COST-EFFECTIVENESS OF VARIOUS FRP REPAIR TECHNIQUES FOR RC STRUCTURES

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ABSTRACT
Advances in the fields of polymers and composites have resulted in a major development of high strength fibre reinforced polymers (FRP). These materials offer great potential for cost-effective retrofitting of concrete structures.

In response to the growing need for concrete repair and rehabilitation, an experimental program was conducted at the University of Manitoba in Winnipeg, Manitoba, Canada, to investigate the feasibility of using different strengthening techniques as well as different types of FRP in strengthening post-tensioned bridge slabs. Half-scale models of a reinforced concrete bridge were constructed and tested to failure. The model, with dimensions of 8.5 x 1.2 x 0.4 meters, consisted of one simple span and two overhanging cantilevers. Each specimen was tested at three different locations. The first and second tests were performed on the two cantilevers with the load applied at the edge of each cantilever, while the third test was conducted on the mid-span.

Five different strengthening techniques were investigated including near surface mounted Leadline bars, C-Bars, CFRP strips and externally bonded CFRP sheets and strips. Ultimate capacity, failure mechanism and cost analysis of various strengthening techniques for concrete bridges are presented.

INTRODUCTION
In an aggressive environment, concrete may be vulnerable to chemical actions such as carbonation and chloride contamination which breaks down the alkaline barrier in the cement matrix. Consequently, the steel reinforcement in concrete structures becomes susceptible to rusting and corrosion. Such a phenomena leads to further cracking and spalling of the concrete and even delamination of the concrete at the reinforcement level under more severe conditions. In the United States, nearly one third of the nation’s 581,000 bridges were graded structurally deficient or functionally obsolete by the FHWA [Us DOT,1997]. A large number of these deficient bridges are reinforced or prestressed concrete structures, and are in urgent need of repair and strengthening. In the United Kingdom, over 10,000 concrete bridges need structural attention. In Europe, it is estimated that the repair of structures due to corrosion of rebars in reinforced concrete structures costs over $600 million annually [Tann and Delpark, 1999].

A possible solution to combat reinforcement corrosion is the use of non-corrosive materials to replace conventional steel bars. High tensile strength, lightweight, adequate ductility and corrosion resistance characteristics make FRP ideal for such applications. FRP also provides cost effectiveness and a practical technique for the repair and strengthening of structures and bridges using externally bonded sheets or prefabricated laminates.

Manitoba Highways and Government Services is considering upgrading a concrete bridge constructed thirty years ago using near surface mounted FRP reinforcement. The analysis conducted using current
codes indicated that the flexural strength of the bridge deck is not sufficient to withstand modern truck loads. To accommodate the HSS30 AASHTO truck design load, the analysis indicates a need of approximately 10 percent flexural strengthening of the existing bridge slab at the negative moment zone over the pier columns. Costs to demolish the existing structure, construct a replacement, and carry out required embankment and pavement modifications was estimated to exceed the two million dollar range. Costs will be lessened if the existing structure can be effectively rehabilitated.

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, an experimental program has been undertaken to investigate the feasibility of using near surface mounted CFRP reinforcement to increase the flexural capacity in the negative moment zones.

Three half-scale models of bridge No. 444 on Highways 101 and 59 in Winnipeg, Manitoba, Canada were cast and post-tensioned. The specimens were tested in simple span with double cantilever configuration. Each specimen was tested at three different locations. The first and second tests are performed on the two cantilevers with the load applied at the edge of each cantilever, while the third test was conducted on the mid-span. Prior to the third test on the mid-span, the cracks resulting from the first two tests 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 and tested. The significance of this project is the strengthening is required to increase the flexural capacity at the location of the maximum shear stresses typically occurring at the maximum negative moment section of cantilevers and continuous beams.

The overall aim of the research is to gain experimental evidence and insight of the performance of various FRP strengthening techniques, recommend innovative and cost-effective methods, and provide complete design guidelines for use in retrofitting projects.

**BRIDGE OUTLINE**

The bridge was constructed in the early 1970s. The bridge is designed for AASHTO HSS20 truck design load. The bridge is located at the intersection of Highways 101 and 59 in Winnipeg, Manitoba, Canada. The bridge consists of four spans of 19.8 m, 29.0 m, 22.9 m, and 19.8 m as measured from west to east. The thickness of the bridge slab is 800 mm, cast in place, partially voided, post-tensioned slab and supported by concrete pier columns and abutments as shown in Figure 1.

