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BEHAVIOUR OF BEAMS USING GLASS FIBER POLYMER

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I. INTRODUCTION

Concrete is a construction material composed of portland cement and water combined with sand, gravel, crushed stone, or other inert material such as expanded slag or vermiculite. The cement and water form a paste which hardens by chemical reaction into a strong, stone-like mass. The inert materials are called aggregates, and for economy no more cement paste is used than is necessary to coat all the aggregate surfaces and fill all the voids.

1.1 Fiber Reinforced Polymer (FRP)

Continuous fiber-reinforced materials with polymeric matrix (FRP) can be considered as composite, heterogeneous, and anisotropic materials with a prevalent linear elastic behavior up to failure. They are widely used for strengthening of civil structures. There are many advantages of using FRPs: lightweight, good mechanical properties, corrosion-resistant, etc. Composites for structural strengthening are available in several geometries from laminates used for strengthening of members with regular surface to bi- directional fabrics easily adaptable to the shape of the member to be strengthened.

1.2 Fiber

A fiber is a material made into a long filament with a diameter generally in the order of 10 tm. The aspect ratio of length and diameter can be ranging from thousand to infinity in continuous fibers. The main functions of the fibers are to carry the load and provide stiffness, strength, thermal stability, and other structural properties in the FRP.

To perform these desirable functions, the fibers in FRP composite must have:

- High modulus of elasticity for use as reinforcement;
- High ultimate strength;
- Low variation of strength among fibers;
- High stability of their strength during handling; and
- High uniformity of diameter and surface dimension among fibers.

II. LITERATURE SURVEY

Many works have been done to explore the benefits of using waste glass powder in making and enhancing the properties of concrete. History of bonded external reinforcement in the UK goes back to 1975 with the strengthening of the Quinton Bridges on the M5 motorway. This scheme followed a number of years of development work by the Transport and Road Research Laboratory (TRRL), (now TRL), in association with adhesive manufacturers and the Department of Transport. In terms of testing programmes, research and development work continued at the TRRL and at several academic institutions in the UK, most notably at the University of Sheffield. Theoretical investigations and the evaluation of suitable adhesives were allied to the extensive beam testing programmes which were undertaken. Preliminary studies were conducted by:

Irwin (1975). Macdonald (1978) and Macdonald and Calder (1982) reported four point loading tests on steel plated RC beams of length 4900mm. These beams were used to provide data for the proposed strengthening of the Quinton Bridges (Raithby, 1980 and 1982), and incorporated two different epoxy adhesives, two plate thicknesses of 10.0mm and 6.5mm giving width-to-thickness (b/t) ratios of 14 and 22, and a plate lap-joint at its centre. In all cases it was found that failure of the beams occurred at one end by horizontal shear in the concrete adjacent to the steel plate, commencing at the plate end and resulting in sudden separation of the plate with the concrete still attached, up to about mid-span.

Ladner et al., (1990) worked on the use of FRP materials as a replacement for steel in plate bonding applications was pioneered at the EMPA in Switzerland. Four point loading tests were initially performed on RC beams 2000mm (Meier, 1987; Kaiser, 1989) or 7000mm () in length. Strengthening was achieved through the use of pultruded carbon fiber/epoxy laminates up to 1.0mm thick bonded with the same epoxy adhesives used in earlier steel plating work (Ladner and Weder, 1981).

Saadatmanesh and Ehsani (1991) conducted an experimental study of the strengthening of reinforced concrete beams using non-prestressed and prestressed GFRP plates. One of the two prestressed beams contained a relatively small amount of internal tensile steel reinforcement, while the other contained larger bars and was precracked prior to bonding of the plate. The plate prestress in the precracked case closed some of the cracks, indicating the benefit of prestressing from a serviceability point of view. The beam with little original reinforcement before plating experienced a large improvement in ultimate capacity due to the additional moment couple provided by the plate prestress.

