REHABILITATION OF CONCRETE STRUCTURES WITH FRP

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ABSTRACT

The need for repair and strengthening of deteriorated, damaged and substandard infrastructures has become an important challenge worldwide. The demand for increasingly heavier truck loads is forcing bridge owners to upgrade existing structures. In response to the growing need for concrete repair and rehabilitation, an experimental program was conducted to investigate the feasibility of using different strengthening techniques as well as different types of FRP in strengthening concrete members. Three half-scale models of a prestressed concrete bridge were constructed and tested to failure. Five different strengthening techniques were investigated including near surface mounted Leadline bars, C-BAR CFRP bars, CFRP strips as well as externally bonded CFRP sheets and strips. The dimensions of the specimens are 8.5 x 1.2 x 0.4 meters and consisted of one simple span and two overhanging cantilevers. Cost-effectiveness of each of the strengthening techniques considered in this study is presented. To evaluate bond characteristics of the most efficient techniques, a total of 24 concrete beams were constructed and tested under monotonic static loading. Design guidelines for the development length of near surface mounted CFRP bars, strips and externally bonded CFRP sheets used in strengthening concrete beams are proposed. Ultimate capacity as well as failure mechanism of concrete beams strengthened with various FRP techniques are presented.

INTRODUCTION

Rapid deterioration of infrastructure became a principal challenge facing all nations worldwide. Both serviceability and ultimate load carrying capacity of most of the existing concrete structures and bridges became inadequate to meet the users demand. An attractive solution is strengthening of the existing structures using FRP materials. High tensile strength, lightweight and corrosion resistance characteristics of FRP make it ideal for retrofitting applications. Many studies [AN et al., 1991; DE LORENZIS and NANNI, 2001; HASSAN AND RIZKALLA, 2002] have shown that significant increase in stiffness and strength can be achieved using FRP strengthening techniques. This paper summarizes a completed study, which provides experimental evidence and detailed performance of various FRP strengthening techniques. The paper also provides cost analysis for each technique to help engineers to judge the cost effectiveness of each system. Design guidelines for the development length for near surface mounted CFRP bars, strips and externally bonded CFRP sheets, used in strengthening concrete beams are proposed.

LARGE-SCALE SLAB SPECIMENS

The need to study the most appropriate strengthening technique for prestressed concrete bridges is initiated by the request of the Highway Department to upgrade a thirty years old concrete bridge in Winnipeg, Manitoba, Canada. Bridge rating analysis conducted using current AASHTO code indicated that the flexural strength of the bridge deck is not sufficient to withstand the modern truck loads. To accommodate the HSS30 AASHTO truck design load, the analysis indicated a need to increase the flexural strength by approximately 10 percent at the negative moment zone over the pier columns where the maximum shear is located. 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 half-scale models of the bridge were cast and post-tensioned. The specimens were tested in simple span with double cantilever configuration. 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 mid-span. Prior to the third test of the mid-span, the cracks resulted from testing of the two cantilevers were sealed entirely by injecting a high strength epoxy resin adhesive into the concrete to restore the slab to its original monolithic condition. The mid-span was then strengthened using FRP and tested. This paper presents only test results of the cantilevers.

Bridge Outline

The bridge was constructed in the early 1970s. The bridge was designed for AASHTO HSS20 truck design load. The bridge consists of four spans of 19.8m, 29.0m, 22.9m, and 19.8m as measured from west to east. The thickness of the bridge slab is 800 mm. The slab is supported by concrete pier columns and abutments as shown in Figure 1. The solid slab over the intermediate pier columns was post-tensioned transversely to resist the negative moments at cantilevers and the positive moment at mid-span.

