Effectiveness of FRP for Strengthening Concrete Bridges

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Summary

This paper demonstrates an alternative use of FRP for strengthening concrete bridges in response to the demand towards increasing the flexural capacity to accommodate new truck loads. Several large-scale models of a prestressed concrete bridge were constructed and tested to failure to investigate the effectiveness of different FRP strengthening techniques. These techniques include near surface mounted bars and strips as well as externally bonded sheets and strips. Bond characteristics of near surface mounted FRP bars and strips are investigated by testing independently 17 concrete beams under monotonic static loading. Mathematical models for the interfacial shear stresses and the minimum anchor length needed for near surface mounted bars and strips are presented. The feasibility of strengthening a prestressed concrete bridge using near surface mounting technique is discussed. The paper presents also an overview of the cost-effectiveness of each of the strengthening techniques considered in this study.

Introduction

The need to rehabilitate deteriorating civil infrastructure is becoming an urgent problem worldwide. In the United States, nearly 11% of the nation's highway bridges are presently structurally deficient and 19% are functionally obsolete [1]. In the United Kingdom, over 10,000 concrete bridges need structural attention. In the rest of Europe, it is estimated that the repair of concrete structures due to corrosion of reinforcing bars costs over USD 600 million annually [2]. FRP offers designers an excellent combination of properties not available for other materials and presents a potential solution to this crisis. High strength-to-weight ratio, ease of installation and corrosion resistance characteristics make FRP ideal for strengthening applications.

Since 1982, externally bonded FRP sheets/strips have been successfully applied to strengthen concrete structures [3]. Although externally bonded FRP reinforcement performed extremely well in practice, premature debonding failure was observed and identified by many researchers [4, 5]. Several details were proposed to avoid this type of failure, which is unacceptable from the point of view of structural safety [6].

Use of near surface mounted FRP rods and strips can preclude delamination-type failures, frequently observed by using externally bonded reinforcement. Near surface mounting technique becomes particularly attractive for flexural strengthening in the negative moment regions of slabs and decks, where external reinforcement would be subjected to mechanical and environmental damage and would require protective cover which could interfere with the presence of floor finishes.

Large-scale Slab Specimens

Large-scale specimens in the range of a half-scale model of typical bridges were tested in this program due to lack of information on the use of near surface mounted FRP reinforcement for flexural strengthening in regions of combined bending and high shear stresses. Three post-tensioned solid slabs, representative of typical bridge slabs over intermediate pier columns, were constructed. The specimens were tested in simple span with a double cantilever configuration. The specimens also represent a portion of continuous bridges between the inflection points beyond two adjacent supports. Each specimen was tested three times using loads applied at different locations in each test. The first and second tests were performed on the two cantilevers where the load was applied at the edge of each cantilever. The third test was conducted using a load applied at the midspan. Prior to the third test of the midspan, the cracks resulted from testing the two cantilevers were sealed entirely by injecting a high strength epoxy resin adhesive into the concrete. The midspan was then strengthened using FRP and tested. This paper briefly presents test results of the cantilevers. Detailed information of the experimental program can be found elsewhere [7].

The number and layout of the prestressing tendons were selected to have the same stress level of a typical prestressed bridge under service loading conditions. The loss of the prestressing force was calculated according to the current AASHTO specifications [8]. The number of the top and bottom mild steel bars was selected to represent the same reinforcement ratio in the cantilever portion of typical prestressed bridges. Reinforcement details of the specimens are shown in Fig. 1. The average compressive strength of concrete after 28 days ranged between 45 and 50 MPa for the three slabs.