Deblois et al. (1992) investigated the application of unidirectional and bidirectional glass fiber reinforced polymer (GFRP) sheets for flexural strengthening. A series of RC beams 1000mm long were tested after strengthening. The use of bidirectional sheets increased the ultimate load by up to 34%, whereas unidirectional GFRP resulted in an increase of only 18%. The authors of this current chapter feel that this is an unexpected conclusion and emphasise that the FRP material used was GFRP.\

Triantafillou et al. (1992) tested reinforced concrete beams in three point bending with various quantities of internal reinforcement and magnitudes of CFRP plate prestress. Improved control of concrete cracking was brought about not only by a greater internal reinforcement provision, but also by higher plate prestress, indicating the serviceability advantage gained by prestressing the composite.

Char et al. (1994) conducted an analytical parameter study to determine the effects of varying the cross-sectional area and material type of the composite plate and the prestress in the plate. The parameter study revealed that pre stressing a GFRP plate would not necessarily increase the ultimate moment capacity over that of a beam with a non-prestressed plate, for the particular beam configuration and prestress level considered.

Hussain et al. (1995) investigated the use of anchor bolts at the ends of steel plated beams, in an attempt to prevent brittle separation of the plate. In agreement with Jones et al. (1988) the bolts, which were 15mm in diameter and penetrated to half the depth of the beam, were found to improve the ductility of the plated beams considerably, but to have only a marginal effect on the ultimate load.

Jones and Swamy (1995) presented a brief summary of some of the research work carried out at the University of Sheffield since the late 1970s has highlighted a number of effects of external, epoxy-bonded steel plates on the serviceability and ultimate load behaviour of RC beams.

Garden and Hollaway (2001) tested 1.0m and 4.5m lengths of reinforced concrete beams in four point bending after strengthening them with externally bonded prestressed CFRP plates. The plates were bonded without prestress and with prestress levels ranging from 25–50% of the plate strength.

He et al. (2004), at the University of Sheffield, used steel and CFRP plates with the same axial stiffness-to-strength precracked reinforced concrete beams in which a new, but unspecified, plate anchorage system was adopted.

Bencardino et al. (2008) tested CFRP plated beams at the University of Calabria, Italy, recording reductions in member ductility due to plating without end anchorage; the ductility was restored when anchorage was fitted in the form of externally bonded U-shaped steel stirrups. The method of CFRP plating was used successfully to strengthen an experimental portal structure.

Wight et al. (2013) reported data on the strengthening and stiffening achieved with prestressed CFRP plates. The control of concrete crack widths and numbers of cracks was improved by prestressing the plates.

2.2 Research Gap

Investigations were carried out on the Reinforced Concrete Beams, whose strength is increased by sheets of glass fiber polymer that are used as a reinforcing material. Thereafter their shear and flexural behavior was tested. Two point loading system is used for failure testing of externally reinforced concrete beams with epoxy-bonded glass fiber reinforced polymer sheets. In which two sets of beams were casted for experiment. In first sets three beams were casted, which were weak in shear.

III. FAILURE MODES

The flexural and shear strength of a section depends on the controlling failure mode. The following flexural and shear failure modes should be investigated for an FRP-strengthened section:

• Crushing of the concrete in compression before yielding of the reinforcing steel;
• Yielding of the steel in tension followed by rupture of the FRP laminate;
• Yielding of the steel in tension followed by concrete crushing;
• Shear/tension delamination of the concrete cover (cover delamination); and
• Debonding of the FRP from the concrete substrate (FRP debonding).

IV. LOAD DEFLECTION HISTORY

The load deflection history of all the beams was recorded. The mid-span deflection of each beam was compared with that of their respective control beams. Also the load deflection behaviour was compared between two wrapping schemes having the same reinforcement. It was noted that the behaviour of the flexure and shear deficient beams when bonded with GFRP sheets were better than their corresponding control beams. The mid-span deflections were much lower when bonded externally with GFRP sheets. The graphs comparing the mid-span deflection of flexure and shear deficient beams and their corresponding control beams are shown in Figs 4.5 and 4.9.

