Experimental Investigation for Moment Redistribution in Continuous RC Beams Top Strengthened with CFRP and Steel Plates

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Abstract. An experimental study was conducted on eight T-section continuous reinforced concrete beams top strengthened with CFRP and steel plates. The beams had a constant cross section and flexural steel reinforcement which was designed to prevent brittle failure due to crushing of concrete before steel yielding. The plates were installed on both sides of the slab above the beam web as near surface mounted or externally bonded to investigate the effect of different plating techniques on the moment redistribution in reinforced concrete beams. The number of plates in each specimen was changed. Steel stirrups were used and designed so that no premature shear failure occurs. The experimental results show that the steel plating can cause up to 40 % redistribution of moments from the sagging to the hogging regions compared to the control specimen as well as a significant increase in the loading capacity of the beams. The results also revealed that the application of CFRP plates can lead to substantial amounts of moment redistribution when they are installed as near surface mounted plates, but without enhancing the load carrying capacity.

1. Introduction

Strengthening of Reinforced Concrete (RC) elements using different materials and techniques is widely used to avoid deterioration of existing structures or to increase the capacity of an RC element along a certain cross-section. Regarding continuous RC members, strengthening at a given section can cause a significant amount of moment redistribution (MR). The concept of MR in RC elements is identified in many codes such as ACI 318-14 \cite{1} and CSA/A23.3-14 \cite{2}. When using conventional steel as reinforcement, the yielding of steel results in most of the MR; also several studies \cite{3,4} reported that after the formation of cracks and before yielding of steel at the critical sections, moment redistribution is likely to occur due to the difference in flexural stiffness along the beam.

MR in continuous members allows more flexibility in structural design, which is usually carried out by reducing the hogging moments over supports \cite{5}. Correspondingly, moments at the sagging areas increase to achieve equilibrium. However, in some cases, MR is required to be achieved by increasing the hogging moments at the expense of the sagging moments.

For example, the designers may consider the MR for strengthening only the upper side of continuous structural elements, beams or slabs, in case of having inaccessibility for strengthening the bottom surface of those elements. This strengthening technique is based on increasing the moment over supports and reducing the moment at mid-span due to the modified stiffness of the strengthening system. Strengthening of the hogging moment areas can be achieved either using steel or carbon fiber reinforced...
polymers (CFRP). Strengthening with steel plates/rebars are characterized with their high ductility which allows for significant values of MR to occur. Steel strengthening also has sufficient bond strength with concrete which ensures that MR can be achieved to the maximum possible limit without debonding [6,7].

On the other side, Carbon Fiber Reinforced Polymers (CFRP) is considered as a good alternative to conventional steel due to the non-corrodible nature and high strength to weight ratio. However, CFRP does not undergo yielding process rather they tend to behave in a linear-elastic manner until failure [8], this behavior affects the ability of RC members strengthened with CFRP to achieve MR between the critical sections. Generally, when strengthening a structural element using FRP material, the failure mode tends to be more brittle which reduces MR as the ductility is a key factor to redistribute moments in a continuous RC member. Moreover, several studies showed that CFRP plates often debond earlier than steel plates [6,7,9].

Furthermore, retrofitting of beams and slabs can be achieved by using externally bonded (EB) plates or near-surface mounted (NSM) CFRP or steel plates. Research has shown that EB plates tend to debond at relatively low strains [10,11,12]. As such, the current guidelines neglect the moment redistribution in plated RC flexural members. On the contrary, recent research on retrofitting using FRP and steel NSM plates has shown that NSM plates tend to debond at high strains which can allow for significant amounts of MR to occur [13,14].

As such, the presented study summarizes the experimental investigation for a novel MR approach for RC continuous beams strengthened at the hogging zones only using conventional steel as well as CFRP plates. The method of installment for those plates will vary from EB plates at the top surface of the beams to NSM plates buried in the upper concrete cover of the tested beams.

