a water-cement ratio by mass of 0.29 (Onoda Corporation 1999). The grouted specimens were allowed to cure for 72 hours before testing. The total length of the test specimens was 5 ft (1.52 m), which included test section and anchoring section. The length of the test section was larger than the minimum value suggested by JSCE (1997) and...
Benmokrane et al. (1998); that is, the greater of 4 in. (100 mm) and 40 times the nominal diameter of the FRP rod.

**Procedure**

An electric strain gauge with a gauge length of 1/2 in. (13 mm) was applied at the center of the test section in the direction of tensioning. The test was performed in displacement control mode with a loading rate of 5 kips (22 kN) per minute, corresponding to 45 ksi (310 MPa) per minute. This rate is within the range recommended by JSCE (1997) and Benmokrane et al. (1998), which is between 14.5 ksi (100 MPa) and 72.5 ksi (500 MPa) per minute.

**Results**

Tensile strength, modulus of elasticity, and ultimate strain were computed using load and strain data recorded during the test and the nominal cross-sectional area of the rod. The average values of tensile strength, elastic modulus, and ultimate strain were 272 ksi (1,875 MPa), 15,200 ksi (104.8 GPa), and 1.79%, with a standard deviation of 6.9 ksi (47.6 MPa), 700 ksi (4.8 GPa), and 0.1%, respectively. The modulus of elasticity was calculated from the points on the stress-strain curves at 20 and 60% of the tensile strength. The ultimate strain was computed by dividing the tensile strength by the calculated modulus of elasticity. All specimens showed linear-elastic behavior up to failure and tensile failure occurred away from the anchors. This failure mode indicated that the bar had developed its full tensile capacity, and the anchor was efficient.

**BOND TEST**

**Specimens**

Bond is of primary importance, since it is the means for the transfer of stress between the concrete and the FRP reinforcement in order to develop composite action. The bond behavior influences the ultimate capacity of the reinforced element as well as serviceability aspects such as crack width and crack spacing. Among the many different types of bond tests reported in the literature, the most common are the direct pull-out test and the beam pull-out test (Nanni et al. 1995). It is generally believed that beam pull-out tests are more representative of the bond behavior in real members.

A beam pull-out test was adopted for this project, based on previous work on bonds between CFRP sheets and concrete (Miller 1999). The specimens were unreinforced concrete beams with an inverted T-shaped cross section. This section was chosen to provide a larger tension area for concrete while minimizing the overall weight of the beam. A larger tension area for concrete was needed to increase the cracking load of the specimen. Furthermore, the position of the bonded length of the rod had to be chosen appropriately in order to prevent flexural cracking before bond failure. Further details are reported elsewhere (De Lorenzis 2000).

The dimensions of the beams are given in Fig. 1. Each beam had a steel hinge at the top and a saw cut at the bottom, both located at midspan. The purpose of the hinge and saw cut was to control the distribution of the internal forces. During loading, the saw cut caused a crack to develop at the center of the beam and extend up to the hinge. Therefore, the compressive force in the beam at midspan was located at the center of the hinge and the internal moment arm was known and constant for any given load level above the cracking load. This allowed an accurate computation of the tensile stress in the rod.

Each of the specimens had a longitudinal groove cut in the tension face, where the FRP rod had to be mounted. The grooves were created by saw-cutting two parallel slits at the desired distance and depth and chiseling off the material in between. Then, they were air blasted to remove the powdered concrete produced by the cutting process and other loose material. The epoxy paste was prepared by mixing the two components (resin and hardener) in 2:1 proportion by volume with a power mixer. The method of application of the rods has been described in the introduction.

Only one side of the beam was the test region, with the FRP rod having a limited bonded length and being unbonded in the remaining part. The rod was fully bonded on the other side of the beam, so that bond failure would occur in the test region.

Five specimens were tested, as indicated in Table 1. The following variables were examined in the experimental test matrix:

- Bonded length: Three different bonded lengths were used, equal to 6, 12, and 18 times the diameter of the rod.
- Size of the groove: For the 12-diameter bonded length, specimens with three different groove sizes were tested. The groove width was maintained equal to the groove depth in all the tested specimens.