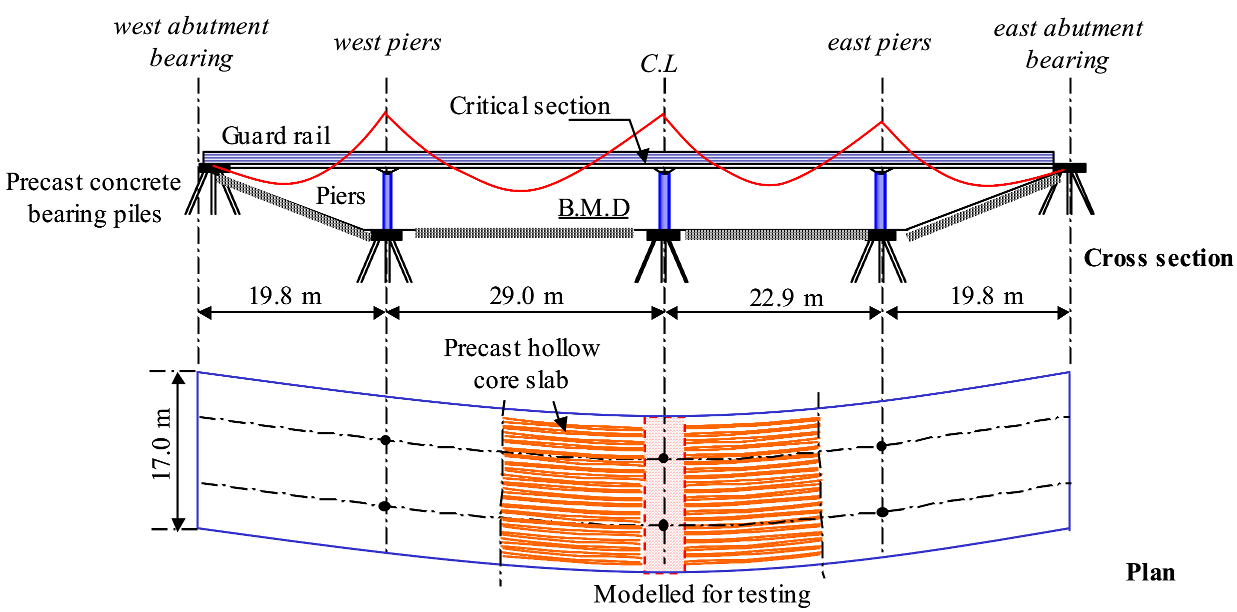

Figure 1: Schematic of bridge No. 444 in Winnipeg - Manitoba - Canada

Test Specimens

To simulate the combined effect of high flexural and shear stresses occurring over the intermediate supports of the bridge, three half-scale models of the solid slab simulating the bridge deck in the transverse direction were constructed. In the specimen, the maximum negative moment at the support of the cantilever coincides with the zone of maximum shear. The number and layout of the tendons were selected to induce the same stress level of the bridge under service loading conditions. The specimens were reinforced with four No. 15 mild steel bars on the top surface and five No. 15 mild steel bars on the bottom surface. The number of bars in the top surface was selected to represent the same mild steel reinforcement ratio in the cantilever portion of the existing bridge. Shear reinforcement consisted of U-shape stirrups spaced at 125 mm centre to centre in the cantilever span and 250 mm centre to centre in the simply supported span. Twelve 15 mm 7-wire strands were used for post-tensioning the specimens. The compressive strength of concrete after 28 days ranged between 45 and 50 MPa for the three slabs.

Strengthening Procedures

Slab S1

To investigate the benefits of embedding CFRP bars in concrete grooves, one cantilever of specimen S1 was strengthened using near surface mounted Leadline CFRP bars while the other cantilever remained unstrengthened. The Leadline bars are produced by Mitsubishi Chemicals Corporation, Japan. The bars have a modulus of elasticity of 147 GPa and an ultimate tensile strength of 1970 MPa. Based on equilibrium and compatibility conditions, six 10 mm diameter Leadline CFRP bars were used to achieve a 30 percent increase of the ultimate load carrying capacity of the slab. The location of the grooves was first marked using a chalk line. The grooves were 200 mm apart. A concrete saw was used to cut six grooves approximately 16 mm wide and 30 mm deep at the tension surface of the cantilever. The groove ends were widened to provide wedge action and hence prevents possible slip of the bars The bars were then placed in the grooves ensuring that they were completely covered with the epoxy.

