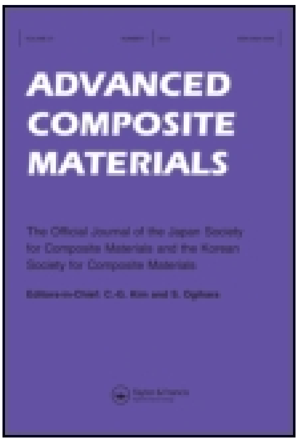

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Reinforced concrete beams externally retrofitted with advanced composites

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Abstract—This study investigates the effect of externally bonded carbon fiber reinforced polymer (CFRP) laminates on the ductility of reinforced concrete beams used in the repair of damaged bridge structures. Reinforced concrete structures deteriorate over time due to environmental aging, fatigue, excessive loading, chemical attack, and other factors. Strengthening and rehabilitating these concrete structures by externally bonding carbon laminates is one of many economical engineering solutions. Eight rectangular beams with varying internal steel reinforcement were retrofitted with CFRP strips on the tension faces and tested under four-point bending. The beams were instrumented to monitor strains, deflection, and curvature over the entire spectrum of loading, and determine the structural response of the beams. An existing analytical model using the discrete yield and ultimate values of the load–deflection and moment–curvature curves was modified to an energy-based model, and used to predict the ductility of the beams. Numerical results indicated an increase in strength, a decrease in ductility, and validated the analytical model. Ultimately, this study will aid in the development of design guidelines governing the use of CFRP.

Keywords: Reinforced concrete beams; retrofit; civil infrastructure; repair; CFRP; advanced composite materials.

1. INTRODUCTION

The application of fiber reinforced polymer (FRP) as external reinforcement to concrete infrastructure repair has received more attention from the Civil Engineering research community than any other engineering concern. The majority of FRP research related to civil infrastructure is conducted in Europe and Japan where design codes governing FRP repair already exist. However, this trend is changing slowly with an increasing number of American researchers joining their European and Japanese counterparts in the investigation. Due to this increasing acceptance of

FRP as external reinforcement, the American Concrete Institute (ACI) recently has formed Committee 440H tasked to develop a set of guidelines for design with FRP.

In the past two decades, several types of structures, including columns, slabs, and beams, have been retrofitted and tested with FRP. Researchers have reported improvements in strength and stiffness of retrofitted members with some studies indicating that the shear and flexural strength of reinforced concrete beams can be increased by 20% to 100% with retrofitted CFRP sheets [1]. Many of the studies on strengthening effects have been both analytical and experimental investigations.

2. BACKGROUND

Extensive work has been conducted on the short-term response of concrete beams strengthened in flexure with FRP plates [2, 3]. From these and other results, a pivotal belief was formed that given the superior properties of CFRP over other composites, CFRP offers the highest potential for strengthening concrete structures in most cases [3, 4]. Studies also have confirmed that the use of epoxy for externally bonding the FRP reinforcement is more advantageous than typical mechanical bonding methods since nuts, bolts, and steel plates are vulnerable to environmental corrosion [5]. Other studies identified mid-span deflection and strain over the entire load spectrum as critical parameters for evaluating the response of a strengthened member [6]. Strain profiles can be used to calculate the stresses at various locations and are necessary to validate the assumptions of reinforced concrete beam flexure theory [7]. The four-point bending test has been shown to be the most appropriate for testing these types of hybrid beams [6, 8].

A critical evaluation of the research data shows that there are still many aspects of material and structural behavior arising from the use of FRP that are not yet clearly understood. One such issue is the ductility of retrofitted reinforced concrete beams. Several recent studies, including one by the authors, have examined this topic [9–11]. This paper outlines the analytical and experimental study performed by the authors.