Fig. 1: Reinforcement details for large-scale specimens

Strengthening Techniques

Slab S1

One cantilever of slab S1 remained unstrengthened (specimen C1) while the second cantilever (specimen C2) was strengthened using near surface mounted Leadline CFRP bars. The bars have a modulus of elasticity of 147 GPa and an ultimate tensile strength of 1970 MPa as reported by the manufacturer.
Based on equilibrium and compatibility conditions, six 10 mm diameter Leadline CFRP bars were used to achieve a 30% increase in the ultimate load carrying capacity of the slab. Six grooves of approximately 16 mm wide and 30 mm deep were cut at the tension surface of the cantilever. The groove ends were widened to provide wedge action and to prevent possible slip of the bars. An epoxy was used for bonding the CFRP bars to the surrounding concrete.

**Slab S2**

The second slab, S2, was used to investigate the performance of both near surface mounted and externally bonded CFRP strips in strengthening of concrete bridges. The analysis indicated a need of six CFRP strips, 50 mm wide and 1.4 mm thick to achieve a 30% increase in the ultimate capacity of the cantilever slab. The first cantilever (specimen C3) was strengthened using six externally bonded CFRP strips. The second cantilever (specimen C4) was strengthened also using six CFRP strips inserted into grooves cut at the top surface of the concrete. In order to insert the strips within the concrete cover layer, the strips were cut into two halves each, 25 mm wide. Using a concrete saw, grooves of approximately 5 mm wide and 25 mm deep were cut at the tension surface of the second cantilever. The CFRP strips have a modulus of elasticity of 150 GPa and an ultimate tensile strength of 2000 MPa as reported by the manufacturer.

**Slab S3**

To investigate the effectiveness of strengthening using externally bonded sheets in comparison to the three previously prescribed techniques, one cantilever of Slab S3 (specimen C5) was strengthened using externally bonded CFRP sheets. The required area of CFRP sheets was calculated to achieve a 30% increase in flexural capacity of the cantilever slab. The sheets have a modulus of elasticity of 228 GPa and an ultimate tensile strength of 4275 MPa as reported by the manufacturer.

The second cantilever (specimen C6) was strengthened using eight near surface mounted C-BAR CFRP bars. Based on testing, the bars have a modulus of elasticity of 111 GPa and an ultimate tensile strength of 1918 MPa. Detailed strengthening procedures for all slabs are reported in [7].

**Testing Scheme**

The slabs were tested under static loading conditions using a uniform line-load acting on a width equivalent to the width of a tire contact patch according to the AASHTO HSS30 design vehicle. A closed-loop MTS, 5000 kN, testing machine was used to apply the load using stroke control mode with a rate of 0.5 mm/min up to failure. Neoprene pads were placed between the steel beam and the slab to simulate the contact surface of a truck tire and to avoid local crushing of the concrete. The load was applied at a distance of 330 mm from the cantilever edge. To prevent possible damage of the other cantilever during the first test, an intermediate support was provided as shown in Fig. 2.

**Test Results and Discussion**

The load-deflection behavior of cantilevers specimens strengthened with near surface mounted Leadline bars (C2), CFRP strips (C4) and C-Bars (C6) is compared to the unstrengthened specimen (C1) as shown in Fig. 3.

CFRP strips and sheets, respectively. The behavior of the control specimen, C1, is also shown for comparison. The figure clearly indicates that the strength, stiffness and ductility are greatly improved with the addition of CFRP reinforcement. Identical behavior was observed for specimens C3 and C5 up to a load level of 500 kN. After yielding of the steel reinforcement, the stiffness of specimen C5 was about 3.3 times higher than that of the unstrengthened cantilever. Initial cracking in the concrete substrate at the anchorage zone was observed at a load level of 400 kN for specimen C3. Upon additional loading the cracks continued to widen up to a load level of 530 kN where unstable delamination occurred resulting in peeling of the
strips as shown in Fig. 5. The observed mode of failure for all other cantilever specimens was due to crushing of the concrete in the compression zone at the face of the support.

