EFFECT OF STRESS LEVEL ON RETROFITTING OF EXTERIOR BEAM COLUMN JOINTS

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CERTIFICATE

This is to certify that thesis entitled **"Effect of Stress Level on Retrofitting of Exterioir Beam Column Joints"**, being submitted by Mr. **Puneet Bhandari**, Roll No **800822006** in partial fulfilment for award of degree **Master of Engineering in Civil (Structures) at Thapar University, Patiala,** is a bonafide work carried out by him under our guidance and supervision and that no part of this thesis has been submitted for the award of any other degree.

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ABSTRACT

Almost all the structures whether industrial, commercial or housing are constructed of RCC. There structures fare nicely under normal circumstances, but in the event of major earthquakes, higher load imposition etc. they may suffer permanent damage. This poses a more difficult scenario for a structural engineer than constructing a new building. This is due to number of restraints an already constructed building throws up like non engineered construction, wear & tear etc. Instead of tearing apart the structure one can strengthen the deficient structural elements of the structure. Thanks to the advancement in technology with the help of non-destructive testing one can easily identify such deficient elements. Once identified the best way out is to retrofit such elements. Retrofitting is different from repair or rehabilitation. It is basically a process of strengthening and enhancement of the performance of deficient structural elements in a structure or of the structure as whole. Retrofitting of deficient buildings can be done by increasing the strength, stiffness and/or ductility of its specific constituent elements or of the whole building. For any building, depending upon the requirement, a combination of the above may also be selected. Retrofitting of individual members or elements is referred to as local retrofitting.

In this present study the effect of stress level on retrofitting of exterior beam column joints is studied. The deflections of retrofitted and non- retrofitted specimens are compared.

Specimens were stressed to ultimate loading (100% damage) and the deflections were noted.

After retrofitting with the ferrocement technique, the retrofitted specimens were compared with the controlled specimens. Results show that there is an increase in load carrying capacity for beam column joint retrofitted with ferrocement varying in different cases. Also increase in yield load is observed.

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List of Notations Used

1. CS = Controlled Specimens
2. RS = Retrofitted Specimens
3. AR = After Retrofitting
4. BR = Before Retrofitting
5. J1 = Joint 1
6. J2 = Joint 2

CHAPTER – 1 INTRODUCTION

1.1 GENERAL

In the analysis of reinforced concrete moment resisting frames the beam-column joints are generally assumed as rigid. In Indian practice, these joints are usually neglected for specific design with attention being restricted to provision of sufficient anchorage for beam longitudinal reinforcement. This may be acceptable when the frame is not subjected to earthquake loads. There have been many catastrophic failures reported in the past earthquakes, in particular with Turkey and Taiwan earthquakes which occurred in 1999, which have been attributed to poor design of beam-column joints. The poor design practice of beam column joints is compounded by the high demand imposed by the adjoining flexural members (beams and columns) in the event of mobilizing their inelastic capacities to dissipate seismic energy. Unsafe design and detailing within the joint region jeopardizes the entire structure, even if other structural members conform to the design requirements.

Since past three decades extensive research has been carried out on studying the behaviour of joints under seismic conditions through experimental and analytical studies. Various international codes of practices have been undergoing periodic revisions to incorporate the research findings into practice.

A significant number of structures in India were designed according to older versions of seismic codes. While these types of structures, in light of the revised codal provisions were very vulnerable to unexpected earthquakes, some modification to their structural configuration and material properties showed improvement in their seismic performance. Thus, retrofitting was suggested to be carried out and practitioners started to apply various interventions to the structures to make them earthquake resistant. In general terms partial improvement of degraded strength of a structure is known as repairing, while retrofitting is the strengthening of structure to a pre- defined level.

When built according to earlier code provisions, beam–column joints in reinforced concrete moment resisting frames have inadequate or no transverse shear reinforcement, and the bottom reinforcement of the beam is anchored only 150 mm from the column face, with inadequate development length when the bars are in tension. This was done under the assumption that the beam positive moment reinforcement at the column face is always in compression. Because of these deficiencies, the joint may experience shear or bond-slip failure modes. These brittle types of failure will significantly reduce the overall ductility of the structure.

Inadequate transverse reinforcement in the joint, and weak-column/strong-beam design, are the main reasons for the observed joint shear failures during recent earthquakes. Joint shear failures may result in non-ductile performance of reinforced concrete moment-resisting frames. Many existing structures were designed and built before the development of current seismic codes, or on the basis of earlier codes much before ductile reinforcement detailing requirements came up.

Evidence from recent earthquakes, such as the 1999 Kocaeli (Turkey) and Chi-Chi (Taiwan) earthquakes, shows that a brittle shear failure in the joint may have caused the total collapse of most structures. Damaged structures after the Kocaeli earthquake, shown in Plate 1.1 and Plate 1.2, offer good example of this type of failure. Due to the significant contribution of joint failures to the collapse of buildings during earthquakes it is necessary to develop economical methods to upgrade the joint's capacity, in order to prevent a brittle shear failure and, instead, shift the failure towards a beam flexural hinging mechanism, which is a more ductile type of behaviour.

Plate 1.1 Beam-column joint shear failure

Plate 1.2 Joint failures during the Kocaeli (Turkey) earthquake

The design of beam-column joints is an important part of earthquake resistant design for reinforced concrete moment-resisting frames. Because of difficulty in repairing and retrofitting of the buildings damaged at beam-column joints due to the seismic attack and structural importance, recent building codes for reinforced concrete buildings provides allowable joint shear stress to preclude premature failure of beam-column joints before beam sway mechanism is developed.

Beam column joints are critical regions in multi-storey moment resisting reinforced concrete frames subject to inelastic response under severe seismic loading. Because seismic moments in columns and beams act in opposite directions across the joint, the beam-column joint is subjected to horizontal and vertical shear forces whose magnitudes are often many times higher than those found in adjacent beams and columns. Since joints are also connecting elements of the load carrying columns, brittle failure such as shear or bond failure in the joints must be avoided. Therefore, in the design of the reinforced concrete beam-column joints against seismic load, it is desirable to limit joint strength degradation until the ductility capacity of the beam reaches the designed capacity. Before 60's, columns, beams and walls exhibited the most damage by earthquakes, not joints. However, serious damage to beam-column joints began to appear due to seismic loads in the 60's. The reason was not because the quality of the joints built before the 60’s were superior, but because the strength of the joints got weaker relative to the adjacent members that were designed for greater capacities *[Paulay & Priestley, 1992]*.

Many theoretical and experimental studies have been carried out regarding beam-column joints since the 70's. Especially, three-country joint research efforts (America, Japan and New Zealand) made remarkable improvements in joint design. Reflecting the results of these studies, ACI Recommendations *[ACI 352-R02]* divide joints into two categories:

Type 1 for structures in a non-seismically hazardous area and

Type 2 for structures in a seismically hazardous area

In spite of the cooperative research efforts, the three countries have proposed different views on shear mechanisms and joint strength. In ACI recommendations *[AIJ, 1999]*, AIJ guidelines (Architectural Institute of Japan and prediction of joint shear strength is based on the concrete arch mechanism, while in NZS (New Zealand Standard) code *[NZS,1982]* , joint shear strength is evaluated by both arch and truss mechanisms. Furthermore, although the three countries agree that the concrete compressive strength and the joint area are the two most important factors to evaluate joint shear strength, they have different opinions on the effect of reinforcement ratio on joints.

Beam-column joints are critical regions of reinforced concrete frames designed for inelastic response to seismic attack. Inadequately detailed joints, especially exterior beam-column joints, may fail prematurely in a brittle manner due to high shear stresses. In earthquake-prone regions, the joints of Ductile Moment Resisting (DMR) frames must be designed and detailed to allow large energy dissipation in adjacent plastic hinges without a significant loss of strength and ductility. Designing beam-column joints is considered to be a complex and challenging task for structural engineers, and careful design of joints in RC frame structures is crucial to the safety of the structure. Although the size of the joint is controlled by the size of the frame members, joints are subjected to a different set of loads from those used in designing beams and columns. As a result, it is necessary to pay special attention to the detailing of reinforcement within a joint region.

When frames are not designed properly, the possibility of plastic hinge formation in the columns increases. This is not desirable for two reasons: firstly, the collapse mechanism associated with hinges in the columns has a lower ultimate load; and secondly, the energy absorption of plastic hinges within the columns is normally less due to reinforcement arrangement and the axial load. Engineers can avoid this when designing DMR frames by employing the principle of *strong-column weak-beam* design. According to this design principle, joints, columns and beams are designed so that the joint region and the column remain essentially elastic under the action of high lateral loads, such as earthquake and high-pressure winds, while the main energy dissipation occurs within the plastic hinges formed in the beams. Care also should be taken to make sure that plastic hinges within the beam are sufficiently distanced away from the joint. This is to ensure that penetration of plasticity to the joint core will not occur, as this may trigger a brittle failure within the core. There are several traditional ways of achieving this.

In each of the following two scenarios, rehabilitation may become necessary within the beam region adjacent to a joint:

i) REPAIR, when after a moderately large earthquake, visibly annoying cracks remain open in the beam end of the joints indicating that some residual plastic deformation is present; and
ii) RETROFIT, when detailing of the beam reinforcement is not done adequately at the design/construction stage and subsequently there is a danger of potential plastic hinge cracks penetrating to the joint core.

1.2 BEAM COLUMN JOINT

The functional requirement of a joint, which is the zone of intersection of beams and columns, is to enable the adjoining members to develop and sustain their ultimate capacity. The demand on this finite size element is always severe especially under seismic loading. The joints should have adequate strength and stiffness to resist the internal forces induced by the framing members.

1.2.1 Types of Joints in Frames

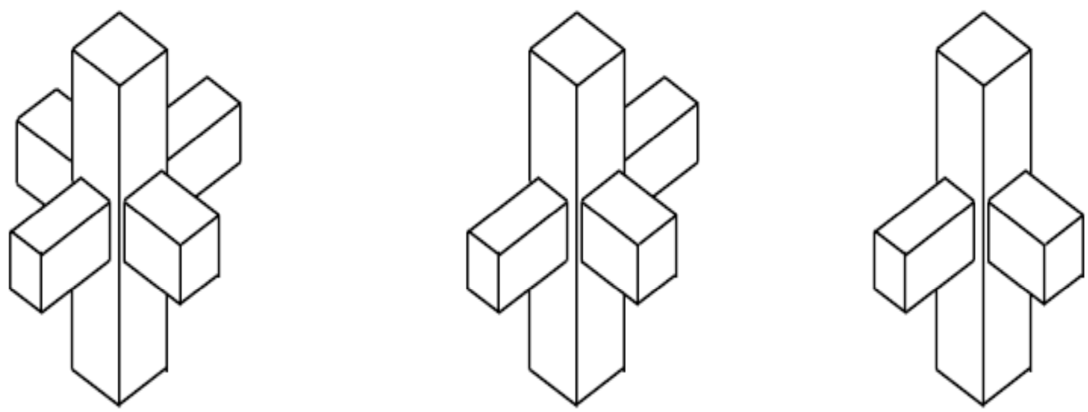
The joint is defined as the portion of the column within the depth of the deepest beam that frames into the column [Uma & Prasad]. In a moment resisting frame, three types of joints can be identified

(a) Interior joint: When four beams frame into the vertical faces of a column, the joint is called as an interior joint.

(b) Exterior joint : When one beam frames into a vertical face of the column and two other beams frame from perpendicular directions into the joint, then the joint is called as an exterior joint.

(c) Corner joint : When a beam each frames into two adjacent vertical faces of a column, then the joint is called as a corner joint. (Figure 1.1)

The severity of forces and demands on the performance of these joints calls for greater understanding of their seismic behaviour. These forces develop complex mechanisms involving bond and shear within the joint.



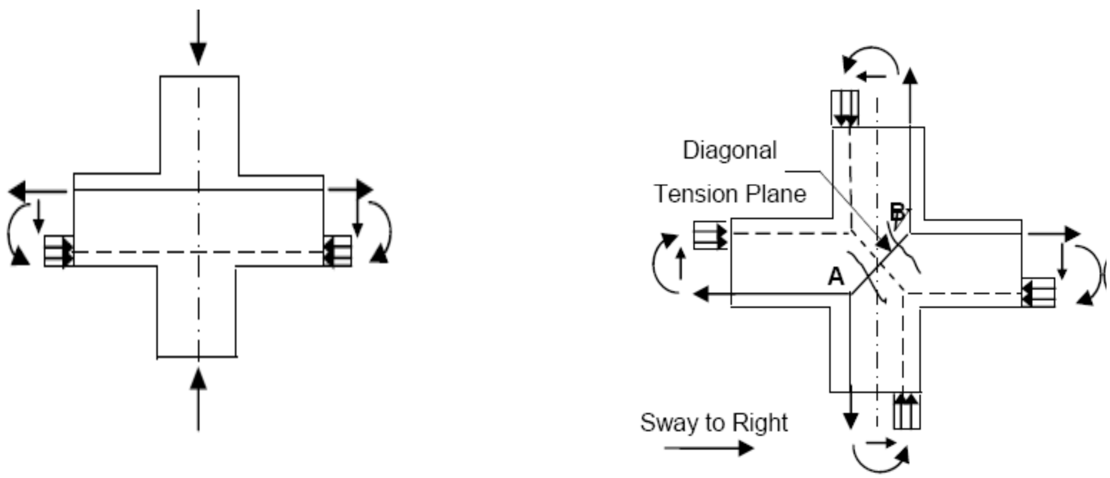
(a) Interior (b) Exterior (c) Corner

Figure 1.1 Types of Joints in a Frame

1.3 FORCES ACTING ON A BEAM-COLUMN JOINT

The pattern of forces acting on a joint depends upon the configuration of the joint and the type of loads acting on it. The effects of loads on the three types of joints are discussed with reference to stresses and the associated crack patterns developed in them [Uma & Prasad].