**TEST SPECIMENS**

To simulate the combined effect of supports of the bridge, three half-span transverse direction were constructed of the cantilever coincides with the were selected to have the same induced critical bending moments for the bridge (SAP2000). The loss of the prestress computer software CONCISE (Co developed by the Canadian Prestress Council). The specimens were reinforced with steel bars on the bottom surface. The actual mild steel reinforcement of the reinforcement consisted of U-shape and 250 mm centre to centre in the shown in Figure 2.

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

Figure 2.
is not sufficient to withstand modern truck design load, the analysis indicates a need of existing bridge slab at the negative moment structure, construct a replacement, and as was estimated to exceed the two million be effectively rehabilitated.

mounted FRP reinforcement for flexural near stresses, an experimental program has surface mounted CFRP reinforcement to

01 and 59 in Winnipeg, Manitoba, Canada ed in simple span with double cantilever at locations. The first and second tests are the edge of each cantilever, while the third on the mid-span, the cracks resulting from high strength epoxy resin adhesive into the tension. The mid-span was then strengthened thening is required to increase the flexural pically occurring at the maximum negative

evidence and insight of the performance of live and cost-effective methods, and provide

g is designed for AASHTO HSS20 truck ghways 01 and 59 in Winnipeg, Manitoba, m, 22.9 m, and 19.8 m as measured from cast in place, partially voided, post-tensioned shown in Figure 1.

east piers east abutment bearing

Cross section

Plan

Winipeg - Manitoba - Canada

The two main reasons for strengthening are:
- Upgrade the section capacity in the negative moment zone above the piers to resist heavier truck loads (HSS30) instead of HSS20 AASHTO truck design load.
- Limit the cracking under the increased service load level (HSS30).

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 point of maximum shear. The number and layout of the tendons were selected to have the same induced stress level of the bridge under service loading conditions. The critical bending moments for the bridge were evaluated based on a linear elastic finite element analysis (SAP2000). The loss of the prestressing force was predicted according to the CSA code using a computer software CONCISE (Computer Analysis and Design for Precast Prestressed Concrete) developed by the Canadian Prestressed Concrete Institute (CPCI).

The specimens were reinforced with four 15M mild steel bars on the top surface and five 15M mild steel bars on the bottom surface. The number of bars in the top surface was chosen to represent the actual 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. Reinforcement details of the specimens are shown in Figure 2.

Figure 2. Reinforcement details of test specimens
Bursting reinforcement was provided using six looped pairs of 10M bars spaced at 70 mm centre to centre. Twelve 15 mm 7-wire strands were used for post-tensioning the specimens.

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 in 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 as shown in Figure 3. The groove ends were widened to provide wedge action and hence prevents any possible slip of the bars. The bars were then placed in the grooves ensuring that they were completely covered with the epoxy as shown in Figure 4.

Figure 3. Cutting grooves for embedded CFRP bars

Figure 4. CFRP bars inserted in epoxy

After completion of the first two tests on both cantilevers, the resulting cracks were injected. A temporary seal [SCB Concrex 1446] was applied at the backside of the slab to prevent the injecting adhesive from running out. Entry ports were placed at the cracked surfaces as shown in Figure 5. Two metering pumps were used to drive the two components (resin and hardener) of a fast-setting epoxy adhesive [SCB Concrex 1360] to a special mixing head. The adhesive was mixed at the nozzle and injected through a special gasket, which prevented leakage on the face of the concrete. The adhesive was pumped into the first entry port until it began to show at the next adjacent port. The first injected port was then plugged and injection was resumed at the second port. This procedure was followed until all major cracks were filled as shown in Figure 6.

Figure 5. Placing entry ports

Figure 6. Injecting the cracks

Based on equilibrium and compatibility conditions, six 10 mm diameter Leadline CFRP bars were used to achieve a 30 percent increase in the ultimate load carrying capacity of the slab.

