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Ductility of Reinforced Concrete Beams Strengthened using Prestressed NSM CFRP Strips/Rebars – Analytical Study

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ABSTRACT: Near Surface Mounted (NSM) Carbon Fiber Reinforced Polymer (CFRP) strengthening technique becomes attractive for flexural strengthening due to its ease of installation and its greater bonding capacity as compared to externally bonded CFRP laminates. Prestressed NSM reinforcements are very effective in increasing flexural capacity, reducing deflection, and addressing serviceability concerns. In this work, a finite element model (FEM) of reinforced concrete (RC) beams strengthened using prestressed NSM CFRP strips and rebars was developed to analytically analyze the strengthening efficiency of the NSM technique. The FEM was validated with experimental results found elsewhere in literature. After verification with experimental results, the analytical model is used to evaluate the ductility of the strengthened beams. The optimum prestress level, prestress level at which the ductility of the strengthened beam is equivalent to that of the un-strengthened beam or the prestress level which provides the maximum ductility index, is in the range of 25.5% to 26.5% and 20.4% to 24.5% of the ultimate tensile strength of the CFRP for beams strengthened with CFRP strips and rebars, respectively.

1 INTRODUCTION

Strengthening using Externally Bonded (EB) Fiber Reinforced Polymer (FRP) plates and sheets proved as an efficient system to upgrade the structural capacity and enhance the performance of Reinforced Concrete (RC) structures. However, strengthening using EB FRP has some drawbacks such as premature debonding failure and exposure to external environment. These drawbacks hinder its effectiveness in strengthening RC structures. Thus, researches were carried out to develop new strengthening systems that overcome the deficiencies of EB FRP. Near-Surface-Mounted (NSM) FRP technique was developed to strengthen RC structures in shear and bending. This innovative strengthening technique overcomes the drawbacks of EB FRP plates and sheets. The strengthening procedure involves embedding CFRP reinforcement inside a pre-cut groove in the tension side of the concrete members then filled with epoxy.

Premature debonding failure is less likely to occur when strengthening using NSM FRP reinforcements due to the fact that three faces of the epoxy block are in contact with the surrounding concrete. This would allow a full utilization of the FRP reinforcement. Moreover, the FRP reinforcement is not exposed to external environment since it is embedded inside the epoxy block.

Many researchers investigated the effectiveness of using NSM FRP for strengthening RC beams, slabs, and columns in shear and bending (Barros et al., 2004). Test results showed that strengthening using NSM would allow full utilzation of the tensile strength of CFRP reinforcement. El-Hacha et al., (2004) compared the effectiveness of using non-prestressed NSM FRP in flexure with EB FRP. For the same axial stiffness, the ultiamte strength increase of beams strengthened with NSM CFRP was around four times that of beams strengthened with EB FRP. Rupture of the NSM was the dominant failure mode while for the EB the failure was by debonding. The flexural performance of concrete slabs strengthened using non-prestressed NSM CFRP strips showed that a percentage of 0.12% of CFRP increased the service load of the concrete slab reinforced with steel ratio of 0.24% by 54%. Also, the ultimate load increased by 390% as compared to that of the un-strengthened slab (Bonaldo, 2005).