Slab S2

The second specimen was used to investigate the performance of both near surface mounted and externally bonded CFRP strips in repair of concrete bridges. Six CFRP strips (50 mm wide and 1.4 mm thick) were used to achieve 30 percent increase in the ultimate capacity of the cantilever slab. The first cantilever was strengthened using externally bonded CFRP strips. The concrete substrate was prepared by grinding at the locations of the strips. The epoxy was then placed over the strips and on the concrete surface. Finally, the strips were placed on the concrete surface and gently pressed into the epoxy using a ribbed roller. The second cantilever was strengthened with near surface mounted CFRP strips inserted inside grooves. In order to insert the strips within the concrete cover layer, the strips were cut into two halves each is 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 specimens. The grooves were then injected with the epoxy adhesive to provide the necessary bond with the surrounding concrete. The strips were then placed in the grooves and they were completely covered with the epoxy. The CFRP strips are produced by S&P Clever

Reinforcement Company, Switzerland. The strips have a modulus of elasticity of 150 GPa and an ultimate tensile strength of 2000 MPa.

Slab S3

One cantilever of specimen S3 was strengthened using externally bonded CFRP sheets. The sheets are manufactured by Master Builders Technologies, Ltd., Ohio, USA. The required area of CFRP sheets was calculated to achieve 30 percent increase in flexural capacity of the cantilever slab. The sheets were applied in two plies. The first ply covered the entire width of the specimen while the second ply covered 480 mm and was centred along the width of the specimen. The second cantilever was strengthened using eight near surface mounted C-BAR CFRP bars. The bars are manufactured by Marshall Industries Composites Inc., USA and characterized by its considerably lower cost compared to Leadline bars used in specimen S1. The bars have a modulus of elasticity of 111 GPa and an ultimate tensile strength of 1918 MPa. The bars were sandblasted first to enhance their bond with the epoxy adhesive. The bars were then inserted inside grooves cut at the top surface of the concrete.

Testing Scheme

The slabs were tested under static loading conditions using a uniform line-load. A closed-loop MTS, 5000 kN, testing machine was used to apply the load using stroke control mode. To prevent possible damage of the other cantilever during the first test, an intermediate support was provided as shown in Figure 2. Neoprene pads were placed between the loading beam and the slab to simulate the contact surface of a truck tire and to avoid local crushing of the concrete.