Concrete strength and epoxy type were not varied, although they are significant parameters. Concrete with 4,000 psi (27.6 MPa) nominal compressive strength and a commercially available epoxy paste were selected as representative systems. The actual average concrete strength, determined according to ASTM C39-97 on three 4 in. (102 mm) diameter by 8 in. (203 mm) concrete cylinders, was 4,090 psi (28.2 MPa). The mechanical properties of the epoxy, as specified by the manufacturer, were 2,000 psi (13.8 MPa) tensile strength (ASTM D638), 4% elongation at break (ASTM D638), 8,000 psi (55.2 MPa) compressive yield strength (ASTM D695), and 400 ksi (2,757 MPa) compressive modulus (ASTM D695).

Electric strain gauges were applied on the surface of the FRP rod to monitor the strain distribution along the rod during the test.

**TABLE 1. Bond Test Specimens and Results**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Bonded length to rod diameter ratio</th>
<th>Groove size (in.)</th>
<th>Ultimate tensile load (lbs)</th>
<th>Average bond strength (psi)</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-a</td>
<td>6</td>
<td>0.5</td>
<td>3,523</td>
<td>1,329</td>
<td>Splitting of epoxy</td>
</tr>
<tr>
<td>12-a</td>
<td>12</td>
<td>0.5</td>
<td>6,006</td>
<td>1,133</td>
<td>Splitting of epoxy</td>
</tr>
<tr>
<td>12-b</td>
<td>12</td>
<td>3/4</td>
<td>6,880</td>
<td>1,293</td>
<td>Splitting of epoxy + concrete cracking</td>
</tr>
<tr>
<td>12-c</td>
<td>12</td>
<td>1</td>
<td>6,472</td>
<td>1,221</td>
<td>Cracking of concrete</td>
</tr>
<tr>
<td>18-a</td>
<td>18</td>
<td>0.5</td>
<td>9,452</td>
<td>1,189</td>
<td>Splitting of epoxy</td>
</tr>
</tbody>
</table>

Note: 1 in. = 25.4 mm; 1 lb = 4.448 N; 1 psi = 0.006895 MPa.
the tests. Three strain gauges were placed within the bonded length and one was applied in the unbonded region. No strain gauges were applied to the specimens with a bonded length of six diameters.

Procedure

The epoxy paste was allowed to cure for at least 15 days (full cure time at room temperature) prior to testing (Master Builders Inc. 1996). The beams were loaded under four-point bending with a shear span of 19 in. (483 mm). Each beam was instrumented with two linear variable differential transducers (LVDTs), one placed at midspan to measure the beam deflection and the other one used to measure the slip of the rod at the end of the beam in the test region side (free-end slip).

Testing was performed by first loading the beam until a crack formed at midspan, starting from the saw cut and extending up to the hinge. Next, the beam was unloaded to approximately 500 lbs (2.22 kN). Finally, load was applied again until failure. The purpose of unloading after cracking was to collect data from the strain gauges with the crack already formed. This allowed an accurate computation of the tensile force in the rod, as a result of the specimen configuration.

Results

Test results in terms of ultimate tensile load in the rod, average bond strength, and failure mode are summarized in Table 1. The ultimate tensile load in the rod was computed from the ultimate load applied to the specimen, knowing the value of the lever arm. The average bond strength was calculated as follows:

\[
\tau_b = \frac{T_u}{\pi \cdot d_b \cdot l_b}
\]

where \(T_u\) = ultimate tensile load; \(d_b\) = nominal diameter of the rod; and \(l_b\) = bonded length.

