USE OF GLASS FRP SHEETS AS EXTERNAL FLEXURE REINFORCEMENT IN RC BEAMS

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ABSTRACT

Experimental data of strengthened or repaired (retrofitted) reinforced concrete (RC) specimens are presented in this study. Retrofitting was attained with the adhesion of glass fiber reinforced polymer (GFRP) sheets to the tension side of the beams. The flexural capacity and central deflection of the strengthened groups of beams were compared with those of their counterpart specimens of the control group to evaluate the effectiveness of strengthening or repairing techniques. The results indicated that the flexural strength of RC beams can be increased by gluing GFRP sheets to the tension face. The increases in the load capacities of the beams due to retrofitting are discussed at service, yield and ultimate load levels. Furthermore, the influence of the retrofitting on the structural ductility and mode of failure are also presented. An analytical model is developed to predict the strength and deflection of RC beams strengthened with FRP sheets. The accuracy of the model is checked by comparing the predicted results with the corresponding measured experimental results.

Keywords: beam, composites, FRP, laminates, plates, reinforced concrete, retrofitting, sheets, strengthening

INTRODUCTION

The prospect of engineering use of any material depends primarily on its stress-strain profile. Fiber reinforced polymer (FRP) materials offer higher strength than steel, but are less stiff than steel, except some types with high modulus such as carbon fiber reinforced polymer (CFRP). The higher strength can result in smaller dimensions of the plate or sheet reinforcement. From a structural point of view, however, absence of a yield plateau may cause a sudden brittle fracture of the plate or sheet, which may not be desirable or acceptable from the design point of view, and much more needs to be understood about the cracking and fracture processes in such materials. The higher strength of the sheet or plate reinforcement may lead to higher ultimate strengths of the sheeted (strengthened) composite member, but this needs to be matched with higher load factor than is

currently specified in building codes. The greatest advantage of FRP materials is that they are not susceptible to the type of rapid electrochemical corrosion that occurs with steel. In economic terms, steel is cheaper than FRP materials, but considering that in repair and strengthening work involving plate or sheet bonding, labor and operational costs often far out weigh material costs, lightweight FRP materials can not only substantially reduce costs, but also greatly minimize site inconveniences and handling problems.

The use of FRP plates or sheets to replace steel in strengthening applications was pioneered in Switzerland [1,2]. Experimental and analytical research work has also been carried out since the late 1980's at the university of Arizona [3], Lehigh university [4] and in Germany [5]. Furthermore, the behavior of RC beams with externally bonded FRP sheets has been investigated in the UK at Oxford Brookes University [6] and in other places [7-13].

Due to the relatively low elastic modulus, linear elastic behavior and limited strain capacity of FRP's, concrete members reinforced with FRP reinforcement exhibit a brittle failure mode. Hence, lack of ductility in such members is one of the key issues facing researchers. Ductility is commonly defined as the ability to undergo large plastic deformations prior to failure. Different approaches for achieving ductility which consists of combining a low modulus, medium strength, polymeric material with CFRP or GFRP as reinforcement are available in different studies [14,15].

The purpose of this paper is to provide experimental data on the response of RC beams strengthened in flexure using GFRP sheets. This includes different strengthening and repairing techniques. Results of the experiments are evaluated in terms of flexural capacity, central deflection, structural ductility and mode of failure. A comparison with the analytical results of predicting the load capacity and central deflection are also presented in order to evaluate the accuracy of the analytical model.

EXPERIMENTAL PROGRAM

Test Specimens

The test program consisted of twelve beams categorized into six groups. The test span of all beams were 2050 mm. The cross section was 150 x 200 mm. All beams were reinforced with 3ϕ10 mm steel bars in tension side (bottom) and 1ϕ6 mm in compression side (top). All beams were provided with ϕ8 mm steel stirrups @ 100 mm center to center. All beams were tested simply supported and were subjected to two points loads symmetrically placed at equal distance (100 mm) from the centerline of the

beam. Further details of the test set-up and cross-section of the beams are shown in Figs. 1 and 2, respectively. The beams of the 1st group (GB1) were tested with no strengthening or repair, considered as control specimens. The beams of the 2nd group (GB2) were strengthened (before loading) with one GFRP sheet at the bottom face of the beam. The 3rd group (GB3) was subjected to 40 kN (equivalent to the yield load of control beams), unloaded, and then repaired by one GFRP sheet at the bottom face of the beam and reloaded up to failure. The 4th and 5th groups (GB4, GB5) were strengthened similar to GB2 and GB3 respectively, but retrofitted using U-shape (GFRP) sheet. The 6th group (GB6) strengthened (before loading) with two layers of (GFRP) sheets at the bottom face of the beam. Figure 2 shows all groups of beams considered in this study.