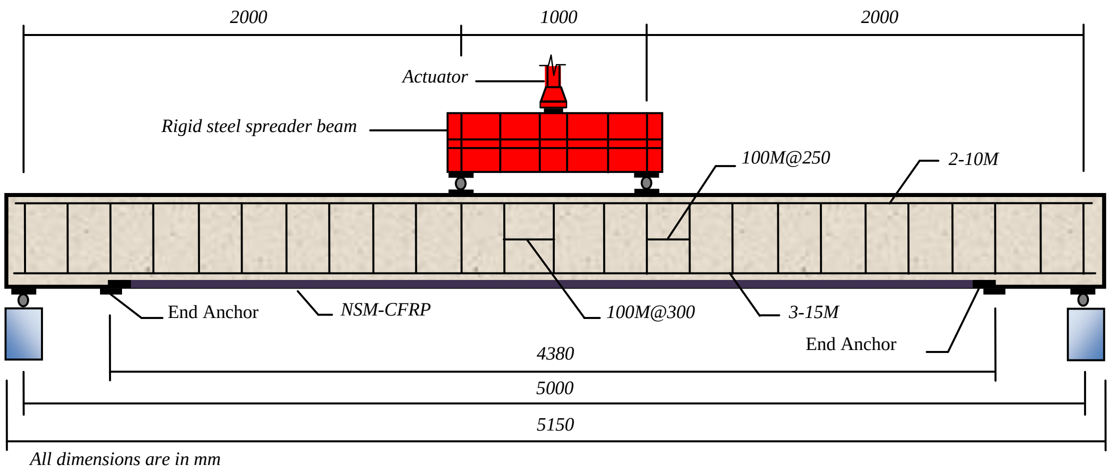
The efficiency of the NSM strengthening system can be enhanced further if the CFRP reinforcement is prestressed before bonding to the member. Several studies investigated the effectiveness of using prestressed CFRP material in strengthening RC structures. An increase of 50% of the yield load of beams strengthened using prestressed CFRP plates was achieved compared to un-strengthened beams and up to 25% compared to beams strengthened using non-prestressed CFRP plates (Nordin et al., 2001). Prestressing NSM CFRP proved to enhance the monotonic and fatigue flexural performance of RC beams (Badawi, 2007). Gaafar (2007) tested beams strengthened with prestressed CFRP strips and rods designed to have the same axial stiffness. Results indicated the effectiveness of the prestressing system and that all beams failed due to rupture of the CFRP without any debonding failure. Damaged prestressed concrete 11m long bridge I-girders strengthened using prestressed NSM CFRP rods performed in a more ductile manner when compared to girder strengthened using EB CFRP plates (Casadei et al., 2006).

This paper presents an analytical evaluation of the performance of RC beams strengthened using NSM CFRP prestressed reinforcement. The analytical model is validated with experimental test results found elsewhere in literature (Gaafar 2007). Ductility comparison is made between the analytical and experimental results in order to determine the optimum prestressing level at which the ductility of the prestressed beam is equal to that of the un-strengthened beam or the prestress level which provides the maximum ductility index.

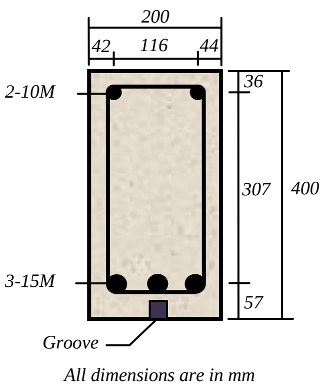
2 BEAMS TO BE MODELED

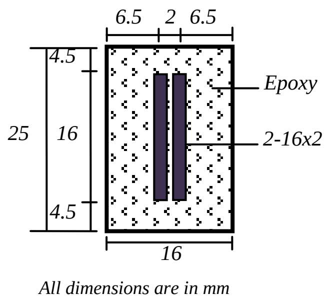
Figure 1 shows typical cross-section of the beams to be simulated. Two sets of strengthened beams and one un-strengthened control beam were tested in flexure until failure. Set B1 consists of four beams strengthened with CFRP strips prestressed to 00, 20, 40, and 60% of the ultimate CFRP tensile strength. Set B2 consists of four beams strengthened with CFRP rebars prestressed to the same prestress levels as set B1.

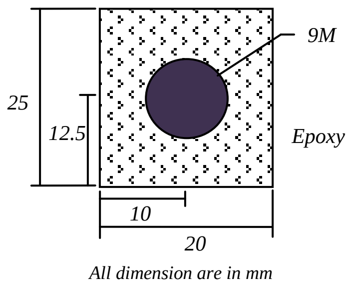
The strengthened beams were designed to increase the strength of the control un-strengthened beam by 30% and to fail by rupture of the CFRP after yielding of the steel reinforcement. All beams were strengthened using same axial stiffness (*EA*, where *E* is the CFRP modulus of elasticity and *A* is the area of CFRP used) of the CFRP, thus, beams of set B1 were strengthened using two strips (2×32 mm^2) to have the same axial stiffness of the beams of set B1 which were strengthened using one rebar (63.61 mm^2) (Gaafar 2007).



(a) Beam dimensions







(b) Cross-section (c) Strips (d) Rebar

Figure 1. Beam details.

