REPAIRING AND STRENGTHENING REINFORCED CONCRETE STRUCTURES USING FIBER-REINFORCED PLASTICS

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SUMMARY: This study reports on experimental and theoretical investigations of the behavior of concrete members that have been strengthened or repaired using externally bonded advanced composite materials. The experimental work begins with an investigation of the tensile properties of glass GFRP sheets, and proceeds to study concrete cylinders and columns strengthened using GFRP sheets, and beams repaired using GFRP sheets. Preliminary design formulas are developed to anticipate the loading capacity of the repaired members. The experimental and theoretical results are in good agreement. The results for beams indicate that GFRP, bonded externally to concrete members, improves both their strength limit state, in the form of increased flexural and shear load capacities, and their serviceability limit state, in the form of reduced cracks. The results for columns indicate that GFRP significantly increases the strength and ductility of reinforced concrete circular columns.

KEYWORDS: Concrete**,** Repair, Glass Fiber Reinforced Plastic (GFRP).

INTRODUCTION

The backbone of a nation is made up of constructed facilities that include public buildings, airports, highways, and other types of infrastructure. Many such structures suffer from continuous deterioration. A reliable system to maintain the structural integrity and extend the life of constructed facilities would save an enormous amount of money in repair costs. The emergence of high-strength resins has made possible the strengthening and repair of concrete members using externally bonded fiber-reinforced plastic sheets. The aim of the work reported here is to investigate external strengthening and repair of concrete cylinders, beams, and columns using glass fiber reinforced plastic (GFRP) sheets. GFRP sheets have superior properties, including a high ratio of strength to weight and a high resistance to chemical attack, and they are

also noncorroding. The properties of GFRP used in this research are given in Table 1. Values in the table represent testing that was not parallel to the strong fiber direction.

Table 1: Properties of fiber reinforced plastic materials used in this research

GFRP	Ultimate Stress (N/mm2)	Modulus (N/mm2)	Ultimate Strain (%)
0/90 GFRP	410	20600	2

1 TESTS ON CONCRETE CYLINDERS

A summary of tests performed on concrete cylinders appears in this section.

1.1 Casting of Cylinders

The concrete used for casting the cylinders was designed to have an ultimate compressive strength of 160 Kg/cm^2 at 28 days. Very low-strength concrete was used to simulate damaged concrete.

1.2 Compression Tests on Concrete Cylinders

The compressive strengths of the control cylinders and wrapped cylinders are listed in Table 2. The failure of all the wrapped concrete cylinders initiated by the crushing of the concrete was followed by failure of the GFRP jacket at higher loads. Fig. 1 shows the cylinders at failure. The cup and cone failure shows that the failure is ductile.

Table 2: Experimental results of the compressive strength of concrete cylinders

Cylinder No.	No. of GFRP Layers	Compressive Strength of Concrete Cylinders (Kg/cm^2)
C1	0	185
C2	0	190
C3	0	180
C4	1	400
C5	1	410
C6	1	405
C7	2	600
C8	2	605
C9	3	900

Fig. 1: Concrete cylinders at failure

2 TESTS ON REINFORCED CONCRETE BEAMS

This section reports on experimental and theoretical investigation of reinforced concrete beams. Design formulas are developed to anticipate the loading capacity of the repaired beams.

2.1 Design and Casting of Concrete Beams

The concrete beams were designed and cast with steel reinforcement at far below the maximum limit (As/bd=0.006) to allow for external reinforcement using FRP without causing a brittle compression failure. The dimensions of the beam were 15 cm wide by 20 cm deep by 115 cm long. The tested span was 100 cm. The internal reinforcement consisted of 2 φ 10 mm steel bars (10 mm in diameter, with 4200 Kg/cm^2 yield stress). A shear reinforcement of 7 φ 6 mm stirrups ran along the whole length of the beam. Fourteen batches of concrete, one for each beam, were needed to fabricate the beams. Cast with every beam were concrete test cylinders whose strength was 300 Kg/cm^2.

2.2 Preloading

The first two beams (Control 1, Control 2) were loaded to failure. Beams BM1 and BM2 were not preloaded. The remaining beams (beams BM3, BM4, BM5, BM6, BM7, BM8, BM9, BM10, BM11, BM12) were loaded to 85 percent of their ultimate capacity and then strengthened using GFRP sheets. All beams were simply supported on a clear span 100 cm long and subjected to two concentrated loads placed 17 cm apart.