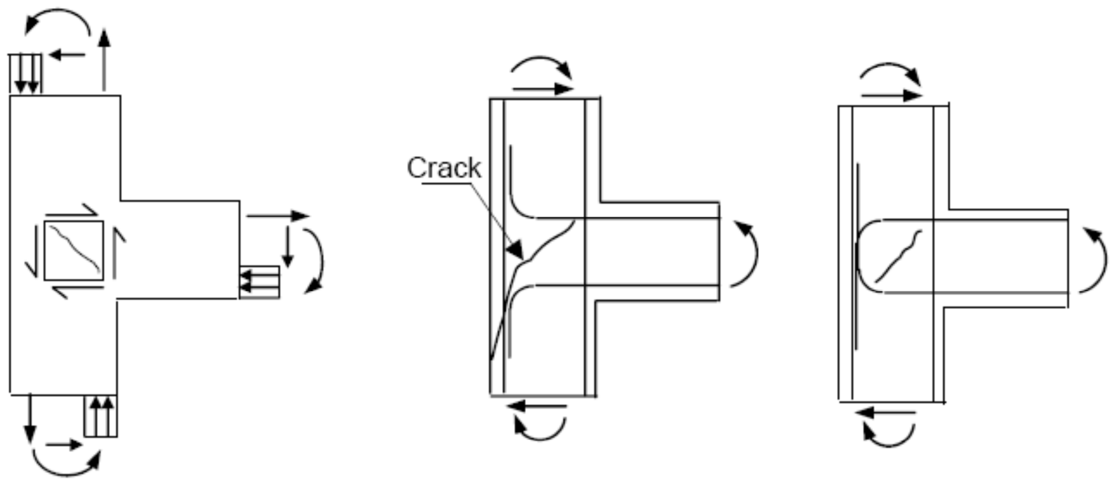
The forces on an interior joint subjected to gravity loading can be depicted as shown in Figure 1.4 (a).The tension and compression from the beam ends and axial loads from the columns can be transmitted directly through the joint. In the case of lateral (or seismic) loading, the equilibrating forces from beams and columns, as shown in Figure 4(b) develop diagonal tensile and compressive stresses within the joint. Cracks develop perpendicular to the tension diagonal *A-B* in the joint and at the faces of the joint where the beams frame into the joint. The compression struts are shown by dashed lines and tension ties are shown by solid lines. Concrete being weak in tension, transverse reinforcements are provided in such a way that they cross the plane of failure to resist the diagonal tensile forces.



a. Gravity loading **b. Seismic loading**

Figure 1.2 Interior joint

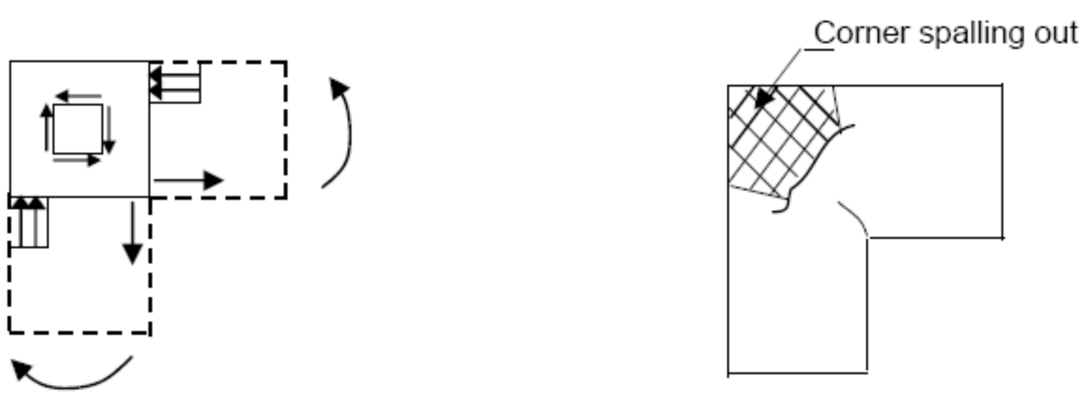
The forces acting on an exterior joint can be idealized as shown in Figure 1.5. The shear force in the joint gives rise to diagonal cracks thus requiring reinforcement of the joint. The detailing patterns of longitudinal reinforcements significantly affect joint efficiency. Some of the detailing patterns for exterior joints are shown in Figure 1.5(b) and Figure 1.5(c). The bars bent away from the joint core (Figure 1.5(b)) result in efficiencies of 25-40 % while those passing through and anchored in the joint core show 85- 100% efficiency. However, the stirrups have to be provided to confine the concrete core within the joint.



(a) Forces detail (b) Poor detail (c) Satisfactory detail

Figure 1.3 Exterior Joint

The forces in a corner joint with a continuous column above the joint (Figure 1.6(c)) can be understood in the same way as that in an exterior joint with respect to the considered direction of loading. Wall type corners form another category of joints wherein the applied moments tend to either close or open the corners. Such joints may also be referred as knee joints or L-joints. The stresses and cracks developed in such a joints are shown in Figure 1.6.



(a) Opening Joint (Top View) (b) Cracks in an Opening Joint

(c) Closing Joint (Top View) (d) Cracks in a Closing Joint

Figure 1.4 Corner joints

1.4 BOND REQUIREMENTS IN THE BEAM COLUMN JOINT

(a) Interior joint

In an interior joint, the force in a bar passing continuously through the joint changes from compression to tension. This causes a push-pull effect which imposes severe demand on bond strength and necessitates adequate development length within the joint. The development length has to satisfy the requirements for compression and for tension forces in the same bar. The distribution of bond along the longitudinal bars is shown in Figure 1.7. Insufficient development length and the spread of splitting cracks into the joint core may result in slippage of bars in the joint. *[Uma & Prasad]*

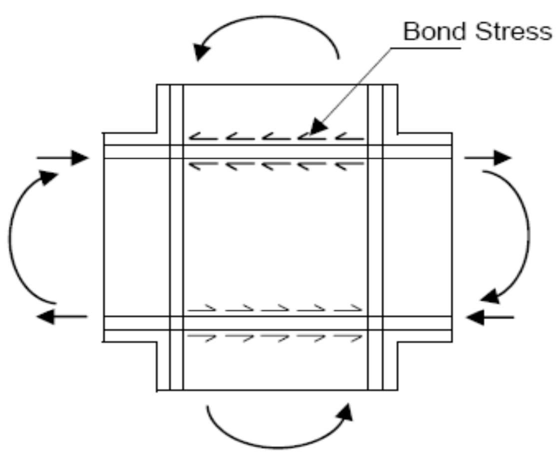


Figure 1.5 Bond stress in interior joint

Slippage of bar occurs when the limiting bond stress is exceeded within the available development length. In the case of interior joints, the column depth is the available development length for the straight longitudinal bars passing through the joint. Hence, for a given limiting bond stress, the ratio of development length to the bar diameter becomes a constant value. Research has shown that when the development length is greater than 28 bar diameters little or no bond degradation was observed with respect to various shear stress levels in the joint 7. In other words, to avoid bond deterioration, the column depth should be around 28 times the diameter of the bar. This observation suggests the adoption of relatively smaller bar diameters so as to obtain with smaller depth of columns. For example, if 20 mm nominal bar size is to be used, the member depth to be provided is 560 mm.

(b) Exterior Joint

In exterior joints the beam longitudinal reinforcement that frames into the column terminates within the joint core. After a few cycles of inelastic loading, the bond deterioration initiated at the column face due to yield penetration and splitting cracks, progresses towards the joint core. Repeated loading will aggravate the situation and a complete loss of bond up to the beginning of the bent portion of the bar may take place. The longitudinal reinforcement bar, if terminating straight, will get pulled out due to progressive loss of bond. The pull out failure of the longitudinal bars of the beam results in complete loss of flexural strength. This kind of failure is unacceptable at any stage. Hence, proper anchorage of the beam longitudinal reinforcement bars in the joint core is of utmost importance.

The pull out failure of bars in exterior joints can be prevented by the provision of hooks or by some positive anchorage *[Uma & Prasad]*. Hooks, as shown in Figure 1.8 are helpful in providing adequate anchorage when furnished with sufficient horizontal development length and a tail extension.

Because of the likelihood of yield penetration into the joint core, the development length is to be considered effective from the critical section beyond the zone of yield penetration. Thus, the size of the member should accommodate the development length considering the possibility of yield penetration.

When the reinforcement is subjected to compression, the tail end of hooks is not generally helpful to cater to the requirements of development length in compression. However, the horizontal ties in the form of transverse reinforcement in the joint provide effective restraints against the hook when the beam bar is in compression.

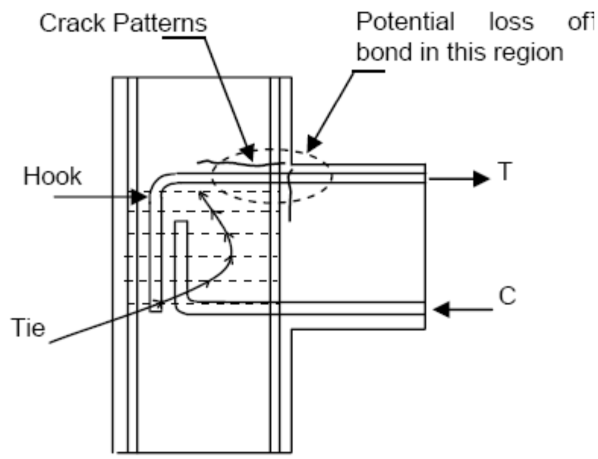


Figure.1.6 Hook in an Exterior Joint

(c) Corner Joint

In a corner joint with column continuing above and in Knee type joints, the bond requirements of longitudinal bars of beams will be similar to that in an exterior joint though there are no specific code requirements related to bond for Knee joints. However, the performance of these joints is significantly influenced by shear diagonal cracks.

Depending upon the type of improvement required for strength enhancement of beam-column joints the engineer must know about which retrofitting scheme is to be adopted. The subsequent section present details about materials used for general retrofitting.

1.5 VARIOUS MATERIALS FOR RETROFITTING

The retrofit engineer needs to have information about these materials for designing the retrofit scheme. The repair and retrofit materials can generally be classified into grouts, jacketing materials:

1.5.1 Grouts

Grout is a flow able material, which can be injected into the structure member under pressure. The grout should have negligible shrinkage to fill the gap/void completely and it should remain stable without cracking, delimitation or crumbling. Injection grout is used to fill interior space within concrete or masonry created due to cracks, voids or honey combs.

Various types of grouts used are:

(a) Injection grout

The injection grouts can be used for strengthening of those old masonry structures, in which mortar has degraded and in honey combed concrete.

(b) Cement sand grout

Cement sand grouts are cheapest. For injection purpose, the grout requires high water and cement contents. This results in shrinkage and cracking of grout at hardening. Suitable shrinkage compensating agents are required to minimize this. Use of cement- sand grout is very common in masonry buildings, but not very common in concrete.

(c) Sulfo-aluminate grout

In these grouts either shrinkage – compensating cement or anhydrous Sulfo-aluminate expensive additive is used with Portland cement. The dosages of additive are recommended at 6% to 10% by weight of cement.

(d) Polymer grout

The polymer resins grouts are most commonly used in concrete. The commonly used polymers are polyester, epoxy, vinyl ester, polyurethane and acrylic. Out of these epoxy is the most popular one. In case of underground and water seepage conditions, polyethane and acrylic resins are used. Polymer grouts can be injected by pre-mixing the resins and hardener and injecting the mix through a pressure gun fitted with a nozzle. The automatic injection machine has a con of the controlled supply of resin and the hardener through two separate pipes.

1.5.2 Bonding Agents

These agents provide enhanced bond between existing concrete and new concrete and also between concrete and reinforcement. These are very important for effective repair/retrofitting of systems. There are three methods available for enhancing the bond:

(i) Application of adhesive at the interface.

(ii) Surface interlocking

(iii) Mechanical bonding

Polymer and epoxy is the adhesive used for bonding between old and new concrete and reinforcement. After removal of the concrete cover the existing concrete surface and steel are cleaned by sand or water blasting. After cleaning and drying, concrete and steel is painted by epoxy/polymer or polymer modified cement grout. If the new steel is to be welded, it is welded prior to coating of the concrete and steel. The coating provides enhanced bond between the old and the new material and reduce the risk of corrosion in steel as well.

1.5.3 Replacement and Jacketing Material

In case of damaged structures, materials in some parts of members are to be replaced by new material. For strengthening existing members in deficient buildings, additional material including reinforcement is to be provided. The material used for replacement should have a good bond with the existing material and it should be non-shrinking. Presently a variety of strengthening and replacement material is available.

(i) Steel plate bonding

Steel plate can be bonded to concrete members as external reinforcement to increase their strength. The plates are glued to the member surface by epoxies. This requires a careful preparation of the member surface and application of epoxy layer. Steel plates can also be provided in the form of jackets either by gluing to surface or by grouting.

(ii) Polymer modified concrete and mortar (PMM/PMC)

Polymers are long molecules hydrocarbons, built by a combination of single units called monomers. The process is called polymerization. Small diameter particles of polymer emulsified in water are called polymer latexes. These latexes form continuous film at drying. The polymer can also be mixed in the form of re dispersible powder in the dry cement aggregate mix. When water is added to the mixture, a process similar to that described above takes place. Some polymers are water soluble. The PMM/PMC has better workability and water retention properties than ordinary concrete/mortar. The main advantage of PMM/PMC is its improved adhesion and bonding with existing concrete and significantly reduced permeability.