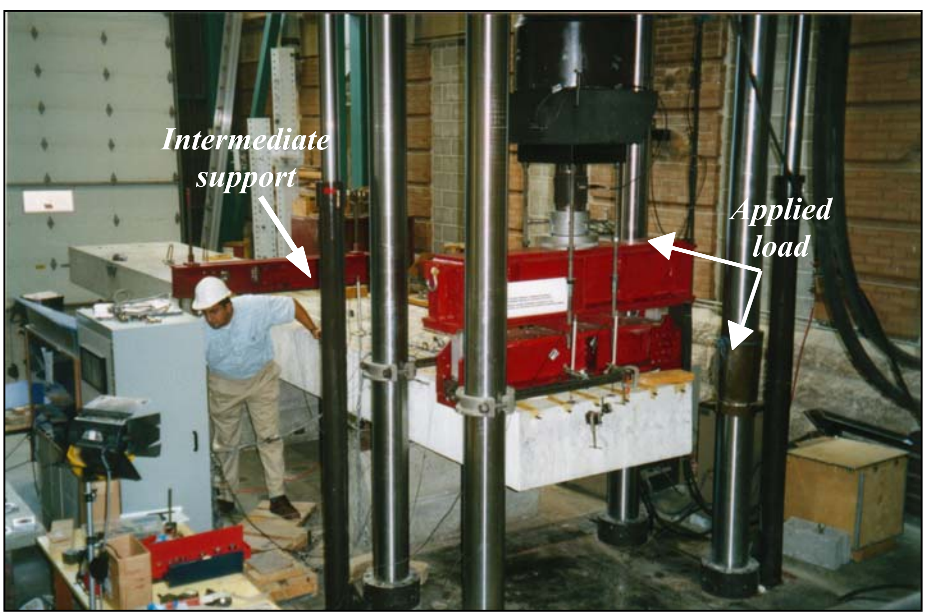

Figure 2: Test set-up for cantilever tests

Test Results and Discussion

The load-deflection behaviour of the cantilevers strengthened with near surface mounted Leadline bars, CFRP strips and C-Bars, C2, C4 and C6, respectively are compared to the unstrengthened specimen, C1, as shown in Figure 3. Test results indicate identical behaviour for all the specimens until cracking occurred at a load level of 180 kN for the unstrengthened cantilever and 190 kN for the strengthened cantilevers. After cracking, a non-linear behaviour was observed up to failure. The measured stiffnesses for the strengthened specimens are higher due to the addition of the CFRP reinforcement. The presence of CFRP reinforcement precluded the flattening of the load-deflection curve, which was clear in the control specimen between 440 kN and 466 kN. Prior to yielding of the steel reinforcement, the stiffnesses of all strengthened cantilevers are almost the same and are 1.5 times higher than the stiffness of the unstrengthened cantilever. The presence of the CFRP reinforcement provided constrains to the cracks to open. Therefore, the deflections were reduced and consequently appeared to increase the stiffness. After yielding of the tension steel reinforcement at a load level of 440 kN, the stiffness of the cantilever specimen strengthened with Leadline bars, specimen C2, was three times higher than that of the unstrengthened one. Using C-BAR CFRP bars instead of Leadline bars increased the stiffness by an extra 20 percent. Using near surface mounted CFRP strips yielded a supreme stiffness increase by an extra 35 percent in comparison to Leadline bars.

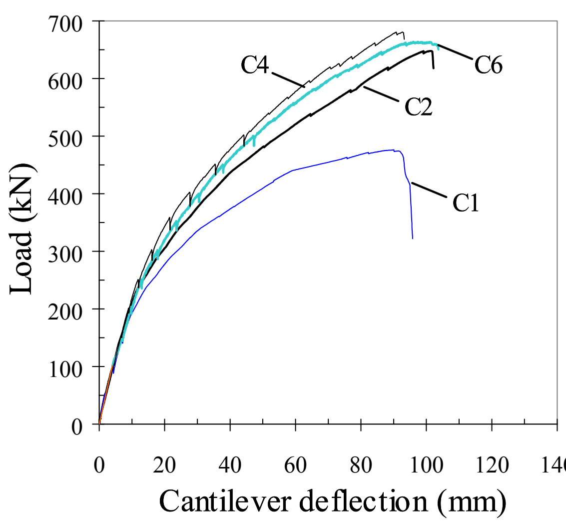

Figure 3: Load-deflection behaviour of cantilever specimens strengthened with near surface mounted CFRP reinforcement

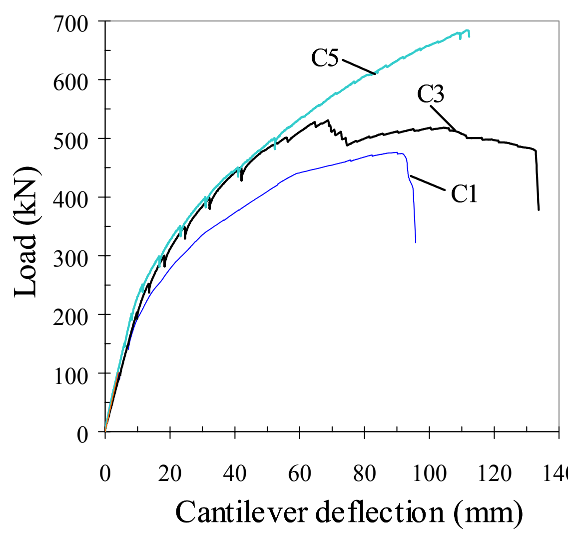

Figure 4: Load-deflection behaviour of cantilever specimens strengthened with externally bonded CFRP reinforcement

Figure 4 shows the load-deflection behaviour of cantilever specimens, C3 and C5, strengthened with externally bonded CFRP strips and sheets, respectively. The behaviour of the control specimen 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 behaviour was observed for specimens C3 and C5 until 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/epoxy interface 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. The load was dropped to a load level corresponding to the yield strength of the cross-section until crushing of concrete occurred. The observed mode of failure for all other cantilever specimens occurred due to crushing of the concrete in the compression zone at the face of the support.