Fig. 5: Delamination failure of externally bonded CFRP strips (specimen C3)

The failure load of the control specimen was 476 kN. Strengthening the specimen using near surface mounted Leadline bars increased the strength by 36% in comparison to the design value of 30%. Using C-BAR CFRP bars instead of Leadline bars increased the strength by 39%. The cantilever specimen strengthened with near surface mounted CFRP strips showed the highest increase in strength by 43%. Using the same area of CFRP strips as externally bonded reinforcement increased the strength by only 11% due to the premature peeling failure of the strips. Using externally bonded CFRP sheets provided superior strength above all the techniques considered in this study and increased the strength by 44%.

Cost Analysis

One of the prime objectives of this study is to provide a cost-effective analysis for each of the strengthening techniques considered in this investigation. All techniques considered in this study were designed to increase the strength by 30% using the characteristics of each FRP material. The total cost includes the cost of materials, equipment needed during construction, and labor cost. The analysis indicates that using near surface mounted CFRP strips and externally bonded CFRP sheets provided the maximum increase in strength. The construction cost of externally bonded CFRP sheets is only 35% in comparison to near surface mounted strips. Using either near surface mounted Leadline bars or C-BAR CFRP bars provided approximately the same increase in ultimate load carrying capacity. With respect to cost, strengthening using C-BAR CFRP bars is 50% less. Using an efficiency scale (E) defined by Eq. (1), the efficiency of each technique was evaluated as shown in Fig. 6.

$$ E = \frac{\% \text{ Increase in strength}}{\text{Construction cost in USD} \times 100} $$

The results show that strengthening using externally bonded CFRP sheets is the most efficient technique in terms of strength improvement and construction cost.

Bond Specimens

Test results of the presented large-scale specimens provided sufficient evidence and confidence in near surface mounted FRP bars and strips as innovative strengthening techniques. To evaluate the bond characteristics and load transfer mechanism between near surface mounted FRP reinforcement and concrete, a total of 17 simply supported T-beams, with a span of 2.5 m and a depth of 300 mm, were tested. Reinforcement details of the bond specimens are given in Fig. 7.

The arrangement of the bottom reinforcement was selected to identify the location of the flexural failure of the strengthened specimens. The T-section configuration was selected to avoid compression failure due to crushing of the concrete. Two series of beam specimens designated as A and B were strengthened using near surface mounted FRP bars and strips, respectively. The beams were tested under a concentrated load applied at midspan. A closed-loop MTS, 1000 kN testing machine was used to apply the load using stroke control mode. The rate of loading was 1.0 mm/min up to yielding of the internal steel reinforcement, beyond which the rate was increased to 3.0 mm/min up to failure.

With the maximum moment occurring at the midspan section of the beam, failure could be due to:

1) debonding of the CFRP reinforcement or
2) rupture of the CFRP reinforcement.

Specimens were adequately designed to avoid concrete crushing and premature failure due to shear. In case of bond failure, the bond length of the CFRP reinforcement was increased in the subsequent specimens. In case of flexural failure, the bond length was decreased in the following specimens. This scheme was applied until an accurate development length of near surface mounted CFRP bars and strips was achieved.