(iii) Fibre reinforced polymer/plastic

Fibre reinforced polymer/plastic is a recently developed material for strengthening of RC and masonry structure. It has been found to be an effective replacement of steel plates for strengthening of columns by exterior wrapping. The main advantage of FRP is its high strength to weight ratio and high corrosion resistance. FRP plates are 2 to 10 times stronger than steel plates, while their weight is just 20% of that of steel. However, at present their cost is high. FRP composites are formed by embedding continuous fibre matrix in resin matrix. The resin matrix binds the fibre together and also provides bond between concrete and FRP. The commonly used polymers are carbon fibre reinforced polymer (CFRP) and Glass Fibre Reinforced polymer (GFRP). These fibres are available in two forms

(i) Unidirectional tow sheets

(ii) Woven fabrics

The application of resin can be in situ or in the form of pre-fabrication of FRP plates. On the other hand, prefabricated systems offer better quality control. It is important to note the difference between the properties of steel and FRP and it should be understood that FRP cannot be treated as reinforcement in conventional RC design methods.

(iv) Ferrocement

Ferrocement is a term commonly used to describe a steel and mortar composite material. Essentially a form of reinforced concrete, it exhibits behaviour so different from conventional reinforced concrete in performance, strength, and potential application that it must be classed as a completely separate material. Ferrocement can be formed into a section less than 1 inch thick, with only an only fraction of an inch of cover over the outermost mesh layer. Conventional concrete is inch or so of concrete cover over the outermost steel rods. Ferrocement reinforcement can be assembled over a light framework into the final desired shape and mortared directly in place, even upside down, with a thick mortar paste. Conventional concrete must be cast into forms. These fairly simple differences lead to other, more remarkable differences. Thin panels of ferrocement can be designed to levels of strain or deformation, with complete structural integrity and water tightness , far beyond limits that render conventional concrete useless .Ease of fabrication makes it possible to form compound shapes with simple techniques; with inexpensive material ; and , if necessary ,unskilled (but supervised) labour.

Ferrocement is a versatile construction material and confidence in the material is building up resulting in its wider application especially in developing countries such as for housing, sanitation, agriculture, fisheries, water resources, water transportation freshwater and marine environment, biogas structure, repair and strengthening of older structures, and others. Considered to be an extension of reinforced concrete, ferrocement has relatively better mechanical properties and durability than ordinary reinforced concrete. Within certain loading limits, it behaves as a homogeneous elastic material and these limits are wider than for normal reinforced concrete reinforced concrete. The uniform distribution and high surface area to volume ratio of its reinforced results in better crack arrest mechanism i.e. the propagation of cracks are arrested resulting in high tensile strength the material.

1.6 FERROCEMENT

Ferrocement is also written as ferrociment, ferrocemento, ferrocimento, and ferrozement, which literally means much steel rather than much concrete. Ferrocement is sometimes referred to as *thin-shell concrete*. In some cases, it is more desirable to have only one strong direction. This type of ferrocement, that has an orthotropic behaviour, is achieved by using expanded steel meshes. Slitting steel sheets and expanding them in a direction perpendicular to the slits form expanded steel sheets. Expanded steel sheets have a diamond-shaped mesh pattern. Rolling could flatten these sheets and enhances their performance as reinforcement in concrete or mortar *[Khaloo & Morshed 2000]*. This type of ferrocement is stronger and relatively stiffer in the long diagonal direction of diamonds and has lower strength and stiffness in the perpendicular direction *[Naaman 2000].* Ferrocement is a composite material consisting of rich cement mortar matrix uniformly reinforced with one or more layers of very thin wire mesh with or without supporting skeletal steel. Its properties vis-a-vis RCC are given here in **Table 1.1**

Ferrocement when used in retrofitting requires primarily looking at the point of application, and then the meshing or reinforcement is applied at the required point. This can be done with the use of studs, fasteners and covering it with cement plaster. The development of ferrocement evolved from the fundamental concept behind reinforced concrete i.e. concrete can withstand large strains in the vicinity of the reinforcement & magnitude of the strains depends on the distribution & subdivision of the reinforcement throughout the mass of concrete. Ferrocement behaves as a composite because the properties of its brittle mortar matrix are improved due to the presence of ductile wire mesh reinforcement. Its closer spacing of wire meshes (distribution) in the rich cement sand mortar & the smaller spacing of wires in the mesh (subdivision) impart ductility & better crack arrest mechanism to the material.

Over the last two decades, many researchers initiated studies to determine the mechanical properties of ferrocement as well as its potential use in construction applications. It has been showed that the conventional methods used for reinforced concrete analysis are valid to predict load–deflection relationship of ferrocement. Studies on the tensile properties of ferrocement indicated that the ultimate load of the composite material was equal to the load carrying capacity of the reinforcement in the loading direction and that the geometry of the mesh influenced the behaviour of ferrocement *[Naaman & Shah 1971].*

Table 1.1: Properties of Ferrocement & RCC

S. No.	Ferrocement	Reinforced cement concrete
1	It is a steel mortar composite material	It is a mix of coarse aggregate, fine aggregate, cement & steel
2	Its reinforcement consists of closely spaced, multiple layer of steel mesh completely impregnated with cement mortar	Reinforcement is at tensile & compressive faces with shear stirrups
3	It can be formed into sections about 20- 40 mm thick	Minimum section thickness varies from 100 mm for thin shells to 1200 mm in large beams
4	Only a fraction of an inch or 2.5 mm of cover over the outermost mesh layer	Minimum cover to reinforcement is one inch or 25 mm
5	Ferrocement reinforcing can be assembled over a light or no framework into the final desired shape and mortared directly in place, even upside down, with a thick mortar paste	Heavy framework required to support massive weight of concrete
6	Thin panels of ferrocement can be designed to levels of strain or deformation, with complete structural integrity and water tightness	Very thick panels of concrete required for water tight section due to relatively high permeability of concrete
7	Ease of fabrication makes it possible to form compound shapes	Compound shapes require difficult & cumbersome framework.
8	The uniform distribution and high surface area to volume ratio of its reinforcement	Percentage of reinforcement varies from 1-4%
9	A high degree of toughness, ductility, durability, strength & crack resistance	Relatively low degree of toughness, ductility, durability, strength & crack resistance
10	Self weight of ferrocement elements per unit area is quite small	About 5-10 times heavier

1.7 HISTORICAL BACKGROUND

The credit of using ferrocement in the present day goes to Joseph Louis Lambot who in 1848 constructed several rowing boats, plant pots, seats & other items from a material he called fercement. Lambot's construction consisted of a mesh or grid reinforcement made of two layers of small diameter on bars at right angle & plastered with cement mortar with a thin cover to reinforcement Lambot's rowboats were 3.66 m long, 1.22 m wide & 25 mm to 38 mm thick. These were reinforced with grid & wire netting. One of the boats built by him, still in remarkably good condition, is on display in the museum at Brignoles, France.

There was very little application of true ferrocement construction between 1888 & 1942 when Pier Luigi Nervi began a series of experiments on ferrocement. He observed that reinforcing concrete with layers of wire mesh produced a material possessing the mechanical characteristics of an approximately homogenous material capable of resisting high impact. After the Second World War, Nervi demonstrated the utility of ferrocement as a boat building material.

In 1945, Nervi built the 165 ton Motor Yatch "Prune" on a supporting frame of 6.35 mm dia rods spaced 106 mm apart with 4 layers of wire mesh on each side of rods with total thickness of 35 mm. It weighed 5% less than a comparable wooden hull & cost 40% less at that time.

In 1947, Nervi built first terrestrial ferrocement structure, a storage warehouse of about 10.7 m x 21.3 m .size. The strength of the structure was due to the corrugations of the wall & the roof which were 44.45 mm thick.

In 1948 Nervi used ferrocement in first public structure, the Tutrin Exhibition building. The central hall of the building which spans 91.4 m, was built of prefabricated elements Connected by reinforced concrete arches at the top & bottom of the undulations. In 1958, the first ferrocement structure - a vaulted roof over shopping centre was built in Leningrad in Soviet Union.

In 1970, a prototype prefabricated ferrocement home was constructed in U.S.A. The house was found much lighter in weight & higher in resistance to dynamic load than the conventionally built brick or block house.

In 1972, the US National Academy of sciences through its board on sciences & technology for international Development established an adhoc panel on the utilization of ferrocement in developing countries.

In 1974, the American Concrete Institute formed committee 549 on ferrocement.

In 1975, two ferrocement aqueducts were designed & built for rural irrigation in China.

In 1976, the International Ferrocement Information Centre (IFIC) was founded at Asian institute of Technology, Bangkok, Thailand. The centre is financed by the United States Agency for International development, the Government of New Zealand & the International Development Research Centre of Canada.

In 1978 an elevated metro station of 43.5 m x 1.6 m in size with continuous ferrocement roofing was erected in Leningrad.

In 1979 RILEM (International Union of Testing & research Laboratories of materials & structures) established a Committee (48-FC) to evaluate testing methods for ferrocement.

In 1984, ferrocement was used in the construction of a shaking table of large scale earthquake simulation facility at the state university of New York at Buffalo.

Recently, it has been reported that the Chinese have been building ferrocement boats even before world war second. It is estimated that they have built 2000 boats. Most of these boats are 12 m to 15 m long & are mainly used in carrying goods.

1.8 CONSTITUENT MATERIALS

The constituent materials of ferrocement are discussed in the succeeding sub sections.

(a) Reinforcing Mesh

One of the essential components of ferrocement is wire mesh. Different types of wire meshes are shown in figure 1.9 and these are available almost everywhere, they generally consist of thin wires, either woven or welded into the mesh but main requirement is that it must be easily handled and if necessary, flexible enough to be bent around sharp corners. The function of wire mesh and reinforcing rod is to provide the form and to support the mortar in its green state. In the hardened state, its function is to absorb the tensile stresses on the structure which the mortar on its own would not be able to withstand. The specifications and properties of wire mesh & skeletal steel are discussed in **Table 1.2.**

(b) Cement

The cement used should confirm to IS specifications. There are several types of cements which are available commercially in the market of which Ordinary Portland cement, Portland Pozzolona cement are the two most commonly used.

(c) Aggregates

The most common aggregate used in ferrocement is sand. Sand should comply with IS standard fine aggregate. Aggregate is the term given to the inert material & it occupies 60 to

80 % of the volume of mortar. Aggregates to be used for the production of high quality mortar for ferrocement structure must be strong enough, impermeable & capable of producing a sufficiently workable mix with minimum water / cement ratio to achieve proper penetration of wire mesh. The sand cement ratio is kept from 2 to 4.

(d) Water

The quality of mixing water for mortar has a visual effect on the resulting hardened ferrocement. Impurities in water may interfere with setting of cement & will adversely effect the strength of cause staining of its surface & may also lead to its corrosion of ferrocement. Usually water that is piped from the public supplies is regarded as satisfactory. The water cement ratio is generally kept from 0.3 to 0.5.

(e) Admixtures

Admixtures are used to alter or improve one or more properties of cement mortar or concrete. Most of the admixtures are used to improve the workability, to lesson water demand & to prolong mortar setting. Admixtures can be classified into groups according to the effect they are expected to achieve. The commonly used admixtures

1. Accelerating admixtures
2. Retarding admixtures
3. Water reducing admixtures
4. Air entraining admixtures.

A new class of water reducing admixtures has emerged during last two decades, KNown as "super plasticizer". There are the high range water reducers.

(f) Coatings

To increase the durability of ferrocement, it may be protected by surface coatings, such as acrylic, latex, polyester & cement based paints.

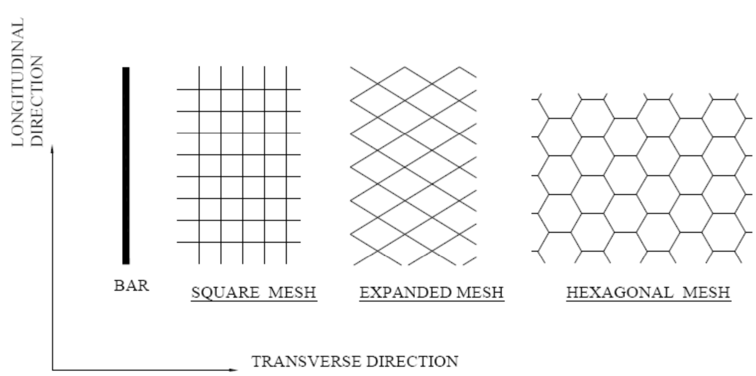


Figure 1.7 Different Types of Wire Meshes

Table 1.2 Properties of Wire-Mesh & Skeletal Steel

(A) Wire mesh		
1.	Wire diameter	0.5 mm to 3 mm
2	Size of mesh openings	Size of mesh openings
3	Volume fraction of reinforcement	Up to 8% in both directions corresponding to up to 630 kg/m as steel per cubic meter of mortar.
4	Specific surface of reinforcement	Up to 4 cm^2 /cm^3 in both directions.
(B) Intermediate skeletal reinforcement (if used) consists of wire, wire fabrics, rods & strands		
1	Diameter	3mm to 10 mm
2	Grid size	5 cm
3	Portland cement	Any type depending on application
4	Sand to cement ratio	1 to 3 by weight
5	Water to cement ratio	0.4 to 0.5
6	Sand	Fine sand passing I.S sieve No 8 & having 5% by weight passing No 100 with a continuous grading curve in between
7	Thickness	6 to 50 mm
8	Steel cover	1.5 to 5.0 mm
9	Ultimate tensile strength	34.5 N/mm2
10	Compressive strength	27.6 to 68.9 N/mm2
11	Allowable tensile strength	10.3 N/mm2
12	Modulus of rupture	55.1 N/mm2
13	Cube Strength of mortar	29.9 N/mm2

14	Young's modulus of wire mesh	2 × 105 N/mm2 for welded wire mesh 1.38 × 105 N/mm2 for woven mesh.
15	Yield strength of wire mesh	410 N/mm2 for welded wire mesh. 385 N/mm2 for woven mesh.

