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AN EXPERIMENTAL STUDY ON SHELLACKING OF CORROSION PROBLEM IN CONCRETE STRUCTURE (DEEP BEAM) BY SUBSTITUTING GFRP AS A STEEL REINFORCEMENT

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Abstract: Corrosion of steel in reinforced concrete structures is one of the biggest challenges that is faced by the civil construction industry to-day. Corrosion of steel reinforcement in Reinforced Concrete (RC) structures considerably reduces the durability and life span of these structures. The problem of corrosion is also a matter of concern especially when the RC structures are exposed to severe adverse environmental conditions such as in an urban or in a coastal area. In cold region countries, corrosion is a menace at places where de-icing salts are used over RC structures in cold region countries. To overcome this corrosion problem, many new techniques have been tried and tested and these tests were found to be either expensive or ineffective. The use of protective coating by epoxy polymers, the use of stainless steel in the place of conventional steel, controlling corrosion by cathodic protection were some of the methods tried earlier to overcome corrosion. But none of them were fully effective from the functionality or economical point of view. Fiber Reinforced Polymer (FRP) materials which are anticorrosive, were found to be a prospective substitute to conventional steel reinforcement used in RC structures.

Keywords: Corrosion, epoxy polymers, Reinforced Concrete (RC) structures

1.0 Introduction

Corrosion of steel in reinforced concrete structures is one of the biggest challenges that is faced by the civil construction industry to-day. Corrosion of steel reinforcement in Reinforced Concrete (RC) structures considerably reduces the durability and life span of these structures. The problem of corrosion is also a matter of concern especially when the RC structures are exposed to severe adverse environmental conditions such as in an urban or in a coastal area. In cold region countries, corrosion is a threat at places where de-icing salts are used over RC structures in cold region countries. To overcome this corrosion problem, many new techniques have been tried and tested and these tests were found to be either expensive or ineffective. The use of protective coating by epoxy polymers, the use of stainless steel in the place of conventional steel, controlling corrosion by catholic protection were some of the methods tried earlier to overcome corrosion. But none of them were fully effective from the functionality or economical point of view.

Thus, Fiber Reinforced Polymer (FRP) materials which are anticorrosive were found to be a prospective substitute to conventional steel reinforcement used in RC structures. During the last two decades, FRP materials in the form of solid bars have been successfully tried as a substitute for steel reinforcement in concrete structures. The corrosion resistance of FRP (Fiber-Reinforced Polymers) bars added to their high strength advantage makes it a promising alternative reinforcement material in RC structures, which are prone to corrosion. FRP materials are anti-corrosive and have a low weight to strength ratio. All types of FRP materials are widely used for various modern engineering applications. Due to relatively low elastic modulus of GFRP reinforcement and its brittleness, the crack width at serviceability is often made as the controlling factor in designing FRP reinforced concrete structures. Factors such as high cost of FRP materials and lack of proper data about its durable usage have been a detriment to its growth in the past. At present there are many organizations which are working hard to develop and promote FRP materials for its use in the construction industry. It is hoped that over a period of time, the cost will reduce due to increase in the number of users and manufacturers. Many research works have been carried out on FRP reinforced concrete structures. Many standards and codes for designing FRP reinforced concrete structures were developed in many countries where it was predominantly used. Although much research work has been carried out in FRP reinforced concrete beams, there is no data on record to confirm any work related to FRP reinforced concrete deep beams tested with in a 'shear span to effective depth ratio' less than 1.0. The earlier studies on FRP reinforced deep beams were confined only to beams reinforced without web reinforcement. This experimental study can be considered as the foremost work done on deep beams with GFRP web reinforcement.

1. 1 Objectives of the Present Work

The main objective of this study is to investigate the structural performance of deep beams reinforced with Glass Fiber Reinforced (GFRP) web reinforcements. The other objectives include:

1. To evaluate the ultimate load carrying capacity of deep beams reinforced with GFRP web reinforcement.
2. To evaluate the individual performance of the GFRP vertical and horizontal web reinforcements which were used as web reinforcement in concrete deep beams to resist shear.
3. To observe and study the mode of failure and crack patterns in GFRP reinforced concrete deep beams reinforced with GFRP subjected to shear failure.
4. To study the effectiveness of GFRP web reinforcement in resisting shear failure.