Discussion

Failure Modes

Four of the five specimens failed by splitting of the epoxy cover. This failure mechanism is similar to splitting of the concrete cover for steel reinforcing bars embedded in concrete. It is well known that, in the case of deformed reinforcement, bond is mainly transferred by bearing on the deformations of the rod (MacGregor 1997). Equal and opposite bearing stresses act on the material surrounding the rod. These stresses have a longitudinal and a radial component, with the latter causing circumferential tensile stresses in the material around the bar. When the maximum tensile stress reaches the tensile strength of the material, the cover splits parallel to the rod. The load at which splitting failure occurs is influenced by the thickness of the cover and the tensile strength of the cover material. Epoxy has typically a much higher tensile strength than concrete. However, the cover thickness of NSM reinforcement is very low compared with the minimum cover thickness of reinforcing bars embedded in concrete.

During testing of the specimens, a crackling noise revealed the progressive cracking of the epoxy paste. Eventually, the epoxy cover was completely split and the load suddenly dropped. Only specimen 12-c, which had the largest groove size, experienced a different mode of failure. Inclined cracks propagated in the concrete surface at one side of the groove (Fig. 2) and led to a gradual dropping of the load. Visual inspection after failure revealed also the presence of inclined cracks in the epoxy close to the rod’s loaded end. In this specimen, the cover was thick enough to offer a higher resistance to splitting, so that the controlling failure mechanism shifted to cracking of the surrounding concrete.

It could be observed that, when failure occurs by splitting of the epoxy cover, the ultimate load is expected to be independent from the concrete tensile strength. However, if the groove is deep enough to cause failure to occur in the concrete, the concrete tensile strength becomes a significant parameter.

Effect of Bonded Length

The ultimate load increased, as expected, with the bonded length of the rod. The average bond strength was found to be approximately constant, which indicates an even distribution of bond stresses along the bonded length at ultimate. This can be verified by means of the data obtained from the strain gauges. Fig. 3 shows the strain distribution along the bonded length in specimen 18-a at different load levels. At ultimate, the strain varies almost linearly along the bonded length, which means that the bond stress has a constant value throughout.

Effect of Groove Size

For the specimens with the 12-diameter bonded length, the ultimate load increased by 15 and 8% as the groove size increased from 0.5 in (13 mm) to 3/4 in. (19 mm) and 1 in. (25 mm), respectively. The smaller increase in the second case corresponded to a different mode of failure, as previously discussed. As the groove size increases, the thickness of the ep-
oxy cover increases, so offering a higher resistance to splitting. The ultimate load increases correspondingly, and failure may eventually shift from the epoxy to the surrounding concrete.

**Bond Stress-Slip Relationship**

Fig. 4 shows the experimental average bond stress versus free-end slip diagrams for the tested specimens. These experimental curves and the data recorded from the strain gauges can be used to obtain the local bond stress-slip relationship of NSM CFRP deformed rods. The importance of finding the local bond-slip constitutive law is that it completely characterizes the bond behavior of the system and can be used, for example, to evaluate the development length of the given type of FRP rod for each different groove size. Different methods can be used to derive the local bond stress-slip curve from slip and strain data. One of them is outlined in the following.

The bond stress, \( \tau \), can be found by equilibrium of forces, knowing the linear stress-strain relationship of the FRP rod:

\[
\tau(x) = \frac{d_s}{4} E_b \frac{d\epsilon_e(x)}{dx}
\]

where \( x \) = coordinate along the longitudinal axis of the FRP rod within the bonded length, with \( x = 0 \) corresponding to the free end and \( x = b \) to the loaded end. Therefore, the \( \tau \) versus location diagram at a given load level can be obtained from the first derivative of the strain versus location diagram at that load level multiplied by the elastic modulus \( E_b \) and the nominal diameter of the FRP rod. Recall the definition of slip:

\[
s = u_b - u_e
\]

where \( u_b \) and \( u_e \) = displacements of the FRP reinforcement and of the epoxy, respectively. Since

\[
\epsilon_e = \frac{du_e}{dx} \quad \text{and} \quad \epsilon_s = \frac{du_b}{dx}
\]

and assuming that the epoxy strain, \( \epsilon_e \), may be considered negligible when compared with the FRP strain, \( \epsilon_s \), it follows that

\[
\frac{ds}{dx} = \epsilon_e
\]

from which

\[
s(x) = s(0) + \int_0^x \epsilon_e(x) \, dx
\]