3. EXPERIMENTAL METHODOLOGY

A series of eight reinforced concrete beams were designed to represent typical in-service reinforced concrete beams. Standard construction techniques were closely followed and the quality of the constructed beams closely mirrored normal construction standards. All beams had identical nominal dimensions. Each beam was 2.896 m in length, 1500 mm in width, and had a 4500 mm nominal depth. The internal longitudinal flexural reinforcement was varied for the beams from #4 to #9 re-bar, but the vertical shear reinforcement had a constant stirrup arrangement designed to minimize the possible occurrence of a transverse shear-induced failure (see Table 1). Varying the internal reinforcement while maintaining constant CFRP

Table 1.
Beam specimen designation and details

Beam	Longitudinal reinforcement				CFRP type	No. of strips
	Number of re-bars	Size of re-bars	Area of steel (mm^2)	Reinforcement ratio (%)		
4A	2	#4	258	0.404	S512	1
4B	2	#4	258	0.404	S812	1
5	2	#5	400	0.626	S512	1
6	2	#6	568	0.889	S512	1
7	2	#7	774	1.212	S512	1
8	2	#8	1020	1.596	S512	1
9A	2	#9	1290	2.020	S512	0
9B	2	#9	1290	2.020	S512	1

properties provided a mechanism for accessing the effect of the bonded sheet on the performance of the hybrid beam.

Each beam was tested under four-point bending. Unique to this test configuration was that the beam was mounted so that its tension face was located on the top rather than the bottom. This allowed ease of access to this surface for the *in-situ* repair. The constant moment region was three feet under this configuration. This region was more than adequate for strain and displacement instrumentation.

Each beam was instrumented with three vertical linear variable displacement transducers (LVDTs) in contact with the tension face: one at mid-span and two directly under the outer load points. These LVDTs provided the essential data for computing the curvature of the beam. In addition, the concrete beam, CFRP sheet, and steel re-bars were instrumented with strain gages. Recording the strain measurements in these materials allowed one to check the strain compatibility and validate Hooke's Law. Finally, eight LVDTs were mounted horizontally on the sides of each beam, at the mid-span, and provided measurements to compute through-the-thickness strain distribution.

Testing consisted of several steps. First the beam was loaded to initial cracking. After reaching this load, the CFRP sheet was bonded to the tension face of the beam. The bonding of the sheet was performed under load. Loading then was resumed until visible failure of each beam was reached. The parameters of interest included mid-span deflection, strains in concrete and CFRP laminate sheets, and ultimate moment capacity. A digital data-acquisition system was used to monitor loading, mid-span deflection, and deformations in the concrete and in the reinforcement.

Figures 1 and 2 represent load–deflection and moment–curvature results for Beam 4A. For this beam, the maximum bending moment at failure was computed as 102.26 kN·m. Similar results were determined for the remaining beams.

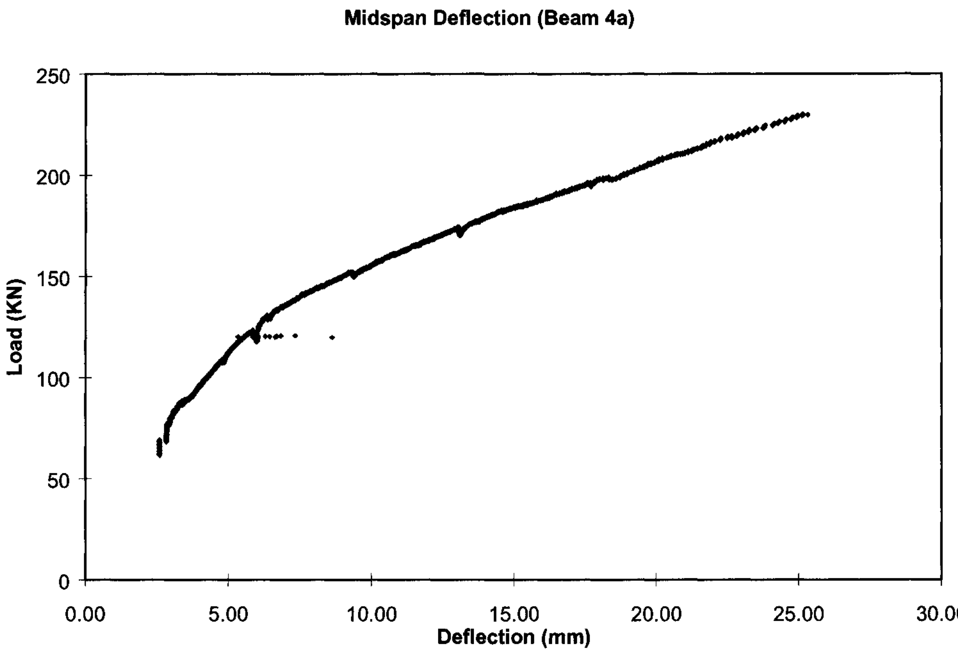

Figure 1. Typical load–deflection curve.