Test Series

Series A

A total of eight beams were tested from series A. One beam was tested as a control specimen while the other seven beams were strengthened with near surface mounted C-BAR CFRP bars. Each beam was strengthened using one bar inserted into a groove cut at the bottom surface of the beam. Beams A1, A2, A3, and A4 with embedment lengths of 150, 550, 800, and 1200 mm, respectively were tested using an epoxy adhesive for bonding the bars. This particular adhesive is commonly used as a mortar binder for vertical and overhead repairs of structural concrete. To investigate the suitability of the epoxy adhesive, beams A5, A6 and A7 with embedment lengths of 550, 800, and 1200 mm, respectively were tested using an alternative epoxy adhesive which is designed specifically for grouting bolts, dowels and steel rebars in concrete.
The control specimen, A0, failed due to crushing of concrete at a load level of 56 kN. Using an embedment length of 150 mm (beam A1) provided insignificant increase in stiffness or strength. Failure of beams A2, A3, and A4 with embedment lengths of 550, 800, and 1200 mm, respectively, was due to debonding of the CFRP bar. The ultimate loads for these beams ranged between 68 kN to 78 kN, with an increase of 21 to 40% over the control specimen. Identical behavior was observed for the beams retrofitted with the two sets of adhesives at different embedment lengths. Altering the type of the epoxy adhesive has a negligible effect on the ultimate load carrying capacity of the strengthened beams. The observed mode of failure for all beams strengthened with CFRP bars was cracking of the concrete surrounding the epoxy paste followed by complete debonding of the bars. After debonding failure, the beams behaved as conventional concrete beams reinforced with steel bars. The load dropped to the yielding load of the steel bars until crushing of the concrete occurred. Debonding was observed to initiate at the location where the bottom steel reinforcement was terminated due to high shear stress concentration. Such a phenomenon was confirmed through nonlinear finite element modeling. Using an embedment length greater than 800 mm provided considerable enhancement in the ultimate load carrying capacity of the strengthened beams. Test results indicated a failure load of 73 kN for an embedment length of 800 mm. Increasing the embedment length by 50% resulted in an increase in the ultimate load carrying capacity by only 7%. Test results showed that the maximum measured tensile strain in the CFRP bars at failure did not exceed 42% of the rupture strain of bars, regardless of the embedment length used. Such a limiting strain is highly dependent on the configuration of the bottom steel reinforcement inside the beam as well as on the stress level at the concrete-epoxy interface. Non-linear finite element modeling showed that the maximum tensile strain in the CFRP bars at debonding could reach as high as 60% when no bottom steel reinforcement is terminated. Therefore, a limiting debonding strain of 40% of the rupture strain of the bars will be used in the analytical modeling. The efficiency of using CFRP bars as near surface mounted reinforcement is controlled primarily by the bond characteristics of the bars as well as the bond between the adhesive material and the concrete. This behavior has been confirmed and reported by other researchers [9].

**Series B**

A total of nine concrete beams were tested from series B. Beam B0 was tested as a control specimen. The remaining beams were strengthened using near surface mounted CFRP strips (25 x 1.2 mm). Each beam was strengthened using one strip inserted into a groove cut at the bottom surface of the beam. The sequence of testing started first by testing specimens B1, B2, B3, and B4 with embedment lengths of 150, 250, 500, and 750 mm, respectively. Based on the results of these tests, specimens B5 to B8 with embedment lengths ranging from 850 mm to 1200 mm were tested. The control specimen, B0, failed due to crushing of concrete at a load level of 52 kN. Test results indicate that using embedment lengths up to 250 mm provides insignificant improvement in strength. This is attributed to the early debonding of the CFRP strips.

A considerable enhancement in strength was observed for embedment lengths greater than 250 mm. Specimens B3 and B4 with embedment lengths of 500 mm and 750 mm, respectively failed also due to debonding of the CFRP strips. Debonding was observed at both ends of the strips as well as at midspan. This is attributed to high shear stress concentration at cutoff point as well as at the vicinity of flexural cracks. The failure loads for both beams were 60 kN and 74 kN, respectively. This indicates that full composite action has not yet been developed and therefore, the measured ultimate load was increasing with the increase of the embedment length. Beams B5, B6, B7, and B8 were strengthened using embedment lengths ranging from 850 mm to 1200 mm. The failure of these beams was due to rupture of the CFRP strips as shown in Fig. 8. The failure loads for specimens B5, B6, B7 and B8 were almost identical and ranged from 75 kN to 80 kN. The maximum measured tensile strain in the CFRP strips used for specimens B5, B6, B7 and B8 was approximately 1.3%. The measured failure loads for the four beams were almost identical. These results suggest that the minimum embedment length needed to rupture the near surface mounted CFRP strips, with the given dimensions used in this program is 850 mm.