3 DESCRIPTION OF FINITE ELEMENT MODEL

The reinforced concrete beam has two planes of symmetry, the *x-y* plane cutting the beam in half longitudinally and the *y-z* plane cutting beam in half transversely, thus, only one fourth of the beam is modeled. Test observation shows that all beams failed due to FRP rupture and a full composite action was achieved (Gaafar 2007). Thus, a 2D model was sufficient to simulate the flexural behaviour of the strengthened beams where the epoxy block and the FRP reinforcement were treated as embedded elements in the concrete elements. ABAQUS/Standard was used to model the beams. An 8-node continuum element with reduced integration was used for modeling the concrete. Concrete damaged plasticity model was used to simulate the behaviour of the concrete material. The model assumes that the main two failure mechanisms are tensile cracking and compressive crushing of concrete. Steel reinforcement was modeled by 3-node 2D truss element and was embedded inside the concrete elements, epoxy block was modeled by 2-node beam element, and CFRP strips and rebars were modeled by 3-node beam element. The prestressing force was applied to the CFRP reinforcement using the initial condition which assumes that stress values are applied uniformly over the element.

3.1 Material properties

3.1.1 Steel reinforcement

Steel stress-strain relationship obtained from tension tests is simplified into the tri-liner pattern. Tension, compression, and stirrups steel reinforcement have the same mechanical properties; yield stress of 475 MPa and a modulus of elasticity of 200 GPa, the strain at the end of the yield plateau is 28000με and the ultimate stress and strain are 600 MPa and 120000με, respectively (Gaafar 2007).

3.1.2 Concrete

Concrete compression stress-strain relationship was modeled using the following model developed by Saenz (1964):

$$\sigma_c = \frac{E_c \varepsilon_c}{1 + (R + R_E - 2)\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - (2R - 1)\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 + R\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^3} \quad (1)$$

where σ_c and ε_c are concrete compressive stress and concrete compressive strain respectively.

and $R = \frac{R_E (R_\sigma - 1)}{(R_\varepsilon - 1)^2} - \frac{1}{R_\varepsilon}$, $R_E = \frac{E_c}{E_o}$, $E_o = \frac{f_c'}{\varepsilon_o}$, R_a =4, R_ε =4, and ε_o =0.003.

The tension stiffening curve has an initial slop of 21.55 GPa up to the tensile strength of 2.47 MPa and a descending branch calculated using the following equation (Kang et al., 2005):

$$\sigma_t = 10^6 \varepsilon_t^2 - 3133.7\varepsilon_t + 2.6771 \quad (2)$$

where σ_t and ε_t are concrete tensile stress and concrete tensile strain, respectively.

3.1.3 Epoxy

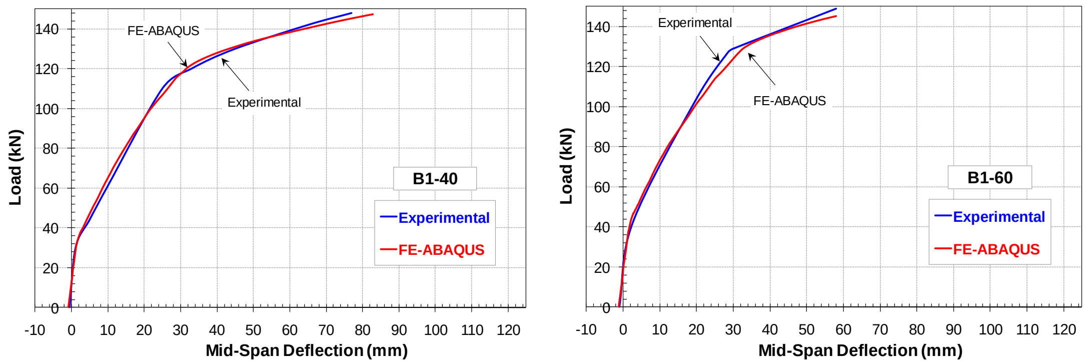
The epoxy adhesive used had a tensile strength of 24 MPa and an elongation at break of 1% (SIKA Canada).

3.1.4 CFRP strips/rebars

Both CFRP strips and rebars had the same mechanical properties; ultimate tensile strength of 2068 MPa, modulus of elasticity of 124 GPa, and fracture strain of 1.67%. The nominal areas of the strip and the rebar are 32 and 63.6 mm^2, respectively (Hughes Brothers).

