BEHAVIOUR OF EXTERIOR RC BEAM COLUMN JOINT RETROFITTED WITH HPFRCC

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IN

STRUCTURAL ENGINEERING

By

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DECLARATION

I hereby declare that this dissertation entitled **"BEHAVIOUR OF EXTERIOR RC BEAM COLUMN JOINT RETROFITTED WITH HPFRCC"** which is submitted in partial fulfilment of the requirement for the award of the Degree of **Masters of Engineering in Structural Engineering** in Civil Engineering Department, Thapar University, Patiala. It is an authentic record of my own independent and original research work carried out by me under the supervision of **Dr. Prem Pal Bansal**, Associate Professor and **Mr. Raju Sharma**, Lecturer, Department of Civil Engineering, Thapar University, Patiala.

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ABSTRACT

Normally the RCC structures constructed these days suffers a lot of damage due to faulty construction, improper design, wear and tear, lack of proper maintenance and the frequent vibrations produced during an earthquake. During the earthquake structure becomes more & more vulnerable to collapse, for such areas the behaviour of structure is needed to be improved. The behaviour of structure is largely dependent upon the behaviour of beam-column joint. More attention is needed paid in reinforcing bars, their sufficient development length, proper binding and in concrete casting. To overcome the damage problems of the structure, retrofitting technique is adopted in which different components of structures are strengthened. Every retrofitting technique has its unique advantages in strengthening structural members like it may either improve strength, ductility, stiffness or any combination of these. So, depending upon the type of requirement, particular strengthening technique has to be selected. High performance fibre reinforced cement composite (HPFRCC) is one of the retrofitting concrete which is widely being developed in different countries. The application of HPFRCC in structural building has not yet gained much attention due to the lack of proper guidelines, coherent information and available literatures. So in order to have better understanding about HPFRCC in structural buildings, the dissertation work has been carried out in the beam column joint with an application of HPFRCC.

The aim of dissertation is to evaluate the performance of non seismic beam column joint retrofitted with high performance fibre reinforced cement composite under quasi static reverse cyclic loading at different stress levels. The analysis of specimen performance is done at two different stress levels by carrying out its retrofitting. The results obtained in reference specimen is compared with its retrofitted specimens in terms of hysteresis curve, energy dissipation capacity, energy absorption, stiffness variation, joint stresses and joint failure analysis.

It was observed that retrofitting the joint with HPFRCC improved the overall specimen performance. In addition to these hooked steel fibres has helped the specimen in regaining its original strength and in bridging the concrete from its spalling. Secant stiffness variation also displays that the retrofitted specimen helped in improving stiffness of joint.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

Over a period of time the beam column joint has always been critical zone within the frame. Beam column joints ensure continuity of a structure and transfer the forces to the adjoining member. This region possesses maximum moment by undergoing bending. The philosophy behind the concept of good building design resides in strong column weak beam mechanism and ensuring that more energy is dissipated by the hinges developed at the joint region. On the basis of capacity design approach, the strength of the joint should be more than the maximum strength of the weakest member meeting at the joint. The poor design practice of beam column joints has resulted in ineffective inelastic capability of joint to release energy. Poor detailing of joint puts entire structure at risk even if rest of the members are safely designed. Poor detailing increases the chances of crack development at the joint region. When it comes to seismic forces, this zone becomes more vulnerable due to additional cyclic loading effect. So by the above mentioned reasons, beam column joint becomes more vulnerable component of the building which needed more attention. Thus, an attempt has been made to improve the behaviour of joint by using retrofitting technique which is essential part of building maintenance.

1.2 BEHAVIOUR OF RC BEAM-COLUMN JOINTS

1.2.1 Significance of Joint Geometry

The joint size definitely has some affect on its load bearing capacity. To assess small joint in an ideal condition of bond and anchorage is a difficult task. Normally reinforcement is kept less for smaller beam column joint. Apart from overall size of joint, relative member size, method of detailing and the force intensity also affects the behaviour of joint. However joint is designed on the basis of strong column and weak beam concept that is beam width has to be equal to or less than column width. So beams yielding starts prior to column yielding.

This will make possibility of beam bending than that of columns as in columns shear and flexure both will act in compression which would lead to spalling of concrete and early failure. Development length is an important criterion in determining the geometry of joint. They help in resisting in horizontal shear forces. However, in exterior joint, column width has to be more than that of beams as this would strengthen bond condition at joint. Along with this there are several codes across the globe which specifies minimum condition of column width at these joints.

1.2.2 Significance of Concrete Strength

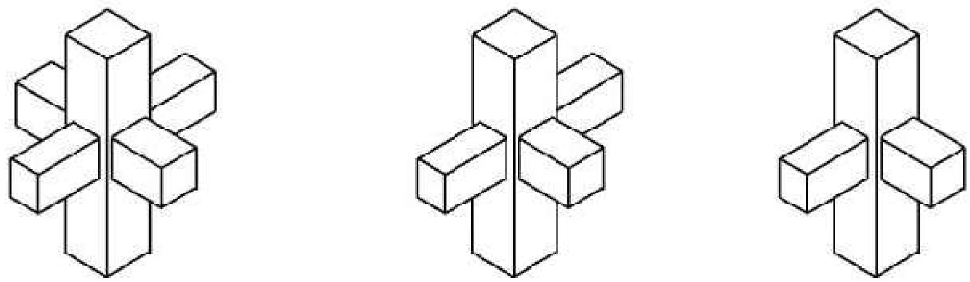
Concrete strength is something which directly affects the strength, ductility and failure of joint. When beam is made of stronger concrete than column, the column fails at low loads in tension state while when the column is made stronger than that of beams, the moment resisting capacity increases up to certain level while when beam and column are made of same grade, joint attains more rotation capacity than that of other combinations. These rotations at joint are dependent upon the ductility of concrete. High strength concrete is used only when joint has low shear stress and high joint confinement for attainment of expected strength.

1.2.3 Significance of Steel Bars

Steel bars directly affect the strength of member. As these reinforcement bars are ductile in nature, they carry both compression as well as tension without undergoing immediate failure. Reinforcement bars gets bonded with surrounding concrete, to develop strong, durable structure. They have an important role in bearing loads within a structure. These bars are surrounded by steel stirrups which help main bars in bearing more loads. Stirrup prevents main bars from splitting, by holding them together and also prevents from early formation of cracks on concrete surface.

1.3 TYPES OF JOINTS IN BUILDING FRAMES

Joint is region in a building where beam and column intersect each other. These joint are made in such a manner so that adjoining column is able to withstand all the forces developed in beam and transfer then successfully to the bottom member in a building. Normally these joints are of three type's namely interior joint, exterior joint and corner joint that has been shown in Figure 1.1. When four beams intersect at same location in the column, the joint is said to be an interior joint. For an exterior joint, when three beam frames (two in opposite directions) intersect at the same location in a column, the joint is said to be an exterior joint. When only one beam intersects a column, the joint is called corner joint.

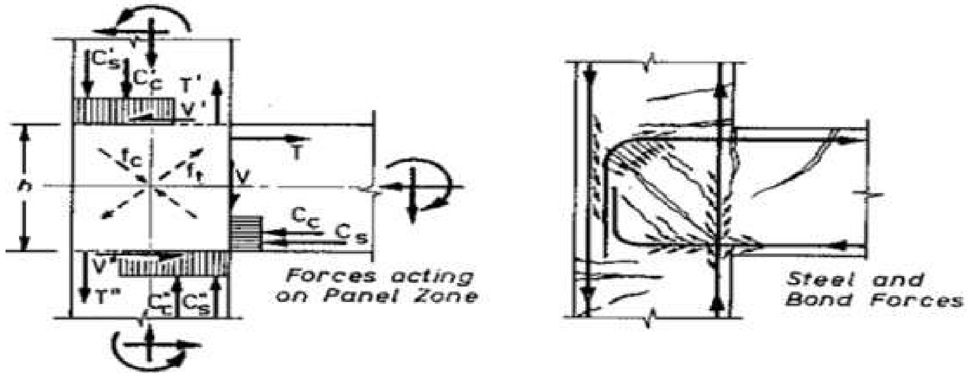


(a) Interior (b) Exterior (c) Corner

Figure 1.1: Types of joints in structure (Joint ACI-ASCE Committee 352, 2002)

1.4 FORCES ACTING ON A BEAM COLUMN JOINT

Forces acting on joint depend upon the configuration of joint and how loads are acting over it. It mainly depends upon the intensity of stress acting within a joint and crack development pattern over the surface of concrete. Various stresses like shear and flexural both acts together in such combination in a joint with its ability to remain in equilibrium as it is under any combination of loads. The shear forces and moments developed at a joint introduce stress resultant at the joint. The stress resultant creates both vertical and horizontal shear force in the joint leading to joint core to develop internal diagonal compressive & diagonal tensile stresses. Depending upon the type of stresses, diagonal cracking or crushing would occur i.e. in case of tension and compression. Without sufficient shear reinforcement in the joint region, most of the time crack formation is observed in corner to corner diagonal plane of joint core.

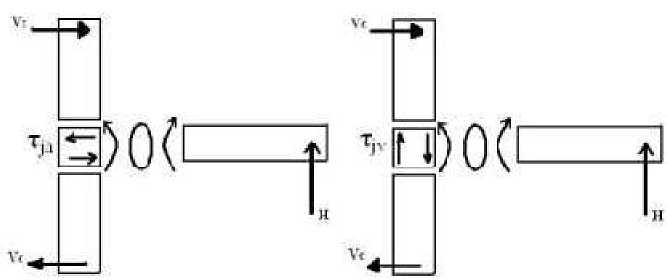


(a) Stress formation (b) Cracking pattern and bond forces

Figure 1.2: Actions at stress development in beam-column joint (Siva chidambaram &Thirugnanam, 2012)

From the position of the stress resultants in the beam column joint, it is clear that diagonal tension and compressive stresses (*fc* & *ft*) are introduced in the critical zone of the joint. The diagonal tension will be highest when the ultimate capacity of the adjoining member is achieved and results in formation of wide cracks in perpendicular direction at the joint core. The critical behaviour of diagonal tension depends upon the ductile capacity of reinforcement and the magnitude of the axial compressive load on the column. Due to vertical and horizontal shear force, large shear force is developed in joint irrespective of plastic hinge development in column or beam. These shear forces sometimes causes damage in the joint region due to failure of bond or shear mechanism or even both mechanisms.

Due to the large shear forces developed in the beam-column joints irrespective of its position whether plastic hinge is developed on column face or at some other locations of adjoining beam. The shear forces may create a failure in the joint region which may be either shear failure, bond failure or combined failure. With the development of ultimate horizontal and vertical shear forces in joint region, the joint becomes more vulnerable to collapse. With the seismic load application, moments are induced in the joint region due to the compressive and tensile stresses developed in the beam. From the excessive moments developed in the joint region from the reversal in cyclic loading, the beam reinforcement is required to be in compression on one side of the joint and at tension on the other side of the joint. The high bond stresses is required to sustain this force gradient across the joint. If the joint fails to sustain the force, it may result bond failure and degradation of moment carrying capacity of joint takes place along with excessive drift. As different mechanisms are involved in the shear transfer after the onset of diagonal cracking, it will finally result in failure of joint.



(a) Horizontal shear (b) Vertical shear

Figure 1.3: Forces acting on the beam column Joint (ACI 318-02, 2005)

When the joint transfers the shear force (uncracked or cracked state) across the joint core, a large amount of shear stress is developed on the joints outer surface resulting in formation of diagonal stresses and leading to formation of diagonal cracks which directly affects strength, stiffness and behaviour of joint. In due course, joint starts becoming more and more flexible eventually leading to shear deformation as shown in Figure 1.3.

1.5 THE JOINT VULNERABILITY PROBLEM

The intrinsic problem of normal concrete is its brittle nature which may cause collapse in non seismically detailed structural members after the first crack during a large earthquake. The use of steel fibres may convert the brittle characteristics to ductile ones. The principal role of fibres is to bridge cracks and resist their formation. Therefore a considerable improvement in tensile strength and higher ultimate strain can be obtained.

Since the 1960s, several theoretical and experimental researches have been done to assess and determine the seismic resistance of beam-column joints. The performance of tested structural frames indicated that the joints have frequently been found to be the critical component of beam-column joint after seismic cycle load. The improper behaviour of joints is mainly caused by lack of shear reinforcement and insufficient anchorage capacity in the joint. Such effects have been observed in many earthquakes across the world which highlighted brittle behaviour of joint when subjected to seismic force. An example of joint failure is shown in Figure 1.4 representing brittle behaviour of joint causing decrease in moment resisting capacity of joint led to accumulation of stress at joint region and inefficient performance of joint leading it to vulnerability state. Thus it became more important to develop new techniques so that the lost strength of the joint can be regained. It was not necessary that this new strength regaining technique has to be used in fully damaged joint instead of this it can also be used at partial damaged joints. So that the structure will able to sustain more seismic load and lead to increased life of building. This newly developed technique which is used to regain lost strength of structure (here joint) is called retrofitting.

Moreover, column should be designed in such a way that should not adversely affect the joint strength. In some cases it has been observed that by lower strength of column or accumulation of stress occurs in column and sometimes column may sway the frame sideways in multi storey building. However in capacity design case, the energy dissipation occurs mainly in the beam plastic hinges, which prevents the joint suffering from excessive strength and stiffness degradation when subjected to seismic load in the inelastic range.

Figure 1.4: Damage of joint in an earthquake (1999 Kocaeli, Turkey)

1.6 BEAM COLUMN JOINT FAILURE MECHANISM

Joint failure mechanism is understood by the development of stress in the joint region. As the beam or column starts behaving like inelastic, the probability of failure at joint increases which starts from development of hinge. The modern buildings are designed on strong column weak beam concept. The formation of hinge is generally formed around joint interface depending upon joint detailing. There are locations where failure of joint may occur

a) Plastic hinge formation in beam region of joint interface.
b) Plastic hinge formation in column region of joint interface.
c) Diagonal crack formation around the joint.

Some of the other reasons of failure of joint are crushing of concrete, improper detailing and slippage of steel bars. The propagation of cracks helps in understanding the type of crack developed like flexural crack, shear crack and flexural shear crack.

1.7 NEED FOR RETROFITTING

The scope of retrofitting has always been a topic of research to improve the durability of earthquake vulnerable building. In every earthquake it is found that several cracks are developed at joint region in many buildings which becomes more vulnerable for bearing more seismic loads. To improve joint performance or to make it more durable retrofitting is adopted. Retrofitting not only strengthens the building but also improves the performance for better load transfer mechanism.

With the development of technology, more efforts are given by Civil Engineers towards retrofitting of building. As retrofitting is found effective in imparting strength to building more than its original strength which was lost earlier. To overcome the damage problem of the structure retrofitting is adopted. The techniques adopted in retrofitting are based upon the requirement of structure. This requirement varies from need of strength, stiffness & ductility improvement or any combination of these. Depending upon the improvement parameter, suitable type of retrofitting technique is adopted.

1.8 RETROFITTING

Retrofitting is strengthening of structure or elements of a building to a pre-defined stress level to enhance the performance of elements in terms of energy dissipation, energy absorption, stiffness, load carrying capacity and ductility. Retrofitting is basically classified in three categories:

a) Global Retrofitting
b) Local Retrofitting
c) Base isolation of a building.

a) Global Retrofitting is retrofitting of a building as a whole. This improves overall behaviour of a structure by providing additional building elements like shear wall, improving joint connections, removing various irregularities within structure, adding infill walls, wall thickening, and mass reduction in storeys.

b) Retrofitting of local or single element is called as local retrofitting. These elements include beam, column, joint, walls, strengthening individual footing to fulfil the requirement of predicted demand. There are various techniques under local retrofitting such as

i) Mortar injection technique - In mortar joint technique repairing is done by injecting mortar in an uninterrupted manner so that the mortar does not set. These are mainly adopted in retaining walls or piers for retrofitting.

ii) Epoxy injection technique - In epoxy injection technique, the high strength epoxy is used to rebind the cracked concrete by sending epoxy into the cracks. This injection is done under pressure to fulfil the crack voids. This technique mainly depends on the viscosity of epoxy and the type of crack. Various steps followed in this method are -

1 Cleaning the crack either by water or by compressed air.
2 Sealing the surface to prevent epoxy from leaking.
3 To mix the epoxy in batch either manually or mechanically.
4 To inject the epoxy by pressure pump.

