FINITE ELEMENT MODELLING OF REINFORCED CONCRETE BEAM COLUMN JOINT RETROFITTED WITH CFRP

A dissertation submitted in the partial fulfillment for the award of the degree of

MASTER OF ENGINEERING

IN

STRUCTURAL ENGINEERING

Submitted By

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DECLARATION

I, Faisal Mehraj Zargar, hereby declare that the work which is presented in this thesis entitled "**Finite element modelling of reinforced concrete beam column joint retrofitted with CFRP**" in partial fulfilment of requirements for the award of the Master of Engineering in Structural Engineering, submitted in the Civil Engineering Department, Thapar Institute of Engineering and Technology, Patiala, is an authentic record of the work carried out by me under the guidance of Dr. Prem Pal Bansal and Dr. Trishna Choudhury, Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala.

The matter embodied in this thesis has not been submitted in part or full to any other institute or university for the award of any degree.

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ACKNOWLEDGEMENT

A dissertation cannot be completed without the help of many peoples who contribute directly or indirectly through there constructive criticism in the evolution and preparation of this work. It would not be fair on my part, if I don't say a word of thanks to all those whose sincere advice made this period a real educative, enlightening, pleasurable and memorable one.

I would like to express my deepest appreciation to my thesis supervisor, Head and Associate Professor **Dr. Prem Pal Bansal** and **Dr. Trishna Choudhury** for their gracious efforts and keen pursuits, which has remained as an asset for the successful instrument of my thesis report. Their dynamism and diligent enthusiasm has been highly instrumental in keeping my spirit high.

I would also like to express my gratitude to my family, for their unconditional support and prayers always and constant encouragement during the entire course of my thesis work.

I would also like to offer my sincere thanks to all the faculty members and non-teaching staff of Civil Engineering Department (CED) T.I.E.T, Patiala for their assistance.

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ABSTRACT

To display the perplexing conduct of retrofitted concrete samples numerically in its non-linear zone is troublesome. This has driven engineers in the past to depend intensely on observational equations that were obtained from various tests for the plan of strengthened solid structures.

The Finite Element technique makes it conceivable to consider non-linear reaction. The FE strategy is an explanatory instrument which can show RCC or retrofitted structure and can figure the non-linear conduct of the basic individuals is Finite component technique. For the auxiliary structure and evaluation of reinforced concrete individuals, the non-linear FE examination has turned into a significant apparatus. The strategy can be utilized to ponder the conduct of strengthened and pre-focused on solid structures including both strength and stress redistribution.

The finite element strategy permits complex examinations of the nonlinear reaction of RC structures to be done in a normal style. FEM helps in the examination of the conduct of the structure under various loading conditions, its load deflection behavior and the cracks pattern. Outside wrapping with carbon fiber-reinforced polymer (CFRP) is a promising answer for retrofit of bar segment joints due to favorable circumstances, for example, high-quality weight proportion, consumption opposition, simplicity of utilization, low work costs, and no noteworthy increment in part estimate over other fortifying procedures. Additionally, ongoing exploration has endeavored to reenact the conduct of retrofitting concrete structures with CFRP composites utilizing the limited component strategy (FEM).

In the present examination, the non-linear reaction of RC structural joint under the steady state monotonic loading is examined and the focus on retrofitted (50%, 85%, and 100%) damaged RC structural joint under the monotonic loading utilizing FE modeling has been completed with the expectation to research the overall significance of a few factors in the non-linear FE investigation of RC structural joints. These incorporate the variety in load displacement diagram, the split examples, the proliferation of the damage, and the impact of the non-linear conduct of concrete and steel on the reaction of control and focused retrofitted RC structural joint.

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Chapter 1 INTRODUCTION

1.1 GENERAL

Among most of the lateral forces functioning on the structure Earthquake inflicting lateral force is one among the foremost distinguished one in several areas around the world depending on the place of the structure.

Earthquakes occur due to the slip and sliding of the masses of the rock with in the earth crust against one another. This sort of movement is principally common on the fault, an opening on the body of rock that extends for miles. Once these pieces of rocks suddenly slip and move, a colossal quantity of energy is discharged that gets propagated through the earth crust as precarious waves called as seismic waves. That progressively causes the ground a to shake and to vibrate, typically violently.

The seismic waves are categorized into 2 predominant category's: Surface & Body waves. Body waves, embody P and S type of waves, that travel via the earth interior waves act same like the sound waves that is they pack and enlarge the material as they pass. Further, the P waves tour through each solid and liquids while as, the S-waves correspond to the water waves that is they move the material up and down and experience solids solely.

After an earthquake strikes, P waves tend to ripple through the planet first, accompanied by the S waves. Then comes the slower waves, also acknowledged as the surface waves also referred as the Love and Rayleigh waves. Both sorts of waves have tendency to make the ground move horizontally though only the Rayleigh waves tend to move the ground vertically too.

1.2 STRUCTURAL BEHAVIOUR OF BUILDING UNDER SEISMIC ACTIONS

Buildings might suffer little damage if earthquakes only moved the ground vertically because all structures are designed to withstand vertical forces. Though this is not the case, surface waves of an earthquake especially the Love waves, exert extreme horizontal forces on the standing structures causing lateral accelerations in the structure, measured as G-force. An earthquake with a magnitude of 7, for example, can produce a G-force of 1G and relatively result in a velocity of 40 inches (102 centimetre) per second. Such a sudden movement to the side creates enormous stresses for a building's structural elements, including beams, columns, walls, and floors, as well as the connectors that hold these elements together. The building can collapse or suffer crippling damage if those stresses are large enough.

Another factor is the substructure of the soil profile of a house or skyscraper. Constructions on bedrock usually perform well as the ground is firm. Structures that sit atop of a soft or filled-in soil often completely fail. The greatest risk in this situation is a phenomenon known as liquefaction, which occurs when loosely packed, water-logged soils temporarily behave like liquids, causing the ground to sink or slide and the buildings along with it.

Earthquakes can cause reinforced concrete buildings to collapse, loss of lives and also staggering economic losses. Under seismic loading, it is needed for RC building to have lateral resistance capacity to counter act for brittle failure. In RC buildings, part of columns that intersects with beams are called beam-column joints. The constituent materials at joints have limited strength, thus result in having limited force carrying capacity. Thus beam-column joints in the moment frames are more susceptible to seismic loading and may experience differen modes of failure. As often observed in recent earthquakes, failure of beam-column joint connections can easily lead to the catastrophic collapse of a frame building. Buildings which are designed using the non-seismic code of practice are vulnerable to earthquake excitation's. Evidence from past earthquakes has shown that during significant events, there are always several structures that stay stable but experience different forms of damage in their members. There are various characteristics commonly observed in damaged beam-column joints, though some dominantly seen characteristics are:

- Poor shear strength
- insufficient anchorage or bonding and
- inadequate flexural strength or ductility

Under earthquake excitation's, the beams connected to the joint experience moments in same (clockwise or anti-clockwise) direction. Due to these moments, the top reinforcement of the beam column joint experience a pull in one direction and the bottom reinforcement experience push in the other direction, which is balanced by bond stress that gets developed between concrete and steel in the joint region. Now if the concrete is weak or the column is not wide enough, there is poor grip on the steel by the concrete, and therefore the beams lose the load carrying capacity. Therefore, the structure becomes more susceptible to damage and experiences a partial collapse.

Further, the reaction from the above push-pull forces at the top and bottom ends, is geometric distortion i.e. one diagonal length of the joint elongates and other compresses at the joint. This leads to the shear failure in the joint region of the beam-column joint, due to inadequate shear fortifications or reinforcement present at the joint region of the frame. Further leading to the damage of the columns with further leads to the formation of the plastic hinge zone (PHZ) near the column head which can in turn lead to global collapse of the structure i.e. the whole structure can collapse.

Thus, the essential requirement for any structure is to enable adjoining areas of the joint say beams and columns to develop and sustain ultimate loads. While providing sufficient energy dissipation to sustain the moments due to earthquake.

1.2.1 Seismic design philosophy

The basic seismic design philosophy is to provide sufficient ductility to the structural member that it dissipates the seismic wave energy acting up on it during earthquake excitations. This ductility is provided in the form of inelastic deformations. During this the material properties for design are taken beyond the elastic limit that is the material is expected to fail beyond this region, which in turn leads to energy dissipation in the structure. The basic ideology for the design for seismic resistant structure is to take material properties of such elements in the in elastic range that on failure would not cause sudden failure of the structure and thus on damage will help in energy dissipation of seismic wave energy.

There are basically two types of failures generally in the structure that can occur due to any type of loading acting up on the structure. 1) Global failure. 2) Local failure. The first type of failure i.e. Global failure is the complete collapse of all the structural elements of the structure. Basically, it's the formation of plastic hinges at the top of the column leading to high inelastic rotational demands which are very high to be catered with any possible detailing and further led to failure of transfer of axial load of the structure. Which in turn leads to the failure of the beams and thus the whole structure collapses. While the local failure means the failure of beams in the structure which is due to the formation of plastic hinges in the beam that leads to partial failure of the structure as columns are still carrying load and transferring the same to the foundation levels of the structure. In the above two kinds of failure the partial failure or the local failure of the structure is more favourable as it gives sufficient warning before the complete collapse of the structure, and thus designing of the structure should be done for this kind of failure.

One of the basic requirement of the design is to provide sufficient flexural strength above and below the beam-column joint. To ensure that hinging occurs in beams rather than in columns

1.3 BEAM-COLUMN JOINT

The basic requirement of a beam-column joint is to make the adjoining elements at the joint i.e. Beams and Columns to develope sufficient and sustain their ultimate capacity. Further as the demand on this small size of element is high under the vertical gravity loads, it gets severe during the lateral loads acting up on the structure i.e.

during seismic loading. Thus, there should always be sufficient strength and astiffness in the joint to resist the forces internally induced in the structural members

1.3.1 Types of joints in frames

Joint is defined as that portion of the column, which is within the depth of the deepest beam framing into the column. There are basically three types of joints that can be identified in the moment resisting frames:

1) Interior Joint

2) Exterior Joint

3) Corner Joint

The assembly of four beam framing in to the column on all sides of the column is called an interior joint, while the assembly of one beam framing in to the vertical face of the column and other two frame in the perpendicular direction at the joint, the joint is called as the exterior joint and at last when the beams frame into adjacent vertical faces of the column such a joint is joint.

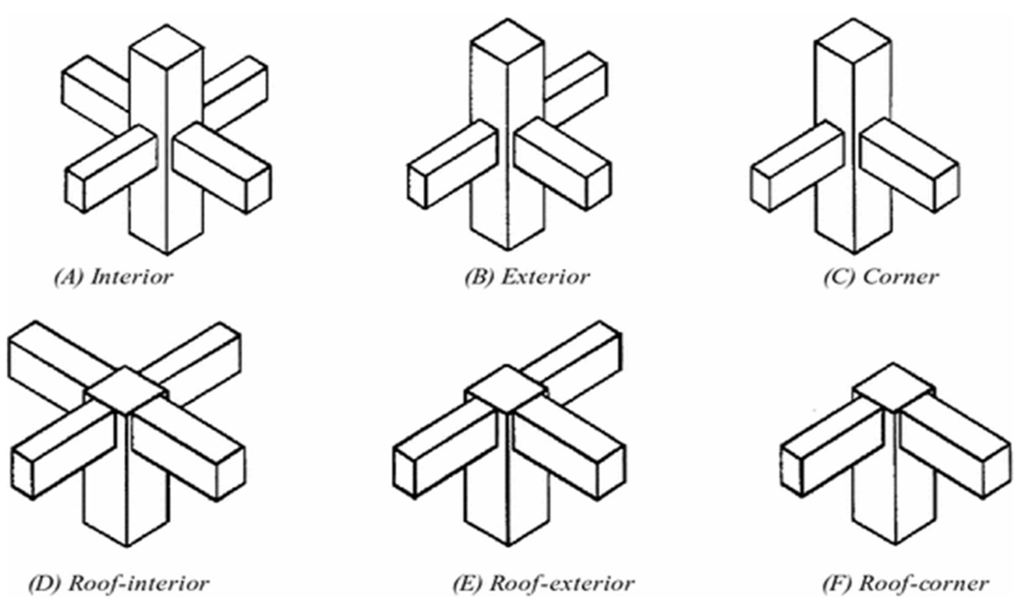


Figure 1.1 Types of beam column joint. (Uma and Prasad, 2003)

1.3.2 Reinforcement detailing of beam- column joints

The detailing of the reinforcement in the beam-column joint plays the vital role in enabling the joint to resist and sustain the load generated internally during lateral or vertical forces acting on the structure. Indian building codes provide two versions for detailing the reinforcements in the beams, column and the joints of beams and columns. These two versions are based on the loading acting on the elements i.e.(gravity and gravity plus lateral loads) and there is a significant changes in both the versions for the same elements of structures. The figure below shows the detailing provisions as per SP34-1987 and IS-13920:1993, representing non-ductile and ductile detailing respectively.

