RETROFITTING OF REINFORCED CONCRETE CURVED BEAMS FAILED IN SHEAR UNDER OUT OF PLANE LOADING USING CARBON FIBRE REINFORCED POLYMER

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DECLARATION

I declare that this is my work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other university or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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Name of the supervisor: Prof. (Mrs.) J.C.P.H. Gamage

Signature of the supervisor: Date:

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Under the Research Project, I had the opportunity of gaining valuable knowledge and first-hand experience on retrofitting methods and techniques. There are a number of persons whom I must pay my gratitude to for their assistance towards the successful completion of the research project and report.

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ABSTRACT

Carbon Fibre Reinforced Polymer (CFRP) strengthening techniques have gained popularity compared to other retrofitting methods due to its superior performance. While ample research had been done regarding retrofitting of straight beams that have failed in shear, as per the author's knowledge no research has been done regarding the retrofitting of curved beams that have failed in shear under out of plane loading. Due to the curvature of the beam, bending, shear and torsion all act together in resisting loads and hence it is important to study the effects of these on CFRP retrofitted curved beams.

An experimental study consisting of four curved beams of 2m and 4m radii of curvature, which had previously failed under the combined effect of shear and torsion for out of plane loading were selected and were externally retrofitted using CFRP strips. The beams were then re-tested for their new failure loads and the load carrying capacities were compared with their original values.

From the experimental program it was observed that two beams exhibited load carrying capacities of 122% and 117% of their original value while the other two exhibited load carrying capacities of 85% and 90%. The predominant mode of failure was CFRP strip de-bonding from its substrate. Therefore it was identified that by anchoring the CFRP strips to the substrate the carrying capacity could be significantly increased by delaying de-bonding load by distributing stresses across two of more CFRP strips.

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1. INTRODUCTION

1.1. Background

In the earlier days before reinforced concrete, plain concrete curved beams were primarily used vertically and in plane with the applied loads. Arched bridges and doorways are examples for such. The reason for plain concrete to be primarily used in in-plane loaded vertical applications was that the concrete was axially stressed in compression in such applications for which concrete had ample strength. With the introduction of steel embedded into concrete to produce ‘reinforced concrete’, the resulting composite now had superior strength in tension too in addition to its inherent compressive strength, thereby allowing the curved beams to resist tensile stresses caused by out of plane loading. Nowadays both in plane and out of plane loaded curved beams are used as architectural and aesthetic features of structures as well as for functional requirements such as horizontally curved viaducts and bridges. In the case of in-plane loaded vertical curved beams, the axial loads could be predominant in addition to the bending moment and shears that could develop depending on curved shape of beam and type of loading. When the loading is out of plane from the plane of curvature, torsional moments are induced in addition to the bending moments and shear forces. Therefore additional stresses are developed. The failure of such beams could be due to bending or the combined effect of shear and torsion.

Reinforced concrete has become a popular construction material for curved beams. Due to the effects of torsion, curved beams which are loaded out of plane are more prone to failure than straight beams. Therefore retrofitting of curved beams has become a necessity. Different retrofitting methods are currently used for straight beams such as steel plates, jackets and Fibre Reinforced Polymer (FRP).

Bonding the steel plate to the tension face of the concrete member with adhesive resin is a technology used to improve its flexural strength (Fleming and King,1967). This technology had been widely used to strengthen many bridges and buildings around the world.

However, the corrosion of steel plates and the subsequent weakening of bond between steel and concrete and difficulty of installation are shortcomings of this method. To overcome these problems researchers came up with FRP as an alternative to steel plates. As early as 1978, Germany reported experimental work using FRP materials to renovate concrete structures (Wolf and Miessler 1989). Swiss research led to the first use of externally bonded FRP systems to reinforced concrete bridges for flexural reinforcement (Meier 1987; Rostasy 1987). In Japan in the 1980s, the FRP system was first applied to reinforced concrete columns to provide more confinement and containment of reinforcement, because the buildings rested on a major earthquake region and needed to be strengthened against earthquakes while causing minimum interference to the function of the building (Fardis and Khalili, 1981; Katsumata et al., 1987). After the Hyogo Prefecture-Nanbu earthquake in 1995, Japan's FRP usage suddenly increased (Nanni, 1995).

Externally bonded fibre reinforced polymer (FRP) systems gained popularity in reinforcing and renovating existing concrete structures since the mid 1980’s. Concrete beams, slabs, columns, walls, vaults, domes, tunnels, silos, pipes and trusses too are retrofitted using FRP. Externally bonded FRP systems are used to strengthen steel, wood and masonry as well.

1.2. Research objectives

- To conduct an experimental program to determine the strength that can be re-achieved by retrofitting curved beams which had failed in shear
- To arrive at a suitable fibre arrangement to effectively strengthen in shear

- To propose recommendations for new and more effective fibre arrangements by extrapolating results of experimental program

1.3. Methodology

The methodology followed in this study is as follows.

A thorough literature review was done in order to identify the current knowledge base regarding curved beams, their applications, failure modes and the retrofitting materials & methods. A research gap in the knowledge base was then identified in an important and practically applied area of study.

An experimental method was then devised with the scope set to be on the identified research gap. A suitable testing setup was then arranged and the experimental program was carried on.

The conclusions were then drawn from the experimental results and recommendations were given on how it could be improved with further research areas on which studies could be proceeded.

A summary of the methodology is presented in Figure 1.

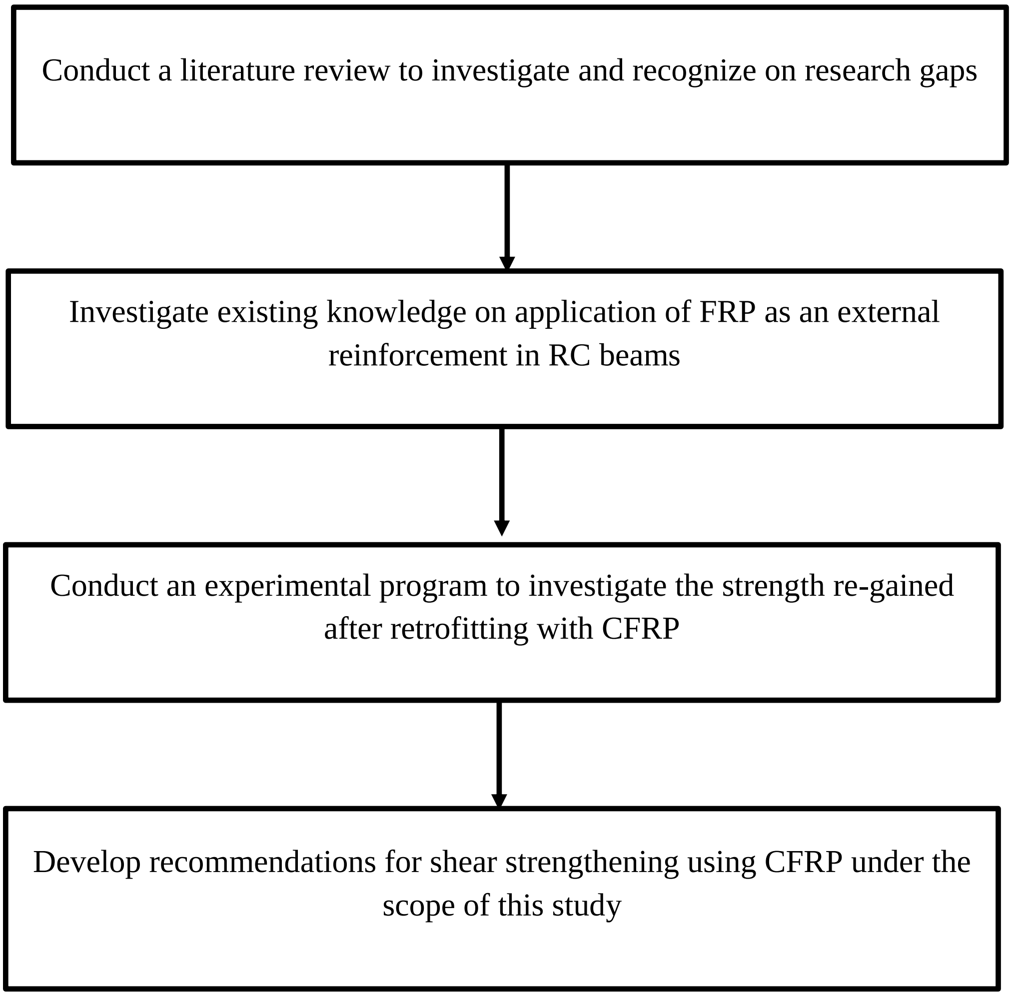


Figure 1: Summary of methodology

1.4. Significance of the Research

It was found out that even though retrofitting of straight RC beams using CFRP is widely used in the industry, there is no experimental data for retrofitting of curved RC beams which are loaded out of plane. Therefore the data of this research can contribute to this knowledge gap under the scope and limitations of this research.

1.5. Thesis arrangement

This thesis is presented in several chapters and the description of each chapter is as follows

Chapter 1: Introduction

A general introduction is given in this chapter with a brief a background and the research objectives. The methodology followed is also presented.

Chapter 2: Literature review

A detailed literature survey regarding the topic is outlined in this chapter. The research gap have been identified and explored henceforth.

Chapter 3: Experimental program

This chapter outlines the properties of material used, specimen preparation and testing procedure followed.

Chapter 4: Observations and Results

This chapter explains the observations and results of the experimental program

Chapter 5: Conclusions and recommendations

This chapter identifies the areas that should be further researched and also provides important aspects to be considered in shear retrofitting using CFRP for out of plane loaded curved beams.

2. LITERATURE SURVEY

This chapter is dedicated to exploring the existing knowledge base regarding application of FRP as an external reinforcement in reinforced concrete members. Though numerous studies and research has been conducted in this area there still are research gaps. The purpose of this chapter is to recognize such research gaps in important study areas and focus the research on them such that the research findings would contribute to shrinking the research gap.

2.1 Curved Beams

Curved beams are popular among architects mainly for its aesthetic appearance. Also in addition to aesthetics, curved elements are required functionally such as for expressway entrance ramps and viaducts as shown in Figure 2. There are two main types of curved beams with respect to the plane of curvature and direction of loading. Those are in-plane loaded, in which loading is in the same plane as curvature and out of plane loaded, where the loading is perpendicular to the plane of curvature. There are significant behavioural differences between straight and curved beams. Therefore curved beams need to be specially considered in design and retrofitting.

Figure 2: Curved bridge (Yindeesuk, 2009)

2.1.1. In-plane loaded curved beams

In plane loaded beams are those that are loaded in the same direction as the plane of curvature, such as arches. (Lee, 1956)

Vertically curved in plane loaded beams have much higher load carrying capacities compared to their straight counterparts due to the loads being transferred to supports mainly through axial forces with bending moments and shear forces of very low magnitudes compared to straight beams. The self-weights too are transferred predominantly through axial thrusts in such beams. Thus, the ultimate carrying capacity of such vertically curved in plane loaded beams depend predominantly on compressive strength of concrete. (Lee, 1956)

2.1.2. Out-of-plane loaded curved beams

Horizontally curved beams can be defined as the beams curved in plan such as curved girder members in circular buildings & domes, beams in curved sections of bridges and curved elevated highway ramps (Lee, 1956).