2.0 Shear Behaviour Of Steel Reinforced Concrete Beams- Background Review

A. 2.2.1 Shear and Its Importance

The study of shear behaviour in concrete structures has been going on since a century. It was until the year 1955, when the shear failure of beams that took place in the warehouse at Wilkins Air Force Depot in Shelby, Ohio, researchers were of the view that shear was simple problem to deal with. Then they realized that shear in concrete beams cannot be designed as traditionally as it was done earlier. There has been a feeling among researchers to go back and rethink about the fundamentals of shear design. Going back, the work done by Talbot (1909) during the year 1909 was considered to give a clear and significant way to analyze the shear for designing concrete structures. been developed to explain the shear behaviour in beams and also to estimate its shear capacity.

B. 2.2.2 Classification of RC Beams

The shear behaviour and capacity of concrete beams depends on various factors and among them the length /span of the beam plays a crucial role. The behaviour of beams varies depending on the span to over all depth ratio. The beams are broadly classified as deep, short and slender depending upon their behaviour and failure mode. 2.2.3 Factors Influencing the Shear Behaviour and Capacity of RC Beams

1) 2.2.3.1 Factors influencing the shear behaviour

Shear failure in concrete beams are brittle in nature and are catastrophic, which is quite contradictory to flexural failure where ductility is dominant. In the concrete beams since the shear failure precedes the flexural failure, the shear strength is designed to be greater than the flexural strength at al1 points along the beam.

2.2.3.2 Factors influencing the shear capacity

In case of concrete beams without shear reinforcement, the load at cracking decides the capacity of the member. However, in the case of beams with shear reinforcement, even after cracking there seems to be some resistance to shear due to the presence of tensile stresses in concrete. This fact about the increased capacity was found by Collins et al (1996). The design capacity of these beams depends on the load at cracking. Concrete beams without stirrups, having a longitudinal reinforcement ratio between 0.75 to 2.5 percentage, fail only due to shear. In this range, beams with Lower reinforcement ratios tend to fail at lower shear stresses.

C. 2.2.4 Shear Resisting Mechanism in RC Beams

Model studies in concrete beams with and without shear reinforcement has been going on since 1973. Most of the model studies related to the shear mechanism of concrete beams was first reviewed by ASCE-ACI Committee in the year 1998, which was published in the ASCEACI Committee report 445. An extensive review of most of the important shear models of RC beams, evolved between 1973 and 1998, was consolidated in this report.

Nilson et al (2004) demonstrated the effect of dowel action which occurs as a result of the vertical forces acting across the longitudinal steel reinforcement. MacGregor and Wight (2005) developed a simple truss model through which they illustrated that a beam which has inclined cracks, formed due to the applied load, develops compressive and tensile forces in the top and bottom flanges together with vertical forces in the stirrups and compressive forces in the diagonals. For members which have very small amount of shear reinforcement, the resistance offered to shear by this model predicts more conservative results.
Later Nilson et al (2004) modified this original truss model into the "variable angle truss model" in which it was assumed that the concrete strut angle was not always inclined at 45°. Instead they vary within the range of 25° to 65°. This new proposal modelled by Nilson and named as "variable angle truss model". The variable angle model comprises of compression fans and compression fields. The compression fans which takes place near the supports or under the direct loads has numerous diagonal struts spread out from this region. It is assumed that the total vertical load is fully resisted by these radiating struts. The compression field consists of diagonal compression struts that are formed parallel between the compression fans. All the stirrups are assumed to have yielded at this point as assumed in the original truss model.
Shioya (1989) studied the influence of beams depth and aggregate size on the shear strength of concrete beams by conducting experiments on beams of varying depth from 100 mm to 3000 mm. The investigation revealed that as the beams depth increased the shear stress decreased. This decrease in shear stress may be attributed to the reason that due to larger area of frictional resistance across the failure crack due to greater depth, the shearing force could have distributed to a relatively larger area. The study also revealed that the size of the aggregate was inversely proportional to the shear stress at failure.