5 After the epoxy has cured properly removing the surface sealant.

iii) Shotcreting - In shotcreting, repair work of concrete is done when a close surface is required by newly made concrete. This is done to improve properties of concrete by using admixtures or other materials.

iv) High Performance Concrete (HPC)/ Ultra High Performance Concrete (UHPC)/ High performance Fibre Reinforced cement Composite (HPFRCC) - Depending upon the strength of concrete, concrete is classified as ultra high performance concrete (UHPC) / high performance concrete (HPC). For HPC compressive strength lies between 60 to 150 MPa and for UHPC compressive strength lies between 150 to 810 MPa. These are used for retrofitting within a structure when there is need to achieve high strength in early days (within a week).

v) Fibre reinforced polymers – Fibre reinforced polymers (FRP) is a newly made material from polymer matrix reinforced with different types of fibres. The fibres mainly used are carbon, glass, aramid or basalt. The polymers normally used are epoxy, vinyl ester or polyester thermosetting plastic. FRP is one of the worlds latest developed techniques for retrofitting of structure which includes different kind of fibres like CFRP, GFRP and Aramid Fibre Reinforced Polymers (AFRP). These are included in continuous polymer matrix. These FRPs increases the strength and stiffness of structure if the structure is retrofitted properly. They certainly increase the durability at various situations within the structure and these are handy at installation. The materials used in improving seismic / non seismic performance of striucture are mainly FRP based materials such as glass, carbon, basalt or aramid fibres.

Carbon fibres are made from Pitch resins, polyacrylonitrile fibres (PAN) or Rayon when carbonised (by oxidation or pyrolysis) at high temperatures. Elasticity is improved by the process of graphitising or stretching the fibres. Carbon fibres are manufactured in diameters varying from 4 to 17 μm. further processes involved are weaving or braiding in carbon fabrics can be used as reinforcement.

In GFRPs fibres use textile grade glass fibres. These fibres are heated to direct up to 1300 degrees Celsius, then dies extrude the filaments in diameter varying from 10 to 18 μm. these filaments are then wound into larger threads for their transportation and then woven in mat form. In mat, glass fibres used has length in between 3 to 26 mm.

FRP composite materials are developed for providing required strength to the masonry structure. The application of FRP composite materials externally bonded fibre reinforced polymer in retrofitting masonry structure and for strengthening several new techniques have been adopted on the basis of non metallic elements like polymeric materials as they are lowly sensible to environmental hazard making the retrofitting more appropriate for its application under severe operating conditions.

On the other hand, with the working of fibre reinforced polymer (FRPs) for retrofitting of concrete structure has obtained more attention. The reinforcement with FRP is done by two types in the case of masonry retrofitting. The two types are externally bonded retrofitting and near surface mounting retrofitting in which grooves are made into the surface of wall. From the last two decade, near surface mounting fibre reinforced polymer (NSMFRP) technique has gained more attention as it provides more advantages in aesthetics, lesser surface damage & better prevention from ultraviolet ray's exposure by delaying deterioration. Another major advantage of using NSM FRP retrofitting technique is that it debonds at higher strain than other retrofitting technique.

c) Base isolation within a structure is a technique to dissipate vibration energy from the base of structure into a tuned mass system. This is done to prevent the structure from harmful effect of earthquake when the structure feels seismic motion. The damper does not transfer the vibration to the structure rather than it transfers the motion to the ground.

1.9 DIFFERENCE BETWEEN HPC/ UHPC/ HSC/ HPFRCC

Concrete are normally divided into normal strength concrete (NSC), High strength concrete (HSC), ultra high strength concrete (UHPC). With the extent of technology these is not any specific boundary between them but it is generally considered concrete having strength more than 40 MPa and less than 170 MPa are said to be high strength concrete. The ratio of water to cement also varies from 0.2 to 0.45. There are several methods out there from which high strength concrete is made like

a) Seeding
b) Revibration
c) High speed slurry mixing
d) Use of admixtures
e) Inhibition of cracks
f) Sulphur impregnation

g) Use of cementitious aggregates.

High performance concrete (HPC) is new type of concrete which was developed after the development of high strength concrete. HPC describes HSC in more wide and efficient sense. HPC has water cement ration of less than 0.3 and with the help of superplasticizer more dense packing is achieved which help in strengthening of transition zone. The maximum size of aggregate used to achieve strength more than 100 MPa should be 10- 12 mm. HPC has following qualities –

- high workability
- high strength
- high modulus of elasticity
- high density
- high dimensional stability
- Low permeability and resistance to chemical attack.

Ultra high strength concrete (UHPC) is developed after the advancement of technology. The strength of UHPC lies between 150 - 800 MPa. The major techniques adopted to prepare UHPC are

- Compaction by pressure which enable densification of concrete and reduction of porosity up to the highest level.
- Helical binding is done by high tensile strength steel wire to produce high strength concrete.
- Polymer concrete is done by impregnation of monomers within the pores appeared over the surface of concrete causing the polymerization of concrete.
- Reactive powder concrete (RPC) is developed for those structures which require high ductility along with high strength. In RPC, conventional sand and aggregates gets replaced by finer sand, quartz powder less than 300 microns, silica fume and steel fibres.

In High performance fibre reinforced cement composite (HPFRCC), steel fibres are added to improve tensile strength of concrete which helps in enhancing of post cracking behaviour of concrete mix. HPFRCC does not contain a coarse aggregate which helps in attaining high strength of concrete. Other advantages of HPFRCC include advantages of steel fibres and high performance concrete. Slurry Infiltrated concrete (SIFCON) and engineering cement composites (ECC) are types of HPFRCC.

1.10 NEED FOR DEVELOPING HPFRCC

With the advancement in construction industries, the need for retrofiiting structures has become priority for civil engineers. Nowadays rehabilitation / retrofitting of structures in nation building are playing a significant role. Sardar sarovar dam, bhakara dam which was constructed in 1963, hoover dam constructed in 1935, bridges, roads, High rise building are some structures which are built once and are then rehabilitated/ retrofitted after or within the service life of structure, so that it could last for some more years without any trouble.

After the introduction of fibres in concrete, it has become possible for engineers to prevent the sudden failure mode of joint by improving the joint behaviour to deform more and dissipate more energy and to make it more compatible with adjoining beam column for transferring loads to successive structural members. In the thesis, HPFRCC was developed due to its requirement of concrete which is easy to develop at normal room temperature without containing any aggregates. The materials used for development of HPFRCC are silica fume, metakaolin, nano silica, steel fibres, etc. The properties of following materials which are used to develop HPFRCC are mentioned in next section.

1.11 OBJECTIVES OF DISSERTATION

The objectives of this thesis are:

1. To develop high performance fibre reinforced cement composite at normal room temperature and to understand the effect of different materials used for its composition.
2. To experimentally investigate the effect on the cyclic behaviour of beam-column joints retrofitted with high performance fibre reinforced cement composite in terms of energy dissipation and energy absorption.
3 To compare the results of stiffness degradation of beam column joint before and after retrofitting the specimens at different stress levels.
4. To draw comparative analysis of developed joint stresses of specimens before and after retrofitting it with HPFRCC.
5. To understand and analyse the cracking behaviour of joint when subjected to seismic motion and draw suitable conclusions.

1.12 STRUCTURE OF DISSERTATION

The complete investigation has been performed under this dissertation which is divided in following five chapters:

Chapter 1 tells brief introduction about beam column joint and about the types of forces acting over the joint used in performing the thesis.

Chapter 2 reviews about the previous research work that are related to current dissertation work such as tests performed with different retrofitting techniques on beam column joint.

Chapter 3 reports about the experimental study conducted on inelastic behaviour of beam column joint. It contains the complete information of specimen description, materials used, followed mix design, casting procedure and type of loading adopted.

Chapter 4 presents the results obtained after testing the specimen in quasi static reverse cyclic loading. More emphasis has been given to represent comparative analysis of joint i.e. results compared between before retrofitting and after retrofitting in terms of hysteresis curve, energy dissipation, energy absorption, stiffness degradation, joint stresses and cracking pattern analysis.

Chapter 5 provides conclusion of the performed dissertation with the efficiency of HPFRCC in retrofitting the specimen.

CHAPTER 2

LITERATURE REVIEW

2.1. BEAM COLUMN JOINT BEHAVIOUR

2.1.1 Chidambaram et al. (2015) reported the behaviour of beam column joint having different concrete composition (hooked end steel fibres, brass coated steel fibre, polypropylene fibre) with Engineered Cement Composites (ECC) were tested under cyclic load to study various parameters such as hysteresis behaviour, ductility response, crack pattern with damage index, energy dissipation with damping characteristics of various specimens. Initially test is conducted on cylinders to determine compressive strength and split tensile strength of various specimens. The hooked end steel fibre used has length of 35 mm, diameter of 0.60 mm and aspect ratio of 60 with tensile strength of 1100 MPa. The water cement ratio was kept 0.45 and 0.5% super plasticizer was added further.

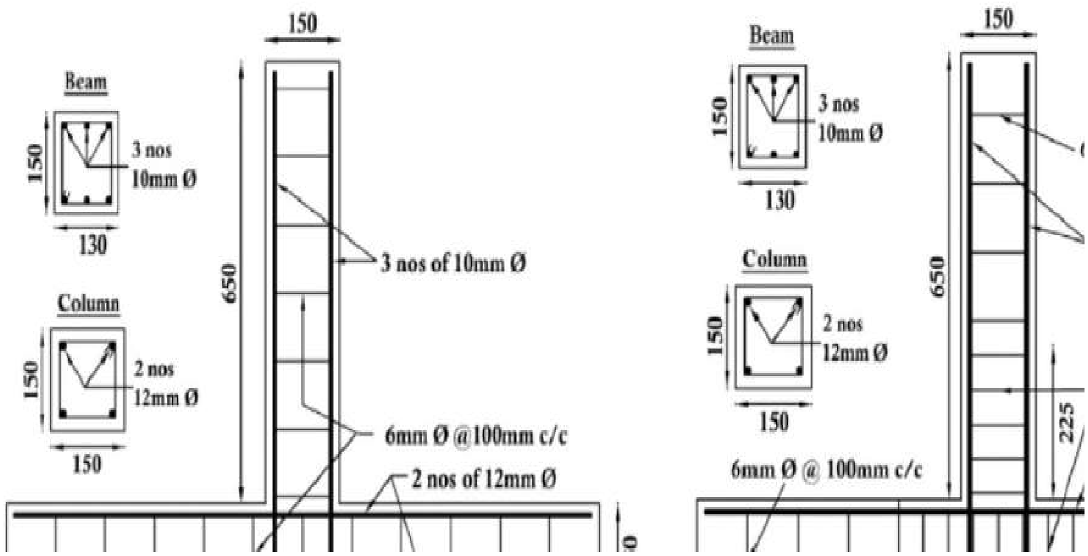
Table 2.1: Cylindrical specimen details and strength comparision.

Specimen ID	Description	Volume of fibre			Cylinder compressive strength (MPa)	Split tensile strength (MPa)
		PP (%)	HSF (%)	BSF (%)		
SC1	Conventional	-	-	-	27	3.8
SC2	SFRC	-	2	-	35	4.8
SC3	ECC	3	-	-	26	4.5
SC4	HECC	1.5	2	-	39	6.5
SC5	BECC	1.5	-	2	33	4.5

Table 2.2: Detailing of joints.

Specimen id	Transverse Reinforcement	Material used	Beam Reinforcement	Column Reinforcement
SJ1	6 mm Ø bar @ 100 mm c/c	Control specimen	3-10 Ø bars at top & bottom.	4 - 12 Ø bars
SJ2	6 mm Ø bar @ 100 mm c/c & 50 mm c/c in the hinge region.			
SJ3	6 mm Ø bar @ 100 mm c/c	SFRC		
SJ4	6 mm Ø bar @ 100 mm c/c	ECC		
SJ5	6 mm Ø bar @ 100 mm c/c	HECC		
SJ6	6 mm Ø bar @ 100 mm c/c	BECC		

Six beam column joints were tested under quasi static test to analyse various hysteresis curve under reverse cyclic loading. Table 2.2 represents detailing of joints and Figure 2.1 represents dimensions of control specimens. Due to close confinement in SJ2 from specimen SJ1, the area under hysteresis curve is increased and indicates increased yielding & post peak load carrying capacity. SJ3 performed better than SJ2 due to presence of steel fibre (bond strength) and SJ4 shows best load carrying capacity when compared to SJ1, SJ2 and SJ3 in terms of post peak deformation (25 – 50%). SJ5 showed similar performance as of SJ4 due to the presence of fibres while SJ6 showed lower load carrying capacity as compared to SJ4, SJ5 but has better post peak load deformation response.



a) SJ1 b) SJ2

Figure 2.1: Control sample dimensions and detailing representation.

2.1.2 Chidambaram et al. (2015) studied performance evaluation of beam column joints with steel fibres reinforced concrete under cyclic loading. Table 2.3 represents reinforcement detailing of beam column joints. The purpose was to compare the behaviour of hysteresis curve, load deformation envelop curve, energy dissipation capacity, failure pattern and damage index by using geo-grid polymer and SFRC. The concrete was prepared using a mix proportion of 1: 1.45: 2.25 of cement: sand: aggregates. The water cement ratio was kept as 0.45 and OPC 43 grade cement was used. The dimensions of joint are same as that of above mentioned R. Shiva paper. The hooked end steel fibres used has length 35 mm, diameter 0.60 mm, tensile strength 1100 MPa.

It was concluded that load deformation pattern after yield of specimen is increased due to the usage of geo-grid polymer and SFRC. The energy dissipation capacity in geo-grid confined specimen performs well with 1% hooked fibres rather than using 2% hooked fibres and geo-grid confined joint specimen has more damage tolerance capacity.

Table 2.3: Specimens reinforcement detailing.

Specimen Id	Transverse reinforcement spacing (mm)	Special configuration	Volume of steel fibres (%)
EJ1	100 c/c	-	-
EJ2	50 c/c in joint (JR)+ 100 c/c in other region (OR)	-	-
EJ3	100 c/c	Geo-grid confinement in hinge	-
EJ4	100 c/c	Geo-grid confinement in hinge	1
EJ5	100 c/c	-	2
EJ6	100 c/c	3 geo-grid layers in beam	-
REJ1	150 c/c JR + 100 c/c OR	Geo-grid confinement in hinge	-
REJ2	150 c/c JR + 100 c/c OR	Geo-grid confinement in hinge	1
REJ3	200 c/c JR + 150 c/c OR	Geo-grid confinement in hinge	-
REJ4	200 c/c JR + 150 c/c OR	Geo-grid confinement in hinge	1

2.2 JOINT STRENGTH COMPARISON WHEN DETAILING IS DONE FROM THREE DIFFERENT CODES

2.2.1 Kiran et al. (2014) studied over the behaviour of 1970s constructed beam column joint with three types of reinforcement detailing. These detailing are ACI 318-1971, old Japanese practice code and old Indian practice code. The comparison in finite element results between ACI and old Japanese code has been shown in Figure 2.2.