When a horizontally curved beam is loaded out of plane, a twist as shown in Figure 3, resulting in a torsional force is applied in addition to the normal shear forces and bending moments. This torsion component is not applied to a straight horizontal beam with vertical loads. A part of the torsional stresses will also be added to the shear stresses, which will create a tendency for the beam to fail in shear prematurely. Thus horizontally curved beams loaded out of plane are more prone to shear failure compared to straight beams (Wong, 1970)

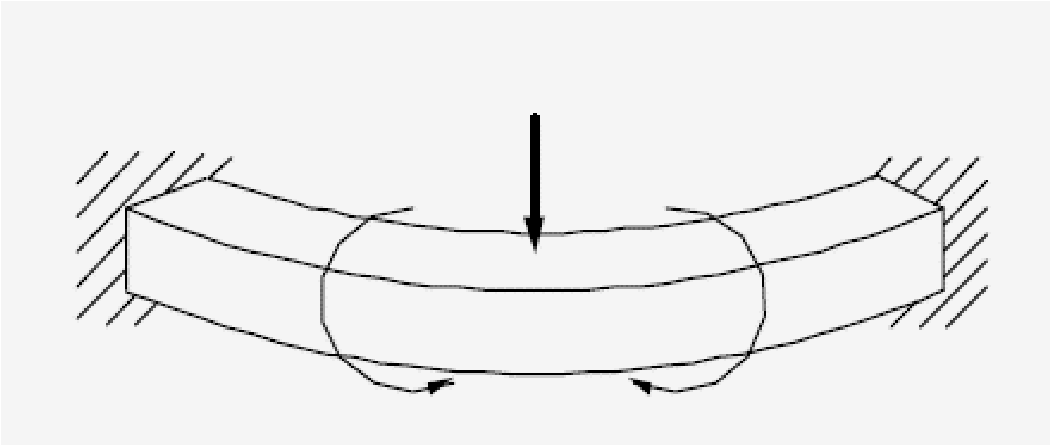


Figure 3 : Out of plane loaded horizontally curved beam (*Tamura and Murata, 2010*)

Therefore the need can arise to retrofit out of plane loaded curved beams which have failed in shear. Fibre reinforced polymer is a suitable option to retrofit such beams. (Kachalakev, 2000)

2.2. FRP materials

Fibres give strength and stiffness to Fibre Reinforced Polymer composites. Carbon, glass and aramid fibres are primarily used in the construction industry. Selection of the type of fibre is done considering factors such as cost, strength requirement and environmental conditions. Basalt fibres, although not as common as the other three fibres, are under consideration as an alternative for glass fibres. (Karaca and Buyukozturk, 2002; Nanni et al., 2014)

2.2.1. Carbon Fibre

There could be a wide variation in strength and stiffness of carbon fibres depending on the manufacturing process. Hence, carbon fibres are classified as high modulus and low modulus, based on the Young's modulus. PAN based (Polyacrylonitrile, a thermoplastic polymer derived from oil or coal) carbon fibres are used for civil engineering applications due to its high strength and high Young's modulus. Pitch based carbon fibre (also known as asphalt or refinery bottoms which comes from the 'undistillable' products at the bottom of the petroleum or coal distillation column/tower) has a higher Young's modulus but a relatively low strength. Thus it is mainly used for aerospace application. Viscose or rayon based carbon fibres belong to low modulus carbon fibres. Carbon fibre has superior chemical properties such as higher alkali/acid resistance, high electric conductivity. Carbon fibre also possesses the ability to cause galvanic corrosion on metals which could be used for the benefit of certain applications. Low thermal expansion, low impact resistance and brittleness can be identified as significant physical properties of carbon fibre. (Karaca and Buyukozturk, 2002; Nanni et al., 2014)

2.2.2. Glass Fibre

Electrical Glass Fibre (E-Glass), High Strength Glass Fibre (S-glass) and Alkali Resistant Glass Fibre (AR-glass) are the three main types of glass fibre used in civil engineering applications. S glass has higher stiffness and strength while E glass provides higher corrosion resistance and electrical insulation. S glass is costlier than E glass. As the name suggests, AR-glass provides superior alkali resistance compared to the other two. Thus a suitable type must be used depending on application (Karaca and Buyukozturk, 2002; Nanni et al., 2014)

2.2.3. Aramid Fibre

Aramid fibre is an organic synthetic material with superior mechanical properties containing high strength, high toughness, high resistance to creep and high impact resistance. Because of its light weight and high strength, aramid fibres are preferred in sports industry and military applications such as bullet resistant Kevlar vests, in addition to civil engineering applications. Aramid fibre also possesses electrical, fire and chemical resistance but is vulnerable to Ultra Violet light (Karaca and Buyukozturk, 2002; Nanni et al., 2014).

2.2.4. Basalt Fibre

Basalt Fibres are relatively new to civil engineering. It is mainly used as an alternative to glass fibre due to its better strength, stiffness, ease of production and thus low manufacturing cost and higher availability of basalt material. (Nanni et al., 2014)

A summary of various properties of fibres are given in Table 1.

Table 1: Material and physical properties of fibres (Nanni et al, 2014)

Fibre type	Density (kg/m3)	Tensile strength (Gpa)	Tensile Modulus (Gpa)	Ultimate Tensile Strain (%)	Poisson's ratio
E-glass	2501	3.45	72	2.4	0.22
S-glass	2501	4.55	86	3.3	0.22
AR-glass	2254	1.79 – 3.45	70 – 76	2.0 – 3.3	N/A
High modulus carbon	1952	2.48 – 4.00	350 - 650	0.5	0.2
Low modulus carbon	1750	3.5	240	1.1	0.2
Aramid (Kevlar 29)	1440	2.76	62	4.4	0.35
Aramid (Kevlar 29)	1440	3.62	123.5	2.2	0.35
Aramid (Kevlar 29)	1440	3.45	174	1.4	0.35
Basalt	2799	4.82	88.5	3.1	N/A

2.3. Fibre Reinforced Polymer Composites

Polymer composites are multi-phase materials made by combining a polymer matrix with fillers and reinforcing fibres in order to create a bulk material that outperforms the separate base materials (Structures & Science, 2014). Fibres are employed to strengthen the polymer and improve mechanical qualities like stiffness and strength (Structures & Science, 2014). Generally, fibres have high resistance and brittle behaviour. The polymer matrix (resin + filler + additives) commonly have low resistance, load transfer and stress distribution ability between fibres, function of protecting fibres, and keeping the fibres in position.

Most commonly used FRP materials in structures are Carbon fibre reinforced polymer (CFRP), Glass fibre reinforced polymer (GFRP), Aramid fibre reinforced polymers (AFRP) and Basalt fibre reinforced polymers (BFRP). The suitability of the various fibres for specific applications depends on several factors including the required strength, stiffness, durability considerations, cost constraints, and the availability of component materials (Structures & Science, 2014). A comparison of the properties of various FRPs are presented in Table 2.

Table 2: Comparison of the properties of different types of FRPs
(Structures & Science, 2014)**,**(Islam, 2014)**,** (Sonnenschein, Gajdosova, & Holly, 2016)

Criterion	Carbon	Glass	Aramid	Basalt
Tensile strength MPa	1200 -2250	400 - 1800	1000-1800	1000 - 1650
Density kg/m^3	1600 - 1900	1600 - 2000	1050 - 1250	2700
Elastic modulus GPa	147 - 165	30 - 55	50 - 74	45 -60
Fibre Alkaline resistance %	95	15	92	More than GFRP
Fibre Acidic resistance %	100	60 - 85	100	-
Fibre content %	65 -70	50 - 80	60 - 70	-
Cost $/m^2	10 - 500	1 - 5	10 - 50	Same as GFRP

2.3.1. Types of Matrix Resins

The purpose of matrix resins is to transfer stresses to the fibres, which are the main stress bearing components of the polymer. The resin also provides protection to the fibres as well as giving shear and compressive rigidity to the fibre composite. There are two types of matrix materials; thermosetting polymers and Thermoplastic polymers. Both polymers have a liquid or semi-solid consistency and harden only after curing. However, thermoplastic polymers can be reverted back to its liquid state even after curing, which makes it unsuitable for civil engineering applications. Thus, thermoset polymers are the most widely used type of matrix material for civil engineering retrofitting applications (Grumezescu and Grumezescu, 2019; Karaca and Buyukozturk, 2002; Nanni et al., 2014)

There are three major types of thermoset polymers that are used in practice.

i. ***Epoxies***

Epoxies are dominantly used in civil engineering applications due to their superior mechanical and physical properties and low shrinkage. However, the cost of epoxies are higher, the curing time is longer and is more difficult to apply to concrete. Still it is preferred over the other two types of polymers owing to their advantages mentioned above.

Good adhesion to fibres, better resistance against environmental conditions and high compatibility with all four main types of fibres can be recognized as favourable properties of epoxies (Karaca and Buyukozturk, 2002; Nanni et al., 2014).

ii. ***Vinyl esters***

Vinyl esters come close to mechanical properties of epoxies, but has a higher cost compared to epoxies. For GFRP applications vinyl esters are often used due to its impressive resistance to alkaline environments and good wetting and adhesion with glass fibres (Karaca and Buyukozturk, 2002; Nanni et al., 2014)

iii. ***Polyesters***

Two types of polyesters are commonly used: orthophthalic and isophthalic acidic polyesters. Iso polyesters have better mechanical properties and better thermal and chemical resistance compared to ortho polyesters, even though they are costlier than ortho polyesters.

However, polyesters are not much used for FRPs in civil engineering applications due to its low strength and low resistance to environmental conditions in comparison to epoxies and vinyl esters (Karaca and Buyukozturk, 2002; Nanni et al., 2014).

Furthermore, fillers are also used to engineer the properties of polymers and reduce the overall cost of the materials (Grumezescu and Grumezescu, 2019; Karaca and Buyukozturk, 2002; Nanni et al., 2014).

A comparison of these thermosetting polymer matrices is presented in Table 3.

Table 3: Material and Physical Properties of resin matrix (Nanni et al., 2014)

Resin type	Density (kg/m3)	Tensile strength (MPa)	Longitudinal modulus (GPa)	Poisson’s ratio (%)	Glass Transition temperature (C^0)
Vinyl ester	1127 - 1365	3.0 – 8.9	3.00 – 3.45	0.36 – 0.39	70 - 165
Polyester	1187 - 1424	4.2 – 11.3	2.76 – 4.14	0.38 – 0.40	70 - 100
Epoxy	1187 - 1424	5.9 – 6.5	2.07 – 3.45	0.35 – 0.39	95 - 175

2.4. FRP Application Methods

The need for FRP application for RC members arises due to structural failures caused by design errors and construction errors, change in purpose of structure or structure deterioration due to environmental attacks. Following the proper application method is crucial since if not applied properly, premature delamination could occur, thereby not being able to achieve the expected strength gain. Hence the proper form of FRP, proper type of FRP and proper arrangement of FRP and proper constituent materials need to be selected depending on requirement (Ehsani, M. R., Saadatmanesh, H. & Tao, 1996).

FRP can be installed in two methods primarily.

1. Near Surface Mounted Method (NSM)

2. Externally Bonded Reinforcement Method (EBR)

An adhesive is required for both methods, which could either be a cement grout or specific epoxy for the purpose. Out of the above two methods, NSM method has shown to be advantageous over EBR method. NSM provides better adhesion to the substrate due to the larger area for binding. Also NSM method has proved to be more durable and easier to protect thermally. The NSM method has exhibited a flexural capacity improvement of approximately two times compared to the EBR method (N.A.Amarasinghe, J.C.P.H. Gamage, 2015).

The beams can fail in shear when the flexural capacity of beam is enhanced due to various methods of FRP application for flexure. Therefore the beams will have to be enhanced for shear as well to prevent this from occurring. When the two methods are used in parallel, the NSM method dominates the strength. The plates also act as a transverse clamping to the FRP fabric strips adhered by EBR method. If the two methods were done perpendicular tone another, the EBR method dominated the failure pattern. The NSM method and parallel direction integration method exhibit more ductile behaviour, thereby making it the preferable choice for earthquake resistance (N.A.Amarasinghe, J.C.P.H. Gamage, 2015).

2.5. Flexural enhancement using FRP composites

For flexure enhancement NSM method using FRP plates is preferred since FRP cannot be aligned properly if laminate is used in EBR method, nor held in place without crumpling in the case of fabric. In the NSM method the plates could be aligned as required along the curvature, thus making it the preferred method in flexure enhancement (N.A.Amarasinghe, J.C.P.H. Gamage, 2015).

From experimental studies it is proposed that flexural strength enhancement by FRP external reinforcement can vary between 10 to 160 percent (Diab et al., 2020; Ebead, 2010; Guo et al., 2021; Lorenzis and Nanni, 2002; Meier, 1995; Paul and Datta, 2018; Ritchie et al., 1991; Sharif et al., 1994). This wide variation can be attributed to the presence of several factors such as the condition of the bond between concrete and FRP, form and type of FRP and the failure mode of the system. The main failure modes in FRP strengthened RC beams under flexure are as follows.

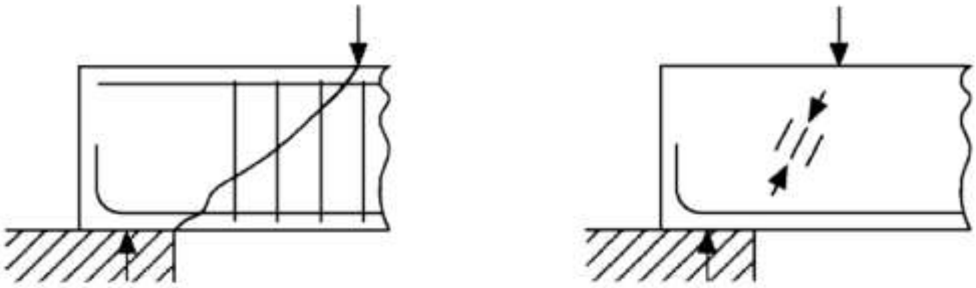
a) Concrete crushing prior to reinforcing steel yielding
b) Steel yielding under tension followed by FRP laminate rupturing
c) Steel yielding under tension followed by concrete crushing
d) Concrete cover delamination due to shear or tension
e) Plate end interfacial de-bonding failure
f) Intermediate crack induced interfacial de-bonding failure

(Hota V. S. GangaRao, 1998; Teng et al., 2001; Zhang et al., 2016)

The flexure enhanced RC beam can fail in shear if enhanced flexural capacity exceeds shear capacity. In such cases shear enhancement would also have to be done in order to avoid sudden brittle failure due to shear (N.A.Amarasinghe, J.C.P.H. Gamage, 2015).

2.6. Beam failure by shear

Shear failure may arise either by diagonal tensile failure or diagonal compressive failure as in Figure 4.