Material Properties

Similar concrete mix was used for all beams. The proportions in the concrete mix were 1.0 (cement) : 1.4 (sand) : 2.1 (gravel) by weight. The water/cement ratio was 0.52 and type I portland cement was used. The average compressive strength was determined from concrete cylinders tested after 28 days of curing and given in Table 1. The average yield stress of main steel bars used in all experiments was 537 MPa and an elastic modulus of 200 GPa. One type of FRP sheet was used during the tests: a bidirectional FRP with the fibers oriented in both longitudinal and transverse directions.

The fiber-composite material consisted of glass bonded together with an epoxy matrix. The sheet was subjected to longitudinal tensile tests to determine elastic modulus and ultimate strength. The GFRP exhibited a linear elastic behavior up to failure. The method of testing utilized to determine the properties of GFRP sheets was performed according to ASTM D 3039-76 to evaluate the tensile properties of oriented fiber composites [16,17]. The test results gave an average ultimate strength of 600 MPa and elastic modulus of 30 GPa for the GFRP sheets. The construction epoxy adhesive used in bonding the GFRP sheets to the surface of the beam was of two-component cold-curing type. The ultimate tensile strength of the adhesive was about 25 MPa and the elastic modulus was 8.5 GPa. Other material properties and plating system are given in Table 1.

Preparation of Test Specimens

The GFRP sheets were bonded to the tension face of the specimens after 28 days of casting. Before applying the epoxy, the concrete surface was roughened and cleaned to insure a good bond between the epoxy glue and the concrete surface. The epoxy was hand-mixed and hand-applied at an

approximate thickness of about 1 mm. The bond thickness was not specifically controlled, but the excess epoxy was squeezed out along the edges of the sheet, assuming a complete epoxy coverage. More details about the methodology utilized to fix the GFRP sheets to the RC beams is discussed elsewhere [17,18].

TABLE 1

Beam test parameters and material properties

Beam ID	f_c' (MPa)	Tension reinf.	Yield stress, f_y (MPa)	Material Type	Sheet thickness (mm)	Strengthening system with GFRP sheets
GB1	35.5	3ϕ10	537	--	--	Control beams (No sheets)
GB2	36.0	3ϕ10	537	GFRP	1.0	One layer bonded to the tension side.
GB3	35.5	3ϕ10	537	GFRP	1.0	Beam loaded up to 40 kN then one layer bonded to the tension side.
GB4	36.6	3ϕ10	537	GFRP	1.0	One layer (U-shape) bonded to the tension side.
GB5	36.6	3ϕ10	537	GFRP	1.0	Beam loaded up to 40 kN then one layer (U- shape) bonded to the tension side.
GB6	33.8	3ϕ10	537	GFRP	2.0	Two layers bonded to the tension side.

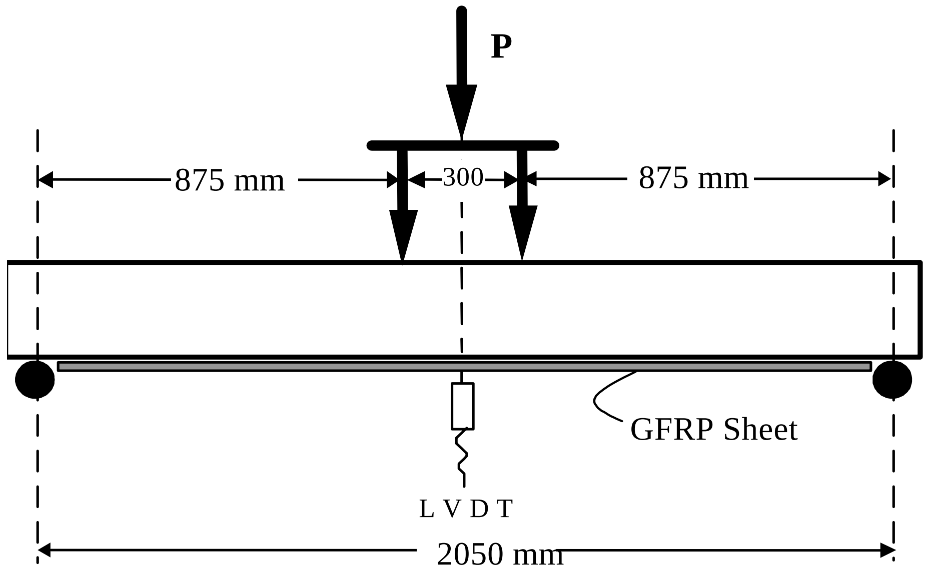

Fig. 1 The beam set up (dimensions are in mm)