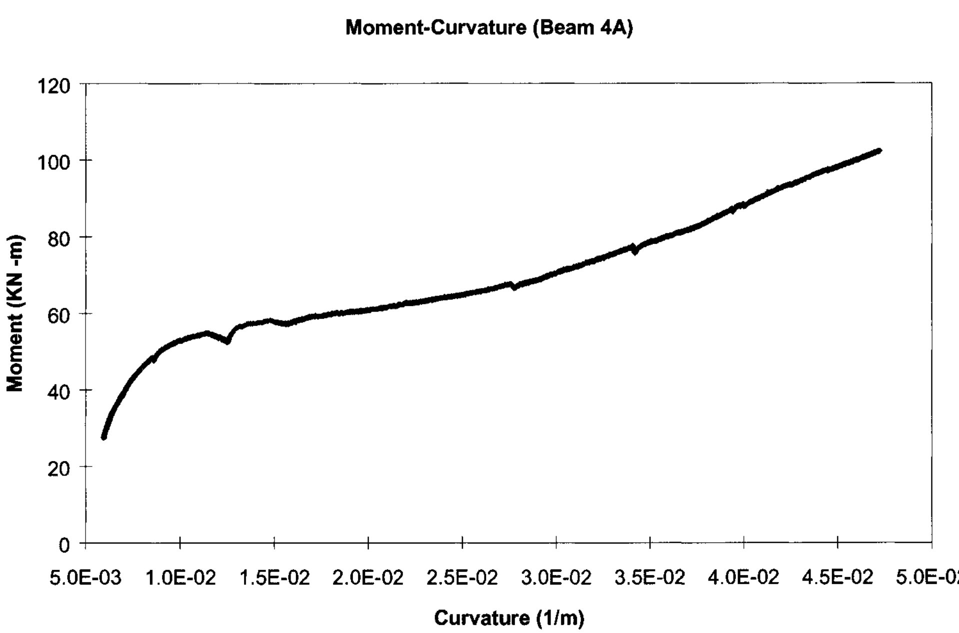

Figure 2. Typical moment–curvature curve.

4. ANALYTICAL METHODOLOGY

In order to predict the mode of failure, the reinforcement ratio, ρ, needs to be calculated. As prescribed by ACI 318-95 10.2.7.3, the reinforcement ratio is given by:

$$\rho = \frac{A_s}{bd}, \tag{1}$$

where A_s is the area of steel, b is the beam nominal width, and d is the beam effective depth. However, this definition for ρ is inadequate for reinforced concrete beams with externally bonded CFRP sheets. For these beams, one must account for the area of the CFRP. Therefore, introduce A_c. Now the reinforcement ratio becomes ρ^*, and is termed the adjusted reinforcement ratio. It accounts for the difference between the ultimate strength of CFRP and the yield strength of steel by adjusting the area of steel used in the calculation of the reinforcement ratio. The adjusted area of steel, A_s^*, should be calculated as follows:

$$A_s^* = A_s + n\alpha A_c. \tag{2}$$

The compositeness reduction factor, α, indicates the degree of composite action predicted for the externally applied CFRP laminate, and was chosen to be 0.7 in this study. The composite-to-steel weighting factor, n, is given by:

$$n = \frac{f_{\text{ult composite}}}{f_{\text{y steel}}}, \tag{3}$$

where $f_{\text{y steel}}$ is the steel yield strength and $f_{\text{ult composite}}$ is the composite ultimate strength. Replacing the area of steel with the adjusted area of steel gives ρ^* as:

$$\rho^* = \frac{A_s^*}{bd}. \tag{4}$$

In order to predict the ductility of the beams, an existing analytical model by Spadea *et al.* [7] using the discrete yield and ultimate values of the load–deflection ($P-\Delta$) and moment–curvature ($M-\phi$) curves was modified to an energy-based model. The modified model evaluates the ductility in terms of the energy at these two points. The energy is the area under the curves at these points. The modified deflection ductility index, μ_Δ calculated from the load–deflection curve and modified curvature ductility index, μ_Φ calculated from the moment-curvature curve are given as:

$$\mu_\Delta = \frac{\int_0^{\text{ultimate}} P(\Delta)\mathrm{d}\Delta}{\int_0^{\text{yield}} P(\Delta)\mathrm{d}\Delta}, \tag{5}$$

$$\mu_\phi = \frac{\int_0^{\text{ultimate}} M(\phi)\mathrm{d}\phi}{\int_0^{\text{yield}} M(\phi)\mathrm{d}\phi}. \tag{6}$$