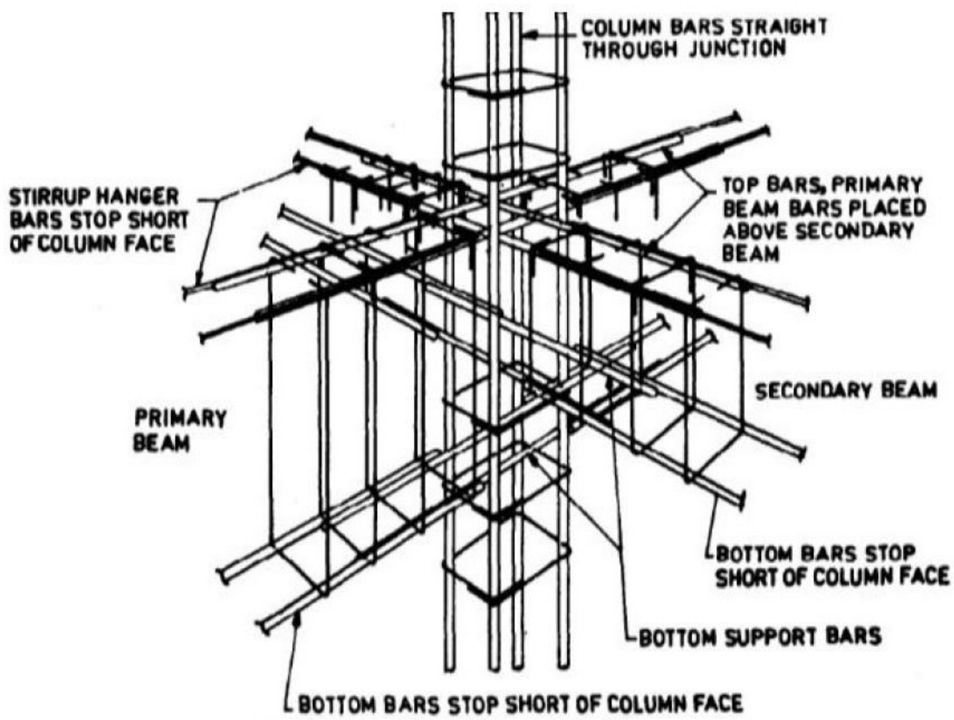


Figure 1.2 Beam column joint detailing (SP34-1987)

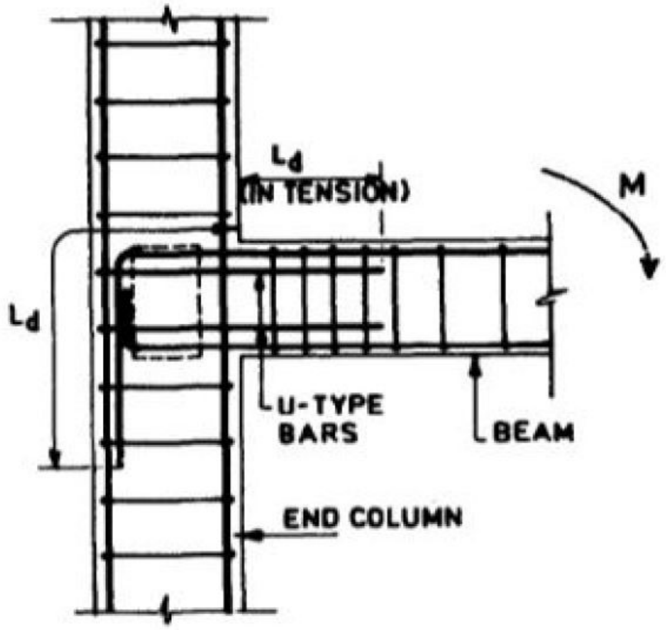


Figure 1.3 Non-ductile detailing of a beam-column exterior joint (SP34-1987)

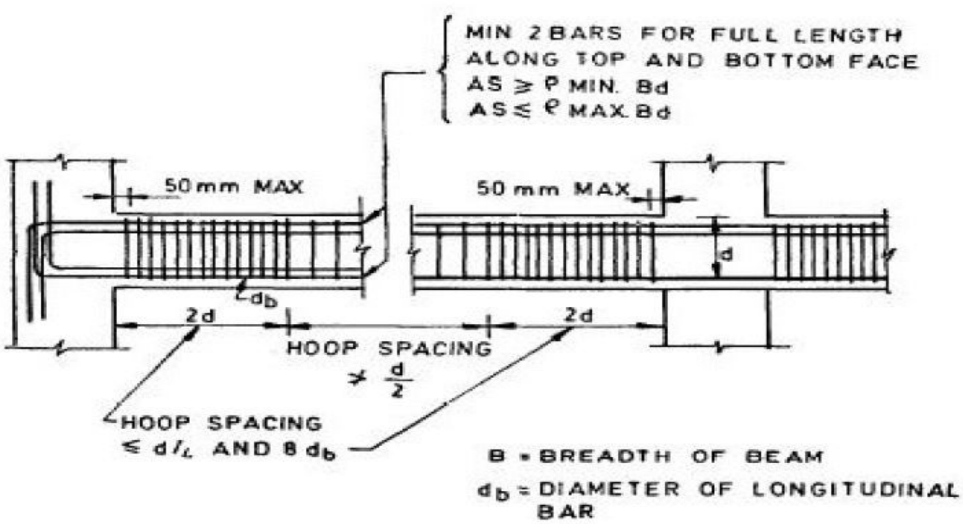


Figure 1.4 Ductile detailing of beam (IS-13920:2016)

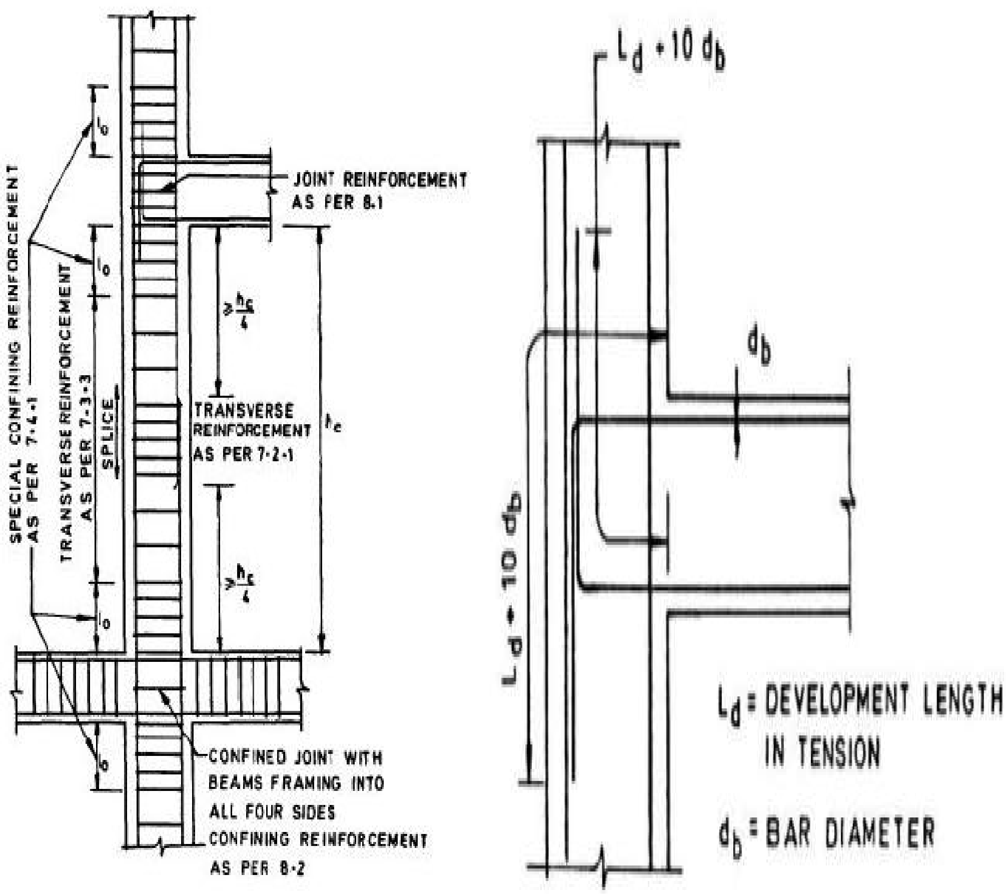


Figure 1.5 Ductile detailing of a beam column joint (IS-13920:2016)

The difference can be noticed between the non-ductile and ductile detailing of the reinforcement. The brief comparison is drawn below:

1) The spacing of shear reinforcement is reduced in the ductile detailing so as to incorporate more shear reinforcement that would cater for the shear developed in the elements of the structure and further would avoid shear failure.

2) The anchorage length in the ductile detailing is increased from the non-ductile detailed specimen by an amount of 10 times diameter of the longitudinal bar, this is to increase the anchorage of the beam with the column.

3) Further the splice length and splice positioning is also changed that can be seen from the figure 1.5.

The development length **Ld** for **IS-13920:1993** is same as described in **IS-456:2000**.

That is given by:

$$\mathbf{Ld = 0.87Fy\phi/4\zeta bd} \qquad \text{Eq. 1.1}$$

Where:

- **Fy** is the Yield strength of steel bars used.
- **Φ** is the nominal diameter of the bar used.
- **ζbd** is the design bond strength.

Further, value of Bond stress can be increased by 60% when high yield strength deformed (HYSD) bars are used.

1.4 RETROFITTING

Retrofitting refers to the addition of new technology or features to the older systems. Seismic retrofitting is the adjustment of existing structures to make them progressively impervious to seismic movement, ground movement or soil disappointment because of tremors. Retrofitting of Reinforced concrete (RC) structures can prompt expanded solidness, quality, and disappointment distortion. The strengthening and retrofitting of reinforced concrete (RC) structures are difficult but essential construction tasks. Such activities are increasingly becoming significant because of the insufficient capacity of structures that have been designed using old design codes. In addition, RC structures are often damaged by numerous factors, such as natural disasters, fire or environmental effects. Thus, structures are weakened and must therefore be either strengthened or retrofitted. Effective and construable techniques and materials should be used to improve deteriorated or substandard structural members. Deteriorated structural members must be examined and analyzed carefully to determine its in situ condition prior to strengthening work.

Furthermore, strengthening measures must be determined based on the in-situ condition of structures.

The basic classification is based on the opted reconditioning of the structures:

1. Global Retrofitting Technique.

2. Local Retrofitting Technique.

Further Global Retrofitting involves processes like

- Adding shear wall
- Adding infill walls
- Adding bracings.
- Wall thickening
- Base isolation
- Mass dampers.

While Local retrofitting involves

- Jacketing of Beams.
- Jacketing of Columns.
- Jacketing of Beam-Column joints.
- Strengthening of individual Footings.

As discussed earlier about Beam column joint severity under lateral loading, jacketing of Beam-Column joints gives a local solution to the demand on structures under loading.

1.5 FINITE ELEMENT METHOD

Finite element method is a numerical analysis technique that can be used to find approximate solution to the complex problem with the ease of material saving, cost saving and further time saving. The basic idea behind Finite element methodology of analysis is discretization of the model to be studied into smaller components and then analyzing the behaviour of each element and finally combining the results for the whole model.

Various steps involved in Finite Element Method:

1. Discretize the Solid model.
2. Selecting of functions for interpolation.
3. Identifying the material properties.
4. Obtaining System Equations by assembling material properties.
5. Application of Boundary Conditions required for the system.
6. Solving System Equations.
7. Additional Computations if necessary.

1.5.1 Applications of finite element method

1. FEM permits point by point representation of where structures bends or twists, and shows the distribution of stresses and displacement.
2. FEM can easy take the analysis of very complex geometry.
3. FEM can be used to replicate complex loading conditions as done in experimentally.
4. FEM can also be used to predict the future behaviour of assembly of elements that work together in a system.
5. FEM allows optimization, refinement and visual rendering of the assembly of geometry before the design is manufactured.
6. FEM provides stiffness and strength visualizations.
7. FEM also provides various simulation options to cater for the both modelling and analysis complexity of a system.
8. FEM allows the user to make various iterations without any loss of material, and money on the failed samples.

1.5.2 Working principle of finite element method

The limited component strategy (FEM) is predominantly discretization procedure in structural mechanics. The idea of FEM is the division of numerical model into discrete elements of fine size of straightforward geometry. The reaction of every component is expressed through a limited number of degrees of freedom described as the value of a function.

The limited component strategy is appropriate for super imposition of material models for the constituent pieces of a composite material. Progressed constitutive models executed in the finite component framework Abaqus serves with proper tools to determine the conduct of association among steel and cement. Nonlinear re-creations utilizing the models in Abaqus can be effectively used to help and stretch out trial examinations and to foresee conduct of structures and auxiliary subtleties.

A few constitutive models covering these impacts are executed in the PC code Abaqus, which is a finite component bundle intended for PC re-enactment of solid structures. The graphical UI in Abaqus gives an effective and amazing environment takes care of many securing issues. Abaqus empowers virtual testing of structures utilizing PCs, which is the present pattern in the innovative work world. Use of Abaqus for reenactment of associations among steel and cement is great. In Abaqus, concrete is defined and represented by a solid brick element, reinforcement by wire components with a beam type sub category and FRP by shell element. Material properties take a significant role in representation of a structure.

1.5.3 Importance of finite element modelling

The analytical representation of the nonlinear behavior of a material like concrete in conjunction with reinforcing steel is difficult. Which in past used to be determined using various empirical formulas that were derived from the experimental results of the RCC specimens, making it time consuming and cumbersome, to evaluate a particular behavior of the material like RCC.

FEM makes it possible to consider non-linear response. The FEM is a numerical analysis of the assembly of various materials formed in to a particular geometry that can be used to study the linear and nonlinear behavior of the assembly like RC concrete and can be equally be made to evaluate the response of the retrofitting of such structural components. For auxiliary structure and appraisal of strengthened solid individuals, the non-straight limited component (FE) investigation has turned into a significant device.

With the coming of computerized PCs and ground-breaking techniques for examination, for example, the FEM (limited component strategy) numerous endeavors to create expository arrangements which would hinder the requirement for tests have been embraced by specialists. The limited component technique has hence turned into a ground-breaking computational device, which permits complex examinations of the nonlinear reaction of RC structures to be completed in a basic manner.

FEM is useful for obtaining the load deflection behavior and its crack patterns in various loading conditions.

1.6 OBJECTIVES

In the present study, FE modelling of the RC beam-column joint in a FEM based software Abaqus was carried out, on the experimental results done by Bansal and Kumar (2014). The non-linear response of the beam-column joint and retrofitted RC beam-column joint as done in the experiment was carried out in Abaqus under the static monotonic incremental loading. Further, the load-displacement graph variations, along with damage in tension and compression in the control and retrofitted specimens were produced.

The major objectives of the analysis carried out are:

1. To analyse the load-carrying capacity and the non-linear response of the beam-column joint using the FE based software Abaqus.