D. 2.2.5 Effect of Shear Reinforcements

The flexural failure in RC beams does not take place suddenly but instead it shows some warning of distress going to take place in the near future. Contradictory to this, the shear failure is sudden, catastrophic and devastating. To avoid any such sudden shear failure, shear reinforcements are provided. Also, the shear reinforcement has a control over the shear strength of the beam. The shear stirrups and are used to increase the shear strength of concrete beams, to avoid the shear failure and to cause a flexural failure. Shear reinforcements, which are normally provided as vertical stirrups, are placed at varying intervals depending upon the shear conditions acting on a beam. Different configurations of stirrups are being used, such as an open or closed stirrup, or stirrups with multiple legs which depends upon the amount of applied shear. Shear reinforcement are also provided as inclined bars in some cases. Shear reinforcement comes into effectiveness only after the formation of diagonal cracks either crossing them or in its vicinity.

3.0 Materials And Experimental Procedure

A. 3.2.1 Concrete

The concrete used for casting was prepared in the testing laboratory using a portable concrete mixture machine. All the specimens which were tested were cast by using cement concrete and the cement used was confirming to the specification of IS 8112 (1989) code. The concrete was designed to achieve the 28 day compressive strength of 40 N/mm2 (M40 Grade). The concrete mix proportion adopted was 1: 1.02: 1.93 with water/cement ratio of 0.38. The material proportions per cubic meter of concrete:
1) 1059 kgs of coarse aggregate (maximum size 20mm)
2) 560 kgs of natural river sand (sp.gr =2.53)
3) 548.5 kgs of ordinary port land cement(43 grade)
4) 208 litres of water
The coarse aggregate used was crushed granite stones of 20 mm and less in size with specific gravity of 2.58 and fineness modulus of 6.9. The locally available river sand which was sieved by passing through 2.36 mm sieve and retained on 75µm sieve and which had a fineness modulus of 3.1 was used as the fine aggregate. The cement used for preparing the concrete was tested in the laboratory conforming to IS 8112 (1989) standards with respect to initial and final setting time, specific gravity, fineness and compressive strength. Along with each batch of the concrete prepared for casting the deep beam specimens, concrete cubes were cast to determine the material properties and to confirm its strength.
Testing of these cubes was done after 7 days and then 28 days after casting. The cubes were de-moulded the next day of casting and it was cured in water before it is tested. The details of cube tests results are shown in Table 3.1.

Table 3.1 Concrete cube test results

BEAMS	Compressive Strength N/mm^2 (7Days)	Compressive Strength N/mm^2 (28Days)
GFRDB-1	28.7	41.9
GFRDB-2	30.9	43.3
GFRDB-3	28.8	40.8
GFRDB-4	34.9	40.1
GFRDB-5	33.4	41.3
GFRDB-6	28.5	42.9
GFRDB-7	29.8	42.5
GFRDB-8	29.5	40.7
GFRDB-9	30.5	41.9

A. 3.2.2 About GFRP Constituent Materials

1) 3.2.2.1 Glass Fiber

The glass fiber used for fabricating the GFRP reinforcements is of E- Glass fiber type of Saint-Gobain Vetrotex with identification number as RO 99 2400 P 566. The fiber which was available in roving of 2400 normal liner density (tex) was specifically selected for the purpose of manufacturing by 'Manual Fiber-Trusion'. The proprietary sizing system of the selected P566 type fiber was designed to give a high level of performance with greater compatibility with Polyester, epoxy, vinyl and phenolic resins.
The test specifications supplied by the fiber manufacturer are as shown in Table 3.2.

Table 3.2 Specification of E-Glass Fiber

PROPERTY	VALUE	TEST METHOD
Glass content (%)	60-65	BS 3691
Tensile Strength (Mpa)	1700-1800	BS 3691
Tensile Modulus (Gpa)	65-75	BS 3691

Some of the other advantages of using this type of fiber include fast and complete wet-out of fibers with most resins, good resin wettability by fibers, excellent spreading on the mandrel which gives a smooth and regular surface, high mechanical properties, good composite translucency. These were found to be needed for this type of manufacturing process to be efficient and hence this type of fiber was adopted.