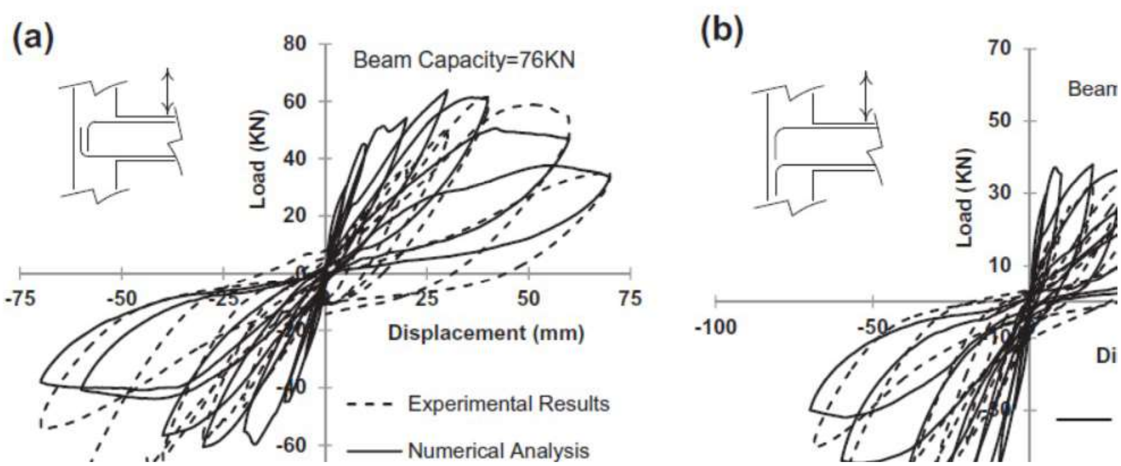
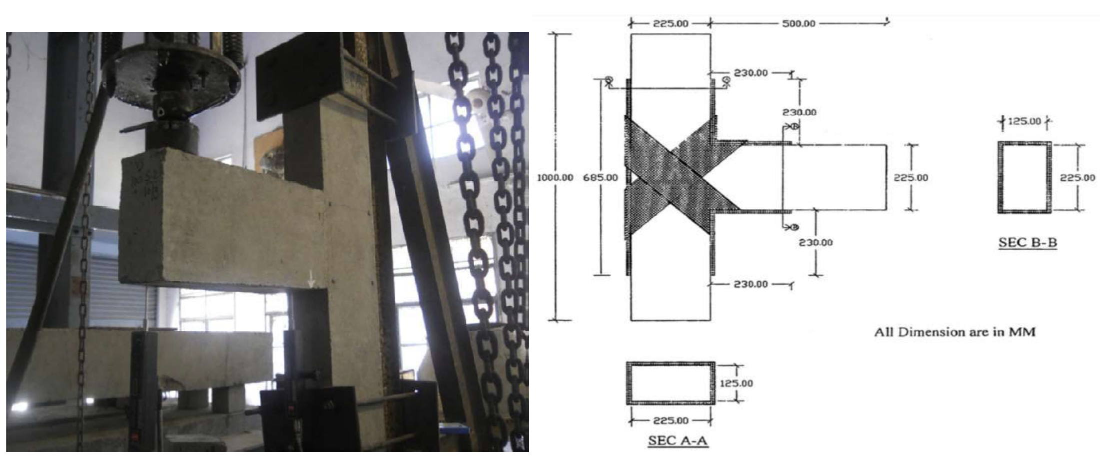


Figure 2.2: Comparison of finite element results with experimental results for joint
a) as per ACI 318-1971 b) old Japanese code.

On comparing monotonic load anchorage response of specimens by three different codes, it was found that joint designed as per Indian code was found to be most brittle while all three joints failed in shear behaviour and due to slippage of bars. Other two types of joint were failed in shear reinforcement. The joint designed as per ACI 318-1971 performed better than Japanese code due to proper core confinement.

2.3 BEAM COLUMN JOINT VARIOUS RETROFITTING/ REHABILITATION TECHNIQUES

2.3.1 Singh et al. (2014) studied the behaviour of RC beam column joints when stressed to three different stress levels, under static loading conditions and then the joint is retrofitted with CFRP jacket. The dimension of joint 225 mm X 125 mm with an overall length of column as 1 m and beam has the same dimension with a length of 0.5 m. The grade of concrete is M20 and the steel used is Fe500. A total of 9 RC joints were prepared with 4- 10 mm DIA longitudinal bars used in column and 2-10 DIA bars provided in tension side of beam and 2-8 DIA bars provided in compression side of beam with lateral ties of 6 mm DIA provided at 100 mm c/c throughout the joint as per IS 456:2000 code. Within the 9 specimens, 3 were used as control specimen & loaded up to ultimate failure load while the remaining specimen were grouped in 3 parts, each having two specimens which were stressed up to 50% ultimate load, 85% ultimate load and 100% ultimate load respectively. Figure 2.3 represents the set up arrangement and the CFRP wrapping pattern of the specimen.



a) Loading frame set up. b) CFRP retrofitting.

Figure 2.3: Experimental set up and retrofitting representation.

The joint was then retrofitted with CFRP in two layers of 500 mm wide sheet & 0.117 mm thickness. For proper bonding between CFRP sheet & specimen surface, blue pigmented epoxy resin was used. The specimen was then cured for 7 days and again testing was done until its ultimate failure load. From the results represented in Table 2.4, it was observed that the average ultimate loads of controlled specimens were found to be 22.68 KN with a maximum deflection of 23.24 mm at free end of cantilever beam. With the application of CFRP jacket in beam column joint, improvement in ultimate load carrying capacity is observed that is 9.47%, 11.99% and 7.76 % respectively for different stress conditions.

Table 2.4: Representation of joint results at different stress levels.

Stress level (%)	Beam column design	Pu (KN)	ΔY (mm)	ΔU (mm)	Ductility ratio	Stiffness
-	Control sample	22.68	8.38	23.24	2.77	4.34
100	SL-1	24.83	12.40	22	1.77	1.67
85	SL-2	25.40	11.33	22.66	2	1.96
50	SL-3	24.44	12.52	20.12	21.61	2.12

A similar trend is observed in stiffness improvement that is 17.36% and 26.94% in retrofitted specimen when stressed from 100% to 85% & 100% to 50% of ultimate load.

2.3.2 Shannag et al. (2005) reported six interior beam column joint specimens casted and retrofitted using high performance fibre reinforced composites up to 200 mm length in beam and column from the face of joint. Testing was done for ultimate strength, ductility, energy dissipation capacity and joint stiffness behaviour under reverse cyclic loading. Four specimens were retrofitted using 2% high strength steel fibres in the joint region, one specimen was used as control specimen and last specimen has column main reinforcement lapped- spliced after the joint region. they observed that hooked steel fibres displayed three times increased load carrying capacity and initial secant stiffness, two times increase in ductility and twenty times increase in energy dissipation capacity.

2.3.3 Holschemacher et al. (2010) reported 6 out of 8 retrofitted non seismic joint with different configurations of CFRP sheet to determine improvement in lateral load bearing capacity and ductility. The various ways for wrapping CFRP were T-shape, L shape, X-shape and strip combinations. From the results, it was observed that the strength was increased up to 17.5% and ductility increase was found up to 5.3%. The best & worst results came out with X-shaped and L-shaped wrapping methodology. He also reported that more the layer of CFRP more will be the improvement in strength and ductility.

2.3.4 Saber et al. (2017) studied the effect of macropolymeric and polypropylene fibres on nano silica and silica fume. He prepared 280 specimens which were divided into 28 groups to determine compressive strength, modulus of elasticity, tensile strength, porosity and water absorption of concrete. Properties of fibres are mentioned in Table 2.5.

Table 2.5: Properties of fibres

FIBRE TYPE	SHAPE OF FIBRE	LENGTH (mm)	DIAMETER (mm)	ASPECT RATIO	ELASTIC MODULUS (GPa)	TENSILE STRENGTH (MPa)
MP	CRIMPED	39	0.78	50	3.6	500
PP	STRAIGHT	12	0.019	631	3.5	350

Through his studies he concluded that a polypropylene fibre reduces the slump (workability) of concrete while nano silica and silica fume addition further reduces the workability of concrete. He observed 8 and 11% improvement of compressive strength on adding macro polymeric and polypropylene fibres to concrete. 2% nano silica and 12% silica fume showed good influence in compressive strength of concrete while 3% nano silica and 10% silica fume showed good influence in tensile strength of concrete.

2.3.5 Ghobarah et al. (2005) reported seismic rehabilitation of non seismic exterior beam column joint with steel plate, CFRP, GFRP. The author figured out a splitting crack was formed at joint region which was rehabilitated with angle bars. Another joint rehabilitated with CFRP sheets showed improvement in tensile strain behaviour up to three times with wide crack formation at junction of beam column joint while corresponding control specimen failed at lower load with shear and bonding type failure. Next joint exhibit bond slippage of existing beam reinforcement and the best behaviour was shown by joint which was rehabilitated with GFRP sheet as it showed no debonding. Overall the failure mode was improved from brittle failure to ductile failure by increasing load carrying capacity and tensile strength of joint.

2.3.6 Barakat et al. (2002) conducted experiment on five beam column joint specimens which were tested under cyclic loading. The failed specimens were repaired by HPFRCC jacket. The repaired specimen performed better in load carrying capacity than its prior testing. The repaired specimens displayed better flexural capacity on comparing with joints earlier testing which helped the joint in plastic hinge formation and more ductile performance. The energy dissipation capacity of repaired specimen was much more than reference specimen.

2.4 DEVELOPMENT OF UHPC/ HPC/ HSC/ HPFRCC

2.4.1 Effect of Silica Fume and Metakaolin in UHPC

2.4.1.1 Duan et al. (2013) studied the effect of silica fume and metakaolin on interfacial transition zone and compressive strength of concrete. Techniques employed were Mercury Intrusion Porosimetry (MIP), micro hardness testing, Scanning Electronic Microscopy (SEM) at 28 and 180 days. Mix proportioning of concrete are given in Table 2.6.

Table 2.6: Mix proportions of concrete (kg/m^3)

SERIES	CEMENT	SLAG	SILICA FUME	METAKAOLIN	WATER	FINE AGGREGATE	COARSE AGGREGATE
NORMAL	360	0	0	0	180	681	1160
SL	324	36	0	0	180	681	1160
SF	324	0	36	0	180	681	1160
MK	324	0	0	36	180	681	1160

He concluded that the ITZ becomes denser with the addition of mineral admixtures and more refined internal structure is obtained. The results of Metakaolin produces better internal structure than silica fume as it has higher ratio of micro morphology which helps in attaining more compressive strength. The fine particles present in metakaolin helps in filling the gap between cement particles resulting in formation of denser ITZ.

2.4.2 Effect of Nano Silica in UHPC

2.4.2.1 Yu et al. (2014) performed experiment on the effect of nano silica on the microstructure development of ultra high performance concrete in which nano silica was added to different percentages as a replacement of cement. Mix design for uhpc is mentioned in Table 2.7.

It was concluded that by the addition of nano silica water demand of mix increases. It also help in decrease in slump value of UHPC which is due to the fact that viscosity of mix increases with the addition of nano silica and entrapped air within mix finds it difficult to escape. Nano silica shows nucleation effect which directly promotes hydration of cement by forming more CSH gel. So optimum amount of nano silica has to be utilized for getting highest strength. Figure 2.4 compares compressive strength of concrete with various nano silica content after 3, 7 and 28 days.

Table 2.7: Mix design for uhpc.

(kg/m^3)	REF.	UHPC1%	UHPC2%	UHPC3%	UHPC4%	UHPC5%
CEMENT	439	435.1	430.7	426.3	421.9	417.5
LIMESTONE	263.7	263.7	263.7	263.7	263.7	263.7
QUARTZ	175.9	175.9	175.9	175.9	175.9	175.9
MICROSAND	218.7	218.7	218.7	218.7	218.7	218.7
SAND	1054.7	1054.7	1054.7	1054.7	1054.7	1054.7
NANO SILICA	0	4.4	8.8	13.2	17.6	22
WATER	175.8	175.8	175.8	175.8	175.8	175.8
SUPERPLASTICIZER	43.9	43.9	43.9	43.9	43.9	43.9

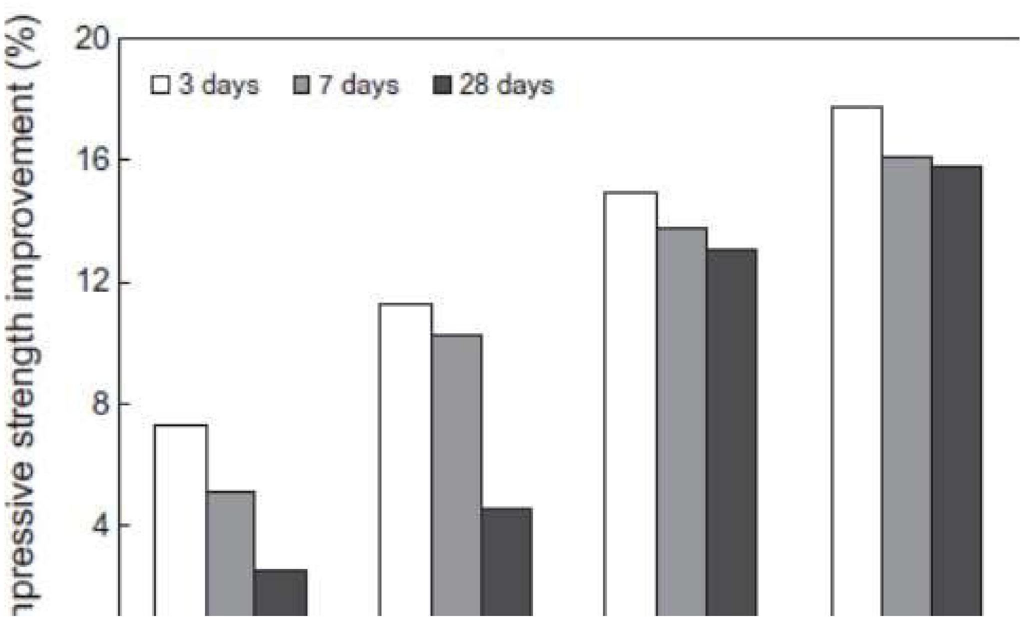


Figure 2.4: Compressive strength of concrete with various nano silica content at 3, 7 and 28 days.

2.4.3. Effect of Steel Fibres in UHPC

2.4.3.1 Yu et al. (2016) studied the effect of steel fibres on dynamic strength of concrete in which 270 specimens were tested in split Hopkinson pressure bar (SHPB). The test was conducted with 4 different steel fibres which has difference in distribution, orientation and densities. The properties of steel fibres are mentioned in Table 2.8.

Table 2.8: Properties of fibres.

FIBRES	M.O.E. (GPa)	TENSILE STRENGTH (MPa)	LENGTH (mm)	DIAMETER (mm)	ASPECT RATIO
MF06	210	4295	6	0.12	50
MF15	210	4295	15	0.12	125
TF03	210	1500	30	0.30	100
TF05	210	1500	30	0.50	60

He concluded that fibre length and aspect ratio has positive effect on concrete dynamic strength and they directly affect the UHPC strength achievement i.e. micro fibres are comparatively less sensitively in bearing higher loading rate than that of long fibres. The maximum compressive strength came out with the MF15 as 145 KN while for MF6, TF3 and TF5 strength came out as 114, 132 and 113 KN respectively.

2.4.3.2 R. Siva et al. (2015) investigated the effect of flexural & shear behaviour of geo-grid confined RC beams of length one meter with steel fibres. Here 24 beams having different configurations were divided into three types are tested under 3 point monotonic loading.

Table 2.9: Different specifications of beams.

Specimen ID	Longitudinal reinforcement		Transverse reinforcement		Steel fibres (%)	Description
	Top (mm)	Bottom (mm)	Size (mm)	Spacing (mm)		
Type A	Confined					
B1	2-8Ø	2-10Ø	6Ø	150	-	Control beam-I
B2	2-8Ø	2-10Ø	6Ø	150	1	SFRC beam-I
B3	2-8Ø	2-10Ø	6Ø	150	0.5	Geo-grid with SFRC-I
Type B	Moderately confined					
B4	2-8Ø	2-10Ø	6Ø	250	-	Control beam-II
B5	2-8Ø	2-10Ø	6Ø	250	1	SFRC beam-II
B6	2-8Ø	2-10Ø	6Ø	250	1	Geo-grid with SFRC-II
Type C	Lightly confined					
B7	2-8Ø	2-10Ø	6Ø	450	-	Control beam-III
B8	2-8Ø	2-10Ø	6Ø	450	1	SFRC beam-III
B9	2-8Ø	2-10Ø	6Ø	450	0.5	100 KN/m geo-grid +SFRC
B10	2-8Ø	2-10Ø	6Ø	450	0.5	200 KN/m geo-grid +SFRC
B11	2-8Ø	2-10Ø	6Ø	450	1	100 KN/m geo-grid +SFRC
B12	2-8Ø	2-10Ø	6Ø	450	1	200 KN/m geo-grid +SFRC

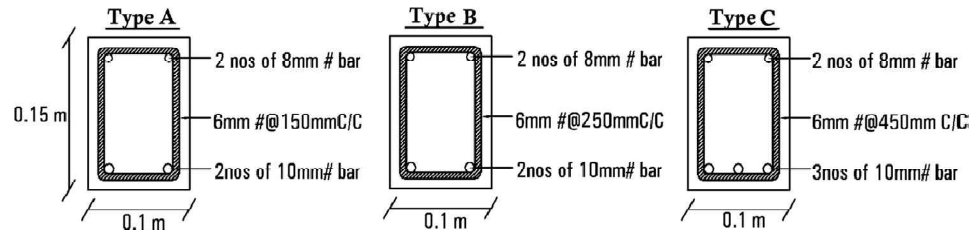


Figure 2.5: Dimensions and detailing of different specimens.