1.9 APPLICATIONS OF FERROCEMENT

Ferrocement has found wide spread applications in housing particularly in roofs, floors, slabs, & walls. Some researchers were also made on the use of ferrocement in beams & columns. Ferrocement roofs investigated included shell roofs, folded plates & the channel shaped roofs, box girders & secondary roofing.

Ferrocement can be effectively used for roofing for short spans. Ferrocement technology package for roofing uses state of-the-art design principles to manufacture reinforced shells. Commonly called channels they are produced on specially designed vibrating tables and profiled moulds. The production system is uniquely tailored to provide special end details, consistent shape and thickness; all crucial for high performance. They have a very high density, are impervious to penetration of water and provide high structural strength. Ferrocement roofing technology offers a viable alternative to conventional flat roofing systems such as reinforced cement concrete, reinforced brick cement, sand stone, etc. in both rural and urban areas of the country.

Ferrocement roofing channels are manufactured using designed mix of cement, sand and water to give high strength mortar that is reinforced with a layer of galvanised iron chicken wire mesh of 22 gauge and tor steel bars of 8-12 mm diameter provided in the bottom nibs of the channel. Ferrocement roofing channels can be safely transported for the application after a curing period of 14 days.

a) Advantages of ferrocement channels

● Fast construction – prefabricated channels enable to construct a roof in just 3 days

● No shuttering required, unlike in-slab slab casting

● 30% cost saving over RCC roofing

● Less dead load on the walls

● High strength to weight ratio

● Appealing aesthetics - elegant profile and uniform size

1.10 FERROCEMENT FOR BUILDINGS

Ferrocement, as already sated, is a boon for the construction industry particularly the retrofitting trade. It is also used in the pre-fabricated industry due to its light weight. Generally ferrocement roofing units are produced in factories or fabricated on site. It provides savings in the use of materials and labour for joining the smaller units. The result is a structure that is more stable, durable and requires little maintenance. Some researchers were also made on the use of ferrocement in beams and columns. Analytical and experimental investigation of hollow ferrocement units were studied by *[Mathews et, al 1998]* the system consists of top and bottom flanges connected by webs, there by leaving hollow spaces in between. The hollow section is selected mainly the passage of heat from outside. Based on the investigation the load deflection of the developed section is quite similar to that of a typical ferrocement element. There appears to be good potential for the use of these elements for roof/floor in residential buildings for span up to 3.5m *[Kaushik, et.al. 1987]* investigated the behaviour of eight simply supported concrete steel and concrete ferrocement composite slabs of span 1.5m

and 3.0 the results show that the ferrocement and corrugated galvanized iron composite slabs can be safely used for roofing and flooring purposes The ferrocement composites exhibit better performance as compare to CGI composite in terms of load carrying capacity, energy absorption capacity, ductility and recovery in unloaded.

The behaviour and performance of composite ferrocement brick reinforced slab without ferrocement panels especially to be shaped into simple geometric forms was carried out by *[Mattone 1992]*. The advantages afforded by this building technique are numerous: prefabrication ensures product quality by optimizing aggregate grain, the water cement ratio binder and addictive quantities and may entail a reduction in cost, while the simplicity of the operation to be performed to obtain a structural element from the semi-finished product make the process ideally suitable for self-help activities, enabling even unskilled workers to participate in the construction of their homes.

1.11. TRAITS OF FERROCEMENT

The traits to be kept in mind when using ferrocement are discussed below:

(i) Ease of Placement

Performance of ferrocement jacketing is highly enhanced as it can be easily placed with minimal or no form work. Cement slurry, the generally used bonding agent is very cheap & quite effective. Even though the mortar used has very less water-cement ratio still it can be worked onto the inverted beams.

(ii) Workmanship

The application of ferrocement jacketing does not require skilled labour. Even masons well versed with the art of cement plastering can do this job satisfactorily. Only minimal supervision will suffice the job in hand.

(iii) Durability

Durability is the main question about performance of ferrocement and reinforced mortar elements. Reinforced corrosion particularly seems as a first problem to be solved to give a safe margin of quality assurance to thin-walled constructions. Ferrocement or reinforced mortar members are typically built with 3 mm to 8 mm reinforcement cover thicKNess. Despite relatively low water/cement ratio recommended for the mortar mix 0.35-0.50. This is not itself enough to ensure reinforcement protection against corrosion, even if it is in orderly aggressive environments. Direct approaches to ferrocement durability problems are not given in a sufficient number.

(iv) Cost

Ferrocement uses steel wire meshes that are about 2 to 5 times more expensive by weight than ordinary steel bars. The assemblage of those meshes required medium level or no skilled labour, which is an advantage in developing countries where the cost of labour is relatively low. However, this work often takes much time and the productivity goes down. In Prefabrication plants this lack of productivity can raise the cost and so ferrocement or reinforced mortar may become non competitive against other industrialized products. The tendencies are in general to reduce the mesh content or to substitute them for other suitable meshes and fibres that may reduce the production cost. There are examples of production rationalization, by using long beds and stretching the meshes, or by using pre stressing. Application of short fibres in conjunction with continuous wires also has been proved to be economical in many situations. The application of pre-stressing techniques to ferrocement (or generally to thin walled reinforced mortar or "fine grain concrete" has a great potential in the light weight prefabrication and some of the pre-cast concrete production techniques can be adapted to ferrocement. This also should reduce the cost, because mesh content and wiring labour could be minimized.

Quality control is another important aspect in prefabrication, not only because a good quality of the elements must be reached, but also because quality control can reduce the cost.

1.12 OBJECTIVE OF THE PRESENT WORK AND ORGANIZATION OF THE THESIS

The objective of the present study is to study the effect of stress level on the retrofitting of Beam Column Joint using the ferrocement jacketing. In this study single layer of mesh wire is used as ferrocement laminates.

The thesis is organized in five chapters as follows:

Chapter-1 This chapter consist of general introduction about various retrofitting techniques and technique using ferrocement

Chapter 2 This chapter consists of literature review related to ferrocement as a retrofitting material and its effect on various properties of concrete.

Chapter 3 This chapter details experimental program. In this chapter properties of various materials used in the work have been discussed. The process has also been discussed.

Chapter 4 In this chapter the effect of stress level on the retrofitting of beam column joint has been discussed. A comparative study of ultimate load of control beam and retrofitted beam column joints are discussed.

Chapter 5 This chapter contains the salient features drawn from the experimental program and its analysis

References follow in sequence and form the end of the thesis.

2.1 PRELIMINARY REMARKS

Earthquakes are the major nemesis of all the buildings. While analysing and assessing the damage caused by the earthquakes it was inferred that a building should not suffer collapse total i.e. after a devastating earthquake it should not suffer such irreparable damage which would require demolishing and rebuilding and in case it sustain such damage it could be repaired quickly and easily to bring it to its usual functioning *[ISET, 1981]*.

In this scenario ferrocement comes to our rescue due to its ease in use and its flexibility. It is used in retro-fitting so as to strengthen the damaged structural members *[Singh and Kaushik et al, 1998]*.

Ductility requirements are the main feature of an efficient earthquake restraint design process, and Ferrocement being highly ductile material have led to its application in rehabilitation of houses damaged by earthquake and the effectiveness of its use has been reported by many researchers *[Desia, 1999, Wasti and Erberik et al, 1998]*. Reinforced concrete elements are designed to fail in a ductile manner by emphasizing on the detailing requirements due to the brittle nature of concrete *[IS 1893:2000]*.

Shear failure are also classified as brittle and shear zones are therefore reinforced by provision of stirrups for transformation to ductile failure, however, a limit is imposed on the provision to avoid brittle shear-compression failure. In the event of an earthquake, however, the shear loads can exceed shear capacities, and damage in shear zones may lead to catastrophic failure of such members.

Many experimental studies have been conducted in recent years to strengthen flexure members by using various materials.

[Andrew and Sharma 1998] in an experimental study compared the flexural performance of reinforced concrete beams repaired with conventional method and Ferrocement. They concluded that beams repaired with Ferrocement showed superior performance both at service and ultimate load. The flexural strength and ductility of beams repaired by Ferrocement was reported to be greater than the corresponding original beams and the beams repaired by conventional method.

[Al-Farabi et al 1993] while investigating the effectiveness of Fiberglass bonded plates for capacity enhancement, reported increased strength and reduced ductility. Premature failure by

plate separation was also identified as a potential problem at the plate curtailment place. Steel plates bonded by epoxy were used to repair shear cracked beams utilizing various forms of plate bonding by *[Basunbul et al 1993]*. The experimental investigation clearly demonstrated that the effectiveness of the repair primarily depends on how effectively the diagonal tension cracks in the shear-damaged beams were trapped. Flexural mode of failure was observed surpassing shear capacity for only those specimens where full encasement of the shear zone was carried.

2.2 PROPERTIES OF FERROCEMENT

Lots of research has gone into ascertaining the properties of ferrocement, its behaviour in flexure & shear, advantages & disadvantages of different types of mesh etc. The pioneer work done by few researchers is mentioned here.

[Yuzugullu 1991] reported that using expanded mesh reinforcement increases the load carrying capacity of ferrocement elements.

[*Desayi and EI- Kholy 1992]* studied the deflection and cracking of lightweight fiber reinforced ferrocement in bending proposing a bilinear equation for predicting the deflection in the portion of load-deflection curve

[G.J.Al-Sulamani et al 1992] studied the behaviour of Ferrocement under direct shear by conducting compression tests on Z-shaped specimens reinforced with wire mesh producing pure shear on shear plane. Tests results indicate that Ferrocement under direct shear exhibits two stages of behaviour (cracked and un cracked) while under flexure it exhibits a third stage i.e. plastic stage in addition. The cracking and ultimate shear stresses increase with increasing mortar strength and wire mesh reinforcement. Empirical equations have been developed here using regression on analysis to predict the cracking and ultimate shear stresses in terms of the mortar tensile strength f_t and V_f. It indicates that the shear stiffness in the un cracked stage is not significantly affected by the amount of wire mesh; it is mainly affected by the mortar strength. The shear stiffness in the cracked stage is affected by both amount of wire mesh and mortar strength. Ductility of ferrocement material under direct shear increases with increasing wire mesh reinforcement and decreases with higher mortar strength. The behaviour of ferrocement in flexure has received adequate attention by many researchers and it has been observed to be similar to the reinforced concrete members. The behaviour of ferrocement material under direct shear was investigated by conducting axial load tests on direct shear specimen. The direct shear specimen used in this study has Z-shape. It has width of 300 mm; 100 mm thickness height pf 600mm. There is a triangular notch in the middle of each side of

the specimen to force failure along the shear plane which has dimensions of 30mm×220mm.The wire mesh Layers are placed to cross the shear plane. Regular reinforcing bars are placed top and bottom blocks of the specimens to avoid any premature failure of these end blocks. Ferrocement when subjected to flexure, exhibits three stages of behaviour; un cracked, cracked and yield or ultimate stage. The third stage is an indication of the ductility that ferrocement possesses under flexure.

[Xiong and Singh 1992] investigated a qualitative mechanistic model to show the flexural fatigue of ferrocement, they showed that the rectangular stress distribution assumption is better for estimating steel stress when designing weld mesh ferrocement against fatigue.

[Kobayashi et. Al 1992] reported the properties of impact damage obtained from lateral impact test of ferrocement.

[Kahn et al 1975] to study the composite behaviour of ferrocement, they tested forty composite beams made of 0.25 in. thick steel plates and 1 in. thick plates made of either reinforced concrete (RCC) or ferrocement. They concluded the necessity of using sandblasted plates to improve the composite action between layers

[Ong et. al. 1992], provided additional data on the performance of reinforced concrete beams strengthen and repair with ferrocement laminate. The study focused on Shear connection using Ramset nails at various spacing, epoxy resin adhesive and Hilti bolts. The effects of studied. The performances of the strengthened beams were compared to the control beams with respect to cracking, deflection and ultimate strength. The results showed that all the strengthened beams exhibited higher ultimate flexural capacity and greater stiffness. The performances of the strengthened beams were compared to the control beams with respect to cracking, deflection and ultimate strength. The results showed that all the strengthened beams exhibited higher ultimate flexural capacity and greater stiffness.

2.3 REVIEW OF PAPERS OF RETROFITTING ON BEAM COLUMN JOINTS

Various papers which presented work done on the retrofitting of beam column joints are as follows:-

[Jung-Yoon Lee et al 2009], reported a method to predict the ductile capacity of reinforced concrete beam-column joints failing in shear after the development of plastic hinges at both ends of the adjacent beams. After the plastic hinges occur at both ends of the beams, the longitudinal axial strain at the centre of the beam section in the plastic hinge region is expected to increase abruptly because the neutral axis continues to move toward the extreme compressive fibre and the residual strains of the longitudinal bars continue to increase with

each cycle of additional inelastic loading cycles. An increase in the axial strain of the beam section after flexural yielding contributes to a widening of the cracks in the beam-column joints, thus leading to a reduction in the shear strength of the beam-column joints. The proposed method includes the effect of longitudinal axial strain of a beam in the plastic hinge region of the beam on the joint longitudinal strain and the strength deterioration of the joint. In order to verify the shear strength and the corresponding deformability of the proposed method, test results of RC beam-column assembly were compared. Comparisons between the observed and calculated shear strengths and their corresponding deformability of the tested assemblies showed reasonable agreement.