where \( s(0) \) = free-end slip of the FRP rod. Therefore, the slip versus location diagram for each given load level can be obtained by integrating the strain versus location curve and adding the free-end slip at that load level. The local \( \tau \)-slip relationship can be obtained by combining the two curves \( \tau(x) \) and \( s(x) \), expressed by (2) and (6). A \( \tau \) versus \( x \) curve and an \( s \) versus \( x \) curve can be obtained for each value of the applied load. Therefore, a \( \tau \) versus slip diagram corresponding to each value of applied load can be drawn. At a given load level, the \( \tau \) versus slip diagram covers the range of slip between the free-end slip and the loaded-end slip at that load level. The entire curve can be obtained as the envelope of the partial curves at all load levels between zero and ultimate. Once a local \( \tau \)-slip relationship is obtained, the next step is to find an analytical expression capable to model such a relationship. For steel and FRP reinforcing bars in concrete, various equations have been previously proposed (Eligehausen et al. 1983; Cosenza et al. 1997). Each of them contains a certain number of unknown parameters which need to be calibrated by comparison with experimental data. Provided that one of these laws is found to be representative of the bond behavior of NSM FRP rods, the values of the unknown parameters may be determined by best fitting techniques using the experimental curves. Once the analytical \( \tau \)-slip relationship is known, it can be used to analytically solve all problems related to the bond behavior, particularly to calculate the development length of NSM FRP rods.

**Test Method**

The bond test method adopted in this experimental program appears to be an efficient protocol for investigation of bond. It gives reliable data while maintaining a manageable specimen size. Furthermore, specimen details, such as the saw cut and the top hinge at midspan, allow accurate data analysis. It is suggested as a possible new standard test method.

**BEAM SHEAR TEST**

**Specimens**

This part of the experimental program involved testing of three full-scale RC beams with a T-shaped cross section and a total length of 10 ft. (3 m). All the beams had no internal shear reinforcement. The amount of steel flexural reinforcement was designed to obtain a shear failure in spite of the envisioned shear capacity enhancement provided by NSM FRP rods. As a result, all beams were reinforced with two No. 9 [nominal diameter 1.128 in. (28.7 mm)] longitudinal steel rebars. The average concrete strength determined on three 6 in. (152 mm) diameter by 12 in. (305 mm) concrete cylinders according to ASTM C39-97 was 4,560 psi (31.4 MPa). The internal steel flexural reinforcement had a yield strength of 62.7 ksi (432 MPa), as determined from testing of three coupon specimens according to ASTM A370-97. The epoxy paste was the same used for the bond specimens. The dimensions of the beams are given in Fig. 5. All the grooves had a square size. Furthermore, specimen details, such as the saw cut and the top hinge at midspan, allow accurate data analysis. It is suggested as a possible new standard test method.

**FIG. 5.** Dimensions of Beams and Loading Scheme

**FIG. 4.** Average Bond Stress versus Free-End Slip

JOURNAL OF COMPOSITES FOR CONSTRUCTION / MAY 2001 / 117
Beam B1 (no shear strengthening) was used as a baseline comparison to evaluate the strength enhancement provided by the NSM FRP rods. The other two beams (B2 and B3) were strengthened for shear with No. 3 CFRP deformed rods applied in vertical grooves cut in both sides of the beams. The length of the grooves was equal to the height of the beam web. The two beams differed in the spacing of the NSM FRP rods, which was equal to 7 in. (178 mm) and 5 in. (127 mm) for B2 and B3, respectively.

Procedure

The epoxy paste was allowed to cure for 15 days prior to testing. The beams were loaded under four-point bending with a shear span of 42 in. (1,067 mm), as shown in Fig. 5. Load was applied by means of a 400 kip (1,780 kN) hydraulic jack connected to an electric pump and recorded with a 200 kip (890 kN) load cell. Each beam was instrumented with four LVDTs placed at midspan on the two sides and at each support to measure net deflection.

