

RETROFITTING OF RC BEAMS USING ULTRA HIGH PERFORMANCE FIBER REINFORCED CONCRETE

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Submitted by

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DECLARATION

I hereby declare that this dissertation titled **“RETROFITTING OF RC BEAMS USING ULTRA HIGH PERFORMANCE FIBER REINFORCED CONCRETE”** is an authentic record of the work carried out as per the requirement for the award of degree of **Masters of Engineering in Structural Engineering** in the Civil Engineering Department of **Thapar University, Patiala** under the guidance of **Dr. Prem Pal Bansal, Associate Professor, Department of Civil Engineering, Thapar University, Patiala** during July 2015 to July 2016. The matter embodied in this report has not been submitted in part or full to any other university or institute for the award of any degree.

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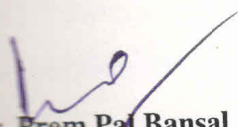
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CERTIFICATE

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(Varun Garg)

ABSTRACT

Reinforced concrete structural components are found to exhibit distress, even before their service period is over due to several causes. Such unserviceable structures require immediate attention, enquiry into the cause of distress and suitable remedial measures, so as to bring the structures back to their functional use again.

This strengthening and enhancement of the performance of such deficient structural elements in a structure or a structure as a whole is referred to as retrofitting. The all important issue to be addressed in retrofitting is life safety .What can be done to prevent collapse of the structure and prevent injury or death to occupants? Some retrofit requirements may try to address only the issue of life safety, while acknowledging that some structural damage may occur.

Structures that are deficient in shear can be strengthened or repaired by using various methods, e.g., external prestressing, shotcreting, polymer impregnation, steel plate bonding, fiber reinforced polymers (FRPs) and other cementitious materials including ultra high performance fiber reinforced concrete (UHPFRC).

To overcome all the above problems faced in different retrofitting techniques, a new cementitious material with ultra high performance characteristics like very high compressive strength (above 110 MPa), high tensile strength due to addition of fibers, high flexural strength, very less permeable and high workability i.e. UHPFRC came into existence. UHPFRC with its remarkable characteristics as a repair material for concrete structures in harsh environmental conditions and high loads works excellent.

In the present thesis shear deficient RC beams initially stressed to a prefixed percentage i.e. 60% of the failure load are retrofitted using UHPFRC. Retrofitting is done using three different techniques i.e. Side face retrofitting, U-Type retrofitting and External 25 mm thick jacketing.

From the study it is seen that the ultimate load carrying capacity of RC beams retrofitted with UHPFRC is significantly increased with Side face and U-type retrofitting. Moreover, this technique also increased the ductility and energy absorption of RC beams which resulted in the usefulness of UHPFRC as a retrofitted material. A comprehensive literature review is carried out for better understanding of UHPFRC as a retrofitting

material compiled from many references representing research, development, and deployment efforts around the world, this thesis report provides a framework for gaining a deeper understanding of UHPFRC as a retrofitting material.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

Concrete construction is generally expected to give trouble free service throughout its intended design life. However, these expectations are not realized in many constructions because of structural deficiency, material deterioration, unanticipated over loadings or physical damage. Premature material deterioration can arise from a number of causes, the most common being when the construction specifications are violated or when the facility is exposed to harsher service environment than those expected during the planning and design stages. Physical damage can also arise from fire, explosion -as well as restraints, both internal and external, against structural movement. Except in extreme cases, most of the structures require restoration to meet its functional requirements by appropriate repair techniques.

The strengthening and enhancement of the performance of deficient structural elements in a structure or a structure as a whole is referred to as retrofitting. Repair refers to partial improvement of the degraded strength of a building after an earthquake. In effect, it is only a cosmetic enhancement. Rehabilitation is a functional improvement, wherein the aim is to achieve the original strength of a building after an earthquake. Retrofitting means structural strengthening of a building to a Predefined performance level, whether or not an earthquake has occurred. The seismic performance of a retrofitted building is aimed higher than that of the original building.

Flexural failure and shear failure are the two primary modes of failure in reinforced concrete (RC) beams. Flexural failure of a beam is ductile in nature, i.e., it occurs gradually with large deflections and cracking, which provide a warning of incipient failure. Conversely, shear failure is brittle in nature and does not allow substantial redistribution of loads; thus, shear failure occurs without any prior warning and is often catastrophic. Poorly designed beams may fail in shear before reaching the flexural strengths. Hence, RC beams must have sufficient shear strength, higher than flexural strength, in order to ensure a ductile failure mode.

Shear failure of RC structures may be due to many factors, e.g., insufficient shear reinforcement, reduction of steel area due to corrosion and spalling of concrete caused by aggressive environmental conditions, increased service load due to change in usage of the structure, and any detailing, design, and/or construction error. Thus, strengthening and rehabilitation of RC structures may be needed to increase the ultimate load carrying capacity of shear-deficient beams.

Structures that are deficient in shear can be strengthened or repaired by using various methods, e.g., external prestressing, shotcreting, polymer impregnation, steel plate bonding, fiber reinforced polymers (FRPs) and other cementitious materials including ultra high performance fiber reinforced concrete (UHPFRC). Among these retrofit solutions, the use of externally bonded FRPs has been more frequently used as FRP shear retrofit of RC structures presents numerous advantages compared to other more traditional techniques, e.g., light weight and ease of installation, high strength to weight ratio high stiffness to weight ratio, and corrosion resistance. But Retrofitting using FRPs is also vulnerable to undesirable brittle failures due to a large mismatch in the tensile strength and stiffness with that of the concrete.

So to overcome all the above problems faced in different retrofitting techniques, a new cementitious material with ultra high performance characteristics like very high compressive strength (above 110 MPa), high tensile strength due to addition of fibers, high flexural strength, very less permeable and high workability i.e. UHPFRC came into existence. UHPFRC with its remarkable characteristics as a repair material for concrete structures in harsh environmental conditions and high loads works excellent.

1.2 UHPFRC

UHPFRC belong to the group of High Performance Fiber Reinforced Cement Composites (HPFRCC), where HPFRCC defined as the kind of Fiber Reinforced Concretes (FRC) that exhibit strain-hardening under uniaxial tension force. In addition, UHPFRC is characterized by a dense matrix (removal of coarse aggregates from the mix and the maximum particle size in the mix is 2 mm), high mechanical properties like compressive strength, tensile strength, flexural strength and consequently a very low permeability when compared to HPFRCC and normal strength concretes.

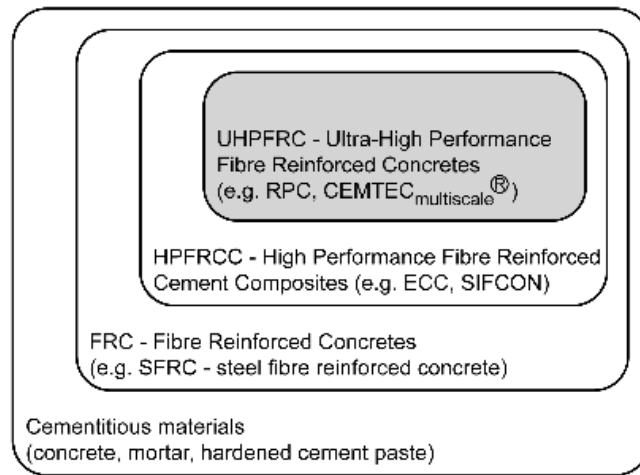


Fig. 1.1 Definition of UHPFRC (Yen L.V. et al., 2012)

1.2.1 Historical Background

UHPFRC is an advanced reinforced cementitious material. “Ultra-high performance” refers principally to improved mechanical strengths, fractural toughness, and durability. Over the last few years, considerable efforts to improve the behavior of cementitious materials by incorporating fibers have led to the emergence of Ultra-High Performance Fiber Reinforced Concretes (UHPFRC).

Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) is the outcome of a demand that began in the 1930s to seek for approaches to increase the mechanical properties of concrete, in particular its compressive strength. At the beginning of the 1950s, Graf produced a type of concrete with strength of 70 MPa (**Bache, 1991**). This did not change until 1966, when a special production method allowed the production of concrete with the strength of 140 MPa.

Yudenfreund M. et al (1972) investigated high strength pastes with low w/c-ratios ($w/c = 0.2$ to 0.3) whose main characteristic was the low porosity leading to high compressive strengths (up to 200 MPa) and to low dimensional changes. Strength enhancement by hot pressing techniques was first applied by **Roy D.M. et al (1972)** and resulted in very high strength cement pastes with compressive strengths up to 680 MPa.

With the development of superplasticizers and pozzolanic admixtures such as silica fume, two kinds of materials emerged in the 1980s, **Birchall et al.(1982)** developed polymer modified

cementitious materials called Macro-Defect-Free (MDF) cements. The pores are filled by polymerization leading to a compact matrix. However, these concretes are susceptible to water and have high creep. **Bache H.H. (1987)** developed the DSP (Densified Small Particles) which use the interaction of superplasticizers and silica fume to decrease the porosity of the material and to increase the strengths. That way, he prepared the ground for modern UHPFRC development.

After that, it was demonstrated that the addition of silica fume and water reducing agents/plasticizers is a simple way to produce a concrete with high strength and good workability. In this field, a significant innovation was the development of the material SIFCON (Slurry Infiltrated Fibre Concrete), in which the fibres are preplaced in the formwork moulds to its full capacity and the resulting fibre network is infiltrated by a cement-based slurry (**Naaman and Homrich, 1989**). In 1994, a new mix named "Reactive Powder Concrete" was developed, which continues to exist today as "Ductal" (**Richard and Cheyrezy, 1995**). With further developments of the concrete technology, engineers realized that the advanced concrete, besides the high strength, should also has other excellent properties, which led to the term "Ultra-High Performance Concrete (UHPC)" and "Ultra-High Performance Fibre Reinforced Concrete (UHPFRC)" (**Acker and Behloil, 2004**). Then, the UHPC and UHPFRC attracted more and more attention (Yang et al., 2009; Tuan et al., 2011a; Hassan et al., 2012; Tayeh et al., 2012; Park et al., 2012; Rossi, 2013; Wang et al.,s 2014; Yu et al., 2014a; Yu et al., 2014c; Yu et al., 2015). In 2002, the first design rules for UHPFRC were published in France (French Interim Recommendations, 2002).

The development of UHPFRC and the tailoring of their specifically properties is still not entirely known and presents a research topic at several universities.

1.2.2 Definition of UHPFRC

Ultra-High Performance Fiber-Reinforced Concrete (UHPFRC) is relatively new construction material. Hence, it is a combination of high performance concrete and fiber reinforcement. UHPFRC is generally made using fine, coarse aggregates, very low amounts of water and high amounts of cement. Silica fume is generally considered in the preparation of UHPFRC as it provides more strength. These materials are characterized by a dense microstructure. The sufficient workability is obtained by using superplastisizers in combination with the low-water demand of the fresh concrete. The Compressive strength ranges between 120MPa to 150MPa, which allows the construction of sustainable and economic buildings with an extraordinarily slim design. The mechanical performance, durability and ductility behavior of UHPFRC differs

scientifically from normal and high strength concretes due to the high-packing density of these materials. Apparently high brittleness of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) is a major problem. The increase in compressive strength decreases the ductility. This matter limits its use in structures.

1.2.3 Principles of UHPFRC

The main principles for UHPFRC design are (Richard P., Cheyrezy M;1995)

- **Homogeneity enhancement:** The homogeneity of the material is improved by eliminating coarse aggregates.
- **Compacity enhancement:** The density of the matrix is increased by optimizing the packing density. The different particle size classes are silica fume (mean size: 0.1 to 0.2 μm), cement (mean size: 15 μm) and fine sand (mean size: 0.2 mm). The optimum packing density can be determined with granular packing models by calculating the optimum ratio of the different aggregate classes.
- **Ductility by fibres:** As the matrix of DSP (Densified Small Particles) is very brittle, steel or organic (e.g. carbon or glass) fibres have to be added to obtain strain-hardening behaviour in tension.

UHPFRC may be subjected to heat and / or pressure treatment. Pressure treatment of the fresh material increases the density by reducing the entrapped air, by removing excess water and by accelerating chemical shrinkage. Post-set heat treatment of 90 °C accelerates the pozzolanic reaction and modifies the microstructure of the hydrates.

1.2.4 Constituents of UHPFRC

UHPFRC consists of cement, silica fume, sand, fibres, water and superplasticizer. Typical water/cement-ratios are 0.15 to 0.25 with 20 to 30% of silica fume.

1.2.4.1 Cement

The cement content ($\geq 700 \text{ kg/m}^3$) is more than two times higher than for normal strength concrete. There is general agreement that the cement should have a low alkali content, low to medium fineness and a low C_3A -content, thus, reducing water need, ettringite formation and heat of

hydration. Any variation in cement content causes the compressive strength of concrete to fluctuate more than any other single material.

1.2.4.2 Silica fume

Silica fume fulfils three functions in UHPFRC: it fills voids between cement grains, it enhances the rheological characteristics and it forms hydration products by pozzolanic activity. Thus, the mechanical strengths are increased and microstructure and compacity of the UHPFRC are enhanced. Best results are reported for silica fume from the zirconium industry having few impurities and a Blaine fineness of $14 \text{ m}^2/\text{g}$. The optimum filling performance of silica fume in cement is reached for silica fume-contents of 25% of cement content..

1.2.4.3 Sand

UHPFRC aggregates are sand. Quartz sand is proposed, since it has a high hardness and provides good paste-aggregate interfaces. The mean particle size is often smaller than 1 mm, but also UHPFRC with maximum particle sizes of 8 to 16 mm are produced (Richard P., Cheyrezy M;1995). It must be noted that the grain size distributions of cement, silica fume and sand have to be optimized in order to obtain high compacity and thus, a dense matrix with a very low permeability

1.2.4.4 Superplasticizer

Superplasticizers are essential for the processing (workability) of UHPFRC with their low water-content. In general, superplasticizers of the third generation – poly-carboxylates and poly-carboxylate ethers - are used, since they are highly efficient and have no efficiency threshold for low w/c-ratios.

1.2.4.5 Fibres

Steel and organic fibres are used in UHPFRC. Naaman defined the demands on fibres as “short, stiff and strong”, Naaman A.E.(2002). Thus, steel fibres seem to be the most adapted. Steel fibres are generally pulled out; therefore, fibre ductility is less important than their strength. Naaman developed limit curves for hardening under bending and uniaxial tension as a function of the fibre properties. Rossi P. et al (1987) developed UHPFRC on the basis of the multiscale concept with several kinds of steel fibres - going from steel wool ($l \approx 1 \text{ mm}$) to small steel fibres (5 to 15 mm) to long ones ($> 15 \text{ mm}$). The different fibres work on the material scale to increase the tensile

strength and on the structural scale to increase resistance and deformation capacity. Typically, UHPFRC have fibre contents of more than 2% by Vol. The maximum fibre content in function of the aspect ratio of the fibres is limited by processing factors such as workability.

1.2.5 Advantages of UHPFRC

The most important advantages of UHPFRC for structural applications can be summarised as follows:

- Mechanical strengths are greatly improved in relation to those of ordinary concrete;
- The use of ordinary reinforcement can be reduced and, in some cases, completely avoided due to the strength and ductility of UHPFRC in tension;
- Hardened UHPFRC is a very dense material, resistant to chemical and physical aggression and consequently far more durable and stable in time than ordinary reinforced concrete;
- UHPFRC is generally self-placing and self-compacting.

Thanks to mechanical strengths, reduction of ordinary reinforcement, and matrix density, very thin and light members can be made of UHPFRC, with advantages from both a structural point of view (e.g. reduction of dead load) and an architectural point of view, since thin members can be freely shaped and sized to meet a wide range of functional and aesthetic requirements. The second point means that working time for shaping and placing ordinary reinforcement is greatly reduced and production speed increased, with valuable effects especially in the precast industry. Light precast members can be easily stored, moved and placed, resulting in faster construction phases and a reduction in construction site size. The dense matrix, resistant to environmental aggression, allows maintenance costs to be decreased. To summarize all the above advantages, the integral quantity of material, labour and invested time are reduced, also resulting in significantly decreased environmental impact when constructing with UHPFRC.

1.2.6 Limitations of UHPFRC

At the moment, however, certain technical aspects partly explain the limited use of UHPFRC in structure:- The high price of the material, mainly due to the high portion of costly constituents (fibres, cement, admixtures) and the small-scale production;

- The sensitivity of the material to mixing procedure and mix proportions;
- The variability of mechanical properties as a function of casting conditions (formwork shape, paste flow).

The fact that, despite its outstanding properties, only a limited number of structures have been built using UHPFRC may also be related to designers' needs and habits and current knowledge on the subject:

- UHPFRC has not existed long enough, from a civil engineering point of view, to overcome the strong influence of traditional design concepts characterising common engineering practice;
- There is insufficient feedback from existing structures;
- Design rules are lacking and partial recommendations have been published only recently.

1.2.7 Applications of UHPFRC

UHPFRC were originally developed for the precast industry. The typical way to use UHPFRC is to produce precast elements that are assembled on the building site. The high strengths and the low permeability suggest the use of UHPFRC for zones subjected to detrimental substances or where high mechanical loads have to be introduced or transferred.

To-date, many prototype UHPFRC structures have been constructed in various countries such as France, USA, Germany, Canada, Japan, South Korea, Australia, New-Zealand, and Malaysia. The first application of UHPFRC was the UHPFRC in filled steel tube composite used in the construction of a footbridge in 1997 at Sherbrooke, Canada. Since then, UHPFRC has caught the attention of academics, engineers and many governmental departments worldwide. The first fully UHPFRC footbridge spanning 120 meters in the world was constructed in Seoul, South Korea in 2002. Subsequently, a motorway bridge was designed by VSL (Australia) at Shepherds Gully Creek, Australia, and was opened to traffic in 2005. UHPFRC can be used in a broad range of highway infrastructure applications due to its high compressive and tensile strengths and its enhanced durability properties; thereby allowing a longer design/service life and thin overlays, claddings, or shells. In addition, UHPFRC is also being considered to be used in a range of other applications such as precast concrete piles, seismic retrofit of substandard bridge substructures, thin-bonded overlays on deteriorated bridge decks, and security and blast mitigation applications.

Figure 1.2 presents a schematic drawing showing the evolution of UHPC technology with respect to structural and architectural applications from 1995 to 2010.

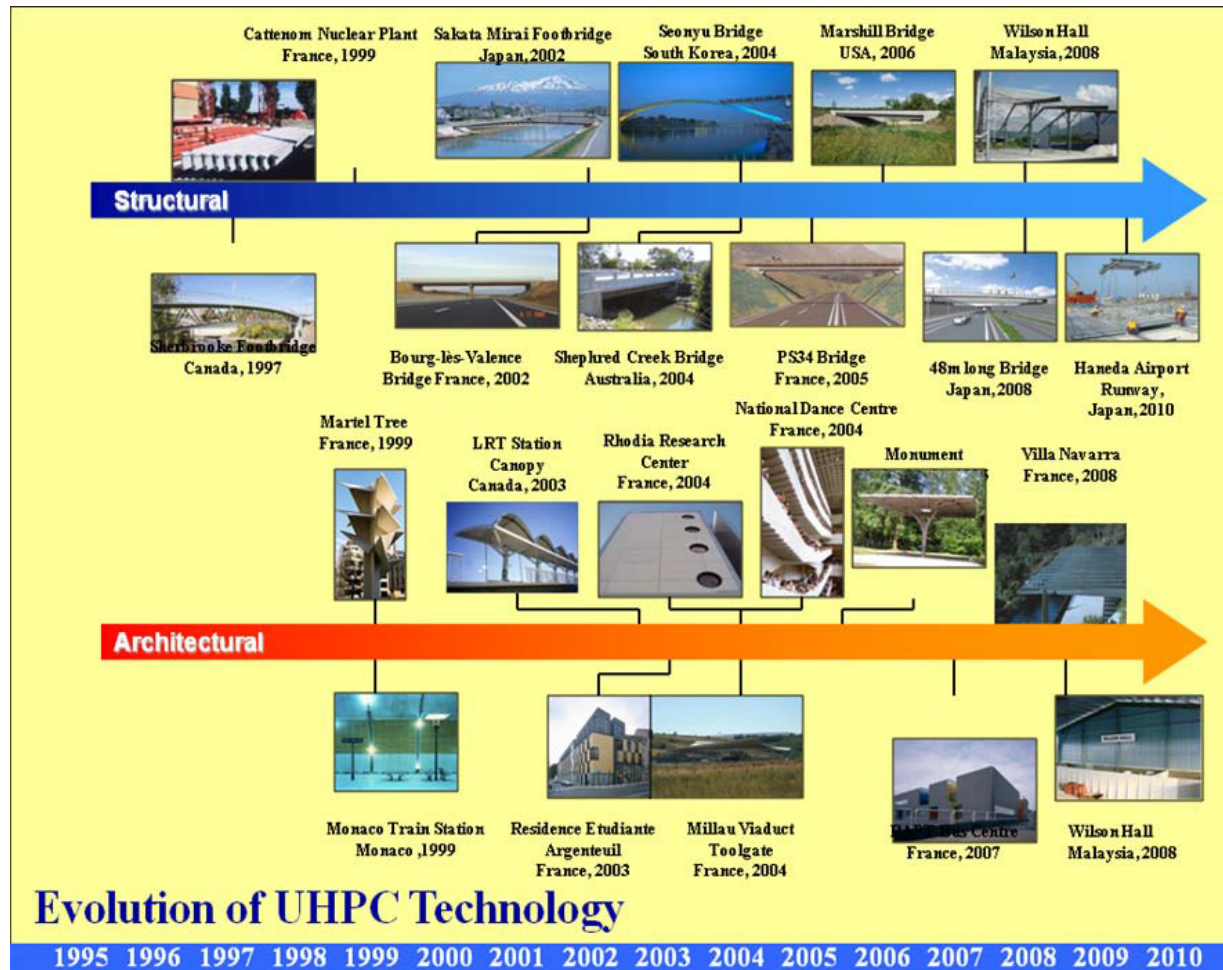


Figure 1.2 Applications of UHPFRC (Yen L.V. et al., 2012)

The current initial price and the technology required for UHPFRC can only be justified if its mechanical or other material properties are well exploited. For this reason, UHPFRC should not be considered an all-field alternative to ordinary concrete: successful applications should be sought in domains where OC barely meets specific design requirements. In accordance with the discussion in the previous paragraph, UHPFRC can be successfully used when durable and strong but light members are required, or in cases where a considerable liberty with regard to form is desirable. From this point of view UHPFRC can be an interesting alternative to structural steel. Until now, UHPFRC has been used structurally mainly for:

- Roof elements
- Foot bridges
- Road bridges
- Protective elements in hydraulic or aggressive environments.