It has been observed from the load deflection curve, specimens B2 and B5 shows higher increase in post yielding deflection when compared to B1 and B4 while B3 is capable in restricting the crack propagation when compared to B2. The average ultimate load of B6 is at least 20% more than B4 and B5. 50% shear strength has been increased in B8 when compared from control specimen (B7). However both (B7, B8) failed in shear. B11 shows higher shear strength when compared with B7 which is due to the presence of 1% steel fibres and geo-grid confinement. Increase in energy dissipation capacity and ductility was found to be 3-4 times. The entire summary of experiment is summarized in Table 2.10.

Table 2.10: Test results

Specimen ID	Yield stage		Maximum stage		Ultimate stage		Ductility factor (μ)	Energy dissipation (KN-mm)
	P_y (KN)	Δ_y (mm)	P_m (KN)	Δ_m (mm)	P_u (KN)	Δ_u (mm)		
B1	48	3.4	49.5	9.1	45.03	12.9	3.8	540
B2	49	3.2	52.2	30.1	48.32	36	11.3	1704
B3	51	4	52.3	32.9	42.31	45	11.3	1976
B4	50	3.2	51.4	11.7	46.61	13.3	4	584
B5	51	3.5	54	13	45	25	7.1	1194
B6	58	4	58.9	16.4	53	34	8.5	1856
B7	44.1	3.1	45	4.26	40.86	4.5	1.4	150
B8	63.8	5	64.6	2.254	57.85	5.8	1.2	230
B9	47.8	3.6	53.5	6.223	43.61	8.8	2.5	354
B10	45.3	3.4	51.5	8.356	41.85	12.4	3.8	538
B11	70	4.9	73.2	7.229	63.21	11.5	2.4	649
B12	74.1	4.4	79.8	8.1	70.25	15.8	3.6	1056

2.4.3.3 Holshemacher et al. (2010) studied the effect of steel fibres on small beam (15×15×70 cm). Three types of fibres were used as shown in Table 2.11.

Table 2.11: Steel fibres details.

DESCRIPTION	TYPE 1 (F1)	TYPE 2 (F2)	TYPE 3 (F3)
SHAPE	STRAIGHT	STRAIGHT	CORRUGATED
CROSS SECTION	CIRCULAR	CIRCULAR	CIRCULAR
ANCHORAGE	HOOKED ENDS	HOOKED ENDS	CONTINUOUS
LENGTH (mm)	50	50	50
DIAMETER (mm)	1	1	1
ASPECT RATIO	50	50	50
TENSILE STRENGTH (MPa)	1100	1900	1100

A total of 18 beams were casted. Out of which 6 beams was 1% SFRC and other 6 with 0.25% SFRC, remaining beams were without SFRC. Four point bending test were conducted on beams. The results obtained shows the maximum compressive strength came out with SFRC with maximum reinforcement (60 kg/m^3) whereas the maximum split tensile strength came out with SFRC with minimum reinforcement (20 kg/m^3). However all specimens showed improvement in load carrying capacity in post cracking range by increasing fibres content. With the increase in fibres content initial crack load was found to be increased which shows clear dependence of fibre content on load bearing capacity of beam. The high strength fibres demonstrated improved ductile capacity of beam and more load bearing capacity. Broken high strength fibres were found in less numbers in failed specimens. Specimens with least fibre content (20 kg/m^3) failed more in shear and lesser in compression. Specimen with average fibre content (40 kg/m^3) failed more in compression and lesser in shear. Specimen with most fibre content (60 kg/m^3) failed mainly in compression.

2.4.3.4 Zemei et al. (2017) studied the behaviour of hybrid steel fibre in ultra high performance concrete. The behaviour is related to static and dynamic properties. Quasi static test was conducted on uhpc. The concrete was prepared by adding fibres of length 6 & 13 mm length with volume of 2% volume of concrete. Dynamic properties were determined by split Hopkinson press bar (SHPB) test. He determined that flowability on concrete was decreasing with the addition of steel fibres. This was found due to more surface area of fibres which requires more water for developing sufficient bonding. The worst flowability was given by mix which used 2% steel fibres of 13 mm length. The steel reinforcement was found effective in restricting transverse deformation of specimen. Short steel fibres prevented the formation of microcracks and long steel fibres helped in bridging the macro cracks. The Dynamic increase factor (DIF) for all specimens were greater than 1 which suggests dynamic compressive strength was more than static compressive strength. Under impact loading, pulverization of concrete was observed which was better than local failure mode. However, author concluded that 1.5% long steel fibres and 0.5% short steel fibres displayed best static and dynamic properties of ultra high performance concrete.

2.4.4 Effect of Steel Fibres when used with Nano Silica and Silica Fume in UHPC

2.4.4.1 Farid et al. (2017) invested the effect of hooked end steel fibres on properties of high strength of concrete with addition of silica fume and nano silica. The fibres he used was 50 mm length, 0.8 mm diameter, and 1400 MPa tensile strength. He concluded that steel fibres was useful in developing compressive strength of HSC due to high tensile strength and modulus of elasticity of steel fibres. The effect on addition of silica fume in association with nano silica exhibits improvement in compressive strength of concrete. With 1% steel fibres, 3% nano silica, 8% silica fume the compressive strength improves upto 18.7%. the effect on adding hooked fibres to high strength concrete helped in improving split tensile strength of concrete while silica fume and nano silica helped in improving tensile strength of concrete. On further addition of silica fume and nano silica, the concrete shows reduction in water absorption due to the filling of concrete pores. This will eventually lead to decrease in compressive strength of concrete on comparing it to normal concrete. The results obtained are represented in Table 2.12.

Table 2.12: Specimen test results

MIX NO	MIX ID	COMPRESSIVE STR.	SPLIT TENSILE STR.	ELASTIC MODULUS
1	PLAIN	55.15	5.52	39986
2	ST0.5	63.08	7.99	43122
3	ST0.75	68.48	9.13	43844
4	ST1.0	62.02	9.21	43440
5	ST1.25	58.22	9.34	42938
6	ST1.5	58.09	10.43	43588
7	ST1SF8	73.72	10.29	47297
8	ST1SF10	70.98	10.22	48395
9	ST1SF12	73.09	9.63	46285
10	ST1NS1	62.21	9.03	47867
11	ST1NS2	66.18	9.97	47162
12	ST1NS3	67.43	9.06	48666

CHAPTER 3

MATERIALS AND METHODOLOGY

3.1 GENERAL

In RC framed structure, joints have always been critical in transferring forces from beams to columns. Since these joints become vulnerable under earthquake loading due to its inability to transfer compressive, tensile and shear forces. The emphasis of this chapter in the dissertation is on the materials properties used to prepare the concrete specimen and to retrofit it. This chapter also contains the HPFRCC development technique, adopted mix design, casting of specimen and the experimental procedure for testing the specimen.

3.2 MATERIALS FOR CASTING CONCRETE SPECIMEN

3.2.1 Steel Bars

The bars used for the reinforcement in the casted beam column joint were TATA TISCON Fe500. TISCON 500 is high strength ribbed TMT bar where 500 refers to the strength of rebar in MPa. Steel bars are made by TATA confirming to latest standards (last revised 2012) set by Bureau of Indian Standards. The ultimate tensile strength of bar is 540 MPa and yield strength of 500 MPa conforming to IS 1786:2008.

3.2.2 Cement

Cement is a basic material that is used for building construction. It helps in making concrete which after setting, adheres to other building materials and becomes very hard. It is used to bind sand and aggregate of concrete. It affects the compressive strength of concrete than any other materials. The standard properties of cement are represented in Table 3.1.

Table 3.1: Standard properties of cement.

Characteristics	PPC	OPC33	OPC43
Fineness, m^2/kg, min	320	225	225
Initial setting time, minutes, min	30	30	30
Final setting time, minutes, max	600	600	600
Compressive strength, MPa, 3days	16	16	23
MPa, 7days	22	22	33
MPa, 28days	33	33	43

Portland pozzolonic cement (PPC) is used for casting of specimen. The pozzolonic material used in PPC is fly ash conforming to IS 1489 part I of 1991. PPC usually improves pore size distribution than OPC as it is finer. Thus it reduces the formation of microcracks at transition zone. The strength of PPC after a couple of months is higher than OPC 33. This is due to the continuity of pozzolonic reaction within the concrete. Nowadays PPC is used in hydraulic structures, marine structures, sewers and sewage disposal units and in mass concreting works like dams, bridges and foundation works, etc. Cement used for casting beam column joint is PPC fly Ash based and the cement used for retrofitting specimen is OPC 43 grade (IS 8112:2013). The properties of cement are represented in Table 3.2.

Table 3.2: Cement Properties tested in Lab.

Properties	Practical values of PPC	Practical values of PPC	Theoretical
Specific Gravity	3.18	3.16	Minimum 3.15
Initial setting time	40 minutes	36 minutes	Minimum 30 minutes
Final setting time	5 hours	5 hours	Maximum 10 hours

3.2.3 Aggregates

Aggregates are the building material used to mix cement, lime, gypsum, bitumen and some other adhesive materials to form concrete. These are obtained from igneous, sedimentary and metamorphic rocks or manufactured from blast furnace slag, etc. It mainly provides stability, resistance to wear & tear and increases the volume to concrete. It also reduces the shrinkage effect Aggregate possess several properties to assess its quality like particle shape, size, texture, specific gravity, water absorption, bulk density, moisture content, strength, toughness, hardness, etc. The tested properties of coarse aggregates and fine aggregates (sands) have been mentioned in Table 3.3 which is further used to prepare the design mix of concrete.

Table 3.3: Properties of Aggregates

Properties	Coarse Aggregate	Fine Aggregate
Code	IS 383	IS 383
Specific gravity	2.68	2.65
Fineness modulus	3.2	2.78
Maximum size aggregate used	Up to 20 mm aggregate	Passing through 4.75 mm is sieve
Water absorption capacity	0.5% by weight of coarse aggregate	0.8% by weight of fine aggregate

3.2.4 Water Content

Water has two important roles in making of concrete i.e. in fresh concrete mixing & in curing purpose. To fulfil strength development requirement and durability criteria, water should be free from impurities like acids, oils, sugar, salts, alkalis and organic materials. The main purpose of adding water to cement is to cause hydration of cement and lubricates both coarse and fine aggregates to produce workable concrete. In case of excess water, cement and water comes to the surface of concrete by capillary action and after evaporation of water from concrete, it becomes porous. In other case when water is less in concrete, mixing of concrete becomes very difficult results in non uniform mixing. The most suitable water for concreting is portable water i.e. water used for human consumption. However chemical analysis of water is checked by IS 3025 (Part 17-32) and pH of water should be more than 6.The water used in casting is tap water available in laboratory.

3.3 MATERIALS USED FOR DEVELOPMENT OF HPFRCC

3.3.1 Silica Fume

Silica fume is a by-product of smelting of metal silicon or ferrosilicon. SiO_2 is produced on reducing quartz at temperature up to 2000 °C. This SiO_2 vapours oxidises and condensed at low temperatures. Condensed silica fume has more than 90% silica fume in crystalline form. Silica fume is also called as silica dust, micro silica or condensed silica fume. Silica fume is extremely fine spherical particle with specific surface area of 21000 m^2/kg and average diameter of 0.1 μ which is about 100 times more than cement particles. Silica fume is a grey or white coloured powder, somewhat similar to PCC or fly ash as shown in Figure 3.1. It has both pozzolanic and cementitious properties. Silica fume primarily works as pozzolonic admixture which is found useful in improving mechanical properties to large extent.

Figure 3.1 (a): Silica fume powdered form

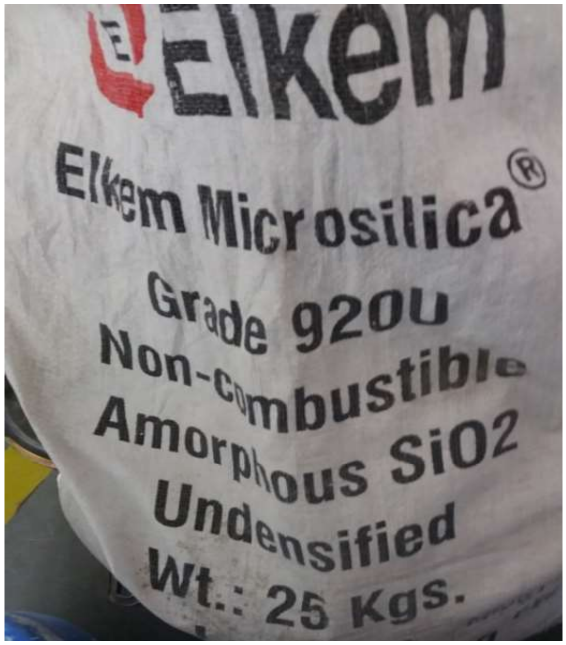


Figure 3.1 (b): Silica fume bag.

Figure 3.1: Silica fume

The specification of silica fume is written in IS 15388:2003. By using superplasticizer and silica fume together it becomes comparative easier to obtain compressive strength up to 90-120 MPa. On adding silica fume to concrete durability of concrete is improved due to its refined structure as the pores gets filled. It also reduces calcium hydroxide content so that it becomes more resistive to sulphate attack and attains higher protection to the reinforced bars. Silica fume is normally available in market in two forms: namely wet & dry form, these are stored in tanks, silos and hoppers. The particles of silica fume are extremely fine, with about 96% of particles are finer than 1 µm. Colour of silica fume is either white or grey. Silica fume used in the development of high performance concrete is in uncondensed form.

Table 3.4: Silica fume properties (manufacturer data)

Properties	Values
Particle size (µm)	<1
Bulk density	
As produced (kg/m^3)	140-440
Slurry (kg/m^3)	1330-1450
Densified (kg/m^3)	475-725
Specific gravity	2.22
Surface area (BET) (m^2/kg)	12000-30000
Standard slurry pH	4.7

Silica fume is mainly available in four types

Table 3.5: Types of silica fume and its density (M.S. Shetty 1982)

Types	Bulk density (kg/m^3)
Undensified	200-300
Densified	500-600
Micro pelletised	600-900
Slurry	1400

3.3.2 Metakaolin

Kaolin is a fine, white, clay mineral. It is widely used in the manufacturing of porcelain. Kaolin is used as raw material for manufacture of metakaolin. Metalaolin is a type of pozzolonic material. Metakaolin is produced by dehydroxylation of the clay mineral kaolinite. This is formed by calcinations of kaolinite clay at temperature of 500 to 800°C. In the initial phase of heating from 100 to 200°C, most of adsorbed water is lost from clay and on higher temperature it undergoes dehydroxilization which is an endothermic process. Above this temperature range kaolinite becomes metakaolin and further properties are improved as per requirement of metakaolin. Image of used metakaolin has been shown in Figure 3.2 while the properties of metakaolin provided by manufacturer are shown in Table 3.6 and Table 3.7. The code in which methods for test in pozzolonic material is given is IS 1727:1967.

Figure 3.2: Metakaolin

Metakaolin has smaller particle size with average particle size is of 3 µm. it is off-white in colour. The main constituent materials for metakaolin are silica oxide (SiO_2) and alumina oxide (Al_2O_3) while other components are present in small quantity. Metakaolin is prepared as per specifications of ASTM C618. In PCC, Metakaolin normally reacts with CaOH at normal temperature to form CSH, C_2ASH_8 and C_4AH_{13}.So this reaction helps in reducing the porosity and the refinement of pores in terms of structure, strength and impermeability of cement paste.

Table 3.6: Typical chemical composition of metakaolin (manufacturer's data)

Ingredients	Values (%)
SiO_2	51.55
Al_2O_3	40.13
Fe_2O_3	1.22
CaO	2.0
MgO	0.13
K_2O	0.54
SO_3	-
TiO_2	2.26
Na_2O	0.07
L.O.I	2.02

Table 3.7: Physical properties of Metakaolin (manufacturer's data)

Properties	Values
Average particle size	<3 μm
Specific gravity	2.5
Fineness (m^2/kg)	12000

3.3.3 Sand

Sands act as fine aggregates in concrete or mortar. Sands are classified depending upon source, mineralogical composition, size of the particle and particle size distribution. Here two

Figure 3.3: Ennore sand carry bag.

two types of sand are used. One sand is used for casting beam column joint while other sand is used for making high performance concrete. For casting beam column joint, sand of zone II was used and Ennore sand is used for making high performance concrete. This Ennore Sand is granular form of silica. In India normally standard sand is available (IS:650) which are referred as Ennore sand. It has quartz, light grey or white in colour and is silt less and fineness modulus varies from 2.2 to 2.6. Ennore sand of grade III is used for retrofitting purpose. The composition of ennore sand is mentioned in Table 3.8.