Results

All the tested beams experienced shear failure. The ultimate load of the control beam was 40.6 kips (180.6 kN). Beams B2 and B3 failed at 51.8 kips (230.4 kN) and 57.4 kips (255.3 kN), showing an increase in the shear capacity with respect to the control beam of 28 and 41%, respectively. The load versus deflection curves of the three beams are reported in Fig. 6. In the control beam, diagonal shear cracks formed throughout the shear span, widened, and propagated up to failure. Diagonal shear cracks developed also in beams B2 and B3. Failure of these beams occurred by splitting of the epoxy cover in one of the NSM FRP rods intersected by the major shear crack (Fig. 7).

PROPOSED DESIGN APPROACH

Background

The experimental results obtained in the present study suggest that, in RC beams strengthened in shear with NSM FRP rods, the controlling failure mechanism is related to bond of the FRP shear reinforcement. Results of the shear tests are consistent with those of the bond tests in that bond failure appears to be controlled by splitting of the epoxy paste cover. However, it should be noted that the obtained results are related to the materials used in this experimental study. The use of FRP rods having different properties, especially in terms of surface configuration, and the use of epoxy paste having a different tensile strength may lead to different results. In the following, a simple design approach to compute the contribution of NSM FRP rods to the shear capacity of an RC beam is proposed. On the basis of the results obtained in the present study by tensile tests (elastic modulus of the FRP rods) and bond tests (average bond strength), the proposed approach is then applied to the beams tested in shear.

Shear Strength of RC Beams Strengthened with FRP Reinforcement

The nominal shear strength of an RC beam may be computed by the basic design equation presented in ACI 318-95 (American Concrete Institute 1995):

\[ V_s = V_c + V_r \]  

(7)

The nominal shear strength is given by the sum of the shear strength of the concrete, \( V_c \), and the shear strength provided by the steel shear reinforcement, \( V_r \). In the case of beams externally strengthened with FRP, the nominal shear strength can be computed by adding a third term to account for the contribution of the FRP reinforcement (Khalifa et al. 1998):

\[ V_s = V_c + V_r + V_{FRP} \]  

(8)

The design shear strength is obtained by applying a strength reduction factor, \( \phi \), to the nominal shear strength. When proposing a design procedure for RC beams strengthened in shear with externally bonded FRP sheets, Khalifa et al. (1998) suggested to maintain the reduction factor of \( \phi = 0.85 \) given in ACI 318-95 for the concrete and steel terms, and to apply a more conservative reduction factor (\( \phi = 0.70 \)) to the FRP contribution, to account for the novelty of this strengthening technique. The same reasoning could be applied to shear strengthening with NSM FRP rods.

Contribution of NSM FRP Rods to Shear Capacity

To compute the nominal shear strength given by (8), the contribution of NSM FRP rods to the shear capacity needs to be quantified. Although several parameters are believed to exert an influence on this contribution, it is not possible yet to develop a comprehensive design approach including all the significant parameters. To the best of the writers’ knowledge, no experimental data on RC beams strengthened in shear with NSM FRP rods, other than that presented herein, is available to date.

This preliminary design approach includes two equations that may be used to obtain \( V_{FPRP} \) and suggests taking the lower of the two results as the contribution of NSM FRP rods to the shear capacity. The first equation computes the FRP shear
strength contribution related to bond-controlled shear failure, \( V_{1,\text{FRP}} \). The second equation calculates the shear resisted by NSM FRP rods when the maximum strain in the rods is equal to 4,000 \( \mu \varepsilon \), \( V_{2,\text{FRP}} \). This limit is suggested to maintain the shear integrity of the concrete (Khalifa et al. 1998). At higher levels of strain, the shear crack width would be such that aggregate interlock would be lost and the shear capacity of the concrete compromised.

It is proposed that, in the calculations, a reduced value is used for the length of NSM rods:

\[
d_{\text{net}} = d_e - 2 \cdot c
\]

(9)

where \( d_e \) = length of the rods; and \( c \) = concrete cover of the internal longitudinal reinforcement.