2. Experimental Program

Eight two-span continuous beams with a span length of two meters were tested using two concentrated loads loaded at their mid-spans as shown in Figure 1. The loads were applied in a monolithic static way until failure. Figure 1 also demonstrates the typical reinforcement details of the tested beams. The beams were designed as T-sections, as shown in Figure 2, to increase the flexural capacity of the section at mid-span between the supports and to avoid early failure of the beams at mid-span sections. Figure 2 also discloses the positions of the used plates in each strengthening technique (1500 mm at the location of the intermediate support). The used plates were implemented at the flanges of the beam within the effective width of the slab rather than in the web of the beam to avoid the presence of an intermediate column if any.

Table 1 summarizes the test matrix and demonstrates the tested parameters. The steel plates were 50 mm in width, 6 mm in thickness, while the CFRP laminates were 50 mm in width, 1.2 mm in thickness. The steel plates and the CFRP laminates were coated by an epoxy adhesive material. The tested beams were named after the type (Steel or CFRP), method of installation number (EB or NSM), and of the number of the used plates. For example, beam CN1 is strengthened with one NSM CFRP plate. For the sake of clarity, the control beam was named “B1”. Figure 3 shows that all the beams were instrumented with six Linear Variable deferential transducers (LVDT) and nine strain gauges (three measuring the concrete compression strain (CS1 to CS3), five gauges measuring the main lower and upper reinforcement strains (SS1 to SS5), and one strain gauge measuring the strengthening plate strain (SS6). Figure 4 summarizes the typical locations of the used instrumentation.

3. Material Properies

The characteristic compressive strength of concrete was 28.7 MPa, with a coefficient of variation (COV of about 9.8%) based on 18 cubes tested according to the British Standards [15]. Moreover, the yield and ultimate tensile strength of the main reinforcement of the steel beams were 360 and 520 MPa respectively, while the yield and ultimate tensile strength of the strengthening steel plates were 425 and 566 MPa with percentage of elongation of 21.9%. The ultimate tensile strength of the CFRP plates used (Sika Carbodur S-512) was 3100 MPa based on the manufacturer’s data, while the modulus of elasticity was 165 GPa with mean percentage of elongation of 1.8%
4. Failure Modes

Generally, cracks first appeared at the load application points and then, for higher loads, at the central support. The control beam underwent an ordinary flexural failure at one of the two load application sections with the yielding of the steel reinforcement. For the control beam, initial cracks were observed at a load level of 160 kN at the positive moment zone, then narrow cracks appeared at the negative moment zone at 300 kN. The failure commenced at the sagging regions at \( P = 435 \) kN through concrete cracking as depicted in Figure 4. The behaviors of the strengthened beams SN1, SE1, CN1, and CN2 were close to the control specimen in terms of the crack initiation, pattern and failure mode while having different failure loads as shown in Figure 5.

On the contrary, beams SN2 and SE2 failed at one of the sections near the load application points by crushing the concrete at the bottom side and at the middle support section as shown in Figure 6. Initial hair cracks appeared at the sagging zone at \( P = 80 \) kN, while cracks began to be visible at the central support at \( P = 220 \) kN. The cracks began to widen and spread almost simultaneously until failure.

Regarding CE1, the cracks first appeared under the load application point at \( P = 80 \) kN, however, the cracks were noticed at the central support at \( P = 160 \) kN. At \( P = 360 \) kN the CFRP start to debond as shown in Figure 7. After debonding, the beam continued to bear more loads but with the rapid increase of the vertical displacement under the load application point. Then the applied load remained nearly constant until the test was stopped at \( P_u = 404.71 \) mm.