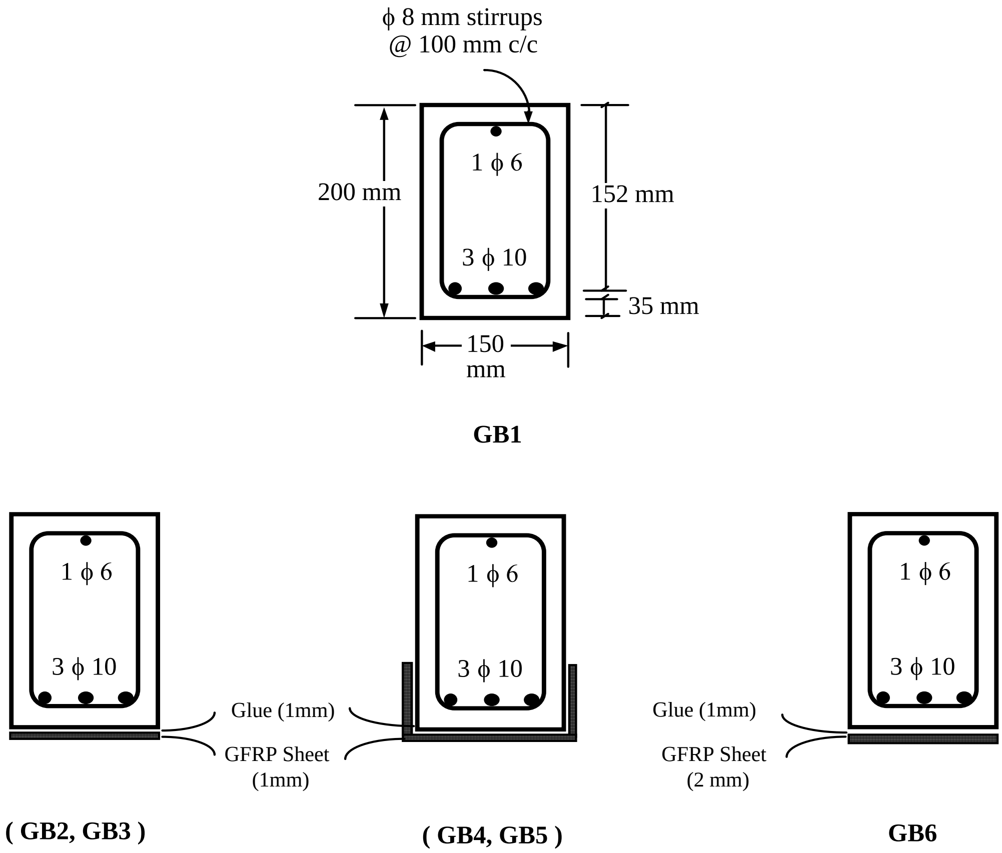

Fig. 2 The cross section details for all groups

Test Procedure

The beams were tested using a load control Amsler testing machine. The central deflections were monitored using a linear variable displacement transducer (LVDT) whereas the strains in the longitudinal bars were recorded using electrical strain gages. The loads were applied continuously and recorded, along with the corresponding LVDT and strain gages readings, using a data acquisition system. Application of the loads and the recording process continued until complete failure of the beam occurred.

TEST RESULTS AND DISCUSSION

The averages for the load-deflection relationships for all groups tested in this series are plotted in Fig. 3. The relationships between the applied loads and the strains in the longitudinal steel bars measured at the center of the beams are shown in Fig. 4. A summary of the test results, which include the values of loads at service, yield and ultimate levels, is given in Table 2. The load deflection behavior, the load-strain behavior, ductilty, and the failure modes of all specimens are discussed in the following sections.

TABLE 2

Test results and mode of failure

Beam ID	Service: Load * (kN)	Service: Increase over unstreng-thened (%)	Yield: Load (kN)	Yield: Increase over unstreng-thened (%)	Ultimate: Load (kN)	Ultimate: Increase over unstreng-thened (%)	Mode of failure
GB1	16.8	0	36.52	0	47.97	0	Ductile flexural failure, steel yield, and concrete crushing.
GB2	19.52	16.2	39.39	7.86	56.22	17.2	Flexural failure after detachment of GFRP sheet.
GB3	16.1	-	38.61	5.73	56.05	16.8	Rupture of GFRP sheet and crushing of concrete.
GB4	19.02	13.2	42.83	17.28	58.67	22.31	Rupture of GFRP sheet and crushing of concrete.
GB5	17.3	3	44.17	20.95	60.11	25.31	Rupture of GFRP sheet and crushing of concrete.
GB6	19.95	19	42.4	16.1	63.46	32.3	Flexural by crushing of concrete.

* *Referenced to the control beam GB1, Δ service = 3.1 mm*

Load-Deflection and Load-strain Relationships

The values of loads at service, yield and ultimate levels are given in Table 2. The failure load for beams GB2 and GB3 were about 56 KN with an increase of 17% over the unstrengthened (unplated) beam. For beams GB4 and GB5 the failure loads were 58.67 kN and 60.11 kN with 22% and 25% increase over that of the unstrengthened beam, respectively. Beams in group GB6 had a failure load of 63.5 kN with an increase of 32% over that of unstrengthened beam. The service load value of the control beam was calculated as 35% of its ultimate load with a corresponding deflection, Δ = 3.1 mm. This value of deflection was used as a reference value to find the service load for all strengthened beams.

