EFFECT OF DIFFERENT WRAPPING TECHNIQUES ON RETROFITTING OF RCC BEAM COLUMN JOINTS USING FERROCEMENT

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by:-

Manzoor Ahmad Dar

Roll No.800922006

Under the guidance of

Dr. Maneek Kumar
Professor & Head, CED
Thapar University,
Patiala

Dr. Prem Pal Bansal
Assistant Prof., CED
Thapar University,
Patiala

DEPARTMENT OF CIVIL ENGINEERING

THAPAR UNIVERSITY, PATIALA-147004, INDIA

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CERTIFICATE

This is to certify that thesis entitled **"Effect of Different Wrapping Techniques on Retrofitting of RCC Beam Column Joints Using Ferrocement"**, being submitted by Mr. Manzoor Ahmad Dar (Roll No. 800922006), in partial fulfillment for award of the degree of Master of Engineering in Civil Engineering (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.

(Dr. Maneek Kumar)
Professor & Head
Department of Civil Engineering
TU Patiala (PUNJAB)

(Dr. Prem Pal Bansal)
Assistant Professor
Department of Civil Engineering
TU Patiala (PUNJAB).

Countersigned by:

(Chairman, Board of Studies)
Department of Civil Engineering
TU Patiala (PUNJAB)

(Dr. S.K. Mohapatra)
Dean Academic Affairs
TU Patiala (PUNJAB)

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Manzoor Ahmad Dar

800922006

ABSTRACT

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

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

Ferrocement as a retrofitting material can be pretty useful because it can be applied quickly to the surface of the damaged element without the requirement of any special bonding material and also it requires less skilled labour, as compared to other retrofitting solutions presently existing. The ferrocement construction has an edge over the conventional reinforced concrete material because of its lighter weight, ease of construction, low self-weight, thinner section as compared to RCC & a high tensile strength which makes it a favourable material for prefabrication also.

In the present thesis RCC beam-column joints initially loaded to a prefixed percentage of the ultimate load are retrofitted using different technique of wrapping of ferrocement to increase the strength of beam-column joint. Then the results of retrofitted beam-column joints are compared with the controlled beam-column joint. Results show that there is an increase in load carrying capacity for beam-column joint retrofitted with ferrocement and the results vary from one technique of wrapping to second technique of wrapping. Also increase in yield load is observed.

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CHAPTER 1 INTRODUCTION

1.1GENERAL

In various parts of the world, Reinforced Concrete (RC) structures, even in seismic zones are still being designed only for gravity loads. Such structures, though performing well under conventional gravity load case, could lead to a questionable structural performance under seismic or wind loads. In most cases, those structures are highly vulnerable to any moderate or a major earthquake. Along with the seismic prone zones like Himalayan region in India, Iran, Turkey, New Zealand and fault regions in US etc., devastations from earthquake have also been seen at the places believed to be seismically not-so-active (as shown in Fig. 1.1) and hence, the existing structures need immediate assessment to avoid collapse which brings a huge loss of human lives and economy that the world has witnessed for several times. Moreover, for new structures, the specifications and detailing provisions, though available to a certain extent, have to be considered in such a way that the structure would be able to efficiently resist seismic actions. Generally, a three phase approach (*Sasmal; 2009)* is followed to describe a structure under seismic loading, as underlined below:-

(1) The structure must have adequate lateral stiffness to control the inter-story drifts such that no damage would occur to non- structural elements during minor but frequently occurring earthquakes,

(2) During moderate earthquakes, some damage to non- structural elements is permitted, but the structural element must have adequate strength to remain elastic so that no damage would occur, and

(3) During rare and strong earthquakes, the structure must be ductile enough to prevent collapse by aiming for repairable damage which would ascertain economic feasibility.

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

Getting it right the first time is always the aim of designer for a new building or site. However, for various reasons, it is often necessary to improve or restore the seismic resistance of an existing building. Considering the situation for code requirements for wind and earthquakes, an example of various revisions of the UBC can be considered. This model code has been regularly issued in a new edition –lately, every three years.

The 1997 edition followed editions published in 1994, 1991, 1988, 1985, 1982, 1979, 1976, 1973, and 1971 and so on. That means that a particular building constructed in1972, for example, was probably built in confirmation with the requirements of the 1971 UBC, if that was the code jurisdiction. In 1999, therefore the building cannot be expected to have all of the features that were required by 9 upgrades of the code since 1971.

Positioning a building in time with regard to code changes may be used to anticipate what might be required for a code retrofit. If a retrofit is required for remodeling, as an example, the building must be made to comply with the present code. However, if upgrading is desired simply for its benefit, the needs may be more specifically identified by determining the construction and its design at the time that work was done.

Since codes are clearly documented, the significant changes for a particular building can be relatively easily determined. However, code changes are not the only picture; other factors may be of equal or even greater significances. Of course not all code changes have necessarily resulted in improved resistances to wind or earthquake. Adjustments in structural requirements also reflect evidence from research and changes in the availability and use of building materials and products overtime .Some of these examples follow as below:

1. The commonly used grade of steel prior to the World War II was A7 steel, shortly after the war the common grade become A36 steel. As a result, steel structure built after 1945 can be expected to have slightly bouncier floors, skinnier columns, and possible more side-sway (drift) from lateral loads.
2. Starting with the 1963 edition of the ACI code most reinforced concrete structures were increasingly designed by what is now called the strength method versus the older working stress method. In some ways, the strength method permits smaller concrete dimensions and the use of more steel and higher grades of steel. Thus as with steel structures, some increased flexibility is a possibility. But since concrete remains essentially a brittle material, some increased amount of cracking in structural actions can be expected.

3. In the 1960's increasing pressure from the building industry resulted in building code acknowledgements of the usable diaphragm capacities of wood frame surfacing materials other than plywood. Thus, gypsum drywall, plaster, stucco, and fiber panels were permitted for shear walls. This is now viewed as highly questionable by many engineers on the basis of performance of buildings in windstorms and earthquakes in the past two decades notably.

For long range survival we may eventually learn how to make stratifies and indeed whole buildings as well that can withstand the force of many earthquakes (and windstorms, firestorms, floods, etc.), and bounce back quickly with easy repairs or replacements of damaged parts. Like the demolished derby race car, they can be quickly and easily patched up and put back into the race, almost as good as new.

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 behavior 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 Figure 1.2 and Figure 1.3, offer good examples 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 behavior *(Ghobarah et al; 2002)*.

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-story moment resisting reinforced concrete frames subjected 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 et al; 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 divide joints into two categories *(ACI 352-R02)*:

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)*and AIJ guidelines (Architectural Institute of Japan), 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 *(Paulay et al; 1992)*.

1.2 RETROFITTING

The strengthening and enhancement of the performance of deficient structural elements in a structure or the structure as a whole is referred to as retrofitting. Retrofitting of a building is not same as repair or rehabilitation. Repair refers to partial improvement of the degraded strength of a building after an earthquake. In fact, it is only a cosmetic enhancement. Rehabilitation is a functional improvement, wherein the aim is to achieve the original strength of a building after an earthquake. Retrofitting means structural strengthening of a building after or before an earthquake to a predefined performance. The seismic performance of a retrofitted building is aimed higher than that of the original building. A survey of existing residential building reveals that many buildings are not adequately designed to resist earthquake. In the recent revision of the Indian earthquake code, (IS1893-2002) many regions of the country were placed in higher seismic zones. As a result many buildings designed prior to the revision of the code may fail to perform adequately as per the provisions of the new code. It is, therefore, recommended that the existing deficient buildings be retrofitted to improve their performance in the event of an earthquake and to avoid large scale damage to life and property *(Dowrick; 2003)*.

1.3 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 be classified into three categories:

1.3.1 Grouts

Grout is a flowable material, which can be injected into the structural member under pressure. The grout should have negligible shrinkage to fill the gap/void completely and it should remain stable without cracking, delamination 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:-

(i) Injection grout

The injection grouts can be used for strengthening of old masonry structures, in those cases where mortar has degraded as well as in honey combed concrete.

(ii) 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.

(iii) 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.

(iv) 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.

1.3.2 Bonding Agents

These agents provide enhanced bond between existing concrete and new concrete and 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.

1.3.3 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 good bond with existing material and it should be non-shrinking. 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 molecule 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 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) Fiber-reinforced polymer/plastic

Fiber 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 two to ten 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 fiber matrix in resin matrix. The resin matrix binds the fiber together and also provides bond between concrete and FRP. The commonly used polymers are carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP). These fibers are available in two forms:

(i) Uni-directional 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 one 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.

1.4 FERROCEMENT

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.

American Concrete Institute Committee 549 has defined ferrocement in broader sense as "a type of thin wall reinforced concrete commonly constructed of hydraulic cement mortar, reinforced with closely spaced layers of continuous & relatively small diameter mesh". The mesh may be metallic or may be made of other suitable materials. Ferrocement possesses a degree of toughness, ductility, durability, strength & crack resistance which is considerably greater than that found in other forms of concrete construction. These properties are achieved in the structures with a thickness that is generally less than 25 mm, a dimension that is nearly unthinkable in other forms of construction & a clear improvement over conventional reinforced concrete .One can certainly call it a high technology material.

The construction of ferrocement can be divided into four phases:

1. Fabricating the skeletal framing system.
2. Applying meshes.
3. Plastering.
4. Curing phase

Phase 1 & 3 require special skill while phase 2 is very labour intensive. The development of ferrocement evolved from the fundamental concept behind reinforced concrete i.e. concrete can withstand large strains in the neighbour hood 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. Due to its small thickness, the self weight of ferrocement elements per unit area is quite small as compared to reinforced concrete elements. The thickness of ferrocement elements normally ranges from 10mm to 40mm whereas in reinforced concrete elements the minimum thickness used for shell or plate element is around 75mm.Low self-weight and high tensile strength make ferrocement a favourable material for fabrication. With the distribution of small diameter wire mesh reinforcement over the entire surface, a very high resistance to cracking is obtained & other properties such as toughness, fatigue resistance, impermeability also get improved. In the past 20 years there has been an increase in the field applications & the laboratory research with this type of construction. The major differences between a conventional reinforced concrete structural element & a ferrocement member can be enumerated as follows:

1. Ferrocement structural elements are normally consists of thin sections with thickness rarely exceeding 25mm. On the other hand conventional concrete members consist of relatively thick sections with thickness often exceeding 100 mm.
2. Matrix in ferrocement mainly consists of portland cement instead regular concrete consist of coarse aggregate.
3. The reinforcement provided in the ferrocement consists of large amount of smaller dia wire or wire meshes instead of directly-placed reinforcing bars used in reinforced concrete. Moreover, ferrocement normally contains a greater percentage of reinforcement, distributed throughout the section.
4. In terms of structural behaviour, ferrocement exhibits a very high tensile strength & superior cracking performance.
5. In terms of construction, form work is very rarely needed for fabrication.

Metallic meshes are the most common type of reinforcement; meshes of alkali resistant glass fibers & woven fabric, of vegetable fibers such as jute burlaps & bamboo have also been tried as reinforcement.

1.5 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 boat build 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 × 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 91.4m, was built of pre-fabricated 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 1971 a ferrocement trowler named "Rosy in I" was built in Hong Kong. It had an overall length of 26 m & is claimed to be the world's most longest ferrocement fishing boat.

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 × 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.

1.6 CONSTITUENT MATERIALS

The constituent materials of ferrocement are:

a) Reinforcing Mesh

b) Cement

c) Aggregates

d) Mixing Water

e) Admixtures.

(a) Reinforcing Mesh

One of the essential components of ferrocement is wire mesh, different type of wire mesh shown in Fig. 1.4 is available almost everywhere. These 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 *(ACI-549)*.

(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 Portland cement is the most well-known & available everywhere. Cement of Portland variety produced today is satisfactory enough to serve the purpose of ferrocement construction *(ACI-549)*.

(c) Aggregates

The most common aggregate used in ferrocement is sand. Sand should comply with IS standard for 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 *(ACI-549)*.

(d) Mixing 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 *(ACI-549)*.

(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 lessen 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 are:

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”. These are the high range water reducers *(ACI-549)*.

1.7 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 galvanized iron chicken wire mesh of 22 gauge and tor steel bars of 8-12 mm diameter provided in the bottom ribs of the channel. Ferrocement roofing channels can be safely transported for the application after a curing period of 14 days.

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. *Kaushik et al; 1987* investigated the behaviour of eight simply supported concrete steel and concrete ferrocement composite slabs of span 1.5m and 3.0m, 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 semifinished product make the process ideally suitable for self-help activities, enabling even unskilled workers to participate in the construction of their homes.

Advantages of ferrocement channels

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

● No shuttering required, unlike in-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.

1.8 OTHER APPLICATIONS

Ferrocement applications to water resources structures are numerous. Ferrocement has been used for:-

1. Water tanks
2. Canal linings
3. Aqueducts
4. Pipes
5. Ferrocement gates
6. Culverts

Ferrocement has been widely accepted as a suitable building material for Biogas structures & marine applications such as boats, ships, pontoons, treatment plants, & floating docks etc.

In India Structural Engineering Research Centre (SERC) Ghaziabad is conducting extensive research on development of ferrocement for rural applications. This centre is concentrating efforts towards solving problems of farmers viz grain storage & water storage structures by conducting research on how ferrocement could economically be used for the manufacture of bins, silos & water tanks.