2) 3.2.2.2 Epoxy resin

Epoxy resin is the extensively used matrix material compared to other category of resins particularly for reinforcement bars due to their high mechanical property to bond with fibers. Epoxy is a thermosetting polymer formed from after mixing a peroxide resin with a hardener in a suitable proportion. Epoxy resins have high strength, good corrosion resistant and can very well bond with fibers, high glass transition temperature, superior electrical properties, high dimensional stability, and low shrinkage. Epoxy resins which have a wide range of mechanical properties can be used in all FRP manufacturing processes. The slow curing epoxy resins used in this work takes a long gel time to become solid, which is considered to be more advantageous for the 'Manual Fiber-Trusion' process as it requires more time for moulding the uncured impregnated fibers pulled from the Fiber-Trusion die. Epoxies used for this work was found to have an average tensile strength of 95 MPa. Their density varied between 1.2 -1.4 Kg/m3. The

II. PERCENTAGE OF CURE SHRINKING WAS OBSERVED TO BE 3.1%.

III. MANUAL FIBER-TRUSION

A. 3.3.1 'Manual Fiber-Trusion'- a new method of Manufacturing GFRP Reinforcement

FRP Composites are being manufactured by various techniques and with emerging new technologies, production is easy and fast. Among the various methods of manufacturing FRP composites, pultrusion by machine is one way of manufacturing. The Pultrusion method employs machines to manufacture FRP composite which make the process expensive. Moreover they require high initial investment and the process of manufacturing is slow. In addition to all this, there exist some shortcomings with respect to the strength criteria of FRP bars manufactured through mechanized pultrusion. This is due to the use of low strength filler material during the manufacture of pultruded FRP composites. These A schematic diagram showing the various stages of 'Manual Fiber-Trusion' process is shown in Figure 3.1.

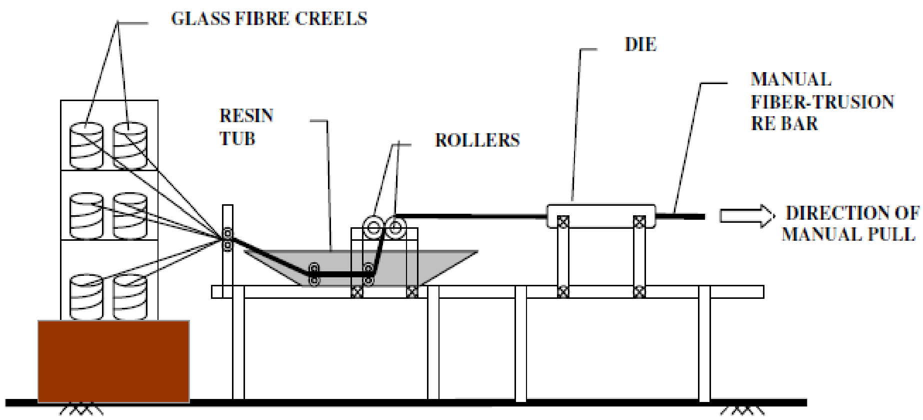

Figure 3.1 The 'Manual Fiber-Trusion' process

Although there are many FRP composite manufacturing companies in India, most of them mainly manufacture GFRP bars for electrical applications only. In such GFRP bars, the insulation property is considered primarily important than the strength of the bar. When tested the strength of these bars were found to be less as their fiber content was less than 65% and a substantial amount of filler materials was present in it. Such a combination of low fiber and high matrix materials used for manufacturing these GFRP bars leads to low strength and thereby reduces their load carrying capacity. These GFRP bars which were manufactured for electrical applications were tested earlier, before adopting this 'Manual Fiber-Trusion' method of manufacturing. Thus the option of using the readily available GFRP bars from the market was not preferred. To overcome the entire above problem, it was decided to produce the GFRP reinforcement bars having high strength. After experimenting by various methods, many trails were carried out to manufacture high strength bars economically. Some of the trail specimens are as shown in Figure 3.2.