5. DISCUSSION AND RESULTS

The results of the reinforcement ratio calculations and the predicted modes of failure for the specimens are shown in Table 2. The modes of failure were predicted by comparing the adjusted reinforcement ratio, ρ^*, to the maximum design reinforcement ratio, $\rho_{\max}$, as specified by ACI Code. (Note: $\rho_{\max}$ is 75% of balanced reinforcement ratio, ρ_{balanced}.) Since ρ^* did not exceed $\rho_{\max}$ for any of the beams, a typical tension failure by yielding of steel was predicted for the beams. These predicted modes compared well to the observed failure modes given in Table 2. The only difference was found in the results of Beam 9B. Beam 9B displayed a semi-brittle collapse that can be termed as a tension/balanced failure. These data, more than any others, show the possible dangers associated with CFRP retrofit. As a result of adding CFRP, Beam 9B experienced a change in failure mode from tension to balanced. This is an undesirable sudden concrete compressive failure.

Computed ductility indices for deflection and curvature cases also are given in Table 2. As expected, the control beam, 9A, which displayed a typical tension failure, had a ductility index greater than 1. Beam 9B experienced an almost balanced failure and correspondingly had a lower ductility index. Beams 4A and 4B, which had the lowest laminated reinforcement ratios, displayed the most ductile behavior. These beams showed increases in flexural capacity while retaining significant ductile qualities.

As reinforcement ratio increases, the ductility indices decrease. This is because the beam is becoming stiffer as more steel or carbon is added and after the steel has

Table 2.
Experimental and analytical results

Beam	ρ^*	$\rho_{\max}$	Predicted failure	Observed failure	μ_Δ	μ_ϕ
4A	0.0078	0.0283	Tension	Tension	5.67	10.2
4B	0.0102	0.0283	Tension	Tension	5.63	7.63
5	0.0101	0.0283	Tension	Tension	3.50	8.94
6	0.0127	0.0283	Tension	Tension	3.77	8.24
7	0.0159	0.0283	Tension	Tension	—	7.81
8	0.0198	0.0283	Tension	Tension	2.55	2.82
9A	0.0202	0.0283	Tension	Tension	1.93	2.05
9B	0.0240	0.0283	Tension	Tension/Balanced	1.60	1.59

yielded, this stiffness limits the amount of deflection and curvature possible. The reinforcement ratio is approaching the balanced condition when no deflection after yield occurs.

Another observation shows that as the laminate percentage of total reinforcement increases, the ductility indices decrease. The CFRP laminate accounted for 23.3% and 37.3% of the total tensile reinforcement in Beams 4A and 4B, respectively, while for Beams 9A and 9B, the CFRP contribution was 0% and 4.7%, respectively. In both cases, the increases in CFRP contribution corresponded to a decrease in ductility.

Deflection and curvature ductility indices appear to follow similar trends. The curvature ductility index is greater than the deflection ductility index for all beams except Beam 9B, where the values are essentially the same. The differences in these two ductility indices range from as low as 10.85% for Beam 9A to a high of 60.85% for Beam 5. This shows the unpredictable behavior of retrofitted beams and reinforces the need for an increased understanding of the behavior of such beams.

6. CONCLUSIONS

Several conclusions were drawn from this research: (1) bonding of CFRP composites to damaged reinforced concrete beams is a viable technology for repair; (2) strengthening technology is easy to perform and results in significant improvements in ultimate load capacity; (3) existing ACI guidelines do not accurately model the experimental behavior of CFRP retrofitted concrete beams; however, minor modifications to account for the strength and constituent behavior of CFRP enable reasonably accurate predictions to be made; (4) energy-based definition of ductility better assesses the ductile behavior of reinforced concrete beams than traditional discrete-based definitions; and (5) the use of CFRP as external reinforcement reduces the ductility of an under-reinforced concrete beam, but it is possible to design beams with FRP that still exhibit ductile failure by ensuring that the reinforcement ratio of the retrofitted beam does not exceed the balanced reinforcement ratio.

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