Slab S2: The second specimen surface mounted and externally bonded surface mounted CFRP strip. Technical University in Munich, existed between the strip and the concrete surface. To compare the behaviour (50 mm wide and 1.4 mm thick) of the cantilever slab. The first cantilever concrete substrate was prepared and pre-cut prior to installation. The strips were then placed on rollers. Finally, the strips were placed on rollers.

The second cantilever and the mid order to insert the strips with the concrete slab to the tension surface of the specimen. The concrete did not provide the necessary bond with the pre-cut strips placed in the grooves ensuring that the CFRP strips are pre-cut prior to installation. The strips were then placed on rollers. The same procedures, in terms of testing both cantilevers, the further testing of the mid-span, th previously for specimen S1. 18 CF to centre were used to achieve a slab. The same procedures, in terms of testing both cantilevers, the further testing of the mid-span, th previously for specimen S1. 18 CF to centre were used to achieve a slab. The same procedures, in terms of testing both cantilevers, the further testing of the mid-span, th previously for specimen S1.

Slab S3: A widespread method fit onto the concrete surface. This n detailing problems and design as to investigate the effectiveness of the.
Based on equilibrium and compatibility conditions, ten 10 mm diameter Leadline CFRP bars were used to achieve a 30 percent increase in the ultimate capacity of the simply supported slab. The grooves were 120 mm apart. The same procedures as described before for cutting the grooves and placing the bars were applied.

Slab S2: The second specimen was intended to investigate the potential application of both near surface mounted and externally bonded CFRP strips in repair of concrete bridges. The performance of near surface mounted CFRP strips has been recently tested by Blaschko and Zilch, 1999 at the Technical University in Munich. Test results positively determined that a good and uniform bond existed between the strip and the concrete. The high tensile strength of the strip was fully utilized up to failure. To compare the behaviour of the second specimen with that of the first one, six CFRP strips (50 mm wide and 1.4 mm thick) were used to achieve a 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 strips were measured and pre-cut prior to installation. 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 and the mid-span were strengthened with near surface mounted CFRP strips. 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 specimen. The grooves were then injected with the epoxy adhesive to provide the necessary bond with the surrounding concrete as shown in Figure 7a. The strips were then placed in the grooves ensuring that they were completely covered with the epoxy as shown in Figure 7b. 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.

After testing both cantilevers, the areas above the supports were substantially cracked. To enable further testing of the mid-span, the cracks were injected following the same procedures as described previously for specimen S1. 18 CFRP strips (25 mm wide and 1.4 mm thick) spaced by 66 mm centre to centre were used to achieve a 30 percent increase in the ultimate capacity of the simply supported slab. The same procedures, in terms of cutting the grooves, placing the CFRP strips were applied.

Slab S3: A widespread method for the rehabilitation of concrete structures is to bond CFRP sheets onto the concrete surface. This method can be seen as state-of-the-art technique in spite of some detailing problems and design aspects which could influence the failure modes. To experimentally investigate the effectiveness of this strengthening method compared to previous tested techniques, one
cantilever of specimen S3 as well as the simply supported span were 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 a 30 percent increase in flexural capacity of the cantilever slab. For the first cantilever, the sheets were applied in two plies. The first ply covered the whole width of the specimen (1200 mm) while the second ply covered 480 mm and was centred along the width of the specimen. Installation procedures are illustrated in Figure 8.

![Figure 8. Installation procedures for CFRP sheets](image)

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 low 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.

The simply supported span was strengthened with externally bonded CFRP sheets after injecting the cracks resulting from cantilever tests. The same technique used for specimens S1 and S2 for the injection process was followed. Three plies of CFRP sheets were used to achieve a 30 percent increase in flexural capacity. The first two plies covered the whole width of the specimen, while the third ply covered 400 mm and was centred along the width of the specimen. Detailed information about the tested specimens is provided in Table 1. The designation of the tested specimens have the first letters either C, or SS refers to Cantilever or Simply Supported specimens, respectively.