1.9 FUTURE OF FERROCEMENT IN CONSTRUCTION

Some questions are launched for argumentation on the future of ferrocement in construction which are factors that have inhibited the full development and dissemination of ferrocement technology e.g. is ferrocement cost- competitive? Is high structural performance always needed in ferrocement applications/is ferrocement durability reliable?

a) 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 construction. Ferrocement or reinforced mortar members are typically built with3mm to 8mm reinforcement cover thickness. Despite relatively low water /cement ratio recommended for the mortar mix (0.38-0.45). This is not itself enough to ensure reinforcement protection against corrosion, even if it is in orderly aggressive environments.

b) Cost

Ferrocement uses steel wires meshes that are about 2 to5 times more expensive by weight than ordinary steel bars. The assemblage of those meshes medium level or non-skilled labor, which is an advantage in developing countries where the cost of labor is relatively low. However, this work often takes much time and the productively goes down. In prefabrication plants this lack of productivity can raise the cost and so ferrocement or reinforced mortar may become no competitive against other industrialized products. The tendencies are in general to reduce the mesh content or to substitute them for other suitable meshes and fibers that may reduce the production cost. There are examples of production rationalization, by using long beds and stretching the meshes, or by using prestressing. Application of short fibers 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 lightweight prefabrication and some of the precast concrete production techniques can be adopted to ferrocement. This also should reduce the cost, because mesh content and wiring labor could be minimized. Quality control is a very important aspect in pre-fabrication, not only because a good quality of the elements must be reached, but also because quality control can reduce the cost.

1.10 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.

1.10.1 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. In a moment resisting frame, three types of joints can be identified *(Uma et al; 2003)*.

(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 1.5).

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.

1.11 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 et al; 2003)*.The forces on an interior joint subjected to gravity loading can be depicted as shown in Figure 1.6 (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 1.6(b) develop diagonal tensile and compressive stresses within the joint. Cracks develop perpendicular to the tension diagonal A-Bin 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.

The forces acting on an exterior joint can be idealized as shown in Figure 1.7. 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 1.7(b) and Figure 1.7(c). The bars bent away from the joint core Figure 1.7(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.
The forces in a corner joint with a continuous column above the joint Figure 1.8(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 1.8.

1.12 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 1.9. Insufficient development length and the spread of splitting cracks into the joint core may result in slippage of bars in the 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 *(Subramanian et al; 2003)*.

(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. Hooks, as shown in Figure 1.10 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 *(Subramanian et al; 2003)*.

(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 *(Subramanian et al; 2003)*.

1.13 LIMITATIONS OF FERROCEMENT

The disadvantage of ferrocement constructions is the labor intensive nature of it, which makes it expensive for industrial application in the western world. This "disadvantage" is the primary advantage for those who compete with western world corporations. High labor content fosters small-scale enterprises by employing low cost marginal labor to fabricate artifacts which require large labor inputs. When large industrial corporations are outside their own economic system they must compete directly without government protection. Highly motivated ferrocement entrepreneurs build aqueducts, drainage systems, water and septic tanks, large flower pots for hotels and parks, water troughs, shade roofs, small houses, etc.

1.15OBJECTIVE OF THE PRESENT WORK ANDORGANIZATION OF THESIS

The main objective of the present study, is to study the effect of different wrapping techniques on retrofitting of RCC beam-column joints using ferrocement.

The thesis has been organized into the following five chapters:

Chapter 1: This chapter deals with general introduction of retrofitting and ferrocement.

Chapter 2: This chapter consists of literature review related to ferrocement as a retrofitting material.

Chapter 3: This chapter details the experimental program. In this properties of various material used in work have been discussed.

Chapter 4: This chapter deals with the results and discussion related to the work.

Chapter 5: It details the conclusions of the work carried out

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

Fig. 1.1 Earthquake damage of RC building inter-story collapse in Bhuj, India (2001) (Sasmal, 2009).

Figure 1.2 Beam-column joint shear failure (Ghobarah et al; 2002).

Figure 1.3 Joint failures during the Kocaeli (Turkey) earthquake (Ghobarah et al; 2002).

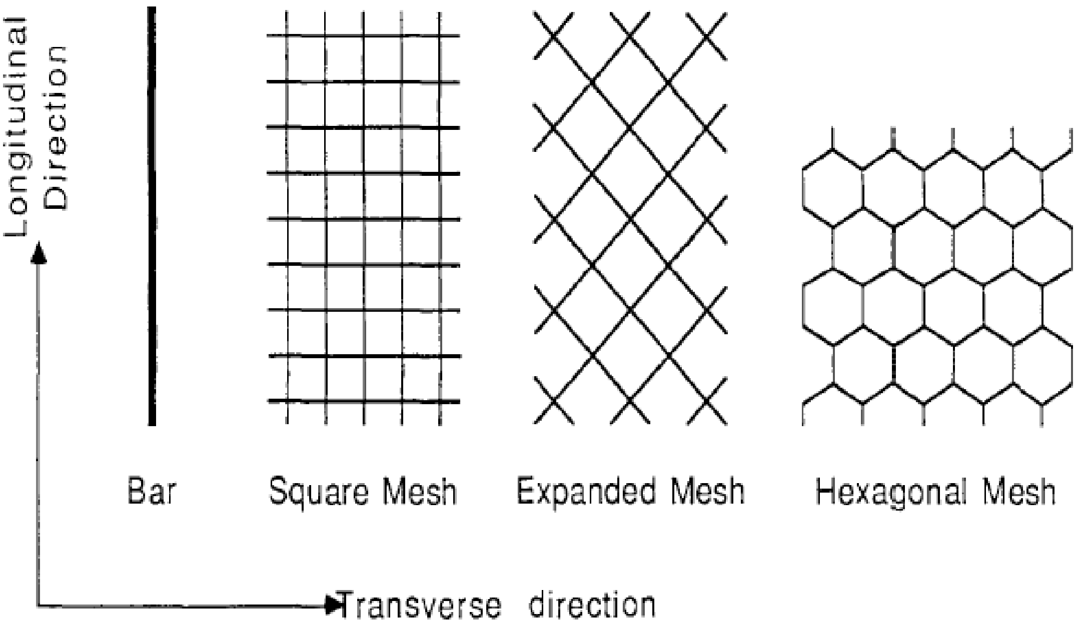
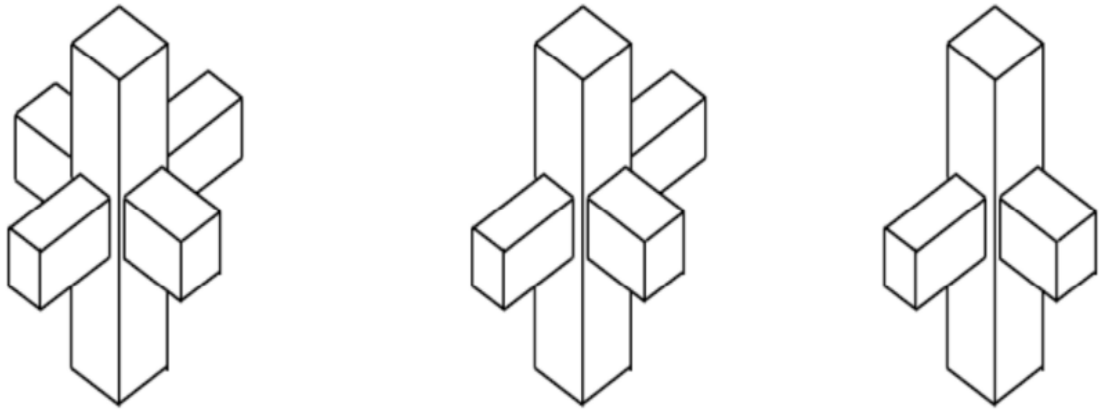


Figure 1.4 Different Types of Wire Meshes (ACI-549).

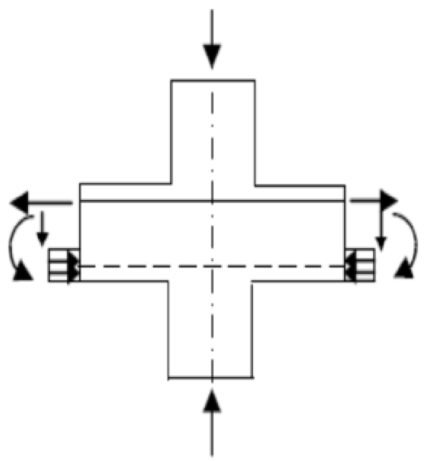


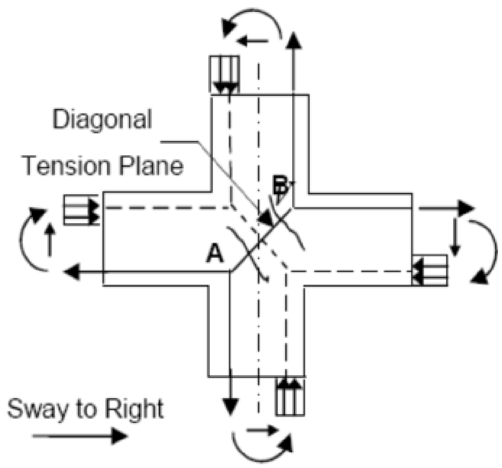
(a) Interior

(b) Exterior

(c) Corner

Figure 1.5 Types of Joints in a Frame (Uma et al; 2003).

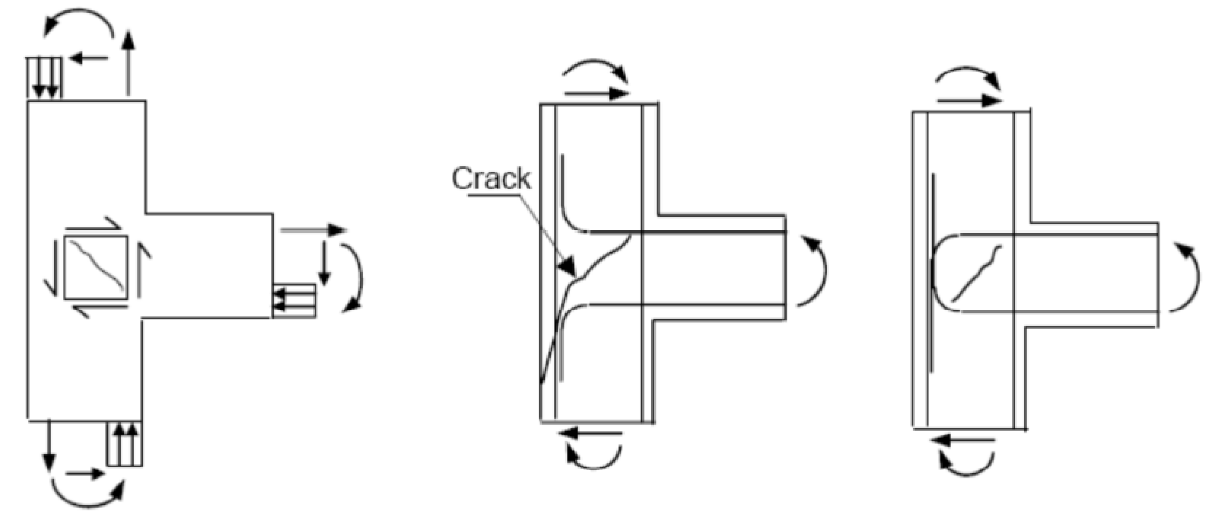




a. Gravity loading

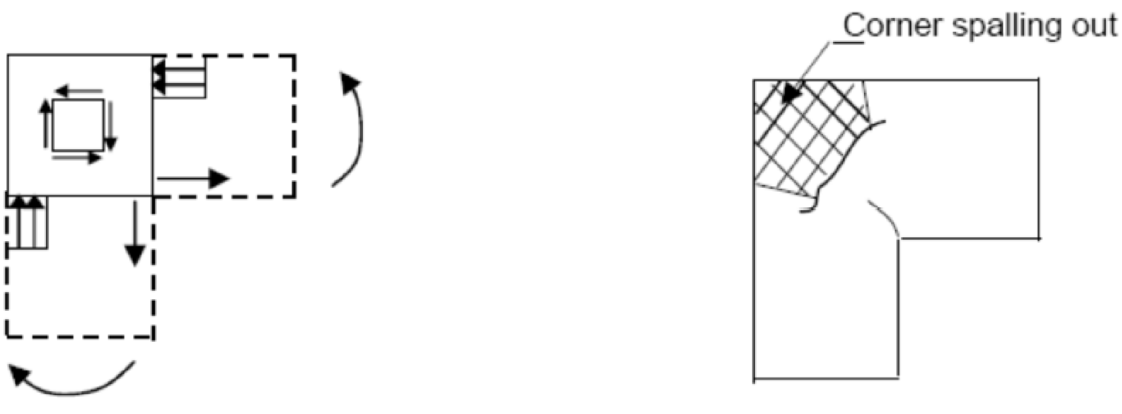
b. Seismic loading

Figure 1.6 Interior joint (Uma et al; 2003).



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

Figure 1.7 Exterior Joint (Uma et al; 2003).



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

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

Figure 1.8 Corner joints (Uma et al; 2003).