1.2.7.1 *Sherbrooke Footbridge*

The world's first engineering structure designed with UHPC was the Sherbrooke footbridge in Sherbrooke, Quebec, built in 1997. Spanning 60 m, this precast, prestressed pedestrian bridge is a post-tensioned open-web space RPC truss, shown in Figure 1.3(a) below, with 4 access spans made of Ultra High Performance Concrete (UHPC). The main span is an assembly of six 10 m prefabricated match-cast segments. The cross section is made of a ribbed slab 30 mm thick, with a transverse prestressing made of greased-sheathed monostrands. The truss webs are made of RPC confined in stainless steel tubes.

The structure is longitudinally prestressed by an internal prestressing placed in each longitudinal flange and an external prestressing anchored at the upper part of the end diaphragms and deviated in blocks placed at the level of the lower flange. The connection between the flanges and truss diagonals is ensured by greased-sheathed monostrands and miniaturized anchorage. Vibration calculation has been carried out and tuned mass dampers have been installed to reduce the effect of the first three modes of vibration of the footbridge. It is worth mentioning that it is the first Ductal footbridge built in Japan with a span of 50 m. The deck is a simple beam of 2.4 m wide with circular web holes. The structure is longitudinally prestressed by an external prestressing and has no passive reinforcement.

1.2.7.2 *The Seoul and Sakata Miral Footbridges*

In 2001 and 2002, a footbridge was built over the Han River running across Seoul in South Korea. It is made of an arch spanning 120 m, with two steel access spans, shown in Fig. 1.4 (a). The arch has a π -shaped cross-section, shown in Fig. 1.4 (b), 1.3 m deep. The upper flange is a ribbed slab 30 mm thick, with a transverse prestressing made of greased sheathed monostrands. The webs are 160 mm thick and are inclined outward. The arch is an assembly of six 20 m prefabricated segments, connected on site by means of temporary supports. The elements are

stitched together by an internal longitudinal prestressing placed in haunches in the lower and the upper parts of the webs. This very slim structure has frequencies of vibration sensitive to the pedestrian traff



Fig.1.3 Sherbrooke Footbridge



Fig 1.4 (a) General View of Seoul Footbridge [Karmout, M., (2009)]

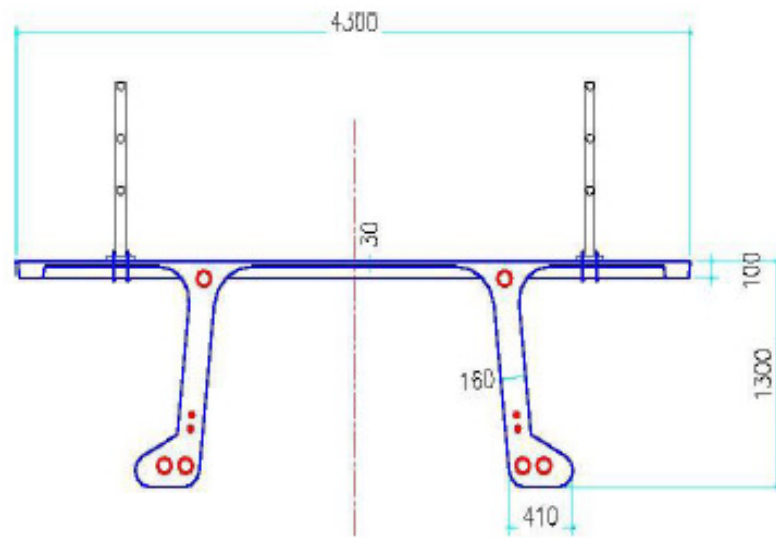


Fig. 1.4 (b) Cross-section [Karmout, M., (2009)]

1.2.7.3 18 M Kampung Ulu Kampar Bridge (Medium Traffic Bridge)

Figure 1.5 shows example on a short span bridge crossing a river with a total span length of 18 m was constructed by JKR Perak (Kinta Daerah). The bridge is located at a small village call Kampung Ulu Kampar, which is approximately 30 km from the capital city of Perak, Ipoh. Similar to the Kampung Ulu Geroh Bridge (refer to Section 2.6), this bridge also uses the full UHPFRC bridge/deck system. This bridge was designed to withstand 30 units HB loading and HA + KEL loading as per BD37/01. Construction of the bridge commenced at mid January 2012, and the bridge work was complete at end of February.

Similarly, the major challenge of the project is the poor access road to the job site. No long trailer is able to access to the job site. Although UHPFRC girder system has weight advantage over conventional system, the bridge designer or contractors still have bridge length issue to consider. The bridge designer eventually comes up with an idea to break the 18 m girder into three segments, thus having each segment measured 6 m long and weighted merely 6 tonnes. Thus a simple ten wheels truck can be used to transport the bridge segment (one at a time) and later on aligned off-site, then post-tensioned to form a single girder. (<http://penerbit.uthm.edu.my/ojs/index.php/IJSCET>)

1.2.7.4 UHPFRC Portal Frame Building (Warehouse Solution)

In year 2008, a portal frame building named *Wilson Hall* with a roof coverage area of 2,861m² was built using the prefabricated system of UHPFRC technology. The total transverse width and longitudinal length of the building is 67 m and 42.7 m, respectively. Each UHPFRC portal frame was spaced at 12.2 m c/c and the building consists of eight pieces of UHPFRC prestressed columns, internal rafters, cantilever rafters and connections as shown in Figure 1.6. To-date, this building is the world first attempt to replace conventional steel beam with the UHPFRC prestressed beams/columns. This building has earned a place in the *Malaysia Book of Records* in year 2010.

Figure 1.6 (b) presents the environmental impact calculation (EIC) of the UHPFRC portal frame system against the conventional steel portal frame system. In terms of material consumption, the UHPFRC portal frame system consumed 13% less material than the conventional steel portal frame solution. With regard to immediate construction cost, the UHPFRC system is 16% more economical than the conventional steel structural system. More cost savings can be realised for those factory buildings that are located in corrosive environment or places constantly subjected to chemical attack such as chemical plants, due to conventional steel structure would require periodic maintenance. In terms of environmental indexes, the UHPFRC solution has 24% less embodied energy and 19% less CO₂ emissions. For the 100-year GWP, the UHPFRC solution provides a reduction of 16% to the conventional solution. Thus this shows that the UHPFRC system can give a more sustainable solution against the conventional method.



Figure 1.5 The New UHPFRC Bridge At Kampung Ulu Kampar, Gopeng, Perak.



Fig . 1.6 (a)

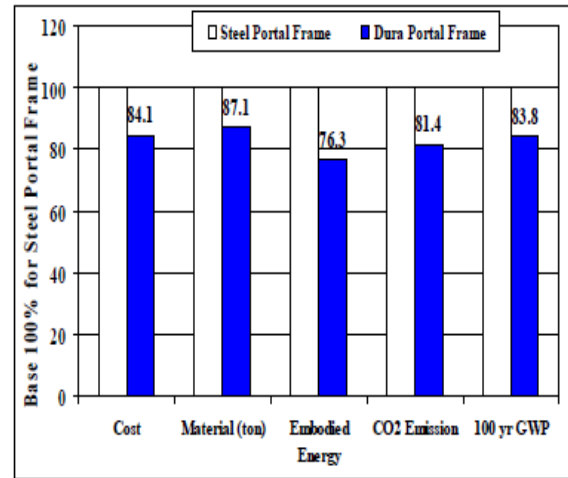


Fig. 1.6 (b)



Fig. 1.6 (c)

Figure 1.6 (a) UHPFRC Portal Frame (before completion at year 2008), (b) Environmental Impact Calculation and (c) Completed *Wilson Hall* (2012).

1.3 ORGANIZATION OF THESIS

Many researchers put their theory for use of cement based composites for retrofitting but very less work specifically regarding the behavior of ultra high performance fiber reinforced concrete

(UHPRFC) for strengthening of shear deficient beams has been done. So the aim of this thesis is to study the effect of retrofitting with UHPRFC on the strength and deflection characteristics of shear deficient beams initially stressed to a specified limit. Organization of thesis has been organized in the following five chapters:

Chapter 1: This chapter deals with general introduction and history of Ultra High Performance Fiber Reinforced Concrete (UHPRFC).

Chapter 2: This chapter deals with review of literature work and the need for the present study.

Chapter 3: This chapter details the experimental programme. In this, properties of various material used, testing equipments and procedures adopted in this work have been discussed.

Chapter 4: This chapter is about the results and discussion of the work carried out.

Chapter 5: This chapter details the conclusions of the work carried out.

CHAPTER 2

REVIEW OF LITERATURE

2.1 GENERAL

This chapter presents a review of the existing literature regarding the development of UHPFRC and study of beams retrofitted using UHPFRC

2.2 REVIEW OF PREVIOUS WORK

2.2.1 Development of UHPFRC

This chapter presents a review of literature highlighting the work done by various researchers with regards to the strength of UHPFRC.

Al-Azzawi et al. (2011) investigated a study to understand the behavior of UHPC members with steel fibres. They used two different kinds of admixtures for ultra-high performance concrete (namely, high reactivity metakaolin and silica fume) were used separately with three different steel fibers ratios (1.0 %, 1.5% or 2.0%) for each mix. Therefore, the total number of UHPC mixes considered in the experimental investigation was six. From each specific UHPC mix, three cubes (50 mm×50 mm× 50 mm) were tested under uniaxial compression to assess the compressive strength of UHPC at 3,7, and 28 days of age.

They used Sulfate resisting portland cement in UHPC because it contains low C_3A content. Independent of other variables, mixes with cement of low C_3A content always require lower w/c ratios as compared to mixes with ordinary cement. Also, very fine sand with maximum size 600µm was used. This sand was separated by sieving, its grading satisfied the fine grading in accordance with the B.S. specification No. 882/1992. The steel fibres used during their investigation were Straight brass coated steel fibers 13 mm long with a diameter of 0.18 mm and an aspect ratio of 72. Mineral admixtures like High Reactivity Metakaolin (HRM) and Silica fume (SF) were also used along with a high performance concrete superplasticizer (or named

High Range Water Reduction Agent HRWRA) based on polycarboxylic technology, Structuro 335 in combination with Structuro 480.

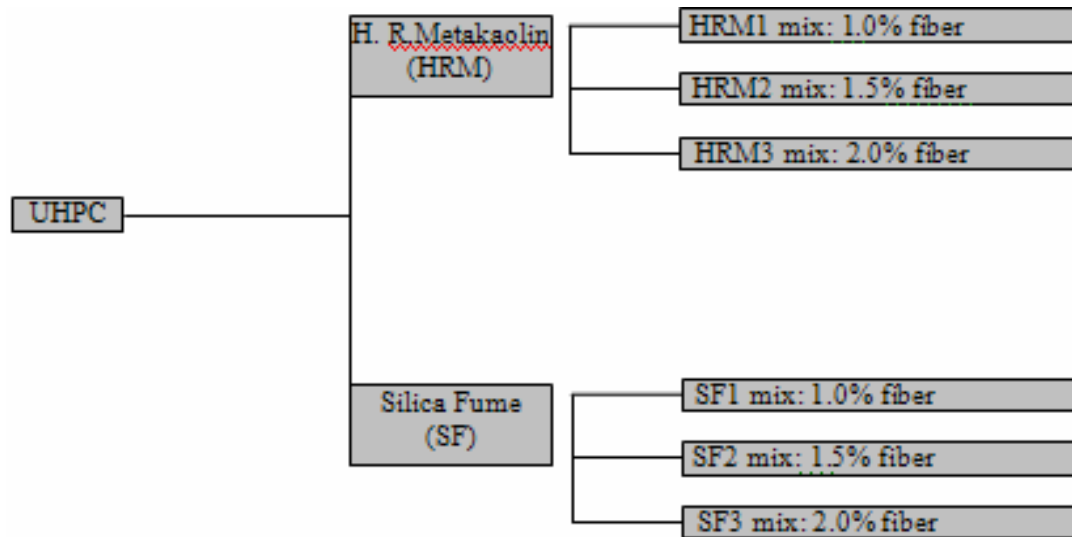


Fig.2.1 Mixes Studied in the Investigation [Al-Azzawi et al., 2011]

They studied the influence of the different mixture ratios on the mechanical properties of Ultra High-Performance Concrete, considering the following aspects;

- Two types of mineral (pozzolanic) admixtures [namely: High Reactivity Metakaolin (HRM) and Silica Fume (SF)].
- Three proportions of steel fibres (1.0%, 1.5%, 2.0%) for each type of mineral admixture.

This gives six different mixes which were used in this investigation as shown in Fig.2.1.

They carried out compression tests on a standard 50 mm cubic specimens tested after curing at the ages of 3, 7, and 28 days for each mix. The average of three cubes was adopted for each age. The testing procedure was carried out using an Avery-Denson 2000 KN capacity compression testing machine. Test results for compressive strength of various mixes of Ultra High Performance Concrete investigated in this study were summarized in Table 2.1. The Table shows the compressive strength values at three ages (3, 7 and 28 days).

Table 2.1 Compressive strength of UHPC mixes at 3, 7 and 28 days age**[Al-Azzawi et al., 2011]**

Mix	Compressive Strength (MPa)							
	Cement (Kg/m ³)	H.R. Metakaoline (%)	Silica fume (%)	Steel fibres (%)	w/c ratio	3 days	7 days	28 days
HRM1	850	15	-	1	0.174	88	103	151
HRM2	850	15	-	1.5	0.178	90	106	155
HRM3	850	15	-	2	0.179	91	109	159
SF1	900	-	10	1	0.186	98	122	191
SF2	900	-	10	1.5	0.189	99	124	195
SF3	900	-	10	2	0.190	102	127	198

They found that a 28 days compressive strength of 198MPa was obtained for SF3 mix (mix included Silica Fume and steel fibres of 2.0 % volume fraction) while a 28 days compressive strength of 159MPa was obtained for HRM3 mix (mix included High Reactivity Metakaolin and steel fibres 2.0 % volume fraction). Also they noticed the results showed that the addition of pozzolanic mineral admixtures (metakaolin or silica fume) could cause a considerable increase in compressive strength at early ages. That was mainly due to the fact that the chemical reaction of pozzolanic materials usually starts at early ages (3 and 7 days), and increases till the age of 28 days. This could be explained by the high pozzolanic reaction of particles of these materials with calcium hydroxide released from cement hydration leading to pore size and grain size refinement processes which can strengthen the microstructure and reduce the microcracking.

Prem et al. (2012) conducted a research program to evaluate the mechanical properties of Ultra High Performance Concrete, target compressive strength at the age of 28 days being more than 150 MPa. For this purpose, they considered the chemical composition of ingredients as given in Table2.2(a).

Table 2.2(a) Chemical composition of ingredients [Prem et al., 2012]

Mix	Cement (Kg/m ³)	Silica fume (Kg/ m ³)	Quartz (Kg/m ³)	Fine Aggre- gates (Kg/m ³)	Water (Kg/ m ³)	SP (Kg/m ³)	RF Index	Steel fibers (%)	w/c ratio
R1	788	197	315	866.8	173	14.77	2.0312	S1-2. 5	0.22
R2	788	197	315	866.8	173	14.77	1.625	S1-2	0.22
R3	788	197	315	866.8	173	14.77	0.9375	S2-2. 5	0.22
R4	788	197	315	866.8	173	14.77	0.75	S2-2	0.22
R5	788	197	315	866.8	173	14.77	0	Nil	0.22

They used the cement during the experiments was Ordinary Portland Cement of Grade 53 conforming to IS 12269:1987. The specific gravity was 3.15; the initial and final setting times are 110 min and 260 min. The normal consistency being 28% and the particle size range lies between 31µm to 7.5µm. The silica fume they used in the experiment conforms to ASTM C1240 – 97b. The specific gravity being 2.25, percentage passing through 45µm sieve in wet sieve analysis is 92% and the particle size range lies between 5.3 µm–1.8 µm. Quartz Powder used had the specific gravity being 2.59, percentage passing through 45µm sieve in wet sieve analysis was 75% and the particle size range lies between 5.3 µm – 1.3 µm. *Sand* - The sand used for the experimental studies are Grade I Coarse (particle size range - 0.6mm -2.36 mm) and Grade III Medium (particle size range -0.075-0.15mm). Also, the Super Plasticizer- Poly-Acrylic Ester based type SP was preferred and S1 (Steel fibres) -13 mm length, 0.16mm diameter, S2 (Steel fibres)-6 mm length, 0.16mm diameter was used in the research.

The compressive strength of mix (R1-R5) evaluated at 7, 14, 21 and 28 days are shown Table 2.2(b). They observed that the specimens attained 90% of the compressive strength around the age of 14 days. The mixes having same fibre volume irrespective of aspect ratio produced 25% increment in compressive strength, to that of the control mix. The results showed that the

compressive strength didn't depend too much on the reinforcement index of fibre. The specimens attain 90% of the compressive strength at the age of 14 days.

Table 2.2(b) Cube Strength Evaluated [Prem et al., 2012]

Mix	Compressive Strength (MPa)			
	7 days	14 days	21 days	28 days
R5	104	123	126	132
R4	129.8	149.7	156	164
R3	149.5	167.2	172	178.70
R2	147.2	162.3	167.3	170.29
R1	152.67	168.5	176.4	180.28

M. Gesoglu et al. (2015) studied the effect of using binary and ternary blends of nanosilica (NS) and microsilica on the mechanical properties of low binder ultra-high performance cementitious composites (UHPCs). For this, two concrete groups were designed with and without silica fume by weight of cement with a constant water/binder ratio and total binder content. Commercially available NS was used in partial substitution of cement at 0%, 0.5%, 1%, 2% and 3% by weight.

As per **M. Gesoglu et al. (2015)**, the mixture proportioning studied in the experimental program is shown in Table 2.3. Group 1 and 2 in the mix design codes show 0% and 10% SF respectively with a mutual NS content of 0%, 0.5%, 1%, 2% and 3%. Superplasticizer was used in varying amounts to adjust the workability enough for the mixtures. The mixtures in Table 2.3(a) were designated according to NS and SF replacement level. For example, SF0NS1 indicates the mixture containing 0% of silica fume and 1% of nanosilica.

They conducted the compression test on 50 mm cubes at 1, 3, 7, 14, 28, 56, and 90 days with respect to ASTM C39 and observed the effect of using nanosilica (NS) on the compressive strength development of the UHPC with and without silica fume (SF) are shown in Fig. 2.2(a) and (b), respectively.