**Analytical Models**

**Near Surface Mounted FRP Bars**

Transfer of stresses from a deformed near surface mounted FRP rod to the concrete is mainly by mechanical interlocking of the lugs with the surrounding epoxy. The resultant force exerted by the lug on the epoxy is inclined at an angle to the axis of the bar as shown in Fig. 9, where \( 1/(\tan \theta) \) is the coefficient of friction between the bar and the adhesive. The radial component of the resultant force creates zones of high tensile stresses at the FRP-epoxy interface as well as at the concrete-epoxy interface. Thick-walled cylinder theory was applied by many researchers [10] to analyze the stresses in a concrete cylinder surrounding a single bar. Nevertheless, the same theory could not be applied for near surface mounted FRP bars. Lack of confinement, uneven distribution of bond stresses and composite interaction between concrete, epoxy and FRP materials complicate the analysis of near surface mounted FRP bars and consequently, thick-walled cylinder theory is no longer valid.
In this section finite element analysis is employed to provide in-depth understanding of the bond characteristics and load transfer mechanism between near surface mounted FRP bars and concrete. The finite element modeling described in this paper was conducted using a special program. Fig. 10 shows the mesh dimensions used in modeling a portion of a concrete beam strengthened with a near surface mounted FRP bar. The concrete and the epoxy were modeled using eight-node plain strain elements with a 3 x 3 Gauss integration scheme. Groove dimensions, bar location and properties of concrete and epoxy were set identical to those used in the bond specimens. Radial pressure was applied at the bar location to simulate the bond stresses transferred from the bar to the surrounding epoxy.

Two different types of debonding failures could occur for near surface mounted FRP bars. The first mode of failure is due to splitting of the epoxy cover as a result of high tensile stresses at the FRP-epoxy interface, and is termed "epoxy split failure". Increasing the thickness of the epoxy cover reduces the induced tensile stresses significantly. Furthermore, using adhesives of high tensile strength delays epoxy split failure. This type of debonding failure forms with longitudinal cracking through the epoxy cover [9]. The second mode of failure is due to cracking of the concrete surrounding the epoxy adhesive and is termed "concrete split failure". This mode of failure will take place when the tensile stresses at the concrete-epoxy interface reaches the tensile strength of the concrete. Widening the groove minimizes the induced tensile stresses at the concrete-epoxy interface and increases the debonding loads of near surface mounted bars. Concrete split failure was the governing mode of failure for the bond specimens reported in this study. Large epoxy cover and high tensile strength of the epoxy adhesive provided high resistance to epoxy split failure and shifted the failure to occur at the concrete-epoxy interface.

Measurements of bar strains along the embedment length of near surface mounted FRP bars shows linear strain distribution at high load levels. Therefore, the tangential bond stress, $\tau$, is constant and can be expressed as:

$$\tau = \frac{d f_{FRP}}{4 L_d}$$

(2)

where $d$ is the diameter of the bar, and $L_d$ is the embedment length needed to develop a stress of $f_{FRP}$ in the near surface mounted bar. If the coefficient of friction between the bar and the epoxy is $\mu$, the radial stresses, $\sigma_{radial}$, can be expressed as:

$$\sigma_{radial} = \frac{\tau}{\mu} = \frac{d f_{FRP}}{4 \mu L_d}$$

(3)

The tensile stresses at the concrete-epoxy interface, $\sigma_{con-epoxy}$, and at the FRP-epoxy interface, $\sigma_{FRP-epoxy}$, can be expressed in terms of the radial stress as follows:

$$\sigma_{con-epoxy} = G_1 \frac{d f_{FRP}}{4 \mu L_d}$$

(4)

$$\sigma_{FRP-epoxy} = G_2 \frac{d f_{FRP}}{4 \mu L_d}$$

(5)

where $G_1$, $G_2$ and $G_2'$ are coefficients determined from the finite element analysis based on a unit radial pressure applied at the bar location and using specified groove dimensions, concrete and adhesive properties. The maximum tensile stresses at the FRP-epoxy interface, $\sigma_{FRP-epoxy}$, depends on the coefficients $G_2$ and $G_2'$, whichever is greater as shown in Fig. 11. Equating the tensile strength of concrete to Eq. (4), the minimum embedment length needed for near surface mounted FRP bars to prevent concrete split failure can be expressed as:

$$L_d = \frac{G f_{FRP}}{\mu f_{ct}}$$

(6)