Table 3.8: Properties of ennore sand (manufacturer's data)

Properties	Ennore Sand
Physical Properties	
Colour	Greyish White
Specific Gravity	2.65
Water absorption	0.8%
Shape of Grains	Sub Angular
Chemical Properties	
SiO_2	99.3%
Fe_2O_3	0.1%
Petrography Analysis	
Quartz	97.4%
Feldspar	2.5%

3.3.4 Admixtures

Admixtures are additional products added to a concrete mix to alter the properties of concrete at fresh & hardened state. Various guidelines are given in IS 9103:1999 for selection of admixture. Admixtures are of two type chemical or mineral type. Chemical type admixture is found in liquid form whereas mineral type is found in fine granular form. These are mainly used to produce high strength or high performance concrete. For preparing high performance concrete, high range water reducing superplasticizer is used namely "Master Glenium sky 8234, batch no - 43/15" which helped in attaining high strength. It helps in

- Elimination of vibration and reduced labour cost in placing.
- Has improved early and ultimate strengths.
- Improved adhesion to reinforcing and stressing steel.
- Better resistance to carbonation and other aggressive atmospheric conditions.

- Lower permeability and increased ductility.
- Reduces creep and shrinkage.

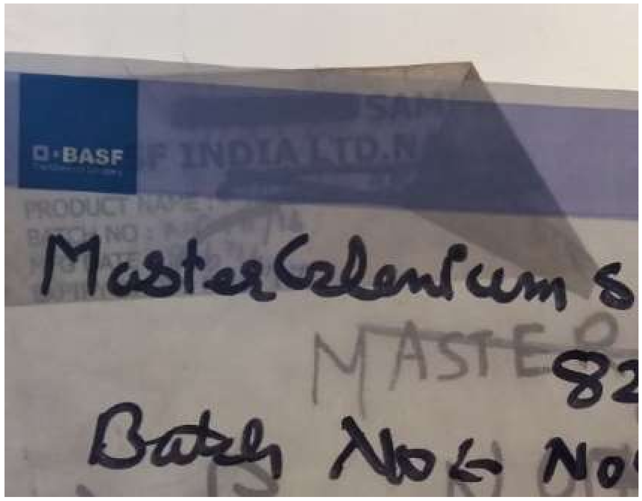


Figure 3.4: Superplasticizer used.

Table 3.9: Types of admixtures.

Chemical Admixture	Mineral Admixture
Accelerators – Used to develop early strength of concrete. e.g. Repair work	They either modify the properties of concrete reduces the cost of concrete. They have pozzolonic properties. Some of them are –
Retarders – mainly sugar, to delay setting time of concrete (by reducing generation of heat). e.g. Ready mix concrete.	Fly Ash - Ash precipitated electro statically or mechanically from exhaust gas in coal fired power plants, conforming to Grade 1 of IS 3812
Water Reducers – are plasticizers, to achieve high strength concrete & to improve workability by producing flowing concrete.	Ground Granulated Blast Furnace Slag – They produces concrete having more resistance to chemical attack, conforming to IS 12089,
Superplasticizers – are high range water reducer, superior to water reducer. It has same purpose as of water reducer.	Silica fume – these are used for producing high-strength and high-performance concrete as the form dense concrete.
Air Entraining Agents – are organic compounds, used to protect concrete ill effects of alternate freezing and thawing.	
Bonding Admixtures – polymer emulsions or latexes, used to bond old or hardened concrete to fresh concrete. e.g. in repair works.	

3.3.5 Nano Silica

Colloidal silica in nano metric size of silica particle is referred as nano silica. Due to the development of new and fine pozzolanic materials, the productivity of high strength concrete, high performance concrete and ultra high performance concrete has gained more attention, for producing new concrete with enhanced properties. Nano silica is also one of those materials which have been developed for improving properties of concrete. These materials are of great importance in research due to their stability, low toxicity and ability to be functionalized with a range of molecules and polymers. Nano silica particles are divided into P-type and S-type according to their structures. The S-type particles have a comparative smaller surface area. The P type nano silica particles exhibit a higher ultraviolet reflectivity when compared to S-type. Silicon dioxide nano particles appear in the form of white powder having density 2.6 g/cm^3. The melting point and boiling point of nano silica are 1600°C and 2230°C.

3.3.6 Steel Fibres

These fibres are drawn from filaments of wire which are further deformed as per requirement and cut to suitable length. These are cold drawn wires fibre which are corrugated and given required shape. These fibres are used as reinforcement in concrete or mortar to improve its properties and make it more durable. There are different types of fibres available in market as per their manufacturing process such as cold drawn, slit sheet, melt extract, mill cut.

Steel fibre finds its application in different fields of engineering such as cellar walls slabs, vibrocompacted piles or pavements, jointless floors, industrial floors, foundation slabs, suspended ground slabs, composite slabs, high strength concrete, ultra high performance concrete, etc.

Steel fibres used in development of HPFRCC are of two types, namely hooked and crimped fibres which have properties as mentioned in Table 3.10 and image is shown in Figure 3.5.

Table 3.10: Specifications of Steel fibres Used (manufacturer data)

Fibres	Length (mm)	Diameter (mm)	Aspect Ratio	Tensile strength (MPa)
Hooked	60	0.75	80	1350+
Crimped	30	0.60	50	1350+

a) Crimped fibres b) Hooked fibres

Figure 3.5: Representation of fibres used.

3.4 MIX DESIGN FOR M-20 CONCRETE

Cement, sand, aggregate were added in ratio 1:1.60:2.61 by weight with the ratio of water to cement as 0.44. Casted cubes were demoulded after 24 hours and were cured in water tank for a period of 28 days. The 7 & 28 days average compressive strength of standard cube of 150 mm dimension was found to be as 18 MPa & 27 MPa respectively.

Figure 3.6: Cubes taken out from its mould.

3.5 HPFRCC DEVELOPMENT

HPFRCC was prepared in the digi mortar mixer (AIM 412-4 with 5 litre nominal capacity, as specified in IS: 4031) available in the laboratory. Samples were prepared on trial basis by varying the quantity of materials such as OPC 43 grade, ennore sand, silica fume, metakaolin, quartz, nano silica, PCE based superplasticizers, hooked and crimped fibres, w/b ratio. The properties of materials used for preparing HPFRCC have been mentioned section 3.3. Based upon the suitable strength of HPFRCC, best suited concrete is selected for retrofitting of all tested beam column joint.

Figure 3.7 (a): Preparation of HPFRCC in digi mortar mixer.   Figure 3.7 (b): Measurement of flowability.

Figure 3.7 (c): Casting of cubes.

Mixing of materials is done continuously for a period of 15- 20 minutes with the addition of water, nano silica and superplasticizer in 3 – 4 cycles at an interval of 40 – 60 seconds. These fibres can be added after the batching of concrete as outlined by ASTM C 94 (10-15 minutes at mixing speed or approximately 50-60 revolutions per minutes). After the fibres are added it

Figure 3.7(d): Testing of cube.

Figure 3.7: Preparation of HPFRCC.

should be noted that mixed rotates at normal speed and fibres are added slowly so that clump (balling effect) should not form. The use of internal or external vibrator (including vibrator screeds) is recommended. Figure 3.7 represents the procedure followed for making HPFRCC in digi mortar mixer. Figure 3.7 (b) represents an image measuring the flowability. Figure 3.7 (c) represents the cubes after immediate casting which is further demoulded after 24 hours and kept in water tank for curing purpose. The representation of testing of concrete cube is depicted in Figure 3.7 (d). Based on different materials, efforts for making HPFRCC were started. The expectation for development of HPFRCC was to develop a concrete which contains both hooked fibres and nano silica. During the initial trial samples, strength was achieved but due to lower percentages of hooked fibres concrete won't show binding action in retrofitting. In further trial samples hooked fibres and crimped fibres were increased and kept constant by varying the percentages of nano silica from 2 to 4 percent. Testing was done in sequence once for metakaolin and then for silica fume. However the ultimate strength achieved was with silica fume. The strength of trial samples (70.5 mm cube dimensions) are primarily obtained after 7 & 28 days. A summary of tested samples for high performance concrete is presented below in the Table 3.11. From the strength obtained after 28 days the best combination came out with sample 10 which is used further for retrofitting the specimen.

Table 3.11: Composition of HPFRCC.

SP*	CEM* (Kg/m^3)	E.S* (Kg/m^3)	QTZ.* (%)	S.F* (%)	M.K* (%)	N.S* (%)	H.F* (%)	C.F* (%)	S.P* (%)	W/B ratio	7DS* (MPa)	28DS* (MPa)
1	800	800	10	25	15	2	0.32	0.21	1.64	0.185	70	-
2	800	800	10	25	15	3	0.166	0.11	1.72	0.185	72	88
3	800	800	11	25	15	4	0.166	0.11	1.66	0.183	55	-
4	800	879	-	-	22	-	-	0.308	2.575	0.195	76	-
5	800	879	-	-	22	2	-	0.308	2.575	0.192	46	70
6	800	879	-	-	22	3	-	0.308	2.575	0.192	55	69
7	800	879	40	25.26	-	-	-	0.308	1.815	0.19	67	79
8	800	879	40	25	-	-	1.5	1	1.905	0.18	80	88
9	800	879	40	25	-	2	1	1	1.8	0.19	62	90
10	800	879	40	25	-	3	1	1	1.785	0.19	68	105
11	800	879	40	25	-	4	1	1	1.775	0.19	56	100
12	800	876	-	-	15	2	0.6	1	1.9	0.19	47	61
13	800	876	-	-	15	3	0.6	1	1.9	0.22	45	58
14	800	879	-	-	15	4	0.6	1	1.8	0.22	41	89
15	800	879	-	20	-	-	-	-	1.8	0.19	42	-
16	800	879	-	20	-	2	-	-	1.8	0.19	67	-
17	800	879	-	20	-	3	-	-	1.8	0.19	80	-
18	800	879	-	20	-	2	-	0.5	1.8	0.19	53	-
19	800	879	-	20	-	2	0.5	-	1.8	0.19	47	-
20	800	879	-	20	-	2	0.5	0.5	1.8	0.19	65	-

* represents acronyms used SP = sample number, CEM = Cement, E.S = Ennore sand, QTZ = quartz, S.F = silica fume, MK= metakaolin, N.S= nano silica, H.F= hooked fibres, C.F= crimped fibres, S.P= superplasticizer, DS = days strength.

3.6 DESCRIPTION OF SPECIMEN

Three joints casted as per IS 456:2000 (non seismic detailing) for seismic ground motion. The reinforcement detailing and its dimensions of the specimen are shown in Figure 3.8. The specimens were cast with beam and column of dimensions 125 mm X 225 mm with column length of 1000 mm and beam length of 900 mm. The specimen casted was of M20 grade concrete and Fe 500 steel. 4- 10 mm dia bars were use in column as reinforcement and beam was reinforced with 2- 8 mm dia bars on one side and 3 -10 mm dia bars on other side. The diameter of stirrups was kept as 8 mm. The spacing between the stirrups in the beam and column was kept as 150 mm c/c and 120 mm c/c respectively.

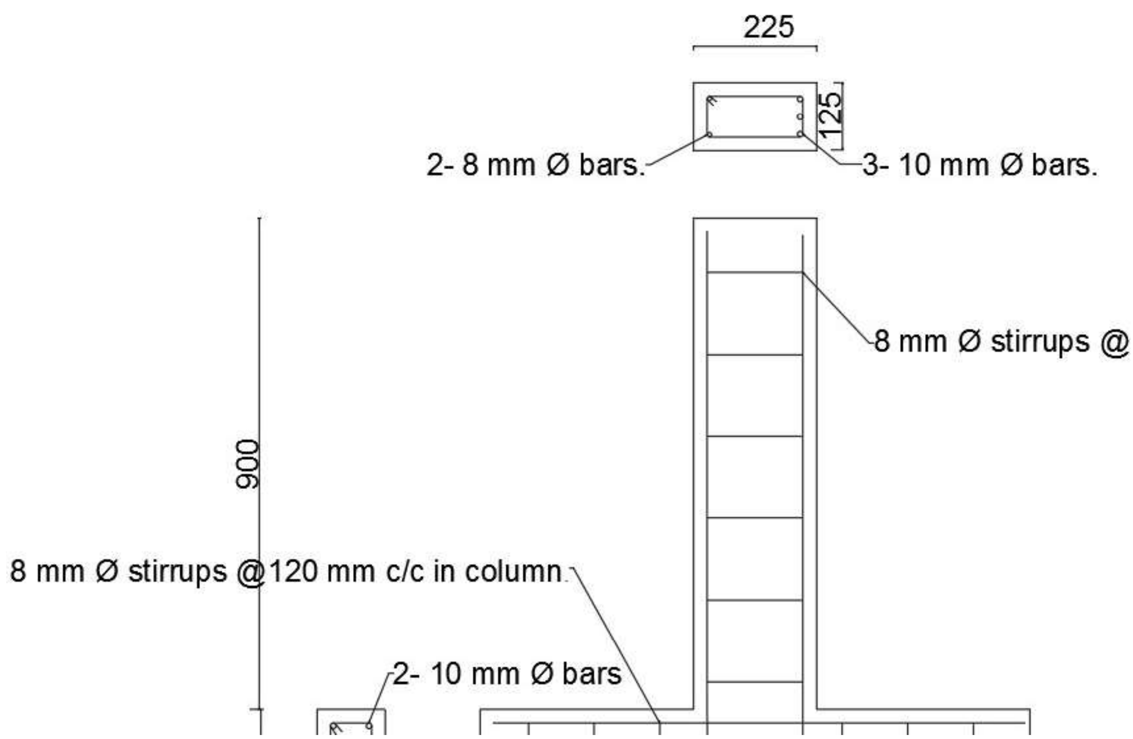


Figure 3.8: Detailing of specimen.

3.7 CASTING OF SPECIMENS

Figure 3.9 (a): Formation of steel mesh

The mix design used in preparation of M-20 concrete is used for casting beam column joint. The specimens were casted in the horizontal position within the steel mould. Vibrators were used to remove the entrained air within specimen. Then, these moulds were removed after period of minimum 24 hours (as per IS 456:2000). The specimens were then cured by wrapping the gunny bags over it and watering the specimens for a period of 28 days.

Figure 3.9 (b): Placement in mould.

Figure 3.9 (c): Casting of specimen.

Figure 3.9 (d): Curing of specimen.

Figure 3.9: Casting and curing of specimen.

Figure 3.9 represents different steps involved in casting and curing of specimen which is started from the formation of steel mesh to the placement of mesh in the mould.

3.8 EXPERIMENTAL SETUP

The test was conducted in laboratory under displacement control facility by subjecting specimen to quasi static reverse cyclic loading. The specimen setup is shown in Figure 3.10 (a**).** The amplitude of loading was gradually increased from 0 to 35 mm, starting with 5 mm and proceeding towards 35 mm, with an improvement of 5 mm in every other cycle. The loading history of the specimen in displacement cycles has been represented in Figure 3.10 (b). The reinforcement detailing of specimen is shown in Figure 3.8 & has been done as per IS 456:2000. The casting and curing procedure of specimen has been represented in Figure 3.9. The load cell used in applying the load to the beam has least count of 0.1 KN is determined on connecting it to the jack. The deflection at either end of beam was measured with the help of LVDT (Linear Variable Differential Transducers) with least count measured of 0.01 mm fixed at upper end of the beam at either side of it. The specimens were fixed to the frame with the nut bolt arrangement holding the column firmly to the base in horizontal direction while the cantilever portion of beam was held vertically by hydraulic jack and then load was applied in cyclic manner through load cell to the beam at a distance 125 mm from the free end of beam. The data logger was used to record the performance of specimen in terms of load and deflection**.** The first specimen tested was used as control specimen and has been loaded up to 30 mm deflection (or up to its ultimate deflection). By analysing hysteresis curve and damage index of control specimen, the damage level of remaining specimen were also determined (moderate damage and severe damage levels) as indicated in Table 4.2. The remaining specimens are also tested up to their decided damage levels.