Table 2.3 Mixture Proportions of UHPC Containing NS and SF (Part 1 and 2)

[M. Gesoglu et al., 2015]

Group	Mix	Cement (Kg/m ³)	Silica fume (Kg/m ³)	Nano silica (Kg/m ³)	Water (Kg/m ³)	Superpla- sticizer (Kg/m ³)	Quartz Aggregate (Kg/m ³)
1	SF0NS0	800	0	0	160	21.6	1471.3
	SF0NS0.5	796	0	4	160	25.2	1461.1
	SF0NS1	792	0	8	160	28.8	1450.8
	SF0NS2	784	0	16	160	36.0	1430.2
	SF0NS3	776	0	24	160	43.2	1409.7
2	SF10NS0	720	80	0	160	29.6	1422.9
	SF10NS0.5	716	80	4	160	33.6	1411.6
	SF10NS1	712	80	8	160	37.6	1400.3
	SF10NS2	704	80	16	160	44.8	1379.8
	SF10NS3	696	80	24	160	52.0	1359.2

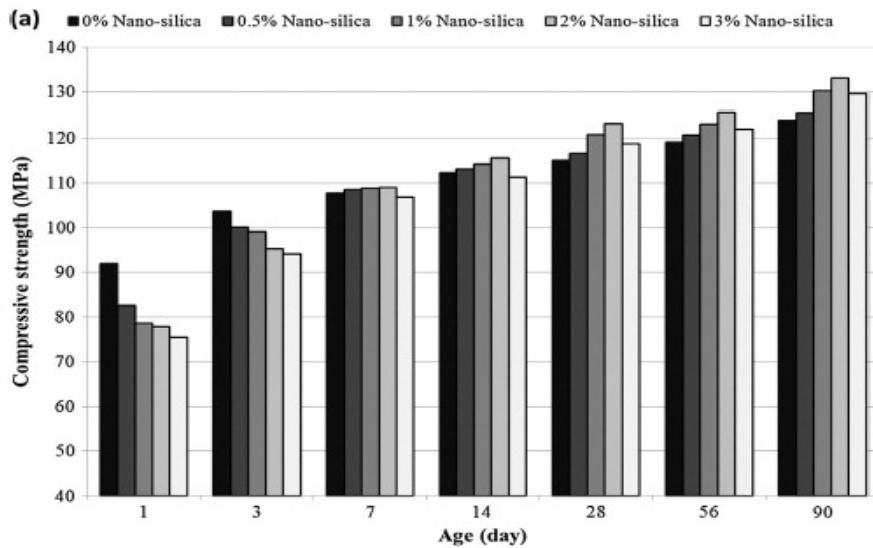


Fig.2.2 (a) Compressive Strength Versus Different Age water Cured in Plain UHPC (0% SF)

[M. Gesoglu et al., 2015]

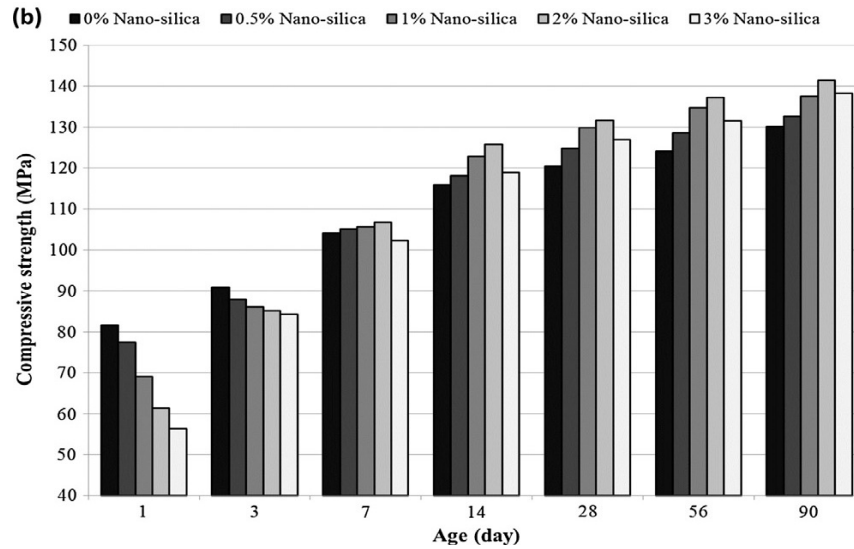


Fig.2.2 (b) Compressive Strength Versus Different Age Water Cured in Plain UHPC (10% SF)
[M. Gesoglu et al., 2015]

2.2.2 Retrofitting of Structural Elements Using HPC/RPC/UHP-FRC

Much research has been done to assess effectiveness of FRC retrofitting technique during the last years. This section presents a brief review of research done related to the use of fiber reinforced composites with cement like matrix for repairing or strengthening purposes. The research done concerning retrofitting with this type of materials has two main motivations. By one side, it is aimed at solving durability or aging problems or prolonging service life of the structure and by the other side, it is aimed at solving strength or structural problems.

The use of fiber reinforced both for the strengthening and repairing of different types concrete and reinforced concrete elements like beams, columns, panels, joints, slabs and pavements has been extensively investigated during the last years. Different types of cement based materials for the matrix like normal strength, high strength concrete or self consolidating concrete, were used and compared. Moreover different types, sizes and shapes of fibers were used.

The reinforced concrete structures retrofitting technique using fiber reinforced concrete (FRC) avoids some of the problems that other systems present like the brittle failure of the interface retrofitting layer/concrete. Compared with fiber reinforced polymers, fiber reinforced cement composites present higher resistance against high temperature and ultraviolet radiation, more

long term durability and fundamentally more compatibility with the substrate. Moreover, the use of fibers in the retrofitting concrete layer helps controlling shrinkage cracking.

Krstulov and Toutanj (1996) used a layer of thin high performance fiber-reinforced cementitious composites (HPFRCC) overlays, between 10 and 50 mm to repair deteriorated cementitious structures such as cracked pavements and bridge decks. It was found out this thin overlay tensile strain-hardening behaviour improves the original structure's deformation and energy dissipation capacity. Thus, HPFRCC overlays are able to bridge existing cracks in the concrete substrate. The cracks in the HPFRCC layer remain small, between 30 μm and 50 μm are closely spaced.

Namaan et al. (1996) studied the use of slurry infiltrated mat concrete (SIMCOM) for repair and rehabilitation of reinforced concrete beams and columns. They concluded that SIMCOM can successfully interact with reinforced concrete elements substantially increasing flexural strength and energy absorption capacity. Adding a layer of SIMCON in the compression zone and/or tension zone increases both the lever arm and strength of the compression and the tension zones respectively. This leads to a significant increase in strength and energy absorption capacity of the beam.

Mesbah et al. (2000) studied the flexure behavior of beams repaired in the bottom concrete cover with self compacting concrete and self compacting FRC was experimentally studied. The beams repaired with FRC presented thinner cracks. The use of self compacting concrete has shown to be a good option to facilitate the pouring. The behavior of the repaired beams seemed to be not influenced by the placing and the length of the repaired zone.

Li et al. (2000) proposed a system for the stiffening of frame structures with precast engineering cement composites (ECC) plates. They studied the performance of the system numerically. The plates contributed to energy dissipation of the structure through the early cracking of ECC. Although the ECC has strength not very different from the FRC or mortar used, structural strength and structural ductility was all much higher when ECC was used, especially when ECC composition (fiber, matrix and interface) was adequately tailored taking into account the influence of material micro-mechanics on composite macromechanics and structural behavior.

Denarie (2003) studied an experimental pilot-scale test on the flexure behavior of various composite UHPFRC and conventional reinforced concrete (RC) beams. The composite beams were comprised of an UHPFRC overlay to replace the standard tensile reinforcing bar in an RC beam and exhibited an ultimate force compared to the standard RC beams. The composite beams exhibited an increased stiffness until the ultimate force was reached and, thereafter, a pronounced softening behavior, while the RC beams showed a slight hardening behaviour commonly observed in concrete flexural members.

Habel (2004) evaluated the performance of (UHPFC) overlay material above ordinary reinforced concrete element subjected to impact loading. The application of a UHPFC overlay significantly improved the structural response of the member subjected to impact loading, with none of the spalling, crushing, or cracking that are typically common for normal concrete overlay, and the results confirmed that applying a UHPFRC layer to form a composite UHPFRC and NC element increases the service condition stiffness, minimizes deformations for given imposed forces, reduces crack widths and crack spacing, and delays the formation of localized macro-cracks as compared to the original conventionally reinforced concrete element. This improved performance is attributed to the UHPFRC layer of high tensile strength and strain hardening properties.

Habel (2007) studied the use of CARDIFF (a new class of high performance fibre reinforced cementitious composite). It that had been used as bonded strips applied to the tensile face to rehabilitate and improve existing reinforced concrete beams. It was found that the rehabilitated composite beams behaved monolithically until fracture and the composite beam ultimate force was equal to or higher than the reference concrete member. Also, the experiment resulted in high tensile/flexure strength and high energy-absorption capacity (i.e.Ductility). So, finially he demonstrated that the damaged reinforced concrete beams can be effectively rehabilitating and strengthening in a variety of different retrofit configurations using CARDIFRC strips adhesively bonded to the prepared surfaces of the damaged beams.

Kim et al (2007) proposed a repair method consisting of the replacement of damaged materials in aged structures due to the lack of maintenance. They used ductile fiber reinforced cementitious composite (DFRCC) as repair material for over reinforced concrete beams under flexure. Neither the strength nor the deformation capacity were significantly changed with this

type of repair and the importance of the interface between the old and the new material and the debonding prevention were showed.

Wang and Lee (2007) proposed the use of ultra-high strength steel fiber-reinforced concrete (UFC) jacketing for the strengthening of internal nodes of three beam-column sub-assemblages used as moment-resisting reinforced concrete frames. They showed that the use of UFC led to an increase of ductility and the formation of plastic hinges in the beams.

Two types of retrofitting schemes were adopted to strengthen the as-built frame: RC jacketing with 100 mm thick NC (as displayed in JI-RC), and replacement of the joint core concrete with UFC (JI-UFC). The behavior of the shear resistance of different concrete materials used for joint shear strengthening was studied and the relationship between the joint shear stress and strain, attained from measurement of the joint area was discussed. The initial shear stiffness (the so-called shear modulus) was similar for all three units but the highest joint shear strength was different. They depicted that the joint shear failure mode of the prototype J1 unit had the maximum joint shear strength of 5.59 MPa whereas the strengthened unit J1-UFC was subject to the ductile failure when the imposed joint shear strength was as high as 7.09 MPa. Also, the shear modulus for UFC joint was linear all the way up to failure but for unit JI, the nonlinearity in joint shear appeared at the post cyclic excursions.

The failure mode of JI-UFC was changed from brittle to ductile behavior because of joint core replaced by UFC. Meanwhile, It was seen that JI-UFC showed stiffer shear resistance than the RC jacketed unit JI-RC. Although the NC jacketing of the JI-RC unit caused the failure mode to become a ductile one, the shear deformation, at the maximum imposed shear, became larger after JI-RC was subjected to a few loading cycles. Their implication was that UFC would be a better material than NC jacketing for strengthening non-ductile beam-column joints.

Farhat et al. (2007) presented a new material called ultra-high performance cement-based fiber composite (CARDIFRC). It is an ultra high performance composite reinforced with 8% in volume of short fibers. They used this material for the reinforcement of under reinforced concrete beams under flexure and over reinforced concrete beams under shear. The reinforcement consisted of thin plates made of CARDIFRC that were glued with epoxy cement

to previously damaged beams. In all strengthening setups the strength of the beams was increased with the reinforcement.

Massicotte and Boucher-Proulx (2008) presented experimental results of two actual scale bridge piles repaired with high performance fiber reinforced concrete (HPFRC). The concrete cover of one of the piles was removed and replaced by HPFRC. The piles were tested under quasi static cyclic load. The strengthened pile presented greater load bearing capacity, increasing with load cycles, and also greater ductility.

Brühwiler and Denarié (2008) used 30 mm thick UHPFC overlay for the rehabilitation of short span road bridge with heavy traffic. The protective function of the UHPFRC overlay was verified by air permeability tests which confirmed the extremely low permeability; approximately 30 times lower than normal concrete. The study of the construction costs showed that the rehabilitation with UHPFRC was 12% more expensive than a more traditional solution with waterproofing membrane and rehabilitation mortar and providing lower quality in terms of durability and life-cycle costs. However, in the latter case the duration of the site would have been largely increased by the drying period of the rehabilitation mortar, prior to the application of the waterproofing membrane (up to 3 weeks).

Marini et al. (2008) numerically studied the behavior of a FRC diaphragm as vertical load transferring element to perimeter structure. They concluded that the use of FRC would allow the use of lower thicknesses than in the case of reinforced concrete and that the thickness could be reduced if the fiber content was increased.

Skazlic et al. (2009) evaluated the efficiency of high performance fiber-reinforced micro-concrete as a repair material when applied on concrete beam. Mix designs of concrete (C25/30) used to prepare 150 × 150 × 600 mm beams and HPFRMC used to reinforce the tension zone of concrete beams. Besides using larger amount of the cement, silica fume and superplasticizer, low water/binder ratio, diabase aggregate with maximum grain size 4 mm and larger amount of fibers were used as well. Steel fibers were used for enhancing behavior of HPFRMC under flexural stress and polypropylene fibers for eliminating the cracking of concrete in case of concreting in bad weather conditions. High values of compressive and tensile strength are preferable for the use of this material for reinforcing concrete in tension zone, while similar

values of modulus of elasticity as those of the base concrete are necessary in order to achieve the compatibility of two materials. They concluded that the use of this material for repairing purposes has both economical and technical advantages.

Boscato and Russo (2009) studied the behavior of rectangular concrete plates used as industrial pavements repaired with FRC. The ordinary RC slabs are 300 *300 cm wide and 15 cm thick. The concrete belongs to 25 MPa strength class; the traditional reinforcement consists of two layers of welded wire of 8 mm diameter and 200 × 200 mm mesh. Three different layers of SFRC reinforcement were taken into account, respectively 1, 2 and 4 cm thick. They achieved excellent adherence between the materials and increase of the first crack load and the collapse load. With respect to the RC slab taken as reference, an increment of the first crack load is attained of 10%, 57% and 71%, corresponding respectively to 1 cm, 2 cm and 4 cm of SFRC reinforcement thickness.

Martinola et al. (2010) used a self compacting FRC jacketing to strength and repair reinforced concrete beams predominantly subjected to flexion. Structural effects of a 40 mm thick HPFRC jacket were investigated by testing full-scale beams having a depth of 500 mm and a span of 4.55 m. Results showed a remarkable increase of the bearing capacity coupled with a great stiffness increase. They obtained good adherence between the jacketing and the beams and the flexural strength and stiffness were increased.

Radik et al (2011) compared two methods, glass-fiber-reinforced polymer (GFRP) sheets and a layer of fiber-reinforced cement, for strengthening two-way reinforced concrete floor slabs subjected to out-of-plane bending. They concluded that fiber-reinforced cement has great potential as a strengthening method, and future work was recommended to further optimize the proposed strengthening technique.

Maringoni et al.(2011) investigated the use of different high strength jackets with or without combination of steel meshes to provide shear strengthening for slender beams. Four beams with effective span of 2.5 m and the height of 450 mm were tested in 4 point bending test. Three of them were upgraded with different configurations. To provide a good bond between existing substrate and new composite layer, the surface was sandblasted to provide an average roughness of about 1 mm. The great potential of using advanced cement based for upgrading was

demonstrated with about double increase in strength and stiffness, by using a jacket with thickness 50 mm. To this purpose two different typologies of HPFRCC were used, one was self levelling matrix that could be casted directly in the formwork, the other was thixotropic that could be applied directly on the surface. In this work the beam was made with concrete class C20/25; the bars had a yield strength of 518 N/mm^2 , whereas the HPFRCC contained 3.9% fibres by volume content (with length 15 mm and diameter 0.175 mm) and tensile strength equal to 6 N/mm^2 and 5 N/mm^2 , for self levelling and thixotropic HPFRCC; a steel wire mesh of 2mm dia spaced at 25 mm with yield strength 550 N/mm^2 was also used. The beam (B) was strengthened with self levelling HPFRCC jacket 50 mm thick, and inside it has been inserted the U shaped stainless steel mesh. The other two beams (D) and (E) at the intrados were upgraded with self levelling HPFRCC and on the lateral surface with thixotropic HPFRCC. Also in these cases were placed the U shaped stainless steel mesh, in beam (D) for sake simplicity on application the U shaped mesh was applied up to a height of 20 cm on lateral surfaces.

Rosignoli, et al. (2012) used a high performance self compacting FRC jacketing for the strengthening of an actual scale column constructed with low strength concrete. To achieve a good adherence between both materials, the column surface was sandblasted. The test showed a remarkable increase of strength and ductility without much weight increase and high performance concrete of the jacketing provided protection to the concrete core increasing durability.

Iskhakov et al. (2013) repaired Reinforced concrete beams by replacing the compression zone concrete by steel fiber reinforced high strength concrete (SFHSC) and creating two layers beams. The beams presented a classical flexure behavior. The addition of steel fibers increased ultimate deformations and provided supplementary energy dissipation to the structure.

Iskhakov et al. (2013) presented a patch repair method that uses high performance fiber reinforced cement composites to repair reinforced concrete members damaged by chloride attack. Strength could be recovered if the amount of corrosion was less than 10%. HPFRCC layer remained impermeable due to its very small crack width.

Tayeh et al.. (2013) carried out an experimental investigation to evaluate the relationship between concrete substrate roughness parameters and the bonding performance of UHPFRC

used as a repair material. The significant influence of substrate surface preparation method on bonding strength between UHPFRC and the concrete substrate was shown. When the surface was sand blasted the composite behaved closely as a monolithic structure.

2.3 NEED FOR PRESENT STUDY

From the review of literature it is clear that most of the research studies regarding the retrofitting using UHPFRC have focused on the strengthening and rehabilitation of structural elements deficient in flexural. Some studies have also been conducted regarding the strengthening of trial sections of existing highway pavements and shoulders and footpaths using UHPFRC. However, there have been very few studies regarding the use of UHPFRC in retrofitting of shear deficient beams.

Shear failure is inherently brittle in nature and therefore, does not offer sufficient warning to the occupants prior to failure. Over the last few decades, researchers have tried to use different materials to improve the load carrying capacity and ductility (and hence energy absorption capacity) of such beams. Use of UHPFRC offers a possible solution to fulfill this requirement.

This research thesis therefore focuses on the use of UHPFRC in different configurations to strengthen the initially stressed shear deficient beams. The objectives of this study are:

1. To compare the failure mode and crack pattern for different configurations (Side Face, U-Type and External Side Face Jacketing) used for retrofitting of shear deficient beams.
2. To compare the effect of different retrofitting configurations on the load carrying capacity and ductile behavior of initially stressed shear deficient beams.

CHAPTER 3

EXPERIMENTAL PROGRAMME

3.1 INTRODUCTION

At present a number of laminates like CFRP (Carbon Fibre Reinforced Polymer), GFRP (Glass Fibre Reinforced Polymer), Ferrocement, cementitious material like UHPFRC (Ultra high strength fiber strength concrete) etc. are being used for retrofitting of structures. Among all these, UHPFRC due to its high tensile/flexural strength and high energy-absorption capacity i.e. ductility is gaining popularity.

Thus in the present study, shear deficient beams are cast and subsequently stressed to 60% of the failure load and are retrofitted with UHPFRC bonded to beam with epoxy in different configurations.

For the proposed work, 16 beams (150 x 150 x 700 mm) were cast including 8 beams each of U/R and O/R section. Out of these two beams each of the U/R and O/R sections are control beams tested to find out the failure load capacity of beams and subsequently 6 beams each of the two different sections are stressed to 60% of the failure load and then retrofitted with UHPFRC in three different configurations i.e. Side Face, U-Type and External Side Face Jacketing.

3.2 TEST PROGRAMME

The test programme is so devised so as to find out the properties of materials to be used for casting of beams and then the behavior of retrofitted beams. The test programme consists of:

1. Determination of basic properties of constituent materials namely cement, sand, coarse aggregates and steel bars as per relevant Indian standard specifications.
2. Casting of sixteen beams (150 x 150 x 700mm) using M 25 grade concrete, the mix of which is designed with evaluated properties.
3. Flexural testing of the beams in two-point loading to obtain load deflection curve of the control beam. The ultimate failure load of each type of beam is noted from the respective curve.

4. The beams are stressed to 60% of ultimate failure load.
5. Development of Ultra High Performance Fiber Reinforced Concrete (UHPFRC).
6. Finally beams are retrofitted with UHPFRC material in different configurations such as side face, U-type and external 25mm thickness side face jacketing in shear failure zone.