The use of GFRP sheet had effect in delaying the growth of crack formation. In SET I when both the wrapping schemes were considered it was found that the beam F3 with GFRP sheet up to the neutral axis along with the soffit had a better load deflection behaviour when compared to the beam F2 with GFRP sheet only at the soffit of the beam. In SET II when both the wrapping schemes were considered it was found that the beam S3 with U wrapping of GFRP sheet had a better load deflection behavior when compared to the beam S2 with GFRP sheet only at the sides of the beam.

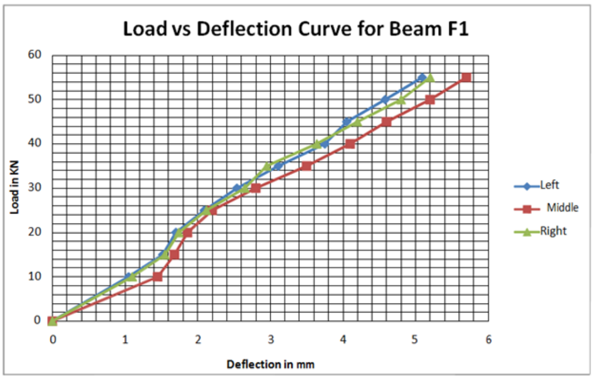

Fig. 4.1 Load vs. Deflection Curve for Beam F1

Beam F1 was the control beam of SET I beams which were weak in flexure but strong in shear. In beam F1 strengthening was not done. Two point static loading was done on the beam and at the each increment of the load, deflection at the left, right and middle dial gauges were taken. Using this load and deflection of data, load vs deflection curve is ploted. At the load of 30 KN initial cracks started coming on the beams. Further with increase in loading propagation of the cracks took place. The beam F1 failed completely in flexure.Beam F2 of SET I beams which were weak in flexure but strong in shear. In beam F2 strengthening is done by application of GFRP sheet only at the soffit of the beam. Two point static loading was done on the beam and at the each increment of the load, deflection at the left, right and middle dial gauges were taken. Using this load and deflection of data, load vs deflection curve is ploted. At the load of 34 KN initial cracks started coming on the beams. Initial cracks started at a higher load in beam F2 compared to beam F1.

