

Investigating the load carrying capacity of RC undamaged and damaged beams strengthened with CFRP

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ABSTRACT

Every year there are unfortunate situations in which thousands of structures experience destruction or severe damage due to natural or manmade disasters. Some of these structures could be repaired or strengthened using Carbon Fibre Reinforced Polymer (CFRP) to restore their load carrying capacity and make them serviceable. In this study two reinforced concrete (RC) beams were casted under controlled laboratory conditions. One of the RC beams was used as a control beam and the other was strengthened with CFRP using the Near Surface Mounted (NMS) technique. Both beams were tested using a 3 point static test until their failure. Samples of concrete cubes, steel reinforcement bars and CFRP strips, used in these RC beams, were tested in the laboratory to determine their precise material properties. The authors then used finite element analysis (FEA) and the first author's previous findings using model updating techniques, to create an analytical model of the RC beam strengthened with the CFRP to demonstrate that the generality proposed technique that is transferable to any RC beams strengthened with CFRP. The research has demonstrated that using this technique can considerably reduce the number of expensive laboratory samples.

INTRODUCTION

Carbon Fibre Reinforced Polymer (CFRP) is a non-corrosive lightweight and high tensile strength material that can be easily fitted to any type of structure. Due to these advantageous characteristics, CFRP has been recently used in new construction and rehabilitation of structures as reinforcement in concrete structures, bridge decks, modular structures, formwork, and external reinforcement for strengthening and seismic upgrade (Jain and Lee, 2012).

The first use of carbon fibre was reported by Bacon (1960) in the Journal of Applied Physics. Fibre Reinforced Polymer (FRP) are composites used in almost every type of advanced engineering structure, with their usage ranging from aircraft and spacecraft

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to ships and offshore platforms. FRPs have also been used in civil infrastructure applications Masuelli (2013).

Carseet *al.*(2002) reported that “Rehabilitation of deteriorated civil engineering infrastructure such as bridge decks, buildings, beams, girders, parking structures, marine structures, roads etc. has been a major issue in the last decades. The deterioration of these structures might be due to ageing, poor maintenance, corrosion due to poor environmental conditions, poor initial design or construction and accidental situations like earthquakes. The need to upgrade the deteriorated civil engineering infrastructure greatly enhances with the ever increasing demands”(Carseet *al.*, 2002).

Strengthening of structural members, as reported by Norris *et al.* (1997) is a method that has been used in Europe and South Africa since the mid 1960's, is to use epoxy to bond steel plates to flexural members. Beberet *al.* (2001) indicates that the epoxy glued plates that have been retrofitted to structural members in the last 30 years have been very effective. An example of the first large scale application of this type of strengthening was of two pre-stressed concrete bridges over the Nete Canal at Lier Belgium constructed in 1955 (Brosens and Van Gemert, 2001). The bridges were found to have lost part of their pre-stressing as a result of corrosion and in 1980 a repair took place in which steel plates bonded along the axis of the girders, Van Gemert (1980).

According to Duthinh, and Starnes (2004), after the publication of the outcome of the research at the Swiss Federal Laboratories for Materials Testing and Research, or EMPA (Meier, 1987), in Europe the use of CFRP composites for the retrofitting of beams and slabs started in the late 1980s. Recently the use of Fibre Reinforced Polymer (FRP) for strengthening and upgrading of existing structural elements has gained particular attention (Hamed and Rabinovitch, 2005). The use of FRPs on structures has become more frequent in the UK, where it is estimated that it has been used on several hundred structures (TR55 2012). Design guidance for this has been produced by the Concrete Centre; the most recent edition was published in 2012. This guidance shows techniques and advancement in the use FRPs in the construction industry.

Hamed and Rabinovitch (2005) and Aldajahet *al.* (2008) both commented that FRP has emerged as a promising alternative to steel and are rapidly becoming materials of choice for the strengthening and rehabilitation of concrete infrastructure.

As the literature has shown, although the CFRP has proven to be a suitable material for repairing and strengthening of concrete structures, its high cost makes its use to some degree prohibitive in structural engineering. This is one of the reasons that it has not been widely used in this area. Another reason for this lack of popularity is due to the cost of testing large numbers of structural members in the laboratory.