<table>
<thead>
<tr>
<th>Slab No.</th>
<th>Specimen</th>
<th>Strengthening technique</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C1</td>
<td>Control specimen</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>6 near surface mounted Leadline bars</td>
</tr>
<tr>
<td></td>
<td>SS1</td>
<td>10 near surface mounted Leadline bars</td>
</tr>
<tr>
<td>2</td>
<td>C3</td>
<td>6 externally bonded CFRP strips (50x1.4mm)</td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>12 near surface mounted CFRP strips (25x1.4mm)</td>
</tr>
<tr>
<td></td>
<td>SS2</td>
<td>18 near surface mounted CFRP strips (25x1.4mm)</td>
</tr>
<tr>
<td>3</td>
<td>C5</td>
<td>2 plies of CFRP sheets</td>
</tr>
<tr>
<td></td>
<td>C6</td>
<td>8 near surface mounted C-BAR CFRP bars</td>
</tr>
<tr>
<td></td>
<td>SS3</td>
<td>3 plies of CFRP sheets</td>
</tr>
</tbody>
</table>

Table 1. Details of test specimens

TESTING SCHEME
The slab was tested under static loading conditions using a uniform line-load acting on a width equivalent to the width of tire contact patch according to AASHTO HSS30 design vehicle. A closed-

loop MTS 5000 kN (1.2 million pc control mode with a rate of 0.5 mm beam and the slab to simulate the tires. For the cantilever tests, the load was prevented possible damage of the slab provided as shown in Figure 9.

![Figure 9. Schematic view](image)

For the mid-span tests, the slab was simply each end with the load applied at the cen

![Figure 10. Schematic view](image)
loop MTS 5000 kN (1.2 million pounds) 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 tires of the truck and to avoid local crushing of the concrete. For the cantilever tests, the load was applied at a distance of 325 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 Figure 9.

(1) Elastomeric bearings (Neoprene pads) 400x400x4
(2) Steel plate 200x20 (Length=1200 mm)
(3) HSS 76x50x5 (Length=230 mm)
(4) HSS 100x100x5 (Length=1270 mm)
(5) Prestressed DIWIDAG bar (Diameter=25.4 mm, Prestressing force=178 kN)
(6) HSS 100x100x5 (Length=1200 mm)

Figure 9. Schematic of test set-up for cantilever tests

For the mid-span tests, the slab was simply supported with a 4.90 m span and a 1.80 m projection from each end with the load applied at the centre of the slab as shown in Figure 10.

(1) Elastomeric bearings 400x400x4

Figure 10 Schematic of test set-up for simply supported tests
TEST RESULTS AND DISCUSSION
Cantilever Tests
The load-deflection behaviour of cantilever specimens strengthened with near surface mounted CFRP reinforcement C2, C4 and C6 are compared to the unstrengthened cantilever, C1, is shown in Figure 11.

![Figure 11. Load-deflection behaviour of cantilever specimens strengthened with near surface mounted CFRP reinforcement](image1)

Identical behaviour was observed for all the specimens until cracking occurred at a load level of 180 kN for the unstrengthened cantilever and 190 kN for strengthened cantilevers. After cracking, a nonlinear 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 were almost the same and were 1.5 times higher than the stiffness of the unstrengthened cantilever. The higher stiffness of the strengthened cantilevers after cracking is attributed to the high elastic modulus of the CFRP reinforcement. 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 20 percent. However, using near surface mounted CFRP strips yielded a supreme stiffness increase by 35 percent. For the control specimen, the increase in the applied load was negligible after yielding of the steel reinforcement. For strengthened cantilevers, the load resistance increased until the concrete was crushed in the compression zone. This is attributed to the linear stress-strain behaviour of the CFRP reinforcement up to failure.

Figure 12 shows the load-deflection behaviour of cantilever specimens, C3 and C5, strengthened with externally bonded CFRP reinforcement. The behaviour of the control specimen was also shown for comparison. The figure indicates that the stiffness, ductility and strength 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.

Figure 12. Load-deflection with externally for specimen C3, as shown in Figure 13.

![Figure 13. Initial cracking](image2)

Upon additional loading the cracks co delamination occurred resulting in peel of the steel reinforcement until crushing.

The observed mode of failure for all concrete at the face of the support, w of failure, the bottom steel bars and bottom steel bars. Flexural shear cracks CFRP-strengthened cantilever specimen exhibited plastic failure with load of the control specimen was 4. Leadline bars increased the strength. Using C-BAR CFRP bars instead of specimen strengthened with externally percent, due to peeling of the strips f
Cantilever specimens strengthened with near surface mounted CFRP (C1) and strengthened cantilever, C3, is shown in Figure 12. Load-deflection behaviour of cantilever specimens strengthened with externally bonded CFRP reinforcement was observed at a load level of 400 kN for specimen C3, as shown in Figure 13.