3.3 MATERIALS

Cement, fine aggregates, coarse aggregates, reinforcing bars are used in designing and casting of beams and micro-silica, metakaoline, steel fibers, superplasticizer, cement, epoxy are used for retrofitting of these beams. The specifications and properties of these materials are as under:

3.3.1 Portland Cement

Ordinary Portland 43 grade cement from a single lot is used for the study. IS 8112:2013 provides the specification of OPC 43 Grade. According to IS 8112 Ordinary Portland cement, 43 grade shall be manufactured by intimately mixing together calcareous and argillaceous and/or other silica, alumina or iron oxide bearing materials, burning them at a clinkering temperature and grinding the resultant clinker so as to produce a cement capable of complying with this standard. No material shall be added after burning, other than gypsum (natural mineral or chemical), water, performance improver(s), and not more than a total of 1.0 percent of air-entraining agents or other agents including coloring agents, which have proved not to be harmful. It was fresh and without any lumps. The physical properties of the cement as determined from various tests conforming to Indian Standard IS: 8112: 2013 are listed in table 3.1. Cement was carefully stored to prevent deterioration in its properties due to contact with the moisture. The various tests conducted on cement are initial and final setting time, specific gravity, fineness and compressive strength. The results of above said tests are shown in table 3.1.

3.3.2 Coarse Aggregates

These are generally the aggregates which are the residue of the 4.75mm IS sieve and passed through 75mm IS sieve. Coarse aggregates consist of crushed gravel, uncrushed gravel and partially crushed gravel. These aggregates are formed by natural disintegration of rocks or by artificial crushing of rock or gravel. As per IS: 383-1970 standards quality of aggregates shall be hard, strong, dense, durable, clear and free from veins and adherent coating; and free from

injurious amounts of disintegrated pieces, alkali, vegetable matter and other deleterious substances. As far as possible, flaky and elongated pieces should be avoided.

The coarse aggregates used in the present study, obtained locally, were a mixture of crushed stones of 20mm and 10mm sizes. The aggregates were thoroughly washed to remove dirt, dust and then dried to surface dry condition. They were then put into the oven for a period of 24 hours so as to remove the excess moisture present. The aggregates taken out were then cooled down to the room temperature to be used in the concrete mix. The specific gravity and other physical properties of coarse aggregates were calculated as per relevant codal guidelines. These values are shown in Table 3.2. The values obtained by the sieve analysis of coarse aggregate done in the laboratory for 20mm aggregate are shown in the Table 3.3 and that for the 10mm aggregate in Table 3.4.

Table 3.1 Properties of OPC 43 Grade

Characteristics	Experimental values Obtained	IS 8112:2013 Specified Values	Test Method Referred to
Specific gravity	3.04	-	IS 4031 Part 11
Standard Consistency (%)	33	-	IS 4031 Part 4
Setting time (min)			IS 4031 Part 5
Initial	40	30 (Minimum)	
Final	320	600 (Maximum)	
Compressive strength (MPa)			IS 4031 Part 6
3 days	24.5	23	
7 days	33.5	33	
28 days	44.5	43	

3.3.3 Fine Aggregates

These are generally the aggregates which passed through 4.75mm IS sieve and are residue of 75 μ m IS sieve. The fine aggregates were washed to remove silt and clay and were then oven dried for a period of 24 hours. They were then brought down to room temperature and were then used in the concrete mix. Depending upon the particle size distribution IS: 383-1970 has divided the fine aggregate into four grading zones (Grade I to IV). The fine aggregates become progressively finer from grading zone I to IV. In this experimental program fine aggregates lying in grading zone II (size less than 2.36 mm) were collected from the local supplier in Patiala.

Table 3.2 Properties of Coarse Aggregates

Characteristics	Values	
Color	Grey	
Shape	Angular	
Maximum Size	20 mm	10 mm
Specific gravity	2.61	2.68

Table 3.3 Sieve Analysis of Coarse Aggregates (20mm)

S.No.	IS Sieve	Wt. Retained(gm)	% age Retained	Cumulative % Retained	% Passing
1	80 mm	0	0	0	100
2	40 mm	0	0	0	100
3	20 mm	0	0	0	100
4	10 mm	2656	88.53	88.53	11.47
5	4.75 mm	326	10.87	99.4	0.6
6	Pan	18	0.6	Sum=187.93	0
Total		3000	FM = (187.93+500)/100 i.e. 6.87		

Table 3.4 Sieve Analysis of Coarse Aggregates (10mm)

S.No.	IS Sieve	Wt. Retained(gm)	% age Retained	Cumulative % Retained	% Passing
1	80 mm	0	0	0	100
2	40 mm	0	0	0	100
3	20 mm	0	0	0	100
4	10 mm	1246	41.53	41.53	58.47
5	4.75 mm	1634	54.47	96	4
6	Pan	120	4	Sum=137.53	0
Total		3000	FM = (137.53+500)/100 i.e. 6.37		

Physical properties and sieve analysis of fine aggregates are tested as per IS: 383-1970 and results are shown in table 3.5 and table 3.6 respectively.

Table 3.5 Physical Properties of Fine Aggregates

Characteristics	Values
Type	Natural Sand
Specific Gravity	2.46
Fineness Modulus	2.56
Grading Zone	II
Total Water Absorption (%)	0.83

Table 3.6 Sieve Analysis of Fine Aggregates

S.No.	IS-Sieve	Wt. Retained(gm)	%age Retained	%age Passing	Cumulative % Retained
1	4.75 mm	7.5	0.75	99.25	0.75
2	2.26 mm	84	8.4	90.85	9.15
3	1.18 mm	167	16.7	74.15	25.85
4	600 μ m	218	21.8	52.35	47.65
5	300 μ m	288	28.8	23.55	76.45
6	150 μ m	199.5	19.95	3.6	96.4
7	Pan	36	3.6	-	-
Total		1000	Sum		256.25
Zone II			FM = 2.56		

3.3.4 Silica Fume

Silica fume, also known as micro silica or condensed silica fume is a pozzolanic admixture. When used in concrete it will fill the void space between cement particles resulting in a more impermeable concrete. Silica fume was obtained from KGR Agro Fusions (P) Ltd., Ludhiana Punjab. The color of silica fume received was bluish grey. KGR Silica Fume consists primarily of amorphous (non-crystalline) silicon dioxide (SiO_2). The individual particles are extremely small, approximately 1/100th the size of an average cement particle. Because of its fine particles, large surface area, and the high SiO_2 content, silica fume is a very reactive pozzolan when used in concrete. The quality of silica fume is specified by ASTM C 1240. Physical and chemical properties of silica fume used as provided by KGR Agro Fusions (P) Ltd.were:

- a) **Physical properties:** The primary physical properties of silica fume are shown in Table 3.7.
- b) **Chemical properties:** Chemical composition of silica fume will mostly depend upon composition of the principle product being made by furnace. Additionally, the composition is also influenced by the furnace design; generally a furnace with a heat

system produces silica fume varies with the silica content of alloy being producer. Unlike other by-products pozzolans such as fly ash, silica fume from a single source is little or no variation in chemical composition from one day to another. In the present study the silica fume that was collected from the supplier had the chemical composition shown in Table 3.8.

Table 3.7 Physical Properties of Silica Fume (Provided by Manufacturer)

Particle size (typical)	<1 μ m
Specific surface	15000-30000 m ² /Kg
Color	Bluish grey
Bulk density	550-700 kg/m ³
Specific gravity	2.40



Fig.3.1 Densified silica fume

Table 3.8 Chemical Composition of Silica Fume (Provided by manufacturer)

SiO ₂ (%)	MgO (%)	SO ₃ (%)	H ₂ O (%)	K ₂ O (%)	Na ₂ O (%)	CaO (%)	Si (%)	Cl (%)	Fe ₂ O ₃ (%)
92.55	<1.55	<1.1	<0.4	<2.25	<1.4	<0.35	<0.5	<0.06	<2

3.3.5 Superplasticizer

To develop UHPC mixes, MASTERGLENium SKY 8866 was obtained from BASF solutions, Chandigarh. It is an admixture of a new generation based on modified *polycarboxylic ether*. The product has been primarily developed for applications in high performance concrete where the highest durability and performance is required. It is free of chloride & low alkali. It is compatible with all types of cements.

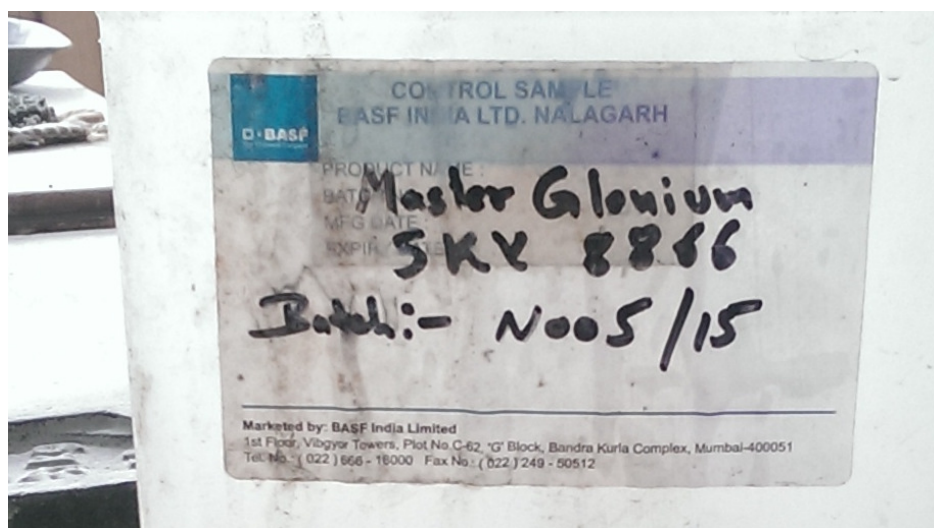


Fig.3.2 Master Glenium Sky 8866 Superplasticizer

Recommended uses:

- Production of Rheodynamic concrete
- High performance concrete for durability
- High early and ultimate strength concrete
- High workability without segregation or bleeding

- Precast & Pre-stressed concrete
- Concrete containing pozzolanas such as microsilica, GGBFS, PFA including high volume fly ash concrete

Advantages:

- Elimination of vibration and reduced labour cost in placing
- Marked increase in early & ultimate strengths
- Higher E modulus
- Improved adhesion to reinforcing and stressing steel
- Better resistance to carbonation and other aggressive atmospheric conditions
- Lower permeability - increased durability
- Reduced shrinkage and creep

The superplasticizer used during this study is specified by ASTM C494 Type F.

The physical properties of Master Glenium sky 8866 is given in the Table 3.9

Table 3.9 Physical Properties of Superplasticizer (Provided by manufacturer)

Aspect	Light Brown Liquid
Relative Density	1.08 ± 0.01 at 25°C
pH	≥6
Chloride ion content	<0.2%
Specific Gravity	1.08
Type	Polycarboxylic Ether Polymer

3.3.6 Water

Water is used as a binding agent for the concrete mix. Potable water is normally considered satisfactory for mixing and curing of concrete. In the current study accordingly potable water available in the material testing laboratory was used for making concrete. The water is relatively free from organic matter, silt, oil, sugar, chloride and acidic material as per requirements of Indian standard.

3.3.7 Reinforcing Steel

HYSD steel of grade Fe-500 D (Tata Tiscon) of 16mm, 12mm and 8mm diameters were used as reinforcement. 16mm and 12mm dia bars are used as tension reinforcement and 8mm bars are used as compression steel. 8mm diameter bars are used as shear stirrups. The mechanical properties are listed in table 3.10

Table 3.10 Mechanical Properties of Tata Tiscon 500 D (Provided by Manufacturer)

Mechanical Properties	Unit	IS:1786 Fe 500D (Not less than)	Tata Tiscon Fe 500D
Yield Stress(YS)	N/mm ²	500	540
Ultimate Tensile Strength(UTS)	N/mm ²	565	600
UTS/YS	Ratio	1.10	1.12
Elongation	%	16	18

3.3.8 Steel Fibers

The main function of steel fibers is to arrest the cracks propagation and to delay the propagation of cracks across the matrix and create a slow cracking propagation stage. This ability to arrest cracks, fiber composites possess increased extensibility and tensile strength, both at first crack and at ultimate, particular under flexural loading; and the fibers are able to hold the matrix together even after extensive cracking. The transformation from a brittle to a ductile type of material would increase substantially the energy absorption characteristics of the fiber composite and its ability to withstand repeatedly applied, shock or impact loading. Steel fibers are of different types namely straight steel fiber, crimped steel fiber and hooked ends steel fiber. In the present study crimped steel fiber and hooked steel fiber (SHAKTIMAN ® Steel Fibers) as shown in fig. 3.3 are used which were obtained from Stewols India (P) Ltd., Nagpur. The specifications of the fibers used are given in Table 3.11.



Fig.3.3 Steel Fibers

Table 3.11 Specification of Steel Fiber

Ultimate strength as per ASTM A820M	>1100MPa
Diameter	0.6 mm
Length	30 mm
Aspect ratio	50

3.3.9 Metakaoline

Metakaolin is a dehydroxylated form of the clay mineral kaolinite. Stone that are rich in kaolinite are known as china clay or kaolin, traditionally used in the manufacture of porcelain. The particle size of metakaolin is smaller than cement particles, but not as fine as silica fume. Like other pozzolans (fly ash and silica fume are two common pozzolans), metakaolin reacts with the calcium hydroxide (lime) byproducts produced during cement hydration. Calcium hydroxide accounts for up to 25% of the hydrated Portland cement, and calcium hydroxide does not contribute to the concrete's strength or durability. Metakaolin combines with the calcium hydroxide to produce additional cementing compounds, the material responsible for holding concrete together.

3.3.10 Epoxy

The success of the strengthening technique primarily depends on the performance of the epoxy resin used for bonding of UHPFRC to concrete surface. Numerous types of epoxy resins with a

wide range of mechanical properties are commercially available in the market. These epoxy resins are generally available in two parts, a resin and a hardener. The epoxy resin used in this study was Dr. Fixit 211 epoxy bonding agent. The properties of the epoxy resin used in the experimental work are given in Table 3.12.

Table 3.12 Properties of Epoxy Resin (provided by manufacturer)

Sr. no.	Physical properties	Value
1	Mix Density	1120 kg/m ³
2	Colour	Grey
3	Pot Life @ 30° C	± 30-40 mins.
4	Cure time	7 days
5	Tensile strength, 7 days	10.4 N/mm ²
6	Compressive strength, 7 days	60 N/mm ²
7	Flexural Strength	28.1 N/mm ²

3.4 Design of Concrete Mix (M25)

M25 concrete mix was designed by using the properties of cement, fine aggregate/sand and crushed stone coarse aggregates as per IS: 10262-2009 guidelines. The water/cement ratio was taken as 0.46. The mix proportions are given in Table 3.13 The cubes were prepared with this mix proportions and were tested at the age of 7days and 28 days. The compressive strength of cubes (average of 3 cubes) at 7days and 28 days was 20.9 MPa and and 34.40 MPa respectively.

Table 3.13 Mix Proportions for M25 Grade Concrete

Water	Cement	Fine Aggregate/sand	Coarse Aggregates
192 kg	417.39 kg	544.38 kg	1190 kg
0.46	1	1.30	2.85

3.5 RCC BEAM DESIGN

In the present study RCC beam is designed using M25 grade concrete and Fe 500D steel. The RCC beam is designed using limit state method and both under-reinforced and over-reinforced sections are designed. The beam is designed having 2 steel bars of 8 mm dia at compression face and 2 bars of 16 mm dia at tension face for over-reinforced section; 2 bars of 12 mm dia at tension face for under-reinforced section and the stirrups used are of 8mm diameter and at the spacing of 85 mm for under-reinforced section and at 75 mm for over-reinforced section. The dimensions of the beam are fixed to 150 x 150 mm overall. Longitudinal section and cross-section of beam is shown Figure 3.4 and Figure 3.5

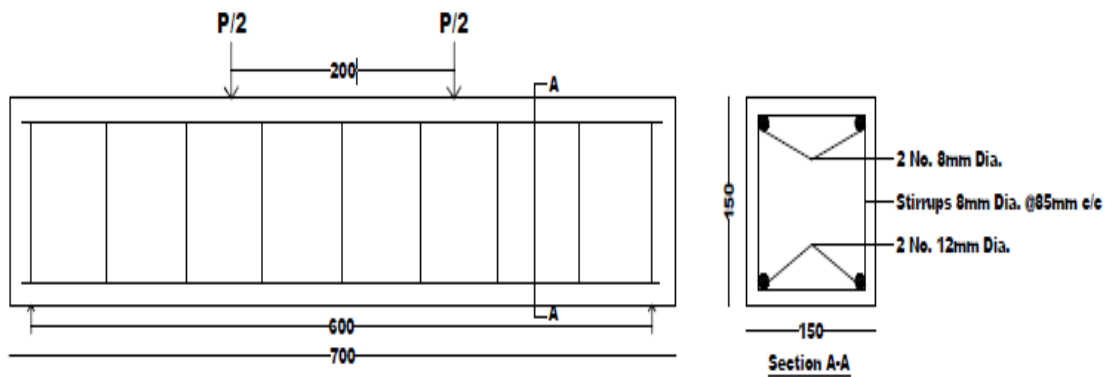


Fig. 3.4 Longitudinal and Cross-section of Under-reinforced Section Beam

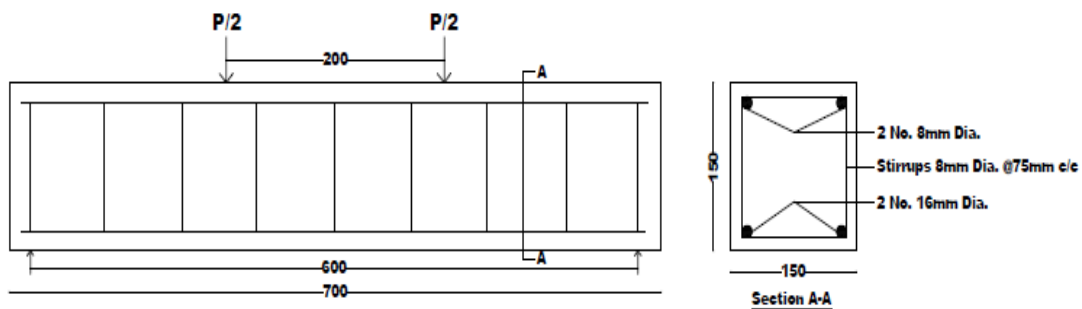


Fig. 3.5 Longitudinal and Cross-section of Over-reinforced Section Beam

3.6 CASTING OF RCC BEAMS

The beams are casted in a mould of size 150 x 150 x 700 mm. First of all the entire beam mould is oiled so that the beam can be easily removed from the mould after the desired period. Spacers of size 25 mm are used to provide uniform cover to the reinforcement. When the bars have been placed in position as per the design, concrete mix is poured in the mould and vibrations are given with the help of vibrator. The vibration is done until the mould is completely filled and there is no gap left. The beams are then removed from the mould after 24 hours. After demoulding the beams are cured for 28 days using jute bags.

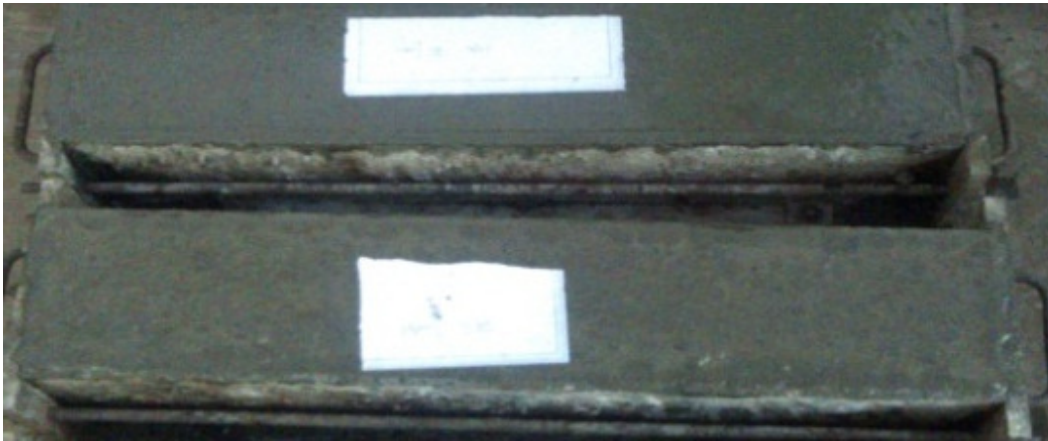


Figure 3.6 Beam specimens at the Time of Casting

3.7 DEVELOPMENT OF UHPFRC

3.7.1 Mix Proportions Used In The Present Study

UHPFRC was used for retrofitting of the beams. For the development of UHPFRC three trials were taken by varying amount of superplasticizer, metakaoline and use of different sand particle sizes to densify the matrix to achieve the target compressive strength of 120 MPa. The three trial mixes considered are shown in the Tables 3.14, 3.16 and 3.18.