In general, CFRP-strengthened cantilever specimens showed considerable enhancement of strength. The control specimen exhibited plastic failure with concrete failing in compression and yielding of the steel. The failure load of the control specimen was 476 kN. Strengthening the specimen using near surface mounted Leadline bars increased the strength by 36 percent in comparison to the design value of 30 percent. Using C-BAR CFRP bars instead of Leadline bars increased the strength by 39 percent. The cantilever specimen strengthened with near surface mounted CFRP strips showed the highest increase in strength by 43 percent. Using the same area of CFRP strips as externally bonded reinforcement increased the strength by only 11 percent 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 percent.

Cost Analysis

One of the prime objectives of this study is to provide a cost-effective analysis for each strengthening technique considered in this study. Using an efficiency scale (E) defined by Eq. (1), the efficiency of each technique was evaluated as shown in Figure 5.

$$E = \frac{\%\, Increase\ in\ strength}{Construction\ cost\ \ in\ USD}\ x\ 100 \tag{1}$$

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

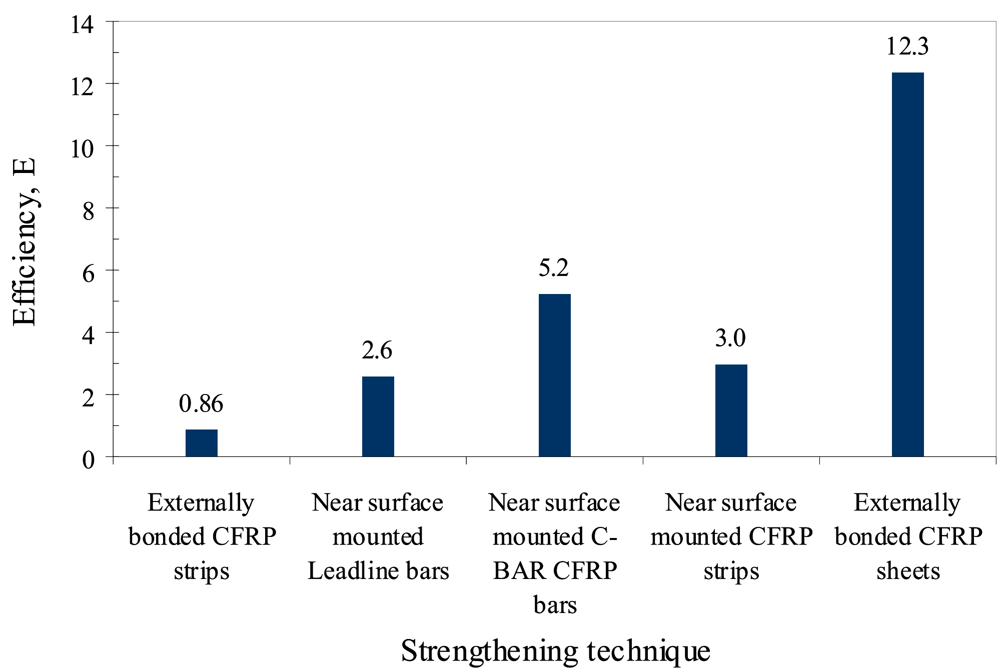

Figure 5. Efficiency of various strengthening techniques

The construction cost accounts for the cost of materials, equipment needed during construction and labor. Test results indicated 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 25 percent 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 construction cost, strengthening using C-BAR CFRP bars is 50 percent less. The results also show that strengthening using externally bonded strips is the least efficient technique in terms of strength improvement and construction cost.

BOND SPECIMENS

Based on test results of large-scale slabs, three different strengthening techniques have proven their efficiency in terms of strength increase and total cost of construction. These techniques are:

a- Near surface mounted C-BAR CFRP bars;
b- Near surface mounted CFRP strips; and
c- Externally bonded CFRP sheets

To characterize the bond mechanism and load transfer between CFRP reinforcement and concrete for these techniques, a total of 24 concrete T-beams with a total length of 2.7 m and a depth of 300 mm were tested. The beams were simply supported with a 2.5 m span. The beams were tested under a concentrated load acting at the middle of the specimen. The arrangement of the bottom reinforcement was selected to ensure that the failure of the strengthened beams would always occur at the mid-span section and not at the section where the FRP reinforcement is terminated as shown in Figure 6.