Initial cracking in the concrete substrate at the anchorage zone was observed at a load level of 400 kN for specimen C3, as shown in Figure 13. 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 then dropped to the yielding load of the steel reinforcement until crushing of concrete occurred.

The observed mode of failure for all cantilever specimens was crushing of the bottom surface of the concrete at the face of the support, where maximum compressive stresses were induced. At the onset of failure, the bottom steel bars and the steel stirrups were exposed, followed by buckling of the bottom steel bars. Flexural shear cracks were also observed before failure of the specimens.

Cantilever specimens strengthened with near surface mounted CFRP strips showed considerable enhancement of strength. The control specimen exhibited plastic failure with concrete failing in compression and steel yielding. 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 enhanced the strength by 39 percent. The cantilever specimen strengthened with externally bonded CFRP strips showed a minor increase in strength by 11 percent, due to peeling of the strips from the concrete surface. Using the same amount of CFRP strips...
but inserted inside grooves increased the ultimate load carrying capacity by 43 percent. Furthermore, using externally bonded CFRP sheets yielded superior strength enhancement by 44 percent. Experimental results for cantilever specimens are summarized in Table 2. Typical failure due to crushing of concrete is shown in Figure 14.

Table 2. Experimental results of cantilever specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strengthening technique</th>
<th>Cracking load (kN)</th>
<th>Deflection at cracking (mm)</th>
<th>Ultimate load (kN)</th>
<th>Deflection at ultimate (mm)</th>
<th>% Increase in capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>N.A (Control)</td>
<td>180</td>
<td>9.1</td>
<td>476</td>
<td>92.5</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>Near surface mounted</td>
<td>189</td>
<td>8.3</td>
<td>647</td>
<td>102</td>
<td>36</td>
</tr>
<tr>
<td>C3</td>
<td>External bonded CFRP</td>
<td>192</td>
<td>9.2</td>
<td>530*</td>
<td>39</td>
<td>11</td>
</tr>
<tr>
<td>C4</td>
<td>Near surface mounted</td>
<td>187</td>
<td>8.5</td>
<td>680</td>
<td>93</td>
<td>43</td>
</tr>
<tr>
<td>C5</td>
<td>External bonded CFRP</td>
<td>194</td>
<td>9.1</td>
<td>683</td>
<td>112</td>
<td>44</td>
</tr>
<tr>
<td>C6</td>
<td>Near surface mounted C-</td>
<td>197</td>
<td>8.3</td>
<td>663</td>
<td>100</td>
<td>39</td>
</tr>
</tbody>
</table>

* Specimen C3 failed due to delamination of CFRP strips, followed by crushing of concrete

All other cantilever specimens failed due to crushing of concrete

Figure 14. Typical failure due to crushing of concrete for cantilever specimens

In general, strengthening using near surface mounted CFRP reinforcement decrease the crack width by a factor ranged between two and three. Comparable crack width was observed for specimens C3 and C5 until separation of the CFRP strips took place at 500 kN, beyond which the crack width for specimens C3 increased significantly, with no increase in the applied load. This is attributed to the sudden transfer of tensile stresses carried by the CFRP strips to the steel reinforcement, which was already yielded. Such a phenomenon resulted in an enormous increase in the strains and consequently the crack width.

Simply Supported Tests
The load-deflection behaviour of the simply supported specimens strengthened with CFRP reinforcement is shown in Figure 15. To investigate different strengthening techniques, it was decided not to test control specimen for the mid-span and to rely completely on the results obtained from non-linear finite element analysis (Hassan et al., 2000)

The strengthened specimens exhibited unstrengthened specimen. Stiffnesses are of the CFRP reinforcement. Figure 15 is an increase in stiffness in the elastic range of slabs were significantly enhanced in specimen. Identical behaviour was observed for the unstrengthened slab and showed traditional non-linearities at crack yielding of the bottom steel reinforcements same and were 1.5 times higher than the showed comparable stiffnesses up to full level of 660 kN, the stiffness of specimen unstrengthened slab. Using externally b percent. Traditional flexural failure due observed for all three specimens.
tly crack development was observed at the control section. Crack spacings were similar to those of the control slab. The controls were positioned in the middle of the slab and were spaced at the same distance from the supports. The load capacity of the specimens was determined using the maximum load at which the cracks did not propagate. The specimens were instrumented with strain gauges and displacement transducers to measure the crack widths and deflections at various points along the length of the slab. The results showed that the load capacity of the specimens was significantly higher than that of the control slab. The load-deflection curves for the specimens are shown in Figure 15.