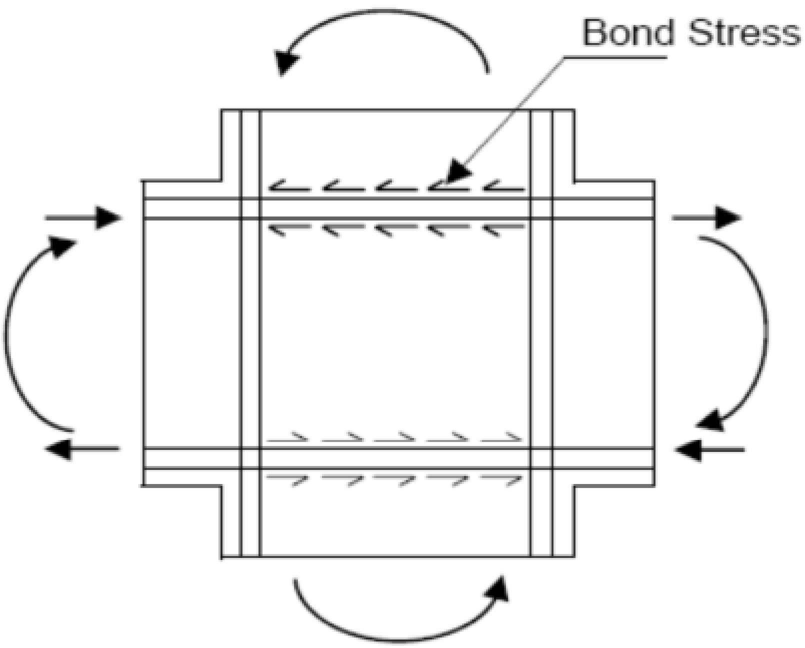


Figure 1.9 Bond stress in interior joint (Subramanian et al; 2003).

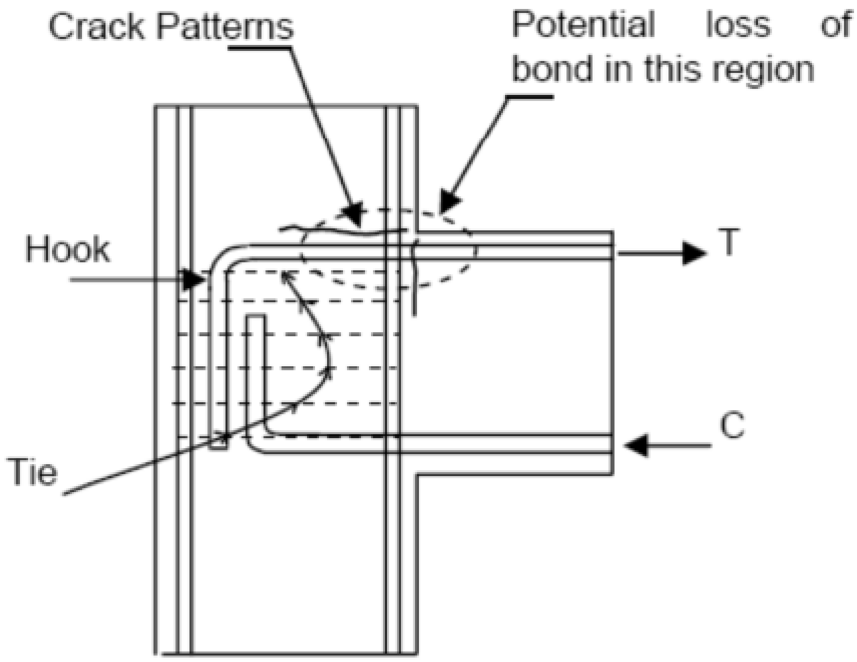


Figure.1.10 Hook in an Exterior Joint (Subramanian et al; 2003).

CHAPTER 2 LITERATURE REVIEW

2.1 INTRODUCTION

Reinforced concrete is one of the most abundantly used construction material not only in the developed world, but also in the remotest parts of the developing world. In the rural areas of the developing world, however, due to transference of expertise and technology know how, reinforced concrete poses threat due to its abuse rather than use, and majority of the houses are constructed in traditional manner using indigenously developed techniques preferably following simpler and economical procedures. Unfortunately such non-engineered construction is mostly prevalent in earthquake prone areas of the developing world e.g. Turkey, Pakistan, India and Iran. The rural populations in the developing world have mostly to rely on local skill, material and technology. The transformation of non-engineered construction into an engineered one therefore needs to be such that it could be sustained. The methodology should be simple in execution, offer better performance even when handled by less experienced workers, must involve materials, which are readily available, and yet durable, strong and economical. Ferrocement is one such material which have excellent ductility and become one of the main structural materials for retrofitting and strengthening of modest span reinforced concrete beams in such construction, specially, in earthquake prone areas.

Singh et al (1998);ferrocement over the years have gained respect in terms of its superior performance and versality, and now is being used not only used in housing industry but its potentials are being continuously explored for its use in retrofitting and strengthening of damaged structural members. 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 the houses damaged by earthquake and the effectiveness of its use has been reported by many researchers.

Andrew (1998);in an experimental study compared the flexural performance of reinforced concrete beams repaired with conventional method and 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 to be greater than the corresponding original beams and the beams repaired by conventional methods.

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. 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 trapped. Flexural mode of failure was observed surpassing shear capacity for only those specimens where full enhancement of the shear zone was carried.

Mays et al (1995); investigated the feasibility of using ferrcement as low permeability cover layer to reinforced concrete members located in environments where there is a high risk of reinforcement corrosion. This protective layer may be precast and could therefore act as permanent formwork to be poured in-situ concrete. It is found that the resistance to chloride penetration in accelerating ageing tests was enhanced by using SBR or acrylic bond coats. The use of permanent ferrocement formwork gave an increase in strength of 15% over conventional reinforced concrete.

Ganesan et al (1993); made experimental investigation on the strength and behavior of short confined concrete columns with and without ferrocement casing is presented. The columns were subjected to monotonic axial compression until failure. The primary variables considered in the study were the volumetric ratio of lateral reinforcement and volume fraction of mesh reinforcement (V_F). They made an attempt to compare the stress-strain characteristics of reinforced concrete columns with ferrocement as confinement (CFRC) and confined reinforced concrete (RC). This investigation revealed that the strength and strain at peak load of reinforced concrete could be enhanced by using ferrocement as confinement. Strength and ductility are two important factors to be considered in the design of seismic resistant reinforced concrete structures. Under seismic conditions the structures may be subjected to large deformations. In the case of reinforced concrete columns, the behavior depend largely on the amount of confinement provided to the core concrete because of the spalling of the concrete cover at compressive strains of about 0.004. Many attempts have been made in the past to improve the strength and strain at peak load in the case of reinforced concrete specimens subjected to uniaxial compression. The experimental work consisted of casting and testing of fourteen reinforced concrete columns of 150mm×150×750mm size. Out of these, ten were confined with ferrocement. Four different values of volumetric ratio of transverse reinforcements, namely 0.3%, 0.6%, 0.9%and 1.8% and three different values of volume fraction of mesh reinforcement (V_F) viz., 1.2%, 1.8%and 2.4% were used. In case of RC specimens vertical cracks appeared at about one- third of the maximum load. As the load increased, the number of cracks increased and width of crack widened. Further increase of load resulted in spalling of cover unevenly.

Nassif et al (2004); had performed an experimental study to examine a shear transfer between composite layers. They have concluded that in order to provide Full composites action between both the layers a minimum of five studs is needed. They also concluded that beams having shear studs with hooks exhibit better pre-cracking stiffness as well as cracking strength than those with l- shaped studs and also beams specimens with square mesh exhibited better cracking capacity than the control beams as well a beam with hexagonal mesh.

Kazemi et al (2005); had performed a study to evaluate a retrofit technique for strengthening shear deficient short column concrete columns. Ferrocement Jacket reinforced with expanded steel mesh is used for retrofitting in this study. They had concluded that expanded meshes were more effective ties in shear strengthening of concrete columns and also specimens strengthened with expanded meshes showed distribution fine shear cracking even at large amounts of displacement ductility capacity.

Abdullah et al (2000); had strengthened reinforced concrete columns with ferrocement jackets. They had used circular and square ferrocement jackets strengthening square reinforced concrete columns with inadequate shear resistance .They had concluded that by providing external confinement over entire length RC columns, the ductility is enhanced tremendously.

Anwar et al (1991); investigated the rehabilitation technique for reinforced concrete structural beam elements using ferrocement. The technique involved strengthening of the reinforced concrete beams by application of hexagonal chicken mesh and sketal steel combined by ordinary plastering. The basic parameters involved were the amount of wire mesh applied, its geometrical configuration and the degree of distress in the beams. From the best test obtained, a design chart was developed to determine the parameters for rehabilitation of the beam elements, the rehabilitation offers several advantages; it is easier to work with as it requires no specialized labor or equipment.

It does not require any formwork. By using ferrocement, with small quantities, considerable improvements can be achieved. The dead weight of the rehabilitation material is almost negligible and hence it does not require catering for additional dead weight as most of the other rehabilitation materials. In view of all these advantages, this method of using ferrocement is appropriate for rehabilitation of structural beam elements. Ngoiro; a wooden ferry in New Zealand has been preserved as a historical artifact and moored in an inner harbor location at which she recently sank. Subsequent examination of the hull showed severe damage by gribble worm. Replanking of the hull was estimated to cost US $ 30000 and US $ 40000 and would have had limited durability. The ferrocement retrofit was estimated to cost US$15000 to US$ 16000 and would in contrast to the other options, provide long term durability. After consideration of the alternatives of replanking or fiber glassing, a retrofitted skin of ferrocement was selected on the ground of durability and cost.

2.2 PROPERTIES OF FERROCEMENT

Al-Sulamani et al (1991);studied the behavior of ferrocement under direct shear by conducting compression tests on Z-shaped specimens reinforced with wire mesh producing pure shear on shear plane. Test results indicate that ferrocement under direct shear exhibits two stages of behavior (cracked and uncracked) 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 uncracked 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 behavior 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 behavior 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 300mm; 100mm thickness height of 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 behavior; uncracked, cracked and yield or ultimate stage. The third stage is an indication of the ductility that ferrocement possesses under flexure.

Mansur et al (1987); have studied the behavior in shear of ferrocement reinforced welded wire mesh by conducting flexural tests on simply supported rectangular beams under two symmetrical point loads. The major variables of the study were the shear to span ratio a/h, volume fraction of reinforcement V_f, strength of mortar f_c, and the amount of reinforcement near the compression face. Their test results indicate the diagonal cracking strength increases as a/h ratio is decreased and V_f, f_c and reinforcement near compression face are increased. Empirical equations are proposed to predict the diagonal cracking strength of ferrocement. Ferrocement beams are found to be susceptible to shear failure at small a/h ratios when v_f, f_c are relatively high. In general similar failure is preceded by the attainment of flexural capacity.

Paramasivam et al (1988); reported the effect of arrangement of reinforcements on mechanical properties of ferrocement. The presence of wire mesh reinforcement in ferrocement improves crack resistance, impact strength, and toughness. Evenly distributing layers of wire mesh across the cross section of ferrocement is a tedious and labor intensive operation. In practice, it might be easier if the layers of wire mesh were bundled. The bundled reinforcement can be placed near the top and bottom surfaces or at the mid-section of the element. The effects of the reinforcement arrangements on strength and deformational characteristics of ferrocement in direct tension and simple bending were studied experimentally. The conclusion is that a reinforcement arrangement in which the wire mesh is bundled and placed near the surfaces is preferred, from the point of view of first crack strength and crack characteristics.

The uniform distribution and high surface area to volume ratio (specific surface) of the reinforcement in ferrocement results in improved crack control, impact resistance, and toughness. Fig 2.1, shows the different arrangement of reinforcement; Fig 2.2 shows the load deflection curve.

Desayi et al (1992);studied the deflection and cracking of lightweight fiber reinforced ferrocement in bending proposing a bi-linear equation for predicting the deflection in the portion of load-deflection curve.

Xiong et al (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.

Kahn et al (1975);to study the composite behaviour of ferrocement, they tested forty composite beams made of 0.25 inch thick steel plates and 1 inch 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.

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:-

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.

Bing 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.

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.

Al-Salloum et al (2002); 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.

Ghobarah 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.

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.

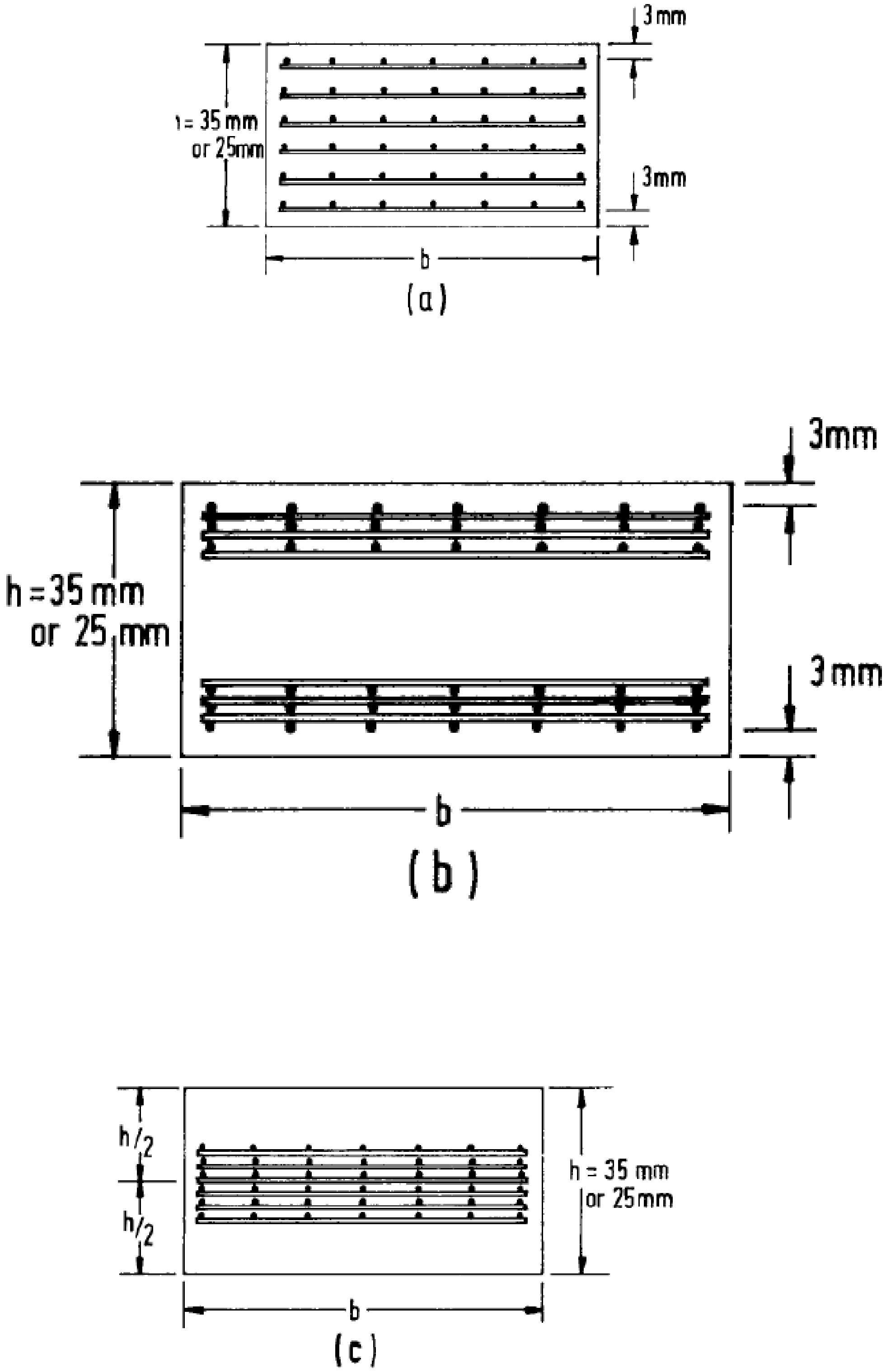


Fig 2.1:- Different arrangements of wire mesh in ferrocement (Paramasivam et al; 1988).