2. To compare the various modelling technique available in the Abaqus, to get the results that are comparable to the experimental data.

3. To compare the results obtained for the control specimen and the retrofitted specimens for various damage levels (50%, 60%, 70%, 85% and 100%) for two types of CFRP retrofitting with two and one layer of CFRP strengthening, with the

available experimental results from the previous study done by Bansal and Kumar (2014)

1.7 SCOPE OF THE WORK

The present work expects to contemplate diagnostically the exhibition of beam-column section joint under static monotonic incremental loading. In the primary period of the present investigation of FE demonstrating of the control RC beam-column joint under the monotonic loading has been broke down utilizing Abaqus programming and the outcomes so acquired have been contrasted with the accessible exploratory outcomes from Bansal and Kumar (2014).

In the second phase of the investigation, FE displaying the damaged/harmed retrofitted RC beam-column joint is examined. The casing retrofitted by utilizing CFRP is demonstrated and broke down. The outcomes obtained from the examination of the FE model have been plotted. Correlations are made with load-deflection plots and values. Displacements, deflections and breaking conduct of the RC beam-column joint are likewise contemplated.

The following parameters are proposed to be measured:

1. Non-linear behavior of the assembly of materials of different geometry
2. Damage location in the samples.
3. Study the damage parameters of the samples on different load steps
4. FEM retrofitting techniques
5. Effects of retrofitting of damage samples.

The effect on the above parameters due to use of the retrofitting materials like CFRP for strengthening of the joint (subjected to damage) is also studied.

1.8 ORGANISATION OF THE THESIS

The thesis is organized as par detail given below:

Chapter 1: Introduces to the topic of thesis in brief.

Chapter 2: Discusses the literature review i.e. the work done by various researchers in the field of performance of beam column joint and retrofitted beam column joint under monotonic and cyclic loading. Past research on use of finite element modeling for concrete structures is also taken.

Chapter 3: Deals with the details of beam column joint modelled in ABAQUS in its first part. Second part comprises of FEM modelling, theory related to the ABAQUS, material modelling and analytical programming procedure steps involved modelling of the control RC beam column joint. It also deals with the description of the material behaviour of concrete, reinforced steel bars.

Chapter 4: The results format he analysis, comparison between the analytical and the experimental results, results comparison between the control beam column joint and the cracking behaviour of it, all are discussed in.

Chapter 5: The results from the analysis for 50%, 85% and 100% stressed retrofitted, comparison of results b/w FE analysis and experimental results of beam column joint are discussed in this chapter.

Chapter 6: Finally, salient conclusions and recommendations of the present study are given in this chapter followed by the references.

2.1 GENERAL

It is hard to show the nitty-gritty survey of the writing identified with FE modelling of RC beam-column joint and retrofitted/strengthened beam-column structural element, so a concise audit of past investigations on the use of the finite element method (limited component strategy) and exploratory examination of the structurally horizontal and vertical element joint and retrofitted structures is exhibited in this part.

Researchers have made a significant stride over the most recent two decades to improve the seismic performance of structural beam-column joint sections. This has brought about the utilization of materials, for example, a fiber-strengthened polymer (FRP). These materials give persistent control of the joint zone consequently upgrading yield load, initial stiffness, and energy scattering limit extensively. Lately, the utilization of limited component investigation to dissect concrete basic parts has likewise expanded. The limited component examination or finite element analysis gives an incredible as set to research the intricate conduct of beam-column joints, being a lot quicker and very practical when contrasted with the test-based testing.

2.2 FINITE ELEMENT MODELLING AND THE STRENGTHENING OF RC MEMBERS

Mukherjee and Joshi, (2005) analysed the effect of the carbon fibre reiforced polymer composites (FRP) in the strengthening of sound beam-column joints with satisfactory and inadequate reinforcement and furthermore in the recovery of damaged joints. One lot of joints had sufficient steel reinforced with the proper enumeration of reinforcement at the basic segments called "Ductile specimens". The other lot of samples had lacking bond lengths of the beam reinforcements at the intersections with the columns and were designated "Non-Ductile specimens". The samples were likewise reinforced by utilizing carbon and glass FRP materials. The control sample without FRP were utilized subsequent for testing to assess the restoration of joints with FRP called "Retrofitted samples". It was seen that for ductile samples the load at yield was extensively higher in the FRP strengthened samples than the control sample. For a similar tip load, the tensile forces in steel were lower in the carbon-strengthened sample than in the glass-reinforced samples. Hence, the steel in the carbon-reinforced samples yielded at higher ultimate loads. The displacement at yield expanded to a lot lesser degree than the force due to FRP. The load-displacement envelopes demonstrated that all the FRP fortified samples had higher ultimate loads than the control samples Stiffness displacement plots demonstrated that all the FRP strengthened samples had a loss of solidness (stiffness) at a higher deformation level than the control sample. The carbon samples had higher starting stiffness and a slower rate of degradation. The aggregate energy scattering diagrams demonstrated that the energy dissemination of the FRP strengthened samples pursued intently that of the control example in light of the fact that FRPs stay elastic until failing and very little dispersal of energy were normal through the disfigurement of the FRP. Be that as it may, the FRPs had expanded a definitive distortion of the structure to a huge degree. The photos of the damaged samples demonstrated that the constrainment given by the FRP strengthened joints obstructed the making of pivot through the spalling of cement. The load versus displacement graphs encompass for the rehabbed samples demonstrated that the utilization of a composite framework not just re-established the first limit of the damaged ample, in addition updated a definitive ultimate limit by 55%. Likewise, deformation at a definitive load was expanded by 30%. Stiffness versus deformation plots demonstrated that there was a 48% expansion in introductory stiffness. An addition of 57% was seen in the energy dispersal limit of the structure after the restoration [18].

Gencoglu M., et al., (2007) conducted an analysis on the reinforcing of the lacking RC exterior Beam-Column Joints, utilizing CFRP for Seismic Excitation. So as to strengthen the lacking exterior beam-column joints, CFRP layers were spread out on the tension face of column and beam and after that, both column and beam were wrapped. The test after-effects of the retrofitted beam-column joints were contrasted and the test consequences of both RC outside beam-column joint implicit agreement with the prerequisites of ACI 318-02 and RC exterior beam-column joint slighted the transverse necessities at ACI 318-02 as far as total load conveying limit, total energy amounts and ductility. Examination of the samples in the wake of testing showed that the retrofitting strategy moved the confinement pivot of the sample to the beam a nd the method of the failure of beam-column joints could be straightforwardly influenced. CFRP sheets mounted onto the solid surfaces of beam and column by utilizing epoxy tars increased the ultimate load limit, beam tip displacements, the absorbed energy, and the RC exterior beam-column joint reinforced by two layers of L-shaped CFRP sheets has the most noteworthy cyclic load conveying limit and the absolute absorbed energy sums among other beam-column joint samples [5]

Ganesan .N et al., (2007) investigated ten steel fiber reinforced high-performance concrete (SFRHPC) exterior beam-column joints under cyclic load application. It was planned to utilize a changed ACI strategy proposed by Aïtcin, under which a M60 grade of concrete was used. The volume portion of the fibers utilized in this examination changed from 0 to 1% with an addition of 0.25% for each other iteration. Joints were tried under positive cyclic load application, and the outcomes were assessed as for quality, ductility and solidness/stiffness. It was observed that there were enormous deformations and displacements in the SFRHPC specimen, without creating extensive splits which when contrasted with the HPC joint demonstrated that the SFRHPC joints due to steel filaments get high pliability, which is one of the fundamental properties for the beam-column joints. Further the extension of fibers to the beam-column joints reduced the rate of firmness degradation considerably when contrasted with the joints without strands. The ultimate load of the joints likewise increased with the increase in the amount of fiber content up to the workability limit for concrete[4].

Mahini S.S. et al., (2008) considered the capacity of nonlinear semi-static finite element modeling in re-enacting the hysteretic conduct of CFRP-retrofitted RC exterior beam-column joints under cyclic load application, utilizing FE component strategy embraced by ANSYS. The FE component models are created utilizing an altered Hognestead model for concrete and an-isotropic multi-straight model for displaying the stress-strain relations in strengthening bars while an-isotropic pliancy is considered for the FRP composite. Both cement and FRP are displayed utilizing strong components though space interface components are utilized for steel bars. The outcomes got from the ANSYS FE component investigation are contrasted and the exploratory information for two RC beam-column joints after and before retrofitting. In this investigation, a consequently transforming stiffness grid procedure was utilized so as to mimic the genuine seismic presentation of the RC concrete in the wake of splitting, steel yielding, and cement smashing during the push and pull stacking cycles. The examinations were made for load-displacement bends at mid-length; and ultimate load. The outcomes from the FE component investigation were determined at a similar area as the test trial of the beams. The precision of the FE component models is surveyed by examination with the test results, which are to be in great understanding. The load-displacement graphs from the FE component investigation concur well with the trial results in the direct range till the ultimate load, as concrete strain-softening can't be demonstrated by ANSYS [17].

Pannirselvam N. et al., (2008) exhibited the trial work for Strength Modeling of Reinforced Concrete Beam with Externally Bonded Fiber Reinforcement Polymer. In this investigation, three diverse steel proportions are utilized with two distinctive Glass Fiber Reinforced Polymer (GFRP) types and two unique thicknesses in each kind of GFRP were utilized. 15 beams were cast for this work wherein 3 were utilized as a control beam and the remaining were fixed with the GFRP cover on the soffit. Flexural test, utilizing basic beam with a two-point load point application was done to ponder the exhibition of FRP plated beams regarding flexural quality, redirection, pliability and was contrasted and the un-plated beams. The outcomes got from this examination demonstrated that the beams reinforced with GFRP overlays show better results. The flexural quality and pliability increment with increment in the thickness of the GFRP plate. The expansion in first crack load was hysterically up to 88.89% for 3 mm thick Woven Roving's GFRP plates and 100.00% for 5 mm WRGFRP plated beams and increment in flexibility as far as energy and redirection was observed to be 56.01 and 64.69% separately with a 5 mm thick GFRP plated beam. Quality mode ls were produced for foreseeing the flexural strength and pliability of FRP beams [20]

Hasballa M., et al., (2009) contemplated the practicality of utilizing the GFRP bars as a longitudinal and transverse fortification for reinforced RC frames exposed to high seismic load application. This paper centers around the test outcomes and examination of two test models. One model is completely reinforced with steel, while the other one is reinforced with glass FRP bars and stirrups. The test results demonstrated that the drift limit of the joint can achieve over 3.0% securely with no extensive harm; additionally, GFRP bars were fit for resisting tension-compression cycles with no quality degradation. GFRP-reinforced joints fulfilled both strength and pliability (deformability) prerequisites of earthquake-safe structures. The steel-reinforced sample showed the joint-shear method of failure, which brought about the deformation of the joint. Then again, GFRP reinforced example displayed a flexural slippage failure bec ause of the inadequate improvement length for the GFRP longitudinal support of the bar [7].

Robert .R.S and Prince .A.G., (2010) displayed the exploratory work on Behavior of Reinforced Concrete Beam-Column Joints Retrofitted with GFRP-AFRP Hybrid Wrapping. Under this work, three external reinforced column-beam joint control samples (The segments had a cross-area of 200 mm x 200 mm with the length of 1500 mm and the bars had a section of 200 mm x 200 mm with a cantilevered part of length 600 mm) were loaded to complete damage. Two examples had been detailed according to code IS 456:2000. The other example had been detailed according to code IS 13920:1993. A pivotal loading was connected to the column (vertical segment). Cyclic load application at the free end of the cantilever bar till complete damage. The two sample joint samples structured according to code IS 456:2000 were retrofitted with GFRP-AFRP/AFRP-GFRP half and half fiber sheets wrapping to reinforce the samples. The exhibition of the retrofitted column- beam joints were contrasted with the control joint sample. The ultimate load conveying limit and energy-absorbing limit of the reinforced sample examples structured and defined according to code IS 13920:1993 was 10% - 11% and10 % - 13.8% more than the samples defined according to code IS 456:2000 individually. The load conveying limit and energy-absorbing limit of the sample retrofitted with the GFRP-AFRP cross breed sheet was 18.3 % and 26.6 % more than the control samples.[25].

Perumal.P and Thanukumari.B., (2010) analyzed the impact of mixed fiber reinforced cement (1.5% of steel fiber and 0 to 0.6% polypropylene fiber) to add to the Seismic Performance of Beam-Column Joints utilizing M20 concrete. Six of one fourth scale samples were built according to IS 456:2000 and one structured according to IS 1893 (Part 1): 2002 and point by point according to IS 13920-1993. The five samples were like the first however different mixes of mixed fiber concrete in the joint area. Out of five fiber samples, four samples were built by utilizing (steady 1.5% of steel fiber and 0 to 0.6% polypropylene filaments). The fifth fiber sample was built by utilizing 1.5 % polypropylene fiber. The properties like strength, energy dispersal and stiffness of the joint were looked at. The raise in polypropylene fiber diminished a definitive load conveying limit. The energy absorption limit expanded by 87% by including just steel fiber and 205% by including a mix of 1.5% fiber and 0.2% polypropylene fiber. The sample with 1.5% of steel fiber and 0.6% polypropylene fiber had the most extreme ductility factor. The abundance of poly- propylene fiber raised flexibility. In this paper, it was seen that examples comprising of 1.5% of steel fiber and 0.2% of polypropylene fiber have the best execution considering the energy dissemination limit and pliability factor however a definitive load conveying limit is diminished by including polypropylene fiber.[22].

Rajaram.P et al., (2010) examined scientifically the auxiliary conduct of the RC be am-column joint utilizing standard programming bundles STAAD Pro and ANSYS. A two narrows five-story support RC moment-resisting frame has been broke down and planned in STAAD Pro according to IS 1893:2002 code method and point by poi nt as IS 13920:1993 proposals. A beam-column joint has been scaled to 1/5th from the model and the model has been exposed to cyclic load application to discover its conduct during an earthquake. Test results were decrease contrasted with FEM model investigation in ANSYS, the conduct of the beam-column joint is comparative. The greatest anxieties are created in the FEM model at the intersection. With the increase expansion in the load, there is a decrease of steadiness. The relative energy retained during loading cycles raised with cycles of loading [24].