(a) (b)

Figure 4: Types of shear failure: (a) diagonal tensile failure (b) diagonal compressive failure (Arya, 2009)

Tension is induced in a diagonal direction a result of shear and the beam could fail in this tension if no shear reinforcement is provided or if the provided shear reinforcement yields. This is a brittle failure and hence at least nominal shear reinforcement is provided to avoid catastrophic brittle failures. Once shear links are provided, the failure can either be due to yielding of shear reinforcement or by concrete compressive failure in the opposite diagonal direction. Therefore if applied loads induce greater compressive stresses in diagonal direction than what concrete can bear, the beam would have to be re-sized to bring diagonal compressive stresses under allowable limits.

This will ensure that the failure mode to be yielding of shear links which is a relatively ductile failure mode (Arya, 2009; BS 8110-1:1997, 1997; BS EN 1992-1-1, 2004). Figure 5 illustrates results exhibited by different shear failure modes (Dinh et al., 2011)

(a) (b)

(c)

Figure 5: Shear failure modes and crack patterns (a) diagonal tensile failure with shear links (b) diagonal compressive failure (c) diagonal tensile failure without shear links (Dinh et al., 2011)

Type of shear failure is affected by three main factors and effects. Grade of concrete, aggregate interlock across cracked region and the dowel action provided by the tensile reinforcement. The compressive and tensile strength of concrete is governed by the grade of concrete. Once initial cracking occurs, the aggregate across crack surface interlocks, thereafter resisting shear force through aggregate interlocking. Further increase in shear force would exceed the shear resisted by aggregate interlock and loads will be transferred to shear links and the dowel action of longitudinal reinforcement. The beam will continue to resist shear until yielding of shear links occur or until concrete fails in compression in diagonal direction (Arya, 2009; Cranston et al., 1996; Dinh et al., 2011; Johnoehlers and Rudolf, 2004)

The aggregate interlock and dowel action are illustrated well in Figure 6.

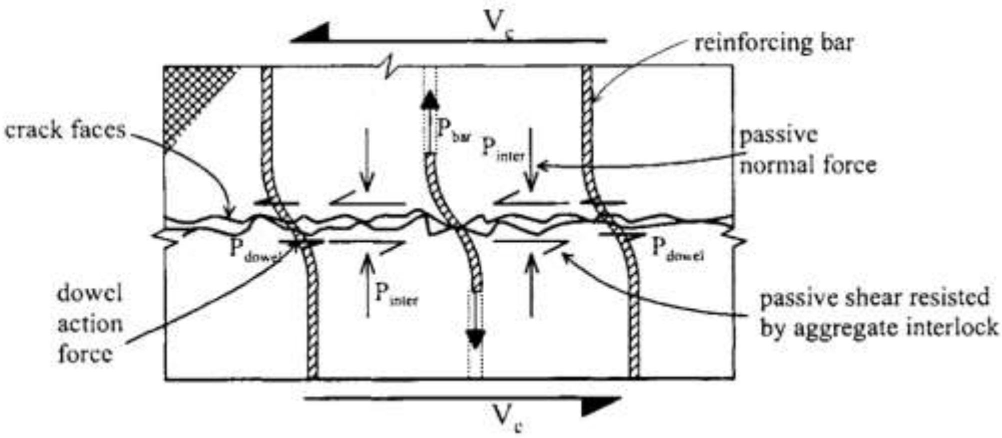


Figure 6: Aggregate interlock and dowel action of tensile reinforcement (*Johnoehlers and Rudolf, 2004*)

2.7. Shear enhancement using FRP material

Beams will have to be retrofitted for shear as a result of shear damaging or shear enhancement would have to be done as a result of flexural strength enhancing using FRP. Various methods of shear enhancing have been studied and researched as outlined below.

2.7.1. Externally bonded fibre laminates (EB method)

Four full-scale reinforced concrete beams from an existing bridge were duplicated (Horsetail Creek Bridge, Oregon, USA). The original beams were significantly under strengthened for shear, especially in light of the anticipated rise in traffic loads. One replicated beam acted as a control beam, while the other three were retrofitted using different configurations of carbon fibre reinforced polymers (CFRP) and glass fibre reinforced polymer (GFRP) composites to mimic the retrofitting of the original structure. As indicated in the Figure 7, CFRP unidirectional sheets were used to boost flexural capacity while GFRP unidirectional sheets were used to prevent shear failure and four-point bending test was carried out. Beams reinforced with FRP showed considerably larger load carrying capacities compared to the control beam. Static load capacities even higher than 150% of the original beam capacity could be achieved through retrofitting using FRP (D. Kachlakev & D.D. McCurryb, 2000).

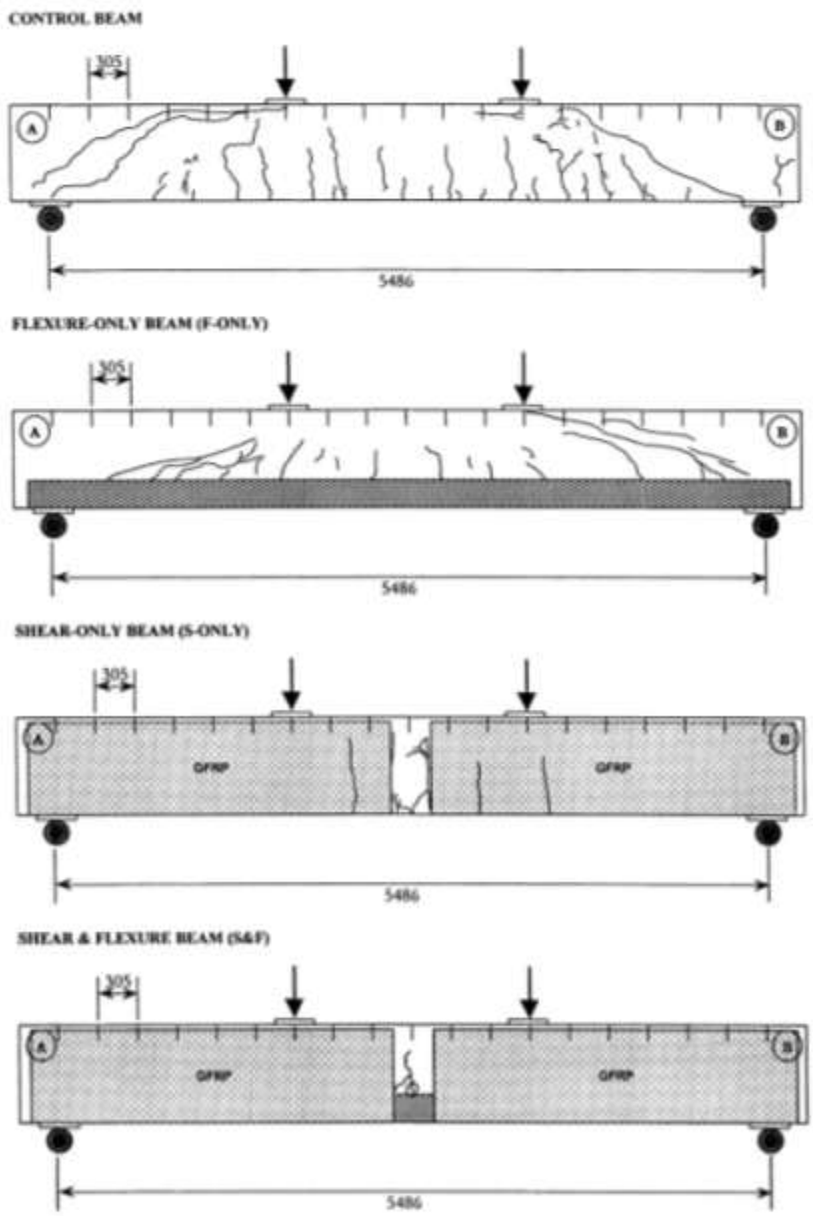


Figure 7: Experiment by D. Kachlakev & D.D. McCurryb(2000)

2.7.1.1. *Wrapping with fibre sheets*

A total of sixteen reinforced concrete beam specimens were designed and built according to AS 360. Depending on the beam reinforcement arrangement, the specimens were divided into four groups. Beams in Group 1 were strengthened with longitudinal bars of 2T16 and 500MPa tensile strength. Beams strengthened with

2T16 bars and helices within the compressive zone were assigned to Group 2. Groups 3 and 4 were made up of 2T20 and 2T24 longitudinal bars with helical reinforcement in the compressive zone of the beams. The helical reinforcement had a fixed pitch of 30 mm and an internal diameter of 50 mm, with an overall length of 400 mm to allow the helix to encapsulate the compression zone of each beam. FRP Strengthening materials were solely applied to the beam's pure shear span as in Figure 8. The results of testing the sixteen beam specimens showed that all of the tested parameters contribute to the beams' strength, and the findings confirm that FRP sheet strengthening techniques can be used to significantly increase shear capacity, with efficiency varying depending on the tested variables. Retrofitting with FRP is a viable rehabilitation procedure for both repair and strengthening. The higher the number of layers of FRP materials used in wrapping, the higher the flexural strength. The increase of wrapping layers would result in a proportional increase in beams shear capacity (M.N.S. Hadi, 2003)

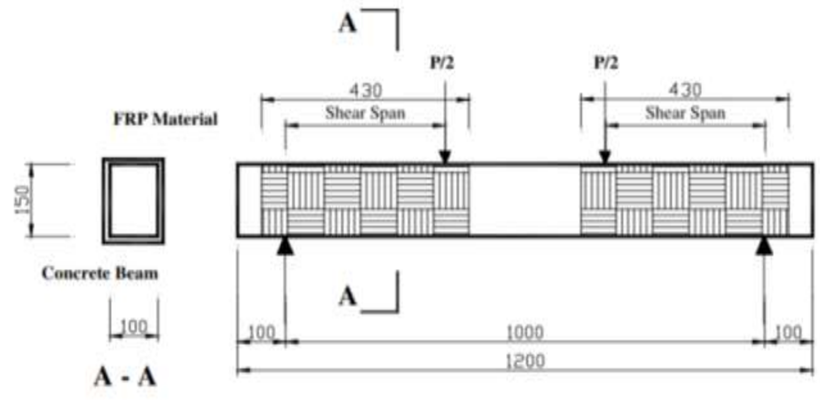


Figure 8: Experiment by M.N.S. Hadi (2003)

Retrofitting of RC beams pre-damaged in shear with CFRP sheets using different fibre arrangements were studied and the results were compared (Said M. Allam & Tarek I. Ebeido , 2003). Fibre was installed in Strips, U- strips, Wings and U - Wings and was reloaded to find the failure load as shown in Figure 9.

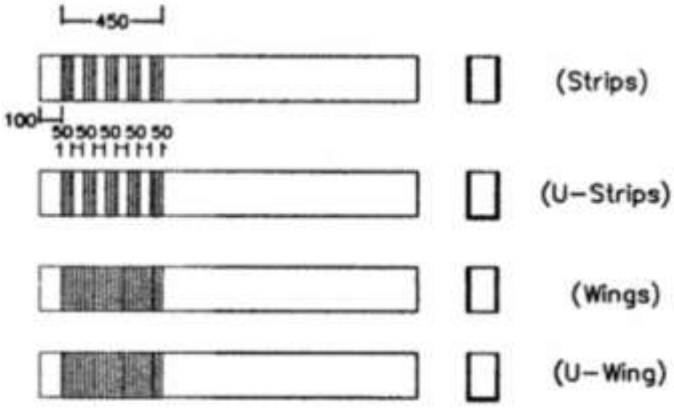


Figure 9: Experiment by Said M. Allam & Tarek I. Ebeido (2003)

Two control beams, having a shear span to depth ratio a/d = 2.57, were tested to failure in their original condition without any strengthening. Secondly, twelve beams, having a shear span to depth ratio a/d = 2.57, were preloaded to different shear cracking levels. Following that the beams were retrofitted using four different schemes of CFRP as illustrated in Figure 9 above. Then the beams were reloaded to failure. Another four beams having shear span to depth ratio a/d =1.71 were first strengthened with the same four different schemes of CFRP and then they were loaded to failure. Finally, the experimental results for the contribution of the CFRP sheets to the total shear force capacity of beams were compared to theoretical results from an analytical model found in the literature. Based on this research the following conclusions had been drawn:

- The application of CFRP sheets is an efficient method for retrofitting preloaded cracked reinforced concrete beams. The application of such CFRP sheets significantly enhanced the ultimate shear force capacity of the beams and increased their stiffness thus significantly reduced the deflection.
- The efficiency of CFRP in retrofitting or strengthening reinforced concrete beams deficient in shear depends mainly on the scheme of CFRP applied to the beam. Such efficiency increases as the area of CFRP sheets bonded to the beams increases. Also, retrofitting schemes that provide enough

anchorage length are more efficient than other schemes since de-bonding failure of CFRP is prevented or delayed.