![Figure 1. Test setup (dimensions in mm). 1 — Steel or CFRP EB plate; 2 — Steel or CFRP NSM plate; 3 — roller support, 4 — load cell.](image)

| Specimen | Material of plate | Number of plates | Method of Installment |
|----------|-------------------|------------------|-----------------------|
| B1       | No plates (Control) | ---              | ---                   |
| SN1      | Steel             | 1                | NSM                   |
| SE1      | Steel             | 1                | EB                    |
| SN2      | Steel             | 2                | NSM                   |
| SE2      | Steel             | 2                | EB                    |
| CN1      | CFRP              | 1                | NSM                   |
| CE1      | CFRP              | 1                | EB                    |
| CN2      | CFRP              | 2                | NSM                   |
5. Test Results
Table 2 summarizes the test results, where $P_u$ is the maximum load applied on the beam, $R_m$ is the vertical reaction of the central support at load $P_u$, $R_e$ is the end support reaction, while, $M_{sag}$ and $M_{hog}$ are the experimental flexural moments at the positions of mid-span and at the central support respectively. It is worth mentioning, that the own weight of the beams was neglected in the calculations. The percentages of increase in the load capacity of the beam, reduction of $M_{sag}$ and increase in $M_{hog}$ were calculated relative to the control specimen.

The load capacity and the percentage of MR for SN1 and SE1 were similar, while SN2 had higher capacity and MR than SE2. For example, SN2 had 7 and 11% more in terms of capacity and reduction in hogging moment respectively than SE2. Furthermore, SN2 showed higher load capacity, reduction in $M_{sag}$, increase in $M_{hog}$ than SN1 by about 17, 8, and 31% respectively. Moreover, SE2 showed higher load capacity, reduction in $M_{sag}$, increase in $M_{hog}$ than SE1 by about 1.5, 3.5, and 19% respectively. As such, it can be observed that the application of NSM plates was more effective than EB plates when using steel plates as was shown in the comparison between SN2 and SE2.

For CE1, debonding of the CFRP plate from the concrete surface occurred at $P_u = 360$ kN, however, the beam continued to bear extra load until the failure load of 404.71 kN, which resulted in a negligible MR at the maximum load, however, some MR was observed during loading of the beam before debonding. Similarly, the ultimate capacity of CN1 did not increase, however, MR was observed at the sagging and hogging moment locations. CN2 showed MR of 15% more than CN1. As such, The
application of CFRP NSM plates proved to be more effective than EB plates and the usage of two plates gave higher percentages of moment redistribution. However, the beams strengthened with CFRP plates gave smaller values of increased load capacity and MR than the beams strengthened with steel plates.

Table 2. Test Results

| Beam | $P_u$ (kN) | $R_m$ (kN) | $R_e$ (kN) | $M_{sag}$ (kN.m) | $M_{hog}$ (kN.m) | $\%$ of load capacity increase | $\%$ of reduction, $M_{sag}$ | $\%$ of increase, $M_{hog}$ | Average $\%$ of MR |
|------|-----------|-----------|-----------|-----------------|-----------------|-------------------------------|---------------------------|------------------------|---------------------|
| B1   | 467.19    | 360.40    | 53.40     | 53.40           | -126.81         | N.A.                          | N.A.                      | N.A.                   | N.A.                |
| SN1  | 480.86    | 391.53    | 44.67     | 44.67           | -151.10         | 2.93                          | 16.35                     | 19.16                  | 17.75               |
| SE1  | 475.24    | 387.25    | 43.88     | 43.88           | -149.75         | 1.67                          | 17.83                     | 18.09                  | 17.96               |
| SN2  | 561.37    | 478.95    | 41.21     | 41.21           | -198.27         | 20.16                         | 22.82                     | 56.35                  | 39.59               |
| SE2  | 525.72    | 441.12    | 42.30     | 42.30           | -178.26         | 12.53                         | 20.78                     | 40.58                  | 30.68               |
| CN1  | 462.27    | 369.84    | 46.22     | 46.22           | -138.71         | -1.05                         | 13.45                     | 9.38                   | 11.42               |
| CE1  | 404.71    | 315.72    | 44.50     | 44.50           | -113.37         | -13.37                        | 16.67                     | -10.60                 | 3.03                |
| CN2  | 468.55    | 380.41    | 44.07     | 44.07           | -146.14         | 0.29                          | 17.46                     | 15.24                  | 16.35               |