Beams GB2, GB4 and GB6 showed an increase of 16%, 13% and 19% at service load level over that of the unstrengthened beam, respectively. However, GB3 and GB5 did not show any increase in the service load. This may be due to the precracking of the beams before strengthening. The increase in the yield load (defined as the load at which the steel reinforcement starts to yield) over that of the unstrengthened beam was higher for beams GB4 and GB5 than that for other strengthened beams (GB2 and GB3). These results clearly show the effectiveness of using U-

shape sheets which provided good anchorage for the strengthening technique. The same increase was recorded for GB6 beams which were strengthened with two layers of GFRP sheet on the tension side only (no U-shape).

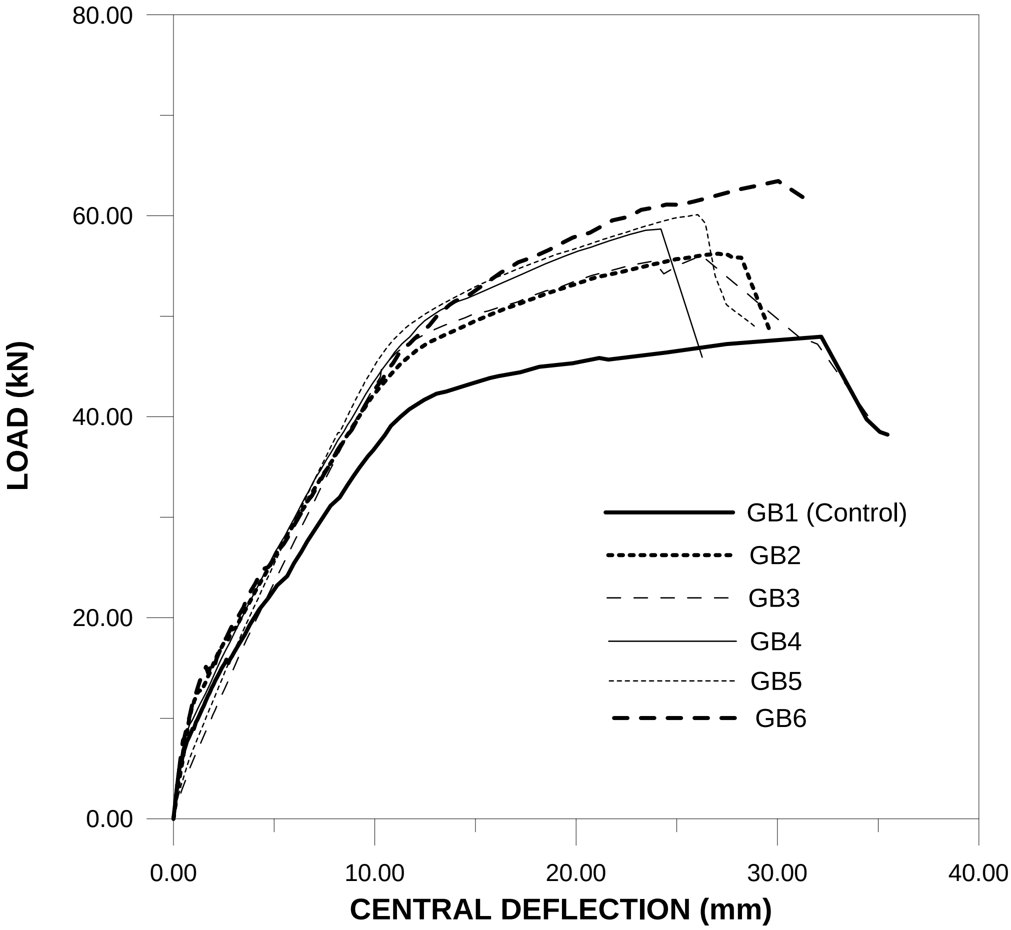

Fig. 3 Measured load versus midspan deflection for all beams

The results in Fig. 3 indicate that the strengthened beams carried more load than the control beam but lost some of their ductility. The measured deflections corresponding to the yield loads are about the same almost for all beams with only marginal variation from the value of deflection of the control beam. However, after yielding, the FRP sheets continued to carry the tensile component in bending and thus the post yield capacity was higher for the strengthened beams.

Also from the strain curves shown in Fig. 4, it can be observed that the yield strains in steel were reached for all strengthened and unstrengthened beams. But, the measured strains in steel for strengthened beams were less than those of the unstrengthened ones under the same load level. This clearly shows the advantage of using FRP sheets in strengthening or upgrading RC beams.