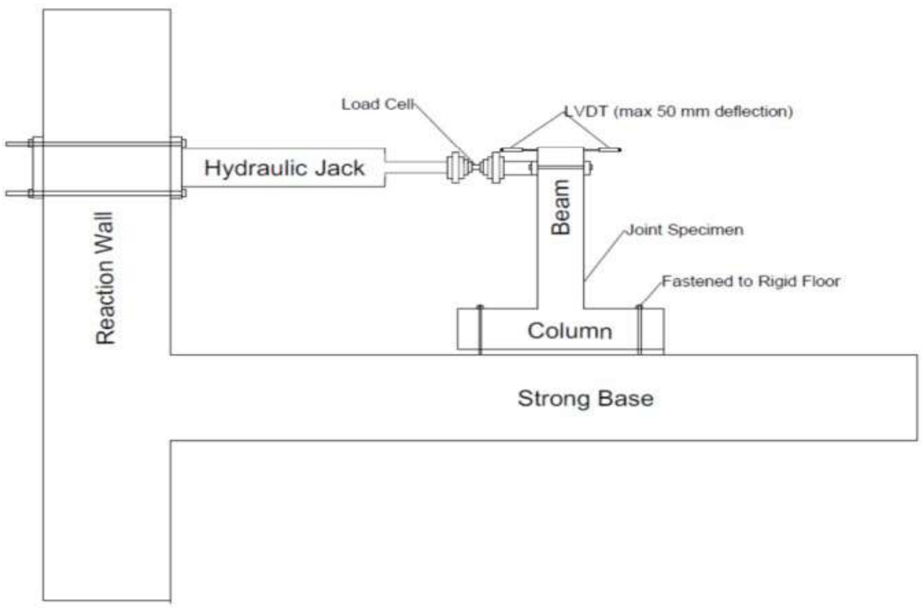


Figure 3.10 (a): Specimen testing arrangement.

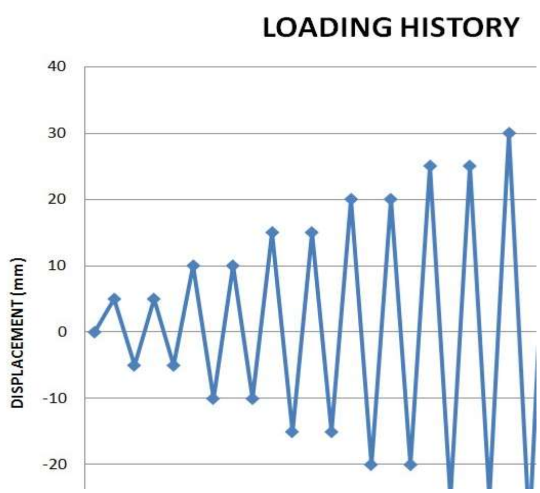


Figure 3.10 (b): Loading history for reverse cyclic loading.

Figure 3.10: Testing setup with loading history in displacement control mode adopted for reverse cyclic loading.

Figure 3.11: Monitoring response of specimen in attached computer.

3.9 RETROFITTING OF SPECIMEN

Before retrofitting already tested specimens, all the cracked locations were marked and surface around cracks were removed by chipping off the cracked concrete with the help of chisel and hammer. Separation of cracked surface was done in uniform manner throughout

Figure 3.12 (a): Chipping off cover of specimen in central region.

the concrete surface up to the decided depth and distance from the face of joint for all the three specimens. It was ensured that there should not be any loose surface left over the specimen during concrete removal. The concrete removal depth was up to the surface of steel

Figure 3.12 (b): Specimen ready for retrofitting.

Figure 3.12 (c): Retrofitting done.

Figure 3.12 (d): Specimen ready for testing.

Figure 3.12: Retrofitting procedure.

bars or clear cover distance. This distance was up to the distance of 200 mm from the face of joint. A layer of cement water paste was prepared to adhere the hardened surface of concrete with the new HPFRCC layer. This helps in improving the bonding of hardened concrete to the HPFRCC layer. Now the chosen sample was prepared in required amount to retrofit all three joints. These retrofitted specimens were again tested after curing it for a period of 7 days. The procedure for retrofitting is represented in Figure 3.12.

3.10 TESTING OF RETROFITTED SPECIMENS

After retrofitting the specimens, the specimens were further tested in the same manner as it was tested earlier to determine and compare specimen behaviour with the previous tested results. Images of tested specimens before and after retrofitting have been shown in Figure 4.1, Figure 4.7 and Figure 4.8 for specimens S1, S2 and S3 respectively. The test results are analysed in next chapter in terms hysteresis curve, energy dissipation capacity, energy absorption capacity, principal tensile stresses, stiffness degradation, and crack pattern and failure analysis.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 GENERAL

The beam column joint which was casted in previous chapter were tested and retrofitted with the developed HPFRCC concrete. The testing of retrofitted specimen was done again after curing the specimen for a period of 7 days and the results obtained were analysed and compared with the results of control specimen. . These results are analysed and presented in tabular and graphical form for easy understanding. The results obtained displays a comparative study in terms of hysteresis curve, energy dissipation, energy absorption, stiffness degradation, developed joint stresses and their cracking pattern.

4.2 BEHAVIOUR OF CONTROL SPECIMEN

During the testing of specimen the joint was subjected to displacement cycles of magnitude of 0 to 35 mm in steps of 5 mm increment every other cycle. Since the specimen was held at its ends the cracks propagated all round its intersection region of beam column joint as shown in Figure 4.1 (a). Initial cracks were observed in intersection region of joint and propagated towards the other end of joint. However some crack lines were observed completely in column region but these crack lines were limited to the same end of column. This was found due to sufficient development length provided in joint section of the specimen so that the specimen remain intact after the application of all displacement cycles. Few cracks were also observed in the middle section and on opposite faces of the beam as shown in Figure 4.1 (a). After retrofitting the specimen the joint displayed better performance on restricting cracks propagation. The subsequent effect of improved performance is reflected in the figures of Hysteresis curve, ductility, stiffness degradation, joint stresses, and cracking pattern. In Hysteresis curve, the maximum load carrying capacity which was declining prior to retrofitting at its last displacement cycle (14.3 KN & 4.1 KN) got increased up to (17 KN & 11.3 KN) after retrofitting the specimen. The ductility factor which was 3.43 before retrofitting improved up to 4.3 after retrofitting. A similar improvement in stiffness behaviour is also observed in Table 4.5 which represents almost full recovery in stiffness behaviour before and after retrofitting the specimen.

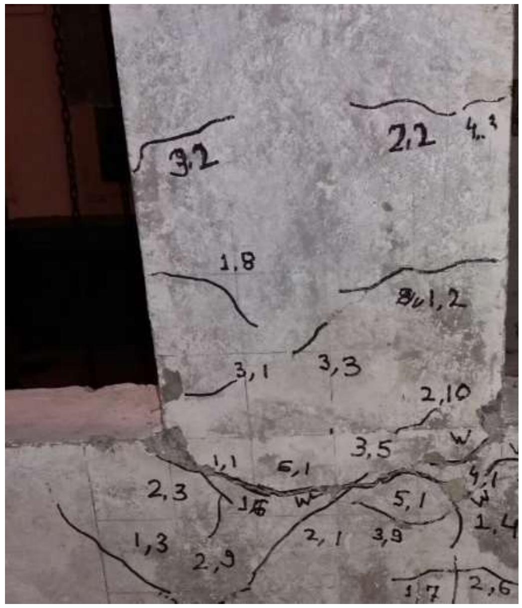


Figure 4.1 (a): Cracks formation prior to retrofitting.

Figure 4.1 (b): Cracks formation after retrofitting.

Figure 4.1 (c): Fibres bridging action.

Figure 4.1 (d): Cracks formation at joint intersection.

Figure 4.1 (e): Cracks formation at other side of joint.

Figure 4.1: Representation of cracks in sample S1.

4.3 LEVEL OF STRESSING IN BEAM COLUMN JOINT

The different damage levels were computed based on damage index as shown in Table 4.1. The complete damage, moderate damage and severe damage are selected for retrofitting the joint. The experimental study considered for stressing joint is shown in Table 4.2. The computation for damage index of control specimen has been used by Park and Ang formula

$$DI = \frac{\delta m}{\delta u} + \frac{\beta}{Fy.\delta u} \int dE$$

Where

δm = maximum displacement under each cyclic load,

δu = ultimate displacement capacity, Fy = yield strength of structure,

dE = Energy dissipation, β = strength degradation parameter.

The value for damage index varies from 0 to 1, i.e. from no damage to complete damage. In this study it is assumed that

Table 4.1: Damage Index categorisation.

Damage Index Variation	Damage level
0 to 0.2	Elastic behaviour
0.2 to 0.4	Slight damage
0.4 to 0.6	Moderate damage
0.6 to 0.8	Severe damage
0.8 to 1	Complete damage

Table 4.2: Damage Index adopted.

Specimens name before retrofitting	Damage Index	Damage level	Specimens name after retrofitting
S1	1.00	Complete damage	RS1
S2	0.50	Moderate damage	RS2
S3	0.66	Severe damage	RS3

These different stressing of joints will help in comparing the results of retrofitted specimens with that of control specimen. The results are analysed in terms of energy dissipation, energy absorption, stiffness degradation, ductility, joint stresses and crack pattern and failure analysis.

4.4 EFFECT OF RETROFITTING ON HYSTERESIS CURVE

During the initial displacement cycles of the control specimen, the load carrying capacity of specimen was seen to be increased while at the later cycles of loading, the load carrying capacity of specimen was seen to be decreased which can be observed from the Figure 4.2. However on retrofitting the specimen by HPFRCC, the specimen regained its strength up to certain extent. From hysteresis curve of RS1, RS2, RS3 it was observed that graph initially increased up to third displacement cycle (12 KN load) later on it started decreasing and went up to 7 KN. The increment and decrement pattern of load at different displacement cycles was almost linear. This shows that retrofitting with HPFRCC was successful as specimen attains sufficient strength within 7 days and is producing reliable results in critical conditions i.e. if there is not much fluctuation in load action over specimen then HPFRCC can be implemented in retrofitting. Another change was observed at zero displacement in the initial displacement cycles of all the retrofitted specimens, the load bearing capacity of specimen suddenly increases without any displacement at its initial position. This is due to the fact that HPFRCC has prevented the specimen from any deformations by offering maximum resistance to the applied external force.

However before retrofitting at the opposite side of specimen (Figure 4.2 (a)), it was observed that the peak load reaches up to 21 KN and after retrofitting (Figure 4.2 (b)) these fibres were not able to regain its ultimate peak load and eventually reached up to 19 KN. This implies that HPFRCC became more insufficient in retrofitting such side of beam which has more diameter reinforced bars but these fibres has also helped in preventing spalling of concrete at both sides of specimen as shown in Figures 4.1 (c), 4.7 (c) and 4.8 (d).

From the last two loading cycles of retrofitted specimens (30 mm and 35 mm) the pattern of their peak load variation helped in regaining some strength to specimen which is found due to random distribution of fibres helped in offering sudden increase in resistance to the displacement as shown in Figure 4.2 (b) & (f) while the presence of horizontal line in hysteresis curve of retrofitted specimens displayed that they offered no resistance in bearing loads at that point of displacement cycles.