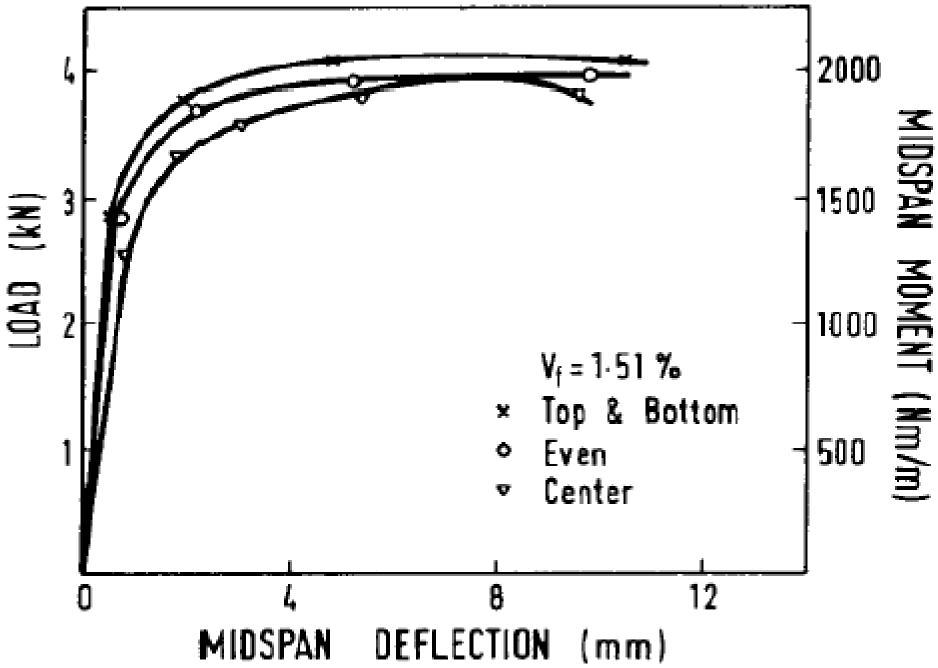


Fig 2.2:- Load deflection curve for different arrangement of reinforcement (Paramasivam et al; 1988).

CHAPTER 3 EXPERIMENTAL PROGRAM

3.1 TEST PROGRAM

The test program is so devised so as to study the behavior of retrofitted beam-column joints subjected to different ways of wrapping the retrofit material. The test program consists of:

1. First is the determination of basic properties of constituent materials namely cement, fine and coarse aggregates and steel bars as per relevant Indian standard specifications and designing the relevant concrete mix proportions.
2. Casting of five beam-column joints, with column rectangular shape of dimensions 225 mm x 150 mm and length of 1000 mm and the beam with dimensions 225mm x150 mm in all test specimens and length of 500 mm, using M 20 grade concrete.
3. One beam-column joint is considered as control beam. The remaining are stressed and retrofitted with ferrocement, in-order to find out the load carrying capacity. The stress levels maintained are 80% of the maximum load carrying found out by testing the control beam.

The details of the test program are discussed in subsequent sub-sections.

3.2 MATERIALS USED

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

3.2.1 Cement

Portland pozzolana cement of Ultra Tech make from a single lot is 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, valid for pozzolana cements.

3.2.2 Fine Aggregates

Locally available sand is used as fine aggregates both in the preparation of cement mortar as well as for the concrete mix. The physical properties and sieve analysis results of sand are shown in Tables 3.2 and 3.3.

3.2.3 Coarse Aggregates

Crushed stone aggregates (locally available) of 20 mm and 10 mm are used through-out the experimental study. The physical properties and sieve analysis results of coarse aggregates is given in Tables 3.4 to 3.6.

3.2.4 Water

Fresh and clean water is used for casting and curing the specimens. The water is relatively free from organic matter, silt, oil, sugar, chloride and acidic material as per requirements of Indian standard.

3.2.5 Reinforcing Steel

HYSD steel of grade Fe-500 of 10mm, 8mm diameters and mild steel of 6mm diameters are used in the experimental program. 10mm diameter bars are used as tension reinforcement and 8mm diameter bars are used as compression steel. 6mm diameter bars are used as stirrups. The properties of these bars are shown in Table 3.7.

3.2.6 Wire mesh

GI steel wire mesh of diameter 2.4 mm with rectangular grids pattern is used as a part of the ferrocement in jackets. The grid size of mesh used was 45mm×30mm.The properties of mesh wire are given in Table 3.7.

3.2.7 Concrete Mix

M20 grade concrete mix is designed as per IS code design procedure using the properties of materials as discussed above and presented in Tables 3.1 to 3.6. The water cement ratio achieved in the design is 0.48. The mix proportion of material came out to be 1: 1.46: 2.94 (cement: sand: coarse aggregate) and compressive strength of materials after 7 days and 28 days is 21.5 MPa & 29 MPa respectively.

3.2.8 Mortar Mix

The range of mix proportion recommended for common ferrocement applications are cement: sand ratio by weight of 1:1.15 to 1: 1.25, but not greater than 1:3 and water cement ratio by weight of 0.35 to 0.5. The higher the sand content the higher is the required water content to maintain same workability. Fineness modulus of the sand, water cement ratio and sand-cement ratio are determined from trial batches to ensure a mix that can infiltrate the mesh and develop a strong and denser mortar. The proportion of cement-sand mortar used for ferrocement jackets is 1:3.The water-cement ratio for mortar is 0.45.

3.3 DESIGN OF BEAM-COULMN JOINT

To study the proposed behaviour, five external beam column joint specimens are cast using M-20 grade concrete and Fe-500 grade steel. The column is rectangular in shape with dimensions 225 mm x 150 mm and a length of 1000 mm. The beam has dimensions 225 mm x 150 mm in all test specimens and length of 500 mm. In all five joints 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 10 mm diameter bars in tension zone and 2 no's of 8 mm diameter in the compression zone and, from the face of beam, an anchorage length of 600 mm to both sides of column is provided. The RCC beam-column joint is designed using limit state method considering it to be an under-reinforced section. The ties for both the specimens consist of rectangular hoops of 6 mm diameter of size 185 mm x 110mm placed 100 mm c/c in the column portion as well as in the beam portion. The reinforcement detailing is shown in Figure 3.1.

The specimen is fixed on loading frame using the arrangement shown in the Figure 3.2.The joints are subjected to a point load at a distance of 300 mm from the face of column. The value of deflection has been taken with the help of three LVDT's, one LVDT is set at the free end of a beam, second at a distance of 150mm from free end and third at 100 mm from column face to note the deflection in the beam. The casted specimens are tested using the hydraulic jack. The Figures 3.3 and 3.4 show the details of hydraulic jack and data acquisition system used for the test.

3.3 DATA ACQUISITION SYSTEM

Working principle for this system: The load, deformation and displacement measuring system signals through the A /D transformation, after the real-time computer acquisition are stored in the computer. After the test, in accordance with testing standard, the computer will automatically process the testing data, and obtain the test results, then store the results for long-term test data-base establishment. Meanwhile testing data is open to the user, so that users can realize their own data processing according to different requirements. The system also has the test data re-processing and re-analysis function. The testing process can be re-asserted on the computer, and gain further data processing and analysis to confirm the reliability of previous results. The testing curve can be enlarged so that more careful observation and test results analysis and processing are available. Also the batch test results can be analyzed and processed, as well as overlapping comparison test of the batch of test curves.

3.4 CASTING OF COMPOSITE BEAM-COLUMN JOINTS

The casting of the joints is done in the single stage. A steel mould is made of dimensions 225mm x 150 mm for the beam portion and of length 500mm and 225 x 150 mm for the column portion with length 1000mm. The steel mould is shown in the Figure 3.5. Cover blocks of 20 mm are placed under the reinforcement to provide uniform cover. Coarse aggregates, fine aggregates, cement and water are mixed manually as per the proportions of design mix.

After placing the desired reinforcement, concrete is poured in the mould and vibrations are 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 are then removed from the mould after 24 hours. After de-moulding the beams are cured for 28 days in water.

3.5 TESTINGOF BEAM COLUMN JOINTS

For testing the joints under a hydraulic jack a triangular frame is fabricated. 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 Fig. 3.2. One specimen is loaded to ultimate loading (i.e., 100% damage) and the remaining are loaded upto 80% of ultimate load. After retrofitting the ultimate strength of the remaining four specimens are checked.

3.6 PROCESSOF RETROFITTING

The four beam column joints which are loaded upto 80% of the ultimate load are retrofitted using two different schemes. The retrofitting schemes are discussed below. The retrofitting scheme consists of wrapping the beam portion and column portion with the help of the rectangular wire mesh. Firstly, the surfaces of specimens are cleaned. After the wrapping of specimen with wire mesh is done, the cement slurry is applied as bonding agent to the surfaces of beam-column joints. The cement mortar of 20mm thick made of ratio 1:3 and having water cement ratio (w/c) equal to 0.45 is applied on the specimen. 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 beam to calculate ultimate load and corresponding deflections.

3.6.1 RETROFITTING SCHEMES

The two types of retrofitting schemes used for wrapping of wire mesh are categorized as:-

1) Type one retrofitting, and

2) Type two retrofitting.

1) Type one retrofitting: - In this retrofitting we make two L-shapes of appropriate size from the wire mesh and wrap these on the lower and upper faces of the beam at the joint.

Then we use cement mortar of thickness 20mm on the wire mesh bonded on the beam-column joint as shown in Fig.3.6.

2) Type two retrofitting: - In this retrofitting we make again two L-shapes of appropriate size from the wire mesh and wrap these on the lower and upper faces of the beam at the joint but in this type we use some extra mesh of appropriate size diagonal to the joint. Then we use cement mortar of thickness 20mm on the wire mesh bonded on the beam-column joint as shown in the Fig.3.7.

Table 3.1 Physical Properties of Portland Pozzolana Cement

Sr. No.	Characteristics	Test Values	Value specified by IS :1489-1991 (Part 1)
1.	Standard Consistency	32	---
2.	Fineness of cement as retained on 90 micron sieve (%)	0.7%	Maximum 10%
3.	Setting time (mints) 1.Initial 2.Final	 105 255	 Minimum 30 Maximum 600
4.	Specific gravity (Specific gravity bottle)	3.10	-
5.	Compressive Strength(N/mm^2) 7 days 28 days	 23.50 35.60	 Minimum 22.0 Minimum 33.0

Table 3.2 Physical Properties of Fine Aggregates

Sr. No.	Characteristics	Value
1.	Specific gravity (oven dry basis)	2.69
3.	Fineness modulus	2.30
4.	Water absorption	2.39
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 (in grams)	Percentage Retained (%)	Cumulative Percentage Retained (%)	Percent Passing (%)
1	10mm	Nil	Nil	Nil	100
2	4.75mm	27.0	2.7	2.7	97.30
3	2.36 mm	50.5	5.05	7.75	92.25
4	1.18 mm	120.0	12.0	19.75	80.25
5	600µm	186.5	18.65	38.40	61.60
6	300µm	287.0	28.70	67.10	32.9
6	150µm	280.0	28.0	95.10	4.9
7	Pan	49.0	4.9	SUM=230.80	

Total weight taken = 1000 gm.

Fineness Modulus of sand = SUM/100= 2.30

Table 3.4 Sieve Analysis of 10mm Coarse Aggregate

Weight of sample taken = 3 Kg.

S. No.	Sieve No.	Mass retained (g)	Percentage retained (%)	Cumulative percentage retained,(C)	Percentage Passing, (100- C)
1	80 mm	0	0	0	100
2	40 mm	0	0	0	100
3	20 mm	0	0	0	100
4	10 mm	1225	40.83	40.83	59.17
5	4.75 mm	1624	54.13	94.96	5.04
6	Pan	151	5.04	SUM=135.79	-

Fineness Modulus= (SUM +500)/100 =635.79/100= 6.358= 6.36

Table 3.5 Sieve Analysis of 20mm Coarse Aggregate

Weight of sample taken= 3 Kg.