As per the definition of Ultra High Strength Fiber Reinforced Concrete (UHPFRC), the coarse aggregates were removed and to densify the matrix; an initial trial mix was decided to use 2 different sizes of sand particles i.e. ($< 600\mu\text{m}$ and 1.18-2.26mm). Cement content is usually

higher in ultra high strength concretes and water-cement ratio ($w/c=0.23$) was taken. Also two different types of high strength fibers were used i.e. crimmed and hooked which increase the tensile behavior and helps in bridging the cracks. Silica-fume used in the mix design serves two purposes i.e. as filler which fills the gap within the cement particles and as a pozzolanic material which helps in reducing $CaCO_3$ which liberates in the hydration reaction of concrete.

Firstly, cubes of size 150 mm were casted using trial mix 1 and then a set of three cubes were tested after 7 and 28 days of curing. The cubes were cured in normal water and tested after 7 and 28 days of curing. The test results of same are shown in Table 3.15. From the test results and during the process of casting it is observed that cubes cannot be demoulded even after 3-4 days of casting. It is anticipated that this may be due to higher quantity of superplasticizer. Moreover, the 7 and 28 days compressive strength of cubes was also less than the targeted value of 120 MPa. Hence second trial mix was prepared by decreasing the superplasticizer quantity to achieve the targeted strength of 120 MPa.

Table 3.14 Design Mix Trial- 1

Items	Quantity(Kg/m ³)
Cement	875
Sand (<600 μ m)	270
Sand(1.18-2.26mm)	1055
Silicafume	300
Superplasticizer	45
Fibers (hooked)	1.5% by vol. of concrete
Fibers (crimmed)	0.5% by vol. of concrete
Water	200

Table 3.15 Results of Compressive Strength for Trial 1

7 day Strength	48 MPa
28 day Strength	75 MPa

A second trial mix was prepared by decreasing the quantity of superplasticizer from 45 Kg/m³ to 35 Kg/m³ keeping all other composition to be same. Similarly, cubes were casted using trial mix 2 and a set of three cubes was tested after 7 and 28 days of curing. The results of trial mix 2 after 7 and 28 days of curing are shown in table 3.17. From the results it is observed that the cubes were able to be demoulded after 24 hours so that proper curing could be done to achieve the targeted strength of 120 MPa at 28 days. This may be due to decrease in the amount of superplasticizer. But there was a slight increment in the compressive strength was observed after 28 days of curing so there was a need to further densify the matrix to achieve the targeted strength of 120 MPa. So an another trial mix was prepared which consisted of two different size range of sand particles i.e.($<300\mu\text{m}$ and $300\mu\text{m}$ -1.18 mm).

Table 3.16 Design Mix Trial - 2

Items	Quantity(Kg/m³)
Cement	875
Sand ($<600\mu\text{m}$)	270
Sand(1.18-2.26mm)	1055
Silica fume	300
Superplasticizer	35
Fibers (hooked)	1.5% of vol. of concrete
Fibers (crimped)	0.5% of vol. of concrete
Water	200

Table 3.17 Results of Compressive Strength for Trial 2

7 day Strength	52 MPa
28 day Strength	79 MPa

Thus maximum particle size reduced to 1.18 mm from 2.26 mm resulted in densification of the matrix. Also the quantity of cement increased in this mix keeping the water-cement to be same and metakaoline as a pozzolanic material added along with silica fume to increase the strength. Also, metakaoline has higher particle size than silica fume but smaller than sand particle sizes used in the mix trial so the addition of metakaoline helped to further densification. But metakaoline due to its higher surface area, needed more water resulted in decrease in workability so amount of superplasticizer was increased to 45 Kg/m³. The trial mix 3 is tabulated in table 3.18.

Table 3.18 Design Mix Trial -3

Items	Quantity(Kg/m³)
Cement	1000
Sand (<300µm)	270
Sand(300µm-1.18mm)	750
Silica fume	150
Metakaoline	150
Superplasticizer	45
Fibers (hooked)	1.5% of vol. of concrete
Fibers (crimped)	0.5% of vol. of concrete
Water	230

In the trial mix 3 as the quantity of superplasticizer increased but due to higher cement content and presence of metakaoline, the cubes were able to be demoulded after 24 hours. A set of three cubes of size 150 mm were tested after a curing period of 7 and 28 days. The results obtained are shown below in Table 3.19. From the results it is observed that the compressive strength of cubes at 7 and 28 days are in the range of the targeted strength of 120 MPa. The compressive strength after 28 days of curing was observed to be 118 MPa which is in the range of the targeted strength. So retrofitting of beams was done using trial mix 3.

Table 3.19 Results of Compressive Strength for Trial 3

7 day Strength	75.8 MPa
28 day Strength	118 MPa

3.8 TESTING ARRANGEMENT

The beams were tested after 28 days from the date of casting. All the beam specimens were tested as simply supported beams subject to two-point loading with loads applied at distance $L/3$ from the each end; where L is the effective span i.e. 600 mm. The shear span to effective depth ratio (a/d) was equal to 1.6. This load case was chosen because it gives constant moment and zero shear in the section between the loads, and constant maximum shear force between support and load. The moment was linearly varying between supports and load. The testing procedure for the entire beam specimens was same. The tests were performed using the setup as shown in the Figure 3.7.



Figure 3.7 Experimental Setup Used For Testing

The testing of beams was carried out in Universal testing machine (UTM) of 1000kN load capacity. The tests were carried out using displacement control, and the applied loads were monitored through a high accuracy load cell with a load sensitivity of 0.001kN. The load was applied at the rate of 4 kN/min. One dial gauge was placed just below the center of the beam, i.e. at $L/2$ distance to measure the deflection. The reading was taken after regular intervals. During loading, the specimen was visually inspected and cracks were marked. For the clear visibility of cracks, the surfaces were painted white prior to testing.

3.9 RETROFITTING OF BEAMS

All beams are initially stressed to 60% of its load carrying capacity and then retrofitted with ultra high performance fiber reinforced concrete using Epoxy as bonding agent between old concrete and UHPFRC. Three different retrofitting techniques adopted for retrofitting are:

- (i) Side Face Retrofitting
- (ii) U-Type Retrofitting
- (iii) External Side Face Jacketing

Retrofitting of beams is done in three stages- chipping off side cover to expose rings, epoxy layer applied and then finally retrofitting with new UHPFRC layer. The three different techniques are discussed below.

1. Side Face Retrofitting

In this technique retrofitting is done in three stages. In stage 1, the damaged concrete cover from the sides of the beam was removed in the shear span. In Fig. 3.8 the chipped off portion on the either side until the ring is exposed is shown. In stage 2, the exposed beam surface was cleaned and epoxy is applied on the chipped off portion as shown in the Fig. 3.9. The epoxy layer is used as the bonding agent between the old chipped off portion and the new UHPFRC. In stage 3, the resulting beam is then retrofitted with UHPFRC. The section is retrofitted till it is restored to its original dimensions of 150*150 mm with UHPFRC as shown in fig. 3.10.



Fig. 3.8 Side Face Chipped off Beam (Stage 1)



Fig. 3.9 Epoxy Layer Applied After Chipping off (Stage 2)



Fig. 3.10 Beam Retrofitted with UHPFRC (Stage 3)

2. U-type Retrofitting

In this technique the damaged concrete cover from the side faces and the bottom face of the beam was removed in the shear span. The portion is chipped off until the ring is exposed. The

resulting beam is then retrofitted with UHPFRC after applying a layer of epoxy resin as a bonding agent between the old concrete surface and the new UHPFRC. The first two stages are same as for the Side Face retrofitting so only final stage 3 is shown in fig. 3.11. The section is retrofitted till it is restored to its original dimensions of 150*150 mm.



Fig. 3.11 Beam Retrofitted in U-Type Configuration (Stage 3)

3. External Side Face Strengthening

In this technique the side faces are externally strengthened with 25 mm layer of UHPFRC over the existing section of 150*150 mm. Initially the side faces are roughened by manually making small indentations. A layer of epoxy resin is then applied to the roughened surface to act as a bonding agent with the old concrete and the new UHPFRC. Then finally a 25 mm thick layer of UHPFRC is applied to the epoxy treated surface as shown in fig. 3.12.



Fig. 3.12 Beam Strengthened with 25 mm Thick UHPFRC layer

CHAPTER 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION

The results of experimental work carried out as discussed in previous chapter are presented and discussed in this chapter. In the present study, the effect on the behavior of initially stressed shear deficient beams retrofitted using UHPFRC is studied. For the purpose the beams were firstly stressed to pre-decided stress level of 60% of the ultimate load carrying capacity of control beam and then were retrofitted by three different methods using UHPFRC as discussed in previous chapter section. The load deflection graphs of both controlled and initially stressed retrofitted beams were plotted and a comparative study on the strength variation of retrofitted and control beams is done with above parameters and is presented in the subsequent section.

4.2 TESTING METHODOLOGY

Firstly control beams were tested to failure and the data corresponding to it was recorded. Then two beams each of the three different retrofitted techniques were stressed to 60% of the ultimate load carrying capacity for both under-reinforced (U/R) and over-reinforced (O/R) section beams. The ultimate failure load was calculated from the load deflection curve of the control beams tested in two point loading. Then the retrofitting of beams was done by three different methods as discussed in previous chapter with UHPFRC using epoxy resin as bonding agent between the old concrete surface and the new UHPFRC. The retrofitted beams were cured with jute bags for 28 days and tested again under two point loading and the corresponding results are recorded and shown in the sections to follow.

The beam designations for all the retrofitted and control under-reinforced and over-reinforced beams used are presented in Table 4.1.

Table 4.1 Beam Designations

Particulars	Designation
Control Beam(CB-U/R)	Under-reinforced section control beam
Control Beam(CB-O/R)	Over-reinforced section control beam
R1, R6 (U/R SIDE FACE)	Under-reinforced section beams retrofitted by replacing side face cover in shear span using UHPFRC
R2, R3 (U/R U-TYPE)	Under-reinforced section beams retrofitted by replacing side face and bottom cover in shear span using UHPFRC
R8, R11 (U/R 25 mm THICK JACKETING)	Under-reinforced section beams strengthened side face externally with 25 mm thick layer of UHPFRC.
R4, R9(O/R SIDE FACE)	Over-reinforced section beams retrofitted by replacing side face cover in shear span using UHPFRC
R5, R10 (O/R U-TYPE)	Over-reinforced section beams retrofitted by replacing side face and bottom cover in shear span using UHPFRC
R7, R12 (O/R 25 mm THICK JACKETING)	Over-reinforced section beams strengthened Side face externally with 25 mm thick layer of UHPFRC.

4.3 CONTROL BEAMS

4.3.1 Introduction

Two beams each of Under-reinforced (U/R) and Over-reinforced (O/R) section were tested as control beams under two point loading system. Mid Span deflection and crack patterns were observed during testing. Load deflection curves were plotted using the data and from the curve,

ultimate load, yield load, deflection at the ultimate and yield load, initial stiffness, ductility and energy absorption values were calculated.

4.3.2 U/R Control Beam Section (CB-U/R)

After 28 days of curing, two control beams (CB1 & CB2) were tested under the two point loading system as discussed in section 3.8. The beams were designed with shear stress (τ_v) exceeding the maximum shear capacity of the concrete ($\tau_{c,max}$) and the distance between two loads was so selected that a/d ratio is around 1.67 so that the beam fails in shear. The load at centre and mid-span deflection were recorded for both the control beams. These values are presented in Table 4.2 and the flexural behavior of both the beams is graphically represented in Fig. 4.2. The results of the beams CB1 and CB2 were then averaged to obtain a single set of results (**designated as beam CB-U/R**) as shown in Table 4.2 and Fig. 4.2. The behavior of the average beam specimen CB-U/R was then studied in detail and used for comparison with the retrofitted beams, as illustrated in succeeding paragraphs.

The first flexural crack in beam CB was observed at a load of 32 kN, and it occurred between the left point load and the center of the beam. As the load increased the cracks started to propagate upward. The load was increased in the intervals and the deflection at the center was noticed with the help of the dial gauge. The results of the load and corresponding deflection so obtained are presented in the table 4.2. and the crack pattern are shown in Fig.4.1. At a load of 70 kN load, shear cracks started appear on the beam in shear span. On increasing the load further, shear cracks started to widen and propagate upward toward the point at which loads are applied on the beam and finally beams failed in shear with maximum deflection at the center 3.422 mm.

4.3.3 O/R Control Beam Section (CB-O/R)

Similarly to the U/R sections, two O/R section control beams (CB1 & CB2) were tested under the two point loading system as discussed in section 3.8. The beams are deficient in shear and the distance between two loads is so selected that a/d ratio is around 1.67 so that the beam fails in shear. The load at centre and mid-span deflection were recorded for both the control beams. These values are presented in Table 4.3 and the flexural behavior of both the beams is graphically represented in Fig. 4.4. The results of the beams CB1 and CB2 were then averaged to obtain a single set of results (**designated as beam CB-O/R**) as shown in Table 4.3 and Fig. 4.4.

The behavior of the average beam specimen CB-O/R was then studied in detail and used for comparison with the retrofitted beams, as illustrated in succeeding paragraphs.

Table 4.2 Detail of Load vs Mid Span Deflection of U/R Control beam Section (CB) U/R

Load (kN)	Displacement for Beam CB1 (mm)	Displacement for Beam CB2 (mm)	Displacement for Beam CB-U/R (mm)
0	0.000	0.000	0.000
1	0.009	0.200	0.014
2	0.012	0.029	0.021
3	0.018	0.038	0.024
4	0.025	0.047	0.037
5	0.041	0.062	0.051
6	0.056	0.079	0.069
7	0.072	0.093	0.081
8	0.084	0.112	0.096
9	0.098	0.134	0.115
10	0.108	0.156	0.132
15	0.215	0.259	0.237
20	0.318	0.382	0.348
25	0.412	0.495	0.457

Load (kN)	Displacement for Beam CB1 (mm)	Displacement for Beam CB2 (mm)	Displacement for Beam CB (mm)
30	0.520	0.612	0.569
35	0.653	0.691	0.672
40	0.718	0.815	0.767
45	0.826	0.897	0.862
50	0.907	1.029	0.968
60	1.115	1.192	1.152
70	1.244	1.458	1.357
80	1.422	1.683	1.592
90	1.617	1.977	1.757
100	1.845	2.256	1.959
110	2.285	2.548	2.452
120	2.592	3.322	2.957
121	2.710	3.520	3.110
123	2.972	-	3.422
125	3.224	-	-

The first flexural crack was observed at a load of 48 kN, and it occurred between the right point load and the center of the beam. As the load increased the cracks started to propagate upwards. The load was increased in the intervals and the deflection at the center was noticed with the help of the dial gauge. The results of the load and corresponding deflection so obtained are presented

in the table 4.3 and crack pattern are shown in Fig. 4.3. At a load of 84kN load, shear cracks started appear on the beam in shear span. On increasing the load further, shear cracks started to widen and propagate upward towards the point at which loads are applied on the beam. The beams finally failed in shear with maximum deflection at the center 4.525 mm.

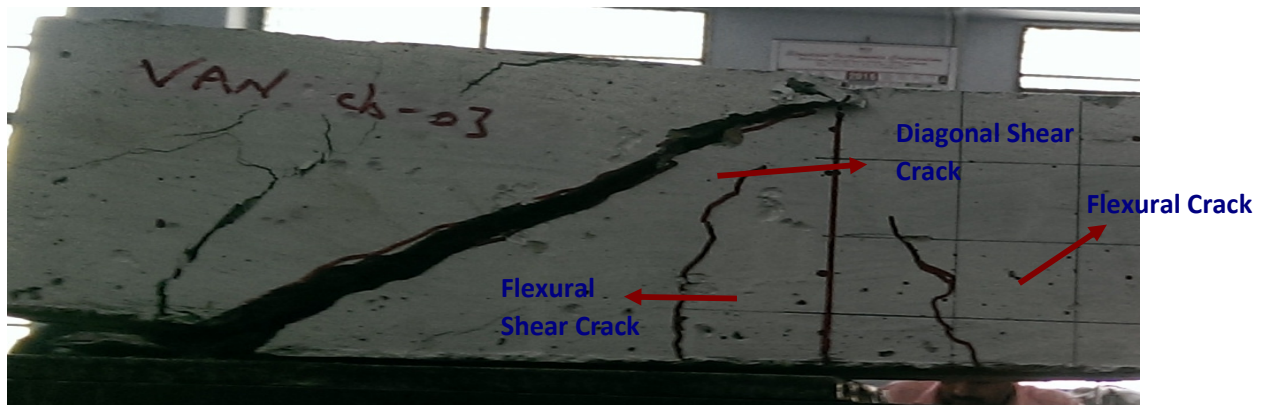


Fig. 4.1 Crack Pattern of Control Beam (CB) U/R at Failure

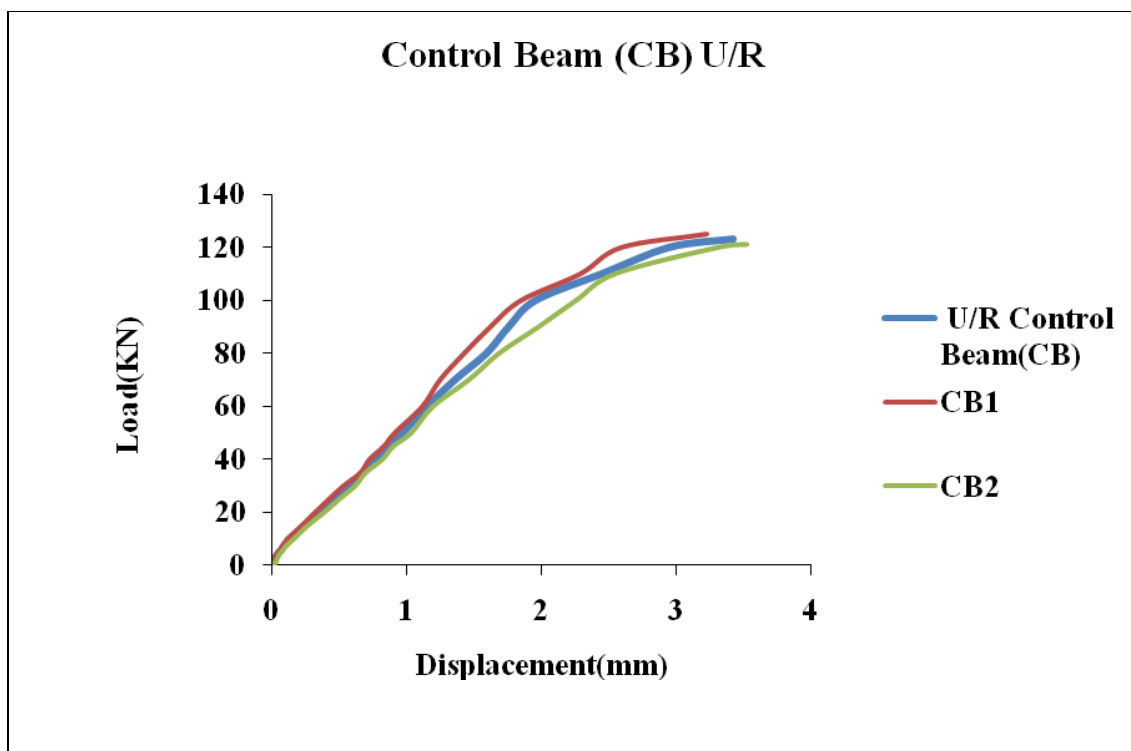


Fig. 4.2 Load v/s Mid Span Deflection of U/R Control Beam (CB) U/R

Table 4.3 Detail of Load vs Mid Span Deflection of O/R Control beam (CB O/R)

Load (kN)	Displacement for Beam CB1 (mm)	Displacement for Beam CB2 (mm)	Displacement for Beam CB-O/R (mm)
0	0.000	0.000	0.000
2	0.015	0.006	0.000
4	0.028	0.015	0.020
6	0.046	0.035	0.040
8	0.076	0.063	0.070
10	0.170	0.070	0.120
15	0.285	0.135	0.210
20	0.390	0.220	0.300
25	0.490	0.320	0.390
30	0.571	0.409	0.490
40	0.751	0.629	0.690
50	0.935	0.805	0.870
60	1.190	0.970	1.080

Load (kN)	Displacement (mm)	Load (kN)	Displacement (mm)
70	1.487	1.195	1.360
80	1.798	1.432	1.590
90	2.178	1.842	2.050
95	2.447	2.113	2.380
100	2.649	2.245	2.550
105	2.752	2.488	2.650
110	2.946	2.734	2.840
115	3.104	2.956	3.030
120	3.289	3.255	3.255
125	3.497	3.433	3.465
130	3.826	3.664	3.745
135	4.215	3.895	4.105
140	4.566	4.124	4.617
143.5	4.812	4.284	-
146.5	-	4.422	-