Recent research by Rafiq and Al-Farttoosi (2013 and 2014) has demonstrated that by using model updating techniques it is possible to develop reliable analytical models that closely predict the behaviour of both strengthened and un-strengthened RC beams. Rafiq and Al-Farttoosi (2014) used test data from Esfahani *et al.* (2007) on a RC beam with and without CFRP to develop a Finite Element (FE) model and they demonstrated that by using the knowledge of failure criteria of RC beams it is possible to develop a generalised FE model, that closely predicts the behaviour of RC beam strengthened with CFRP.

The research presented in this paper is testing the generality of the methodology proposed by Rafiq and Al-Farttoosi (2014) on beams with different dimensions and steel bar details. The aim was to demonstrate that using this approach can reduce number of expensive laboratory testing samples. Details of our findings are presented in the following section.

METHOD

Two concrete beams were cast under strict laboratory conditions and three point load tests were conducted. The first test was a control test on a standard RC beam with no CFRP strengthening. The next test was on an identical RC beam, cast and cured under the same laboratory conditions but strengthened with a near surface mounted (NSM) CFRP strip. The final test was on the control beam, after it had been tested to complete failure. This beam was then repaired and strengthened with a NSM CFRP strip. All beams were tested under a three point static load. The experimental arrangement and beam dimensions are shown in Fig.1.

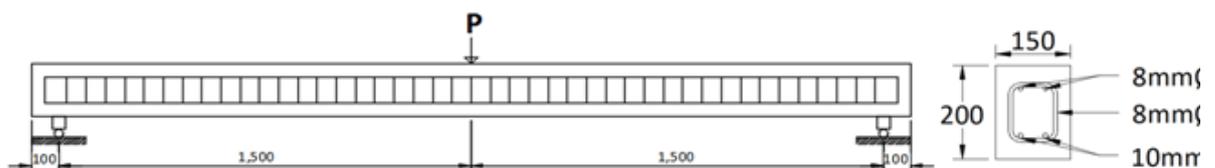


Fig. 1 Experimental Beam Arrangement

The loading on the beams was very slowly increased, so that crack formation patterns and beams behaviour could be observed and recorded. First the control beam, not strengthened with CFRP, was tested. The beam was prepared for the test by bonding a small steel plate, 150mm x 100mm x 20mm, to the beam top face, in the location of the point load application. The purpose of the plate was to protect the top face of the beam and spread the point load over a slightly larger area. This was necessary because without it the point load would have damaged the top face of the beam and would have affected the results.

The second test was on the RC beam strengthened with NSM CFRP. The beam was cast with a slit in the bottom face, as shown in Fig.2. The slit was cleaned and primed using a concrete primer and was left overnight. In the next morning the slit was filled with an epoxy resin and the CFRP strip was bonded into the slit, as shown in Fig.3. The resin and CFRP was left for 2 days to ensure sufficient curing time. A plate was then bonded to the top of the beam, as used in the un-strengthened control beam, to avoid damage to the beams surface.



Fig.2 Slit Cast in Bottom of Beam



Fig.3 CFRP Bonded inside the Slit

The final experiment was conducted on the original un-strengthened control beam that had been damaged under the test and then repaired using a NSM strip of CFRP. After the first experiment the beam had severe residual deflection and cracking, as shown in Fig.4 and Fig.5, therefore the beam needed to be slowly straightened. This was done by inverting the beam and applying a load gradually using the 100 tonne press machine. Then the CFRP was bonded to the beam in the same way as the second test, however, because of the heavy damage to the underside of the beam due to cracking the cleaning out of the slit and applying the epoxy resin was more difficult. The beam was restrained whilst the CFRP cured to ensure it did not spring back to its residual deflection. In addition to the residual deflection there was crushed concrete on the top face of the beam from the previous loading cycles, this needed to be repaired to ensure that when reloaded the load would be transferred uniformly into the beam. A quick repair polyester resin was used to repair the damage and bond the loading plate at the centre of the beam. The resin was only used as a way to bond the steel plate to ensure a uniform spread of load and not to increase the strength or repair the cracks in the beam. The resin application has been shown in Fig.6. The polyester resin was quick setting so within 1 hour of application the test could commence.