Choudhury .A.M., (2011) researched on ductility stressing size impact for plain and Retrofitted Beam-Column joint under reversal displacemental incremental loading. In this investigation, a full-scale private structure with floor to floor stature as 3.3 meters and a beam of 3.0 meters viable range was considered. Four kinds of samples, specifi cally, Beam-column joint with beam feeble in shear: control, Beam-column joint with column deficient in shear: control, Beam-column joint with beam deficient in shear: retrofitted and Beam-column joint with column deficient in shear: retrofitted were considered. In each kind, three geometrically comparable samples of full-scaled, two-third scaled and 33% increase scaled sizes were tried. Subsequently, all-out of twelve examples covering two distinct sorts and three unique scales were tried under cyclic stacking. Cyclic dislodging was connected to every one of the examples with the assi stance of water powered unique actuators. Uprooting controlled burden with a recurrence of 0.025Hz was connected to the test examples. Efficiency of the relocation chronicles were downsized for two-third and 33% models. A definitive burden conveying limit due to retrofitting expanded up to 12.18% for beam feeble in shear enormous example. A definitive burden conveying limit with regards to every one of the examples expanded due to retrofitting and it was comparable for column deficient in shear samples. Likewise the removal ductility for every one of the samples raised due to retrofitting and it was comparable for column deficient in shear samples moreover. In this paper, it was seen that ductility and extreme burden conveying limt due to retrofitting increments as the sample size was reduced.Further the standardof size impact for both control and retrofitted examples [3]

Xilin lu. et al., (2011) exhibited the way of thinking including the utilization of extra inclining bars inside the joint under cyclic stacking. In this paper, ten full-scale beam-column samples were developed with different extra support subtleties. The samples with extra bars contrasted and the ones without them indicated fewer breaks in the column. With the expansion of the proportion of twisting snapshot of the columns to beam, the plastic pivots are bound to create in the beam, and the flexibility of the joint improves. Extra corner to corner bars, anticipated breaks at the edges of joint interface among columns and beam. Besides, these joints have been demonstrated to act in a pliable way as beams experience plastic pivoting sooner than the column The examples with extra bars viably increment the quality limit at the joint region ju as adequate improvement of pliability to the casing individuals under expanding hori zontal stacking. The component created in these samples demonstrated adequate prof to suit a solids column feeble beam idea. The direction of the extra corner to corner bars included quality for individuals they wer arranged to. That is, extra bars along beam included quality towardsthe beam closes and extra bars along column included quality towards the column [16].

Choudhury .A.M. et al., (2010) inspected on vitality dissemination with stress on size impact for plain and Retrofitted Beam-Column joint under cyclic stacking. In this investigation, a run of the mill full scale private structure with floor to floor tallness as 3.3 meters and the light emission meters' successful range was considered. Two classifications of examples, viz. shaft segment joint with pillar frail in shear and bar section joint with segment powerless in shear alongside their comparing retrofitted examples were considered. In each sort, three geometrically comparative examples of full scaled, two third scaled and 33% scaled sizes were tried. In this manner, complete twelve examples covering two distinct sorts and three unique scales were tried under cyclic stacking. Cyclic dislodging was connected to every one of the examples with the assistance of water powered unique actuators. Removal controlled burden with a recurrence of 0.025Hz was connected to the test examples. Plentifulness of the uprooting chronicles were downsized for two-third and 33% models. The heap conveying limit of the retrofitted example expanded in contrast with control example. A definitive burden conveying limit with regards to every one of the examples expanded due to retrofitting and it was comparable for section frail in shear examples. Additionally, the increase in vitality dispersal due to retrofitting was 33.08% at disappointment arrange for bar powerless in shear huge example. The increase in vitality dispersal due to retrofitting was 85.7% at disappointment organize for bar frail in shear medium example and the equivalent is 97.7% for little example. Vitality scattering for every one of the examples expanded due to retrofitting and it was comparable for segment powerless in shear examples moreover. In this paper it was seen that vitality dispersal and extreme burden conveying limit due to retrofitting increments as the example size declines. Both vitality dispersal and extreme burden conveying limit pursued the guideline of size impact for both control and retrofitted examples. Vitality scattering per unit volume additionally expanded as the example size diminished [2].

Patil S.S et al., (2013) analysed systematically the RCC beam-Column junction exposed to Quasi- Static monotonic loading by nonlinear finite element or limited component investigation utilizing commercial software for FEM ANSYS for the nonlinear examination of the reinforced solid structures. The outside beam-column joint is modelled and the loading is done at the free end of the beam, till the failure of the structure (ultimate load carrying capacity is reached). It was surmised from the papers that as the load is increased. The firmness of the structure changes and displacement of the joint or damage of the joint increases with respect to the loading [21].

2.3 GAPS IN RESEARCH AREA

Some researchers have also investigated or work on the area of the failure mode of the CFRP strengthened structural members and some are work on the strengthening of the structural members with different FRP materials.

This research is concerned with the finite element modelling of the retrofitted the RC beam using the CFRP. The use of CFRP sheets for retrofitting and the strengthening of the reinforced concrete structural have been studied extensively in previous studies. However, many researchers performed experimentally and analytically the strengthening of the beams but limited work is done on the study of RC beam column joints and in the field of stressed/damaged retrofitted structural members. Further this research looks at different procedures that can be followed analytically in the FE based software Abaqus to conclude on the effects of retrofitting a damaged beam-column joint with CFRP also a brief comparison has been made of the retrofitted specimens with the ductile detailed specimen to find the effectiveness of the CFRP as a retrofitting material.

Chapter 3 MODELLING OF BEAM COLUMN JOINT

3.1 GENERAL

This chapter presents the modeling of the Beam-Column joint, previously studied experimentally by Bansal and Kumar (2014). The material properties, boundary conditions and loading conditions are taken from the same. The study includes modeling of control specimens and damaged samples retrofitted with CFRP, under quasi-static monotonic loading as is done experimentally by Bansal and Kumar (2014. Finite element based commercially available software **ABAQUS** has been used to conclude this study. Control samples have been stressed up to 50%, 85% and 100% of the ultimate load of the control specimen, and then retrofitted with the one and two layers of the CFRP material to get an idea about the effectiveness of the CFRP to make the stressed specimen regain its strength and further.

This chapter further discusses the modeling ways in the FE based software ABAQUS and the information regarding the steps necessary to create the model needed to generate the analytical load-deflection response of the model experimentally studied by the Bansal and Kumar (2014).

3.2 GENERAL DESCRIPTION OF STRUCTURE

In the experimental program conducted by Bansal and Kumar (2014), behavior of unconfined external RC beam-column joint (T-joint) under monotonic loading is studied. Same specimens after the tests were retrofitted with CFRP sheets in the damage area to restore their strength. Four specimens in general were used among which one specimen was used as a control specimen to get the ultimate load carrying capacity of the samples, the rest of the samples where tested for different percentage of that ultimate load carrying capacity to induce damage in the models. The samples were tested for a stress level of 50%, 85% and 100% of the ultimate load carrying capacity, of the control specimen. The damaged samples were then retrofitted with the layers of CFRP. To conclude the effectiveness of CFRP as a retrofitting material, further to study the damage induced in joint and the counter act created by the application of the CFRP on the damaged specimens.

3.2.1 Material properties

The material used for construction is Reinforced concrete with M-20 grade concrete and a fe-415 grade reinforcing steel for the main longitudinal and vertical bars in the beam and column respectively. Further the shear reinforcement was provided of fe-250 grade of steel for both the beam and column. The RC T-joints were constructed with detailing as per the IS:456-2000. The CFRP used for retrofitting of the damaged specimens was reported to have a tensile strength of 3800MPa.

3.2.2 Model geometry

Length of the beam = 500 mm

Height of the column = 1000 mm

Cross section of the beam = 125*225 mm

Cross section of the column = 125*225 mm

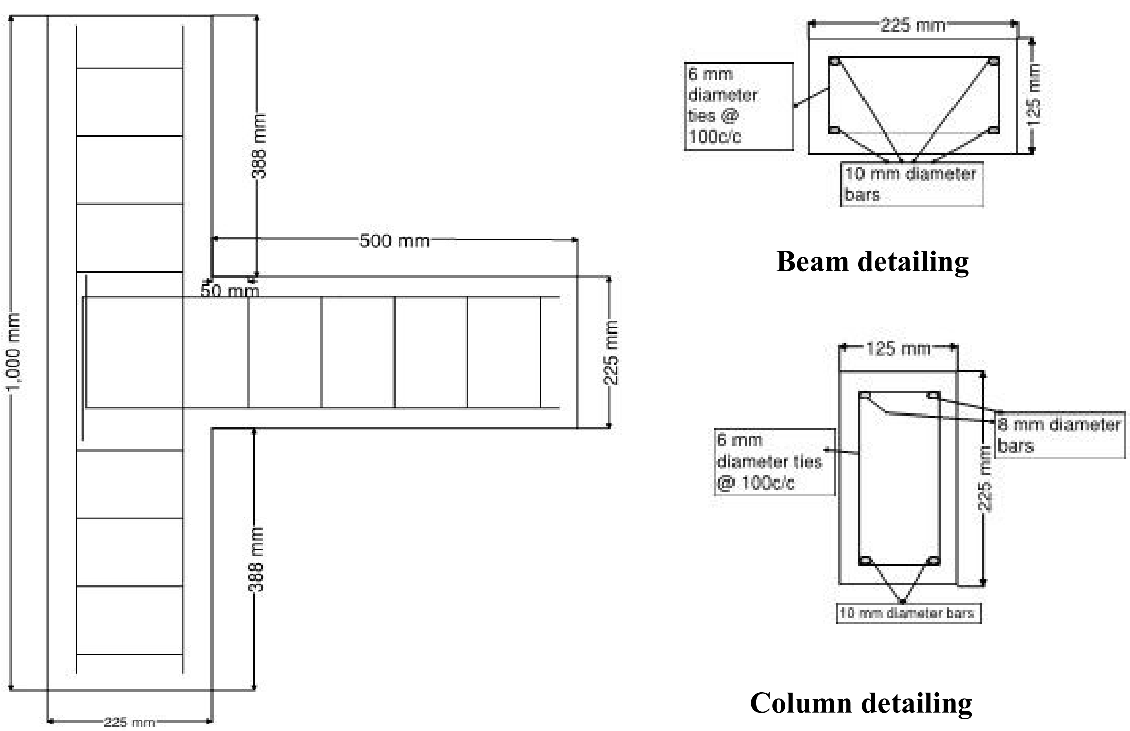


Figure 3.1 Detailing of the Beam-column Joint [27]

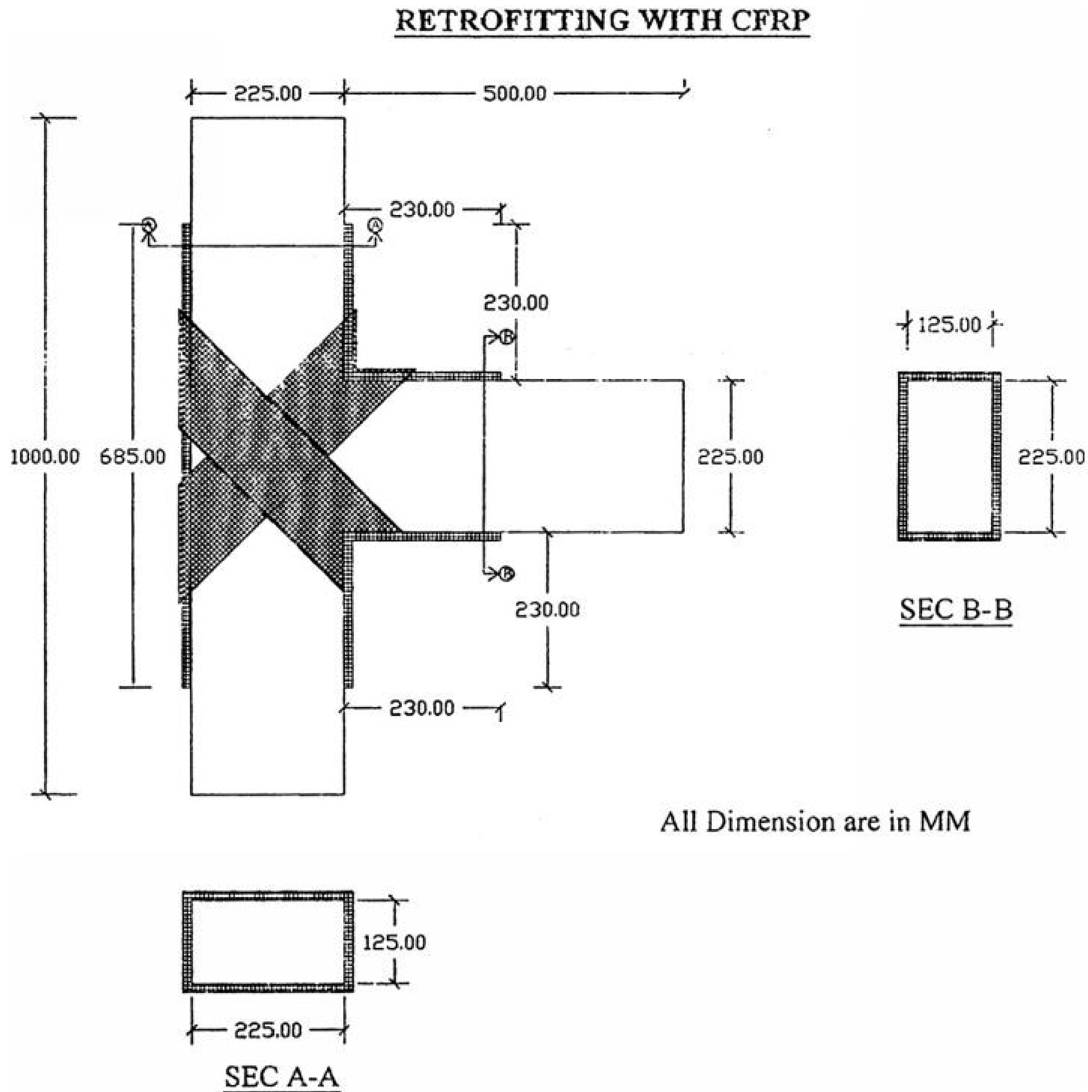


Figure 3.2 Retrofitting of the damaged joint with CFRP layers [27]

3.3 ABAQUS MATERIAL MODELS

The program Abaqus offers an assortment of material models for various materials and purposes. The most significant material models in Abaqus for RCC structure are for concrete and Reinforcements. These propelled models demonstrate all the significana parts of genuine material conduct in strain and stress. CFRP material demonstrating is additionally considered.