- The preloading level that the beam is subjected to significantly affects the efficiency of retrofitting in the case of Strip scheme. As the preloading level increases the efficiency of retrofitting decreases.
- The decrease in the efficiency of CFRP retrofitting with the increase in the preloading level is not significant in the cases of U-Strips, Wings, and U-Wing schemes. This is due to the enough anchorage length (U-Strip) or the large area of CFRP sheets applied to the beam (Wings) or both (U-Wing). The application of large area of CFRP sheets can successfully arrest the cracks and the use of enough anchorage length can confine the tensile reinforcement and thus increase the contribution of the dowel action. Consequently, the beam can restore higher shear strength even if it was pre-cracked to any stage.
- The failure mode of retrofitted reinforced concrete beams depends mainly on the scheme of CFRP applied. A sudden explosive failure mode always takes place in the case of Strips or U-Strips scheme by de-bonding of the strips at their ends showing a de-bonding shear failure mode. Also, as the level of preloading increases the failure becomes less explosive.
- A more ductile mode of failure takes place in the case of beams retrofitted using CFRP sheets in the form of U-Wing. The failure mode in this case is a pure flexural one. In most of the cases, failure of reinforced concrete beams, having a shear span to depth ratio less than 2, and strengthened using CFRP sheets takes place by concrete crushing due to the diagonal compression component of shear forces.
- The existing models found in the literature for estimating the contribution of CFRP to the ultimate shear force capacity of reinforced concrete beams are reliable. Results from applying these models showed good agreement with the current test results. However, theoretical results are always found to be greater than the experimental ones.

- Also the researchers have recommended that more theoretical models be developed for the estimation of the contribution of CFRP to the total shear capacity of beams having low values of shear span to depth ratio (Said M. Allam & Tarek I. Ebeido , 2003)

2.7.1.2. Inclined Fibre Strips

Six RC beams with different CFRP retrofitting arrangements were fabricated and tested (Riyadh Al-Amery & Riadh Al-Mahaidi , 2006). One beam was kept as a control beam without any form of CFRP strengthening and the rest were either strengthened for shear using CFRP straps, strengthened for flexure using CFRP sheets or coupled CFRP straps and sheets were used for both shear and flexure enhancement.

The test results have concluded in the following;

- The CFRP straps contribute to the shear resistance of beams together with the steel stirrups.
- CFRP straps acted as an anchorage mechanism for CFRP sheets thereby preventing the de-bonding failure of sheets.
- A significant reduction of interface slip between CFRP sheet and concrete substrate to one tenth of its un-anchored value was obtained through CFRP strap anchoring. The reduction in slip exhibited better composite behavior between CFRP sheets and concrete beam
- A 95% increase in flexural strength was seen when CFRP sheets were anchored using CFRP straps. When no CFRP strap anchoring was used, only a 15% increase in flexural strength was observed.
- The major mode of failure for the beams that had CFRP strap anchoring was a ductile flexural failure with considerable yielding and elongation of longitudinal reinforcement prior to the rupture of CFRP sheets and concrete crushing.

2.7.2. Using Fibre Rods - Near Surface Mounted (NSM Method)

Steel rods mounted in NSM method have been used in strengthening structures since 1950's even though the use of FRP rods is quite recent. The earliest reference in literature for the use of steel rods dates back to 1949 (Asplund, 1949). An RC bridge in Sweden had exhibited excessive settlement during its construction in 1948, which was identified to have been caused by under- reinforcement for negative moments. Thus the negative moment capacity was increased by inserting steel rods into grooves cut out into the surface and later filling them with cement mortar. Different arrangements for groove were studied to select the most suitable one (Asplund, 1949).

Currently, FRP rods have replaced steel rods and specific epoxy has replaced cement mortar. The primary advantage of FRP over steel in rods used for NSM method is the corrosion resistance of FRP due to the very close proximity of rod to the surface (Laura De Lorenzis & Antonio Nanni, 2001)

The rod application method is as follows. A groove is cut onto the surface. The size of groove is selected allowing for clearance around rod for bonding. The groove is then filled halfway with epoxy and the rod is pressed into groove so that epoxy paste flows around rod. Any left out space is then filled with epoxy to completely enshroud the rod in epoxy inside the groove and to create a levelled surface (Laura De Lorenzis & Antonio Nanni, 2001)

.The following conclusions have been drawn from the experiment (Laura De Lorenzis & Antonio Nanni, 2001).

- NSM FRP rods are an effective way of increasing shear capacity of RC beams. When no steel stirrups have been incorporated in beam, the rods can increase shear capacity up to 106% of that of the control beam
- The NSM rods contribution to further shear strength enhancement is significant even when steel stirrups are present. The Fibre rod reinforced beam showed a shear capacity increase of 35% compared to the non - retrofitted beam when both contained steel stirrups.

- One failure mode observed was de-bonding of FRP rods. This could be prevented by longer bond lengths, anchoring mechanisms or by using 45 degree inclined rods at closer spacing.
- The controlling factor for failure became concrete cover splitting once de-bonding was prevented. Unlike steel stirrups, the fibre rods do not provide any restraint for longitudinal reinforcement. Furthermore the dowel forces exerted to surrounding concrete at rod ends eventually leads to cover delamination and loss of anchorage.
  (Laura De Lorenzis & Antonio Nanni, 2001)

2.7.3. Using fibre rods - Embedded through Section (ETS method)

While the NSM rod embedment method essentially deals in the application of rods on side cover region of beams, the Embedded Through Section (ETS) method drives the rods in the middle of the beam (Chaallal, Mofidi, Benmokrane & K. Neale ,2011).

Although both NSM and ETS methods are efficient to some extent, both exhibit certain shortcomings such as high potential to de-bonding, the need for surface preparation and vulnerability to vandalism and fire. However, ETS method is less time consuming, needs less adhesive and does not require highly specialized skilled workers to install. The purpose of the experimental study was to compare externally bonded FRP methods, NSM rod method and ETS method in enhancing shear (Chaallal, Mofidi, Benmokrane & K. Neale, 2011).

It was seen that incorporation of EB U straps had an average increase in shear capacity of 23%, NSM FRP rods had an average shear capacity increase of 31% and ETS FRP rods had an average shear strength gain of 60%, thereby deeming the ETS method to be the best in terms of shear strength gain (Chaallal, Mofidi, Benmokrane & K. Neale, 2011).

Furthermore, the following conclusions can be drawn from the experimental study.

- Beams strengthened with externally bonded sheets failed by sheet de-bonding whereas the beams strengthened with NSM FRP rods failed by concrete cover separation.
- The failure of beams fitted with ETS rods were mainly from flexure, implying that the rupture force was not developed in ETS rods and the beam prematurely failed in flexure before it could fail in shear. This indicates that the ETS method is very successful.
- The contribution of both EB sheets and NSM rods for shear enhancement was hindered by the transverse steel (stirrups). This however, did not occur in the ETS method, which could be attributed to the fact that the ETS rods were embedded into the core of the concrete beam, which is less cracked than surface and cover regions which act as the substrate for EB and NSM methods.
- ETS method is very effective and appears promising. Further research could be done on effect of spacing, cross section area and different types of FRP rods on the shear capacity of beams.

2.7.4. Shear Retrofitting By Using Fibre Reinforced Concrete Jackets

Jacketing is another method of shear retrofitting, on which research has been conducted.

Fibre-reinforced cement (FRC) composites outperform fibre-reinforced polymers in terms of high temperature and ultraviolet radiation resistance, long-term durability, and fundamental substrate compatibility when poured to act as a jacket as shown in Figure 10 (Gonzalo Ruano, Facundo Isla, Rodrigo Isas Pedraza, Domingo Sfer, Bibiana Luccioni, 2013).

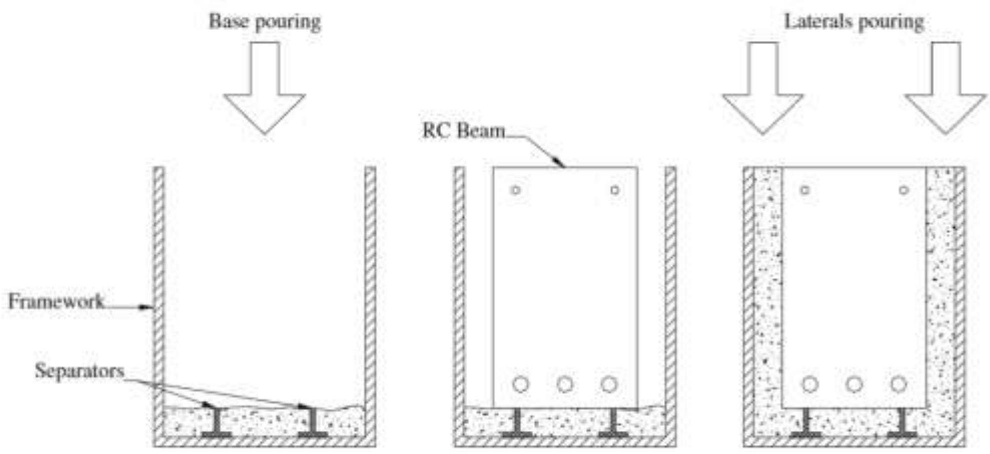


Figure 10: Fibre reinforced concrete jacketing

Experiments on a cracked series of beams with and without shear links, later repaired and strengthened using steel fibre reinforced concrete jacketing, have concluded that the strengthening /repairing technique using a self-compacting concrete matrix with steel fibre reinforcement is feasible to apply in building sites (Gonzalo Ruano, Facundo Isla, Rodrigo Isas Pedraza, Domingo Sfer, Bibiana Luccioni, 2013). Fibre-reinforced concrete with these properties can be poured in thinner jacketing thicknesses. On the other hand, it gives a good surface finish, allowing the plaster layers to be eliminated, somewhat compensating for the structure's extra mass and cost.

The structural qualities of fibre reinforced concrete are improved. Furthermore, the compatibility of the base and retrofitting materials, as well as the longer but thinner cracking pattern, restricts the entry of aggressive agents, enhancing the reinforcement's durability. These effects are critical, given that one of the main purposes of restoration work is to ensure that the structure maintains its integrity and gives a sense of security (Gonzalo Ruano, Facundo Isla, Rodrigo Isas Pedraza, Domingo Sfer, Bibiana Luccioni, 2013). The results of the shear tests on reinforced concrete beams showed a wide range of outcomes. This dispersion can be linked to shear failure's brittleness, and it's similar to what other writers have discovered in similar testing. De-bonding and spalling of the reinforcement were seen in beams strengthened or repaired with plain concrete jacketing. The insertion of fibres to the jacketing prevented de-bonding, ensuring the beams' structural integrity. This characteristic not only increases endurance but is also critical from a structural standpoint because the jacketing no longer contributed if it de-bonds. The transverse section enlargement increased the rigidity of strengthened and repaired beams in general. Statistically, stirrup-strengthened beams with 30 kg/m3 and 60 kg/m3 fibre dosage enhanced their shear strength, whereas beams strengthened with plain concrete jackets had no such improvement. Finally, fibre reinforced concrete jacketing has proven to be an effective solution for the shear strengthening of reinforced concrete stirrup beams (Gonzalo Ruano, Facundo Isla, Rodrigo Isas Pedraza, Domingo Sfer, Bibiana Luccioni, 2013).

The experiment concluded that for beams without stirrups, this retrofitting method is unable to replace the stirrups and does not prevent longitudinal reinforcement buckling. The fibre-reinforced concrete-repaired beams outperformed the original beams in terms of strength. In conclusion, given the conditions they evaluated, fibre reinforced concrete jacketing with fibre concentrations equal to or greater than 30 kg/m3 appears to be an effective shear repairing approach for reinforced concrete beams with stirrups (Gonzalo Ruano, Facundo Isla, Rodrigo Isas Pedraza, Domingo Sfer, Bibiana Luccioni, 2013).

2.8. Summary of the Literature Review

The aim of the literature survey was to identify the research gaps existing in the current research knowledge base regarding shear strength enhancement of curved RC beams using FRP. Only a few research has been carried out on retrofitting of in-plane loaded curved beams and none of those have been done on retrofitting of out of plane loaded curved RC beams for shear failure. Therefore the first part of the survey was about curved beams both in and out of plane loaded and their applications. The second part was dedicated to FRP materials and composites and the application methods of FRP for beams. The third part was regarding flexural enhancement of beams using FRP and how it fails once retrofitted for flexure once retrofitted with FRP. The final part was dedicated to beam failure by shear and various shear enhancing and retrofitting techniques used over time with steel rods, jackets and FRP. It was found out that these retrofitting techniques yielded good results in terms of strength enhancement and capacity restoration in damaged beams.

2.9. Research Gaps

As per the findings of literature review, it was identified that studies have been conducted regarding retrofitting of straight beams for flexure and shear. However, none of the studies have been done regarding the retrofitting of shear failed out of plane loaded curved beams. Therefore it was decided to set the scope of this research to diminish the above mentioned research gap of retrofitting of shear failed, out of plane loaded curved beams.

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3. EXPERIMENTAL PROGRAM

In order to conduct research on the research gap identified, the following experimental program was devised and carried out.

3.1. Overview

The experimental program focuses on retrofitting 2 sets of damaged horizontally curved beams using CFRP laminates. The beams previously tested by Fernando et al (2021) had failed in shear when tested under four point bending.

During the experimental program of Fernando et al (2021) the selected beams exhibited major shear cracks near supports inclined at an angle of approximately 45 degrees as in Figure 11. The beams also exhibited flexural cracks as well in the middle region.

Figure 11: Cracks in a 2m radius curved beam

3.2. Material properties

The material used and their properties are illustrated in following sections.