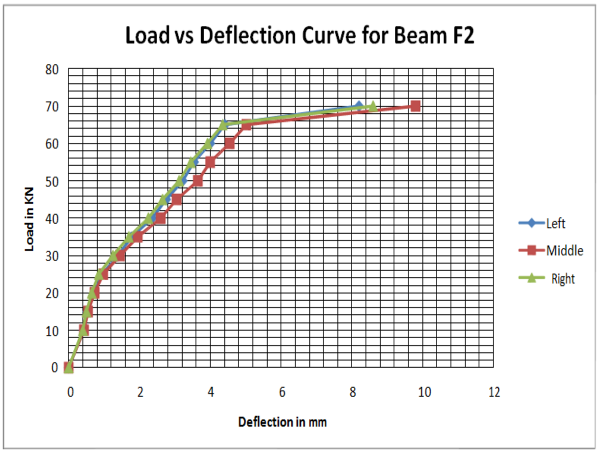

Fig. 4.2 Load vs. Deflection Curve for Beam F2

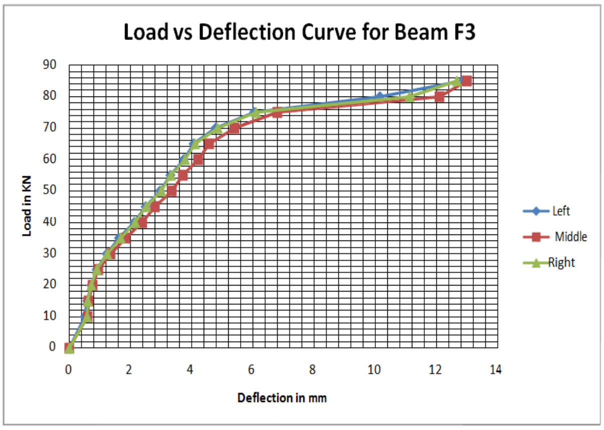

Fig. 4.3 Load vs. Deflection Curve for Beam F3

Further with increase in loading propagation of the cracks took place. The beam F2 failed in flexural shear. Beam F2 carried a higher ultimate load compared to beam F1.Beam F3 of SET I beams which were weak in flexure but strong in shear. In beam F3 strengthening is done by application of GFRP sheet up to the neutral axis along with the soffit of the beam. Two point static loading was done on the beam and at the each increment of the load, deflection at the left, right and middle dial gauges were taken. Using this load and deflection of data, load vs. deflection curve is ploted. Initial cracks are not visible on the beams. Further with increase in loading propagation of the cracks took place but it had poor visibility of cracks due to the covering of the GFRP sheet. The beam F3 also failed in flexural shear like beam F2 but beam F3 carried a higher ultimate load compared to both beam F1 and F2.

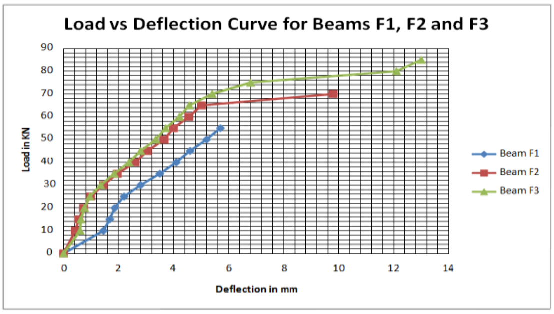

Fig. 4.4 Load vs. Deflection Curves for Beams F1, F2 and F3.

From the load and deflection of data of SET I beams F1, F2 and F3, load vs. deflection curve is plotted for all the three beams. From this load vs deflection curve, it is clear that beam F1 has lower ultimate load carrying capacity compared to beams F2 and F3. Beam F1 had also undergone higher deflection compared to beams F2 and F3 at the same load. Beam F2 had higher ultimate load carrying capacity compared to the controlled beam F1 but lower than beam F3. Beam F3 had higher ultimate load carrying capacity compared to the beams F1 and F2. Both the beams F2 and F3 had undergone almost same deflection up to 65 KN load. After 65 KN load beam F3 had undergone same deflection as beam F2 but at a higher load compared to beam F2. The deflection undergone by beam F3 is highest. Beam F2 had undergone higher deflection than beam F1.

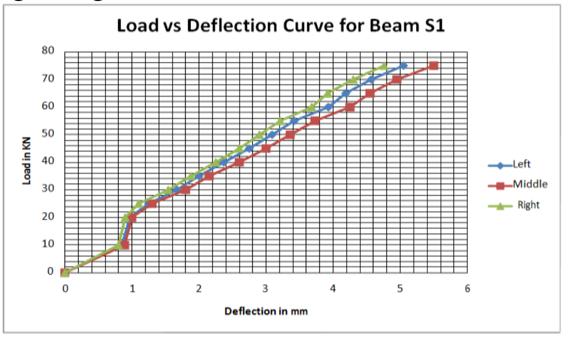

Fig. 4.5 Load vs. Deflection Curve for Beam S1

Beam S1 was the control beam of SET II beams which were weak in shear but strong in flexure. In beam S1 strengthening was not done. Two point static loading was done on the beam and at the each increment of the load, deflection at the left, right and middle dial gauges were taken. Using this load and deflection of data, load vs deflection curve is plotted. At the load of 35 KN initial cracks started coming on the beams. Further with increase in loading propagation of the cracks took place. At first in beam S1 only flexural cracks were developed but ultimately the beam failed in shear.