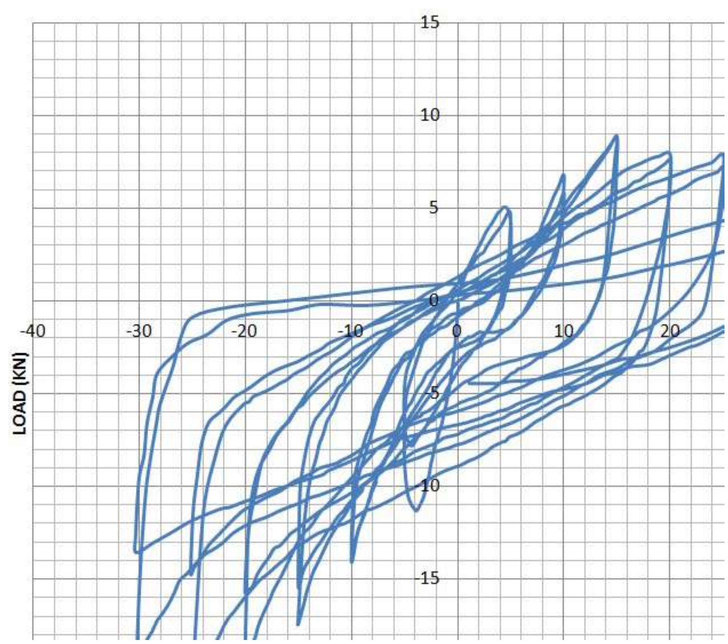


Figure 4.2 (a): S1

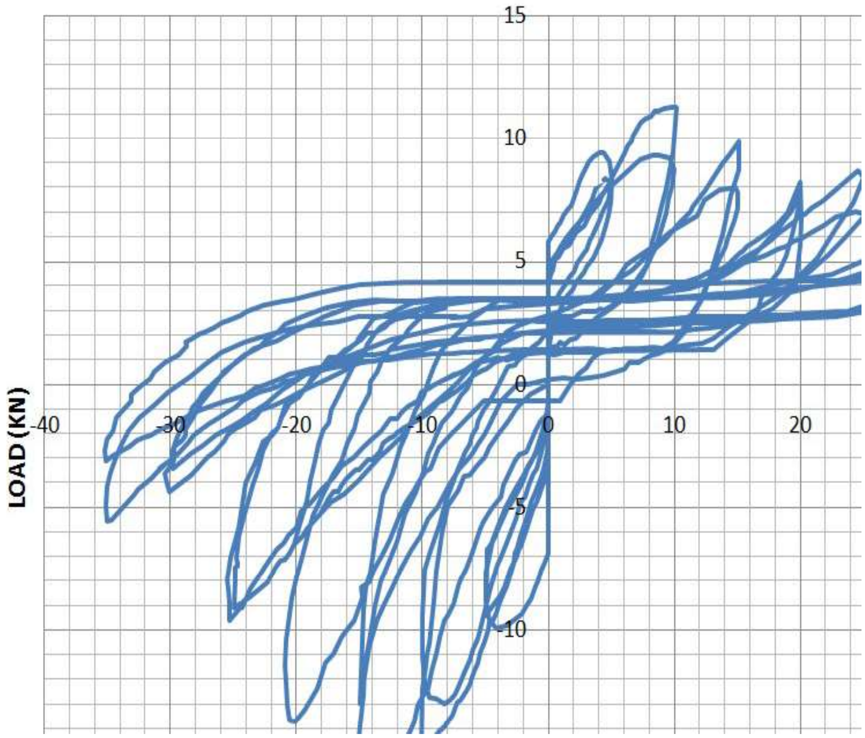


Figure 4.2 (b): RS1

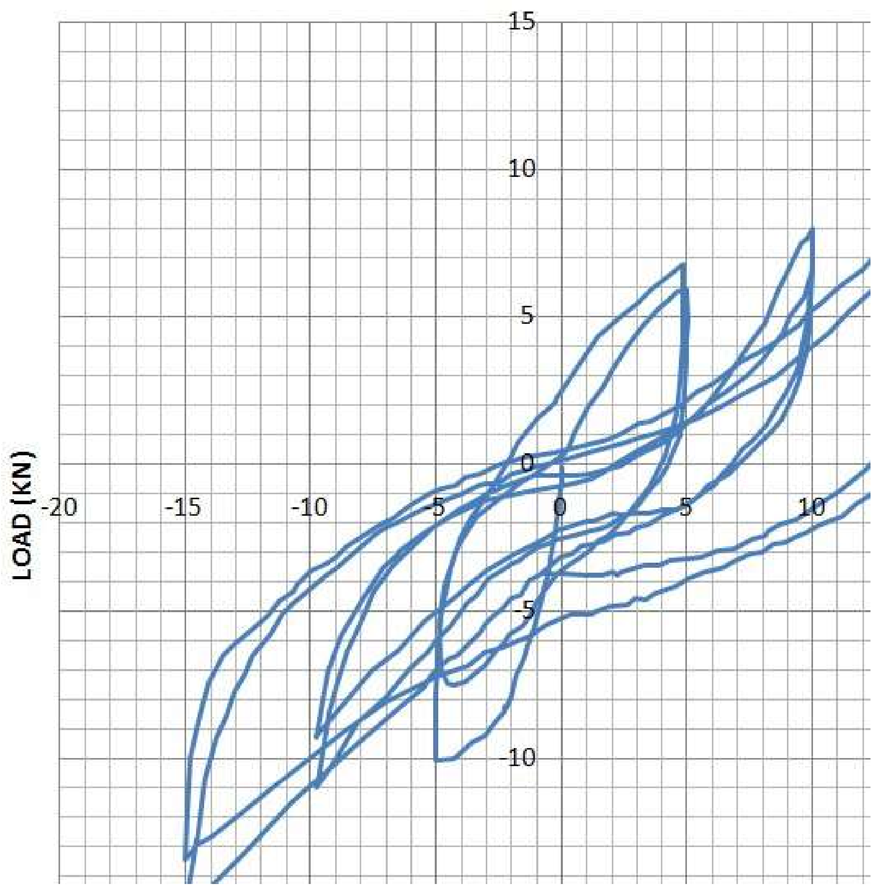


Figure 4.2 (c): S2

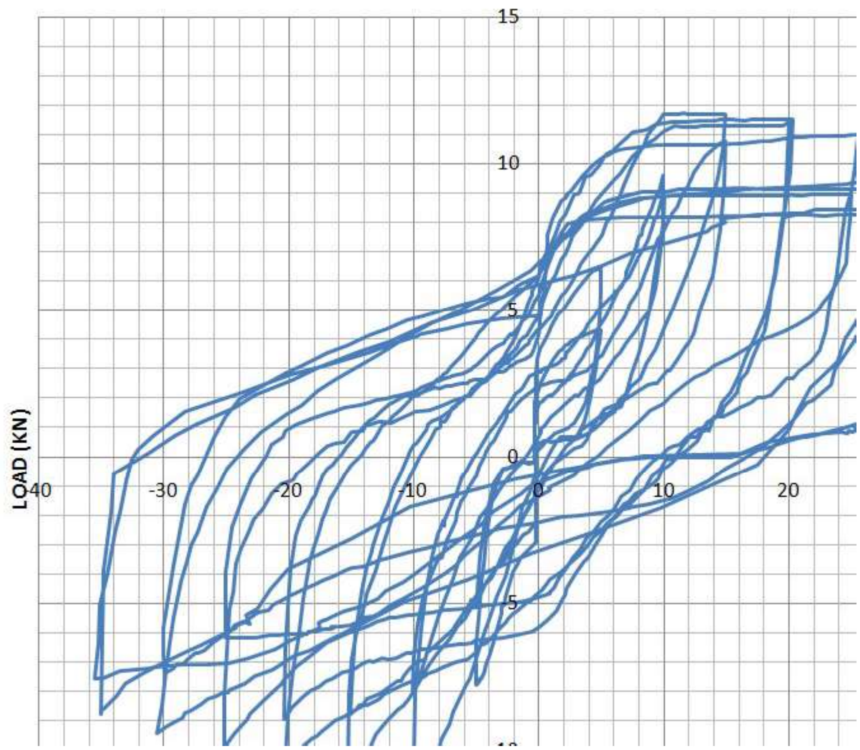


Figure 4.2 (d): RS2

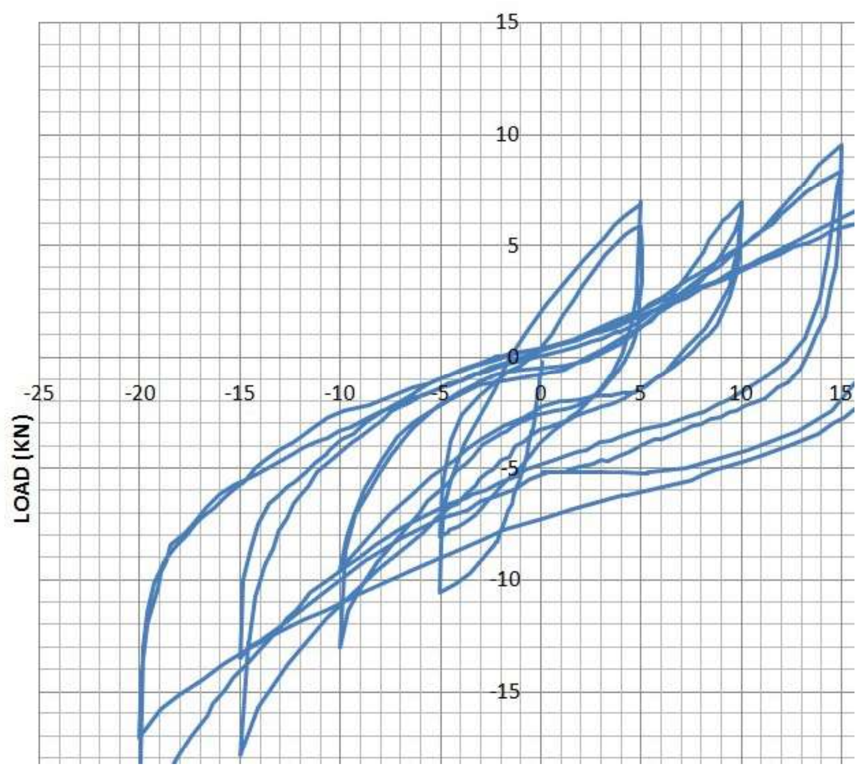


Figure 4.2 (e): S3

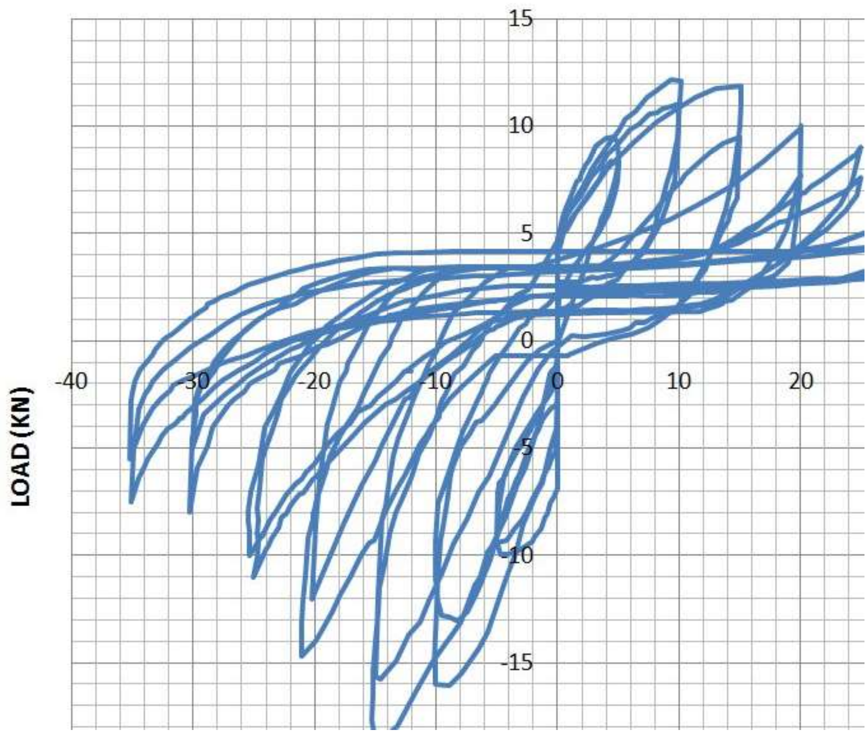


Figure 4.2 (f): RS3

Figure 4.2: Represents the hysteresis curve of all specimens before and after retrofitting.

4.5 EFFECT OF RETROFITTING ON ENERGY DISSIPATION

The area formed under load displacement curve gives the energy dissipation capacity of the specimen. This could be directly understood by the relation between the area under curve and energy dissipation capacity. The more will be the area under curve; more will be the energy dissipation capacity of the specimen. Energy dissipation tells about the ductile behaviour of specimen. There are two parameters for energy dissipation calculation. These are relative energy dissipation (RED) and cumulative energy dissipation (CED). Cumulative energy dissipation capacity after each subsequent cycle is calculated by the total area under hysteresis loop up to the required displacement cycle.

After retrofitting, specimen S2 was the specimen came out with highest energy dissipation capacity (RS2). RS1 has the least energy dissipation capacity as it was already failed before retrofitting. RS3 came out with slightly better performance with RS1 as it was severely failed. These are plotted in Figure 4.3 and corresponding energy dissipation values are mentioned in Table 4.3.

On comparing energy dissipation capacity of specimen S1 & RS1, after first three displacement cycles RS1 was not lagging much behind S1 (588 & 531KN-mm) which indicates that HPFRCC responded well in retrofitting completely failed specimen. However at later loading cycles, the retrofitted specimen showed decrease in energy dissipation capacity. And for specimen RS3 & S3, RS3 being severely damaged performed better than RS1 as it regained dissipated energy up to 77% more energy dissipation as of S3. On comparing specimen S2 & RS2, the specimen S2 regained its entire lost strength after retrofitting at it was least damaged. RS2 was the best suited specimen in having highest energy dissipation capacity.

Table 4.3: Represents RED at different stages of loading.

Specimen	5 mm	10 mm	15 mm	20 mm	25 mm	30 mm	35 mm
S1*	117	105	366	624	806	863	-
S2*	111	106	338	-	-	-	-
S3*	107	109	354	588	-	-	-
RS1*	60	182	289	265	293	179	240
RS2*	65	130	337	764	741	970	1002
RS3*	73	260	450	332	312	207	255

*All energy is in KN-mm

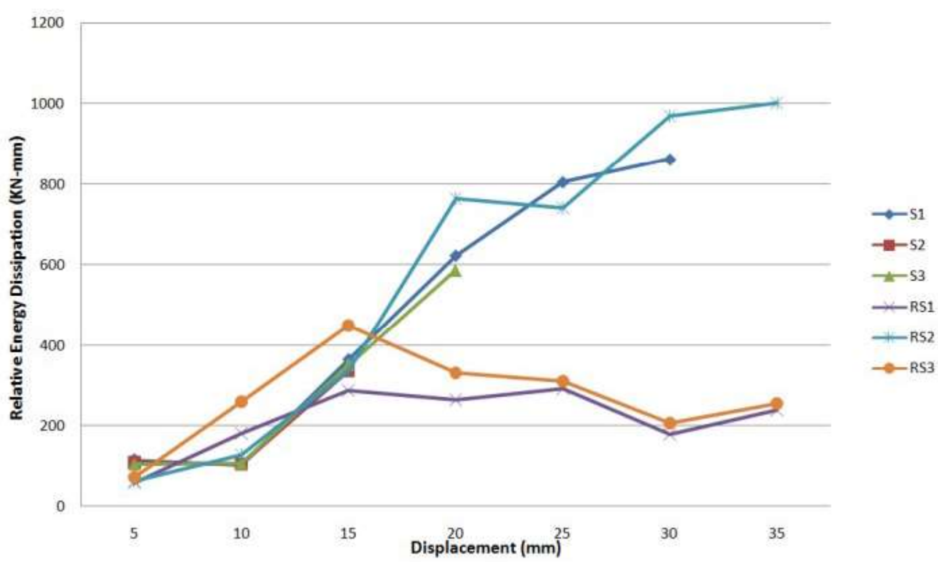


Figure 4.3 (a): Relative Energy Dissipation plot.

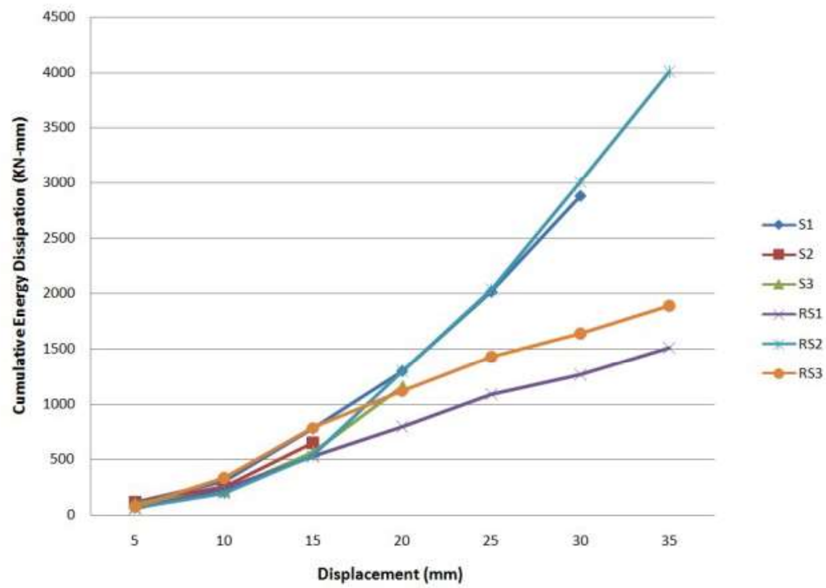


Figure 4.3 (b): Cumulative Energy Dissipation plot.

Figure 4.3: Represents the relation of energy dissipation with loading cycles.

4.6 EFFECT OF RETROFITTING ON LOAD–DEFORMATION ENVELOPE

The load-deformation curve of all the three specimens are plotted from the hysteresis curve as represented in Figure 4.2 by taking its peak values. This load deformation curve helps in understanding the Energy absorption capacity of specimen. This energy absorption capacity determines the capacity of specimen to absorb the energy after it has already been stressed. Energy absorption is always dependent on the energy dissipation capacity and is always lesser than energy dissipation capacity of specimen. The relation between energy dissipation and energy absorption is also dependent upon the ductile behaviour of specimen. More the ductility factor more will be the ratio of energy dissipation and energy absorption. It was evidenced from the Figure 4.4 that there was improvement in the post yield behaviour of specimen after retrofitting as compared to control specimen. On comparing energy absorption capacity of moderately damaged and severely damaged specimens with control specimen, it was found that both had more energy absorption capacity which is 14% and 10% more than that of control specimen. It is also verified by the ductility factor From the Table 4.4. The ductile capacity of specimen was also found to be increased from 25% to 60% in specimen RS2 & RS3 as compared to control specimen. The ultimate displacement of specimen mainly at later stages of loading was found to be increased after retrofitting.

Ductility, $\mu = \frac{U}{Y}$ where U is ultimate displacement and Y is yield displacement.

Table 4.4: Test results

S.N.	Max positive		Max negative		Ultimate Positive		Ultimate Negative		Displacement		Ductility Δ_u/Δ_y
	P (KN)	Δ_m (mm)	P (KN)	Δ_m (mm)	P (KN)	Δ_u (mm)	P (KN)	Δ_u (mm)	Δ_y (mm)	Δ_u (mm)	
S1	9.2	15	21	25	7.21	30	19.2	30	16	55	3.43
RS1	11.25	10	16.47	15	6.8	35	5.5	35	10	43	4.3
RS2	11.67	15	12	15	8.5	35	8.7	35	12	66	5.5
RS3	12.07	10	19	15	6.5	35	7.5	35	9.5	45	4.7

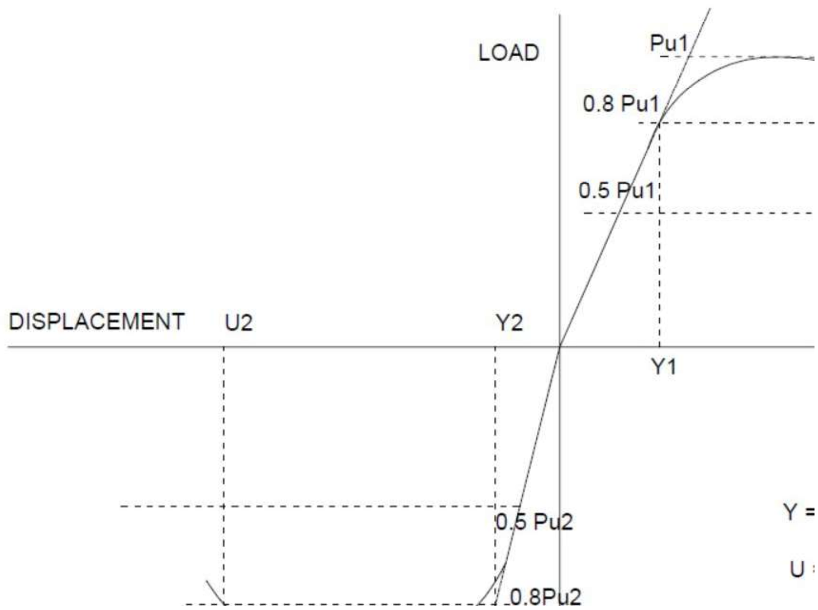


Figure 4.4: Load displacement envelope over ductility.

4.7 EFFECT OF RETROFITTING ON SECANT STIFFNESS DEGRADATION

Stiffness is a property of structure which measures force required for unit deformation. Stiffness is given by –

$$K = \frac{F}{\Delta},$$

Where K = stiffness,

F = Force, Δ = Deflection.

Secant stiffness is a faster way for stiffness calculation. It directly relates force and deformation at furthest point in a curve. It generally gives stiffness on the safer side of curve. On comparing secant stiffness degradation values from Figure 4.5, a uniform stiffness degradation curve has been found on that portion of beam which was reinforced with 2-8 mm dia bars whereas other side of the beam had performed in random manner. Thus it can be said that HPFRCC has helped in regaining lost stiffness of specimen mainly to the one side of beam which had lesser diameter reinforced bars. Overall, improved initial stiffness was found with HPFRCC retrofitting over the conventional specimen.