Figure 3.2 Some trail GFRP reinforcement specimens fabricated

During the trail fabrication, the GFRP rebar specimens were manufactured by altering the composition of individual constituent materials and also by adopting different techniques of manufacturing. Fabrication of GFRP rebars and stirrups with open moulds of different sizes and shapes, fabricating stirrups using continuously looped fiber impregnated in matrix, casting of rebars using ribbed flexible pipes as mould, were some of the techniques that were tried during initial trails for fabricating the GFRP rebars and stirrups. Some of the other procedures adopted were; altering the percentage volume of the resin and fiber combinations in the composites. Ultimately, with a considerable effort, a new method suitable for fabricating the GFRP reinforcement material was found. This new method of fabricating FRP rebars through manual Fiber pulling resulted in the naming of this method as "Manual Fiber-Trusion" method by the author.

IV. Preperation Of Beam Specimens

A. 3.4.1 Sand Coating

After the finishing work, the GFRP bars and stirrups were coated with a layer of epoxy over which coarse sand was spread uniformly over the entire surface. The coarse sand used for this first layer of sand coating was obtained after sieving the sand through a 0.5 mm sieve. After curing for about two days, a second layer of fine sand was applied over the existing layer of coarse sand. The fine sand used for this purpose was obtained after passing through a 100 micron sieve. The fine sand fills between the gaps of the coarse sand in order to develop a complete rough surface over the entire GFRP material. Figure 3.10 shows sand coated GFRP stirrups kept under the sunlight for surface curing.

Figure 3.10 Curing of sand coated GFRP stirrups

B. 3.4.2 Fixing of Strain Gauges

To understand the internal behaviour of GFRP reinforcement in deep beams, all the deep beam specimens were affixed with electric foil strain gauges at critical points where the strains were expected to be more and need to be measured. In case of non-isotropic materials like the GFRP reinforcement, the material is very strong when the fibres are unidirectionally oriented. But in the same material, when the fibers are bent, it becomes weak leading to failure. To effectively understand the response of GFRP reinforcements, the strain gauges were placed at vital places over the surface of the GFRP bars and stirrups, where the cracks in the beam were expected to cause a considerable amount of strain in the GFRP reinforcement. After fixing the strain gauges over the GFRP reinforcements, the electrical resistances of the strain gauges were checked to ensure that all the strain gauges have the required resistance of 120 ohms. This also ensures the working condition of the strain gauges before concealing them inside the water proofing latex cover. Checking the resistance of the strain gauges which were fixed over the GFRP reinforcements is shown in Figure 3.11.

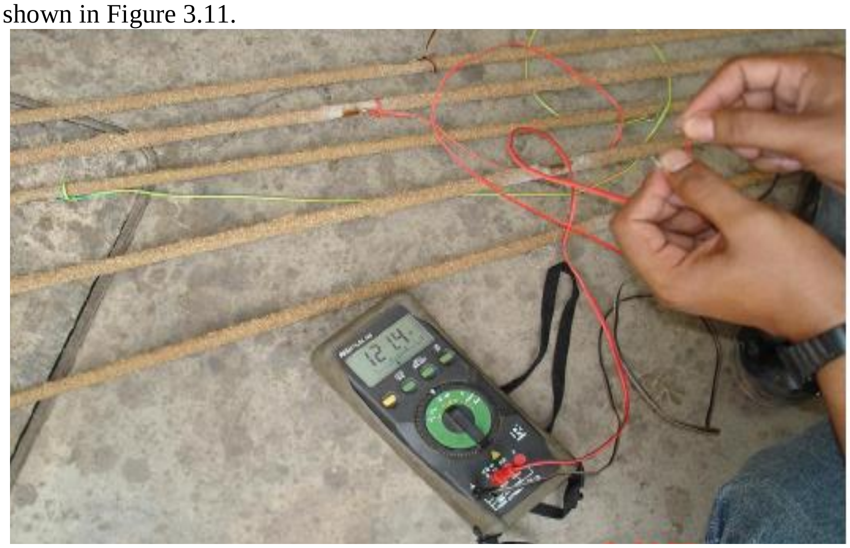

Figure 3.11 Checking of strain gauges fixed over the GFRP bars

C. 3.4.3 Binding of GFRP Reinforcement

After fixing the strain gauges over the GFRP reinforcements at selected locations, the bars and stirrups were tied according to the beams' reinforcement configurations. Self-locking nylon ties were used to bind the GFRP bars and stirrups. At every junction, two nylon ties were provided in a criss-cross fashion to ensure that no slippage of reinforcement takes place. A GFRP cage tied for beam GFRDB-8 by nylon ties is shown in Figure 3.12.