[Jianchun Li et al 2002], reported that the Modelling complex concrete column–beam connection with hybrid fibre reinforced plastic (FRP) reinforcement properly requires understanding of the behaviour of such component and supporting from some experimental data for model updating and refinement. This paper, through a comprehensive experimental work, investigates the behaviour of reinforced concrete frame specimens designed to represent the column–beam connections in plane frames. As a follow-up to the previous reported work, it focuses on details of experimental analyses, in particular, a comprehensive strain analysis. Results of the analysis show that designed hybrid FRP reinforcement greatly improve the stiffness and load carrying capacity of its concrete counterpart. It also delays the crack initiation at the joint through confinement due to FRP reinforcement.

[*Abhijit Mukherjee et al 2005],* investigated the performance of reinforced concrete beam-column joints under cyclic loading is reported. Joints have been cast with adequate and deficient bond of reinforcements at the beam-column joint. FRP sheets and strips have been applied on the joints in different configurations. The columns are subjected to an axial force while the beams are subjected to a cyclic load with controlled displacement. The amplitude of displacement is increased monotonically using a dynamic actuator. The hysteretic curves of the specimens have been plotted. The energy dissipation capacity of various FRP configurations has been compared. In addition, the control specimens have been reused after testing as damaged specimens that are candidates for rehabilitation. The rehabilitation has been carried out using FRP and their performance has been compared with that of the undamaged specimens.

Y. A. Al-Sallow et al. studied that the efficiency and effectiveness of using Carbon Fibre Reinforced Polymers (CFRP) sheets in repairing and upgrading the shear strength and ductility of seismically deficient exterior beam-column joint. For this purpose, a reinforced concrete exterior beam-column sub-assemblage was constructed with non-optimal design parameters (inadequate joint shear strength with no transverse reinforcement) representing pre-seismic code design construction practice of joints and encompassing the vast majority of existing beam-column connections. The specimen was subjected to cyclic lateral load histories so as to provide the equivalent of severe earthquake damage. The damaged specimen was repaired using CFRP sheets and then subjected to the similar cyclic lateral load history and its response history was obtained. Response histories of the specimen before and after repair were then compared. The results were compared through hysteretic loops, load-displacement envelops, ductility and stiffness degradation. The comparison shows that CFRP sheets improve shear resistance and ductility of the joint substantially.

[Ahmad Gobarah et al 2002], identified as the shear failure of beam-column joints and the principal cause of collapse of many moment-resisting frame buildings during recent earthquakes. Effective and economical rehabilitation techniques for the upgrade of the joint shear-resistance capacity in existing structures are needed. The objective of this research is to develop effective selective rehabilitation schemes for reinforced concrete beam-column joints using advanced composite materials. Several reinforced concrete beam-column joints were constructed. The joints were designed to simulate non-ductile detailing characteristics of pre-seismic code construction. The control specimens showed joint shear failure when subjected to cyclic loading at the beam tip. Different fibre-wrap rehabilitation schemes were applied to the joint panel with the objective of upgrading the shear strength of the joint. The tested rehabilitation techniques were successful in improving the shear resistance of the joint and in eliminating or delaying the shear mode of failure.

In this chapter the research work concerning to the various application and methods used for retrofitting of beam-column joint has been discussed.

[Bing Li and H. Y. Grace Chua et al. 2009] reported that three non-seismically detailed interior reinforced concrete beam-column joints, namely, one eccentric and two concentric joints, strengthened with proposed fibre-reinforced polymer (FRP) wrapping configurations using glass fibre-reinforced polymer and carbon fibre-reinforced polymer strips and sheets, were tested under constant axial compression load and reversed cyclic loading which simulated

low to moderate earthquake forces. Seismic performance of the strengthened beam-column joints in terms of their hysteresis response, stiffness, and energy dissipation capacity is evaluated and compared to those of the original and un strengthened beam-column joints. Results indicate that applying strips at 45º on a flushed eccentric joint core and as cross bracing on the beam and confinement round the column is very effective. All specimens failed with gradual strength deterioration, bond degradation, and de-bonding of FRP sheets was observed near the joint core. The proposed strengthening schemes were found to be efficient and economical for mass repair or upgrading of non-seismically detailed structures.

[*Kien Le-Trung et al 2009]* concluded the experimental study to strengthen the shear capacity of non-seismic joints using Carbon Fiber Reinforced Plastic (CFRP) materials. Eight exterior RC beam–column joint specimens including a non-seismic specimen, a seismic specimen and six retrofitted specimens with different configurations of CFRP sheets were developed and tested to find out an effective way to improve the seismic performance of the joints in terms of the lateral strength and ductility. The different configurations of CFRP sheets considered were the T-shape, L-shape, X-shape and strip combinations. The research focused on the effect of using CFRP sheets for enhancing strength and increasing ductility of the non-seismic beam–column joints. The test results showed that appropriately adding CFRP composites to the non-seismic specimen significantly improved the lateral strength as well ductility of the test specimens. Especially, the X-shaped configuration of wrapping, the strips on the column and two layers of the CFRP sheets resulted in a better performance in terms of ductility and strength.

3.1 GENERAL

As very little information is available in the literature on the behaviour of effect of stress level on beam column joints retrofitted using ferrocement jacketing. Thus the main objective of this experimental programme is to investigate the behaviour of beam column joints, stressed to different stress level and then retrofitted with ferrocement jacketing. To meet the objective the experimental program was devised and the details of materials used and the testing program are discussed in the subsequent sections.

3.2 DESIGN OF BEAM-COULMN JOINT

To study the proposed behaviour, eight external beam columns joints specimens were cast using M 20 grade concrete and Fe 500 grade steel. The column was square in shape with dimensions 100mm x 100 mm and the length of column was 1100 mm. The beam had dimensions 100mm x 100 mm in all test specimens and the length of beam was 500 mm. In all eight joints both beam and columns were reinforced with 4 bars of 8mm diameter. However in four joints out of these eight joints, in four joints beam reinforcement is anchored in column upto a length of 500 mm (Joint 1) and in other four joints beam reinforcement is extended upto a outer face of column (Joint 2) as shown in Plate 3.1. The Joint 1 is designed according to the design provisions provided in IS 13920. In the Joint 1 (J 1), the column main reinforcement consisted of 4 no's of 8 mm diameter whereas in the beam portion, the reinforcement consisted of 2 no's of 8 mm diameter bars in tension zone and 2 no's of 8 mm diameter in the compression zone and from the face of beam, anchorage length of 500 mm to both sides of column were provided. In the Joint 2 (J 2), same reinforcement is provided in beam face as well as in the column face, whereas there is no anchorage length from beam face to column face. The anchorage length from the beam face to the column face is not provided in Joint 2 (J 2)

The ties for both the specimens consist of square hoops of 6 mm diameter of size 60 mm x 60 mm placed 100 mm c/c in the column portion as well as in the beam portion. The reinforcement detailing of Joint 1 and Joint 2 are shown in the Plate 3.1.

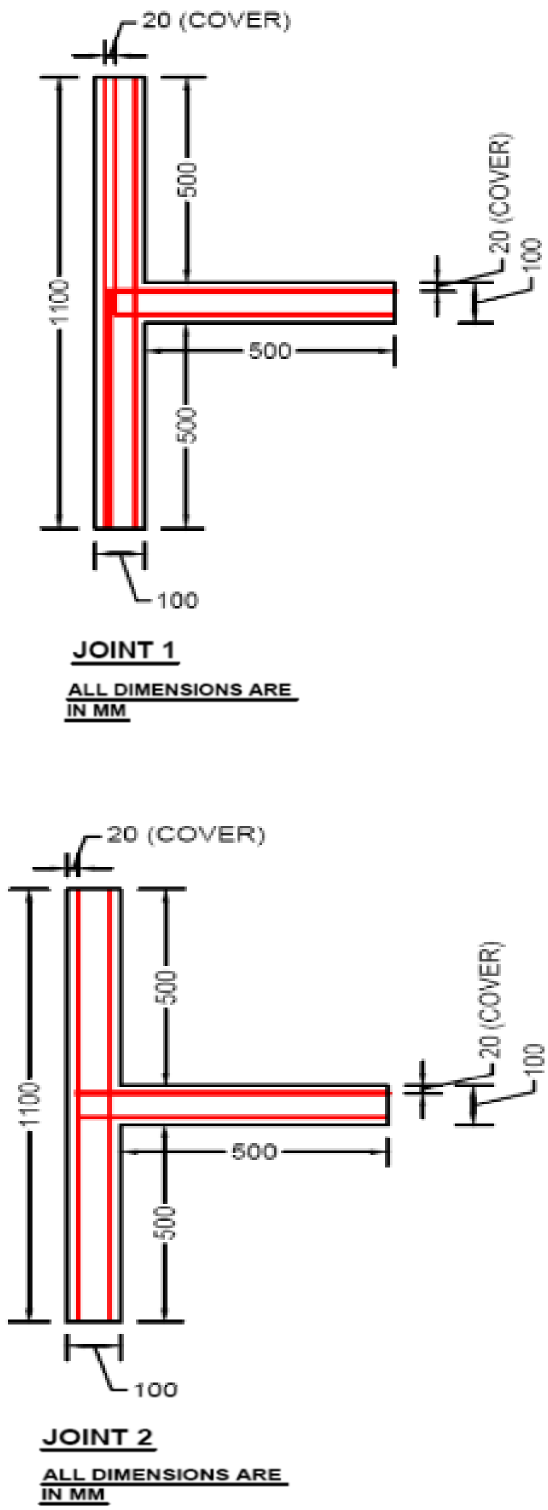


Plate 3.1: Reinforcement detail of Joint 1 and Joint 2

The specimen was fixed on loading frame using the arrangement shown in the Plate 3.2 .The joints were subjected to point load at a distance of 275 mm from the face of column. Three dial gauges were set at a distance of 100 mm, 200 mm, and 500 mm under the beam from the column face to note the deflection in the beam. The casted specimens were tested using the hydraulic jack. The Plate 3.3 and Plate 3.4 show the details of hydraulic jack and data acquisition system used for the test.

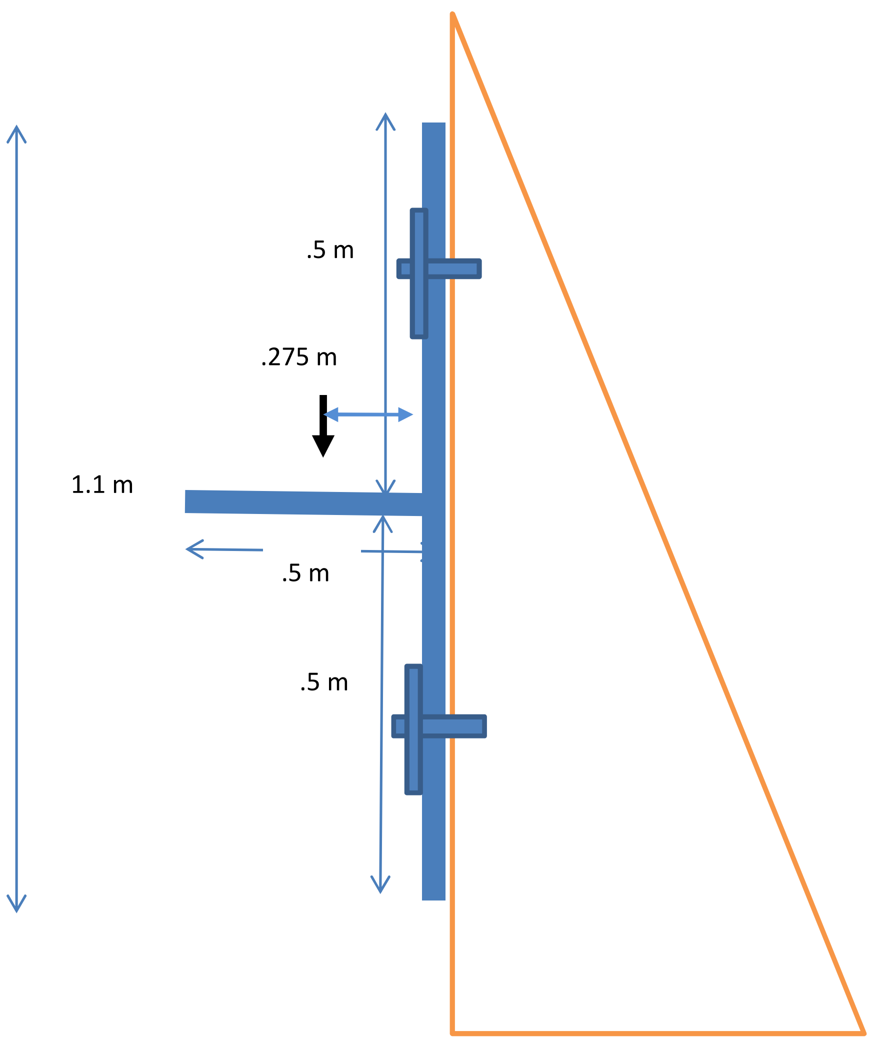


Plate 3.2: Beam Column Specimen attached with frame

Plate 3.3: Testing Jack Showing Full Travel of the Jack

Plate 3.4 Data Acquisition System Connecting to Load Cell

3.3 TESTING PROCEDURE

3.3.1 Isometric and Cross Sectional View of Joints

The isometric view and cross sectional views of the joints are shown in Plate 3.5