Beam S2 of SET II beams which were weak in shear but strong in flexure. In beam S2 strengthening is done by application of GFRP sheet only on the two sides of the beam. Two point static loading was done on the beam and at the each increment of the load, deflection at the left, right and middle dial gauges were taken.

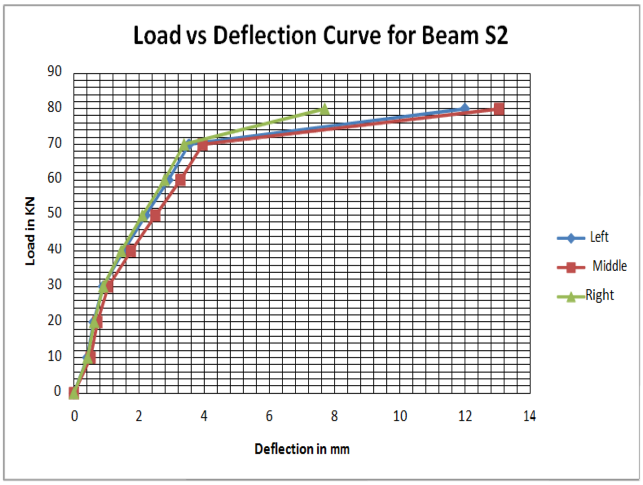

Fig. 4.6 Load vs. Deflection Curve for Beam S2

Using this load and deflection.of data, load vs deflection curve is ploted.At the load of 39 KN initial cracks started coming on the beams. Initial cracks started at a higher load in beam S2 compared to beam S1. Further with increase in loading propagation of the cracks took place. In beam S2 only flexural cracks were developed and finally the beam failed by flexural failure and crushing of concrete. Beam S2 carried a ultimate load higher than beam S1 but lower than beam S3.

Beam S3 of SET II beams which were weak in shear but strong in flexure. In beam S3 strengthening is done by application of GFRP sheet as U-wrap on the beam. Two point static loading was done on the beam and at the each increment of the load, deflection at the left, right and middle dial gauges were taken. Using this load and deflection of data, load vs deflection curve is ploted. At the load of 39 KN initial cracks started coming on the beams. Initial cracks started at a higher load in beam S3 compared to beams S1 and S2. Further with increase in loading propagation of the cracks took place. In beam S3 similar to beam S2 only flexural cracks were developed and finally the beam failed by flexural failure and crushing of concrete, but beam S3 carried a higher ultimate load compared to both beam S1 and S2 .