Fig.4 Residual Damage to Control Beam



Fig. 5 Close Up Residual Damage



Fig.6 Polyester Resin Application

Material Testing

Material property tests were conducted on the individual elements of the experimental beams; the concrete, the steel reinforcing bar and the CFRP strip. These tests were crucial to obtain correct material properties to be used in the FE model. Compression strength tests on concrete cube samples that had been made at the time of casting were conducted to measure the ultimate concrete compressive strength and elastic modulus. Similarly, tensile tests were conducted on the steel reinforcement bar and the CFRP strips used to strengthen the concrete beam. Five samples of each type of reinforcing bar CFRP strips underwent a tensile test.

Previous Research on Analytical Modelling

Uncertainties related to reinforced concrete elastic and non-linear material properties are very difficult to quantify. These uncertainties make the modelling of the reinforced concrete material very difficult as it depends on many variables (e.g variation in aggregate properties from batch to batch and even within each batch; variation in cement content; water-cement ratio etc.).

Rafiq and Al-Fartoosi (2013) used a Genetic Algorithm (GA) to model variation in material properties along the length of the RC beams. In this study they used one of the

RC beams with 20mm main bars, tested in the laboratory by Esfahani *et al.* (2007) using 4 point static load test, see Fig.8 for details. They demonstrated that the GA was able to closely model the behaviour of the RC beam, which was a good match with their experimental results. They discovered that variation in the material properties of the concrete along the beam length was dependent on the propagation of the cracks along the beam length. Variation in the elastic modulus of the concrete along the length of the beam, according to their finding, is shown in Fig. 9.

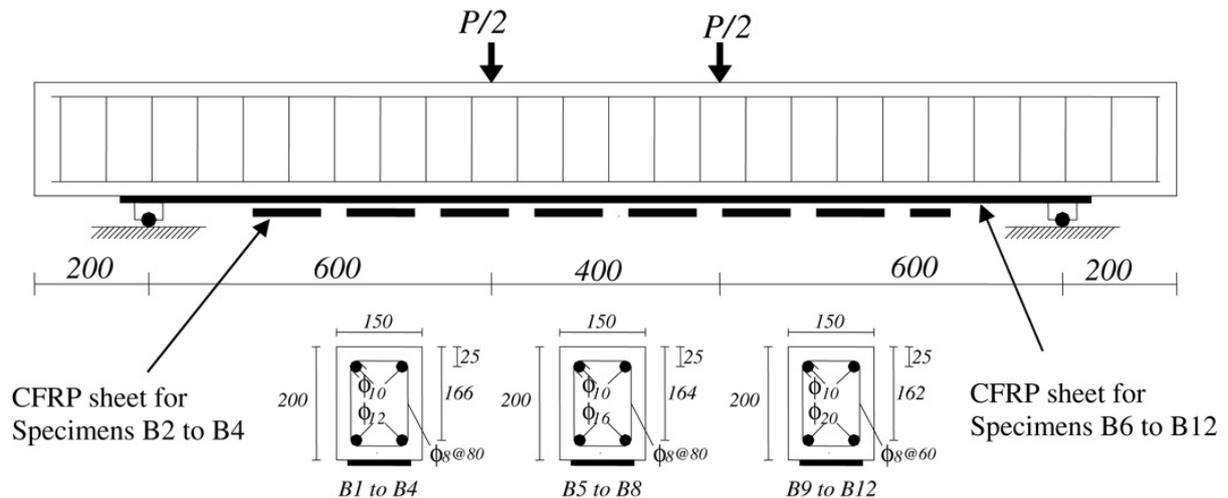


Fig. 8 Esfahani RC beam details (Esfahani *et al.*, 2007)

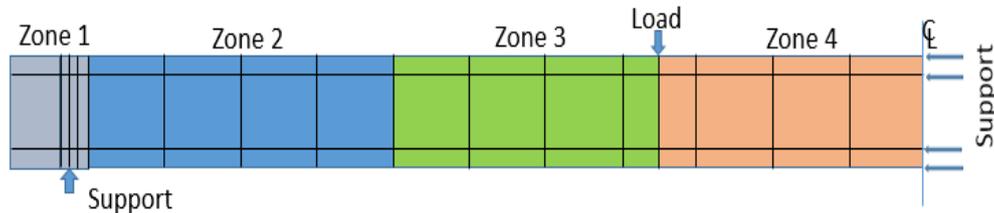


Fig. 9 Variation in concrete elastic modulus along the beam length

To validate the generality of their findings, Rafiq and Al-Farttoosi (2013) used the same variation in the properties of concrete from the test beam with 20mm main bars to two other Esfahani *et al.* (2007) RC beams with 16mm and 12mm main bars. They demonstrated that their proposed analytical model for all three beams closely matched their behaviour with the laboratory test results.