2.3 Repair and Testing of Damaged Beams

One layer of GFRP was bonded to the tension side of beams BM3 and BM4, with the main direction of the fibers oriented along the beam length. Beam BM5 was strengthened using one U-shaped layer with the fibers oriented transverse to the span direction. Beams BM6, BM7, and BM8 were strengthened using one GFRP layer at the bottom and one U-shaped layer. Similarly, beam BM9 was strengthened using one bottom layer and one U-shaped layer, but the U-shaped

layer was cut between the two loads because this is a zero shear zone. Beam BM10 was strengthened using two bottom layers and one U-shaped layer. Beam BM11 was strengthened using three separate layers: one layer of GFRP was bonded to the tension side and two separate layers were bonded to the two sides of the beam with the main direction of the fiber along the length of the beam. Finally, Beam BM12 was strengthened using two bottom layers and two U-shaped layers. Tables 3 and 4 summarize the reinforcement of the beams and the experimental test results.

Table 3: Reinforcement of beams using external GFRP sheets

Beam	Preloading Level	Bottom Layers	U-Shaped Layer	Side Layers
Control 1	-	0	0	0
Control 2	-	0	0	0
BM1	-	1	0	0
BM2	-	0	1	
BM3	85%	1	0	0
BM4	85%	1	0	0
BM5	85%	0	1	0
BM6	85%	1	1	0
BM7	85%	1	1	0
BM8	85%	1	1	0
BM9	85%	1	1	0
BM10	85%	2	1	0
BM11	85%	1	0	2
BM12	85%	2	2	0

Table 4: Experimental test results and mode of failure of beams

Beam	Experimental Failure Load P_{ult} (ton)	* P_{ult} experimental % P_{ult} control	Failure Mode
Control 1	7.1	0%	Shear
Control 2	7.2	0%	Shear
BM1	8.5	18%	Shear
BM2	14.4	104%	Flexural
BM3	8.5	18%	Shear
BM4	9.5	31.9%	Shear
BM5	9.84	36.6%	Flexural
BM6	14.5	104%	Flexural
BM7	14.3	104%	Flexural
BM8	14.7	104%	Flexural
BM9	14.5	101%	Flexural
BM10	15.58	116%	Shear
BM11	11.4	58%	Shear
BM12	18	150%	Compression shear

*Maximum load divided by control load, which is 7.2 tons in percent.

The results of the tests on the reinforced beams appear in Table 4. The repaired beams showed a logical progression of failure modes. Adding one layer of GFRP to the tension face of the beam increased both the flexural and the shear capacities of the beam. For example, Beam BM3 failed at 8.5 tons in shear; Beam BM4, with an additional layer of GFRP, also failed in shear, but at 9.5 tons. The increase in the flexural capacity occurred because the bottom GFRP layer acts as additional reinforcement. The shear capacity of the beam increased when one layer of GFRP was added to the tension side of the beam because the bottom GFRP layer increased the resistance of the concrete to shear by increasing the dual action and the shear interlocking. Adding a U-shaped layer with the main fibers orientated transverse to the span direction of the beam prevented the shear failure; however, a flexural failure of Beam BM5 took place at 9.84 tons because the steel had reached its ultimate tensile capacity. The U-shaped layer increased the shear loading capacity by providing shear resistance similar to that provided by stirrups and by an increase in the shear interlocking. Wrapping the beams with both a U-shaped layer and a longitudinal layer prevented the shear failure, but a flexural failure due to the failure of both the steel and the composite–followed by a compression failure–took place at 14.5, 14.3, and 14.7 tons for Beams BM6, BM7, and BM8, respectively.

In order to save material, a longitudinal layer was used with a U-shaped layer that was cut between the two loads (beam BM9) because this is a zero shear zone and therefore there is no need for shear reinforcement. The results of this experiment indicate that saving material had no effect on the shear capacity of the beam.

Repairing the beam using a single U-shaped layer and two longitudinal layers prevented flexural failure because the longitudinal reinforcement was increased to two layers, and a shear failure of Beam BM10 took place at 15.58 tons. Repairing the beam using a longitudinal layer and two separate layers bonded to both sides of the beam, with the main direction of the fiber oriented along the beam length, prevented flexural failure, and Beam BM11 failed in shear at 11.4 tons. The two separate side layers improved the shear loading capacity of the beam to 11.4 tons, while a single U-shaped layer increased the shear capacity to around 14.5 tons. This result emphasizes the importance of analysis before repair because, if there is no need for a very high shear loading capacity, two separate layers can be used to save effort and cost.