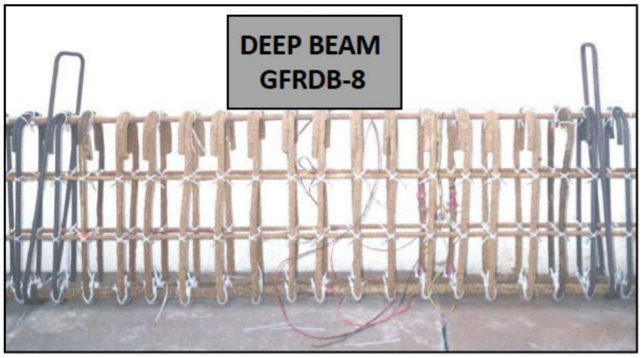

Figure 3.12 Binding of GFRP reinforcement for beam GFRDB-8

The main GFRP bars and horizontal bars were projected beyond the point of support for effective anchorage. The anchorage length was calculated as per the guidelines given in ISIS Design Manual 3(2001). To avoid any rupture in the GFRP bars with the steel stirrups which were provided beyond the test area, the ribs of the steel stirrups were removed by properly grinding at the places of contact with GFRP bars.

4.0 Experimental Results

D. 4.1.1 Results of GFRP Reinforced Deep Beams tested

The experimental testing conducted on GFRP reinforced deep beams to study the effectiveness of web reinforcement in contributing to the shear carrying capacity of the beams has been carried out. Nine GFRP reinforced concrete deep beams were tested. Out of the nine beams tested, one beam was reinforced without web reinforcement and this beam was considered as the ‘control beam’ for Series- I. In this series, two deep beams were reinforced only with vertical web reinforcement and without any horizontal web reinforcement and another two beams were reinforced only with horizontal web reinforcement and without any vertical web reinforcement. The remaining four beams in Series-I were reinforced with both horizontal and vertical web reinforcements but of different spacing. The change in the spacing of web reinforcement was considered as a variable parameter. To compare the effectiveness of the web reinforcement, the ‘control beam’ GFRDB-1 was designed to have no web reinforcement, which can be compared to all other beams which were provided either with vertical web reinforcement as in the case of beams GFRDB-2 & GFRDB-4 or with horizontal web reinforcement as in the case of beams GFRDB-5 & GFRDB-6 or with a combination of both the web reinforcement as in the case of beams from GFRDB-6 through GFRDB-9. The beam GFRDB-1 which is considered as the ‘control beam’ was used to evaluate the remaining all other beams. Except for the variation of the spacing of web reinforcement, there was no other change made either with respect to the materials used or with the dimensions adopted for all the beams tested in Series-I. The beams tested in this series were subjected to four point bending by applying two point loads at the top which are equidistant from the center line of the beam. The position of the point loads was chosen such that a constant shear span length of 350 mm was maintained on either side of the center line of the beam. The effective span was kept at 1050 mm for which a shear span of 350 mm was considered optimal for the study of shear behaviour of the GFRP web reinforcement within the chosen shear span length in this series. The test results of all the beams tested in this work have been discussed in subsequent sections. The discussions are related to the observations made during and after the testing of beams. From the test results observed, factors such as the strain in the concrete, strain in the GFRP reinforcement, deflection of the beam and crack widths in the concrete surface were analyzed. These in turn influence the shear behaviour and shear carrying capacity of deep beams. To understand the effectiveness of the web reinforcement, a correlation of all the observed parameters has also been carried out in this chapter. After a thorough investigation of all the related parameters, an empirical equation to predict the shear capacity in deep beams reinforced with GFRP web reinforcement has been proposed. The proposed design equation is compared with the experimental results obtained in this work to ascertain its credibility. The test results related to the ultimate strength of deep beams tested are shown in Table 4.1. The ultimate shear load carried by the beam is taken as half of the applied load at failure of the beam as it is symmetrically loaded. The applied shear load which is being resisted by concrete along with GFRP main and web reinforcements is analysed in correlation with the experimental strain values of the material.