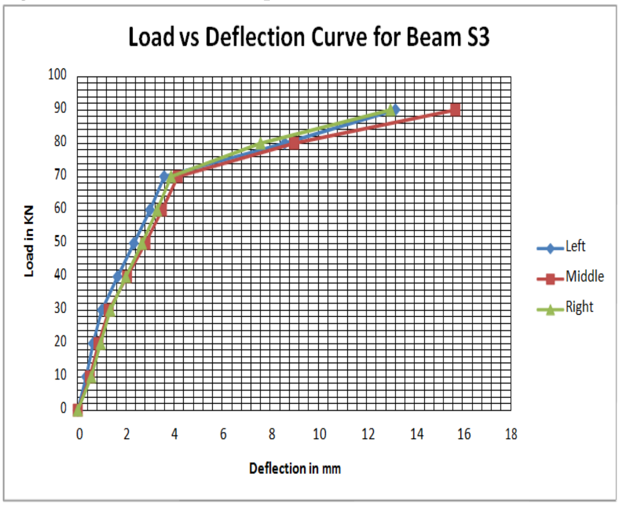

Fig. 4.7 Load vs Deflection Curve for Beam

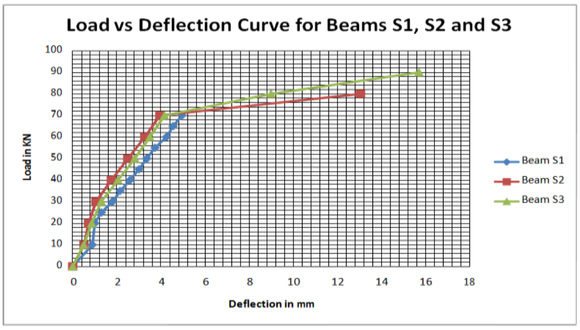

Fig. 4.8 Load vs Deflection Curves for Beams S1, S2 and S3.

From the load and deflection of data of SET II beams S1, S2 and S3, load vs deflection curve is ploted for all the three beams. From this load vs deflection curve, it is clear that beam S1 has lower ultimate load carrying capacity compared to beams S2 and S3. Beam S1 had also undergone higher deflection compared to beams S2 and S3 at the same load. Beam S2 had higher ultimate load carrying capacity compared to the controlled beam S1 but lower than beam S3. Beam S3 had higher ultimate load carrying capacity compared to the beams S1 and S2. Both the beams S2 and S3 had undergone almost same deflection upto 70 KN load. After 70 KN load beam S3 had undergone same deflection as beam S2 but at a higher load compared to beam S2. The deflection undergone by beam S3 is highest. Beam S2 had undergone higher deflection than beam S1.

4. 1 Crack Pattern

The crack patterns at collapse for the tested beams are shown in Figure 4.9 and 4.10. In SET I the controlled beam F1 exhibited widely spaced and lesser number of cracks compared to strengthened beams F2 and F3. The strengthened beams F2 and F3 have also shown cracks at relatively close spacing. This shows the enhanced concrete confinement due to the GFRP strengthening. This composite action has resulted in shifting of failure mode from flexural failure (steel yielding) in case of controlled beam F2 to peeling of GFRP sheet in case of strengthened beams F2 and F3. The debonding of GFRP sheet has taken place due to flexural-shear cracks by giving cracking sound. A crack normally initiates in the vertical direction and as the load increases it moves in inclined direction due to the combined effect of shear and flexure. If the load is increased further, cracks propagate to top and the beam splits. This type of failure is called flexure-shear failure.

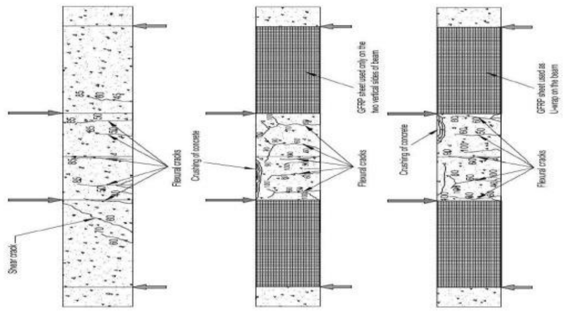

Figure 4.9 Crack patterns at collapse for the tested beams

In SET II beam S1 the shear cracks started at the centre of short shear span. As the load increased, the crack started to widen and propagated towards the location of loading. The

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cracking patterns show that the angle of critical inclined crack with the horizontal axis is about 45°. For strengthened reinforced concrete beams S2 and S3, the numbers of vertical