S. No.	Sieve No.	Mass retained (g)	Percentage mass retained	Cumulative percentage retained, C	Percentage Passing, (100 – C)
1	80 mm	0	0	0	100
2	40 mm	0	0	0	100
3	20 mm	0	0	0	100
4	10 mm	2648	88.26	88.26	11.74
5	4.75 mm	324	10.80	99.06	0.94
6	Pan	28	0.94	SUM=187.32	

Fineness Modulus= (187.32+500)/100= 6.87

Table 3.6 Physical Properties of Coarse Aggregates

S.No.	**Characteristics**	**Value**	
		CA-I	CA-II
1.	Type	Crushed	Crushed
2.	Maximum Nominal Size (mm)	20mm	10mm
3.	Specific gravity	2.60	2.63
4.	Total water absorption (%)	1.95	1.84
5.	Fineness modulus	6.87	6.36

Table 3.7: 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	10mm	445.55	509.2	15.5
2	8mm	559.5	634.13	20.3
3	6mm	442.42	612.7	32.9
4	2.4mm wire mesh	400	511.36	2.52

Table 3.8 Concrete Mix Design for M-20 Grade Concrete (As per I.S.)

Cement	395.62 kg/m^3
F.A.	571.63 kg/m^3
C.A. (CA-I : CA-II)	1163.70 kg/m^3, (50:50)
Water	189.90 kg/m^3
Ratio	1:1.46:2.94
W/C Ratio	0.48

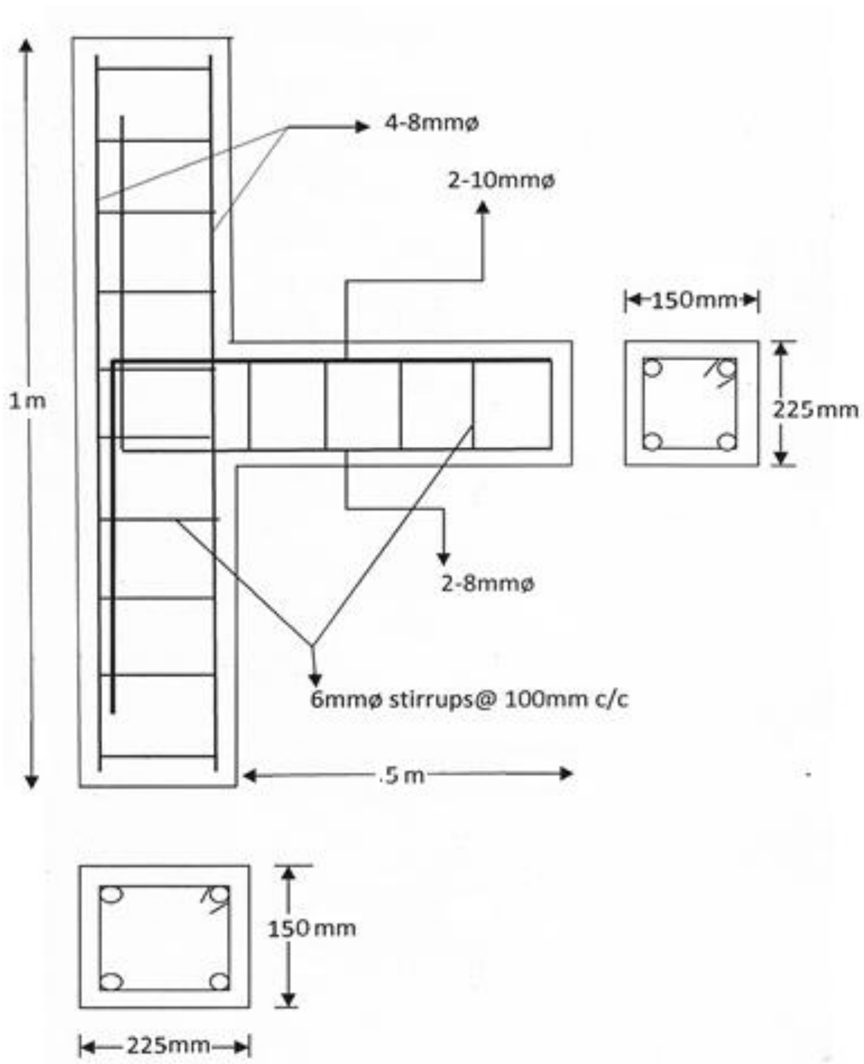


Figure 3.1 Reinforcement Detailing Of Beam Column Joint.

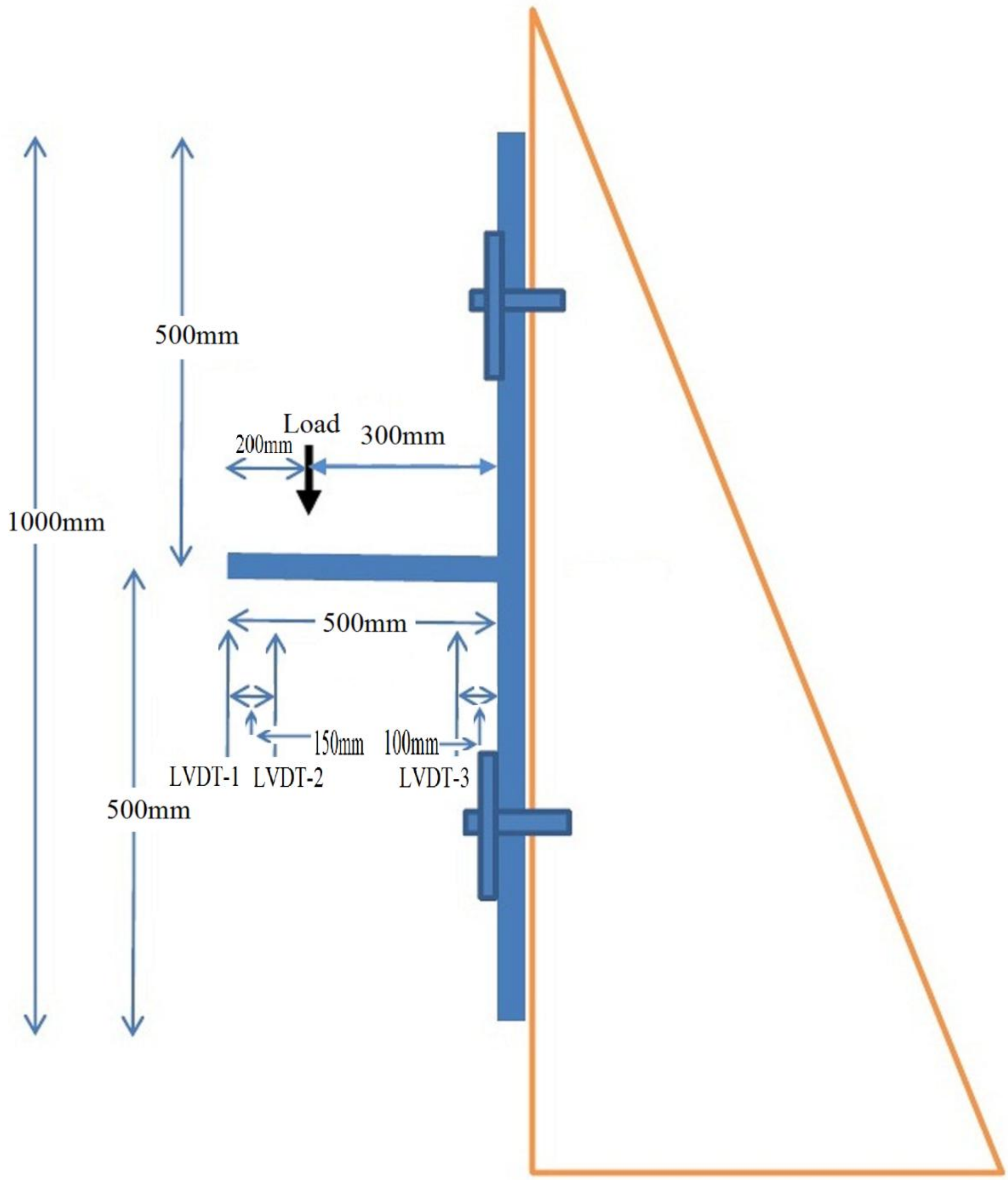


Figure 3.2: Beam Column Specimen Attached with Frame

Figure 3.3 Testing Jack

Figure 3.4 Data Acquisition System Connecting to Jack

Fig 3.5 Steel Mould for Concrete Casting.

Figure 3.6 Specimen Retrofitted with Mesh Wire (Type One Retrofitting).

Figure 3.7 Specimen Retrofitted with Mesh Wire (Type Two Retrofitting).

CHAPTER 4 RESULTS AND DISCUSSIONS

4.1 INTRODUCTION

In this chapter the load carrying capacity of different specimens are discussed. Initially the control specimen is loaded to ultimate load and other four beam-column joints are loaded up to 80% of the ultimate load obtained from testing of control specimen. Out of four, two beam-column joints are retrofitted with type one retrofitting and the other two are retrofitted with type two retrofitting with the help of mesh wire as shown in Figures 3.6 and 3.7in the previous chapter.

4.2 TESTING METHODOLOGY

The testing of beam column joints are done with the help of a servo controlled hydraulically operated jack. With the help of the jack, point load is applied on the beam at a distance of 300mm from the face of the column and the value of load is read from the data acquisition system connected to the jack. Three LVDT's are placed at different locations as shown in Fig. 3.2 in the previous chapter for measuring the deflection at specific locations.

Out of the five specimens cast, one specimen is taken as control specimen and is loaded to ultimate loading and the data corresponding to it is recorded through data acquisition system. The rest four specimens are loaded to 80% of the ultimate load and then are retrofitted using different wrapping techniques.

The ultimate load of the control beam-column joint comes out to be 64.1kN, with a maximum deflection equal to 24.1 mm at free end of the beam. The rest four beam-column joints are loaded to 80% of ultimate load of control specimen i.e.; 51.28 KN. Then the retrofitting of the beam column joints are done with cement mortar of thickness 20mm along with wire mesh bonded on the four beam-column joints. Out of four, two beam-column joints are retrofitted with type one retrofitting technique and the other two are retrofitted with type two retrofitting technique as per the variation in mesh wire wrapping as shown in figures 3.6 and 3.7in the previous chapter. After one week of curing the beam column joints are tested again with the same method as the control beam-column joint was tested initially and the corresponding results are recorded in the form of load and deflection. Then the graphs of these retrofitted specimens are compared with controlled specimen. The graphs of type one and type two retrofitted specimens are also compared with each other.

The beam-column joints designations provided are as under:-

1. Control Specimen - Control beam column joint (CS)
2. Retrofitted Beam column joint 1 - R1 (80% loaded-type one retrofitting)
3. Retrofitted Beam column joint 2 - R2 (80% loaded-type one retrofitting)
4. Retrofitted Beam column joint 3 - R3 (80% loaded-type two retrofitting)
5. Retrofitted Beam column joint 4 - R4 (80% loaded-type two retrofitting)

The subsequent section presents the results of the controlled specimen and the retrofitted specimens.

4.3 CONTROL BEAM-COLUMN JOINT

One beam-column is tested as a control beam. The load is applied and deflection is noted at the three locations with the LVDT's. The results of load and corresponding deflection so obtained are presented in Table 4.1. In the beginning the deflection upto 10 KN is very less and after this it increases almost linearly with the increase in load but after load of 55 KN it increases at much higher rate till the ultimate load of 64.1 KN, as shown in Fig. 4.1. The first crack in the control specimen is observed at a load of 27.56 KN, thereafter number of cracks increased and spread over the entire area of the beam-column joint as shown in Plate 4.1.

4.4 EFFECT OF METHOD OF WRAPPING TECHNIQUE

4.4.1 Effect on Ultimate Load

The effect on strength of retrofitted RCC beam-column joint R1loaded to 80 % level is shown in Fig. 4.1. The Table 4.1 show the load deflection data for control specimen& 80 % loaded retrofitted specimen. Plates 4.2 & 4.3 shows the crack pattern for the retrofitted beam-column joint.

It is observed from the experimental data and the corresponding graph that retrofitting leads to a significant increase in the ultimate load carrying capacity from 64.1KN (control specimen) to 81.45KN whereas the deflection corresponding to ultimate load of 81.45 KN is 16.23 mm as compared to 24.1 mm for the control specimen at 64.1KN. Also there is a considerable increase in the yield load from 55KN (control specimen) to 75 KN for the retrofitted specimen. For the R2 specimen exactly similar trend is observed and increase in load is also of almost of the same order i.e.; from 64.1 KN (control specimen) to 81.52 KN with deflection of about 16.61mm. The yield load also increases from 55 KN (control specimen) to 75 KN. Thus, on an average for type one retrofitting with80% stress level beam-column joints, on retrofitting the ultimate load increase is of the order of 27.12 % and yield load by 36.36 %.

The effect on strength of retrofitted RCC beam-column joint R3loaded to 80 % level is shown in Figs 4.1. The Table 4.4shows the load deflection data for controlledspecimen&80 % loaded retrofitted specimen. Plates 4.4 & 4.5 show the crack pattern for the retrofitted specimen.