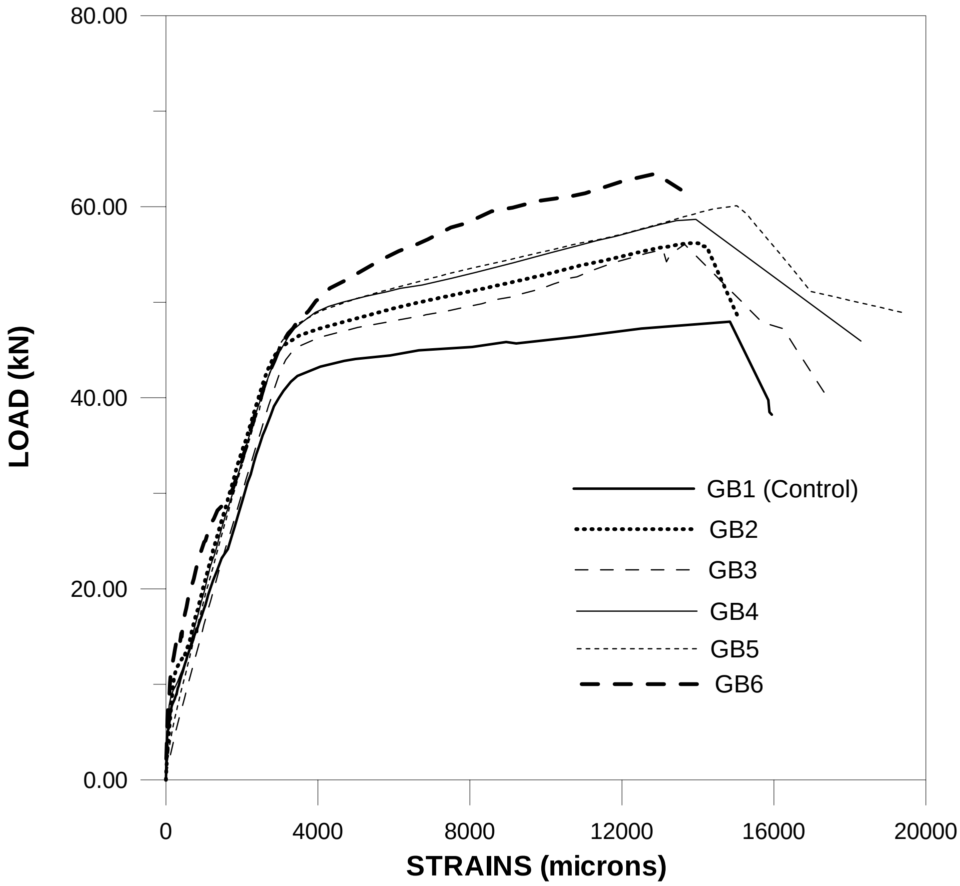

Fig. 4 Measured load versus strains in longitudinal steel for all beams.

Structural Ductility

At the moment, there is no universally accepted definition of structural ductility. In order to develop a rational and meaningful concept of ductility that may be applied to all structural materials, a reference base is required. The yield point of internal steel provides a very objective reference point to define ductility [15]. However, it is the unique yield plateau of the stress-strain curve of the steel at which impart the structural member ability to sustain load while undergoing large deformations. This is not the case when the reinforcing medium is fiber reinforced polymer (FRP), or a mixture of steel and FRP, as in the case of FRP sheet bonded RC beam.

It was shown [15] that deflection and energy based on tension steel yielding can be used as a criterion of ductility to evaluate comparative structural performance of plate/sheet bonded RC beams. The ductility index in this study was obtained based on deflection, μ_D, and energy absorption capability, μ_E, and defined as:

$$\mu_D = \frac{\text{(midspan deflection at peak load)}}{\text{(midspan deflection at tension steel yield)}} \qquad (1)$$

$$\mu_E = \frac{\text{(area under load - deflection curve up to peak load)}}{\text{(area under load - deflection curve up to yield load)}} \qquad (2)$$

The values of ductility indices calculated according to Eqs.(1) and (2) and deflections at yield and ultimate loads are shown in Table 3. The values of μ_D indicated that strengthened beams GB2, GB3 and GB6 had good ductility behavior, especially when compared with unstrengthened beam GB1. On the other hand, GB4 and GB5 beams showed lower ductility than other strengthened beams. However, this reduction in ductility is not significant if compared with that of GB1. This may lead to the conclusion that the strengthened beams had enough ductility that can assure ductile behavior when subjected to flexural stresses.

The same ductility behavior for all beams was obtained based on energy ductility index as computed from Eq (2). Exception of that group GB3 which showed an increase in ductility over the unstrengthened beam GB1 as a result of better anchorage system. Thus, considerable increase in load capacity, due to composite sheeting as in Table 2, was attained without sacrificing the ductility of the beams. Therefore, the results given in Table 3 indicate that since the yield of tension steel is a well-defined reference point in the structural behavior of both unstrengthened and strengthened RC beams, and since deflection and the area under the load-deflection diagram can be precisely defined, measured or calculated, it is suggested that deflection and energy ductility indices are good parameters to evaluate the ductile behavior of strengthened beams.