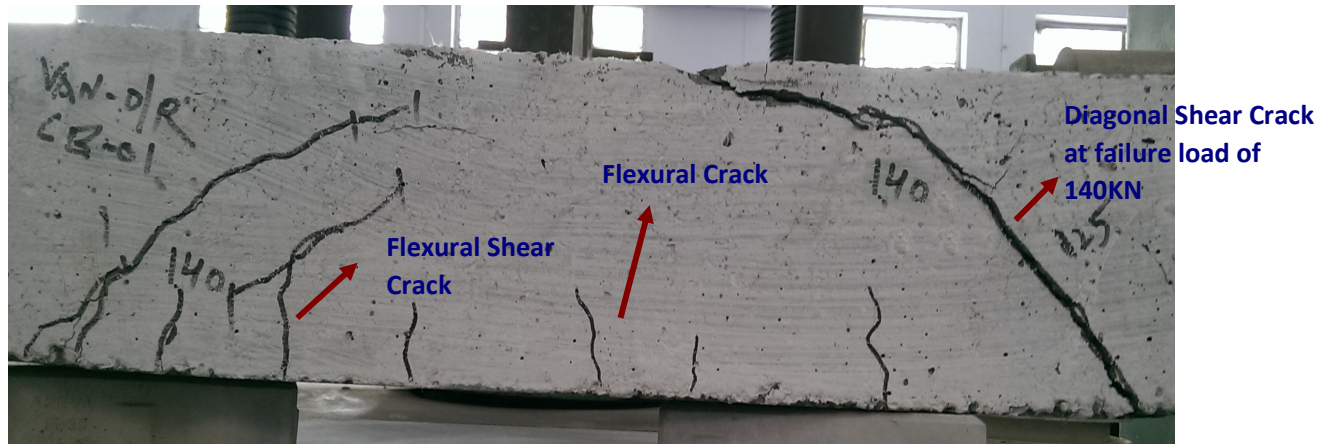


Fig. 4.3 Crack Pattern of Control Beam (CB) O/R at Failure

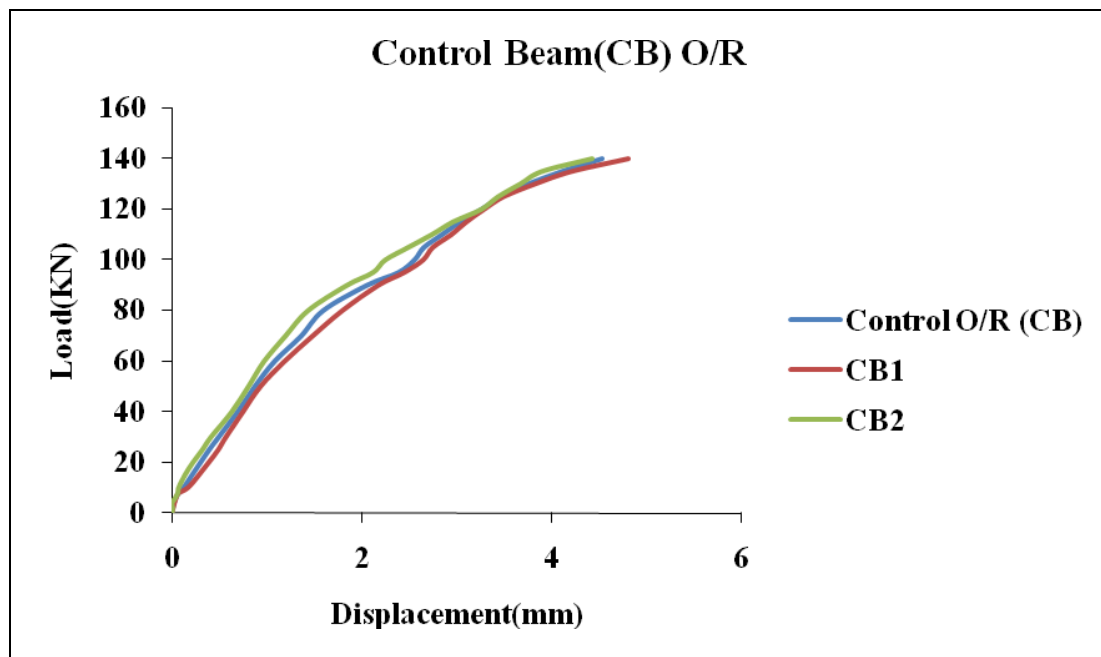


Fig. 4.4 Load v/s Mid Span Deflection of Control Beam (CB) O/R

4.4 EFFECT OF RETROFITTING ON STRENGTH OF BEAMS

The remaining beams were initially stressed to 60 % of their corresponding ultimate load i.e.73.8 kN for U/R section and 84 kN for O/R section beams and were then retrofitted using UHPFRC

by different techniques as discussed in previous chapter. The results of same are present in subsequent section.

4.4.1 Effect of Retrofitting on U/R section Beams Retrofitted by Replacement of Side Face Damaged Concrete Cover in the Shear Span with UHPFRC (Side Face)

A set of initially stressed two beams for both U/R section were retrofitted by replacing damaged concrete cover from the sides in the shear span with new UHPFRC using epoxy resin as bonding agent between old concrete surface and the new UHPFRC. The load at centre and mid-span deflection were recorded for both the beams. These values are presented in Table 4.4 and crack pattern for both beams are presented in Fig. 4.5 & 4.7. The results of the beams R1 and R6 were then averaged to obtain a single set of results (**designated as beam SF-U/R**) as shown in Table 4.4 and Fig. 4.6. The behavior of the average beam specimen SF-U/R was then studied in detail and used for comparison with the average control beam CB-U/R, as illustrated in succeeding paragraph.

It is observed that the retrofitting leads to increase in the load carrying capacity from 123 kN (Control Beam) to 147.9 kN whereas the maximum deflection at the center of the beam corresponding to the failure load of 147.9 kN is 4.71 mm as compared to 3.422 mm for the control beam at 123 kN. First crack load was observed at the load of 36.75 kN as compared to 32 kN for control beam.

Retrofitting of beam (SF-U/R) resulted in 20.24% improvement in load carrying capacity. The possible reason for the above results are (a) UHPFRC has higher compressive strength as compared to control concrete (118 MPa vs 25 MPa) which improve the load carrying capacity of the retrofitted beam. In addition, the higher modulus of elasticity of UHPFRC confined the initial damaged beam from the side faces which results in higher load carrying capacity of the retrofitted beam. (b) The hybrid fibers able to retard and delay the crack propagation so that the load carrying capacity of side face retrofitted initially stressed beam is increased. (c) Because of no debonding observed it can be concluded that the retrofitted material i.e. UHPFRC is able to improve the performance of initially stressed retrofitted beam as higher bond strength of UHPC was reported by **Tayeh et al. (2012)**

Table 4.4 Detail of Load vs Mid Span Deflection of U/R beam Retrofitted in Side Face

Load (kN)	Displacement for Beam R1 (mm)	Displacement for Beam R6 (mm)	Displacement for Beam SF-U/R (mm)
0	0.000	0.000	0.000
2	0.000	0.0000	0.000
3	0.000	0.002	0.001
4	0.000	0.004	0.002
6	0.004	0.008	0.006
8	0.015	0.018	0.016
11.5	0.050	0.060	0.055
14	0.110	0.106	0.108
17	0.160	0.158	0.159
20	0.252	0.241	0.246
22.5	0.310	0.296	0.303
25	0.360	0.354	0.357
28	0.425	0.415	0.420
33.5	0.530	0.518	0.524
39	0.650	0.632	0.641
44.5	0.720	0.738	0.729
50.5	0.860	0.839	0.849
56	1.012	0.970	0.991
67	1.138	1.188	1.163

Load (kN)	Displacement for Beam R1 (mm)	Displacement for Beam R6 (mm)	Displacement for Beam SF-U/R (mm)
78.5	1.345	1.394	1.369
89.5	1.545	1.610	1.577
100.8	1.864	1.880	1.872
112	2.256	2.154	2.205
117	2.425	2.312	2.368
123	2.756	2.640	2.698
128	2.926	2.990	2.958
131	3.228	3.204	3.216
134	3.750	3.518	3.634
140	4.206	3.976	4.091
144.5	4.550	4.280	4.415
146.8	4.639	4.670	4.654
147.9	4.716	-	4.710
149	4.750	-	-

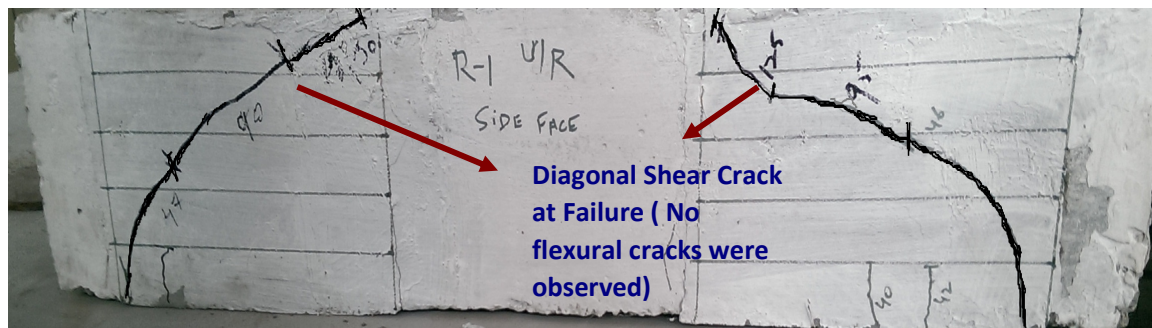


Fig.5 Crack Pattern of Beam R1 at Failure

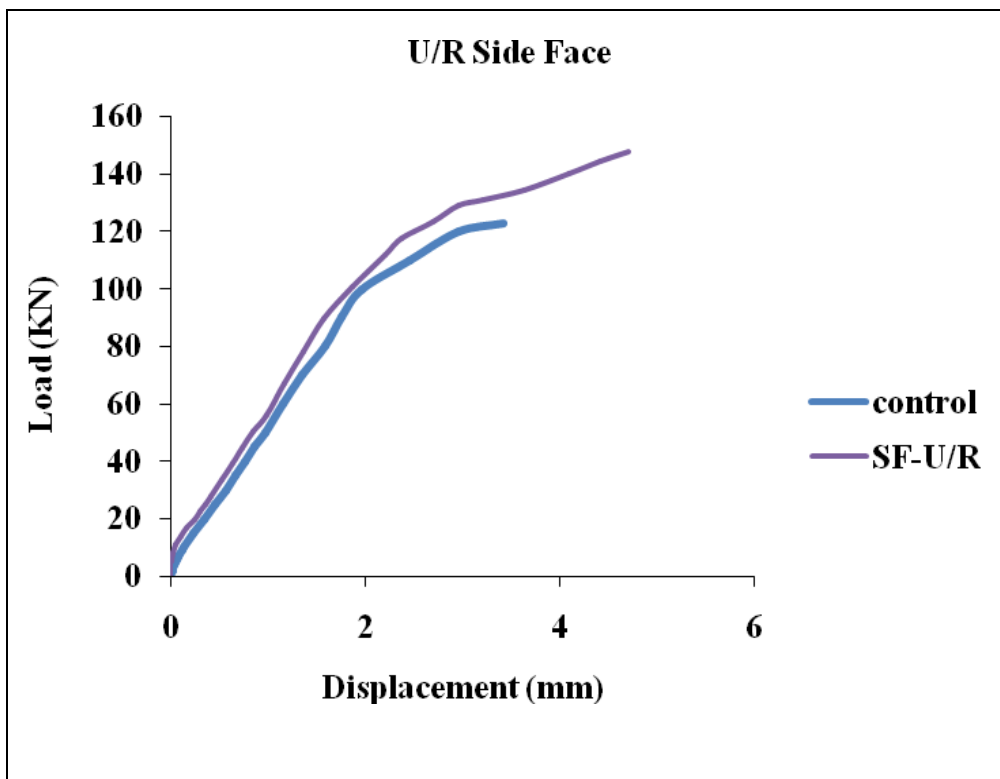


Fig. 4.6 Load v/s Mid Span Deflection of Beam SF-U/R

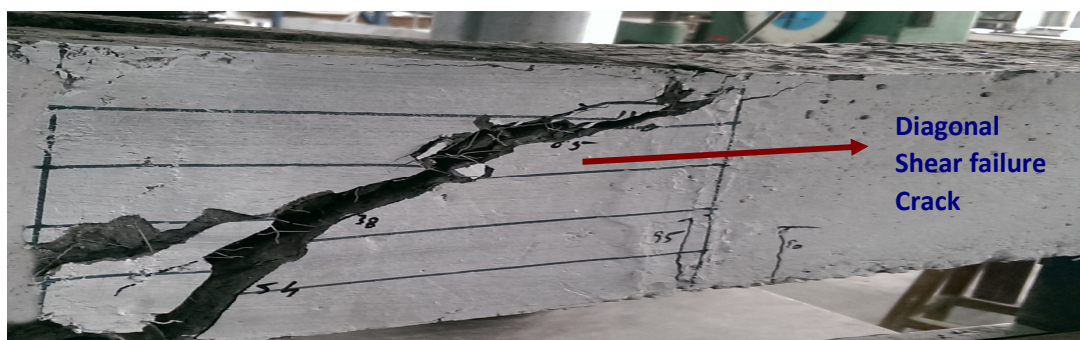


Fig. 4.7 Crack Pattern of Beam R6 at Failure

4.4.2 Effect of Retrofitting on U/R section Beams Retrofitted by Replacement of Side Face and Bottom Face Damaged Concrete Cover in the Shear Span with UHPFRC (U-Type)

A set of initially stressed two beams for both U/R section were retrofitted by replacing damaged concrete cover from the sides and bottom in the shear span with new UHPFRC using epoxy resin as bonding agent between old concrete surface and the new UHPFRC. The load at centre and mid-span deflection were recorded for both the beams. These values are presented in Table 4.5 and crack pattern for both beams are presented in Fig. 4.9 & 4.10. The results of the beams R2 and R3 were then averaged to obtain a single set of results (**designated as beam UT-U/R**) as shown in Table 4.5 and Fig. 4.8. The behavior of the average beam specimen UT-U/R was then studied in detail and used for comparison with the average control beam CB-U/R, as illustrated in succeeding paragraph.

It is observed that the retrofitting leads to increase in the load carrying capacity from 123 kN (Control Beam) to 157.9 kN whereas the maximum deflection at the center of the beam corresponding to the failure load of 157.9 kN is 8.424 mm as compared to 3.422 mm for the control beam at 123 kN.. First crack load was observed to be 38.25 kN as compared to 32 kN for the control beam. From the load-deflection curve, yield load was found to be 121.6 kN and corresponding deflection at the yield load was 3.475 mm.

The improvement in load carrying capacity of U-type retrofitted beam was observed to be highest 28.37% for beam UT-U/R. The possible reasons for the above results are: (a) The confined area is maximum in this case as beams are retrofitted in U-type i.e. along the side faces and the bottom of the beams. The higher modulus of elasticity of UHPFRC confined the initial damaged beam from the side faces and the bottom which results in higher load carrying capacity of the retrofitted beam. However, the bond failure was observed between the old concrete and the UHPFRC at the interface between the two concretes. The crack propagated towards the compression side of the beam. The localization of the crack was observed till the ultimate failure of the specimen. (b) The hybrid fibers are able to retard and delay the crack propagation so that the load carrying capacity of side face retrofitted initially stressed beam is increased.

Table 4.5 Detail of Load vs Mid Span Deflection of U/R Beam Retrofitted in U-Type

Load (kN)	Displacement for Beam R3 (mm)	Displacement for Beam R2(mm)	Displacement for Beam UT-U/R (mm)
0	0.000	0.000	0.000
3	0.009	0.005	0.007
6	0.051	0.038	0.044
9	0.099	0.086	0.093
13	0.145	0.136	0.141
16	0.205	0.196	0.200
19	0.271	0.254	0.263
22	0.341	0.328	0.335
25.5	0.433	0.395	0.414
28.5	0.505	0.485	0.495
32	0.569	0.538	0.554
38.5	0.713	0.684	0.699
45	0.865	0.775	0.820
51	1.009	0.981	0.995
58	1.169	1.089	1.129
64	1.315	1.216	1.266
77	1.601	1.545	1.573
90	1.921	1.865	1.893
102.5	2.425	2.378	2.402

Load (kN)	Displacement for Beam R3 (mm)	Displacement for Beam R2 (mm)	Displacement for Beam UT-U/R (mm)
115	2.968	2.820	2.894
121.5	3.540	3.410	3.475
128	4.102	3.980	4.041
134.5	4.654	4.560	4.607
140.8	4.548	5.350	5.404
150.5	7.230	6.980	7.105
156.8	7.608	8.390	7.814
157.9	8.126	-	8.424
159	8.458	-	-

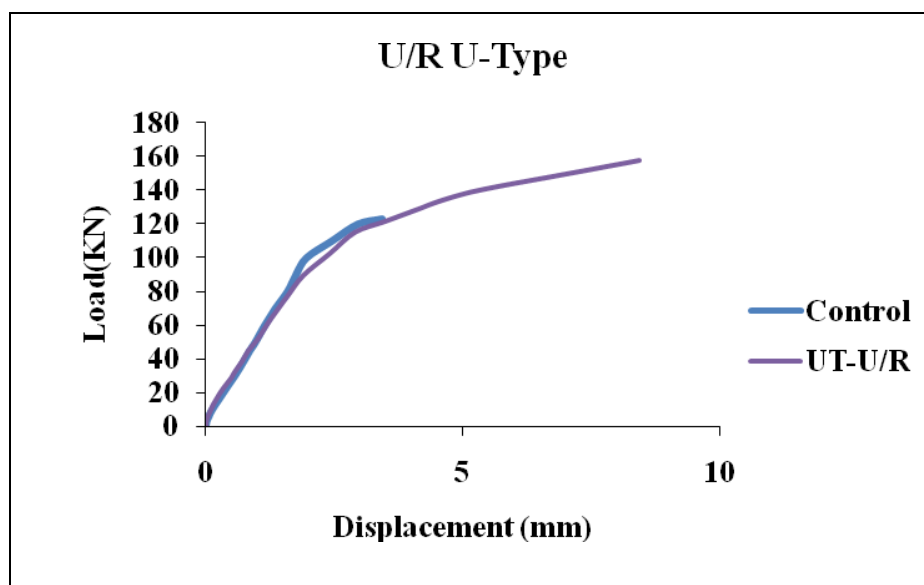


Fig. 4.8 Load v/s Mid Span Deflection of Beam UT-U/R

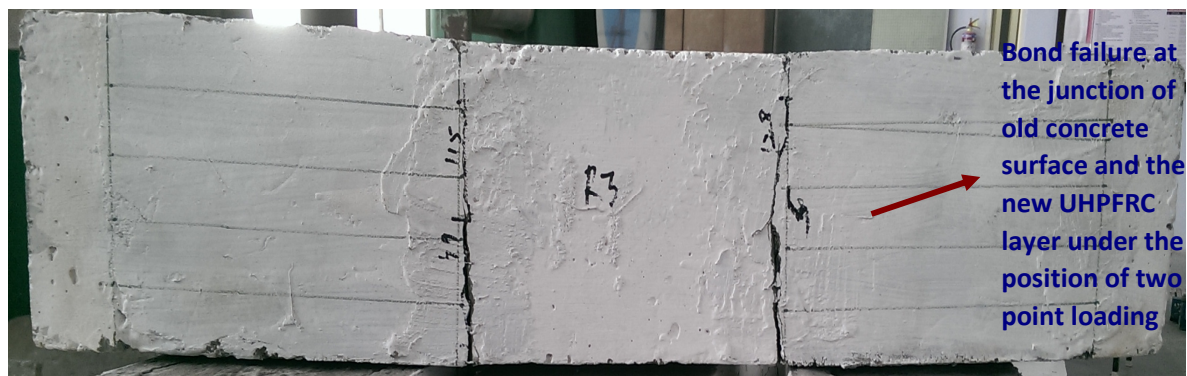


Fig. 4.9 Crack Pattern of Beam R3 at Failure

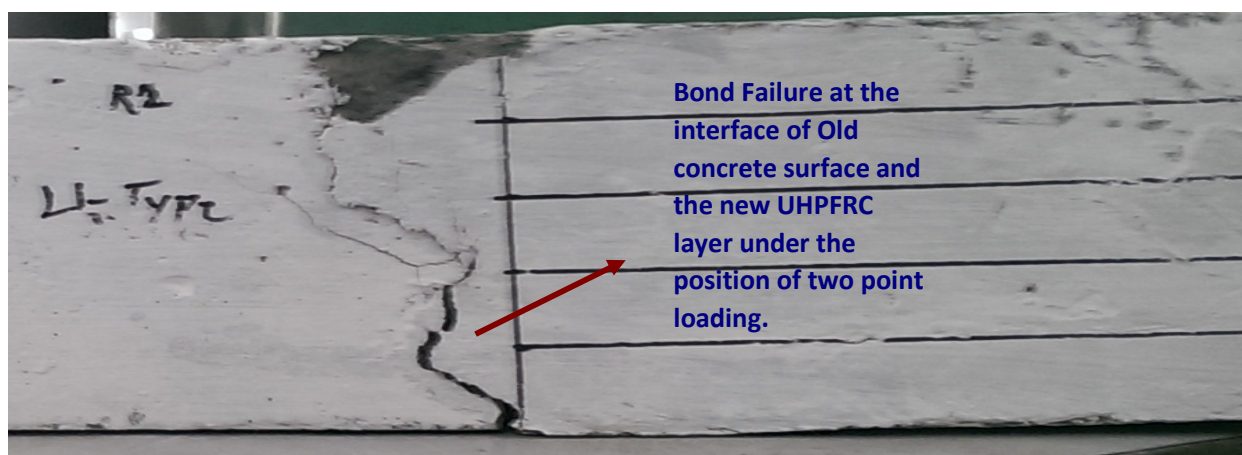


Fig. 4.10 Crack Pattern of Beam R2 at Failure

4.4.3 Effect of Retrofitting on U/R section Beams Retrofitted by External Jacketing 25 mm thick layer of UHPFRC in the Shear Span

When beams are strengthened with 25 mm thick layer of UHPFRC along side faces of shear span then due to the increased area (200*150 mm) to resist shear, different behavior is observed as compared to other two techniques discussed above. The load at centre and mid-span deflection were recorded for both the beams. These values are presented in Table 4.6 and crack pattern for both beams are presented in Fig. 4.11 & 4.12. The results of the beams R8 and R11 were then averaged to obtain a single set of results (**designated as beam EJ-U/R**) as shown in Table 4.6

and Fig. 4.13. The behavior of the average beam specimen EJ-U/R was then studied in detail and used for comparison with the average control beam CB-U/R, as illustrated in succeeding paragraph.