It is observed from the experimental data and the corresponding graph that retrofitting leads to increase in the ultimate load carrying capacity from 64.1 KN (control specimen) to 102.21 KN whereas the deflection corresponding to ultimate load of 102.21 KN is 20.31 mm as compared to 24.1 mm for the control specimen at 64.1 KN. Also there is a considerable increase in the yield load from55 KN (control specimen) to 95 KN for the retrofitted specimen. For the R4 specimen exactly similar trend is observed and increase in load is also of almost of the same order i.e. from 64.1 KN (control specimen) to 102.35 KN with deflection of about 20.35mm. The yield load increases from 55 KN (control specimen) to 95 KN. Thus, on an average for type two retrofitting with 80 % stress level beam-column joints, on retrofitting the ultimate load increase is of the order of 59.56% and yield load increases by 72.73 %

From a comparative point of view it is observed from Fig 4.1 that the beam-column joints of different wrappings show different behaviour, specimens with type two retrofitting scheme show maximum improvement in their ultimate load from 64.1 KN (control specimen) to 102.28 KN without much increase in the deflection if we are going to consider the performance as such. The ultimate load of type two retrofitting scheme is more than type one retrofitting scheme, thus on an average for type two retrofitted beam-column joints, on retrofitting, the ultimate load is of the order of 25.52% and the yield load increases by 26.67%.

From a comparative point of view it is observed from Fig 4.7 and Table 4.8 that percentage increase in the ultimate loads of the retrofitted beams has been able to justify the thesis work till date because the results are in lieu to the economy considerations, all the beams have been able to perform very efficiently increasing the ultimate loads to a percentage as high as 27.12%, 59.56% for type one retrofitted-beam column joints and type two retrofitted beam-column joints for 80% stress level respectively as compared with controlled beam-column joint.

4.4.2 Effect on Ductility

The values of ductility ratio are shown in Table 4.7. The ductility ratio of the controlled specimen is 3.35 and the ductility ratio of type one retrofitted specimen R1 is 1.24. So the ductility ratio of type one retrofitted specimen is less than controlled specimen CS. The ductility ratio of type one retrofitted specimen R2 is 1.27 which is also less than the value of controlled specimen CS. The average value of type one retrofitted specimen is 1.26, which is less than the ductility ratio of controlled specimen.

The ductility ratio of type two retrofitted specimen R3 is 1.52, which is less than the ductility ratio of controlled specimen which is equal to 3.35. The ductility ratio of type two retrofitted specimen R3 is also less than the value of controlled specimen.

The average ductility ratio of type two retrofitted specimen is 1.52, which is also less than the ductility ratio of control specimen which is equal to 3.35.

On comparing the average values of ductility ratio of type one retrofitting with type two retrofitting, the ductility ratio of type one retrofitting is less than type two retrofitting.

4.4.3 Effect on Energy absorption

The values of energy absorption are presented in Table 4.7 and the Figs. 4.4 to 4.6 shows the trilinear curve for control specimen as well as for retrofitted specimen. The value of energy absorption incase of type one retrofitted specimen R1 decreases by nearly 8.176 % than the control specimen. In case of type one retrofitted specimen R2, the value of energy absorption decreases by nearly 4.82 % than the control specimen. On an average the value of energy absorption, incase of type one retrofitting technique decreases by nearly 6.498% than the control specimen.

The value of energy absorption incase of type two retrofitted specimen R3 increases by nearly 46.617 % than the control specimen. And in case of type two retrofitted specimen R4, the value of energy absorption increases by nearly 47.516% than the control specimen. On an average the value of energy absorption incase of type two retrofitted specimen increases by nearly 47.07% than the control specimen.

On comparing the average values of energy absorption of type one retroffiting specimen with type two retrofitting specimen, the energy absorption of type one retrofitting decreases by 56.584% than the type two retrofitting specimen.

4.4.4 Effect on Moment and Rotation

The average value of ultimate moment of the retrofitted specimen is more than the ultimate moment of controlled specimen as shown in Fig. 4.8. The average value of ultimate moment of the type one retrofitting scheme is 24.45 KN-m with corresponding rotation equal to 0.071 and the ultimate moment of the control specimen is 19.23KN-m with corresponding rotation equal to 0.10, the rotation of type one retrofitting scheme is less than the controlled specimen. The data are given in the Table 4.9. The average value of ultimate moment of type one retrofitting is 27.145 % more than the ultimate moment of controlled specimen.

The average ultimate moment of type two retrofitting is significantly more than the ultimate moment of control specimen. The average ultimate moment of type two retrofitting is 30.684 KN-m with corresponding average rotation equal to 0.066, while the ultimate moment of control specimen is 19.23 KN-m with corresponding rotation equal to 0.10, the rotation of type two retrofitting is less than control specimen, as shown in Fig. 4.8. The data are given in the Table 4.10. The maximum moment of type two retrofitting is 59.56% more than controlled specimen.

The maximum moment of type two retrofitting is 25.53% more than the type one retrofitting, as shown in the Fig. 4.8.

Table 4.1 Load and Deflection Values at Free End of Beam of Controlled and Retrofitted Specimens R1 and R2.

S. No.	Control beam column joint (CS)		Retrofitted beam-column joint (R1)-type one retrofitting		Retrofitted beam-column joint (R2)-type one retrofitting		Avg.=(R1+R2)/2	
	Load (KN)	Deflection (mm)	Load (KN)	Deflection (mm)	Load (KN)	Deflection (mm)	Load (KN)	Deflection (mm)
1	0	0	0	**0**	0	0	0	0
2	5	0.01	5	0.43	5	0.55	5	0.49
3	10	0.09	10	1.34	10	1.33	10	1.335
4	15	0.41	15	2.2	15	2.19	15	2.195
5	20	1.12	20	2.92	20	2.92	20	2.92
6	25	1.7	25	3.55	25	3.52	25	3.535
7	30	2.44	30	4.29	30	4.31	30	4.3
8	35	3.49	35	5.04	35	5.05	35	5.045
9	40	4.32	40	5.81	40	5.8	40	5.805
10	45	5.36	45	6.59	45	6.6	45	6.595
11	50	6.28	50	7.38	50	7.37	50	7.375
12	55	7.2	55	8.21	55	8.2	55	8.205
13	60	9.6	60	9.2	60	9.19	60	9.195
14	64.1	24.1	65	10.36	65	10.39	65	10.375
15	60	33.59	70	11.72	70	11.72	70	11.72
16	59.5	35.23	75	13.08	75	13.1	75	13.09
17			80	15.07	80	15.11	80	15.09
18			81.45	16.23	81.52	16.61	81.485	16.42
19			80	17.29	80	17.33	80	17.31
20			75	22.5	75	23.1	75	22.8
21			70	29.59	70	30.32	70	29.955

Table 4.2 Load and Deflection Values at 150mm From Free End of Beam of Controlled and Retrofitted Specimens R1 and R2.

S. No.	Control beam column joint (CS)		Retrofitted beam-column joint (R1)-type one retrofitting		Retrofitted beam-column joint (R2)-type one retrofitting		Avg.=(R1+R2)/2	
	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)
1	0	0	0	0	0	0	0	0
2	5	0	5	0.47	5	0.45	5	0.46
3	10	0.05	10	1.18	10	1.18	10	1.18
4	15	0.38	15	1.82	15	1.78	15	1.8
5	20	1.08	20	2.25	20	2.24	20	2.245
6	25	1.6	25	2.53	25	2.6	25	2.565
7	30	2.35	30	3.1	30	3.03	30	3.065
8	35	3.44	35	3.69	35	3.7	35	3.695
9	40	4.28	40	4.33	40	4.35	40	4.34
10	45	5.32	45	4.98	45	5	45	4.99
11	50	6.23	50	5.64	50	5.65	50	5.645
12	55	7.16	55	6.32	55	6.35	55	6.335
13	60	9.3	60	7.13	60	7.15	60	7.14
14	64.1	23.9	65	8.1	65	8.3	65	8.2
15	60	33.5	70	9.23	70	9.25	70	9.24
16	59.5	35.1	75	10.28	75	10.25	75	10.265
17			80	11.84	80	11.86	80	11.85
18			81.45	12.67	81.52	12.89	81.485	12.78
19			80	13.25	80	13.28	80	13.265
20			75	17.29	75	17.3	75	17.295
21			70	22.22	70	23.13	70	22.675

Table 4.3 Load and Deflection Values at 100mm From Column Face of Controlled and Retrofitted Specimens R1 and R2.

S. No.	Control beam column joint (CS)		Retrofitted beam-column joint (R1)-type one retrofitting		Retrofitted beam-column joint (R2)-type one retrofitting		Avg.=(R1+R2)/2	
	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)
1	0	0	0	0	0	0	0	0
2	5	0	5	0.3	5	0.32	5	0.31
3	10	0.03	10	0.73	10	0.71	10	0.72
4	15	0.27	15	0.84	15	0.83	15	0.835
5	20	0.49	20	1	20	1.02	20	1.01
6	25	0.67	25	1.09	25	1.12	25	1.105
7	30	0.91	30	1.43	30	1.43	30	1.43
8	35	1.23	35	1.83	35	1.82	35	1.825
9	40	1.53	40	2.25	40	2.24	40	2.245
10	45	1.83	45	2.66	45	2.67	45	2.665
11	50	2.16	50	3.07	50	3.07	50	3.07
12	55	2.56	55	3.5	55	3.51	55	3.505
13	60	4.6	60	4	60	3.99	60	3.995
14	64.1	10.09	65	4.61	65	4.6	65	4.605
15	60	19.58	70	5.29	70	5.29	70	5.29
16	59.5	22.22	75	5.9	75	5.93	75	5.915
17			80	6.75	80	6.73	80	6.74
18			81.45	7.07	81.52	7.1	81.485	7.085
19			80	7.39	80	7.45	80	7.42
20			75	8.92	75	8.95	75	8.935
21			70	10.77	70	10.81	70	10.79

Table 4.4 Load and Deflection Values at Free End of Beam of Controlled and Retrofitted Specimens R3 and R4.

S. No.	Control beam column joint (CS)		Retrofitted beam-column joint (R3)-type two retrofitting		Retrofitted beam-column joint (R4)-type two retrofitting		Avg.=(R3+R4)/2	
	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)
1	0	0	0	0	0	0	0	0
2	5	0.01	5	0.23	5	0.25	5	0.24
3	10	0.09	10	0.5	10	0.51	10	0.505
4	15	0.41	15	0.85	15	0.87	15	0.86
5	20	1.12	20	1.21	20	1.22	20	1.215
6	25	1.7	25	1.67	25	1.67	25	1.67
7	30	2.44	30	2.12	30	2.13	30	2.125
8	35	3.49	35	2.55	35	2.57	35	2.56
9	40	4.32	40	3.15	40	3.14	40	3.145
10	45	5.36	45	3.71	45	3.72	45	3.715
11	50	6.28	50	4.5	50	4.48	50	4.49
12	55	7.2	55	5.22	55	5.22	55	5.22
13	60	9.6	60	6.01	60	5.99	60	6
14	64.1	24.1	65	6.81	65	6.8	65	6.805
15	60	33.59	70	7.67	70	7.66	70	7.665
16	59.5	35.23	75	8.55	75	8.56	75	8.555
17			80	9.46	80	9.45	80	9.455
18			85	10.6	85	10.61	85	10.605
19			90	11.91	90	11.9	90	11.905
20			95	13.4	95	13.42	95	13.41
21			100	18.05	100	18.04	100	18.045
22			102.21	20.31	102.35	20.35	102.28	20.33
23			100	30	100	29.94	100	29.97
24			95	35.08	96.5	35.13	95.75	35.105

Table 4.5 Load and Deflection Values at 150mm From Free End of Beam of Controlled and Retrofitted Specimens R3 and R4.

S. No.	Control beam column joint (CS)		Retrofitted beam-column joint (R3)-type two retrofitting		Retrofitted beam-column joint (R4)-type two retrofitting		Avg.=(R3+R4)/2	
	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion(mm)
1	0	0	0	0	0	0	0	0
2	5	0	5	0.19	5	0.2	5	0.195
3	10	0.05	10	0.45	10	0.46	10	0.455
4	15	0.38	15	0.71	15	0.74	15	0.725
5	20	1.08	20	1.11	20	1.12	20	1.115
6	25	1.6	25	1.4	25	1.41	25	1.405
7	30	2.35	30	1.77	30	1.77	30	1.77
8	35	3.44	35	2.16	35	2.17	35	2.165
9	40	4.28	40	2.6	40	2.62	40	2.61
10	45	5.32	45	3.07	45	3.08	45	3.075
11	50	6.23	50	3.69	50	3.71	50	3.7
12	55	7.16	55	4.3	55	4.3	55	4.3
13	60	9.3	60	4.87	60	4.9	60	4.885
14	64.1	23.9	65	5.52	65	5.53	65	5.525
15	60	33.5	70	6.17	70	6.2	70	6.185
16	59.5	35.1	75	6.91	75	6.92	75	6.915
17			80	7.61	80	7.63	80	7.62
18			85	8.57	85	8.58	85	8.575
19			90	9.6	90	9.59	90	9.595
20			95	10.81	95	10.81	95	10.81
21			100	14.24	100	14.92	100	14.58
22			102.21	17.27	102.35	17.32	102.28	17.295
23			100	20.84	100	20.86	100	20.85
24			95	23.56	96.5	23.96	95.75	23.76

Table 4.6 Load and Deflection Values at 100mm From Column Face of Controlled and Retrofitted Specimens R3 and R4.