3.3.1 Modelling of concrete

1) Geometry of the concrete

Element geometric modelling of concrete has been done using 3D solid brick element with 8 to 20 nodes denoted by C3D8R or C3D20R respectively in Abaqus, shown in Figure 3.3

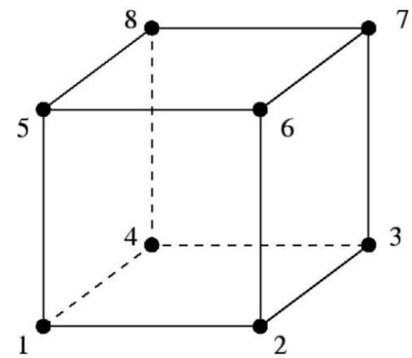


Figure 3.3 C3D8R-8 nodded 3D solid brick element

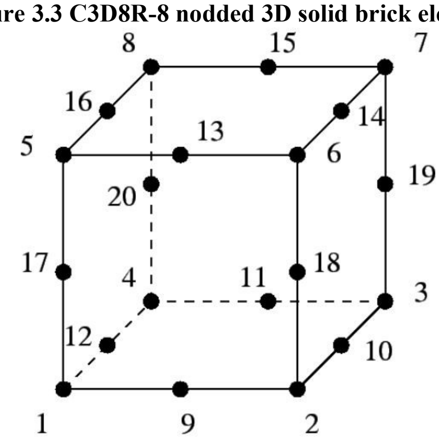


Figure 3.4 C3D20R-20 nodded 3D brick element

2) Element properties

3D solid brick element having three degree of freedom at each node: translations in the nodal x, y and z directions. This is an isoparametric element integrated by Gauss integration at integration points. This element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. The most important aspect of this element is the treatment of non-linear material properties.

3.3.2 Modelling of reinforcement

1) Geometry of the reinforcement

Reinforcement modelling is done as a wire part further sub-categorized as a

Beam element. Denoted in Abaqus as B31. In our work, a discrete modelling of reinforcement has been done. The reinforcement has been modelled using beam elements in Abaqus.

2) Element properties

Support steel is a 3D beam component, which has three a degrees of freedom at every node; interpretations in the nodal x, y and z directions. Beam element is a uniaxial pressure component. The pressure is thought to be uniform over the whole component. Likewise, pliancy, creep, swelling, enormous diversion and stress-solidifying abilities are incorporated into the component.

3.3.3 Modelling of CFRP

The CFRP modelling can be done as a 3D shell element in **Abaqus**. The shell element implemented in **Abaqus**, denoted by S4R shell element. This S4R element belongs to a group of shell element, formulation that is based on 3D elements concept. It can be used to model thin as well as thick shell or plate structures.

1. Geometry of the CFRP

The CFRP can be modelled as a shell element denoted by S4R in **Abaqus**. Shell components are utilized to show structural elements in which one measurement, the thickness, is fundamentally smaller than other measurements. Ordinary shell components utilize this condition to discretize a body by characterizing the geometry at a reference surface. For this situation, the thickness is characterized through the area property definition. Traditional shell components have displacement and rotational degrees of opportunity.

2. Element property of the CFRP

Both general shell sections and shell sections integrated during the analysis allow layers of different materials, in a different orientation, to be used through the cross-section. In these cases, the section definition provides the shell thickness, material, and orientation per layer.

3.4 DEFINING CONCRETE IN ABAQUS

Concrete shows incalculable scaled down scale breaks, especially, at the interface between coarser aggregate and mortar, even before exposed to any load. These scaled down scale parts extraordinarily influence the mechanical behavior of Concrete, since their spread during increment in loading, adds to the nonlinear behavior at low stresses and causes volume expansion near failure. Numerous of these scaled down scale breaks are brought up by shrinkage or warm expansion of the mortar. Some little scale parts may make during loading considering about the firmness among the utilized kind of aggregates and mortar. Since the aggregate mortar interface has lower rigidity nature than mortar, it sets up the weakest association in the composite system. This is the basic clarification behind the low elasticity of mortar. The response of a structure under loading depends to a huge degree on the pressure strain association of the constituent materials and the stresses. Since concrete is used generally in compression, the stress strain association in compression is of fundamental interest.

3.4.1 Elastic behaviour of concrete

The program Abaqus offers an assortment of material models for various materials and purposes. The most significant material models in Abaqus for RCC structure are for concrete. These propelled models demonstrate all the significant parts of genuine material conduct in strain and stress.

Definition of concrete in Abaqus can be defined under material properties, where elastic properties one can simply define the Modulus of Elasticity (Ec), poisons ratio (V) and density of concrete (p) for the gravity or self-weight of the concrete material to be in corporated in the analysis.

In the availability of the experimental data obtained by cube testing on the cubes of the concrete material used, one can with ease make out the Modulus of elasticity of the concrete by dividing the max compressive stress with the corresponding strain:

From Hook's Law:

$$\sigma \equiv E \times \varepsilon \qquad \text{Eq. 3.1}$$

Which can be further derived for Modulus of Elasticity as

$$E \equiv \frac{\sigma}{\varepsilon} \qquad \text{Eq. 3.2}$$

Further if the experimental data is not known one can simply use the codal provision, provided in building codes of every country, that approximates the modulus of Elasticity (Ec) based on the compressive strength of the concrete in use.

- **As per IS 456-2000 (Cl 6.2.3.1)**

$$E_{\varsigma} \equiv 5000 \times \sqrt{f_{ck}} \qquad \text{Eq 3.3}$$

- **As per ACI-318M-14 (Cl 19.2.2.1 b)**

$$E_{\varsigma} \equiv 4700 \times \sqrt{f_{ck}} \qquad \text{Eq 3.4}$$

Where fck and fc' are the compressive strength of the concrete after 28 days of curing The poisons ratio used for the definition of concrete in Abaqus is usually taken in between 0.18 to 0.2. The density of the concrete as known is 24 kN/m^3, as Abaqus is basically unit less and it depends on the user that which system of units is followed during the modeling, for this study all the units are taken in SI(mm) as described in the below figure:

3.4.2 Non-linear behavior of concrete

Concrete is a non-linear material, and to accurately evaluate the behavior of Concrete in Abaqus its necessary to define the parameters required to get the non-linear behavior. For this purpose, only two material models are presents in Abaqus for concrete material. 1) Concrete Damage Plasticity model and 2) Concrete Smeared Cracking model. Both of which are used to induce the non-linearity in Abaqus for Concrete material. While Concrete smeared cracking, model is developed to get the non-linear behavior of concrete when subjected to monotonic loading, the other model Concrete damage plasticity is developed to effectively represent the cyclic as well as monotonic loading behavior of Concrete and visually represent the damage in concrete in tension and compression due to the loading. This study takes Concrete damage plasticity model in to account to get the most damaged areas in the model under loading and retrofit those areas in the later stage.

3.4.2.1 Concrete damage plasticity model

Concrete is not elastic in nature. A better approach is required to describe the non-linear behavior of concrete in the FE based analysis software Abaqus. Therefore, a concrete damage plasticity is provided in Abaqus, so that the user can define the stress strain behavior of the concrete used in the model. Concrete damage plasticity basically assumes that there are two basic modes of failure in concrete:

1. Concrete Crushing
2. Concrete Cracking

Concrete crushing is related to the compression of the concrete, due to the axial loading or the bending caused due to flexure. While the Cracking is due to tensile stress in the concrete, as its well known that concrete performs well for compression and bad for tensile loading. Therefore, determination of both the properties is essential to know accurately the behavior of the concrete used as a material for modeling. The evolution failure surface is controlled for concrete in Abaqus by two hardening variables, ε_{tpl} and ε_{cpl} , linked to failure mechanisms a under tension and a compression loading, respectively. Where the two parameters, ε_{tpl} and ε_{cpl} are referred to as the equivalent plastic strains,in tension and compression respectively.

The concrete damaged plasticity model in Abaqus:

- gives a general capacity for displaying concrete and other materials in a wide range of structures (pillars, supports, shells, and solids);
- utilizes ideas of isotropic damaged elasticity in blend with isotropic elastic and compressive plasticity to determine the inelastic conduct of cement;
- can be used for plain concrete;
- can be used in combination with rebar to model concrete w reinforcement;
- is designed for applications in which concrete is subjected to monotonic, cyclic, and/or dynamic loading under low confining pressures;

There are three different parameters that are required to be put in before defining the concrete material properties in terms of stress and strain values:

1) Dilation angle (Ψ)

Dilation angle is a variable require to characterize the flow potential capacity. The flow potential capacity speaks of the volume change caused because of plastic deformations for solid material (Lubliner J. et al.1989). Mostly used values of Dilation angle for concrete vary between 25° to 40°. For this study a value of 34° was fund more suitable.

2) Eccentricity

Eccentricity characterizes the speed for flow potential function to reach its aasymptote. Default value of eccentricity is 0.1 for concrete.

3) Viscosity parameter

Non-linear exhibiting material model often lead to convergence errors during analysis. Viscosity parameter is used in such situation to remove the instabilities during the step time increments and thus remove the convergence problems in the material model. In general, the value for this parameter is 0 in Abaqus, though various researchers have used values negligibly greater than 0, to get the model to converge though great caution should be taken while using the values for viscosity parameter as with certain values the model may show acceptable results but it would be having a wrong behaviour results. For this study a value of 0.00001 has been found to be enough for the convergence.

1. Concrete in Compression

The stress-strain curve of concrete can be sub-divided in to three phases:

1) Elastic Phase
2) Elastic-Plastic Phase
3) Softening Phase

The input parameters required by Abaqus to evaluate the Elastic Phase as described earlier are the modulus of elasticity (Ec) and poisons ratio (v), for this study the value of modulus of elasticity is taken as per the ACI 318-14, the equation number 3.3 can be referred for the same. Further the value of poisons ratio for concrete is taken as 0.2. The concrete in compression is assumed to be remain elastic up to a stress of 0.4fck.

For Elastic-Plastic phase various models are presented by different researchers over time and an appropriate equation set needs to be chosen. For this study a material model equations prescribed by CEB-FIP,2010 have been considered for the stress range of 0.4fck to fck. The equation 3.5 represents the Elastic-plastic behaviour of the concrete in this phase.

$$\sigma_c = f_c\left(\frac{k.\eta - \eta^2}{1+(k-2).\eta}\right) \qquad \text{Eq. 3.5}$$

$$\eta = \frac{\varepsilon}{\varepsilon_c} \;\; \& \;\; k = \frac{E_0}{E_C}$$

In the above equation the parameters unknown are :

σ_c = Compressive stress of concrete (MPa)
f_c = Compressive strength of concrete (MPa)
η = normalised strain
ε = strain
ε_c = strain at peak stress f_c
E_0 = initial elastic modulus (MPa)
E_c = secant elastic modulus at peak stress = f_c / ε_c (MPa)

Further for the softening phase of the concrete non-linear behaviour in compression is determined as per equation 3.6 taken from (Alfarah, López-Almansa and Oller, 2017)

$$\sigma_c = \left(\frac{2+(\gamma_c f_c \varepsilon_c)}{2f_c} - \gamma_c \varepsilon_c + \frac{\varepsilon_c^2 \gamma_c}{2\varepsilon_c}\right)^{-1} \qquad \text{Eq. 3.6}$$

$$\gamma_c = \frac{\pi^2 f_c \varepsilon_c}{2\left(\frac{G_{ch}}{l_{eq}} - 0.5 f_c\left(\varepsilon_c(1-b) + b\frac{f_c}{E_0}\right)\right)^2} \;\; \& \text{ b} = \frac{\varepsilon_c^{pl}}{\varepsilon_c^{ch}} \qquad \text{Eq. 3.7}$$

G_{ch} = Crushing energy per unit area (N/mm)
l_{eq} = characteristic length of an element
b = 0.9 assumed based on experimental observations(Alfarah, López-Almansa and Oller, 2017)
Crushing energy value determined from the following set of equation (3.7.1) & (3.7.3).

$$G_f = 0.073 f_c^{0.18} \qquad (3.7.1)$$

$$G_{ch} = \left(\frac{f_c}{f_t}\right)^2 G_f \quad (3.7.2)$$

$$f_t = 0.3016(f_c)^{2/3} \quad (3.7.3)$$

G_f = fracture energy per unit area (N/mm)

F_t = tensile strength of concrete (MPa)

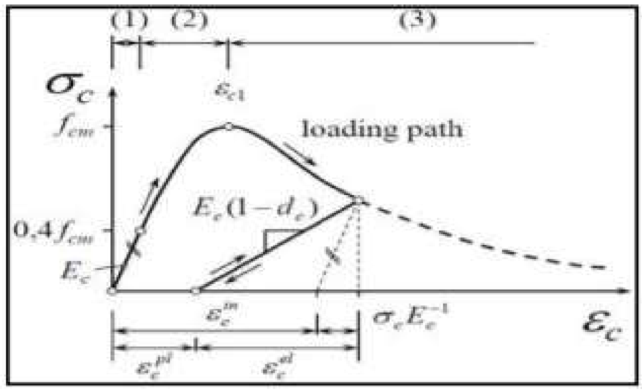


Figure 3.5 Three phase diagram of concrete

2. Concrete in tension

Relation between stress and crack width for concrete in tension was calculated based on Eq. (3.8) (Hordijk DA. 1992). The displacement method was selected in ABAQUS software to incorporate the input data in form of stress displacement variables.