Figure 4. Failure mode for beam (B1)  
Figure 5. Typical failure mode for beams SN1, SE1, CN1 and CN2  
Figure 6. Failure mode for beams SN2 and SE2  
Figure 7. Debonding of the CFRP plate in beam CE1  a. at mid-span; b. at the central support

6. Analysis of Results

6.1 Strains.

Figure 8 demonstrates the tensile strain of the lower reinforcement, showing a similar trend for all the beams except SN2 and SE2 (i.e. with small yielding plateau). This different behavior supports the fact that these beams had smaller ductility controlled by the concrete crushing. It was also noticed that the application of CFRP plates did not affect the behavior of the load-strain relationship in the linear elastic stage. On the other side, the concrete compressive strains measured at the top side at the mid span sections or the lower side at the middle support location did not exceed 0.003 as shown in Figure 9.

The control beam yielded at the lowest load level which is mainly because of that the positive flexural stresses at mid-span caused by sagging moments. Generally, the yielding loads increased with the decrease in the sagging moments in the beams which were clear in the behavior of the rest of the beams, except for CN2 which had a sagging moment less than CN1, SE2, and SN2 but still had a lower yielding load than those beams.

6.2 Deflection.

Figure 10 shows the load-displacement curve at the point of load application. From the figure, both beams B1 and CE1 showed large ultimate deformation, 27.15 mm and 29.75 mm for B1 and CE1
respectively. The similar behavior of beam CE1, behaved to B1, is attributed to the debonding of the CFRP plate which lead to a non-strengthened beam behavior as beam B1.

It is also noted that the application of CFRP strengthening did not affect the flexural stiffness of the beams considerably in the linear elastic range as it is shown that all the beams behaved in an elastic behavior at the beginning until reached a value between 300 and 360 KN, after that, the behavior of the beams was varying depending on the strengthening technique.

All the strengthened beams, except for CE1, had lower values of the ultimate displacement compared to the control specimen. This is mainly due to the redistribution of moments from sagging to hogging zones, which means that the reduction of sagging moments resulted in less displacement values.

![Figure 8. Applied load versus maximum measured tensile strains of lower reinforcement](image1)

![Figure 9. Applied load versus compressive strain of the bottom side of concrete at the central support](image2)

![Figure 10. Applied load versus maximum displacement at points of load application for each beam](image3)

| Beam | $\Delta_u$ (mm) | $\Delta_y$ (mm) | $\Delta_{cr}$ (mm) | $\mu_{\Delta_u-cr}$ | $\mu_{\Delta_u-y}$ |
|------|-----------------|-----------------|-------------------|---------------------|-------------------|
| B1   | 27.15           | 4.55            | 4.74              | 5.73                | 5.97              |
| SN1  | 19.36           | 10.92           | 4.68              | 4.14                | 1.77              |
| SE1  | 17.13           | 8.39            | 3.75              | 4.57                | 2.04              |
| SN2  | 19.61           | 15.42           | 5.45              | 3.60                | 1.27              |
| SE2  | 18.63           | 15.58           | 4.11              | 4.53                | 1.20              |
| CN1  | 15.93           | 10.06           | 4.33              | 3.68                | 1.58              |
| CE1  | 29.75           | 6.65            | 4.22              | 7.05                | 4.47              |
| CN2  | 14.44           | 8.58            | 4.14              | 3.49                | 1.68              |
6.3 Ductility Indices.

Table 3 shows the deflection ductility indices including the ratio of the ultimate to yield deflection, which is known as the ductility factor and is often used as a simple measure of energy absorption [16]. The deflection ductility indices $\mu_{\Delta u-cr} = \Delta u / \Delta cr$ and $\mu_{\Delta u-y} = \Delta u / \Delta y$ which are the ratios of the ultimate deformation ($\Delta u$) to the deformation at the onset of cracking ($\Delta cr$) and the deformation at the point of yielding of the lower steel reinforcement in the sagging zones ($\Delta y$). All the strengthened beams had smaller values of $\mu_{\Delta u-y}$ than the control specimen, regardless the amount of moment redistribution. Thus, it can be concluded that the moment redistribution is related not only to the capacity of inelastic deformation but also to a proper design of the ultimate capacity of critical sections.