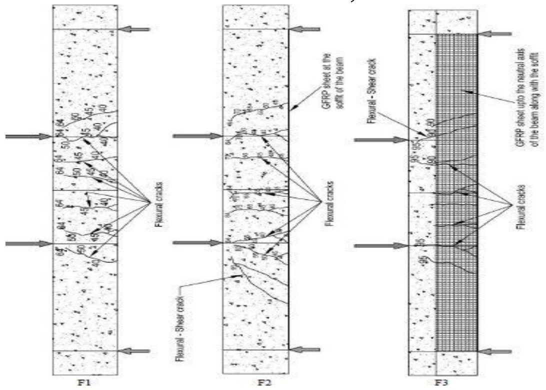

Figure 4.10 Crack patterns at collapse for the tested beams

4.2 Comparison of Results: The results of the two set of beams tested are shown in Table 4.1. The failure mode, load at initial crack and ultimate load of the control beams without strengthening and the beams strengthen with two layers GFRP sheet are presented. The difficulties inherent to the understanding of strengthen structural member behavior subjected to flexure and shear have not allowed to develop a rigorous theoretical design approach. The complexity of the problem has then made necessary an extensive experimental research. Moment of resistance of the SET I beams was calculated analytically and was compared with the obtained

Table: 4.1 Experimental results

SET I Beams	Mu from analytical study	Mu from experimental study
F1	19.06 KN-m	26.00 KN-m
F2	28.07 KN-m	34.68 KN-m

V. CONCLUSION

In this experimental investigation the flexural and shear behaviour of reinforced concrete beams strengthened by GFRP sheets are studied. Two sets of reinforced concrete (RC) beams, in SET I three beams weak in flexure and in SET II three beams weak in shear were casted and tested. From the test results and calculated strength values, the following conclusions are drawn:

A) SET I Beams (F1, F2 and F3)

1. Initial flexural cracks appear at a higher load by strengthening the beam at soffit. The ultimate load carrying capacity of the strengthen beam F2 is 33 % more than the controlled beam F1.
2. Load at initial cracks is further increased by strengthening the beam at the soffit as well as on the two sides of the beam up to the neutral axis from the soffit. The ultimate load carrying capacity of the strengthen beam F3 is 43 % more than the controlled beam F1 and 7 % more than the strengthen beam F2.
3. Analytical analysis is also carried out to find the ultimate moment carrying capacity and compared with the experimental results. It was found that analytical analysis predicts lower value than the experimental findings.
4. When the beam is not strengthen, it failed in flexure but after strengthening the beam in flexure, then flexure-shear failure of the beam takes place which is more dangerous than the flexural failure of the beam as it does not give much warning before failure. Therefore it is recommended to check the shear strength of the beam and carry out shear strengthening along with flexural strengthening if required.
5. Flexural strengthening up to the neutral axis of the beam increases the ultimate load carrying capacity, but the cracks developed were not visible up to a higher load. Due to invisibility of the initial cracks, it gives less warning compared to the beams strengthen only at the soffit of the beam.
6. By strengthening up to the neutral axis of the beam, increase in the ultimate load carrying capacity of the beam is not significant and cost involvement is almost three times compared to the beam strengthen by GFRP sheet at the soffit only.

B) SET II Beams (S1, S2 and S3)

1. The control beam S1 failed in shear as it was made intentionally weak in shear.
2. The initial cracks in the strengthen beams S2 and S3 appears at higher load compared to the un-strengthen beam S1.
3. After strengthening the shear zone of the beam the initial cracks appears at the flexural zone of the beam and the crack widens and propagates towards the neutral axis with increase of the load. The final failure is flexural failure which indicates that the GFRP sheets increase the shear strength of the beam. The ultimate load carrying capacity of the strengthen beam S2 is 31 % more than the controlled beam S1.
4. When the beam is strengthen by U-wrapping in the shear zone, the ultimate load carrying capacity is increased by 48 % compared to the control beam S1 and by 13% compared the beam S2 strengthen by bonding the GFRP sheets on the vertical sides alone in the shear zone of the beam.
5. When the beam is strengthen in shear, then only flexural failure takes place which gives sufficient warning compared to the brittle shear failure which is catastrophic failure of beams.
6. The bonding between GFRP sheet and the concrete is intact up to the failure of the beam which clearly indicates the composite action due to GFRP sheet.
7. Restoring or upgrading the shear strength of beams using GFRP sheet can result in increased shear strength and stiffness with no visible shear cracks. Restoring the shear strength of beams using GFRP is a highly effective technique.

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