3.2.1. CFRP fabric

Material properties of the fibre used are illustrated in Table 4.

Table 4: Properties of carbon fibre fabric
(X-Calibur Construction Chemistry, 2017)

Properties of carbon fibre fabric	
Sheet weight	300g/m2
Carbon content	95%
Net effective thickness	0.166mm
Modulus of elasticity	240GPa
Tensile strength	4000MPa
Elongation at break	2%

3.2.2. Epoxy adhesive

Material properties of the epoxy resin adhesive used are shown in Table 5.

Table 5: Properties of epoxy resin adhesive
(X-Calibur Construction Chemistry, 2017)

Properties of adhesive for fabrics	
Viscosity at 20 ^{0}C	7000 cps
Pot Life at 35 ^{0}C	40 mins
Tensile Strength	45 MPa
Flexural Strength	60 MPa
Compressive Modulus	1.67 GPa
Glass Transition Temperature	65 ^{0}C

3.2.3. Crack repair material

The properties of crack repair material are illustrated in Table 6.

Table 6: crack repair material properties
(Cemscreed HM(F) Special Repair Mortar, 2016)

Colour	Cement Grey
W/P ratio (by weight)	0.14 to 0.16
Fresh wet density	1800 kg/m³
Compressive strength (ASTM C109)	150 kg/cm² @ 1 day 250 kg/cm² @ 3 days 350 kg/cm² @ 7 days 500 kg/cm² @ 28 days

3.3. Specimen preparation

The four beams used in the experimental program were given new notations as shown in Table 7.

Table 7: Details of beams selected for retrofitting

Designation (Fernando et al)	New Designation	Section dimensions (mm x mm)	Radius (m)	Support distance (m)
2-2-4-CO	Beam A	200 x 250	4	1.4
2-1-2-CO	Beam B	200 x 250	2	1.4
2-2-2-CO	Beam C	200 x 250	2	1.4
2-1-4-CO	Beam D	200 x 250	4	1.4

A schematic diagram of the two types of beams is shown in Figure 12.

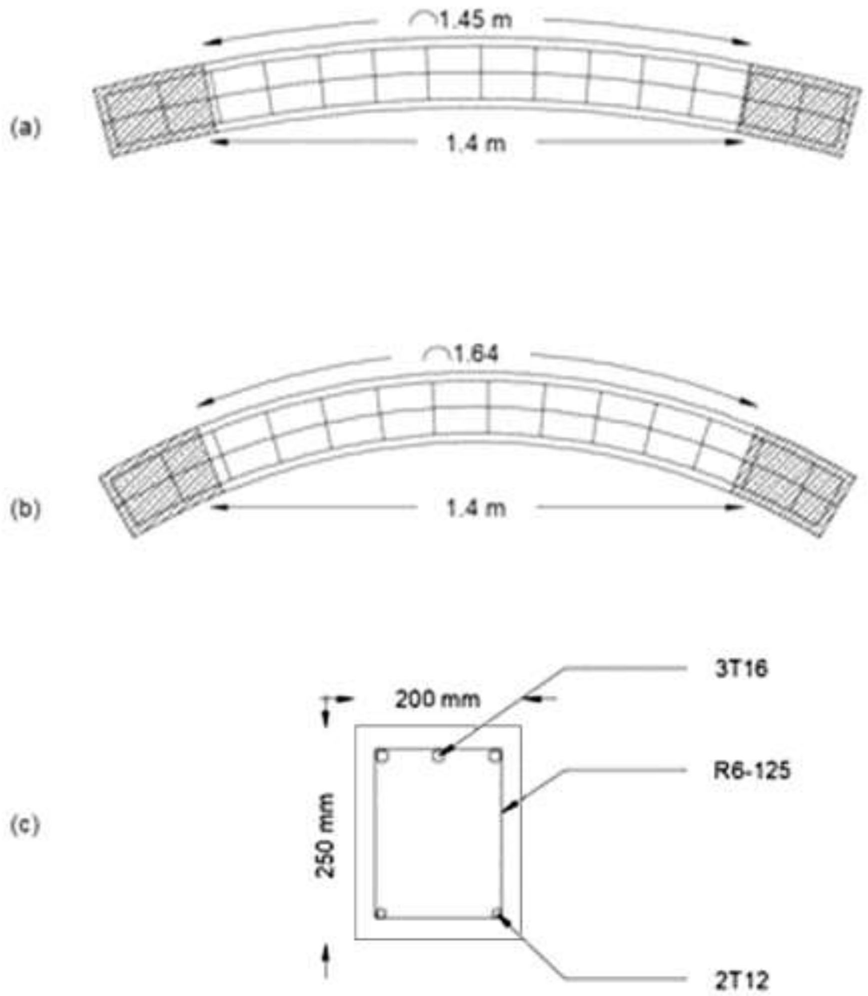

Figure 12: a) 4m radius beam with supports 1.4m apart, b) 2m radius beam with supports 1.4m apart c) Beam section for both types

The beams were tested earlier and therefore exhibited major shear cracks and some flexural cracks too. The cracks had to be repaired to some extent before fibre retrofitting was to be done. Therefore a moderate crack repair method was selected.

3.3.1. Crack repairing

The cracks were repaired using a single component, fibre reinforced, thixotropic repair mortar. The repair mortar was mixed with water at the proportion of 25kg of repair mortar to 4 litres of water. Large cracks were filled to a depth of approximately 25mm using a trowel

Beam after shear crack repairing is shown in Figure 13.

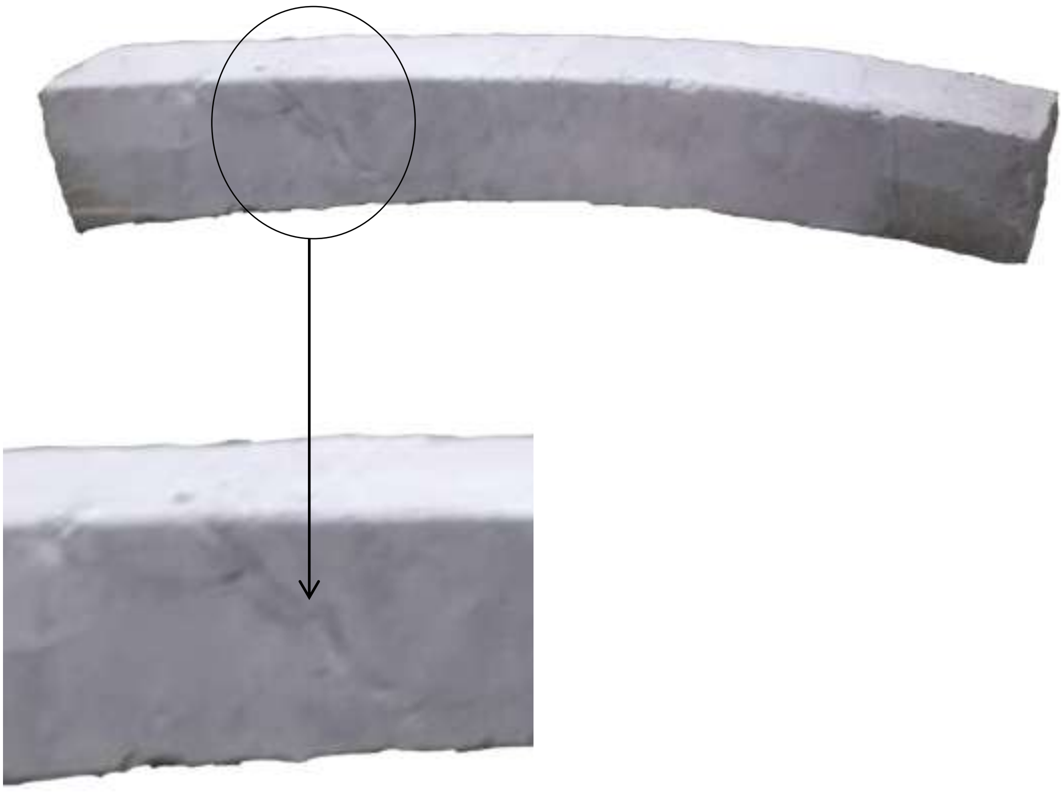

Figure 13: Shear crack repaired in beam

Beam after flexural crack repairing is shown in Figure 14.

Figure 14: Flexural cracks repaired in beam

3.3.2. Retrofitting with fibre

The CFRP configuration was decided based on the design calculations done for the target strength as illustrated in section 4.3 of this thesis.

As per the guidelines (ACI Committee 440, 2017), the outer most concrete layer was ground off using an angle grinder. This resulted in a surface without rough undulations thus preventing stress concentration points. If the fibre strips were to go around a corner the corners too had to be rounded in order to avoid lines of stress concentration. The smoothened surface is shown in Figure 15.

Figure 15: Smoothened substrate after grinding

The two part epoxy was then mixed according to the manufacturer specifications and was applied over the region of fibre retrofitting.

Unidirectional carbon fibre strips of 50mm width and 250mm length were cut off from carbon fibre fabric and were fixed perpendicular to the major shear crack that had propagated during the original load test. The strips were fixed in their correct fibre orientation such that the fibres spanned perpendicular to the crack. The strips were placed in their correct locations with a clear spacing of 15mm-25mm and the strips were spaced out to cover the entirety of the shear crack. 4 shear strips on one side per face were fixed typically with a total of 16 fibre strips per beam. The number of strips was arrived at by considering dimension of beams and to have an economical and practical arrangement.

After the strips were placed at their locations a second coating of the same two part epoxy was applied until the fibre was saturated with epoxy. The epoxy was then left to cure for over 7 days. A beam retrofitted for shear is shown in Figure 16.

Figure 16: Shear retrofitted beam

3.4. Test Setup

Since original loading conditions had to be replicated in order to compare load carrying capacities after replicating, the same support spacing was used.

The beams were retrofit for shear using inclined CFRP strips and were tested for load carrying capacity using 4 point bending test with the load points situated at one third lengths from clear distance of supports. 4 point load test was selected since the original load carrying capacities of the beams too were tested for 4 point load test. The loading conditions in actual scenarios are more likely to be uniformly loaded. However due to the difficulty in simulating this loading condition a point load test was selected. 3 point load test induces higher bending moments in beam as opposed to 4 point load test. Since it was shear that needs to be tested for, 4 point bending test was deemed to be a more suitable load test for this matter.

The behaviour and load carrying capacity of shear retrofitted beams were then compared with the original behaviour of the beams. Also the mode of failure was observed and possible modifications to achieve higher load carrying capacities were identified.

The testing setup used is illustrated in following sections. The testing apparatus used is shown in Figure 17.

Figure 17: Testing apparatus

The beam is set to rest on two I beams on both side and the end rotation and twisting is prevented by a special end support clamp. The beam is then loaded by the reaction generated by jacking against the rig.

3.4.1. End conditions of beams

Most of the practically occurring curved beams have fixed ends and not simply supported ends. Also if torsional effects are to play an impact due to the curvature of the beam, the beam ends have to be fixed. A clamp arrangement was thus used at end to produce a fixed end condition. The clamp used is shown in Figure 18.

The beam is placed on the plate consisting of two angles on either side to prevent twisting of beam. If a gap occurs between angle plates and beam side, an iron plate was wedged to make it tight and to prevent any twisting while loading. Thereafter an angle iron was placed at top and bolted with the plate at bottom to prevent the corner of beam from lifting. By this, the rotation at beam end due to bending could also be prevented, thereby creating an effective 'fixed end' condition.

Figure 18: Clamp and support system

How the beam end is restrained against twisting and bending rotation once placed is shwon in Figure 19.

Figure 19: End clamp to prevent rotation and twisting

3.4.2. Loading conditions

The most commonly occurring loading condition in real life structures such as curved horizontal beams in buildings and viaducts is the uniformly loaded condition. However, this loading condition is difficult to replicate and a 4 point loading setup was used by point loading the beam at $1/3^{rd}$ lengths of the clear span between supports. Rollers were placed at these locations and the loads were made to transfer onto the beam at these points by jacking against the rig using a manually operated hydraulic jack.

The load was then measured using a loading cell. The complete setup is illustrated in Figure 20.