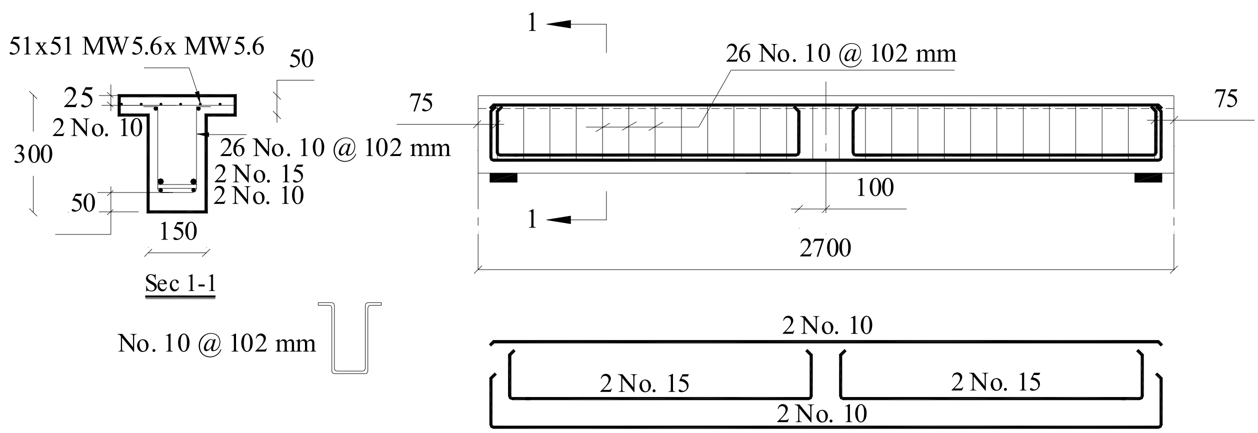

Figure 6. Reinforcement details of the bond specimens

Three series of beam specimens designated as A, B, and C were cast, respectively. In series A, the beams were strengthened using near surface mounted C-BAR CFRP bars. The performance of two different epoxy adhesives used for bonding the bars was investigated (Duralith gel and Kemko 040). In series B, the beams were strengthened using near surface mounted CFRP strips. In series C, the beams were strengthened using externally bonded CFRP sheets. With the maximum moment occurring at the mid-span section of the beam, two types of failure were expected: (1) bond failure; (2) rupture of FRP. Specimens were adequately designed to avoid concrete crushing and premature failure due to shear. In case of bond failure, the bond length of the FRP reinforcement was increased in the following 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 each strengthening technique was achieved.

Figure 7 shows the tensile strain in the CFRP reinforcement at ultimate for different embedment lengths used in this study. The measured strain values suggest the following three mechanisms as the embedment length increases: (1) For small embedment lengths (less than 250 mm) debonding of the CFRP reinforcement occurred before yielding of the internal steel reinforcement without significant development of the bond. For these small embedment lengths, the failure is due to immediate debonding, "Destressed mechanism" and the beams behaved as conventional concrete beams reinforced with steel bars; (2) "bond development mechanism", where the strains at failure are increasing linearly with the increase of the embedment length. For this range of embedment lengths, increasing the bond length results in a considerable enhancement in the ultimate load carrying capacity of the beams; (3) "full composite mechanism", where the CFRP behaved in a full composite action with beam. For these relatively long embedment lengths, increasing the embedment length will not provide extra strength to the retrofitted beam.