Rafiq and Al-Farttoosi (2014) extended their findings using Esfahani *et al.* (2007) other three beams with 20mm, 16mm, and 12mm main bar, but this time strengthened with CFRP. Once again the beam with 20mm bar size was used as the control beam. They found out that the variation in concrete material properties along the length the beam, as found for the beam without CFRP (Fig. 9), did not change for the beam with

CFRP, but the crack pattern along the length of the beam which was strengthened with CFRP changed the modelling of the reinforced steel elements as de-bonding of the steel bars was critical.

To simulate the de-bonding between steel and concrete Rafiq and Al-Farttoosi (2014) used Joint elements with spring constants (K-Springs) to model the bond resistance between steel bars and surrounding concrete at each joint. Similarly the interface layer between the concrete and the CFRP strip was modelled using cohesive element to simulate the de-bonding between the concrete and the CFRP.

Once again a GA was used to model variation in K-Springs at the Joint elements along the length of the beam. The main finding of this study was that values of the spring constants were dependent on the crack and crushing pattern of the concrete along the beam length.

In order to generalise their findings on RC beams strengthened with the CFRP having different bar sizes, a closer look at the spring K-values revealed that the spring k-values were dependent on the bar diameter and these values were close to twice the ratio of the perimeters 20mm bar divided by the perimeters 16mm and 12mm bars respectively. Fig. 10 shows FE discretization of the beam geometry for the FEA modelling purposes.

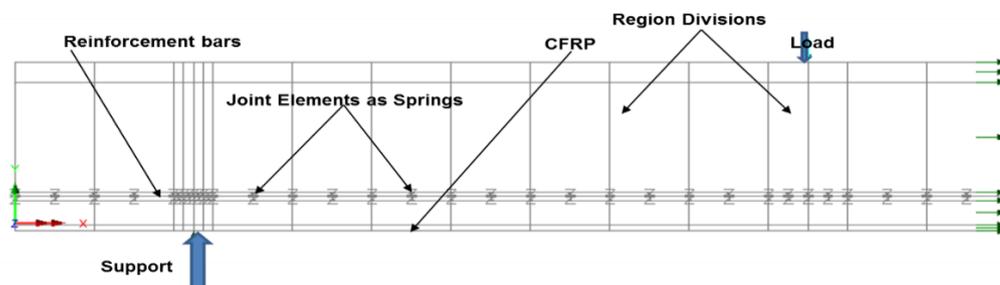


Fig. 10 Showing the FE discretisation of the beam geometry for the Analytical model with the location of springs modelling the bond-slip

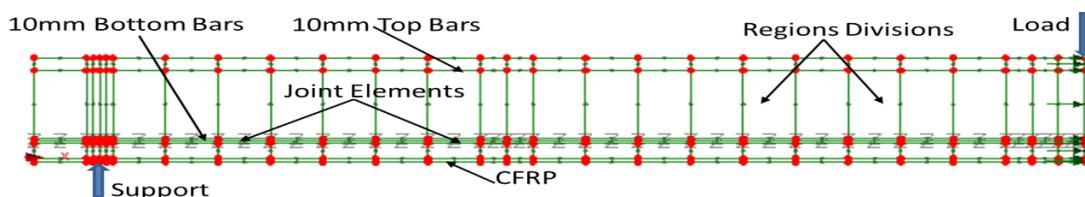


Fig. 11 Showing the FE discretisation of the beam geometry for the Analytical model with the location of springs modelling the bond-slip

In this paper the authors will be trying to further test the generality of the above hypothesis for: (a) a longer RC beam that is representative of real beams in office buildings; (b) using a 3 point test instead of the 4 point test. (c) the spring K-values used for this RC beam will be the same as those found by Rafiq and Al-Farttoosi

(2014). The same rule for proportioning the ratio of twice the ratio of the perimeters 20mm bar divided by perimeter of (10mm) bars, as used for the beams reported in this paper, will be used. Fig 11 shows details of the test beam presented in this paper.