**Calculation of \( V_{1,\text{FRP}} \)**

\( V_{1,\text{FRP}} \) is the FRP shear strength contribution related to bond-controlled shear failure. It is computed using the following assumptions:

- The inclination angle of the shear cracks equals 45°.
- The bond stresses are uniformly distributed along the effective lengths of the FRP rods at ultimate.
- The ultimate bond stress is reached in all the rods intersected by the crack at ultimate.

The first assumption can be easily removed. However, the error it may produce is not significant if considered in the context of approximation of this model. The validity of the other two assumptions is related to the bond behavior of the NSM FRP rods, particularly to the degree of ductility of bond, and to the depth of the beam. For the CFRP rods used in this study, the bond stress distribution at ultimate was shown to be approximately uniform even for the longest of the examined bonded lengths (18 diameters). On the other hand, the depth of the tested beams is such that the longest bonded length considered in the calculations was less than 18 rod diameters. The bond stress distribution for longer bonded lengths can be analytically obtained once the local bond stress-slip relationship is known.

When other types of FRP rods are used, the bond behavior can be substantially different and the assumption of uniform bond stresses at ultimate may be inadequate. In this case, the value of the average bond strength would depend on the bonded length (which in turn, for application in shear strengthening, depends on the beam depth) and could be computed from the local bond stress-slip relationship of the given type of FRP rod.

The shear force resisted by the FRP may be computed as the sum of the forces resisted by the FRP rods intersected by a shear crack. Each rod intersected by a crack may ideally be divided in two parts at the two sides of the crack. The force in each of these rods at the crack location can be calculated as the product of the average bond strength and the surface area of the shortest part, which from now on will be referred to as effective length of the rod. Therefore

\[
V_{1,\text{FRP}} = 2 \cdot \sum A_i f_i = 2 \cdot \pi \cdot d_e \cdot \tau_{\text{eff}} \cdot L_{\text{eff}}
\]

(10)

where \( A_i = \) nominal cross-section area of the rods; \( f_i = \) tensile stress in the rod at the crack location; and the summation is extended to all the rods intersected by a 45° crack. The factor 2 is due to the presence of NSM rods on both sides of the beam, \( L_{\text{eff}} \) is the sum of the effective lengths of all the rods crossed by the crack. \( L_{\text{tot}} \) has to be calculated in the most unfavorable crack position, that is, the position in which it is minimum. Therefore

\[
V_{1,\text{FRP}} = 2 \cdot \pi \cdot d_e \cdot \tau_{\text{eff}} \cdot L_{\text{min}}
\]

(11)

where

\[
L_{\text{min}} = d_{\text{net}} - s \quad \text{if } d_{\text{net}} / 3 < s < d_{\text{net}}
\]

(12)

\[
L_{\text{min}} = 2 \cdot d_{\text{net}} - 4 \cdot s \quad \text{if } d_{\text{net}} / 4 < s < d_{\text{net}} / 3
\]

(13)

In (12) and (13), \( s = \) spacing of the NSM FRP rods.

**Calculation of \( V_{2,\text{FRP}} \)**

\( V_{2,\text{FRP}} \) is the FRP shear strength contribution corresponding to a maximum FRP strain of 4,000 \( \mu \varepsilon \). The effective length of an FRP rod crossed by the crack corresponding to a strain of 4,000 \( \mu \varepsilon \) and the average bond strength \( \tau_{\text{eff}} \) is

\[
L = 0.001 \cdot d_e \cdot E_b \cdot \frac{1}{\tau_{\text{eff}}}
\]

(14)

To compute \( V_{2,\text{FRP}} \), the same assumptions made for \( V_{1,\text{FRP}} \) of 45-degree shear cracks and bond stress redistribution at ultimate can be made. If strain in the longest effective length at the crack location reaches 4,000 \( \mu \varepsilon \) sooner than its average bond stress reaches \( \tau_{\text{eff}} \), that is, if one or more effective lengths are longer than \( L_{\text{eff}} \), at ultimate these effective lengths will carry a tensile load corresponding to 4,000 \( \mu \varepsilon \) strain (which means, they will count as \( L_{\text{eff}} \)), while tensile load in the other effective lengths will be the product of \( \tau_{\text{eff}} \) and their surface area. The following results were obtained in the case of vertical rods for three different spacing ranges:

- \( d_{\text{net}} / 2 < s < d_{\text{net}} / 3 \); bond failure occurs before the maximum strain reaches 4,000 \( \mu \varepsilon \), therefore, \( V_{1,\text{FRP}} \) controls. If \( L_{\text{eff}} < d_{\text{net}} - s \), \( V_{2,\text{FRP}} \) controls with the value:

\[
V_{2,\text{FRP}} = 2 \pi d_e \cdot \tau_{\text{eff}} \cdot L_{\text{eff}}
\]

(15)

- \( d_{\text{net}} / 3 < s < d_{\text{net}} / 2 \); \( V_{1,\text{FRP}} \) controls if \( L_{\text{eff}} > s \). If \( L_{\text{eff}} > s, V_{2,\text{FRP}} \) controls with the value:

\[
V_{2,\text{FRP}} = 2 \pi d_e \cdot \tau_{\text{eff}} (L_{\text{eff}} + d_{\text{net}} - 2s)
\]

(16)

\[
V_{2,\text{FRP}} = 4 \pi d_e \cdot \tau_{\text{eff}} \cdot L_{\text{eff}} \quad \text{if } L_{\text{eff}} < d_{\text{net}} - 2s
\]

(17)

- \( d_{\text{net}} / 4 < s < d_{\text{net}} / 3 \); \( V_{1,\text{FRP}} \) controls if \( L_{\text{eff}} > d_{\text{net}} - 2s \). If \( L_{\text{eff}} < d_{\text{net}} - 2s, V_{2,\text{FRP}} \) controls with the value:

\[
V_{2,\text{FRP}} = 2 \pi d_e \cdot \tau_{\text{eff}} (L_{\text{eff}} + d_{\text{net}} - 2s)
\]

(18)

\[
V_{2,\text{FRP}} = 6 \pi d_e \cdot \tau_{\text{eff}} \cdot L_{\text{eff}} \quad \text{if } L_{\text{eff}} < d_{\text{net}} - 3s
\]

(19)

\[
V_{2,\text{FRP}} = 6 \pi d_e \cdot \tau_{\text{eff}} \cdot L_{\text{eff}}
\]

(20)

**Summary of Proposed Design Procedure**

1. Compute \( d_{\text{net}} \) using (9).
2. Compute the FRP contribution to the shear capacity, based on equations (11)–(20).
3. Apply reduction factor \( \phi \) for design strength.

This procedure was applied to beams B2 and B3. The value of \( \tau_{\text{eff}} \) was taken equal to 1 ksi (6.9 MPa), based on results obtained from the bond tests. Calculated and experimental values of \( V_1 \) and \( V_{2,\text{FRP}} \) are reported in Table 2. The experimental \( V_{1,\text{FRP}} \) was calculated by subtracting the shear strength of beam...
B1 from that of the strengthened beams. $V_c$ was computed using the upper limit indicated by ACI 318-95:

$$V_c = 3.5\sqrt{f'_c} b_w d$$

(21)

where $f'_c$ = concrete compressive strength in psi; $b_w$ = web width in inches; and $d$ = distance from the extreme compression fiber of the cross section to the centroid of the longitudinal reinforcement, in inches. This expression gives a $V_c$ closer to the experimental value, so that $V_c$ (calculated shear strength) and $V_{exp}$ (experimental shear strength) can be compared focusing on the FRP contribution. $V_{exp}$ was equal to $V_{FRP}$ for both beams. The proposed design approach gives results reasonably close to the experimental values. Furthermore, all predictions are conservative.

Further experimental research is needed to assess the validity of the proposed procedure on a wider experimental base. Limits on spacing of the NSM rods should be established from test results.

**CONCLUSIONS**

Tensile and bond tests on CFRP deformed rods for application as NSM reinforcement were carried out using coupon-size specimens. The adopted test methods appeared to perform efficiently and give consistent results. Two bond failure modes were observed: splitting of the epoxy cover and cracking of the concrete surrounding the groove, depending on the groove size. The average bond strength results were approximately constant with the bonded length, indicating an even distribution of bond stresses at failure.