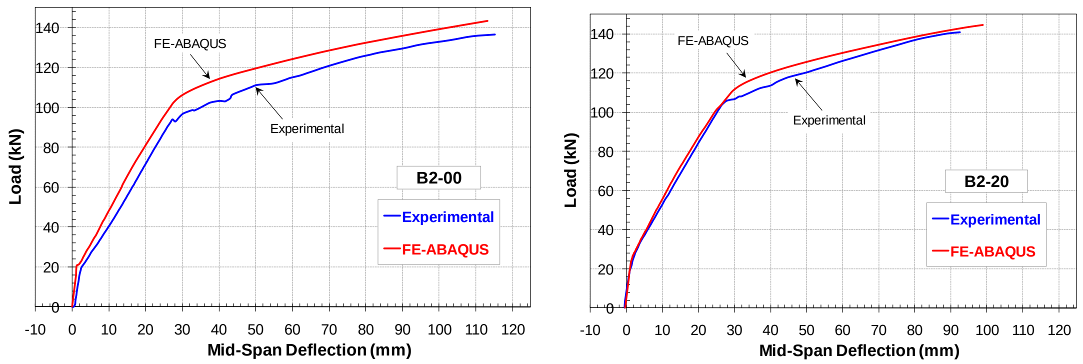
4 VERIFICATION OF ANALYTICAL MODEL

The validity of the model is examined by comparing the analytical results with the results of experimental work by (Gaafar 2007). Figures 2 and 3 show samples of the load-deflection curves by comparing the analytical results with the experimental results. It can be seen that the analytical results are in good agreement with the experimental results indicating the validity of the proposed model. The slight difference between the experimental and analytical results observed might be due to prestress loss and bond effect which are not accounted in the model. Analytical models of beams strengthened with CFRP strips are more accurate in predicting the load-deflection beahviour of the strengthened beams as compared with the models of beams strengthened with CFRP rebars. This difference is attributed to the fact that the method of prestressing the CFRP strips was different than that of the CFRP rebars. As reported by Gaafar (2007), the CFRP strips were prestressed to the required strain measured from the strain gauges mounted on the CFRP strips while CFRP rebars were prestressed to the required force measured from the load cells mounted on the hydraulic jack of the prestressing system. The former method is more precise since it is directly related to the strain of CFRP as compared with the later one which might give misleading readings. More details about the development of the FE model can be found in Oudah and El-Hacha (2010)



(a) 40% prestress (b) 60% prestress

Figure 2. Comparison of selected typical analytical and experimental results of set B1.



(a) 00% prestrss (b) 20% prestress

Figure 3. Comparison of selected typical analytical and experimental results of set B2.

5 DUCTILITY OF STRENGTHENED BEAMS

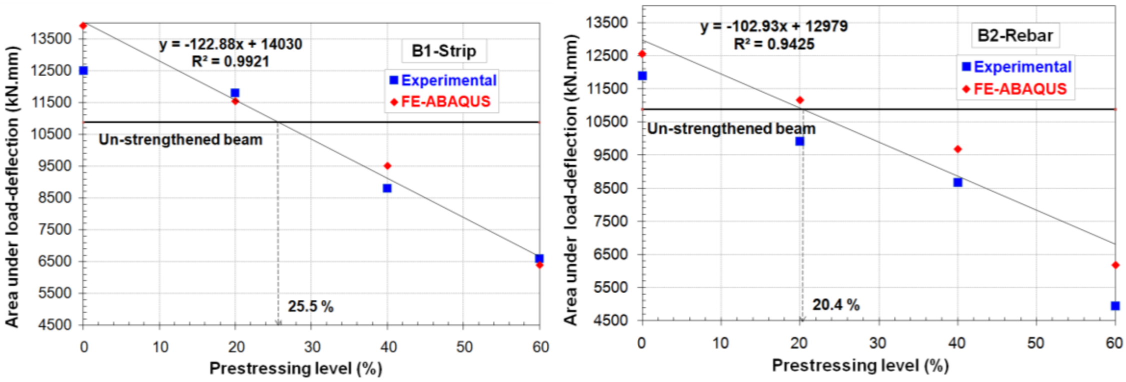
Two approaches have been used in the present study to address the ductility of the strengthened beams.

5.1 Energy based approach

Two methods were adopted to calculate the ductility based on the energy dissipated.