Failure Patterns of the Specimens

The beams of the first group GB1, which represent the control specimens, failed by crushing of concrete after the occurrence of many flexural cracks. On the other hand the strengthened specimens, group GB2, failed by combined effect of crushing of concrete at the top and delamination of the GFRP sheet. Beams in GB3 failed due to the sheet rupture followed by crushing of concrete at the top fiber. Beams GB4 and GB5, failed in the

same manner as GB3 beams. The last group GB6 showed a perfect bond between GFRP sheets and concrete for the entire range of loading until flexural failure occurred followed by crushing of concrete. The modes of failure for all beams are summarized in Table 2. Photopronts of beams GB3, GB4, and GB6 after failure are shown in Figs. 5, 6 and 7, respectively..

TABLE 3

Deflection and energy ductility indices.

Beam designation	Deflection at yield load (mm)	Deflection at ultimate load (mm)	Deflection ductility index (μ_D)	Energy ductility index (μ_E)
GB1	9.85	32.18	3.27	5.67
GB2	9.03	27.05	3.0	5.27
GB3	8.7	26.21	3.01	6.01
GB4	9.75	24.21	2.48	4.18
GB5	9.9	26.04	2.63	4.6
GB6	9.93	30.04	3.03	5.47

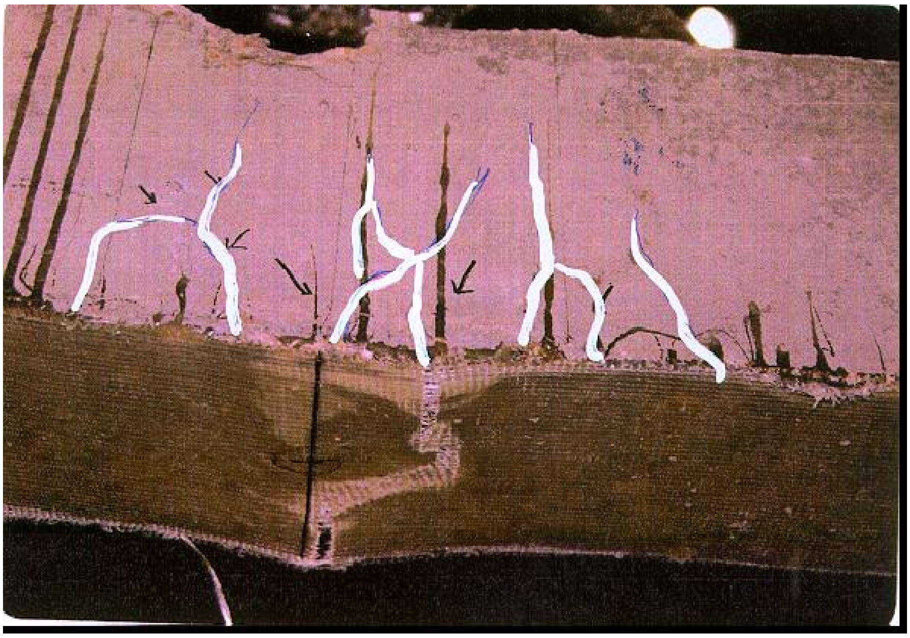

Fig. 5 Failure mode of GB3 beams

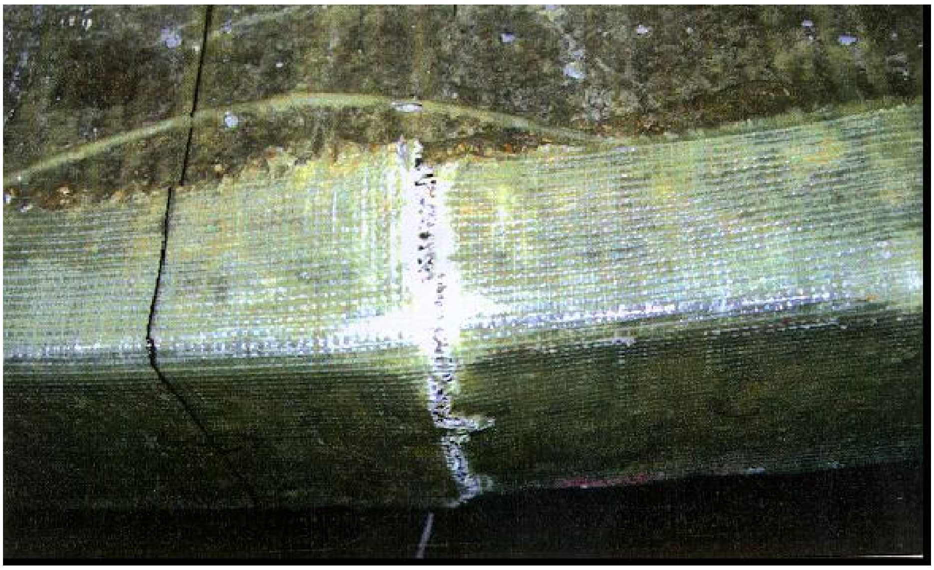

Fig. 6 Failure mode of GB4 beams

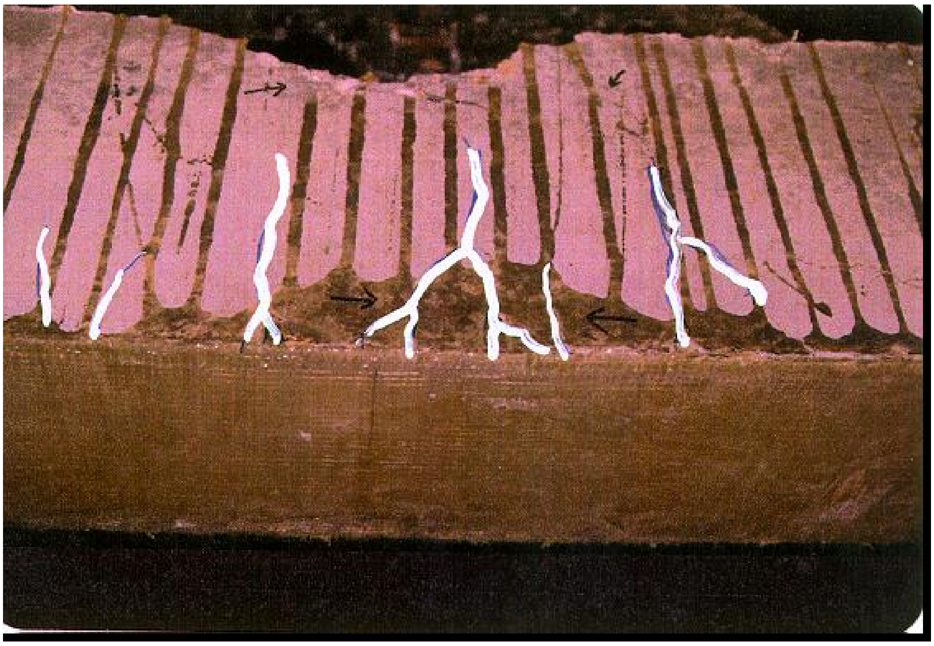

Fig. 7 Failure mode of GB6 beams

ANALYTICAL MODEL

The analytical model used here was developed based on equilibrium of forces and compatibility of the deformations in the adhesive layer [10]. It accounts for influence of variation of stresses, strains and curvatures along the length and across the depth of the beam. The concrete stress-strain

relationship developed by Almusallam and Alsayed [19] was used to represent the concrete behavior. The model assumes linear strain distribution across the depth of the concrete section and accounts for tensile stresses in concrete after cracking. The steel behavior is represented by a bilinear elastic perfectly plastic stress-strain relationship. The GFRP and the epoxy adhesive are represented by a linear line up to the failure point. In order to simplify the analysis, the following assumptions were applied:

(a) Plane sections remain plane (small deformation theory).
(b) Linear strain distribution throughout the full depth of the section.
(c) No slip between the longitudinal reinforcing steel and the surrounding concrete.
(d) No slip between the external plate and the concrete.
(e) No premature plate separation or shear failure; i.e. composite action is maintained up to failure.
(f) Shear deformation is not considered in the analysis.

The required values such as: load, moment, curvature, deflection, and strain for any of the constituents are computed numerically for each increment of loading and printed at the end of the analysis. Thus, the load/tensile strains and load/compressive strain responses, can be obtained at the service, yield and ultimate load levels. Details about the formulation of the analytical model is presented in another study [10].

For simplicity, the results of only two groups of beams were checked against the model prediction. These are of GB2 and GB6 beams, which involve beams strengthened with one and two layers of GFRP sheet. The comparison between the predicted and measured values are presented in Figs. 8 and 9, for beams GB2 and GB6, respectively.

The results show that the ultimate load predictions for beams GB2 and GB6 are in good agreement with the corresponding experimental results. However, the midspan deflections at ultimate were underestimated by about 10% and 22% for beams GB2 and GB6, respectively. These differences may be attributed to the fact that at ultimate load some slippage between the sheet and the beam is expected to occur which was not accounted for in the model. On the other hand, the measured midspan deflections at service load for both beams are in good agreement with the counterpart predicted values.