S. No.	Control beam column joint (CS)		Retrofitted beam-column joint (R3)-type two retrofitting		Retrofitted beam-column joint (R4)-type two retrofitting		Avg.=(R3+R4)/2	
	Load (KN)	Deflection(mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)	Load (KN)	Deflec-tion (mm)
1	0	0	0	0	0	0	0	0
2	5	0	5	0	5	0	5	0
3	10	0.03	10	0	10	0	10	0
4	15	0.27	15	0	15	0	15	0
5	20	0.49	20	0	20	0	20	0
6	25	0.67	25	0	25	0	25	0
7	30	0.91	30	0	30	0	30	0
8	35	1.23	35	0.01	35	0.01	35	0.01
9	40	1.53	40	0.25	40	0.26	40	0.255
10	45	1.83	45	0.49	45	0.51	45	0.5
11	50	2.16	50	0.88	50	0.89	50	0.885
12	55	2.56	55	1.18	55	1.2	55	1.19
13	60	4.6	60	1.55	60	1.57	60	1.56
14	64.1	10.09	65	1.88	65	1.9	65	1.89
15	60	19.58	70	2.31	70	2.31	70	2.31
16	59.5	22.22	75	2.61	75	2.63	75	2.62
17			80	2.99	80	3.01	80	3
18			85	3.47	85	3.47	85	3.47
19			90	3.9	90	3.93	90	3.915
20			95	4.55	95	4.58	95	4.565
21			100	5.94	100	5.95	100	5.945
22			102.21	6.62	102.35	6.65	102.28	6.635
23			100	8.27	100	8.28	100	8.275
24			95	10.09	96.5	10.14	95.75	10.115

Table 4.7 Ductility Ratio and Energy Absorption at Free End of Beam of Controlled and Retrofitted Specimens R1, R2, R3 and R4.

S.No.	Beam-Column Joint Designation	Deflection at Yield load (mm)	Deflection at Ultimate load (mm)	Ductility Ratio	Energy Absorption* (KN-mm)
1	CS	7.2	24.1	3.35	1892.23
2	R1	13.08	16.23	1.24	1748.60
3	R2	13.1	16.61	1.27	1804.61
4	R3	13.4	20.31	1.52	2774.26
5	R4	13.42	20.35	1.52	2790.77

* Area under the load deflection curve.

Table 4.8 Comparison of Experimental Ultimate Loads at a Free End of Beam of Controlled and Retrofitted Specimens R1, R2, R3 and R4.

Specimen Designation	Exp. Ult. Load (KN)	Gain in Ult. Load on strengthening (KN)	
		Value	% age increase
CS	64.1	-	-
R1	81.45	17.35	27.07
R2	81.52	17.42	27.18
R3	102.21	38.11	59.45
R4	102.35	38.25	59.67

Table 4.9 Moment and Rotation at Column Face of Controlled and Retrofitted Specimens for the Average Values of R1 and R2.

S.No.	Control beam column joint (CS)				Avg.=(R1+R2)/2			
	Load (P) in KN	Deflec-tionin mm	B.M. (KN-m)	Rotation (Ɵ)	Load (P)in KN	Deflec-tionin mm	B.M (KN-m)	Rotation (Ɵ)
1	0	0	0	0	0	0	0	0
2	5	0	1.5	0	5	0.31	1.5	0.0031
3	10	0.03	3	0.0003	10	0.72	3	0.0072
4	15	0.27	4.5	0.0027	15	0.835	4.5	0.00835
5	20	0.49	6	0.0049	20	1.01	6	0.0101
6	25	0.67	7.5	0.0067	25	1.105	7.5	0.01105
7	30	0.91	9	0.0091	30	1.43	9	0.0143
8	35	1.23	10.5	0.0123	35	1.825	10.5	0.01825
9	40	1.53	12	0.0153	40	2.245	12	0.02245
10	45	1.83	13.5	0.0183	45	2.665	13.5	0.02665
11	50	2.16	15	0.0216	50	3.07	15	0.0307
12	55	2.56	16.5	0.0256	55	3.505	16.5	0.03505
13	60	4.6	18	0.046	60	3.995	18	0.03995
14	64.1	10.09	19.23	0.1009	65	4.605	19.5	0.04605
15	60	19.58	18	0.1958	70	5.29	21	0.0529
16	59.5	22.22	17.85	0.2222	75	5.915	22.5	0.05915
17					80	6.74	24	0.0674
18					81.485	7.085	24.4455	0.07085
19					80	7.42	24	0.0742
20					75	8.935	22.5	0.08935
21					70	10.79	21	0.1079

Table 4.10 Moment and Rotation at Column Face of Controlled and Retrofitted Specimens For the Average Values of R3 and R4.

S.No.	Control beam column joint (CS)				Avg.=(R3+R4)/2			
	Load (P) in KN	Deflec-tion in mm	B.M.	Rota-tion(Θ)	Load (P)in KN	Deflec-tion in mm	B.M	Rotation (Θ)
1	0	0	0	0	0	0	0	0
2	5	0	1.5	0	5	0	1.5	0
3	10	0.03	3	0.0003	10	0	3	0
4	15	0.27	4.5	0.0027	15	0	4.5	0
5	20	0.49	6	0.0049	20	0	6	0
6	25	0.67	7.5	0.0067	25	0	7.5	0
7	30	0.91	9	0.0091	30	0	9	0
8	35	1.23	10.5	0.0123	35	0.01	10.5	0.0001
9	40	1.53	12	0.0153	40	0.255	12	0.00255
10	45	1.83	13.5	0.0183	45	0.5	13.5	0.005
11	50	2.16	15	0.0216	50	0.885	15	0.00885
12	55	2.56	16.5	0.0256	55	1.19	16.5	0.0119
13	60	4.6	18	0.046	60	1.56	18	0.0156
14	64.1	10.09	19.23	0.1009	65	1.89	19.5	0.0189
15	60	19.58	18	0.1958	70	2.31	21	0.0231
16	59.5	22.22	17.85	0.2222	75	2.62	22.5	0.0262
17					80	3	24	0.03
18					85	3.47	25.5	0.0347
19					90	3.915	27	0.03915
20					95	4.565	28.5	0.04565
21					100	5.945	30	0.05945
22					102.28	6.635	30.684	0.06635
23					100	8.275	30	0.08275
24					95.75	10.115	28.725	0.10115

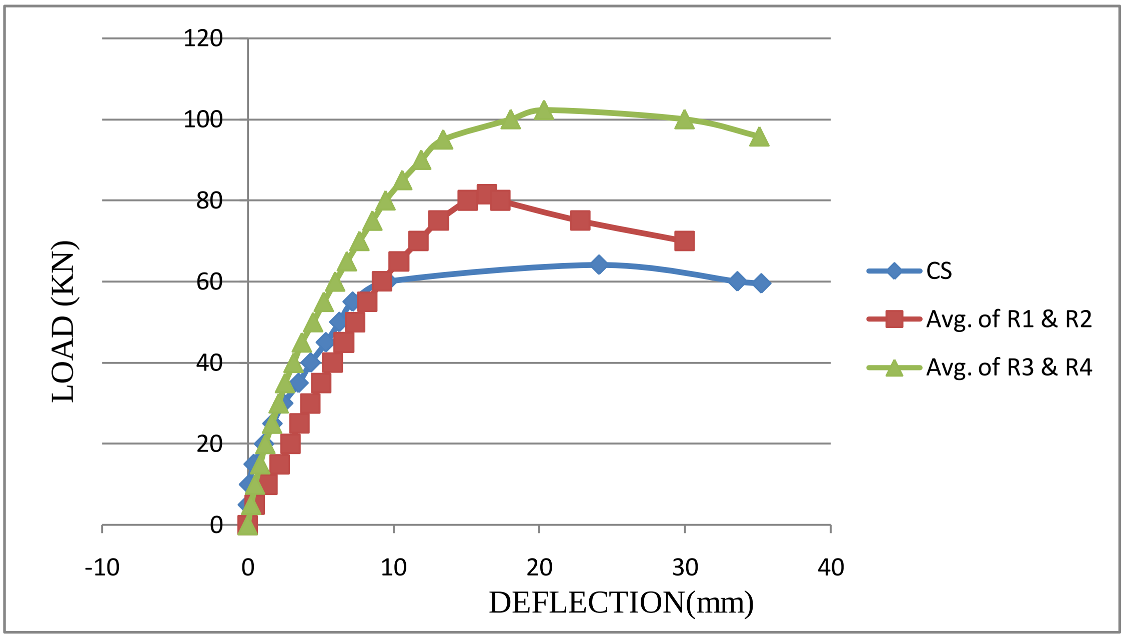


Figure 4.1Average Values of Load and Deflection at Free End of Beam of Controlled and Retrofitted specimens.

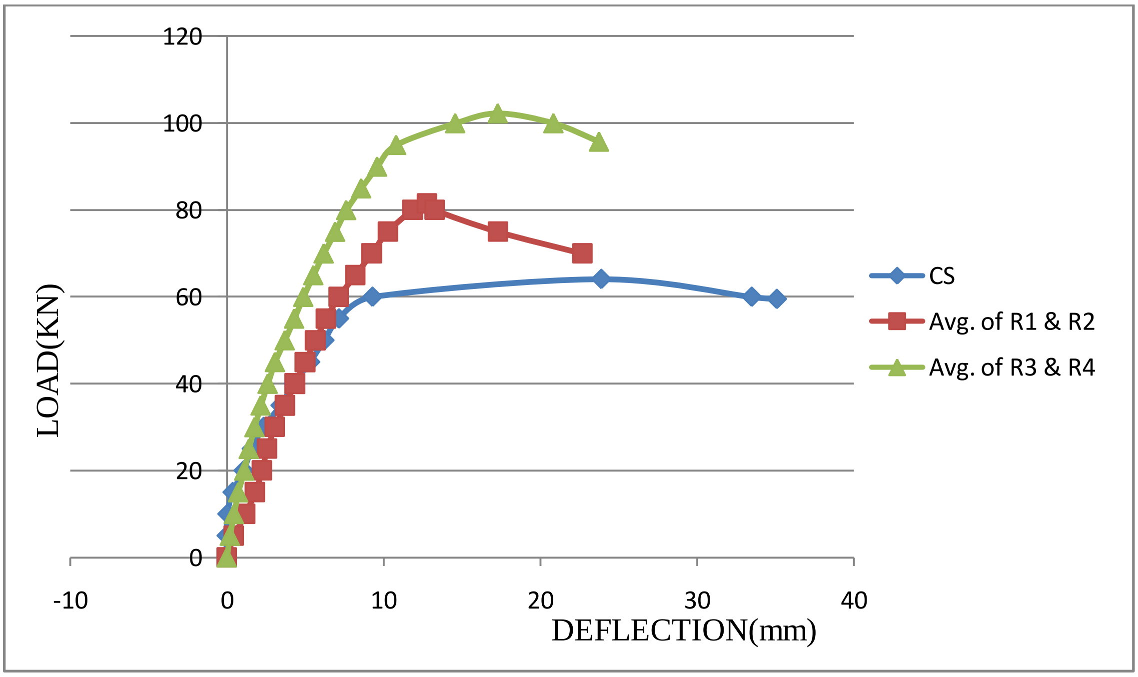


Figure 4.2 Average Values of Load and Deflection at 150mm From Fee end of Beam of Controlled and Retrofitted Specimens.

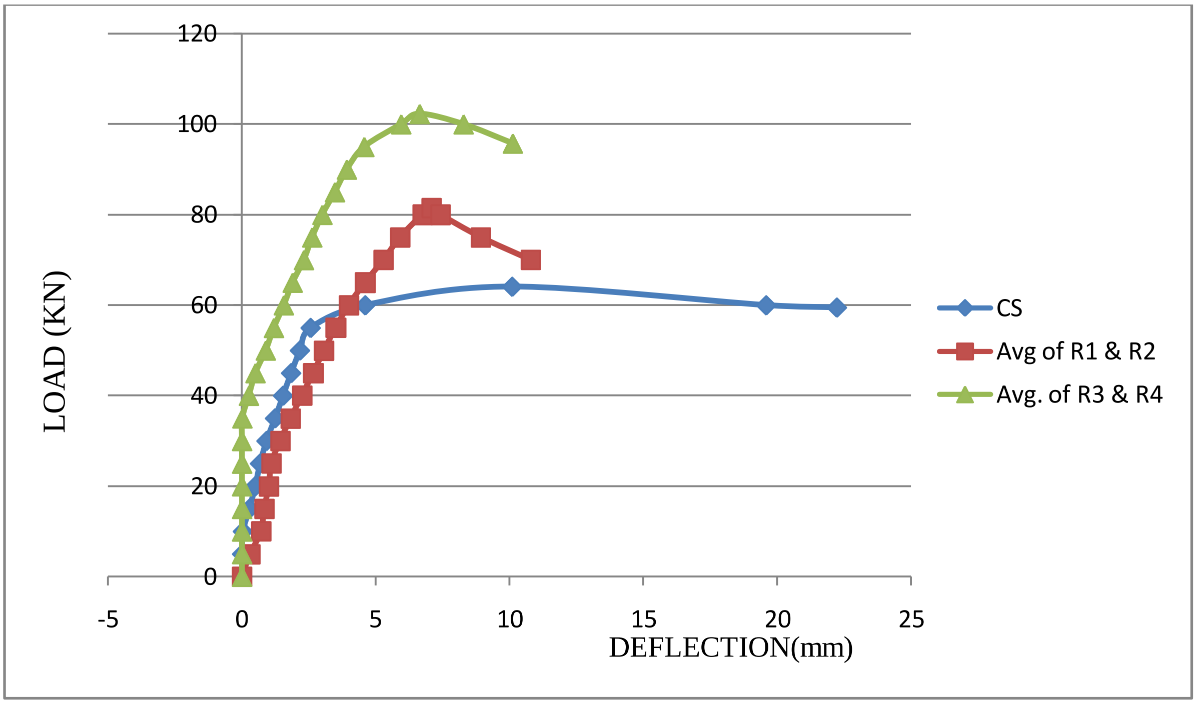


Figure 4.3 Average Values of Load and Deflection at 100mm From Column Face of Controlled and Retrofitted Specimens.