Finally, repairing the beam using two longitudinal layers and two U-shaped layers prevented, as expected, both shear and flexural failures, and Beam BM12 failed in compression at 18 tons. The ultimate capacity of beams BM1 and BM2 clearly show that preloading does not affect the maximum loading capacity. The load versus deflection curves are shown for some beams in Fig. 2. Beams BM3 and BM7 are shown at failure in Figs. 3 and 4, respectively.

2.4 Simple Design Formulas for Predicting Beam Loading Capacity

This section presents simple formulas for calculating the experimental ultimate flexural moment and shear capacities of the test beams. To calculate the ultimate moment capacity, it was assumed that failure occurs after the tensile steel yields and after the longitudinal GFRP reinforcement reaches its tensile capacity. When the resultant tensile forces that acts on a cross-section of the beam are set equal to the compression forces, the beam's moment capacity can be expressed, according to the 1996 Egyptian code of practice, as follows:

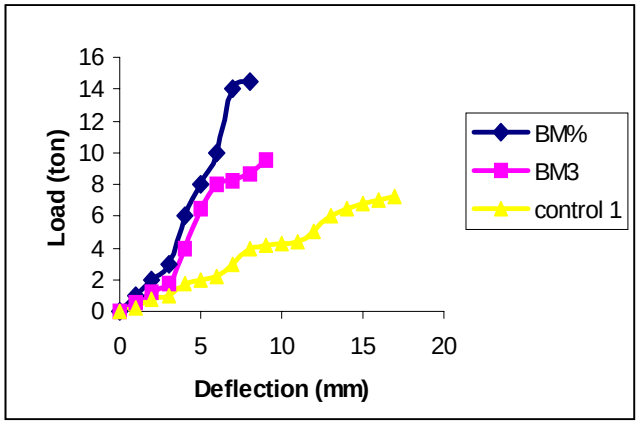

Fig. 2: Mid-span load deflection curve for beams

Fig. 3: Beam BM3 at failure

Fig. 4: Beam BM7 at failure

$$M_u = 0.87\ A_s\ F_y\ (d\text{-}a/2) + 0.87\ A_f\ F_f\ (g\text{-}a/2) \qquad (1)$$

where A_s is the cross-sectional area of the tensile steel, F_y is the steel's yield strength, d is the distance from the steel reinforcement to the top of the beam, A_f is the cross-sectional area of the GFRP longitudinal reinforcement, F_f is the GFRP's ultimate strength, and g is the distance of the GFRP from the top of the beam. Finally, a is given by:

$$a = \frac{1.5}{0.67 F_{cu} b}\left[0.87(A_s F_y + A_f F_f)\right] \qquad (2)$$

where F_{cu} is the cube compressive strength of the concrete and b is the beam's width. Because 0.87 is a factor usually used to account for long term deterioration and other errors, it will be removed from the equations because the beams used for these experiments are still fresh; therefore, the equations can be written as:

$$a = \frac{1.5}{0.67 F_{cu} b}\left[(A_s F_y + A_f F_f)\right] \tag{3}$$

and

$$M_u = A_s F_y (d-a/2) + A_f F_f (g-a/2) \tag{4}$$

The shear capacity Q_u of the beam can be expressed as:

$$Q_u = Q_c + Q_s + Q_f \tag{5}$$

where Q_c, Q_s, and Q_f are the contributions of the concrete, steel, and FRP, respectively. According to the 1996 Egyptian code of concrete practice:

$$Q_c = 0.75/2(\sqrt{F_{cu}}/\delta_c) \text{ x } bd \tag{7}$$

where Fcu is the compressive strength of concrete and δ_c is 1.5; further:

$$Q_s = (A_s/S) \text{ x } (F_y/\delta_s/b) \text{ x } bd \tag{8}$$

where A_s is the cross-sectional area of steel stirrups, S is the spacing between stirrups, F_y is the steel yield strength, δ_s=1.15, and b is the beam width.

In work by Al-Sulaimani, et al., cited by Chajes, et al. [1], the contribution to the total shear capacity made by FRP plates bonded to a beam web can be approximated by:

$$Q_f = (2\,\tau\, d\, g/2) \tag{9}$$

where τ is the interface shear strength**,** which is taken to be 28 Kg/cm2.

Equation (4) was used to predict the ultimate experimental moments. The theoretical predictions were found to be conservative by around 16%; see Table 5.