$$\sigma_t = f_t\left[\left(1+\left(\frac{c_1 W}{W_c}\right)^3\right)e^{-c_2\frac{W}{W_c}} - \frac{W}{W_c}(1+c_1^3)e^{-c_2}\right] \quad \text{Eq. 3.8}$$

Fracture crack opening (W_c) = 0.3 mm

c_1 = 3 & c_2 = 6.93 (Hordijk DA, 1992)

Generally, W_c value assumed as per Eq.(3.9)(CEB-FIP, 2010) which ranges between 0.1 mm to 0.3 mm. A higher value of W_c = 0.3 mm has assumed in the Eq. (3.9) to include tension stiffening effect in the concrete material model. Therefore, Stress value does not reach to zero value corresponding to actual value of W_c. The stress-crack width curve is defined the plastic property of concrete material in tension as shown in fig.-3.4.

$$W_c = \frac{5.14G_f}{f_t} \quad \text{Eq. 3.9}$$

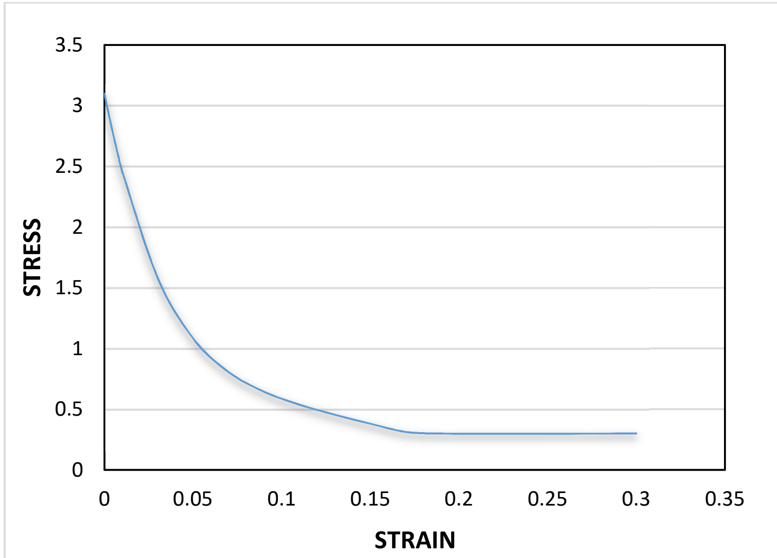


Figure 3.6 Stress-crack width curve of concrete in tension

3.5 DEFINING REINFORCEMENT IN ABAQUS

3.5.1 Introduction

Reinforcement can be modelled in two ways in Abaqus, it can either be made into a 3D solid element or a Wire element of a particular diameter as of the reinforcement bar. The only difference is the ease of analysis. Though taking the reinforcement as a solid 3D bar element is required the same can be made by the wire element where the analysis is much faster due to lesser computation of the surface stresses on the reinforcement. For this study the reinforcement bars have been modeled as a wire element with beam type sub-element category.

3.5.2 Bilinear law

The bilinear law, elastic-perfectly plastic, is assumed as shown in Figure 3.8

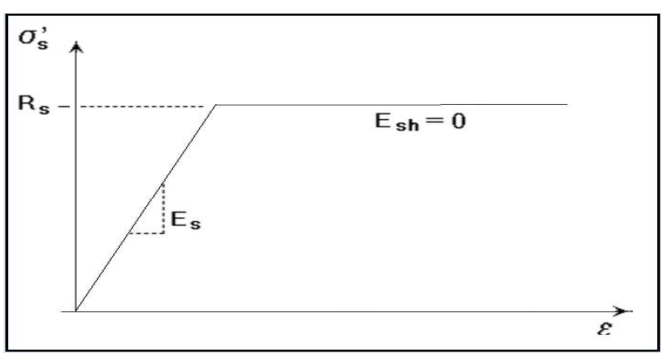


Figure 3.7 The bi-linear stress-strain law for reinforcement

The initial elastic part has the elastic modulus of steel E_s. The second line represents the plasticity of the steel with hardening and its slope is the hardening modulus E_{sh}. In case of perfect plasticity,

E_{sh} =0. Limit strain ε_L represents limited ductility of steel

3.5.3 Multi-linear law

The multi-linear a law consists of four lines as shown in Figure 3.9**.** This law aallows modelling of all four stages of steel behaviour: elastic state, yield plateau, hardening and fracture. The multi-line is defined by four points, which can be specified by input.

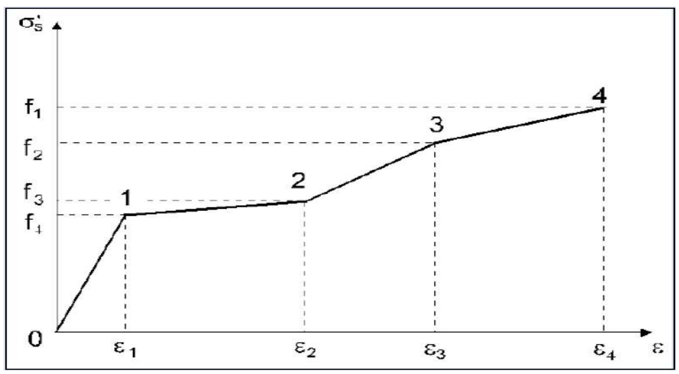


Figure 3.8 The multi-linear stress-strain law for reinforcement

where σ_s is the steel stress between the cracks (the steel stress in smeared reinforcement),

σ_{scr} is the steel stress in a crack. If no a tension stiffening is specified σ_{ts} = 0 and $\sigma_{scr} = \sigma_s$. In case of the discrete reinforcement the steel stress is always σ_s.

Once we understand the finite element modelling, the next step is the analytical programming. In the analytical programming, first we select the materials and its properties and create geometry of the beam column joint. For this, confined and unconfined RC beam column joint (T-joint) under monotonic and cyclic excitation are created and done the FE modelling through automatic FE mesh generator in **Abaqu**s. Same specimens were retrofitted with FRP (stressed to 50% and 75%). All these specimens were tested up to its failure values and in between graphs. For modelling the control and retrofitted specimens in **Abaqus**, concrete, reinforcement bars of different diameters, steel plates, epoxy and CFRP is used a material.

3.6 PHYSICAL MATERIAL PROPERTIES

Concrete, reinforcement steel, rigid plate, Epoxy and CFRP have been used to model the RC beam column joint. The specification and the properties of these materials are as under:

1) Concrete

In Abaqus, concrete material is modelled as a 3D nonlinear brick element. The physical properties of 3D nonlinear brick element material are given in Table 3.1. The values are calculated as per IS code 456:2000 and remaining are the default values.

Table 3.1 Material properties of concrete

Properties	Values
Elastic Modulus (0% and 50% stressed concrete)	20000 MPa
Elastic Modulus (70% stressed concrete)	16000 MPa
Elastic Modulus (85% stressed concrete)	14000 MPa
Elastic Modulus (90% stressed concrete)	10000 MPa
Poisson Ratio	0.2
Tensile Strength	3.130 MPa
Compressive Strength	20 MPa
Specific Material Weight	0.023 MN/mE+3

2) Reinforcement Bars

HYSD steel of grade Fe-415 of 10mm diameter is used as main steel for tension and 8 mm diameter is used in compression while 6mm diameter bars is used as shear reinforcement. The properties of these bars are shown in Table 3.2.

Table 3.2 Material properties of reinforcement

Properties	Values
Elastic modulus	200 GPa
Yield Strength	415 MPa
Specific weight of material	0.785MN/mE+3

3) Rigid plate

The function of the rigid plate in the Abaqus is for support and for loading. Here, the property of rigid plate is of a discrete rigid plate non-deformable.

4) Epoxy

The material properties are taken from the "Bansal and Kumar(2014)". The material properties of epoxy are shown in Table 3.3.

Table 3.3 Material properties of epoxy

Properties	Value
Compressive strength	60 MPa
Tensile Strength	25 MPa
Density	1 kg/m^2

5) Carbon Fiber Reinforcement Polymer

Carbon fiber reinforced polymer is a woven polymer of carbon fibers, that provides strength and stiffness. Carbon fibers provide highly directional properties i.e. they are anisotropic material with major strength properties aligned in one direction.

Table 3.4 Material properties of CFRP

Properties	Values
Tensile Strength	3800 MPa
Specific Material Weight	1.7 g/cm^3
Modulus of Elasticity	240 GPa
Thickness of each layer	0.117 mm

3.7 FE MODELLING OF RC BEAM COLUMN JOINT IN ABAQUS

Modeling in Abaqus, is very basic and easy as compared to other software's in the market for simulation purposes. This ease is due to various basic components included in the package that help in saving time and human hours, further the errors one can make are cut. The various steps involved in modeling using Abaqus are shown in figures below:

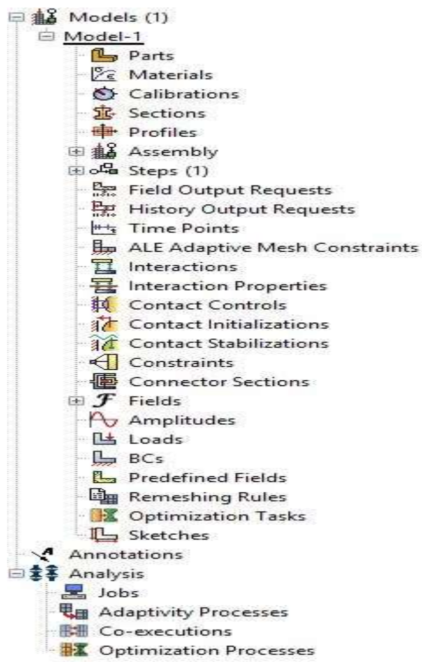


Figure 3.9 Steps involved in Abaqus modeling

STEP 1 Defining parts

Parts are the basic geometry of the model to be analyzed in Abaqus, the user needs to define the type of part as per its use in the analysis and the property it has.

Various Types of parts can be defined such as deformable solid or rigid shell, used for concrete and loading plates respectively, further for reinforcement deformable wire type element can be used. In the below figures the definition of the part for Concrete, steel reinforcement, CFRP and rigid plates have been shown:

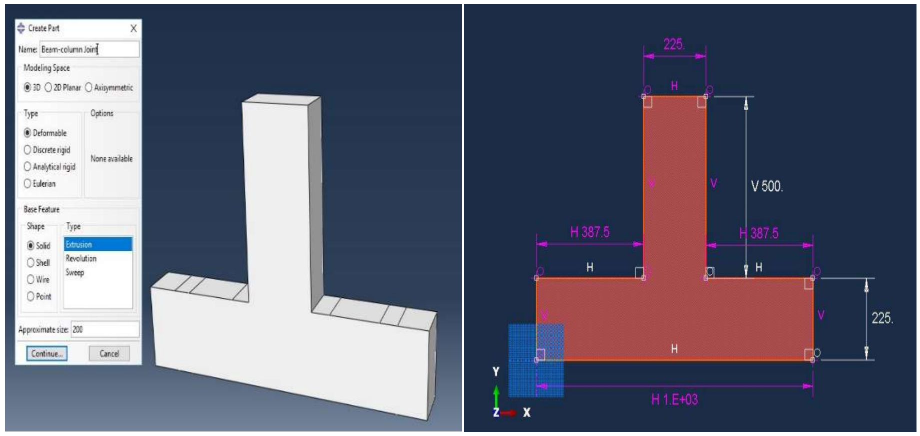


Figure 3.10 Part definition for concrete elements

Figure 3.11 Part definition for reinforcement elements

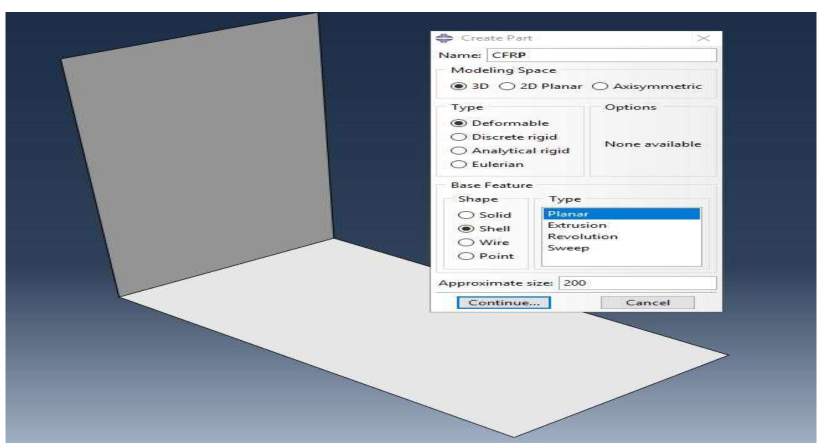


Figure 3.12 Part definition for CFRP element

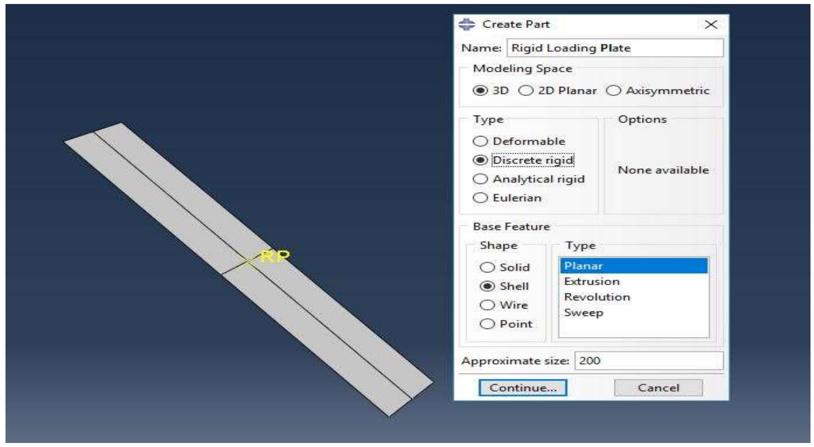


Figure 3.13 Part definition for rigid elements

Figure 3.14 All parts defined for the model

STEP 2 Defining material properties

As defined earlier the input as material properties for Concrete, Steel, CFRP have been defined in Abaqus, as can be seen in the figures given below:

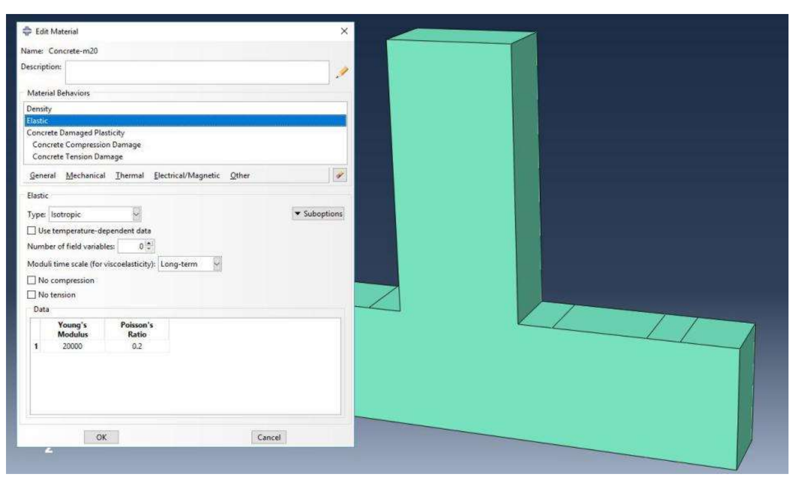


Figure 3.15 Material property for concrete

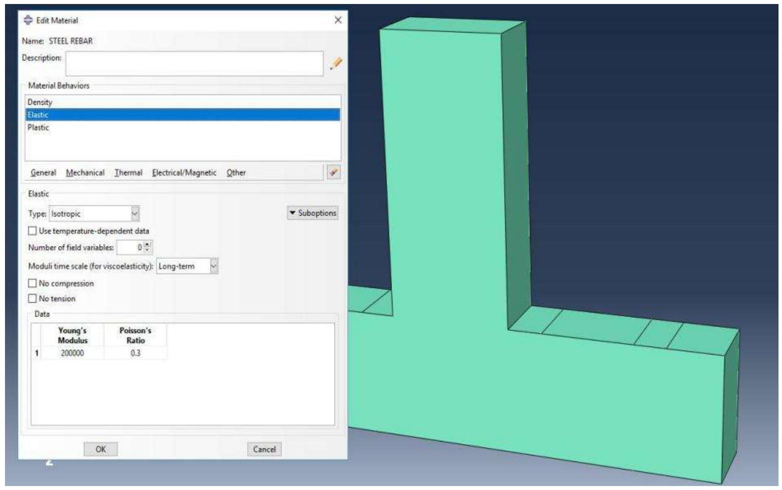


Figure 3.16 Material property for main reinforcement

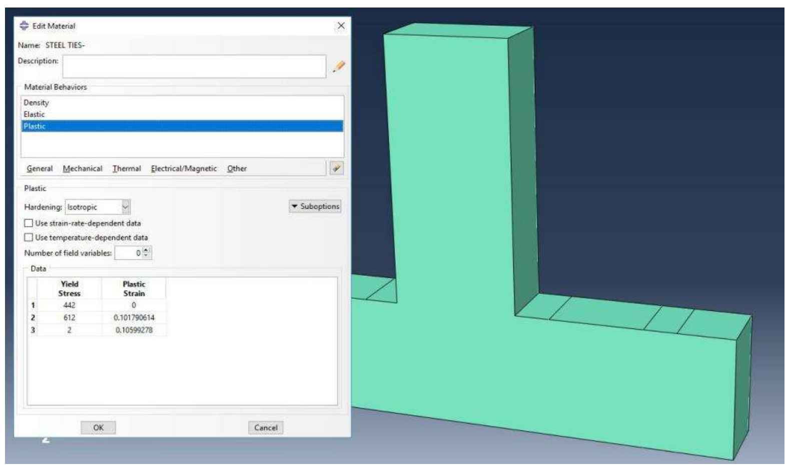


Figure 3.17 Material property for secondary reinforcement (Ties and Stirrups)

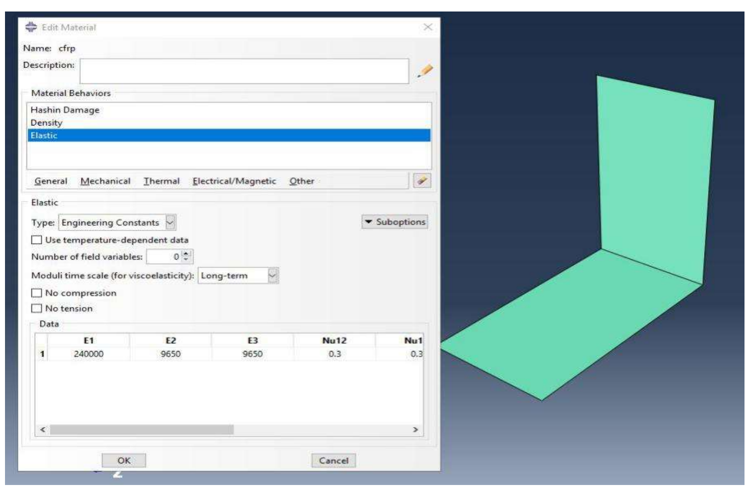


Figure 3.18 Material property for CFRP

STEP 3 Creating sections and assigning sectional properties

In this step the material properties are assigned to the parts created in Step 1.The figure below shows the assignment of concrete to the Beam-Column Joint Part created in Step 1.

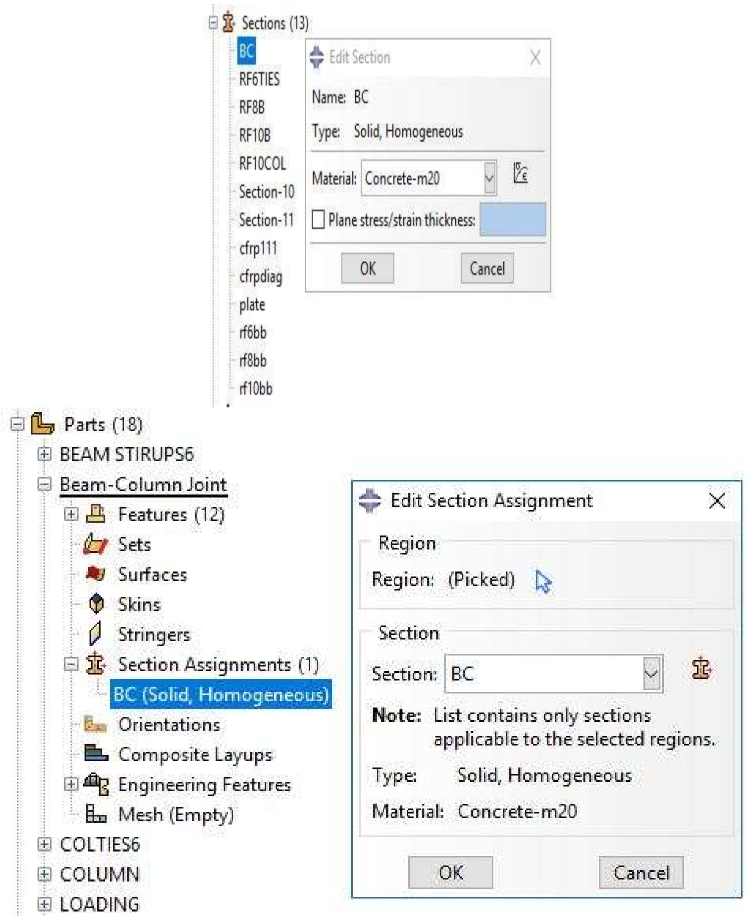


Figure 3.19 Creating a section and assigning it to the part

STEP 4 Assigning orientation of CFRP layers

Orientation of the CFRP fibers in used in retrofitted specimens is important because CFRP is an orthotropic material i.e. the strength properties are aligned all along one direction only. In this study this direction is E1. In the below figures, orientation of various components of CFRP have been shown.

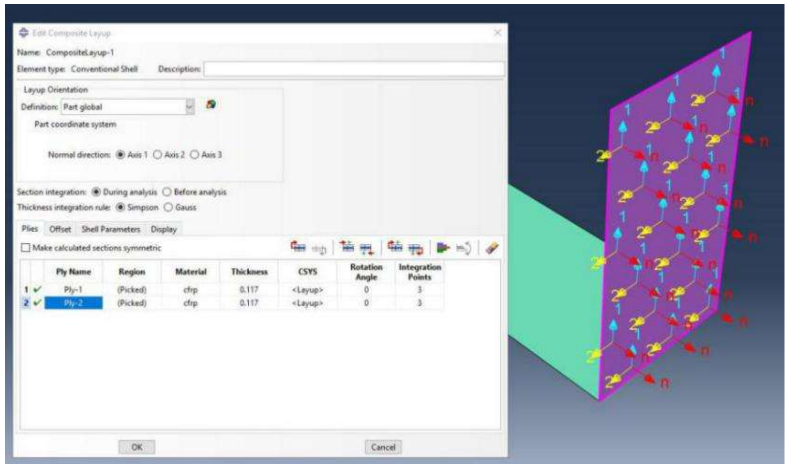


Figure 3.20 Orientation of L-shaped layer-1

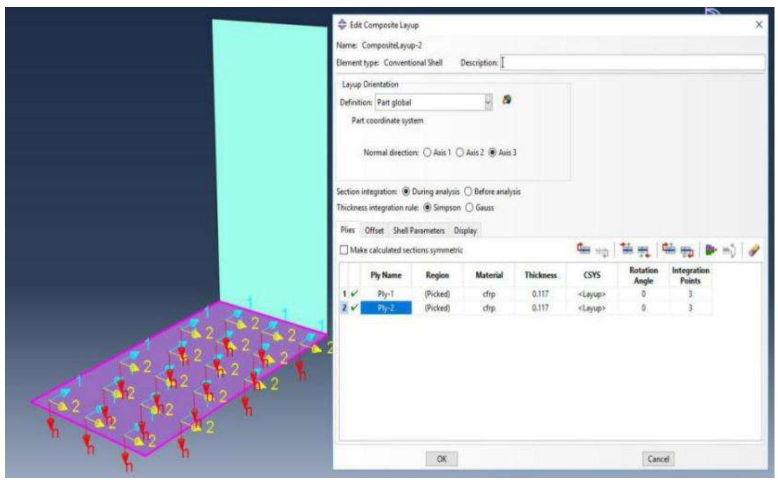


Figure 3.21 Orientation of L-shaped layer-2

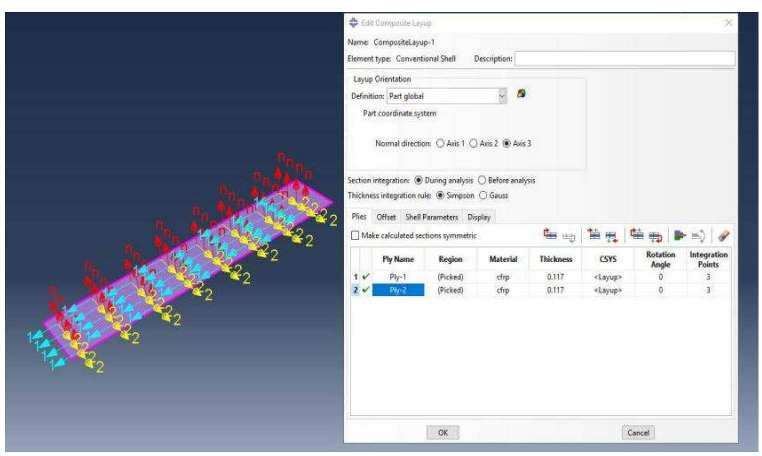


Figure 3.22 Orientation of column retrofitting layer

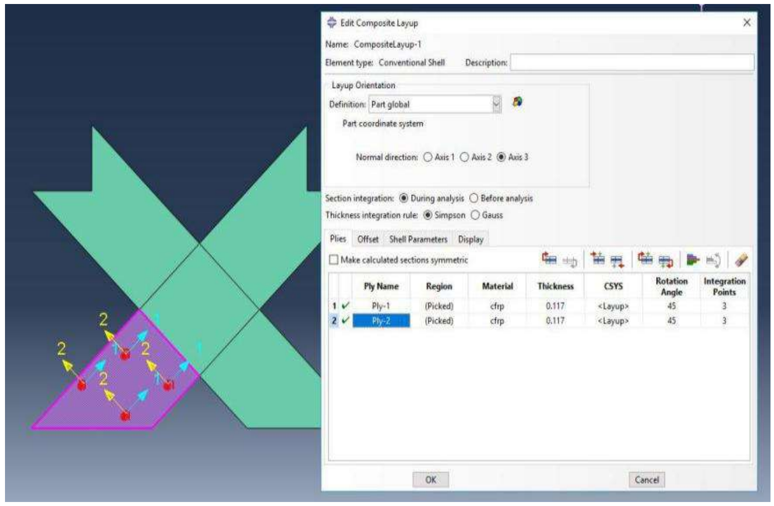


Figure 3.23 Orientation of X-shaped layer-1

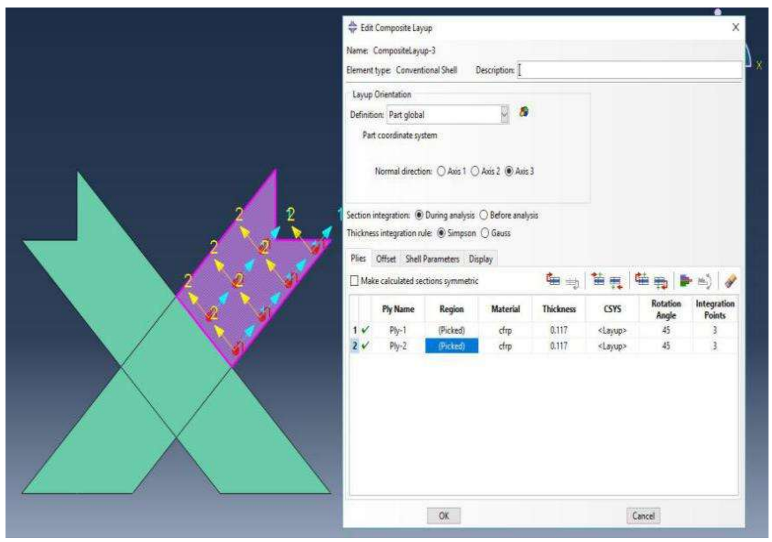


Figure 3.24 Orientation of X-shaped layer-2

STEP 5 Assembly of all parts

In this step, all the Parts created in Step 1 are assembled together to work together as expected in real life. This is the most important part of the whole analysis process as the results can vary a lot due to a small change in assembly from the experiment. The figures below show the assembly process.