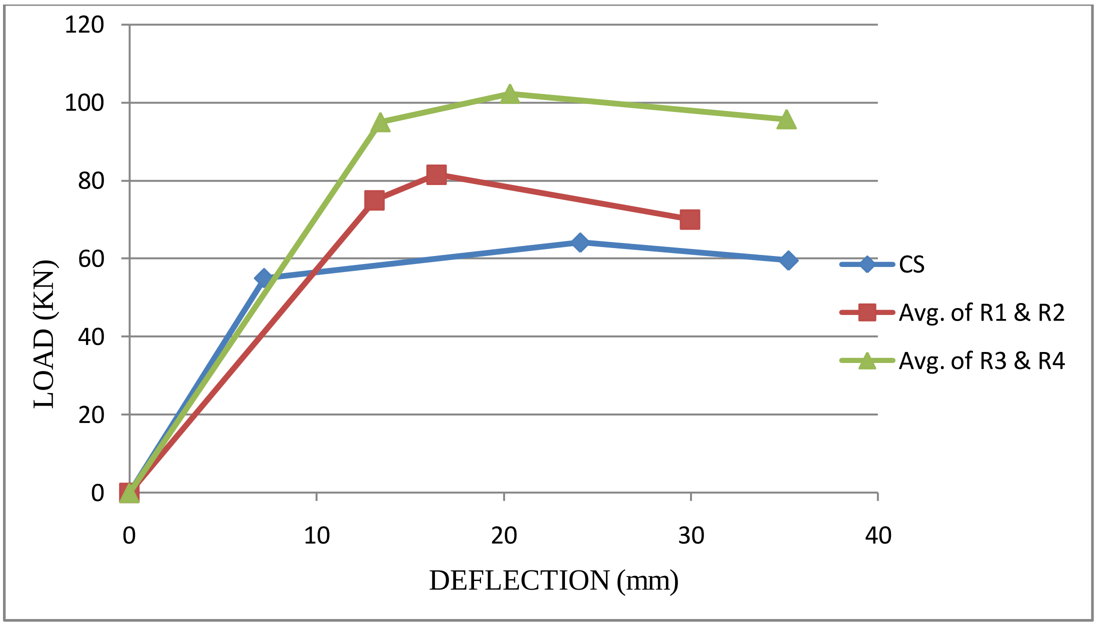


Fig4.4 Trilinear Curves For Average Values of Load and Deflection at Free end of Beam of Controlled and Retrofitted Specimens.

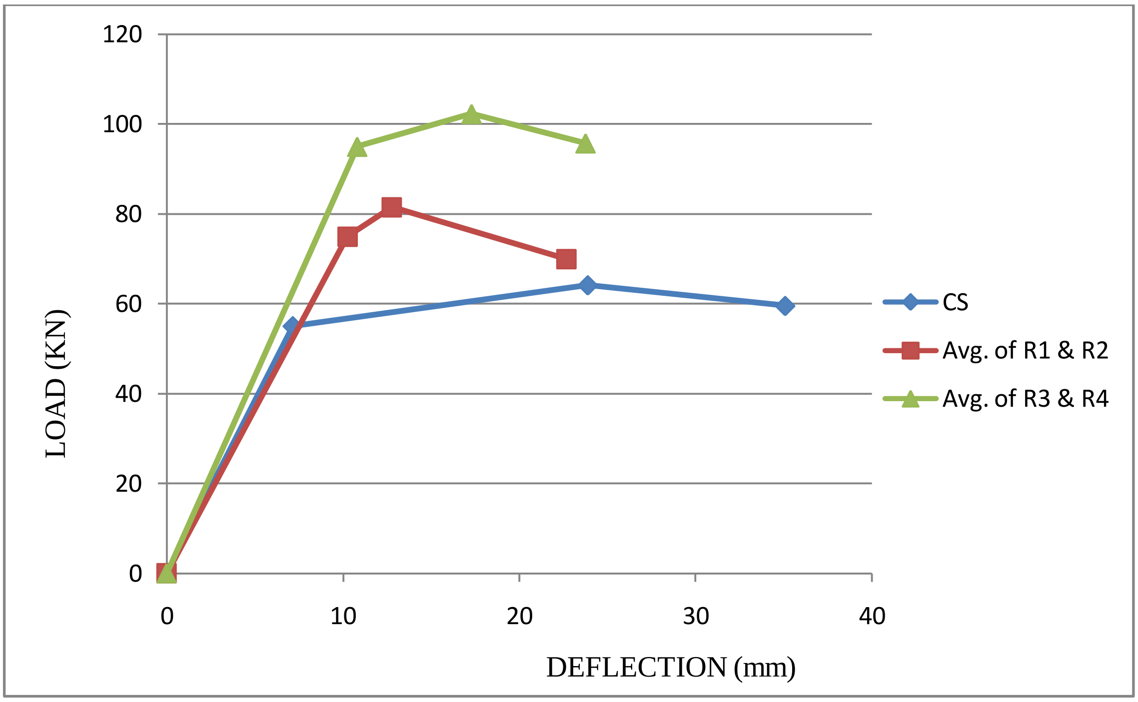


Fig4.5 Trilinear Curves For Average Values of Load and Deflection at 150mm From Free End of Beam of Controlled and Retrofitted Specimens.

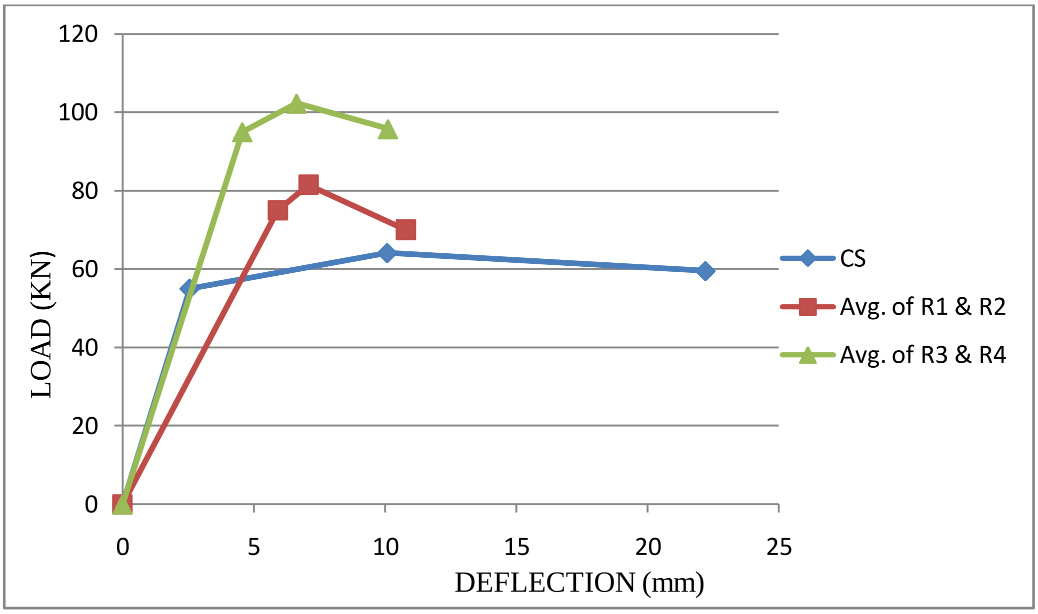


Fig4.6 Trilinear Curves for Average Values of Load and Deflection at 100mm From Column Face of Controlled and Retrofitted Specimens.

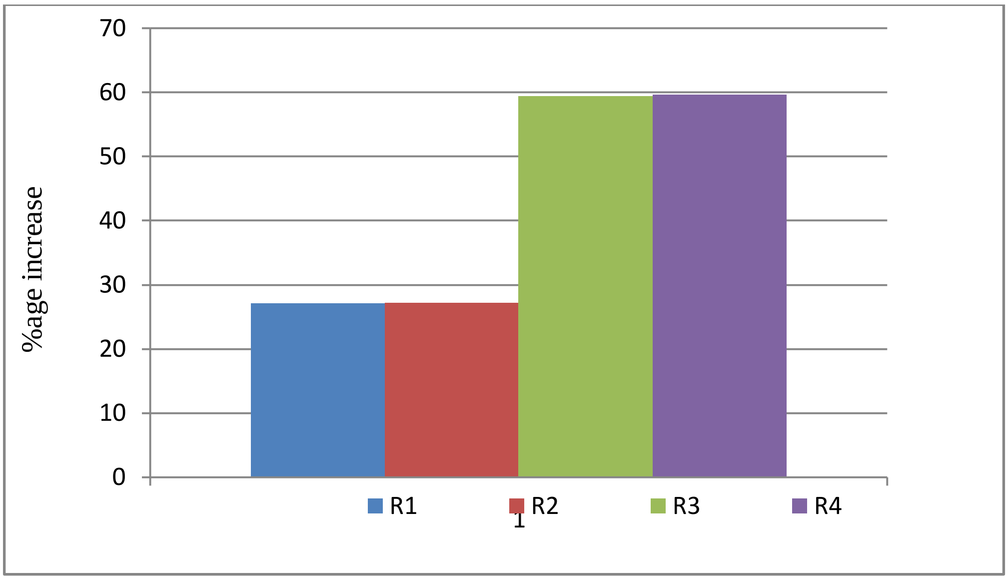


Fig.4.7%age Gain in Ultimate Load

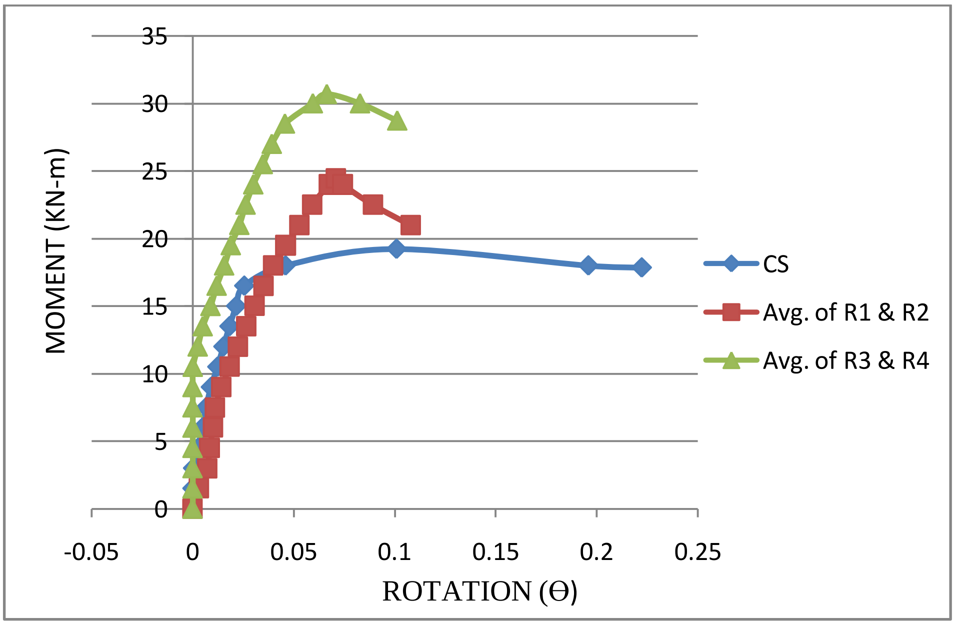


Fig.4.8 Moment and Rotation at Column Face of Controlled and Retrofitted Specimens for the Average Values of R1, R2 and R3, R4.

EXPERIMENTAL RESULT PLATES

PLATE 4.1 CONTROL SPECIMEN

PLATE 4.2 SPECIMEN R1

PLATE 4.3 SPECIMEN R2

PLATE 4.4 SPECIMEN R3

PLATE 4.5 SPECIMEN R4

CHAPTER 5 CONCLUSIONS

5.1 GENERAL

The study is carried out to analyze the Effect of Different Wrapping Techniques on Retrofitting of RCC Beam Column Joints Using Ferrocement. The important conclusions drawn from the study are as listed below:

1. The load carrying capacity of retrofitted beam-column joints for both types of retrofitting techniques increases significantly as compared to control beam-column joint.
2. Specimens with mesh wire wrapped diagonally show maximum improvement in their ultimate load.
3. There is increase in the yield load also in both types of retrofitting; in case of specimens with mesh wire wrapped diagonally there is significant increase in the yield load.
4. There is decrease in the deflection incase of retrofitted specimens as compared to control specimen
5. The ductility ratio of retrofitted specimen is less than the ductility ratio of control specimen.
6. The ductility ratio of those specimens in which mesh wire is wrapped diagonally is more than those specimens in which mesh wire is wrapped in the shape of L.
7. The value of energy absorption, incase of those specimens in which wire mesh is wrapped in the shape of L decreases as compared with control specimen, but the value of energy absorption incase of those specimens increases in which wire mesh is wrapped diagonally than the control specimen.
8. The energy absorption of those specimens in which wire mesh is wrapped in the shape of L decreases than the specimens in which wire mesh is wrapped diagonally.
9. The value of ultimate moment of retrofitted specimen is more than the ultimate moment of controlled specimen, and the ultimate moment of those specimens in which wire mesh is wrapped diagonally is more than the specimens in which wire mesh is wrapped in the shape of L. There is decrease in rotation incase of retrofitted specimen as compared to controlled specimen.

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