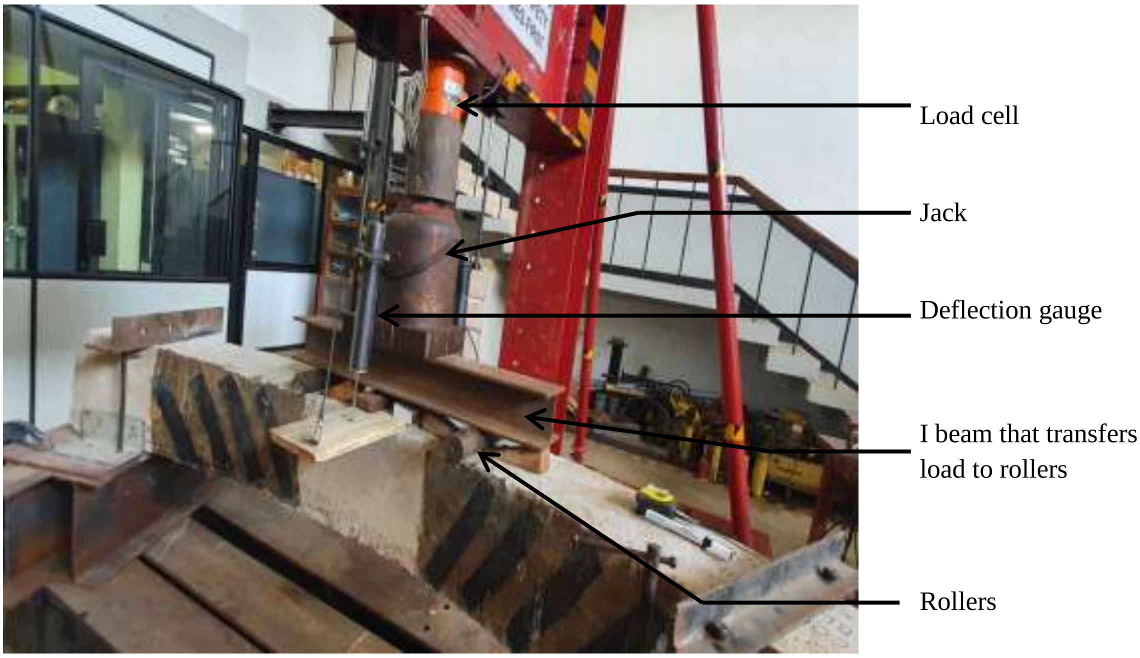

Figure 20: Load points and loading method

3.4.3. Measuring deflection and load

Load was measured using a loading cell as in Figure 20.

The deflection of the beam required is the deflection at centreline of beam at mid span. However due to interference with the testing apparatus, the deflection gauge cannot be fixed directly at the centre. Therefore two deflections on either side at equal offsets from centreline were taken and the average was assumed to be the centre deflection of beam. The reason to take two deflections is that the downward deflection of the outer and inner sides of beam consists of downward translational displacement as well as the rotational displacement. This rotational component is in the same direction as downward translational deflection for the outer side of curved beam, and in the opposite direction for the inner side of curved beam. Hence by averaging the two readings the rotational component can be cancelled out and the translational deflection can be obtained.

3.4.4. Load capacity

Once the beam loading started, the increase of load was observed along with visual observation as shown in Figure 21. The yield point was identified as the point where load started decreasing after reaching a certain value even though the deflection kept on increasing with the jacking.

Figure 21: Beam testing

This maximum load was taken as the load carrying capacity of beam and was compared with the original values. The cracks and any faults developed at this stage too were carefully inspected.

4. THEORETICAL ANALYSIS

The first part of theoretical calculation consists of the determination of forces of the horizontally curved beam under out of plane four point bending load. The second part consists of the determination of flexural and shear capacities of the beam. This capacity calculation aids in designing the experimental program and acts as verification for the experimental results.

4.1. General response expressions of a horizontally curved beam under out-of-plane loads

(Pilkey, 2005) summarizes the expressions for various loading scenarios which will be used in this chapter to calculate the general responses. The notations used are as shown in Figure 22.

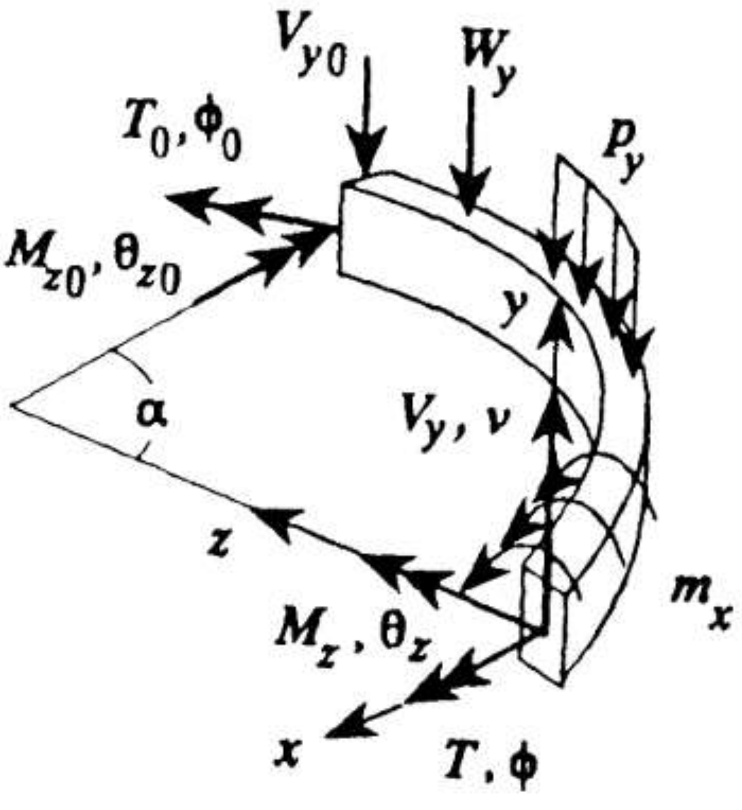

Figure 22: Horizontally curved beam under out-of-plane loads (Pilkey, 2005)

Below notations are valid for all the expressions in this section.

E	=	Modulus of elasticity
G	=	Shear modulus of elasticity
R	=	Radius of arch
J	=	Torsional constant
I_z	=	Moment of inertia about the z-axis
λ_1	=	$\frac{1}{GJ} + \frac{1}{EI_z}$
λ_2	=	$\frac{1}{2}(\sin\alpha - \alpha\cos\alpha)$
λ_3	=	$\frac{1}{2}(2 - 2\cos\alpha - \alpha\sin\alpha)$
λ_4	=	$\frac{1}{2}(2\alpha + \alpha\cos\alpha - 3\sin\alpha)$
λ_5	=	$\frac{1}{2}(\alpha\cos\alpha + \sin\alpha)$
$< \alpha - \alpha_1 >^0$	=	$\begin{cases} 0 & \alpha < \alpha_1 \\ 1 & \alpha \geq \alpha_1 \end{cases}$
$< \alpha - \alpha_1 >$	=	$\begin{cases} 0 & \alpha < \alpha_1 \\ (\alpha - \alpha_1) & \alpha \geq \alpha_1 \end{cases}$

There are six general response expressions for each three deformation parameters and the internal forces depend on the initial loading and boundary conditions of the system. The expressions can only be used for a single loading scenario. It cannot be used for two loads on the same beam. Therefore, to obtain the results for a four-point bending scenario, the general responses for each load should be calculated using the expressions below and the results should be superimposed to obtain the final result. The expressions are shown below.

Angle of twist:

$$\emptyset = \emptyset_0 \cos\alpha + \theta_{z0}\sin\alpha - V_{y0}\lambda_1 R^2 + \frac{1}{2}M_{z0}\lambda_1 R\alpha\sin\alpha + T_0\left(\frac{R}{GJ}\lambda_5 - \frac{R}{EI_z}\lambda_2\right) + F_{\emptyset} \quad (1)$$

Deflection: $$\upsilon = -\phi_0 R(\cos\alpha - 1) + \upsilon_0 + \Theta_{z0} R\sin\alpha + V_{y0}\left(\frac{R^3}{GJ}\lambda_4 - \frac{R^3}{EI_z}\lambda_2\right) + M_{z0}\left(\frac{R^2\alpha}{2EI_z}\sin\alpha - \frac{R^2}{GJ}\lambda_3\right) - T_0\lambda_1 R^2\lambda_2 + F_\upsilon \quad 2$$

Slope: $$\Theta_z = -\phi_0 \sin\alpha + \Theta_{z0}\cos\alpha + V_{z0}\left(\frac{R^2}{GJ}\lambda_3 - \frac{R^2\alpha}{2EI_z}\sin\alpha\right) + M_{z0}\left(\frac{R}{EI_z}\lambda_5 - \frac{R}{GJ}\lambda_2\right) - \frac{1}{2}T_0\lambda_1 R\alpha\sin\alpha + F_{\Theta z} \quad 3$$

Shear force: $$V_y = V_{y0} + F_{vy} \quad 4$$

Bending moment: $$M_z = -V_{y0}R\sin\alpha + M_{z0}\cos\alpha - T_0\sin\alpha + F_{Mz} \quad 5$$

Torque: $$T = V_{y0}(\cos\alpha - 1) + M_{z0}\sin\alpha + T_0\cos\alpha + F_T \quad 6$$

Loading functions $F_\emptyset, F_\upsilon, F_{\Theta z}, F_{vy}, F_{Mz}$ and F_T are defined for an applied concentrated load and concentrated torque scenarios in the below sections.

Loading functions for a concentrated vertical load as shown in Figure 23 are as below.

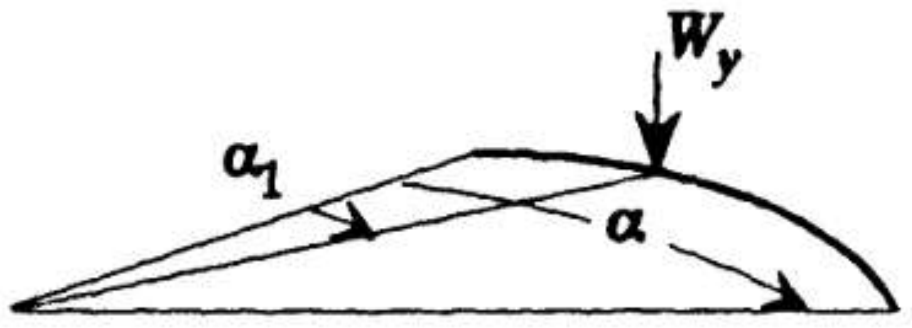

Figure 23: Concentrated vertical load on a horizontally curved beam (Pilkey, 2005)

$$F_{\phi} = \frac{W_y \lambda_1 R^2}{2}[\sin < \alpha - \alpha_1 > - < \alpha - \alpha_1 > \cos(\alpha - \alpha_1)] \quad 7$$

$$F_v = -\frac{W_y R^3}{2}\left\{\frac{1}{GJ}[2 < \alpha - \alpha_1 > + < \alpha - \alpha_1 > \cos(\alpha - \alpha_1) - 3\sin < \alpha - \alpha_1 >] - \frac{1}{EI_z}[\sin < \alpha - \alpha_1 > - < \alpha - \alpha_1 > \cos(\alpha - \alpha_1)]\right\} \quad 8$$

$$F_{\theta z} = \frac{W_y R^2}{2}\left\{\frac{1}{EI_z}[< \alpha - \alpha_1 > \sin(\alpha - \alpha_1)] - \frac{1}{GJ}[2 < \alpha - \alpha_1 >^0 - 2 < \alpha - \alpha_1 >^0 \cos(\alpha - \alpha_1) - < \alpha - \alpha_1 > \sin(\alpha - \alpha_1)]\right\} \quad 9$$

$$F_{vy} = -W_y \quad 10$$

$$F_{Mz} = W_y R \sin < \alpha - \alpha_1 > \quad 11$$

$$F_T = W_y R(< \alpha - \alpha_1 >^0 \cos(\alpha - \alpha_1) - < \alpha - \alpha_1 >^0) \quad 12$$

4.2. Determination of flexural and shear capacities of the beam.

In this section, the flexural and shear load carrying capacities are presented based on the design codes and guidelines which are currently used in the industry.

4.2.1. Flexural capacities of RC beams without CFRP external reinforcement

According to the Euro standard (BS EN 1992-1-1, 2004), the ultimate moment of resistance (M_1) is given by Equation 32.

$$M_1 = 0.25092 f_{ck} b d^2 \qquad 13$$

The following expressions are based on expressions from the Euro standard (BS EN 1992-1-1, 2004) which were modified by removing all the safety factors considered in the code considering the nature of experimental results and the reliability of material properties and quality control. The relationship between the applied moment and the tensile reinforcement area is as follows:

$$M = A_s f_{yk} z \qquad 14$$

Where,

$$z = \frac{d}{2}\left[1 + \sqrt{1 - \frac{2.353M}{f_{ck} b d^2}}\right] \qquad 15$$

Substituting z to the expression of M above and subsequently simplifying Equation 35 for the ultimate moment that tensile reinforcement can resist, can be obtained.

$$M_2 = A_s f_{yk} d - 0.58825 \frac{A_s^2 f_{yk}^2}{b f_{ck}} \qquad 16$$

Then, the ultimate flexural capacity is the minimum of M_1 and M_2.

4.2.2. Shear capacities of RC beams without CFRP external reinforcement

4.2.2.1. Shear capacity with shear links

With the provision of shear links, the failure can occur by either compressive failure of an imagined diagonal concrete strut or tensile failure of the member. For tensile failure, the shear force is given by (BS EN 1992-1-1, 2004),

$$V_{Rd,s} = \frac{A_s}{s} z f_y \cot\theta \quad (17)$$

Where,

The $\cot\theta$ term can vary between 1 and 2.5 depending on the magnitude of the shear force. Hence, for the maximum resistance of shear links, it can be taken as 2.5.