Initially, the displacement is observed at the center of the beam is quite less as compared to the control beam and the load carrying capacity also increased up to a certain load of 115.75 kN. As the load is increased further, the sudden failure of the beam is observed due to the **debonding** of the 25 mm thick layer of UHPFRC from the surface of the RC beam (old concrete). The maximum deflection at the center of the beam at failure of the beam is 1.87 mm. First crack load was observed to occur at load 39.25 kN. Failure of beams is due to debonding of external new UHPFRC layer from the surface of old concrete. So yield load was taken to be 115.75 kN and corresponding deflection at the yield load was 1.87 mm.



New UHPFRC layer
debonding from the
side face surface of old

Fig. 4.11 Crack Pattern of Beam R8



New UHPFRC layer
debonding from the
side face surface of old
concrete surface

Fig. 4.12 Crack Pattern of Beam R11

Table 4.6 Detail of Load vs Mid Span Deflection of U/R beam Retrofitted by External Jacketing

Load (kN)	Displacement for Beam R8 (mm)	Displacement for Beam R11(mm)	Displacement for Beam EJ-U/R (mm)
0	0.000	0.000	0.000
2.5	0.030	0.000	0.015
5	0.045	0.028	0.0365
7.5	0.091	0.070	0.080
10	0.140	0.128	0.134
13	0.180	0.175	0.1775
15.5	0.225	0.218	0.2215
18	0.260	0.254	0.257
21	0.300	0.289	0.294
23.5	0.340	0.334	0.337
26	0.380	0.375	0.377
31	0.470	0.460	0.465
36.5	0.550	0.575	0.562
41.5	0.620	0.634	0.627
47	0.700	0.708	0.704
52	0.780	0.778	0.779
58.5	0.865	0.884	0.874
65	0.940	0.956	0.948

Load (kN)	Displacement for Beam R8 (mm)	Displacement for Beam R11 (mm)	Displacement for Beam EJ-U/R (mm)
71.5	1.030	1.120	1.075
78	1.110	1.216	1.160
84.5	1.201	1.342	1.250
91	1.300	1.390	1.345
97.5	1.460	1.492	1.475
104	1.580	1.614	1.595
110.5	1.700	1.818	1.755
114.5	1.786	1.890	1.847
115.75	1.828	-	1.87
117	1.85	-	-

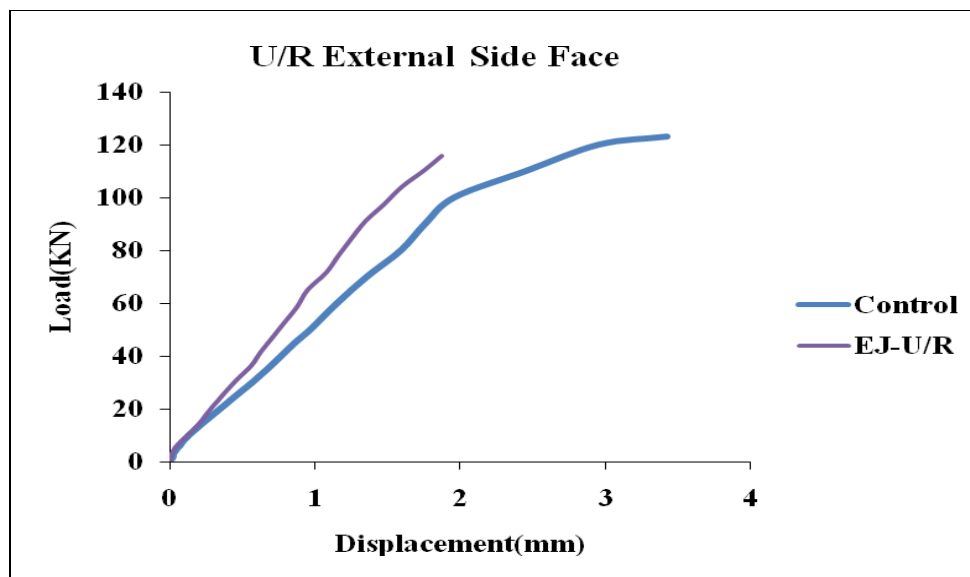


Fig. 4.13 Load v/s Mid Span Deflection of Beam EJ-U/R

4.4.4 Effect of Retrofitting on O/R section Beams Retrofitted by Replacement of Side Face Damaged Concrete Cover in the Shear Span with UHPFRC (Side Face)

A set of initially stressed two beams for both U/R section were retrofitted by replacing damaged concrete cover from the sides in the shear span with new UHPFRC using epoxy resin as bonding agent between old concrete surface and the new UHPFRC. The load at centre and mid-span deflection were recorded for both the beams. These values are presented in Table 4.7 and crack pattern for both beams are presented in Fig. 4.15 & 4.16. The results of the beams R4 and R9 were then averaged to obtain a single set of results (**designated as beam SF-O/R**) as shown in Table 4.7 and Fig. 4.14. The behavior of the average beam specimen SF-O/R was then studied in detail and used for comparison with the average control beam CB-O/R, as illustrated in succeeding paragraph.

It is observed that the retrofitting leads to increase in the load carrying capacity from 140 kN (Control Beam) to 160.75 kN whereas the maximum deflection at the center of the beam corresponding to the failure load of 160.75 kN is 5.331 mm as compared to 4.525 mm for the control beam at 140 kN. First crack load was observed at load 55.85 kN.

For the over-reinforced section, beams failed in shear before the steel yields. So for over-reinforced beams ultimate failure load was taken as the yield load. From the load-deflection curve, yield load was found to be 160.75 kN and corresponding deflection at the yield load was 5.331 mm.

Retrofitting of beam (SF-O/R) resulted in 14.82% improvement in load carrying capacity. The possible reason for the above results are (a) UHPFRC has higher compressive strength as compared to control concrete (118 MPa vs 25 MPa) which improve the load carrying capacity of the retrofitted beam. In addition, the higher modulus of elasticity of UHPFRC confined the initial damaged beam from the side faces which results in higher load carrying capacity of the retrofitted beam. (b) The hybrid fibers able to retard and delay the crack propagation so that the load carrying capacity of side face retrofitted initially stressed beam is increased. (c) Because of no debonding observed it can be concluded that the retrofitted material i.e. UHPFRC is able to improve the performance of initially stressed retrofitted beam as higher bond strength of UHPC as was reported by **Tayeh et al. (2012)**.

Table 4.7 Detail of Load vs Mid Span Deflection of U/R beam Retrofitted in Side Face

Load (kN)	Displacement for Beam R4 (mm)	Displacement for Beam R9(mm)	Displacement for Beam SF-O/R (mm)
0	0.000	0.000	0.000
2.5	0.020	0.010	0.015
5	0.050	0.040	0.045
7.5	0.088	0.075	0.0815
10	0.120	0.115	0.117
12.5	0.155	0.145	0.150
15	0.192	0.186	0.189
17.5	0.230	0.25	0.240
20	0.266	0.275	0.270
22.5	0.302	0.315	0.308
25	0.338	0.345	0.341
30	0.410	0.395	0.402
35	0.484	0.490	0.487
40	0.590	0.578	0.584
45	0.682	0.63	0.656
50	0.782	0.715	0.748
60	0.942	0.884	0.913
70	1.128	1.156	1.142
80	1.334	1.384	1.359
90	1.546	1.548	1.547
95	1.690	1.710	1.700
100	1.980	1.920	1.950
120	2.716	2.760	2.738

Load (kN)	Displacement for Beam R4 (mm)	Displacement for Beam R9 (mm)	Displacement for Beam SF-O/R (mm)
125	2.952	2.965	2.958
131.25	3.186	3.194	3.19
137.5	3.382	3.365	3.373
143.75	3.792	3.785	3.788
150	4.080	3.990	4.035
154.25	4.476	4.398	4.437
159.5	4.966	5.126	5.046
160.75	5.251	-	5.331
162	5.536	-	-

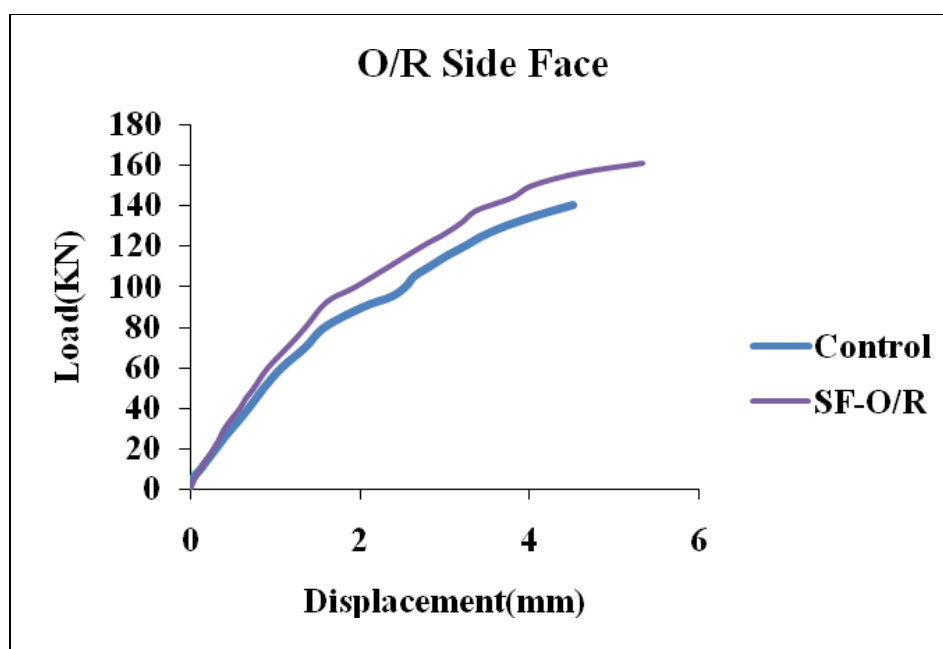


Fig. 4.14 Load v/s Mid Span Deflection of Beam SF-O/R

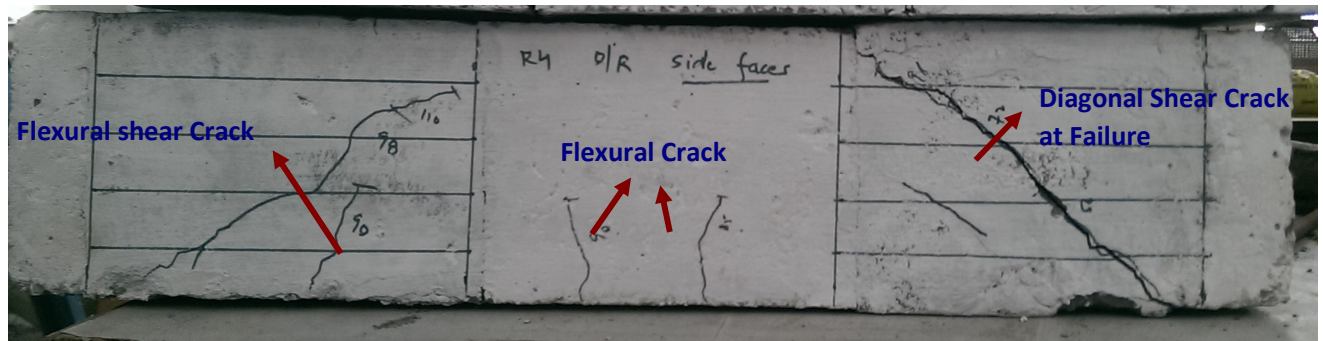


Fig. 4.15 Crack Pattern of Beam R4 at Failure

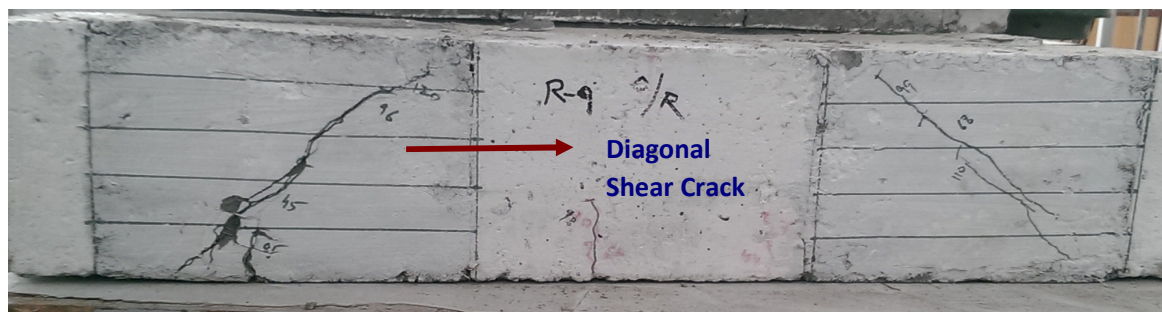


Fig. 4.16 Crack Pattern of Beam R9 at Failure

4.4.5 Effect of Retrofitting on O/R section Beams Retrofitted by Replacement of Side Face and Bottom Face Damaged Concrete Cover in the Shear Span with UHPFRC (U-Type)

A set of initially stressed two beams for both O/R section were retrofitted by replacing damaged concrete cover from the sides and bottom in the shear span with new UHPFRC using epoxy resin as bonding agent between old concrete surface and the new UHPFRC. The load at centre and mid-span deflection were recorded for both the beams. These values are presented in Table 4.8 and crack pattern for both beams are presented in Fig. 4.17 & 4.18. The results of the beams R5 and R10 were then averaged to obtain a single set of results (**designated as beam UT-O/R**) as shown in Table 4.8 and Fig. 4.19. The behavior of the average beam specimen UT-O/R was then studied in detail and used for comparison with the average control beam CB-O/R, as illustrated in succeeding paragraph.

It is observed that the retrofitting leads to increase in the load carrying capacity from 140 kN (Control Beam) to 174 kN whereas the maximum deflection at the center of the beam

corresponding to the failure load of 174 kN is 8.273 mm as compared to 4.525 mm for the control beam at 140 kN. First crack load was observed to occur at load of 56.6 kN.

For the over-reinforced section beams failed in shear before the steel yields. So for over-reinforced beams ultimate failure load was taken as the yield load. From the load-deflection curve, yield load was found to be 174 kN and corresponding deflection at the yield load was 8.273 mm.

The improvement in load carrying capacity of U-type retrofitted beam was observed to be highest 24.285%. The possible reasons for the above results are: (a) the confined area is maximum in this case as beams are retrofitted in U-type i.e. along the side faces and the bottom of the beams. The higher modulus of elasticity of UHPFRC confined the initial damaged beam from the side faces and the bottom which results in higher load carrying capacity of the retrofitted beam. Also similarly for the side face retrofitted beams, higher compressive strength of UHPFRC. However, the bond failure was observed between the old concrete and the UHPFRC under the position of two point loading. The crack was propagated towards the compression side of the beam. The localization of the crack was observed till the ultimate failure of the specimen. (b) The hybrid fibers able to retard and delay the crack propagation so that the load carrying capacity of side face retrofitted initially stressed beam is increased.

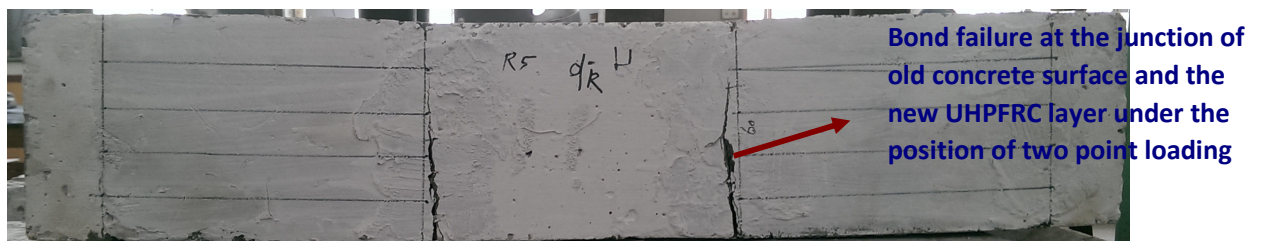


Fig. 4.17 Crack Pattern of Beam R5 at Failure



Fig. 4.18 Crack Pattern of Beam R10 at Failure

Table 4.8 Detail of Load vs Mid Span Deflection of U/R beam Retrofitted in U-Type

Load (kN)	Displacement for Beam R5 (mm)	Displacement for Beam R10 (mm)	Displacement for Beam UT-O/R (mm)
0	0.000	0.000	0.000
4	0.020	0.010	0.015
8	0.030	0.035	0.032
12	0.100	0.055	0.077
16	0.170	0.088	0.129
20	0.240	0.136	0.188
24	0.330	0.221	0.275
28	0.386	0.292	0.339
32	0.445	0.382	0.413
36	0.515	0.449	0.482
40	0.624	0.544	0.584
48	0.795	0.695	0.745
56	0.942	0.816	0.879
64	1.145	0.995	1.070
72	1.328	1.314	1.321
80	1.516	1.616	1.566
96	2.418	2.325	2.371
112	3.086	2.985	3.035
128	3.88	3.763	3.821
144	4.825	4.872	4.848
152	5.620	5.746	5.683
160	6.381	6.422	6.401

Load (kN)	Displacement for Beam R5 (mm)	Displacement for Beam R10 (mm)	Displacement for Beam UT-O/R (mm)
172	7.806	8.216	8.011
174	8.126	-	8.273
176	8.33	-	-

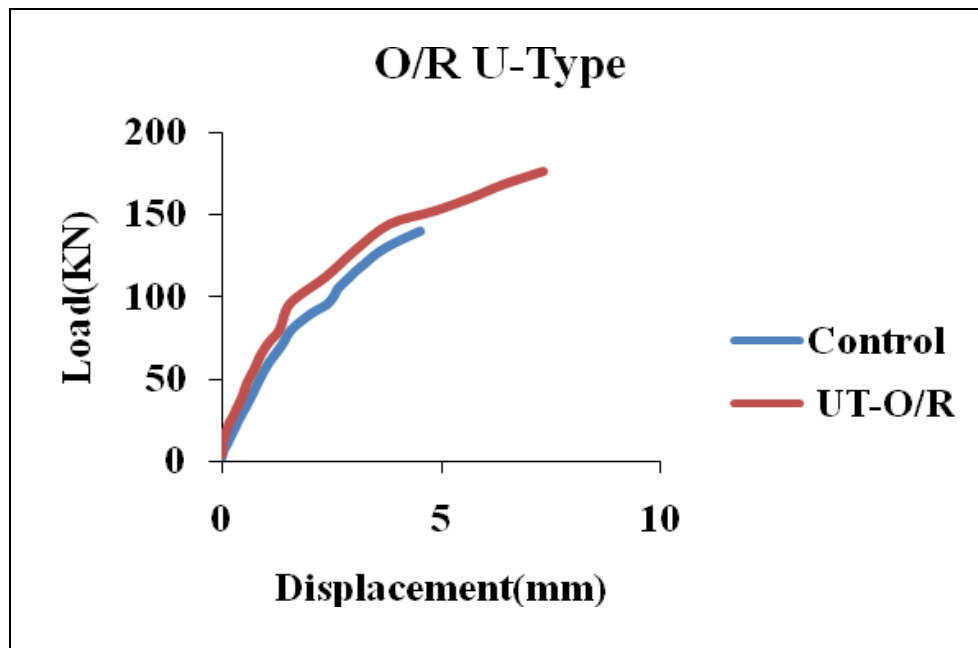


Fig. 4.19 Load v/s Mid Span Deflection of Beam UT-O/R

4.4.6 Effect of Retrofitting on O/R section Beams Retrofitted by External Jacketing 25 mm thick layer of UHPFRC in the Shear Span

When beams are strengthened with 25 mm thick layer of UHPFRC along side faces of shear span then due to the increased area (200*150 mm) to resist shear, different behavior is observed as compared to other two techniques discussed above. The load at centre and mid-span deflection were recorded for both the beams. These values are presented in Table 4.9 and crack pattern for the beam is presented in Fig. 4.20. The results of the beams R7 and R12 were then averaged to obtain a single set of results (**designated as beam EJ-O/R**) as shown in Table 4.9 and Fig. 4.21.