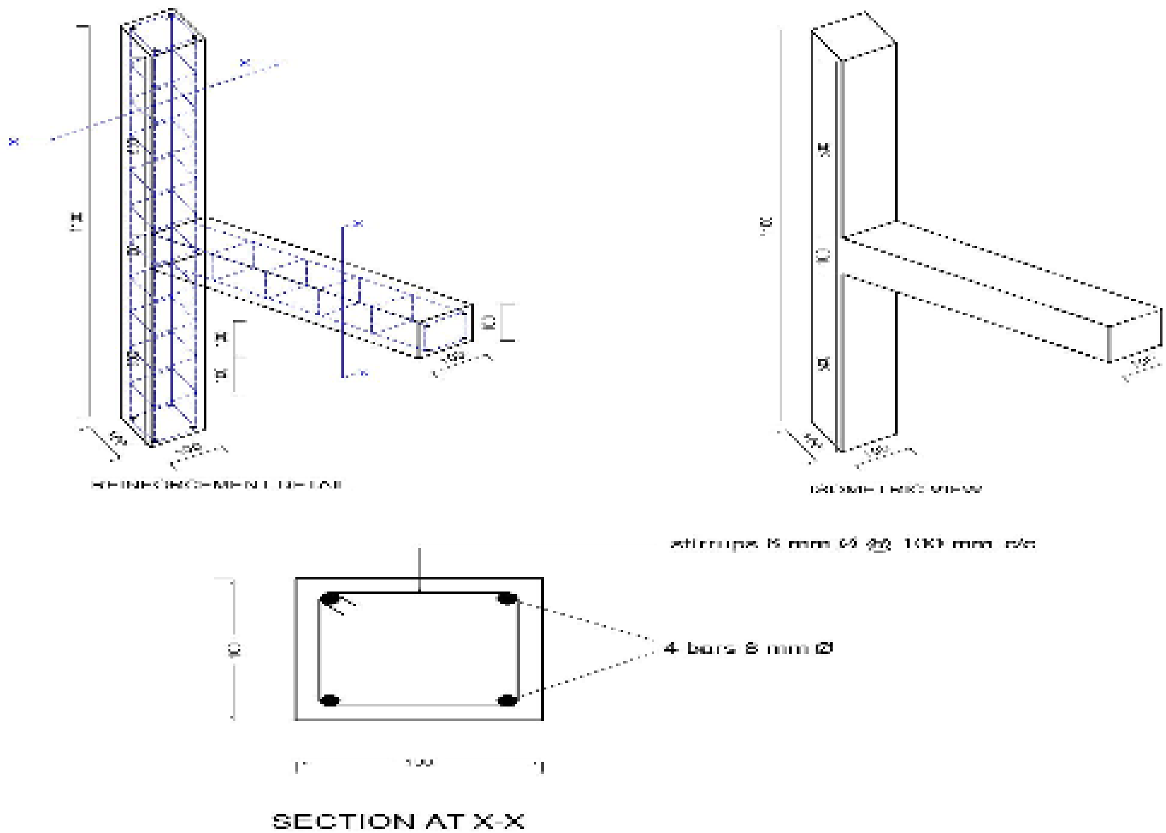


Plate 3.5: Detail of Beam Column Joint

3.3.2 Casting of Beam Column Joints

The casting of the joints was done in the single stage. A wooden mould was made of dimensions 100 x 100 mm for the beam portion and 100 x 100 mm for the column portion. The wooden mould is shown in the Plate 3.6. With the help of the drill machine 8 holes were drilled in the mould. Cover blocks of 20 mm were placed under the reinforcement to provide uniform cover. Coarse aggregates, fine aggregates, cement and water were mixed manually as per the proportion of design mix. The mix proportion came out to be 1: 1.43: 2.3 (cement: sand: coarse aggregate) by weight. The strength at 7 days and at 28 days came out to be 15.56 N/mm^2 and 22.22 N/mm^2. Concrete is then poured in the mould and vibrations were given to the mould with the help of needle vibrator, so that the mix gets compacted. The vibration is done until the mould is completely filled and there is no gap left. The beam column joints were then removed from the mould after 24 hours. After de-moulding the beams are cured for 28 days in water.

Plate 3.6: Wooden Mould Used For Casting of Beam Column Joint

3.3.3 Testing of Beam Columns Joints

For the testing under the hydraulic jack a frame of triangular frame was fabricated. The moment of resistance was calculated by steel beam theory. The total moment of resistance of beam came out to be 10 kNm. Considering the ends of column as fixed ends the moments and the forces were calculated.

The specimens are attached to the frame with the help of nut-bolts. The line diagram of the specimen attached with the frame is as shown in the Plate. 3.2

2 specimens of Joint 1 (J 1) and 2 specimens of Joint 2 (J 2) were stressed to ultimate loading (100% damage). The crack pattern of the failures are shown in the Plate 3.7 and Plate 3.8

Similarly 2 specimens of Joint 1 (J 1) and 2 specimens of Joint 2 (J 2) were stressed to 50 % of ultimate loading (50% damage).

Plate 3.7 Crack Pattern of Joint 1 (100% damage)

Plate 3.8 Crack Pattern of Joint 2 (100% damage)

3.3.4 Process of Retrofitting

The eight beam column joints (4 specimens of Joint 2 and 4 specimens of Joint 1) which were stressed to 100 % and 50% of the ultimate load are now ready for retrofit. Firstly, the surface of beam is cleaned. The slurry is applied on all the sides of beam column joints for ensuring a proper bond of cement mortar with RCC beam. The retrofitting scheme consists of wrapping the beam portion and column portion with the help of the rectangular wire mesh. The arrangement of wire mesh provided is shown in the plate 3.9. Now cement slurry is applied as bonding agent to the surface of beam. After the application of bonding agent retrofitting of beam column joint is done by applying 20 mm thick cement mortar made of ratio 1:3 and having water cement ratio (w/c) equal to .45. The beams are cured with jute bags for 7 days before testing. They are then tested with the same procedure as adopted during the testing of control beams to calculate ultimate load and corresponding deflections.

Plate 3.9 Specimen Retrofitted with Mesh Wire

3.4 MATERIALS USED

Cement, fine aggregates, coarse aggregates, reinforcing bars are used in casting of control beams and ferrocement is used for retrofitting of these beams. The specifications and properties of these materials are as under:

(a) Cement: Portland Pozzolona Cement from a single lot was used for the study. The physical properties of cement as obtained from various tests are listed in Table 3.1. All the tests are carried out in accordance with procedure laid down in IS 1489 (Part 1):1991.

(b) Fine Aggregates: Locally available sand was used as fine aggregate in the cement mortar and concrete mix. The physical properties and sieve analysis results of sand are shown in Table 3.2 and 3.3.

(c) Coarse Aggregates: Crushed stone aggregates (locally available) of 10mm were used throughout the experimental study. The physical properties and sieve analysis of coarse aggregate are given in Table 3.4 and Table 3.5.

(d) Water: Fresh and clean water is used for casting the specimens in the present study. The water is relatively free from organic matter, silt, oil, sugar, chloride and acidic material as per Indian standard.

(e) Reinforcing Steel: HYSD steel of grade Fe-500 of 8mm diameter was used as longitudinal steel. 6mm diameter MS bars are used as shear stirrups. The properties of these bars are shown in Table 3.6

(f) Steel Mesh: GI steel wire mesh of 2.4 mm diameter with rectangular grids was used in ferrocement jacket. The grid size of mesh was 45 × 12 mm. The salient properties of mesh wire used are given in Table 3.6. M20 grade concrete mix is designed as per IS code design procedure using the properties of materials as discussed below i.e. Table 3.1 to Table 3.6. The water cement ratio used in the design is 0.45. The mix proportion of material came out to be 1: 1.43: 2.3 (cement: sand: 10 mm coarse aggregate) by weight and compressive strength of materials after 7 days and 28 days is 21 and 26 N/mm^2 respectively.

(g) Mortar Mix: The range of mix proportion recommended for common Ferrocement applications are cement: sand ratio by weight, 1:1.5 to 1:25, but not greater than 1:3 and water cement ratio by weight, 0.35 to 0.5. The higher the sand contents the higher the required water contents to maintain same workability. Fineness modulus of the sand, water cement ratio contents to maintain ratio should be determined from trail batches to ensure a mix that can infiltrate the mesh and develop a strong and dense matrix. The proportion of cement sand mortar used for the ferrocement sheets was 1:3 (cement: sand) by weight. The water-cement ratio for mortar was 0.45 for given consistency of cement.

Table 3.1: Physical Properties of Cement

S. No	Characteristics	Value obtained Experimentally	Value specified by IS :1489-1991
1.	Standard Consistency (%)	35	
2.	Fineness of cement as retained on 90 micron sieve (in %)	2	Maximum 10
3.	Setting times(Minutes) Initial Final	 119 300	 Minimum 30 Maximum 600
4.	Specific gravity	3.12	-
5.	Compressive Strength(N/mm^2) 3 days 7 days	 16.38 22.5	 Minimum 16 Minimum 22

Table 3.2: Physical Properties of Fine Aggregates

Sr. No.	Characteristics	Value
1.	Specific gravity	2.46
2.	Bulk density(Kg/lt)	1.4
3.	Fineness modulus	2.56
4.	Water absorption (%)	0.85
5.	Grading Zone (Based on percentage passing 600 μm sieve)	Zone II

Table 3.3: Sieve Analysis of Fine Aggregates

Sr. No.	Sieve Size	Mass retained	Percentage Retained	Cumulative Percentage Retained	Percentage Passing
1	4.75mm	4.00	0.40	0.40	99.60
2	2.36 mm	75.00	7.50	7.90	92.10
3	1.18 mm	178	17.80	25.70	74.30
4	600µm	220	22.00	47.70	52.30
5	300µm	274	27.40	75.10	24.90
6	150µm	246.50	24.65	99.75	0.25
7	Pan	0.25	0.25	**∑=256.55**	

Total weight taken = 1000 gm

Fineness Modulus of sand = 2.56

Table 3.4: Sieve Analysis of Coarse Aggregates

Sr. No.	Sieve Size	Mass Retained (gm)	Percentage Retained	Cumulative Percentage Retained	Percent Passing
1	20 mm	0	0	0	100
2	10 mm	2516	83.89	83.87	16.13
3	4.75 mm	474	15.80	99.67	0.33
4	Pan	10	0.33	**∑= 183.54**	

Total weight taken = 3 kg

FM of 10 mm Coarse aggregate = $\frac{183.54+500}{100}$ = 6.83

Table 3.5: Physical Properties of Coarse Aggregates

Sr. No	Characteristics	Value
1	Type	Crushed
2	Specific Gravity	2.66
3	Total Water Absorption	0.56
4	Fineness Modulus	6.83

Table 3.6: Physical Properties of Steel Bars and Steel Mesh Wire

Sr. No	Diameter of bars/ mesh wire	Yield Strength (N/mm^2)	Ultimate Strength(N/mm^2)	Percentage Elongation (%)
1	8 mm, HYSD	560.98	645.32	19.5
2	6 mm, Mild Steel	445.5	609.20	30.2
3	2.4mm wire mesh	390.00	510.50	2.5

4.1 INTRODUCTION

In this chapter the effect of stress level on the retrofitting on different beam column joints has been discussed. Initially the control specimens were stressed to ultimate loading i.e 100% damage and 50% damage and retrofitted with the help of mesh wire. Single layer of mesh wire is used as a ferrocement laminates. A comparative study of ultimate load of control beam and retrofitted beam column joints are discussed.

4.1.1 Test Procedure

The testing of beam column joints has been done with the help of hydraulicallly operated jack connected to load cells. The load cell has been used to apply the load over the beam column specimen and the value of load is read from the data acquisition system connected to the load cell. The value of deflection has been taken with the help of dial gauges, which had been placed at distance 100 mm, 200 mm and at the free end from the face of the column.

Firstly, two specimens of Joint 1 and two specimens of Joint 2 are stressed to ultimate loading i.e (100% damage). The other two specimens of Joint 1 and two joints of Joint 2 are stressed to 50% of the ultimate loading. Then the controlled specimens are retrofitted using ferrocement a single layer of wire mesh and these retrofitted specimens are loaded to failure and the data is recorded in the form of load and deflection. The subsequent section presents the results of the controlled specimens and retrofitted specimens.

4.2 EFFECT OF RETROFITTING ON BEAM COLUMN JOINTS

4.2.1 Effect on Joints Damaged to 100 percent stress level.

This section deals with the effect of retrofitting on the beam column joints. Table 4.1 and Table 4.2 shows the value for the load deflection for control specimens (Joint 1 and Joint 2) and the retrofitted specimens (Joint 1 and Joint 2) and the same is represented grapically in the Figure 4.1 and Figure 4.2. From testing of controlled specimens which are stressed to ultimate loading (100% Damage) it is observed that:

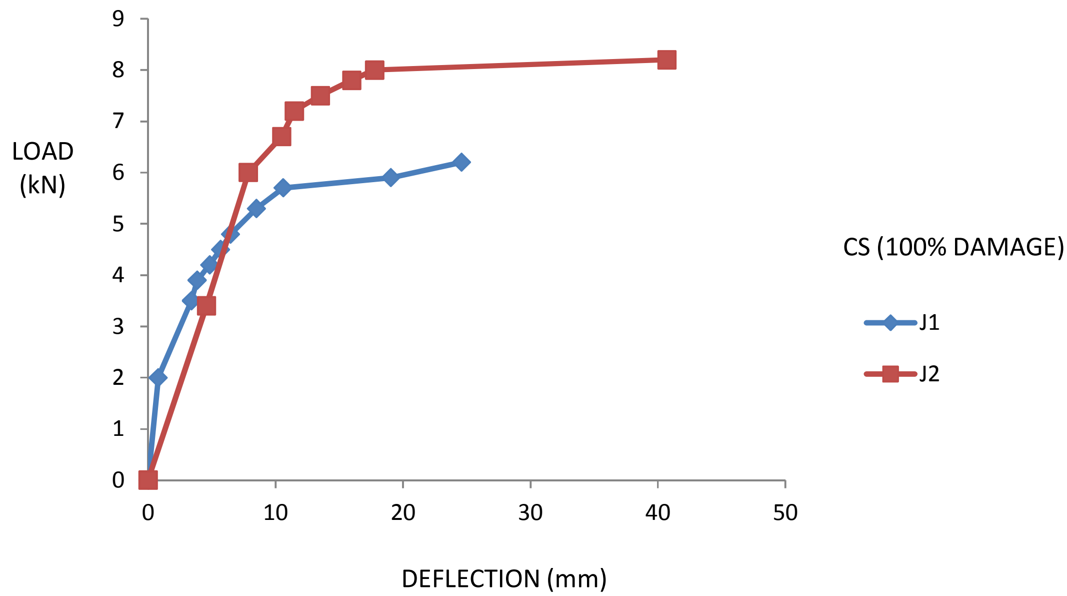


Figure 4.1 Load Deflection Curve for Controlled Specimens (100% Damage)