Implementation to current study

In this study the authors used two full size RC beams 3.2 m span length. Fig. 1 shows the details of the steel and concrete sizes used. Fig.12 shows the load-deflection graphs of each of the test that was conducted (controlled beam without CFRP, beam strengthened with CFRP and beam damaged then strengthened with CFRP and retested).

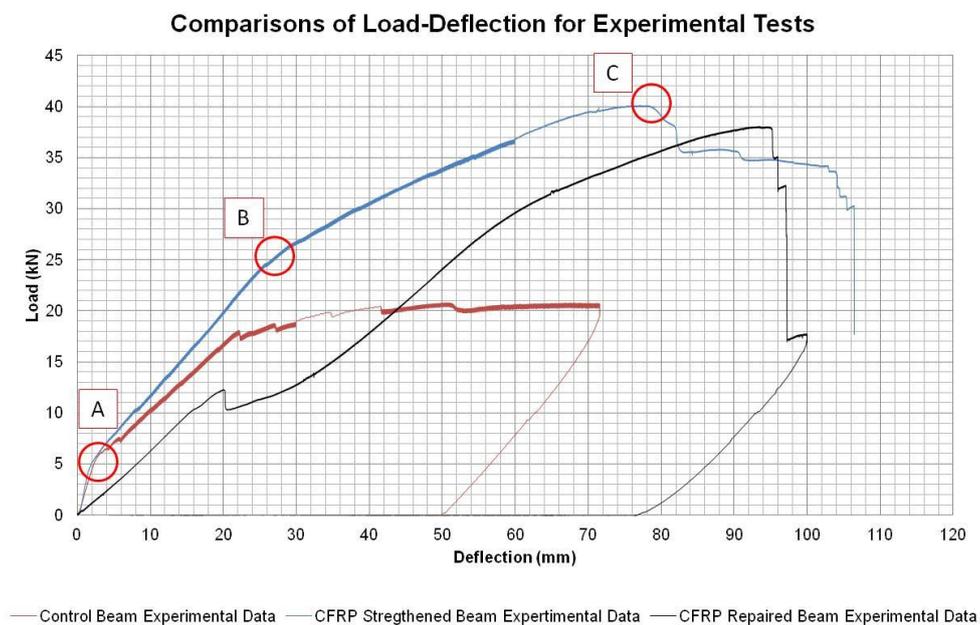


Fig.12 Showing the Load deflection Curves for the Three Tested Beams

The points highlighted in Fig. 12 show the behaviour of the CFRP strengthened beam through the loading cycle.

- A.** First crack approximately 6kN
- B.** Steel yields approximately 26kN
- C.** CFRP failure approximately 40kN

For the FEA model the same concrete, steel and CFRP properties, obtained from physical lab testing of were used. Concrete properties were varied at four zones as discussed in previous section. Similarly reinforcement steel, bond between steel bars and the concrete and CFRP was model as per previous section (see Fig.13 for details).

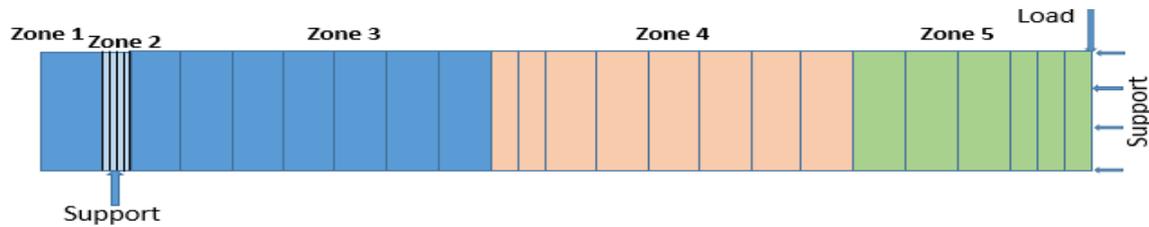


Fig. 13 Variation in concrete elastic modulus along the beam length based on Fig. 9

RESULTS AND DISCUSSION

The LUSAS FEA analysis software (LUSAS, 2015) was used in the analysis process.