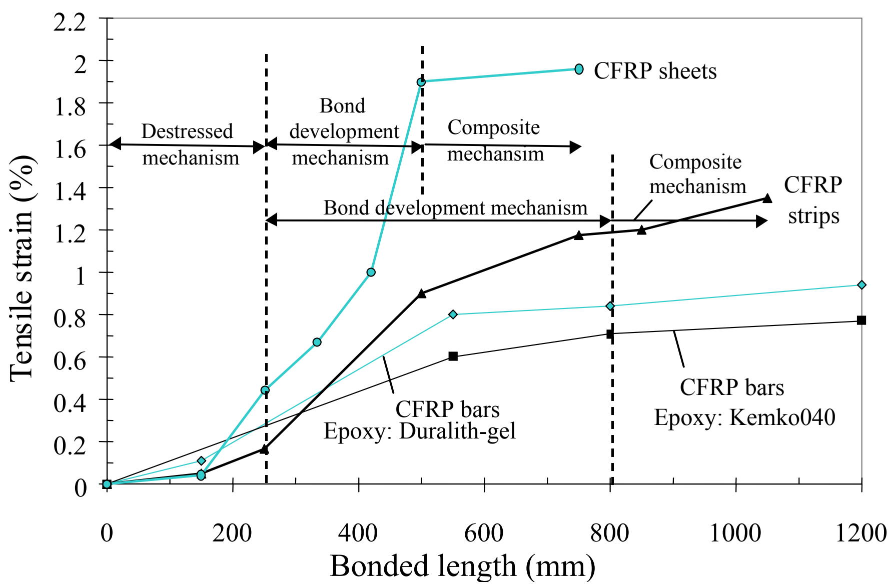

Figure 7. Maximum tensile strain in the CFRP reinforcement vs. bonded length

ANALYTICAL MODEL

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 modelling. The proposed model is based on the combined shear-bending model introduced by MALEK et al. (1998) for externally bonded FRP plates. 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 (HASSAN and RIZKALLA 2002). For simply supported beams subjected to a concentrated load, P, at midspan, the shear stress at the strip cutoff point, τ, can be expressed in terms of the effective moment of inertia, I_{eff}, and the thickness of the CFRP strip, t_f, as follows:

$$\tau = \frac{t_f}{2}\left[\frac{n\,P\,l_o\,y}{2\,I_{eff}}\,\omega + \frac{nPy}{2\,I_{eff}}\right] \tag{2}$$

where,

$$\omega^2 = \frac{2G_a}{t_a t_f E_f} \tag{3}$$

$$n = \frac{E_f}{E_c} \tag{4}$$

E_f is elastic modulus of the FRP strip, E_c is elastic modulus of concrete, G_a is the shear modulus of the adhesive, t_a is the thickness of the adhesive, l_o 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:

$$\tau_{max} = \frac{f'_c\, f_{ct}}{f'_c + f_{ct}} \tag{5}$$

where f'_c is the compressive strength of concrete after 28 days and f_{ct} is the tensile strength of concrete. Equating the shear strength proposed in Eq. (5) to the shear stress given in Eq. (2), 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 (HASSAN and RIZKALLA 2002).

CONCLUSIONS

1. The use of near surface mounted CFRP reinforcement is feasible and effective for strengthening/repair of concrete structures.
2. Strengthening using externally bonded CFRP strips provided the least increase in strength by 11 percent due to peeling of the strips from the concrete surface. Using the same amount of strips but as near surface mounted reinforcement enhanced the ultimate load carrying capacity by 43 percent.
3. Full composite action was observed between near surface mounted CFRP reinforcement and the concrete and no slip was observed throughout the tests.
4. Ultimate loads of concrete beams strengthened with CFRP reinforcement were found to increase with longer bond length. For every strengthening technique, there is a certain length beyond which no further increase in beam strength can be obtained.
5. Using epoxy adhesives that were commonly used for bonding steel rebars into concrete proved its efficiency in bonding near surface mounted CFRP bars to the surrounding concrete.
6. Rupture of C-BAR CFRP bars is not likely to occur regardless of the embedment length or the type of the epoxy adhesive used. The maximum allowable strain in the bars should be limited to 0.7-0.8 percent depending on the type of epoxy adhesive.
7. The development length of near surface mounted C-BAR CFRP bars should not be less than 800 mm for 10 mm diameter bars. The development length of near surface mounted CFRP strips (25x1.2 mm) should not be less than 850 mm. The development length of CFRP sheets bonded to the soffit of concrete specimens should not be less than 500 mm.
8. The proposed analytical model is capable of predicting the interfacial shear stress at the strip cutoff point, ultimate load carrying capacity and mode of failure of concrete beams strengthened with near surface mounted CFRP strips.

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