Table 5: Theoretical versus experimental flexural beam capacity

Beam	M_{ult} Experimental Kg.cm	M_{ult} Theoretical Kg.cm	$\frac{M_{ult}\text{ experimental}}{M_{ult}\text{ theoretical}}$ %
B4	295437.5	247234.6	16.3%
B5	291362.5	247234.6	15.1%
B6	299512.5	247234.6	17.4%

B7	295437.5	247234.6	16.3%

The difference between the experimental and the calculated ultimate moments may be attributed to uncertainty about the behavior of concrete; or to the experimental error in determining F_f, the estimated value of lamination strength; or to code factors.

Equation (8) was used to determine the theoretical shear capacity of beams repaired using GFRP. It was calculated that Q_f = 2x28x18x(20/2) = 10.08 tons. From the experimental work, the shear capacity of both the concrete and the steel was 7.1 tons (Control 1). Thus, experimentally, the load capacity of GFRP is 8.84 tons. This shows that the equation is conservative by about 15%.

3 TESTS ON CIRCULAR COLUMNS

Most research to date into the repair and strengthening of reinforced concrete structures has involved testing standard cylinders with an aspect ratio of approximately 2:1 [2,3,4,5,6,7]. Demers, et al. [5], and Harris, et al. [7], conducted investigations on the axial behavior of reinforced concrete columns confined with FRP jackets; both studies reported increases in the axial load-carrying capacity and axial strain. In the present work, an experimental program was undertaken to investigate further the behavior of concrete columns strengthened with FRP.

3.1 Experimental Setup, Tests, and Results

Six reinforced concrete columns (165mm in diameter and 914 mm high) were tested under a monotonic axial load. The column specimens contained 6#3 (9.5 mm in diameter) vertical bars with #2 (6.35 mm in diameter) ties placed at 15 cm. The results of the tests are shown in Table 6. They illustrate that externally-affixed GFRP jackets greatly improve the compressive strength of circular concrete columns. One 0/90 GFRP layer increased the compressive strength by around 130%. Two 0/90 GFRP layers increased the compressive strength by 270%. It can be seen that an increase in compressive strength is not directly proportional to an increase in the number of GFRP layers. The wrapping of circular columns creates axial hoop tension and provides a uniform confining pressure around the circumference. Results of tests on circular columns appear in Table 6.

Fig. 5 shows columns Col 3 and Col 4 at failure, and Fig. 6 shows the stress strain curve of circular columns strengthened using GFRP.

Table 6: Summary of results on circular reinforced concrete columns

Circular Column	Diameter (mm)	No. of FRP 0/90 Layers	Ultimate Strength (KN)	P_{ult} exp/P_{ult} control (%)
Control 1	16.5	0	410	-
Control 2	16.5	0	409	-
Col 3	16.5	1	930	227%
Col 4	16.5	1	910	222%
Col 5	16.5	2	1430	349%
Col 6	16.5	2	1425	348%

Fig. 5: Columns Col 3 and Col 4 at failure

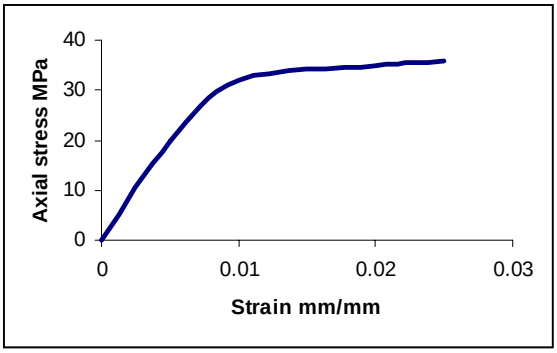

Fig. 6: Axial behavior of confined circular columns

4 CONCLUSIONS

The experimental and theoretical study of the strengthening of cylinders and circular columns using GFRP shows that both the strength and the ductility of circular columns can be increased significantly to any practical design limit and need.

The relationship between an increase in the number of confining layers and an increase in the compressive strength is not directly proportional.

Control cylinders have one plane of failure; however, the GFRP confined cylinders had several planes of failure (cup and cone, for example).

GFRP wrapping of beams was found to improve the shear and flexural loading capacities of damaged beams. In these experiments, the repaired beams exceeded their original loading capacities in flexural and shear by values of up to 150%.
The closeness of the results obtained using equations derived from the Egyptian code, for beams strengthened using GFRP, and those obtained experimentally means that the code-derived equations can be used in designing GFRP externally-reinforced beams.

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