Equating the tensile strength of the adhesive to Eq. (5), the minimum embedment length needed for near surface mounted FRP bars to avoid epoxy split failure shall not be less than:

$$L_d = \frac{G_2}{\mu f_{epoxy}}$$

(7)

Fig. 10: Mesh dimension for a portion of a concrete beam strengthened with near surface mounted FRP bar

Fig. 11: Typical tensile stress distribution around near surface mounted bar

Typical principal tensile stress distribution is shown in Fig. 11. It should be noted that the elastic modulus of the adhesive is generally less than that of the concrete. Such a phenomenon results in a stress discontinuity at the concrete-epoxy interface as shown in Fig. 11. High tensile stresses are observed at the concrete-epoxy interface as well as at the FRP-epoxy interface.

Fig. 11: Typical tensile stress distribution around near surface mounted bar

Fig. 12: Design chart for the development length of near surface mounted FRP bars
The chart covers a wide range of possible epoxy covers and accounts for three different groove sizes. Using the proposed design chart, the coefficients $G_1$ and the greater value of either $G_2$ or $G_2'$ could be evaluated for a given groove width, $w$, and using a specified clear cover to bar diameter ratio ($C/d$). The governing development length for near surface mounted FRP bars could be predicted using the greater of Eqs. (6) and (7).

### Comparison with Experimental Results

Test results showed that the coefficient of friction between C-BAR CFRP bars and different epoxy adhesives used in this study has an average lower bound value of 0.33. Using a groove width equals to two times the diameter of the bar ($w = 2d$) and a clear cover to bar diameter ratio of one ($C/d = 1.0$), the coefficients $G_1$ and the greater of $G_2$ and $G_2'$ or for the bond specimens reported in this study are 0.65 and 1.1, respectively. The diameter of the bar is 9.525 mm. The average tensile strength of the concrete-epoxy interface are reported elsewhere [11].

### Near Surface Mounted FRP Strips

This section presents a closed-form analytical solution to predict the interfacial shear stresses for near surface mounted FRP strips. The model is validated by comparing the predicted values with test results as well as non-linear finite element modeling [12]. The proposed model is based on the combined shear-bending model for externally bonded FRP plates [13]. The model is modified to account for the double bonded area of near surface mounted strips. The model accounts also for the continuous reduction in flexural stiffness due to cracking of the concrete. Debonding of near surface mounted strips is assumed to occur as a result of high shear stress concentration at cutoff point. The derivation of the model is reported elsewhere [12]. For simply supported beams subjected to a concentrated load, $P$, at midspan, the shear stress at the strip cutoff point, $\tau$, can be expressed in terms of the effective moment of inertia, $I_{eff}$, and the thickness of the CFRP strip, $t_f$, as follows:

$$\tau = \frac{f_l}{2} \left( \frac{n}{2} \frac{P f}{I_{eff}} + \frac{n P y}{2 I_{eff}} \right)$$

where,$$\omega^2 = \frac{2G_s}{t_f t_e E_f}$$

$$n = \frac{E_f}{E_c}$$

$E_f$ is elastic modulus of the FRP strip, $E_c$ is elastic modulus of concrete, $G_s$ is the shear modulus of the adhesive, $t_a$ is the thickness of the adhesive, $l_u$ is the unbonded length of the strip; $y$ is the distance from the strip to the neutral axis of the transformed section and $I_{eff}$ is the effective moment of inertia of the transformed section.