A_s is the area of the tensile reinforcement

s is the spacing of stirrups

$z = d - d'$

f_y is the tensile strength of stirrups

For compressive failure, the shear force is given by,

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{ck} / (\cot\theta + \tan\theta) \quad (18)$$

Where,

$\alpha_{cw} = 1$

$v_1 = 0.6\left[1 - \frac{f_{ck}}{250}\right]$

A summary of the flexural and shear capacities calculated as above are presented in Table 8.

Table 8: Moment and Shear Capacities of Beams

Location	Moment Capacity	Shear Capacity
Support (Tension r/f - 3T16)	52.9 kNm	49.5 kN
Span (Tension r/f- 2T12)	22.5 kNm	-

The loads that need to be applied in order to reach the flexural and shear capacities are as given in Table 9.

Table 9: Loads on 4 Point Bending Test to reach flexural and shear capacities as per theoretical analysis

Beam Type	Load on 4 point bending test to reach Moment Capacity		Load on 4 point bending test to reach shear capacity
	Span	Support	
2m radius of curvature beam	176kN	278kN	99kN
4m radius of curvature beams	207kN	325kN	99kN

From the above theoretical analysis it can be seen that

1. Shear failure occurs earlier than flexural failure, which is in agreement with the requirement of the original beam
2. Flexural failure of span due to sagging moment occurs earlier than flexural failure at support due to hogging moment
3. Flexural failure load of 2m radius curvature beams are lower than those of 4m radius of curvature beams

4.3. Shear capacity from FRP strips

ACI guide (ACI Committee 440, 2017) presents an expression for the shear capacity enhancement due to external FRP reinforcement is given in Equation 19. The notations are used as shown in Figure 24.

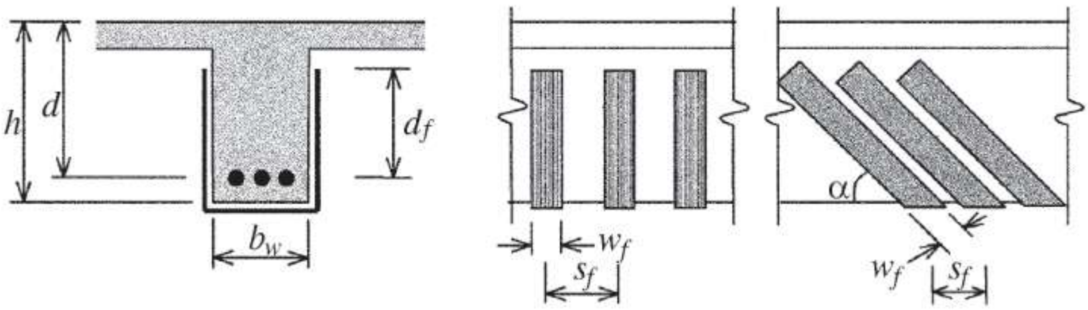

Figure 24: Notations for CFRP shear enhancement calculation (ACI Committee 440, 2017)

$$V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_f}{s_f} \qquad 19$$

For a rectangular section,

$$A_{fv} = 2 n t_f w_f \qquad 20$$

Where,

t_f is the thickness of FRP sheets

By Hook's law

$$f_{fe} = E_f \varepsilon_{fe} \qquad 21$$

For completely wrapped members

$$\varepsilon_{fe} = 0.004 \ \leq 0.75 \varepsilon_{fu} \qquad 22$$

Where,

ε_{fu} is the rupture strain of the FRP and E_f is Young's modulus of FRP.

For U-wraps and side bonding

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \ \leq 0.004 \qquad 23$$

κ_v is a bond-reduction reduction factor which is a function of the active bond length, the strength of concrete and used wrapping scheme and is determined by Equations 24, 25 and 26.

$$\kappa_v = \frac{k_1 k_2 L_e}{11900\varepsilon_{fu}} \leq 0.75 \qquad 24$$

$$k_1 = \left(\frac{f_c'}{27}\right)^{2/3} \qquad 25$$

$$k_2 = \begin{cases} \dfrac{d_{fv} - L_e}{d_{fv}}, & for\ U - wraps \\ \dfrac{d_{fv} - 2L_e}{d_{fv}}, & for\ side\ bonding \end{cases} \qquad 26$$

By providing 50mm wide 250mm long inclined CFRP strips at 55mm spacing it can be shown that CFRP strips on both faces of one end of the beam can resist 49kN, which provides a load carrying capacity of 98kN under four point bending test.

Therefore the CFRP configuration is taken as 4 inclined strips per face of aforementioned dimensions, across shear crack.

5. OBSERVATIONS AND RESULTS

5.1. Comparison with original load carrying capacity

The load capacity results of beams tested after retrofitting are presented in Table 10, along with their original load capacity values.

Table 10 : Comparison with original load carrying capacity

Beam	Radius	Original Load Capacity (kN)	Load capacity after Retrofitting (kN)	As a percentage of original Load Capacity
A	4	95	116	122%
B	2	93	109	117%
C	2	92	79	85%
D	4	95	86	90%

It can be seen that Beams A & B achieved more than its original load carrying capacity after retrofitting while 85% and 90% of its original load carrying capacity was achieved for beams C & D. The reduction in carrying capacities of Beam C & D could be attributed to their severely damaged state before retrofitting.

5.2. Comparison of Theoretical, Experimental and retrofitted beam capacities

The theoretical load carrying capacities calculated and the experimental failure loads, along with the failure loads after retrofitting and re-testing are presented in Table 11. The target strength to be gained by CFRP retrofitting was taken as the maximum load carrying capacity exhibited by each beam radius during original testing.

Table 11: Comparison of Theoretical, Experimental and retrofitted beam capacities

Theoretical Calculation				**Actual Failure Load at Original Testing**		**Level of Damage after Original Testing**	**Target Strength for Shear Retrofitting**	**Actual Failure load after Retrofitting and Re-testing**
Beam Type	Load on 4 point bending test to reach Moment Capacity		Load on 4 point bending test to reach shear capacity					
	Span	Support						
2m radius of curvature beam	176kN	278kN	99kN	Beam B	93kN (shear failure)	**Moderately damaged**. Shear crack width 2mm	93kN	109kN
				Beam C	92kN (shear failure)	**Severely damaged**. Shear crack width approximately 5mm	93kN	79kN
4m radius of curvature beams	207kN	325kN	99kN	Beam A	95kN (shear failure)	**Moderately damaged**. Shear crack width 2mm	95kN	116kN
				Beam D	95kN (shear failure)	**Severely damaged**. Shear crack width approximately 5mm	95kN	86kN

5.2. Comparison with original load deflection curves

The load vs. deflection curves of beam A at original and retrofitted states are shown in Figure 25.

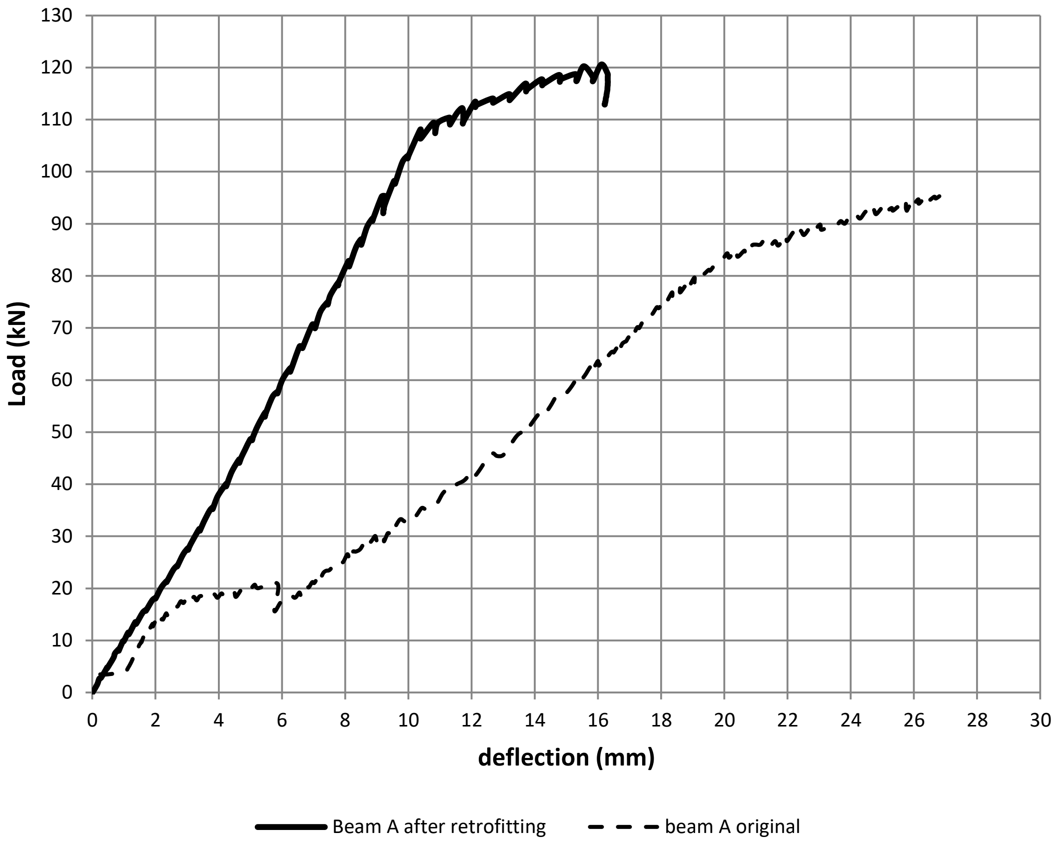

Figure 25: Load Vs. Deflection curves of beam A

It can be seen that the original load vs. deflection behaviour (Fernando et al, 2021) displays an odd behaviour with the curve becoming flatter in the initial loading stage until 6mm of deflection occurred and then suddenly picking up its stiffness again until yielding occurs at around 18mm of deflection. This odd behaviour could be attributed to one or more of the below

1. Erroneously measured deflections with the deflection gauges not being upright

2. Beam not properly resting on support and it falling into place as loading progresses.

The load vs. deflection curve after retrofitting with CFRP, however, displays an agreeable and predictable behaviour with the curve running in a steep, straight and linear elastic manner until the first de-bonding occurred, where after it has become flatter.

The load vs. deflection curves of beam B at original and retrofitted states are shown in Figure 26.

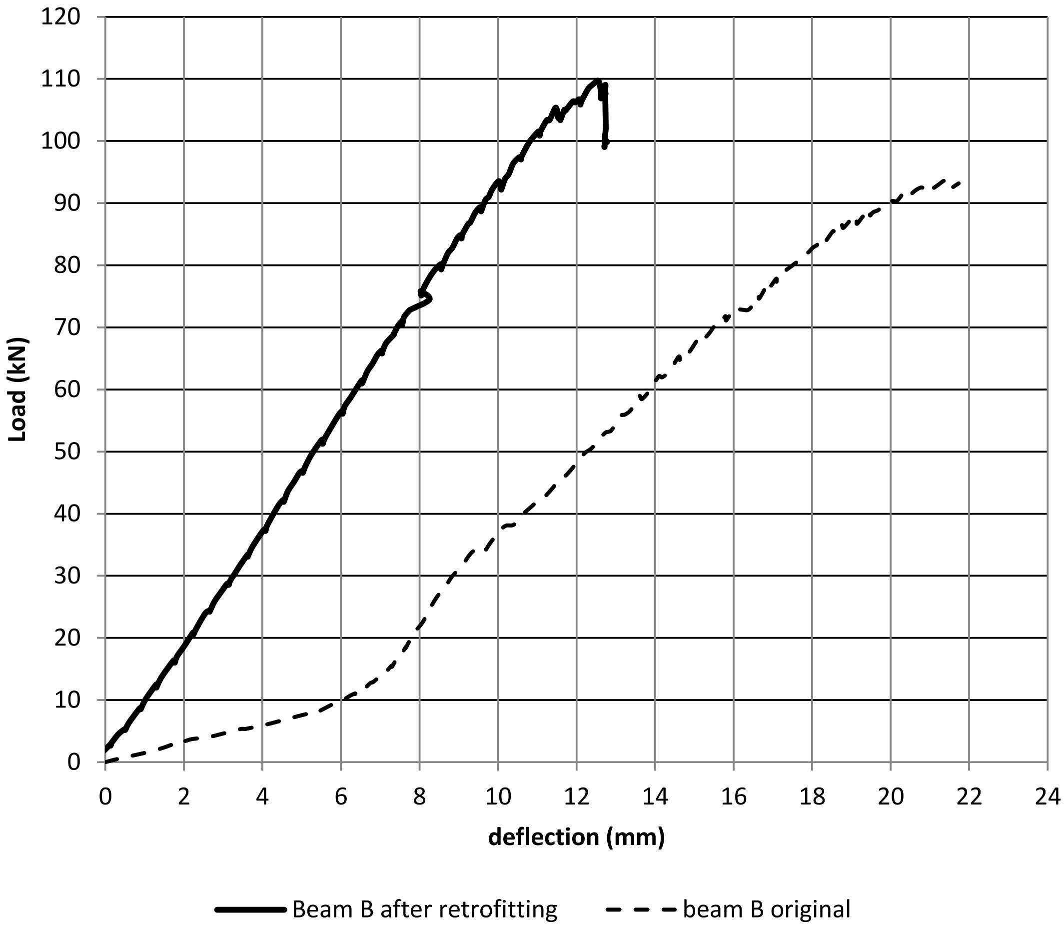

Figure 26: Load Vs. Deflection curves of beam B

The aforementioned anomaly of initial flattening of the curve and its' subsequent picking up of stiffness could be seen in beam B too. This could be attributed to the same reasons mentioned in Beam A.