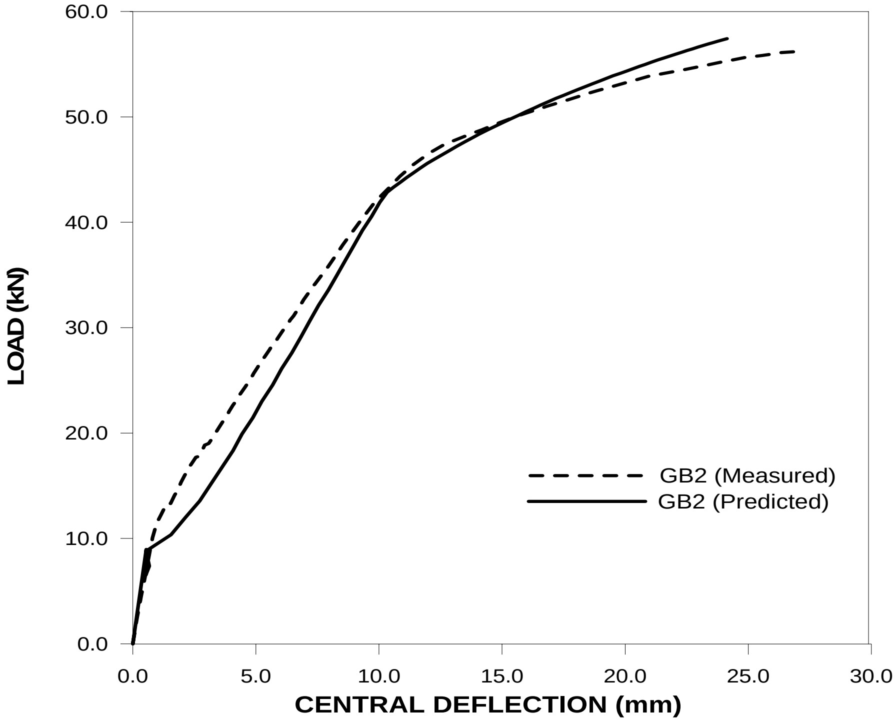

Fig. 8 Calculated and measured load-deflection relationship for GB2 beams

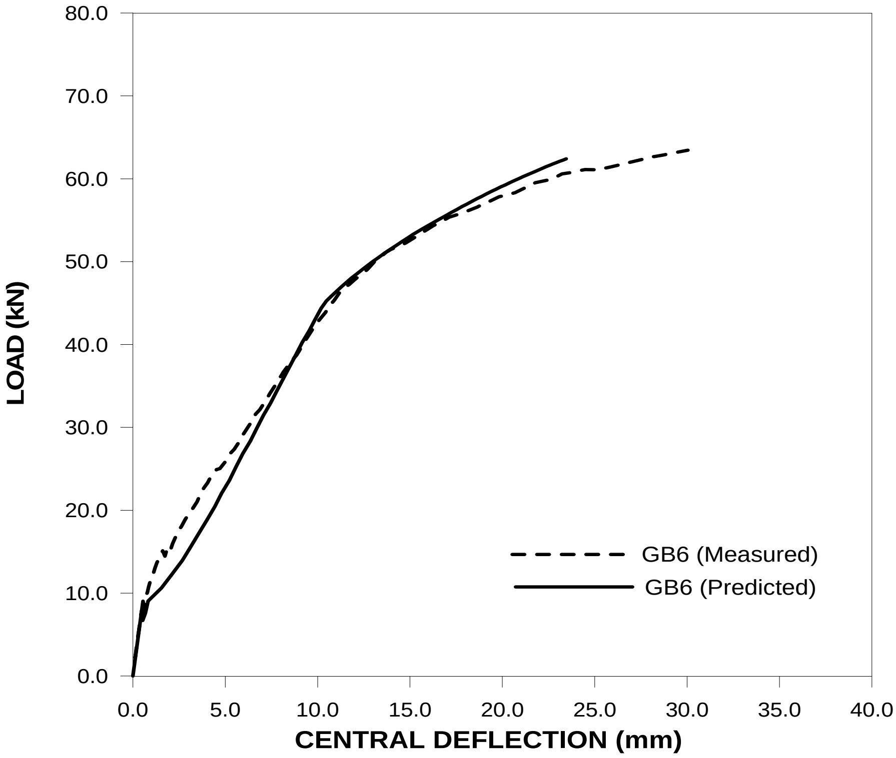

Fig. 9 Calculated and measured load-deflection relationship for GB6 beams

SUMMARY AND CONCLUSIONS

The results of the tests performed in this study indicate that a good increase in the flexural strength can be achieved by bonding GFRP sheets to the tension face of reinforced concrete beams. The proposed anchorage system (U-shape) has a considerable effect on the ultimate strength and the failure mode. It was shown that both deflection and energy based on tension steel yielding can be used as criteria of ductility to evaluate comparative structural performance of plate/sheet bonded RC beams. In this study, the use of GFRP sheets as an external reinforcement to strengthen and upgrade concrete structural members proved to be good and developed enough ductility. The predictions using the suggested analytical model are in good agreement with the experimental results.

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