5.1.1 Total area under the curve

This method involves calculating the total area under the load-deflection curve. Figure 4 shows comparison of ductility between experimental and analytical results. It is seen that analytical results are in good argument with experimental results. However, slight differences are observed when comparing analytical results of set B2 with experimental results. As explained earlier, this is due to the fact that the prestressing is based on the force measured from the load cell resulted in lower prestressing force induced in the CFRP rebars as compared with prestressing based on strain measured from strain gauges. The ductility of the un-strengthened beam is also shown in Figures 4(a) and 4(b) in order to determine the optimum prestressing level at which the ductility of the prestressed strengthened beam is equivalent to that of the un-strengthened beam. This optimum level would allow utilizing the advantages of prestressing the CFRP reinforcement, enhancing the serviceability and deformability of the RC beam, while maintaining the amount of energy dissipated up till failure equivalent to that of the un-strengthened beam. Determining such a prestressing level is beneficial when considering the seismic performance of the strengthened beams. Best fit lines are drawn in Figures 4(a) and (4)b considering the ductility of the analytical results. It is observed that as the prestressing level increases, the ductility of the beams decreases in almost a linear manner. This is approved by the high correlation coefficients indicated in the figure. The intersection of the best fit line with the line representing the ductility of the un-strengthened beam indicates the optimum prestressing level. This level is found to be 25.5 and 20.4% of the ultimate tensile strength of the CFRP strips and rebars for sets B1 and B2, respectively. The higher optimum prestressing level of beams strengthened with strips compared with beams strengthened with rebars is attributed to the higher flexural stiffness of the strips, higher moment of inertia.



(a) Set B1 (b) Set B2

Figure 4. Total energy comparison of analytical and experimental results of sets B1 and B2.

5.1.2 Ratio of the elastic energy and the total energy

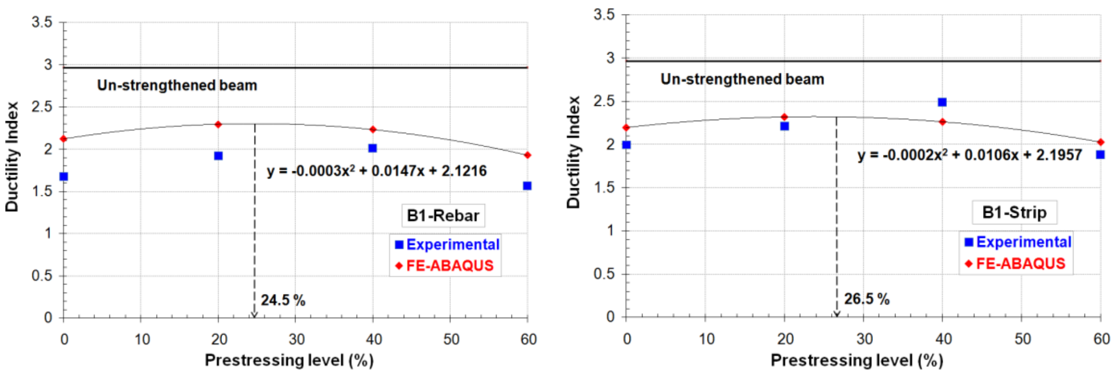
Naaman and Jeong (1995) proposed the following equation to calculate the ductility index based on the energy dissipated:

$$Ductility \quad Index = \frac{1}{2}\left(\frac{E_{total}}{E_{elastic}} + 1\right) \tag{3}$$

where $E_{total} = E_{elastic} + E_{inelastic}$, $E_{elastic}$ is the amount of energy that is recovered, $E_{inelastic}$ is the plastic energy, $s = \frac{P_{cr} s_1 + (P_u - P_{cr}) s_2}{P_u}$, P_{cr} is the cracking load, P_u is the ultimate load, s_1 is the slope up to the cracking load, and s_2 is the slope from cracking load to ultimate load.

Figure 5 shows comparison of ductility indices, calculated using Eq. 3, of sets B1 and B2. The analytical results are in good agreement with the experimental results. It is seen that the ductility index follows a polynomial pattern with respect to the change in the prestress level. The optimum prestress level at which the ductility index is maximum, 2.3, is 26.5% of the ultimate tensile strength of the CFRP strips for set B1 while the optimum prestress level at which the ductility index is maximum, 2.3, is 24.5% of the ultimate tensile strength of the CFRP rebars for set B1. Both the maximum ductility indices of sets B1 and B2 are less than that of the un-strengthened beam.