The load vs. deflection curve after retrofitting shows predictable and agreeable behaviour. The drop in carrying capacity after de-bonding appears to be more sudden and drastic than in beam A. This could be attributed to the fact that several CFRP strips de-bonding at the same time without any stagger.

The load vs. deflection curves of beam C at original and retrofitted states are shown in Figure 27.

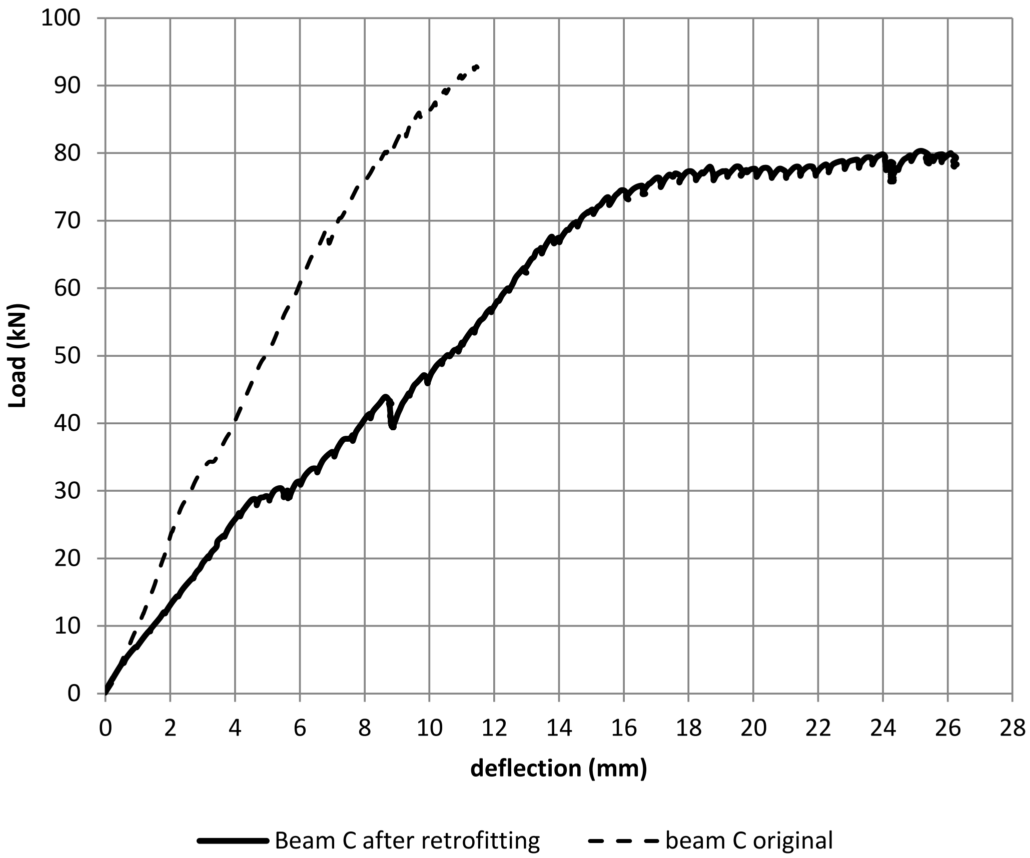

Figure 27: Load Vs. Deflection curves of beam C

The load vs. deflection curve after retrofitting is flatter than its original one. Also the retrofitted beam reaches load capacity at a lower load than the original beam did.

The load vs. deflection curves of beam D at original and retrofitted states are shown in Figure 28.

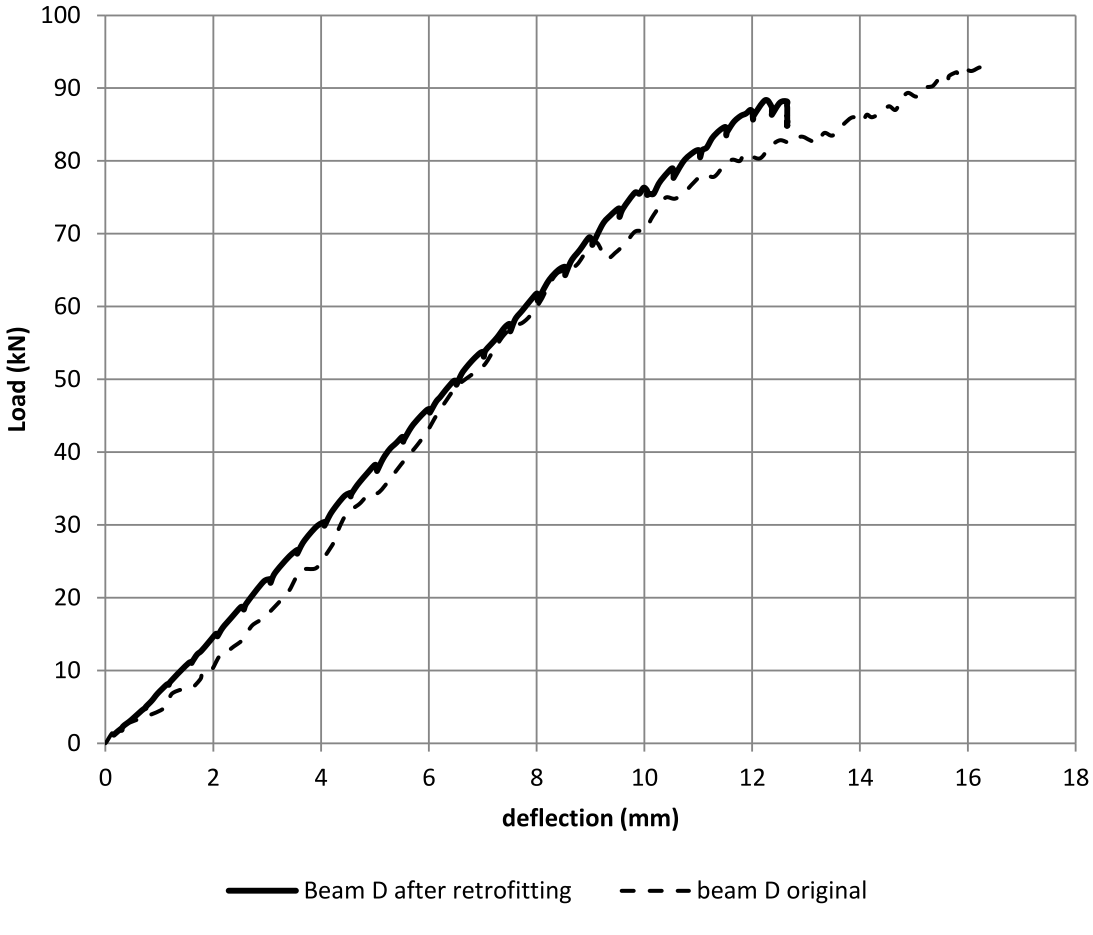

Figure 28: Load Vs. Deflection curves of beam D

The gradient of load vs. deflection curve of retrofitted beam is similar to that of the original beam even though it yields at a lower load than original beam.

5.3. Types of Beam Failure Observed

The major failure mode of all four beams was the de-bonding of fibre strips. Flexural cracks too could be observed over the cracks where a moderate crack repair was done. Furthermore spalling of concrete (separation of concrete cover from reinforcement) in the fibre retrofitted region could also be observed in Beam D.

5.3.1. Failure of Beam A

As loading progressed flexural cracks were seen to re-appear at the locations where a moderate crack repair was done as shown in Figure 29.

Figure 29: Flexural cracks appearing at the locations where a moderate crack repair was done – Beam A

Stretching of CFRP and ultimately de-bonding of could be observed as shown in Figure 30.

Figure 30: De-bonding of shear strips – Beam A

5.3.2. Failure of Beam B

The initial shear crack, on which crack repair was done and CFRP strips fixed across, re-appeared as shown in Figure 31 and rapidly progressed after the de-bonding of several CFRP strips.

Figure 31: Developing shear cracks – Beam B

Several CFRP strips could be observed to have de-bonded as shown in Figure 32.

Figure 32: De-bonding of shear strips – Beam B

5.3.3. Failure of Beam C

A torsional crack propagated at support as illustrated in Figure 33.

Figure 33: Torsional crack at beam end – Beam C

CFRP strip de-bonding could be observed as shown in Figure 34.

Figure 34: De-bonding of shear strips – Beam C

5.3.4. Failure of Beam D

The original shear crack re-appeared as loading progressed as shown in Figure 35.

Figure 35: Shear crack propagated –Beam D

Concrete cover separation could be observed on side of the beam as shown in Figure 36 in addition to the CFRP de-bonding.

Figure 36: De-bonding of fibre strip and spalling of concrete – Beam

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

From observations of behaviour and failure of shear retrofitted beams under loading, the following could be deduced and summarized regarding the effectiveness of retrofitting of curved beams failed in shear using CFRP.

1. Theoretical calculations and experimental results are in good agreement with one another.
   - As per theoretical calculations bending failure could not occur before shear failure. In the original experiment, the beams failed by shear as expected.
   - The experimental shear failure load under four point bending test came very close to the theoretical failure load calculated for both beam types (with a maximum of 6% margin)
   - The failure loads after retrofitting exceeded the target load carrying capacity for beams A & B and fell below the target load carrying capacity for beams C & D.

2. In all beams, failure mode of CFRP was a de-bonding failure.

3. Beam A (4m radius) and Beam B (2m radius) showed load capacities of 122% and 117% respectively from their original load capacities.

4. The Load vs. Deflection curves appeared steeper than their original curves for Beam A and Beam B, essentially meaning that the retrofitted beams have had a stiffening effect caused by the inclined CFRP strips. This could be attributed to the ‘truss action’ caused by the strips acting as an inclined tension member connecting ‘top chord of beam’ in compression with ‘bottom chord of beam’ in tension.

5. For Beam C (2m radius) and Beam D (4m radius) retrofitting was not as effective as for the other two beams, gaining 85% and 90% of their original load capacities. This could be attributed to the fact that these two beams were severely cracked prior to crack repair and retrofitting. However even after such severe damage, regaining strength of this magnitude gives an indication about the success of this method.

6. Bending stiffness of Beam C appears to have reduced compared to its original state, which could be attributed to its severely cracked state originally. Bending stiffness of Beam D remained similar to the state during its original loading, until delamination of strips occurred.

7. Effects of torsion could be seen more on beams of 2m radius since the development of torsional cracks near supports could be observed, whereas beams of 4m curvature did not exhibit such cracks.

6.2. Recommendations for Future Research

It could be observed from the failure of beam series that the mode of failure was by shear strip de-bonding. Since the beams had previously failed in shear, it could be assumed that shear strips were the elements which predominantly held the beam across the shear crack and not the aggregate interlock or shear links (which had already yielded) when the crack was trying to separate due to the loading. Therefore CFRP strips were being stretched as loading continued until the sudden bond failure between CFRP strips and concrete substrate, after which the beam went into a plastic state where deflection increased without any increase in load as jacking continued.

Effects of torsion could be seen on 2m radius beams due to its higher curvature since some torsional cracks could also be observed. Since torsion too adds shear stresses in beam, there is a component of torsion too for the tension in the diagonal direction, which CFRP strips had to resist.

Since the mode of failure was by de-bonding of CFRP strips and not by rupture, an anchorage mechanism would have further increased the load carrying capacity of the beams.

Proposed anchorage methods

1. Individually anchoring each CFRP strip to the substrate using nails
2. Anchoring the group of CFRP strips by connecting with transverse ‘header strips’ as shown in Figure 37.

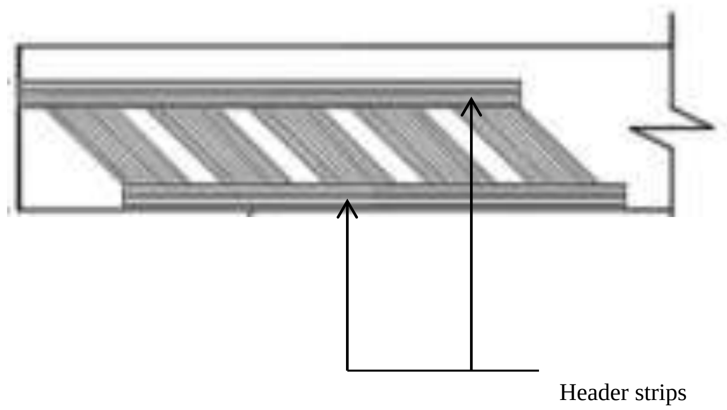

Figure 37: Anchoring using header strips

The header strips would have distributed the stresses across the two or more fibre strips, thereby increasing the load at which CFRP de-bonding occurs. The effectiveness of retrofitting shear failed curved RC beams using CFRP strips anchored utilizing the above anchoring methods remains to be researched upon.

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