The behavior of the average beam specimen EJ-O/R was then studied in detail and used for comparison with the average control beam CB-O/R, as illustrated in succeeding paragraph.

Initially very less displacement was observed at the center of the beam as compared to the control beam due to which the load carrying capacity also increased up to the load of 131.4 kN. As the load increased further, the sudden failure of the beam is observed due to the **debonding** of the 25 mm thick layer of UHPFRC from the surface of the RC beam (old concrete). The maximum deflection at the center of the beam at failure of the beam is 2.686 mm.

Failure of beams is due to debonding of external new UHPFRC layer from the surface of old concrete. So yield load was taken to be 131.4 kN and corresponding deflection at the yield load was 2.686 mm.

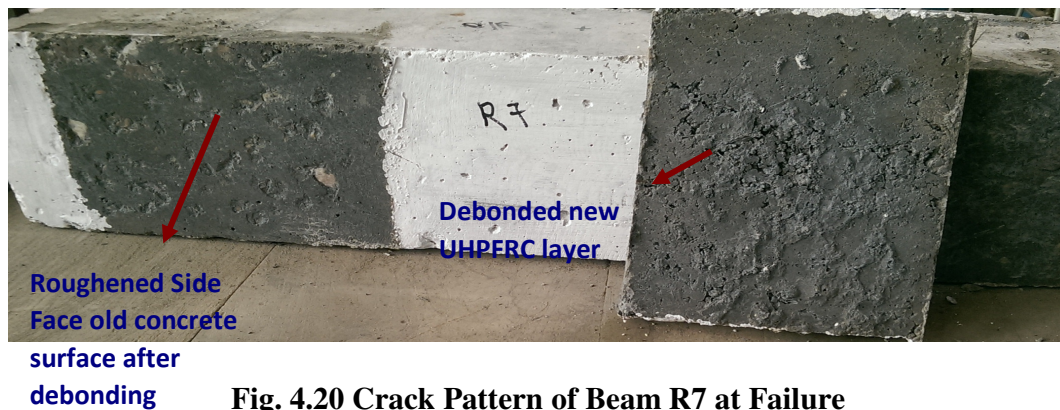


Fig. 4.20 Crack Pattern of Beam R7 at Failure

Table 4.9 Detail of Load vs Mid Span Deflection of O/R beam Retrofitted by External Jacketing

Load (kN)	Displacement for Beam R7 (mm)	Displacement for Beam R12 (mm)	Displacement for Beam EJ-O/R (mm)
0	0.000	0.000	0.000
2.5	0.009	0.000	0.004
5	0.029	0.011	0.020
7	0.057	0.043	0.050
9	0.084	0.072	0.078
11.5	0.098	0.096	0.097

Load (kN)	Displacement for Beam R7 (mm)	Displacement for Beam R12 (mm)	Displacement for Beam EJ-O/R (mm)
13.5	0.116	0.126	0.121
16	0.135	0.145	0.14
18.5	0.188	0.166	0.177
20.5	0.232	0.192	0.212
23	0.266	0.236	0.251
25.5	0.311	0.274	0.292
27.5	0.358	0.328	0.343
30	0.395	0.375	0.385
32.25	0.435	0.418	0.426
34.5	0.476	0.456	0.466
40.25	0.543	0.526	0.534
46	0.634	0.634	0.634
51.75	0.735	0.745	0.74
57.5	0.823	0.823	0.823
63.25	0.904	0.924	0.914
69	0.986	0.998	0.992
80.5	1.190	1.160	1.175
92	1.440	1.380	1.410
103.5	1.766	1.746	1.756
115	1.995	1.988	1.991
120.75	2.254	2.254	2.254
126.5	2.558	2.489	2.523
130.5	2.687	2.634	2.656
131.4	2.719	-	2.689
132.3	2.745	-	-

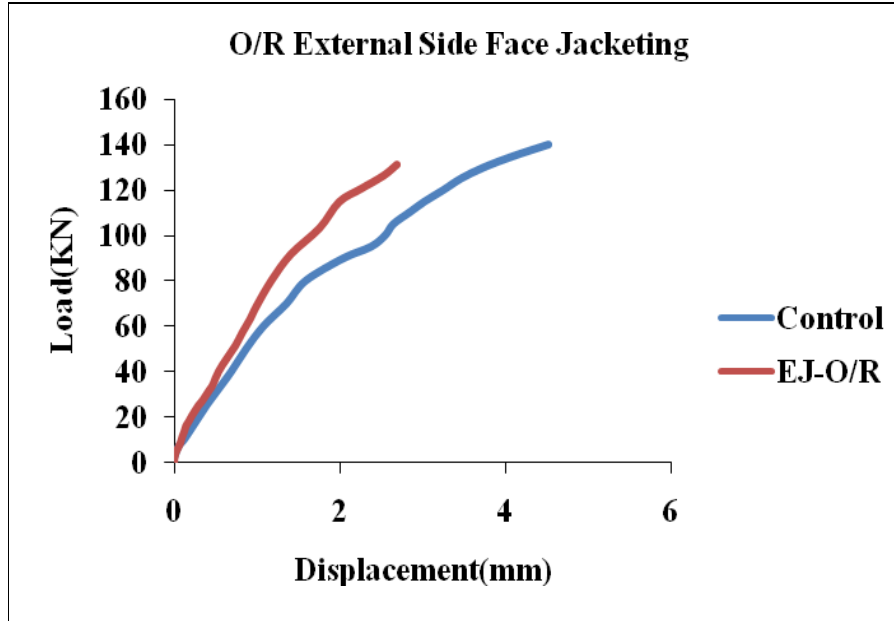


Fig. 4.21 Load v/s Mid Span Deflection of Beam EJ-O/R

4.4.7 Comparison of all Three Different Retrofitting Techniques Compared to Control Beam for U/R Section Beams

Initially stressed shear deficient beams are retrofitted with three different configurations. The load deflection curve for all the three configurations are presented in the Fig. 22.

It is seen from the curve that the load carrying capacity increased maximum in the case of U-Type retrofitting configuration and minimum in the case of external jacketing due to debonding of the new UHPFRC layer from the surface of old concrete. First crack load, yield load, ultimate load and failure modes for all different retrofitted configuration beams are presented in table 4.10.

4.4.8 Comparison of all Three Different Retrofitting Techniques compared to Control Beam for O/R Section Beams

Similarly when initially stressed shear deficient O/R beams are retrofitted with three different configurations, the load carrying capacity of the beams increased maximum for the U-Type retrofitting configuration whereas minimum for the External jacketing configuration due to debonding of new UHPFRC layer from the surface of the old concrete. The load deflection curves for all three different configurations are compared as compared to control beam as shown

in the Fig.23. First crack load, yield load, ultimate load and failure modes for all different retrofitted configuration beams are presented in Table 4.11.

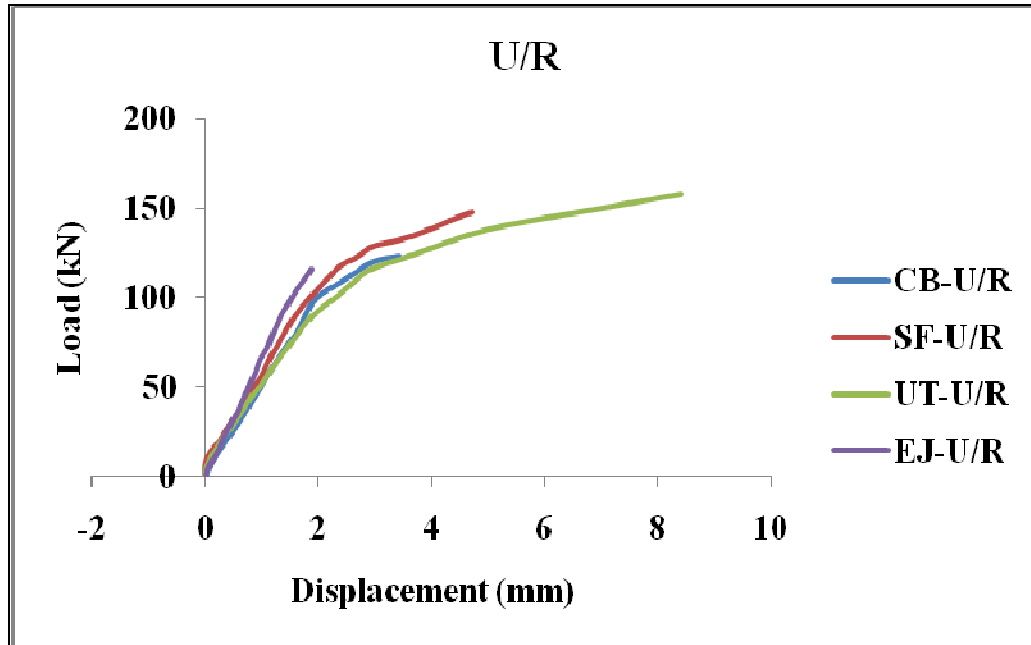


Fig. 4.22 Load vs Mid Span Deflection For All Three Configurations vs Control U/R Section Beam

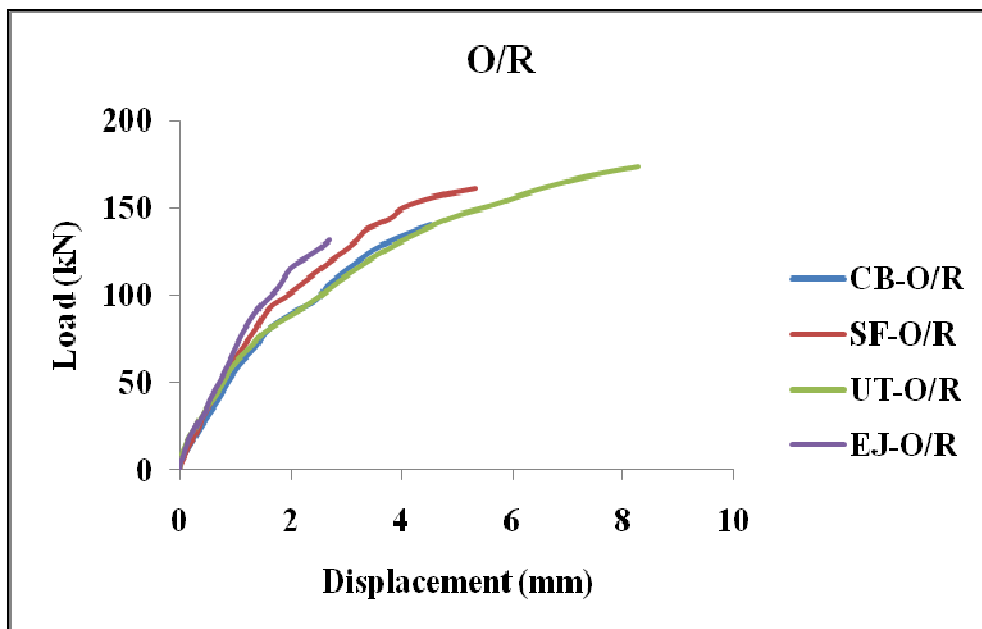


Fig. 4.23 Load vs Deflection curve for all three configurations vs Control O/R Section Beam

Table 4.10 First Crack Load, Yield Load, Ultimate Load and Failure Mode For All Three Configuration vs Control U/R Section Beams

Particulars	First Crack Load (kN)	Yield Load (kN)	Ultimate Load (kN)	Percentage (%) Increment in Ultimate Load	Failure Mode
Control CB-U/R	32	115	123	-	Shear
SF-U/R	36.75	121.6	147.9	20.24	Shear
UT-U/R	38.25	123	157.9	28.37	Bond failure
EJ-U/R	39.25	115.75	115.75	5.895(-)	Debonding

Table 4.11 First Crack Load, Yield Load, Ultimate Load and Failure Mode for All Three Configuration vs Control O/R Section Beams

Particulars	First Crack Load (kN)	Yield Load (kN)	Ultimate Load (kN)	Percentage (%) Increment in Ultimate Load	Failure Mode
CB-O/R	48	140	140	-	Shear
SF-O/R	55.75	160.75	160.75	14.82	Shear
UT-O/R	56.6	174	174	24.28	Bond failure
EJ-O/R	55.2	131.375	131.375	6.16(-)	Debonding

The possible reason for this behavior is the increased area (200*150 mm) due to which confinement increased and ability to deform decreased.

4.5 Effect of Retrofitting on Ductility for all Three Different Retrofitting Techniques

Ductility represents the ability of reinforced concrete to show inelastic deformation prior to collapse. To measure ductility, it is generally accepted to use ductility factor (μ) which represents the ratio of rotation (θ), curvature (ϕ), deflection (Δ), or absorbed energy (E) at the ultimate load to corresponding property when the reinforcement starts yielding (Afefy & Mahmoud, 2014). The inelastic behavior of the reinforced concrete is dictated by property of reinforcement; not by the concrete which is a brittle material. However, concrete does influence the inelastic behavior of reinforced concrete by providing tension stiffness effect.

$$\text{Ductility factor } (\mu) = \Delta_u / \Delta_y \quad (\text{Afefy \& Mahmoud, 2014})$$

As discussed above, the results of both beams of same type for all different retrofitted techniques are averaged to get a single value of ductility for each type of beam. The averaged data for ductility of all those beams are presented in Table 4.12. It can be noted from the table that the ductility increased when the beams are retrofitted by Side Face and U-Type retrofitting technique for U/R section beams. The corresponding values of beam SF-U/R and UT-U/R are 1.708 and 2.424. However, the ductility reduced for the external side face jacketing in the shear span due to debonding of new UHPFRC layer from the old concrete surface.

For over-reinforced section, beams failed in shear before the steel yields so the ultimate load and the yield load are same. It can also be noted from the table that the ductility values are 1 for O/R section beams.

4.6 Effect of Retrofitting on Energy Absorption for all Three Different Retrofitting Techniques

Energy absorption capacity was calculated from load deflection curves. The area under the load deflection curve was calculated using **Trapezoidal rule**. The energy absorption values for all beams are presented in Table 4.13. It can be observed from the table that the energy absorption increases when the beams are retrofitted with UHPFRC for Side Face and U-Type retrofitting technique. The maximum value of energy absorption for both U/R and O/R section beams is observed for U-type retrofitting. The percentage increment in energy absorption is 264% and 150.25% for U/R and O/R U-Type retrofitted beams respectively.

4.7 Effect of Retrofitting on Initial Stiffness for all Three Different Retrofitting Techniques

Initial stiffness was calculated from the load deflection curve. It is taken as the initial slope of the load deflection curve. The initial stiffness of all the beams is presented in Table 4.14. From the table it can be noted that the initial stiffness increased for all the beams retrofitted with UHPFRC. Beams with external side face jacketing showed maximum increment in initial stiffness. This behavior may be due to increased cross-section (200*150 mm) in the shear span.

Table 4.12 Ductility for all Retrofitted Beams as Compared To Control Beams

Particulars	Deflection at Yield Load (Δ_y) (mm)	Deflection at Ultimate Load (Δ_u) (mm)	Ductility Factor(μ)
Control CB-U/R	2.704	3.422	1.266
SF-U/R	2.67	4.71	1.708
UT-U/R	3.475	8.424	2.424
EJ-U/R	1.83	1.83	1
Control CB-O/R	4.525	4.525	1
SF-O/R	5.331	5.331	1
UT-O/R	8.273	8.273	1
EJ-O/R	2.689	2.689	1

Table 4.13 Energy Absorption for all Retrofitted Beams as Compared To Control Beams

Particulars	Energy Absorption (Joules)
Control CB-U/R	266.14

SF-U/R	471.21
UT-U/R	969.565
EJ-U/R	111.445
Control CB-O/R	401.96
SF-O/R	579.76
UT-O/R	998.815
EJ-O/R	215.955

Table 4.14 Initial Stiffness for all Retrofitted Beams as Compared To Control Beams

Particulars	Initial Stiffness (kN/mm)
Control CB-U/R	42.53
SF-U/R	51.5
UT-U/R	44.85
EJ-U/R	65.31
Control CB-O/R	50.31
SF-O/R	58.28
UT-O/R	59.09
EJ-O/R	71.215

CHAPTER 5

CONCLUSIONS

5.1 CONCLUSIONS

Based upon the results of experimental study carried out the following conclusions can be drawn:-

1. The load carrying capacity increases when the initially stressed (60%) beams are retrofitted with UHPFRC except for the case of external side face jacketing whereas initial stiffness increased for all the retrofitted beams.
2. Retrofitting of initially damaged beams by replacing side face concrete cover with UHPFRC in the shear span increases the ultimate load carrying capacity and yield load of Under-reinforced (U/R) section beams by 20.24% and 5.74% respectively. The ultimate load carrying capacity of the Over-reinforced (O/R) section beams retrofitted with the same technique increased by 14.82% as compared to the control beam. Hence, for both U/R and O/R sections, the beam failed in shear but at higher load as compared to control beams.
3. When U-TYPE retrofitting technique i.e. removing damaged concrete cover from side faces and bottom in the shear span and replacing it with a U-shape layer of UHPFRC is used, the ultimate load carrying capacity and yield load of Under-reinforced (U/R) section beams increases by 28.37% and 6.96% respectively. For O/R section beams, the increment in ultimate load carrying capacity is 24.28%. But in this case, bond failure was observed between old concrete and UHPFRC under the points of load application for both U/R and O/R section beams.
4. When external 25 mm layer side face jacketing of UHPFRC in the shear span is done on the initially stressed beams, the beam initially sustained higher loads at lower displacements, i.e. the initial stiffness is improved. However, as the load increased further, debonding between the old concrete surface and the UHPFRC occurred due to lack of confinement which led to sudden failure of beams at a lower ultimate load as compared to the U-type or side face retrofitted beams.

5. The deflection at the failure load increased when beams were retrofitted with UHPFRC in the Side Face and U-Type retrofitting configurations. Maximum ductility and energy absorption capacity were observed for U-Type retrofitting technique. The corresponding value of ductility factor (μ) was 2.46 and increment in energy absorption was 263.45 % for U/R section beam and 150.25 % for O/R section beams. Thus UHPFRC retrofitting results in the increase in the ductility and energy absorption capacity of the RC beams.

5.2 Scope for Future Study

The following are some suggested areas for future studies on the retrofitting of shear deficient beams using UHPFRC:

1. The effect of shear span to depth ratio (a/d) on the failure mode and load carrying behavior of shear deficient beams retrofitted with UHPFRC can be explored. Beams of different lengths and cross-section may be cast to achieve this objective.
2. In this study, all the beams were stressed up to 60% of the control beam failure load and subsequently retrofitted with 25 mm thickness of UHPFRC. The stress level and thickness of UHPFRC may be varied (both may be changed simultaneously or one at a time) and their effect on the behavior of beams may be studied.
3. The effect of different configurations used for retrofitting in this thesis on the behavior of beams may be studied in combination with the above mentioned points. Further, the researchers may also explore the effect of new retrofitting configurations and compare the efficiency of those methods with the contemporary rehabilitation techniques.

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