The percentage difference between optimum prestress levels calculated based on total area under the curve method and the ratio of the elastic energy and the total energy method is 3.85 and 18.21% for sets B1 and B2, respectively. It should be noted that an equilibrium between the unstrengthened and the strengthened beams is established using the total energy method. However, the equilibrium is not achieved using the ratio of elastic energy method. As a conclusion both methods predicts the optimum prestress level at which the ductility of the prestressed strengthened beam is equivalent to that of the un-strengthened beam even though the ratio method fails to achieve ductility equilibrium.



(a) Set B1 (b) Set B2

Figure 5. Ratio of elastic energy and total energy comparison of analytical and experimental results.

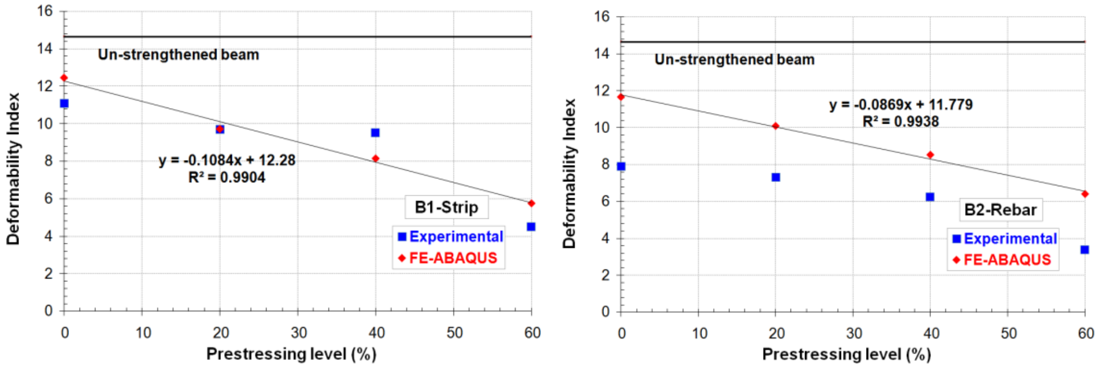
5.2 *Deformation based approach*

The deformability index is calculated based on the following equation developed by Jaeger et al., (1995):

$$Deformability\ Index = \frac{\Delta_u}{\Delta_x} \tag{4}$$

where Δ_u is the deflection at ultimate load and Δ_x is the deflection of the un-cracked section at a load equal to the ultimate load.

Figure 6 shows comparison of deformation indices, calculated using Eq.4, of sets B1 and B2. It is seen that the indices follow a linear trend whereby an increase in the prestress level leads to a decrease in the deformability of the strengthened beams. No optimum prestress levels can be found using this approach of the strengthened beams since the deformability indices are all lower than the deformability index of that of the un-strengthened beam and no parabolic trend is observed. Thus, it is concluded that the energy based approach provides a better measure of the optimum prestress level as compared with deformation based approach.



(a) Set B1 (b) Set B2

Figure 6. Deformability comparison of analytical and experimental results.

6 CONCLUSIONS

This paper presents a 2D finite element model developed to simulate the flexural behavior of RC beams strengthened using prestressed NSM CFRP reinforcements. The good agreement found between the experimental and analytical results indicates the validity of the proposed model. The following conclusions can be drawn: the ductility of beams strengthened using prestressed CFRP strips and rebars decreases linearly with the increase of the prestressing level, To maintain the ductility of the prestressed beam is equal to that of the un-strengthened, the optimum prestressing level using the total energy method is 25.5 and 20.4% of the ultimate tensile strength of the CFRP reinforcement for beams strengthened using strips and rebars, respectively, However, the optimum prestressing level, to achieve the maximum ductility on the prestressed beams, using the ratio of the elastic energy and the total energy method is 26.5 and 24.5% of the ultimate tensile strength of the CFRP reinforcement for beams strengthened using strips and rebars, respectively. The deformation based approach fails to predict the optimum prestress level. Using strips increases the ductility of the strengthened beams as compared with rebars due to its higher flexural stiffness.

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