Table 4.5: Represents the variation of stiffness at different stages of loading.

Def	S1		S2		S3		RS1		RS2		RS3	
(mm)	A*	B*	A*	B*	A*	B*	A*	B*	A*	B*	A*	B*
5	1.48	1.57	1.38	1.62	1.40	2.09	1.85	2.40	1.73	1.97	1.86	1.89
10	0.90	1.40	0.82	1.11	0.90	1.50	1.11	1.48	1.16	1.64	1.18	1.59
15	0.59	1.13	0.59	0.82	0.64	1.19	0.70	1.08	0.74	0.92	0.79	1.27
20	0.39	1.0	-	-	0.50	0.95	0.47	0.68	0.58	0.65	0.50	0.76
25	0.28	0.83	-	-	-	-	0.35	0.38	0.40	0.47	0.36	0.52
30	0.18	0.64	-	-	-	-	0.17	0.14	0.29	0.35	0.23	0.26
35	-	-	-	-	-	-	0.19	0.16	0.18	0.24	0.14	0.21

A* represents that side of beam which had 2 -8 mm diameter bars as reinforcement.

B* represents that side of beam which had 3 -12 mm diameter bars as reinforcement.

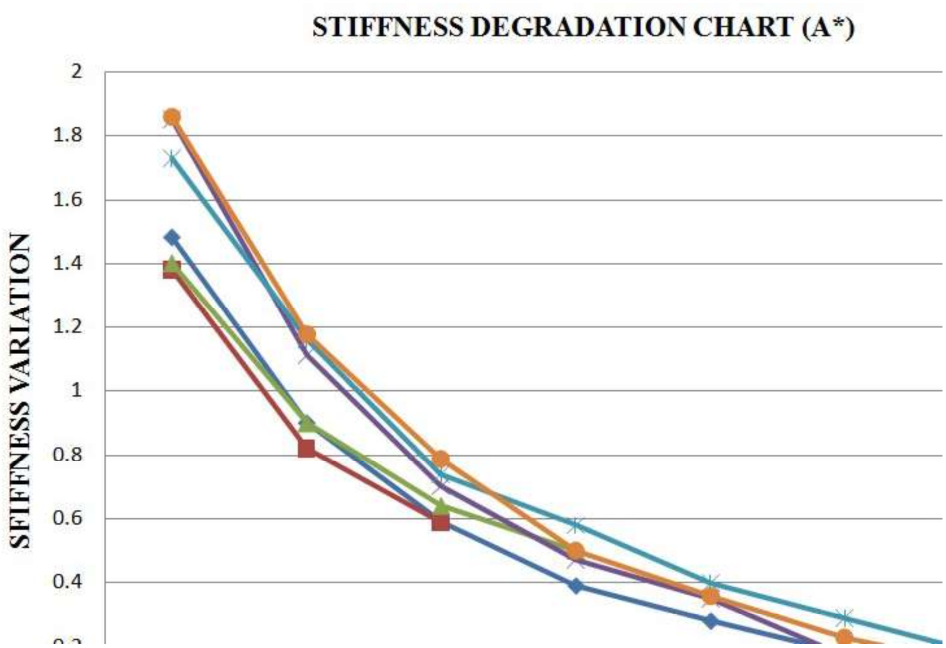


Figure 4.5 (a): Stiffness variation (A*)

The Table 4.5 represents the corresponding effect of stiffness to the specimen before and after retrofitting and their corresponding graphs has been plotted in Figure 4.5 to have better understanding in stiffness variation. This is due to the fact that the fibres incorporated in HPFRCC confined the concrete and improved its strain behaviour for all the tested specimens. The change in stiffness degradation rate directly affects the response of structural component. If the stiffness degradation rate is high the structure is showing brittle response, while if this stiffness degradation rate is low then it means structure is showing ductile response. In the retrofitted specimens, after every cycle of loading on comparing specimen stiffness response with that of other specimens it can be understood that more improvement in stiffness behaviour is observed to the side of beam which was more prone to failure by preventing the separation of steel and concrete.

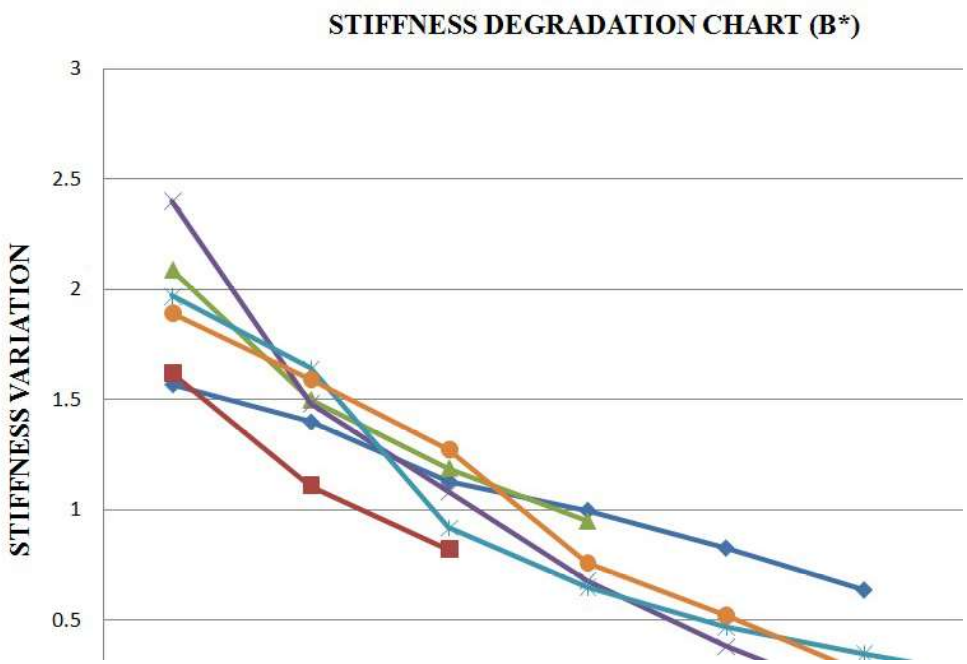


Figure 4.5 (b): Stiffness variation (B*)

Figure 4.5: Stiffness variation in beam-column joint.

4.8 EFFECT OF RETROFITTING ON JOINT STRESSES

The horizontal shear stress developed in the beam column joint is given by (Murthy et al. (2003)).

$$\tau_{jh} = \frac{P}{Ah}\left(\frac{Lb}{db} - \frac{Lb+0.5\,Dc}{Lc}\right)$$

Where P = loads applied at ends of beam (N)

Ah = Joint horizontal cross sectional area (mm^2)

Lb = length of beam (mm)

db = Effective depth of beam (mm)

Dc = Total depth of beam (mm)

The principal tensile stress developed in joint region is given by

$$\sigma_t = \frac{\sigma p}{2} + \sqrt{\frac{\sigma p^2}{4} + \tau jh^2}$$

where σp = axial compressive stress in the joint area (N/mm^2).

$$= \frac{Nc+P}{bc+hc}$$

Nc = Axial compressive load in the column (N).

bc = width of column (mm).

hc = depth of column (mm).

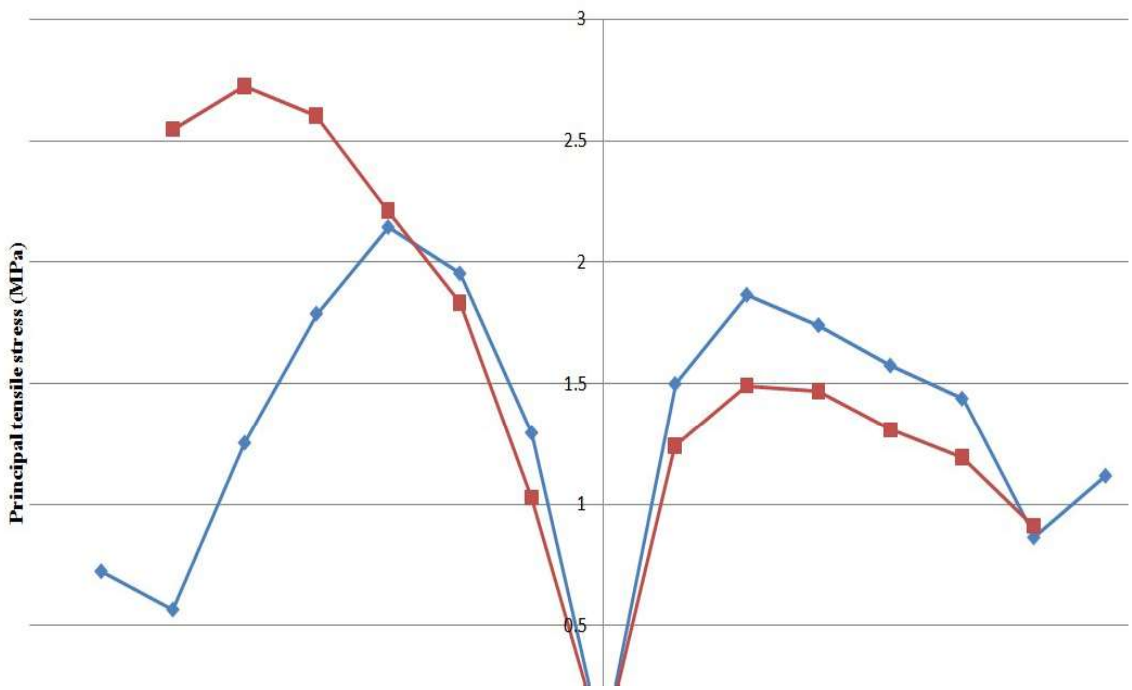


Figure 4.6: Principal tensile stresses comparison.

The graph shown in Figure 4.6 has compared the behaviour of principal tensile stress variation of the reference specimen and the retrofitted specimen with respect to displacement cycles. The figure clearly shows that the retrofitted specimen displayed improvement in principal tensile stress right from the start of first displacement cycle to the third displacement cycle which shows the effectiveness in applying HPFRCC as retrofitting material to the beam column joint. The behaviour of retrofitted specimen after the yielding of joint is mainly unexpected on one side of beam which got 3-12 dia reinforcement bars. One of the reasons for such behaviour of joint may be specimen got insufficient curing days to develop proper bonding between the hardened concrete and HPFRCC layer which is observed in cracking pattern of specimen RS3. On the other side of beam where 2- 8 dia reinforcement bars was embedded showed satisfactory performance up to the last displacement cycle.

4.9 CRACKING PATTERN AND FAILURE ANALYSIS COMPARISON

From the tested specimens, it was observed that the cracks formed in control specimen have spread all over the joint region especially in patches form around the junction region and in beam. However, major crack line was developed at the junction region of the joint starting from beam face as shown in Figure 4.1 (a) after retrofitting with HPFRCC the crack pattern formed was similar to prior developed cracks with an improvement in the peak load carrying capacity of the joint (from 9 KN to 11.225 KN) and fibres prevented the spalling of concrete as shown in Figure 4.1 (c). This shows that retrofitting the specimen was effective in improving the load carrying capacity of specimen and improving behaviour of joint by preventing concrete from spalling. It also showed on later loading cycles, Specimen S2 and S3 displayed a similar failure pattern as of S1 with lesser cracks propagating in column (Figure 4.7 (a) and Figure 4.8 (a), (b)). In retrofitted specimen RS2, spalling of concrete was seen at junction region mainly at upper & lower region of beam and at other locations fibres were preventing concrete from spalling and leading to delamination of HPFRCC. In RS3, the crack mainly propagated at prior failed crack line with large crack width. Along with this delamination of HPFRCC layer had also occurred at later cycles of loading. All specimens shows that fibres (hooked and crimped) displayed good bridging action within the newly formed HPFRCC by preventing the spalling of concrete. It also displayed uniform behaviour

Figure 4.7 (a): Cracks formation prior retrofitting for specimen S2.

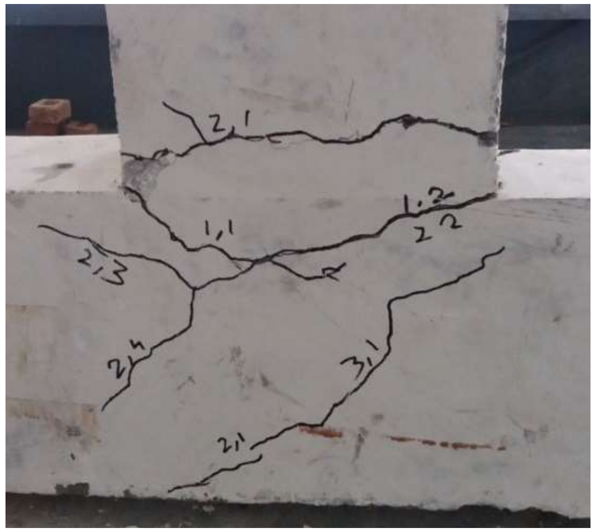


Figure 4.7 (b): Cracks propagation for specimen S2.

at the junction of beam column joint. As the column was held in firm position and beam was subjected to cyclic loading, more cracks were formed within the beam near the junction of the joint both the sides of beam i.e. above top reinforcement and below bottom reinforcement, certain spalling of concrete was seen. HFRCC was found to be more effective as it prevented

Figure 4.7 (c): Cracks formation after retrofitting for specimen RS2.

major portion of concrete from spalling. However at later stage of loading, HPFRCC layer started separating from hardened concrete which was obvious due to the fact that new HPFRCC .

Figure 4.7 (d): Fibres preventing spalling of concrete specimen S2..

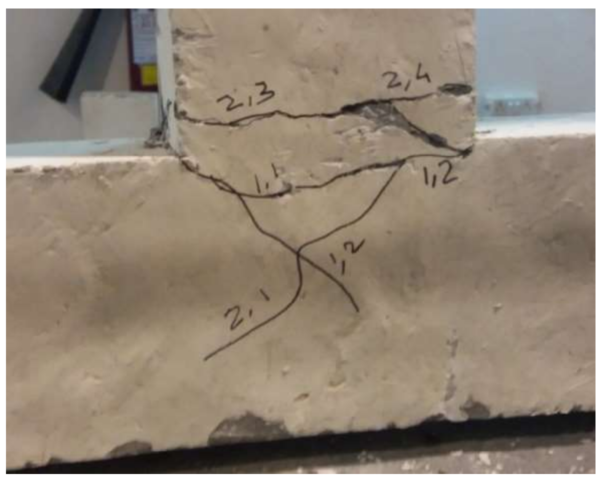


Figure 4.8 (a): Cracks formation prior to retrofitting.

Figure 4.8 (b): Cracks propagation in specimen S3.

Figure 4.8 (c): Separation of HPFRCC layer from hardened concrete in RS3.

Figure 4.8 (d): Cracks after retrofitting.

Figure 4.8: Representation of cracks in specimen S3.

layer haven't got sufficient time for curing. It can be said that if there had been proper binding between the two layers they must have shown better response.

CHAPTER 5

CONCLUSIONS

5.1 GENERAL

The following conclusion can be drawn from the dissertation performed in the laboratory.

1. Improvement in energy dissipation capacity in moderately damaged and severely damaged retrofitted specimen has been found up to 39.15% and 34% respectively when compared to retrofitted completely damaged specimen.
2. Based upon stiffness variation, best results came out with retrofitting of moderately damaged specimen as it was the least damaged specimen and worst stiffness variation was observed in retrofitting of completely damaged specimen. Thus it can be concluded that retrofitting the joint with HPFRCC becomes more effective when it is in the phase of moderately damaged to severely damaged.
3. It was observed that principal joint stresses in retrofitted specimens mainly in first three displacement cycles was more when compared with response of specimen before damaging it. This clearly implies that retrofitting is found effective in development of principal stresses in joint region of specimen.
4. On retrofitting the damaged specimen with HPFRCC, improvement in load carrying capacity and ductility was found. Thus it can be said that retrofitting any joint with HPFRCC is an effective solution to come out from sudden threat of joint failure especially in earthquake vulnerable buildings.
5. Higher percentages of hooked fibres and crimped fibres (up to 1%) helped in attaining higher strength in concrete due to their good bridging action as in lower percentage of fibres, fibres were not evenly distributed over the surface of specimen which led to non uniform strength attainment in HPFRCC.
6. Hooked fibres are found effective in preventing the spalling of concrete and especially to the upper and lower sides of beam in beam column joint. This was found due to the presence longer hooked fibres and its random distribution in concrete. This helps in improving the contact surface area of hooked fibres with concrete. So it eventually helps in restraining the smaller concrete patches from spalling.

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