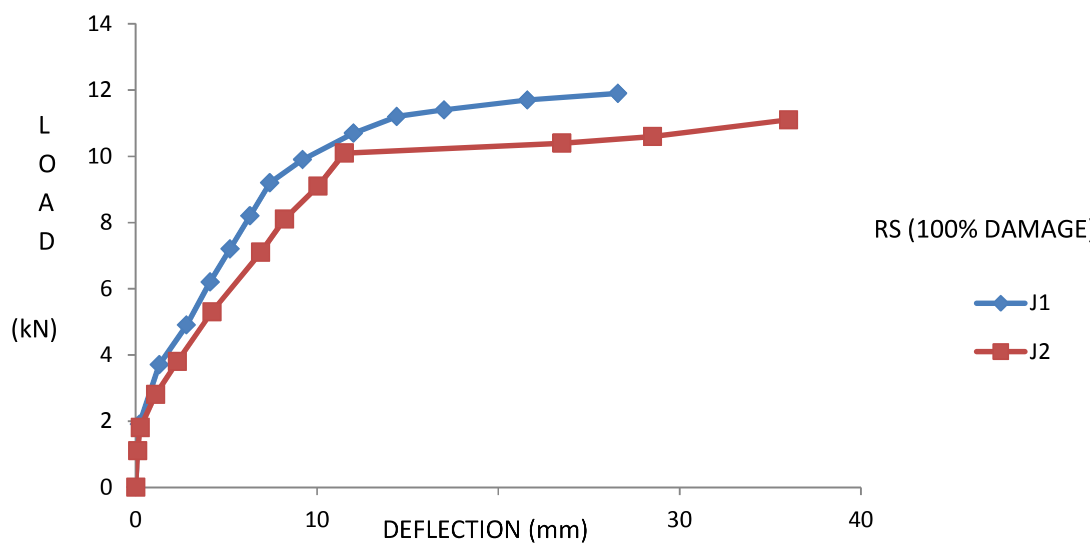


Figure 4.2 Load Deflection Curve for Retrofitted Specimens (100% Damage)

Joint 1 yields at the load of 5.7 KN and the ultimate load of 6.2 KN is reached at a deflection of 24.58 mm. The Joint 2 shows a yield load of 8 KN and the ultimate load of 8.2 KN. However the ultimate deflection for joint 2 is increased to 40.72 mm, indicationg more ductilty in joint 2.

The comparitive load deflection curves for each type of joints are presented separately in fig. 4.3 and fig. 4.4.

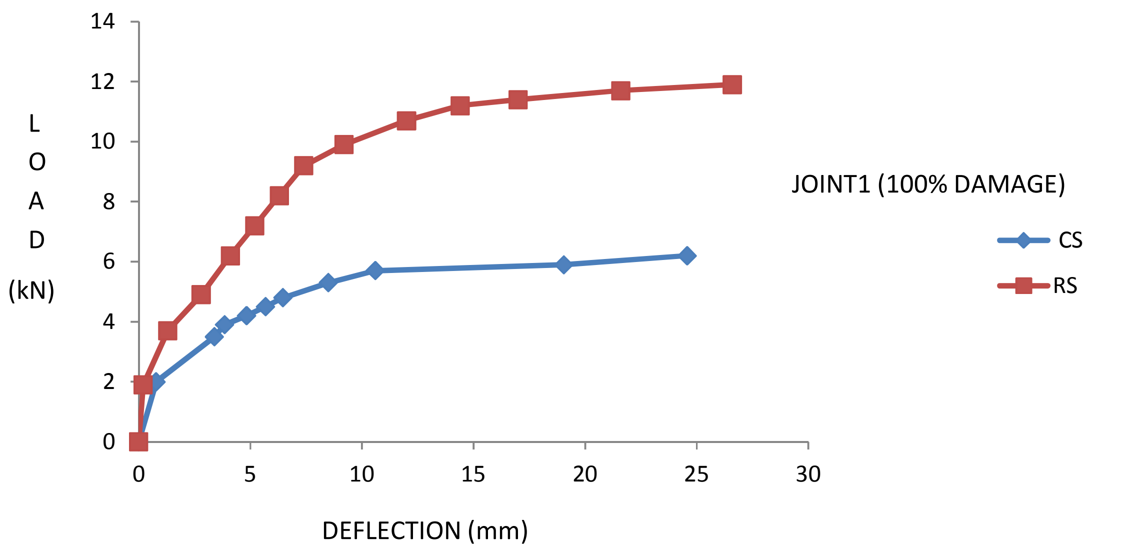


Figure 4.3 Load Deflection Curve for Joint 1 (100% Damage)

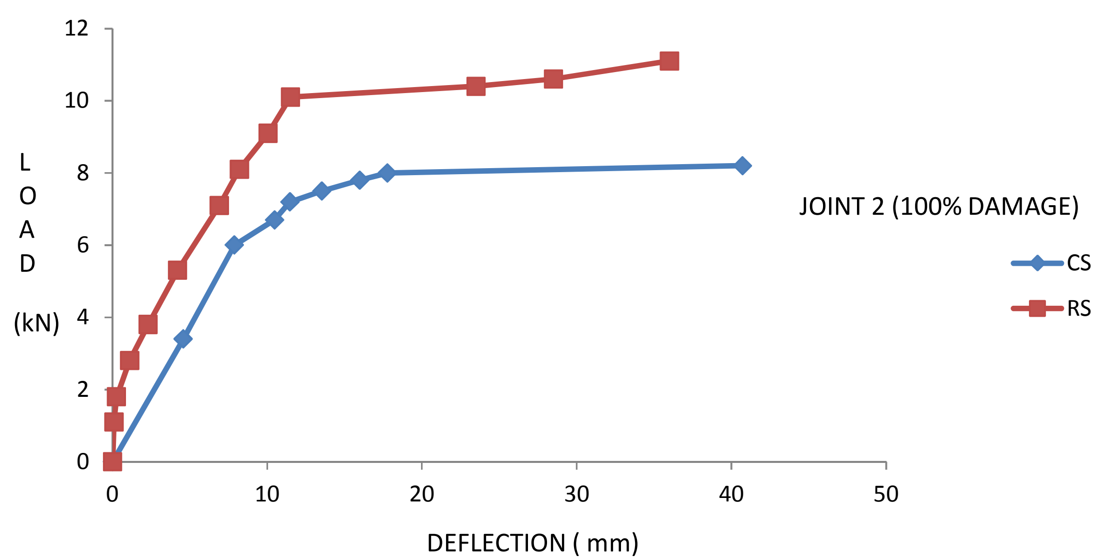


Figure 4.4 Load Deflection Curve for Joint 2 (100% Damage)

In fig. 4.3, Joint 1 when stressed to 100% damage, shows a yield load of 5.7 KN and ultimate load of 6.2 KN. After retrofitting it is observed that there is an increase in yield load and the ultimate load of the specimen. The value of yield load and ultimate load increase to 11.4KN and 11.9 KN respectively, indicating a 92 % increase for the ultimate load and about 96.5% increase in the yield load has been observed. However a very marginal increase is observed for the overall deflection for the joint 1.

In fig. 4.4, Joint 2 when stressed to 100% damage shows a yield load of 8 KN and ultimate load of 8.2 KN. After retrofitting it is obseved that there is an increase in yield load and the ultimate load of the specimen. The valuse of yield load and ultimate load increase to 10.1 KN and 11.1 KN respectively, indicating a 35.36 % increase for the ultimate load and about 26.25% increase in the yield load has been observed. However a very marginal increase is observed for the overall deflection for the joint 2.

4.2.2 Effect on Joint Damaged 50 percent stress level

This section deals with the effect of retrofitting on the beam column joints damaged to 50% stress level Table 4. and Table 4. shows the value for the load deflection for control specimens (Joint 1 and Joint 2) and the retrofitted specimens (Joint 1 and Joint 2) and the same is represented grapically in the Figure 4.5 and Figure 4.6. From testing of controlled specimens which are stressed to 50% damage it is observed that:

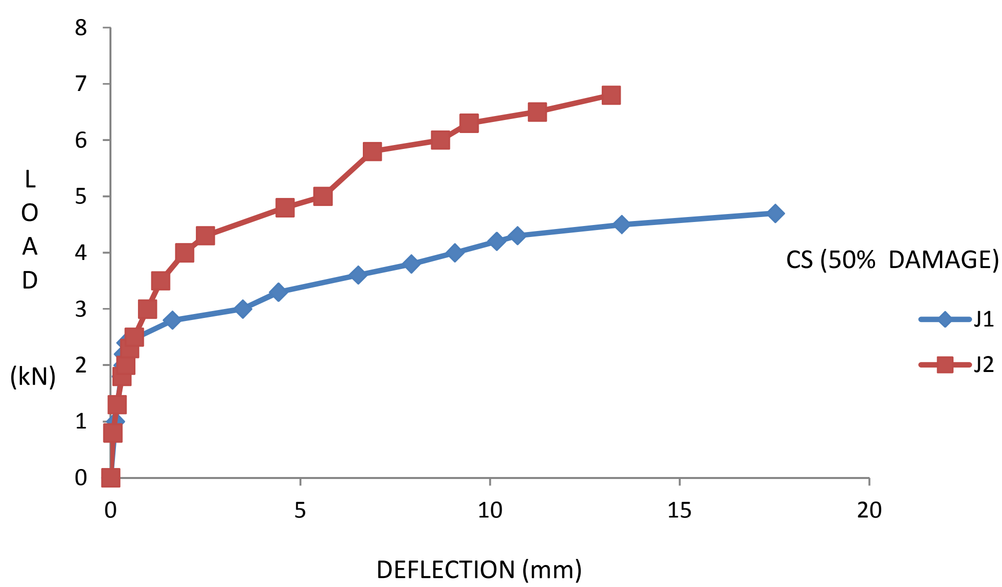


Figure 4.5 Load Deflection Curve for Controlled Specimens (50% Damage)

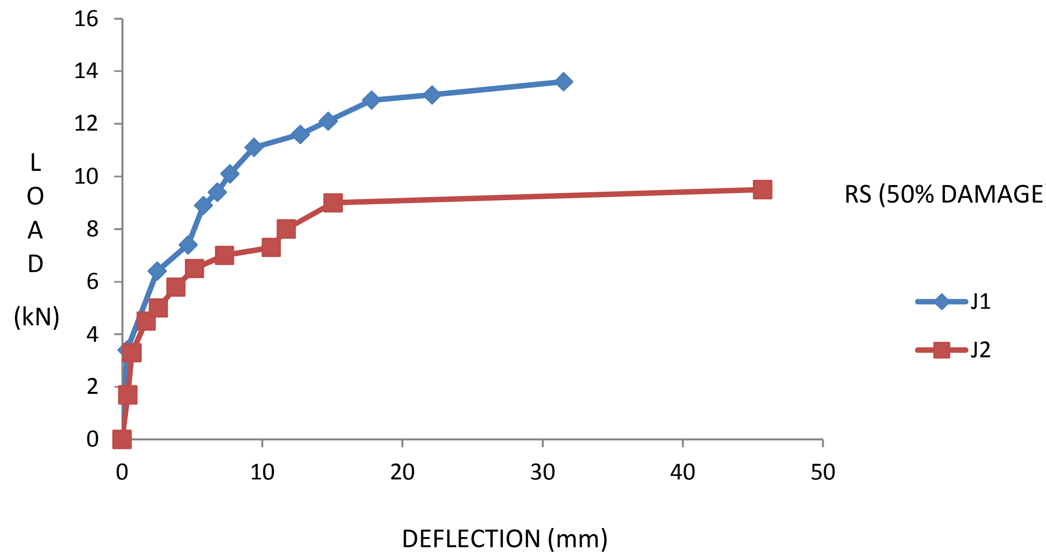


Figure 4.6 Load Deflection Curve for Retrofitted Specimens (50% Damage)

In fig. 4.5, Joint 1 (CS) the ultimate load of 4.7 KN is reached at a deflection of 17.53. The Joint 2 shows the ultimate load of 6.8 KN at a deflection of 13.2 However the ultimate deflection for joint 2 is increased to 45.7 mm, indicating more ductilty in joint 2. After retrofitting, as shown in fig. 4.8, it is observed that there is increase in yield load and the ultimate load for both joint specimens. For joint 1, the ultimate load is 13.6 KN and ultimate load for joint 2 is 9.5 KN. Joint 1 shows a considerable increase in the yield load and the ultimate load after the retrofitting as compared to Joint 2.