Debonding will occur when the shear stress reaches a maximum value, which depends on the concrete properties. Premature debonding of near surface mounted CFRP strips is governed by the shear strength of the concrete. Other components of the system such as the epoxy adhesive and the CFRP strips have superior strength and adhesion properties compared to concrete. Knowing the compressive and tensile strength of concrete, the Mohr-Coulomb line, which is tangential to both Mohr’s circles for pure tension and pure compression, can be represented and the maximum critical shear stress for the pure shear circle can be expressed as:

$$\tau_{max} = \frac{f_c f_{ct}}{f_c + f_{ct}}$$

where $f_{ct}$ is the compressive strength of concrete after 28 days and $f_c$ is the tensile strength of concrete. Equating the shear strength proposed in Eq. (11) to the shear stress given in Eq. (8), debonding loads for near surface mounted CFRP strips can be determined for this specific loading case and embedment length. Other loading cases are reported in [12]. The development length is highly dependent on the dimensions of the strips, concrete properties, adhesive properties, internal steel reinforcement ratio, reinforcement configuration, type of loading, and groove width. The proposed model in Eqs. (8) and (11), can be used to estimate the development length of near surface mounted strips of any configuration as follows:

1. Use the proposed Eqs. (8) and (11) to determine the debonding load of the strip for different embedment lengths as shown in Fig. 13. The resulting curve represents a failure envelope due to debonding of the strip at cutoff point.
2. Use a cracked section analysis at sections of maximum induced normal stresses and determine the ultimate load required to rupture the strip as shown in Fig. 13.
3. Determine the development length at the intersection of the line corresponding to flexural failure of the strip with the curve representing debonding failure at cutoff point. The calculated development length will preclude brittle failure due to debonding of the strips and will ensure full composite action between the strip and concrete up to failure.
The measured debonding loads for different embedment lengths are also shown in Fig. 14. In general, the predicted loads at the maximum shear stress for each length are in a good agreement with the measured debonding loads. The predicted debonding loads for specimens B2, B3 and B4 underestimated the measured values by less than 7%. The midspan section of the test specimen was analyzed using a strain compatibility approach to predict the flexural behavior up to failure. The predicted failure load due to rupture of the CFRP strip and accounting for the tension stiffening of concrete is 75 kN. Failure of specimen B5, with an embedment length of 850 mm was due to rupture of the CFRP strip at a load level of 79 kN, which is 5% higher than the predicted value. Fig. 14 shows also that the minimum embedment length needed to rupture the CFRP strips used in this program is greater than 750 mm and less than 850 mm, which coincides with the experimental results.

Conclusions

Based on the findings of this investigation, the following conclusions can be drawn:

- The use of near surface mounted CFRP reinforcement is feasible and cost-effective for strengthening/repair of concrete structures and bridges.
- The magnitude of strength increase was influenced by the type and configuration of the CFRP materials. In general, near surface mounted CFRP strips and externally bonded CFRP sheets provided superior strength increase. The overall cost of strengthening using CFRP sheets is only 25% of that using near surface mounted CFRP strips.
- Strengthening using either near surface mounted Leadline bars or C-BAR CFRP bars provided approximately the same increase in strength. With respect to construction cost, strengthening using C-BAR CFRP bars is considerably less.
- Increasing the groove width and/or using high strength concrete, increases the resistance to concrete split failure. Using high strength adhesives and/or increasing the epoxy cover layer delays epoxy split failure for near surface mounted FRP bars.

The proposed analytical models are capable of predicting the ultimate load carrying capacity and mode of failure of concrete beams strengthened with near surface mounted FRP bars and strips. Excellent agreement was established between the predicted values using the proposed models and experimental results.

The proposed failure criteria for debonding of near surface mounted FRP bars and/or strips provided sufficient evidence and confidence in predicting debonding loads.

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References