Beam CE1 recorded the highest value of $\mu_{\Delta u-y}$ since it acted as a non-strengthened beam for the late stages of the test after debonding of the CFRP plate. This is so close to the case with $\mu_{\Delta u-cr}$ in which the values determined for the control specimen and CE1 are the highest among all the experimented specimens.

6.4 Load vs percentage of moment redistribution.

As shown in figures 11 and 12, the initial values of moment redistribution were not negligible because the flexural stiffness of the beam in the hogging region (strengthened zone) was higher than in the sagging region. For beam SN2, the percentage of reduction in the sagging moment decreased from the initial values they started with a high rate until the cracks appeared near the central support, when a slight rise occurred and then the reduction decreased with a very low rate until the end of the test to show eventually a final value almost equal to the value occurred at the appearance of the cracks near the intermediate support. For SE2, the values remained almost constant during the test and achieved lower rates than that achieved by SN2 at all load levels.

The behavior of beam CN1 was close to that of SN2 in which the percentages of MR decreased rapidly at first until the moment when the cracks appeared near the central support, and then the values decreased with a smaller rate until the lower steel reinforcement at the sagging regions yielded at $p = 422.75$ kN before the values started to rise given a final value close to what was achieved when the first cracks appeared near the central support.

For CN2, the behavior was similar to that of CN1, the beam had the highest initial MR percentages. These values decreased rapidly until the cracks appeared at the central support when a slight rise took place, then the values continued in decreasing at a high rate until a load level of about 400 kN. That is when the lower steel bars at the sagging zones reached yield and the values began to increase until failure.

Beam CE1 witnessed an initial decrease of MR percentage until the load level of about 250 kN when a significant rise in the values occurred until the load level of 360 kN which is when the CFRP plate was debonded from the concrete surface. After that, the values decreased abruptly until failure at $P_u = 404.71$ kN.

The behavior of beams SN1 and SE1 was different from that of the previous ones. The MR percentages started with negative values and increased during loading until the load level of about 340 kN. After that, the values remained constant until the reinforcement in the sagging zones reached yield at load levels approximately equal to 430 kN before the values rose again reaching significant positive values at the end.

![Figure 11 Applied load versus the percentage of reduction in sagging moments for all beams](image-url)
Figure. 12 Applied load versus the percentage of increase in hogging moments for all beams

7. Conclusions
The NSM steel plating technique at the hogging zones caused up to 40% redistribution of moments from the sagging to the hogging regions compared to the control specimen. Moreover, the NSM plates were found more effective than the EB ones when two plates were used as SN2 had 7 and 11% more in terms of load carrying capacity and decrease in hogging moment respectively than SE2. However, the difference between the aforementioned techniques was negligible when only one plate was used.

The application of two steel plates was more efficient than one steel plate as SN2 showed higher load capacity, reduction in sagging moment, increase in hogging moment than SN1 by about 17, 8, and 31% respectively. Moreover, SE2 showed higher load capacity, reduction in $M_{sag}$, increase in $M_{hog}$ than SE1 by about 1.5, 3.5, and 19% respectively.

Furthermore, the application of CFRP plates can lead to substantial amounts of moment redistribution when they are installed as NSM plates, but without enhancing the loading capacity.

Significant redistribution of moments occurred in the tested beams prior to the plastic limit being reached in either the reinforcement or the concrete. This elastic redistribution took place as a result of the variation in flexural rigidity throughout the span because of the presence of the plates and due to cracking at the critical sections. The application of CFRP strengthening techniques investigated in this study did not affect the flexural stiffness of the beams considerably in the linear elastic range.

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