Table 3.1 Test results of GFRP deep beams tested

S.No.	Beam designation	a /d Ratio	S_v (mm)	S_h (mm)	Ultimate Load P (k N)	ShearLoad V(k N)	Type of Failure
1.	**GFRDB-1**	0.70	-	-	350	175	Flexure -shear with Dowel Splitting
2.	**GFRDB-2**	0.70	150	-	490	245	Flexure -shear with Dowel Splitting
3.	**GFRDB-3**	0.70	75	-	460	230	Flexure -shear with Dowel Splitting
4.	**GFRDB-4**	0.70	-	150	406	203	Flexure -shear with Dowel Splitting
5.	**GFRDB-5**	0.70	-	90	610	305	Shear Failure
6.	**GFRDB-6**	0.70	150	150	560	280	Shear-Compression Failure
7.	**GFRDB-7**	0.70	150	90	740	370	Shear Failure
8.	**GFRDB-8**	0.70	75	150	805	402.5	Shear Failure
9.	**GFRDB-9**	0.70	75	90	900	450	Shear Failure

V. 4.2 BEHAVIOUR OF GFRP RC DEEP BEAMS

A. 4.2.1 FRP Deep Beams without Web Reinforcements

To assess the concrete contribution to shear in GFRP reinforced deep beams, the 'control beams' GFRDB-1 was cast without any web reinforcement. The 'control beams' were reinforced only with the main GFRP reinforcement at the bottom of the beam. The testing conducted on the 'control beams' were similar to that of the other beams tested in their respective series. In order to evaluate the contributions made by the web reinforcements in each series, each beam belonging to a particular series was compared against the control beam of its respective series.

During the tests conducted on the 'control beam' GFRDB-1, it was observed that the first crack was developed in the region between the two point loads. This first crack which happened to be a flexural crack originated from the bottom of the beam near the mid span section and was found to propagate directly upwards as the applied load was gradually increased. The crack moved till it reached almost 90% of the beam's depth and then got terminated when the applied load reached about 70% of ultimate load of the beam. The flexural crack was found to develop rapidly during the initial stages of loading and later the rate of propagation of the crack became relatively less. Just before the flexure cracks could terminate, flexural-shear cracks were developed on either side of the shear span of the beam. These cracks progressed diagonally till they reach the middle of the beam's depth on the south end and thereafter they remained almost vertical till they reached near the bottom of the top loading point. It was observed that the diagonal cracks took the vertical path at a level which is almost at the middle of the beam's depth. From the observations, the mode of failure in case of the beam GFRDB-1 can be postulated to have occurred due to flexure-shear cracking associated with dowel splitting cracks as could be seen in Figure 4.1. The GFRP main bars were sufficiently anchored beyond the supports to about twenty times the bar's diameter. In spite of providing sufficient anchorage to the main bars, some of the beams developed dowel splitting. This was particularly observed in beams without any web reinforcement or with single web reinforcement. The dowel splitting did not occur in beams when both the web reinforcements were present. Ultimately, the beam GFRDB-1 failed due to flexure-shear crack formation combined with dowel splitting. There was no formation of diagonal shear crack in this beam. From the failure mode, it was distinct that there was no arch action developed in this beam for the chosen 'shear span to effective depth' ratio without any web reinforcement. In case of the 'control beams', the absence of both the web reinforcements has led to the failure of the beam at a substantially lower value of the applied load when compared with the other beams.

5.0 Conclusions:

The conclusions arrived at during this study are summarized in this concluding chapter. The scope of future research is also mentioned at the end of this chapter. From the discussions made in the previous chapter and based on some vital observations, the following conclusions have been arrived at on the basis of the parameters involved.

1. The presence of a combination of both the vertical and horizontal GFRP web reinforcements plays a significant role in contributing to the shear carrying capacity of the deep beams. This is apparent by comparing the shear performance of beams GFRDB-9 and GFRDB-1. However the amount of the contribution made by the GFRP web reinforcement to carry shear varies depending upon its amount and its position.

2. Compared to the beams without any web reinforcement, there has been a three-fold increase in the ultimate load carrying capacity of deep beams provided with GFRP web reinforcement. This increase is considered to be

substantial. In case of beams with combined web reinforcement, this increase was even more sizeable which may be attributed to the effect of cage reinforcement.

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