The strengthened slabs exhibited plastic failure with concrete failing in compression and steel yielding at a load level of 741 kN (based on finite element analysis). Strengthening the slab using CFRP strips increased the load capacity by 34 percent in comparison to the control slab. Further strengthening using CFRP sheets enhanced the strength by 44 percent. Experimental results of simply supported specimens are given in Table 3.
Table 3. Experimental results of simply supported specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strengthening technique</th>
<th>Cracking load (kN)</th>
<th>Deflection at cracking (mm)</th>
<th>Ultimate load (kN)</th>
<th>Deflection at ultimate (mm)</th>
<th>% Increase in capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS1</td>
<td>Near surface mounted Leadline bars</td>
<td>350</td>
<td>6.6</td>
<td>993</td>
<td>62.6</td>
<td>34*</td>
</tr>
<tr>
<td>SS2</td>
<td>Near surface mounted CFRP strips</td>
<td>370</td>
<td>6.6</td>
<td>1022</td>
<td>62.0</td>
<td>38*</td>
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<tr>
<td>SS3</td>
<td>Externally bonded CFRP sheets</td>
<td>362</td>
<td>6.8</td>
<td>1115</td>
<td>67.2</td>
<td>50*</td>
</tr>
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</table>

* Ultimate load of unstrengthened specimen was based on non-linear FEM analysis (Hassan et al., 2000)
- Failure of all specimens was due to crushing of the concrete

COST ANALYSIS

The prime objective of this study is to provide better understanding of the efficiency of each strengthening technique adopted. The efficiency needs to be defined in terms of strength enhancement and overall cost associated with it. The percentage increase in capacity, material cost, and overall cost associated with each strengthening technique are shown in Figure 16.

CONCLUSIONS

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

1. The use of near surface mounted CFRP strengthening/repair of concrete members is effective.
2. Using externally bonded CFRP sheets is simple and not supported tests.
3. Strengthening using externally bonded CFRP increased the load capacity compared to 3 percent when embedded in the concrete.
4. Strengthening using externally bonded CFRP significantly increased the load capacity in terms of strength, moment, and strain.
5. The crack width of unstrengthened specimens is smaller than that of strengthened specimens.
6. Full composite action was observed between the steel bar and the concrete, and no slip was observed.

REFERENCES


The figure indicates that using either near surface mounted CFRP strips or externally bonded CFRP sheets yielded the maximum increase in strength. However, the overall cost of strengthening using externally bonded CFRP sheets is only 25 percent of that using near surface mounted CFRP strips. Strengthening using either near surface mounted Leadline bars or C-BAR CFRP bars yielded approximately the same increase in ultimate load carrying capacity. With respect to cost, strengthening using C-BAR CFRP bars is 50 percent less. The results also show that strengthening using externally bonded CFRP strips is less efficient compared to other techniques in terms of strength improvement and overall cost of construction.
CONCLUSIONS
Based on the findings of this investigation, the following conclusions can be drawn:
1. The use of near surface mounted CFRP reinforcement is feasible and effective for strengthening/repair of concrete members especially in negative moment areas.
2. Using externally bonded CFRP sheets yielded superior strength increase for both cantilever and simply supported tests.
3. Strengthening using externally bonded CFRP strips increased the ultimate capacity by 11 percent compared to 43 percent when embedded inside the concrete.
4. Strengthening using externally bonded CFRP sheets has proven to be the most effective method of strengthening in terms of strength, material cost and construction time especially in positive moment areas.
5. The crack width of unstrengthened specimens was two to three times larger than that of strengthened specimens.
6. Full composite action was observed between near surface mounted CFRP reinforcement (bars or strips) and the concrete and no slip was observed throughout the tests.

REFERENCES