Three full-size RC beams were tested, two of which were strengthened in shear with NSM FRP rods. This appears to be an effective technique to enhance the shear capacity of RC beams. ON the basis of the results obtained by tensile tests (elastic modulus of the FRP rods) and bond tests (average bond strength), results of the shear tests were compared with the predictions of a simple design approach, showing reasonable agreement.

Further research is needed to assess the validity of the proposed design approach on a wider experimental base. Even though limited in scope, the protocol presented herein (i.e., characterization at material, system, and structural member level and design approach) is proposed to be followed for other products and strengthening applications of the NSM reinforcement technology.

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**REFERENCES**

ACI Committee 318. (1995). Building code requirements for structural concrete (ACI 318-95) and commentary (ACI 318R-95), American Concrete Institute, Detroit.


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**NOTATION**

The following symbols are used in this paper:

- $A_n =$ nominal cross-sectional area of rod
- $b_w =$ web width
- $c =$ cover of longitudinal steel reinforcement
- $d =$ distance from extreme compression fiber of cross section to centroid of longitudinal reinforcement
- $d_{frp} =$ nominal rod diameter
- $d_{red} =$ reduced length of FRP rods
- $d_l =$ length of FRP rods
- $E_n =$ elastic modulus of FRP rods
- $f'c =$ compressive strength of concrete
- $f_i =$ tensile stress in rod at crack location
- $L_e =$ effective length of rod crossed by crack
- $L_{crack} =$ effective length of rod crossed by crack corresponding to tensile strain of 4,000 $\mu$e
- $L_{max} =$ maximum effective length of rod crossed by crack
- $L_{net} =$ sum of effective lengths of rods crossed by crack
- $L_{net\min} =$ minimum value of $L_{net}$
- $l_b =$ bonded length
- $s =$ spacing of NSM shear reinforcement
- $s_i =$ slip
- $u_0 =$ displacement of FRP rod
- $u_e =$ displacement of epoxy paste
- $x =$ coordinate along longitudinal axis of rod
- $T_u =$ ultimate tensile load
- $V_n =$ nominal shear strength provided by concrete
- $V_{FRP} =$ nominal shear strength provided by FRP shear reinforcement
- $V_c =$ calculated shear strength

---

**TABLE 2. Shear Test Results**

<table>
<thead>
<tr>
<th>Beam</th>
<th>$V_c^*$</th>
<th>$V_{exp}$</th>
<th>$V_{FRP}^*$</th>
<th>$\phi V_{exp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>19.7</td>
<td>20.3</td>
<td>20.3</td>
<td>1.030</td>
</tr>
<tr>
<td>B2</td>
<td>2.4</td>
<td>5.6</td>
<td>5.6</td>
<td>25.9</td>
</tr>
<tr>
<td></td>
<td>26.8</td>
<td>28.7</td>
<td>28.7</td>
<td>1.172</td>
</tr>
<tr>
<td>B3</td>
<td>7.1</td>
<td>5.6</td>
<td>5.6</td>
<td>28.7</td>
</tr>
</tbody>
</table>

Note: 1 kip = 4.488 kN.

*Note that reduction factor 0.5 for $V_c$ due to absence of steel shear reinforcement has not been applied. $\phi = 0.85$ for concrete term and 0.7 for FRP term.

*Shear strength of beam B1.
\( V_s \) = nominal shear strength provided by steel shear reinforcement;
\( V_{1_{FRP}} \) = nominal shear strength provided by FRP related to bond-controlled shear failure;
\( V_{2_{FRP}} \) = nominal shear strength provided by FRP related to maximum strain in rods equal to 4,000 \( \mu \varepsilon \);

\( V_{exp} \) = experimental shear strength;
\( \varepsilon_s \) = tensile strain of FRP rod;
\( \varepsilon_e \) = tensile strain of epoxy paste;
\( \tau \) = bond stress;
\( \tau_b \) = average bond strength;
\( \phi \) = strength reduction factor.