The method used in the FEA process are briefly summarised as follows:

1. RC Beam with 10mm bar size was used for main reinforcement and 8mm as top bar and also for shear links.
2. The same rationalised concrete E-Values at 4 Zones, as reported by Rafiq and Al-Farttoosi (2014), was used in this model.
3. Thirteen different K-Springs values were assigned to each spring representing bond slip between the concrete and reinforcement bars at each location along the length of the RC beam. Values for the spring constants were proportionally calculated for 10mm based on the findings of Rafiq and Al-Farttoosi (2014) for 20mm bars.
4. The CFRP was models as a linear 2D BAR element and its properties were obtained from manufacturer supplied data sheets.
5. These parameters were then passed to a nonlinear FEA analytical model of the RC beam strengthened with the CFRP
6. Results obtained from the FEA modelling were compared with those of the laboratory results.
7. A load deflection graph was used for the comparison purposes.
 - A. First crack approximately 6kN (Experimental and FEA)
 - B. Steel yields (FEA)
 - C. Steel yields (Experimental)
 - D. CFRP failure approximately 40kN (Experimental and FEA)

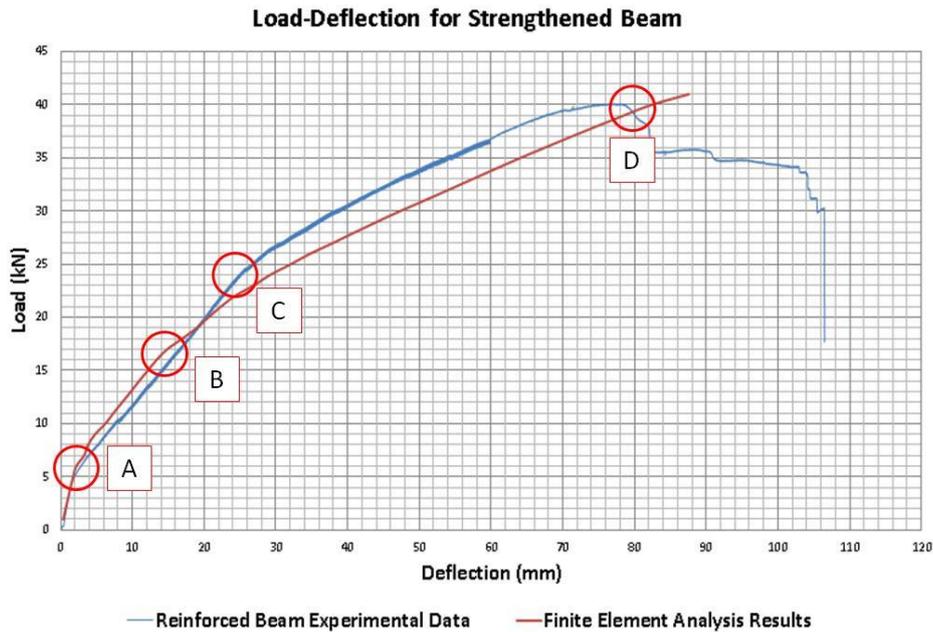


Fig. 14 Comparison of the FEA with the Experimental Results

CONCLUSIONS

Fig. 14 presents a comparison of the FEA non-linear analysis model with the laboratory experimental result. Our findings of the process discussed can be summarised as follows:

1. The first crack in the concrete appears around 6kN. This is the same for both analytical and experimental results.
2. The slope of the line for the region between first concrete cracks and where the steel bars yield (between 6kN and 25kN) are similar for both the FEA and analytical results.
3. Similarly the slope of the line between the Steel yield point and CFRP failure is also very similar.
4. The final failure load and deflection for both the FEA and experimental results is around 40 kN.
5. The only discrepancy between the two results is that in the FE analytical model the yielding of steel reinforcement bar takes place a bit earlier than that of the experimental results.
6. Overall a good correlation is achieved.

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