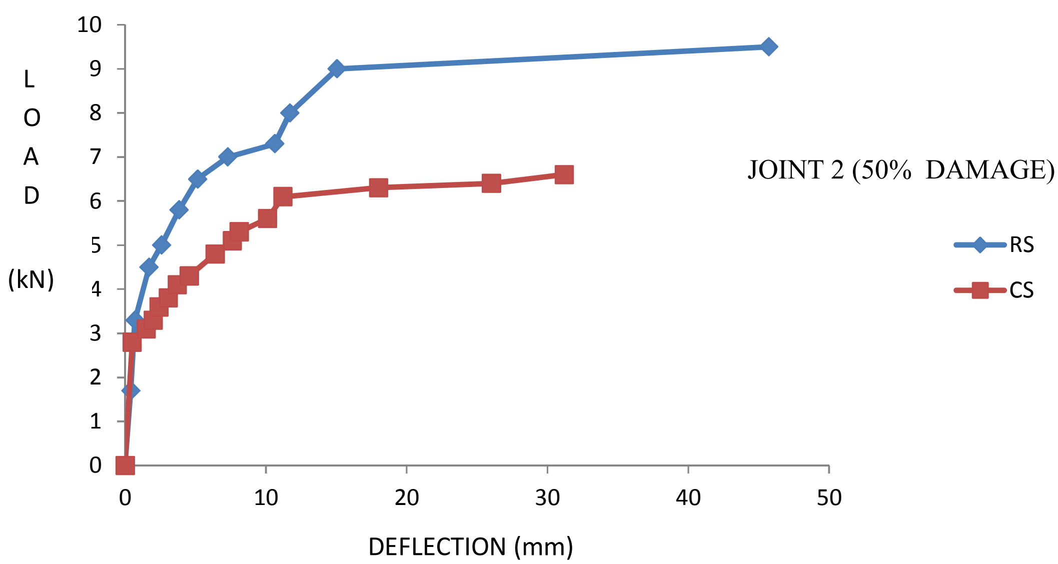


Figure 4.7 Load Deflection Curve for Joint 2 (50% Damage)

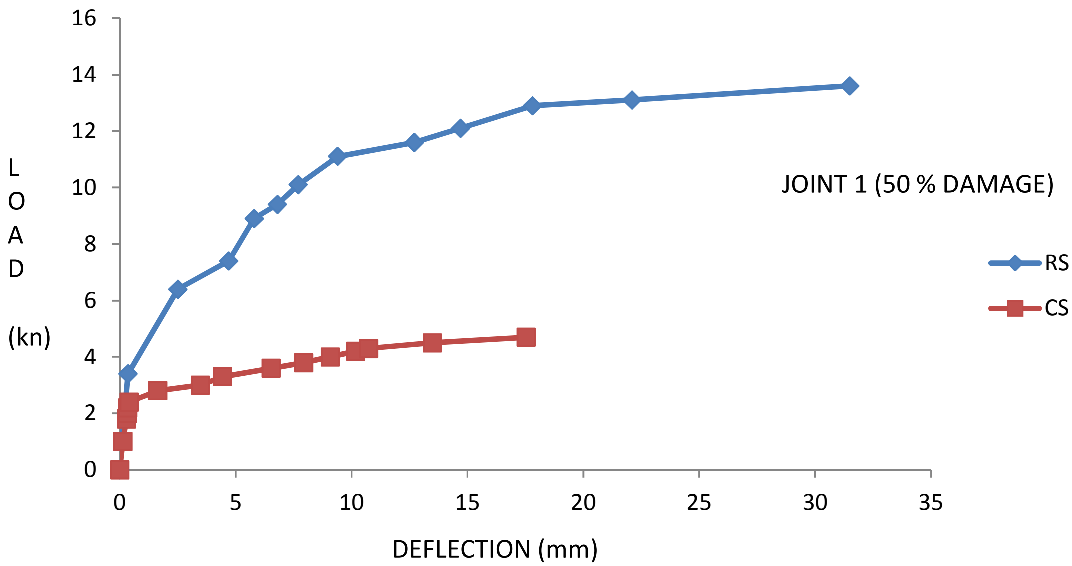


Figure 4.8 Load Deflection Curve for Joint 1 (50% Damage)

In fig. 4.7, controlled specimen Joint 2 (stressed to 50% damage) shows a yield load of 6.1 KN and ultimate load of 6.6 KN. After retrofitting it is observed that there is an increase in yield load and the ultimate load of the specimen. The value of yield load and ultimate load increase to 9 KN and 9.5 KN respectively, indicating a 44 % increase for the ultimate load and about 47.5% increase in the yield load has been observed.

In fig. 4.8, controlled specimen joint 1(stressed to 50% damage) shows a ultimate load of 4.7 KN. After retrofitting it is observed that there is an increase in the ultimate load of the specimen. The value of ultimate load increase to 13.6 KN, indicating a 190 % increase for the ultimate load.

4.3 Effect of Stress Level of Retrofitting on Beam Column Joint

a) Joint 1 (50% Damage):

When the Joint 1 is stressed to 50% damage, it shows a ultimate load of 4.7 KN. After retrofitting it is observed that there is an increase in the ultimate load of the specimen. The value of ultimate load increase to 13.6 KN, indicating a 190 % increase for the ultimate load.

b) Joint 1 (100% Damage):

When the Joint 1 is stressed to 100% damage, it shows a ultimate load of 6.2 KN. After retrofitting it is observed that there is an increase in the ultimate load of the specimen. The value of ultimate load increase to 11.9 KN, indicating a 92% increase for the ultimate load.

c) Comparison of 50% and 100% Damage for Joint 1

The results obtained from the testing of Joint 1, it is observed that ultimate load increase by 92% and 190% in case of joints stressed to 100% and 50% stress levels respectively. Hence it can be concluded that with the increse in initial stress level percentage there is an increase in load carrying capacity. However after retrofitting increase in ultimate load is observed in both cases.

d) For Joint 2 (50% Damage):

When the joint 2 is stressed to 50% damage, it shows a ultimate load of 6.8 KN. After retrofitting it is observed that there is an increase in the ultimate load of the specimen. The value of ultimate load increase to 9.5 KN, indicating a 39.70 % increase for the ultimate load.

e) For Joint 2 (100% Damage):

When the Joint 2 is stressed to 100% damage, it shows a ultimate load of 8.2 KN. After retrofitting it is observed that there is an increase in the ultimate load of the specimen. The value of ultimate load increase to 11.1 KN, indicating a 35.36 % increase for the ultimate load.

f) Comparison for 50% and 100% Damage for Joint 2

The results obtained from the testing of Joint 2, it is observed that ultimate load increase by 35.36% and 39.70% in case of joints stressed to 100% and 50% stress levels respectively. Hence it can be concluded that with the increase in initial stress level percentage there is an increase in load carrying capacity. However after retrofitting increase in ultimate load is observed in both cases.

Table 4.1: Load vs. Deflection for Controlled Specimens Joint 1 and Joint 2 (100% Damage)

Sr. No.	Joint 1		Joint 2	
	Load (kN)	*Deflection(mm)*	*Load (kN)*	*Deflection (mm)*
1	0	0	0	0
2	2	0.78	4.57	3.4
3	3.5	3.39	7.87	6
4	3.9	3.85	10.47	6.7
5	4.2	4.83	11.47	7.2
6	4.5	5.68	13.52	7.5
7	4.8	6.47	15.97	7.8
8	5.3	8.5	17.77	8
9	5.7	10.6	40.72	8.2
10	5.9	19.05	-	-
11	6.2	24.58	-	-

Table 4.2: Load vs. Deflection for Joint 1 & Joint 2 Retrofitted Specimens (100% Damage)

Sr. No.	Joint 1		Joint 2	
	Load (kN)	*Deflection(mm)*	*Load (kN)*	*Deflection (mm)*
1	0	0	0	0
2	1.9	0.2	1.1	0.1
3	3.7	1.3	1.8	0.25
4	4.9	2.8	2.8	1.1
5	6.2	4.1	3.8	2.3
6	7.2	5.2	5.3	4.2
7	8.2	6.3	7.1	6.9
8	9.2	7.4	8.1	8.2
9	9.9	9.2	9.1	10.05
10	10.7	12	10.1	11.5
11	11.2	14.4	10.4	23.5
12	11.4	17	10.6	28.5
13	11.7	21.6	11.1	36
14	11.9	26.6	-	-

Table 4.3: Load vs. Deflection for Joint 1 (100%) Damage

Sr. No.	Controlled Specimen		Retrofitted Specimen	
	Load (kN)	***Deflection(mm)***	***Load (kN)***	***Deflection (mm)***
1	0	0	0	0
2	2	0.78	1.9	0.2
3	3.5	3.39	3.7	1.3
4	3.9	3.85	4.9	2.8
5	4.2	4.83	6.2	4.1
6	4.5	5.68	7.2	5.2
7	4.8	6.47	8.2	6.3
8	5.3	8.5	9.2	7.4
9	5.7	10.6	9.9	9.2
10	5.9	19.05	10.7	12
11	6.2	24.58	11.2	14.4
12	-	-	11.4	17
13	-	-	11.7	21.6
14	-	-	11.9	26.6

Table 4.4: Load vs. Deflection for Joint 2 (100%) Damage

Sr. No.	Controlled Specimen		Retrofitted Specimen	
	Load (kN)	***Deflection(mm)***	***Load (kN)***	***Deflection (mm)***
1	0	0	0	0
2	3.4	4.57	1.1	0.1
3	6	7.87	1.8	0.25
4	6.7	10.47	2.8	1.1
5	7.2	11.47	3.8	2.3
6	7.5	13.52	5.3	4.2
7	7.8	15.97	7.1	6.9
8	8	17.77	8.1	8.2
9	8.2	40.72	9.1	10.05
10	-	-	10.1	11.5

Table 4.5: Load vs. Deflection for Joint 2 (50% Damage)

Sr. No.	Controlled Specimen		Retrofitted Specimen	
	Load (kN)	*Deflection(mm)*	*Load (kN)*	*Deflection (mm)*
1	0	0	0	0
2	2.8	0.5	1.7	0.4
3	3.1	1.5	3.3	0.7
4	3.3	2	4.5	1.7
5	3.6	2.4	5	2.58
6	3.8	3.05	5.8	3.82
7	4.1	3.7	6.5	5.15
8	4.3	4.56	7	7.3
9	4.8	6.4	7.3	10.63
10	5.1	7.6	8	11.7
11	5.3	8.1	9	15.05
12	5.6	10.1	9.5	45.7
13	6.1	11.2	-	-
14	6.3	18	-	-
15	6.4	26	-	-
16	6.6	31.18	-	-

Table 4.6: Load vs. Deflection for Joint 1 (50% Damage)

Sr. No.	Controlled Specimen		Retrofitted Specimen	
	Load (kN)	*Deflection(mm)*	*Load (kN)*	*Deflection (mm)*
1	0	0	0	0
2	1	0.13	3.4	0.35
3	1.8	0.28	6.4	2.5
4	2	0.33	7.4	4.7
5	2.2	0.34	8.9	5.8
6	2.4	0.4	9.4	6.8
7	2.8	1.63	10.1	7.7
8	3	3.48	11.1	9.4
9	3.3	4.43	11.6	12.7

10	3.6	6.53	12.1	14.7
11	3.8	7.93	12.9	17.8
12	4	9.08	13.1	22.1
13	4.2	10.18	13.6	31.5
14	4.3	10.73	-	
15	4.5	13.48	-	
16	4.7	17.53	-	

Table 4.7: Load vs. Deflection for Controlled Specimens (50% Damage)

Sr. No.	Controlled Specimen		Retrofitted Specimen	
	Load (kN)	*Deflection(mm)*	*Load (kN)*	*Deflection (mm)*
1	0	0	0	0
2	1	0.13	0.8	0.06
3	1.8	0.28	1.3	0.17
4	2	0.33	1.8	0.3
5	2.2	0.34	2	0.4
6	2.4	0.4	2.3	0.5
7	2.8	1.63	2.5	0.62
8	3	3.48	3	0.97
9	3.3	4.43	3.5	1.32
10	3.6	6.53	4	1.95
11	3.8	7.93	4.3	2.5
12	4	9.08	4.8	4.6
13	4.2	10.18	5	5.6
14	4.3	10.73	5.8	6.9
15	4.5	13.48	6	8.7
16	4.7	17.53	6.3	9.45
17	-	-	6.5	11.25
18	-	-	6.8	13.2

Table 4.8: Load vs. Deflection for Retrofitted Specimens (50% Damage)

Sr. No.	Joint 1 (50% Damage)		Joint 2 (50% Damage)	
	Load (kN)	*Deflection(mm)*	*Load (kN)*	*Deflection (mm)*
1	0	0	0	0
2	3.4	0.35	1.7	0.4
3	6.4	2.5	3.3	0.7
4	7.4	4.7	4.5	1.7
5	8.9	5.8	5	2.58
6	9.4	6.8	5.8	3.82
7	10.1	7.7	6.5	5.15
8	11.1	9.4	7	7.3
9	11.6	12.7	7.3	10.63
10	12.1	14.7	8	11.7
11	12.9	17.8	9	15.05
12	13.1	22.1	9.5	45.7
13	13.6	31.5	-	-

CHAPTER – 5 CONCLUSIONS

5.1 GENERAL

The study is carried out to analyze the effect of stress level on retrofitting of two types of Beam Column Joints using ferrocement jacketing. The important conclusions drawn from the study are as listed below:

1) Joint 2 has higher ultimate and yield load carrying capacity as compared to Joint 1 and is more ductile.
2) The load carrying capacity for fully damaged Joint 1 is doubled after retrofitting, whereas comparatively less increase is observed for Joint 2.
3) For fully damaged joints the ductility is partially restored for both the types of joints, after retrofitting.
4) For 50% damaged joint, higher increase in load carrying capacity is observed for Joint 1 as compared to Joint 2 after retrofitting.
5) For 50% damaged joint, higher ductility is achieved by Joint 2 after retrofitting.
6) Retrofitting improves the ductility of Joint 2 and the load carrying capacity of Joint 1 for 50% damaged specimens whereas only load carrying capacity is improved for both Joints which are 100% damaged.
7) For the Joint 1 as the stress level increases from 50% to 100% there is decrease in the ultimate load carrying capacity.
8) Similarly for the Joint 2 as the stress level increase from 50% to 100% there is decrease in the ultimate load carrying capacity.

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