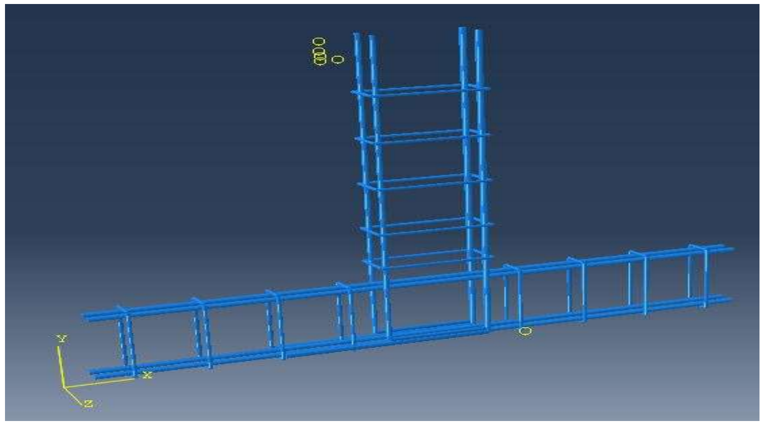


Figure 3.25 Assembly of reinforcement

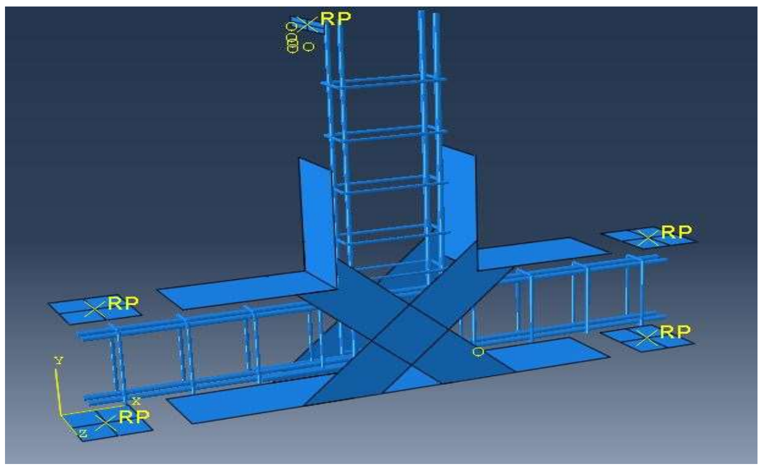


Figure 3.26 Assembly of reinforcement, rigid plates and CFRP for retrofitted samples

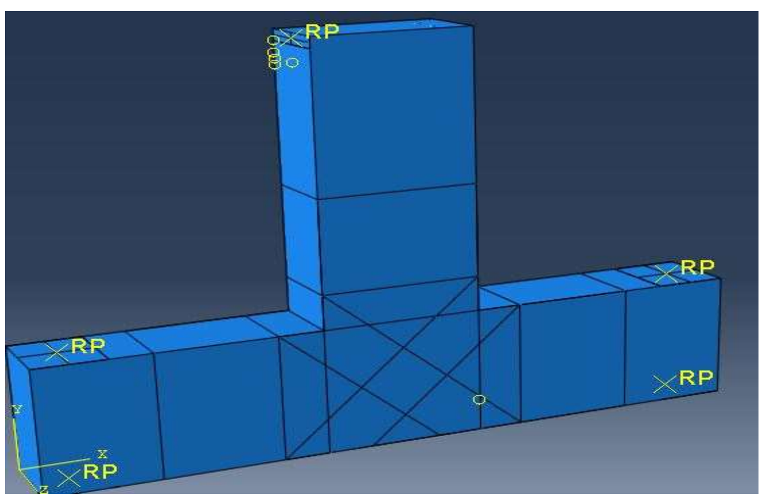


Figure 3.27 Assembly of beam column joint

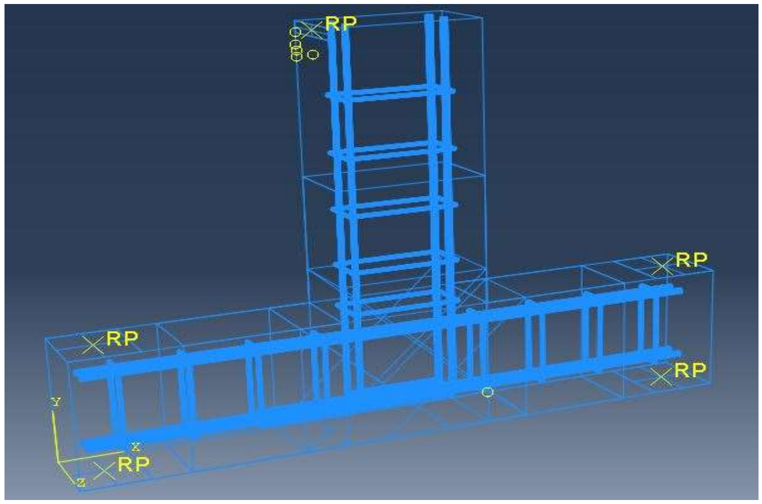


Figure 3.28 Final assembly of all parts

STEP 6 Defining interactions and boundary conditions

This step involves defining interaction between concrete and steel, concrete steel and CFRP layers (for retrofitted samples), concrete and rigid plates. The interaction is set to embeded for the concrete and steel, while a cohesive surface is defined on the concrete surface for attaching CFRP to the surface. Further a tie constraint is defined for the interaction between the rigid plate and Concrete surface.

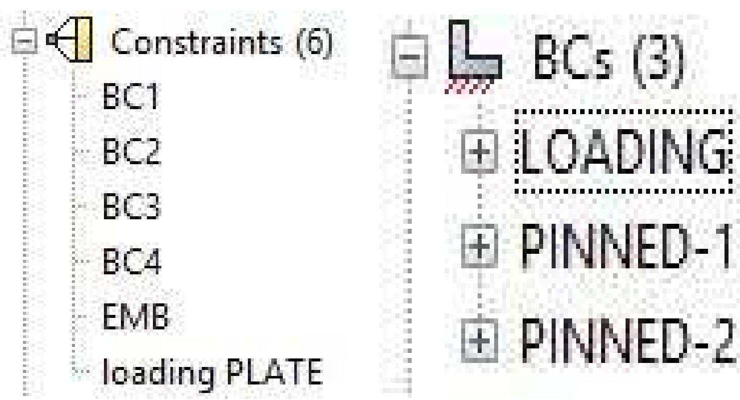


Figure 3.29 Assigning interactions and BC'S

STEP 7 Creating a JOB and submitting it for analysis

The last step is to create a Job for the analysis process and submit it for analysis by Abaqus.

4.0 GENERAL

In this study, non-linear behavior under static loading for a reinforced concrete (RC) beam- column joint(BCJ) detailed as per the provisions of IS456 (BIS_2000) has been studied. The beam-column joint under consideration is described in Chapter 3. The objective of this study is to numerically evaluate the experimental program carried out by Bansal and Kumar (2014). This has been done in terms of variation of load-displacement relation, the initiation of cracks in the joint, and propagation of the damage in the beam-column joint subjected to loading.

Non-linear analysis of the RC beam-column joint under monotonic loading is done using the finite element (FE)program Abaqus. The performance of beam-column joint is assessed in terms of its load deflection and cracking behavior obtained from the analysis. The beam-column specimen after attaining different damage levels are retrofitted using CFRP layers and loaded again to ultimate capacity.

In the analysis, load has been applied at the free end of the beam and corresponding to load values, its deformation is obtained. The deflection values at every increment of load have been recorded, further the crack pattern and its propagation at every step has been studied for the basic beam column joint (control specimen) and the retrofitted specimens. The results of the numerical study with relation to the conducted experiment is studied and discussed in detail in the chapter.

4.1 NONLINEAR ANALYSIS OF THE CONTROL BEAM COLUMN JOINT

4.1.1 Load-deflection behavior

Initially the control beam - column joint (Bansal and Kumar (2014)) is analysed for to monotonic loading which is applied at the free end of the beam as per the details of the experimental program. The numerical study of the considered beam – column joint is done using the FE based software, Abaqus. The control specimen taken as per the experimental study of Bansal and Kumar (2014), modeled in ABAQUS and then subjected to the displacement type loading till ultimate load was attained. Further the Load-deflection curves were plotted for the same and compared to the experimental study. After attained a good acceptance between the two studies the control specimen acted as a base model for the further studies in this study.

Load deflection curve for the control sample has been plotted in Figure 4.1. It can be observed from Figure 4.1 that the model of beam column joint on loading behaved linearly u to the load value of 6.6 kN at 0.2 mm displacement at this stage the damage at the joint region has just started, and then the model starts behaving non-linearly till the ultimate load of 24.38 kN is attained at a displacement of 22.48 mm. After this stage, there is a small amount of decrease in the load with increase in displacement and the Abaqus aborts after that so this point can be taken as point of extreme damage in the model.

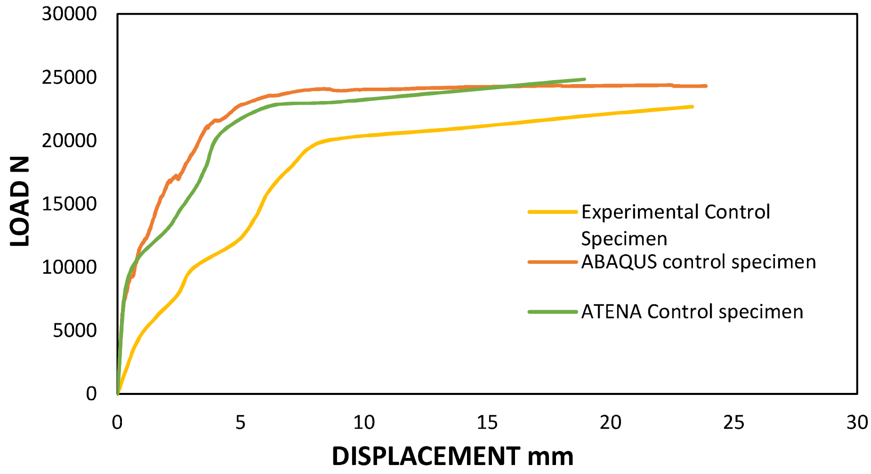


Figure 4.1. Comparison of load-displacement from numerical and experimental study for the control beam-column joint.

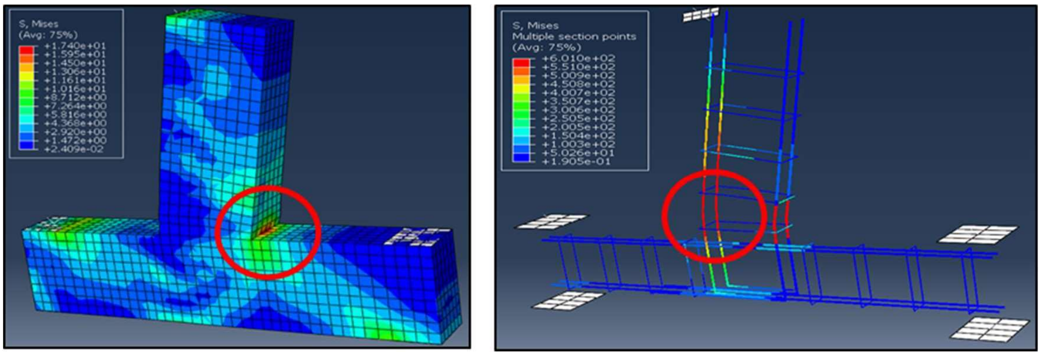
A general comparison can be drawn between the control specimen results from various sources of analysis and experimental. The numerical analysis conducted by (Varinder and Bansal (2014)) in ATENA and by Abaqus are compared together, so as to get the behaviour of the beam-column joint specimen. Figure 4.1 shows the load vs displacement plot of the control specimen. It is observed that the experimental load-deflection curve for the control specimen, shows more or less similar ultimate load carrying capacity for the beam column joint with with a maximum deviation of 7.8% in the FE results. The certain deviation in the results can be attributed to: 1) Material definition in Abaqus which is not exact replica as in experimental procedure. 2)The experimental boundary conditions(BC's) i.e. the both ends of column being pinned cannot be perfectly pinned experimentally due to laboratory constraints, 3) Last but not least, can be the human error that may exist during the experimental program. The numerical load deflection curve obtained for the control specimen shows comparatively more strength and initial stiffness in the beam-column joint as compared to the experimental results for the control sample. The high initial stiffness in the numerical model can be attributed to the fact that in numerical modelling every element of the model has a similar property and thus, the whole model contributes towards the stiffness. In contrast material, non-uniformity exists throughout the section in experimental samples, thus large uncertainty occurs in the material properties are expected in the structure.

The ultimate load for the BCJ under experimental study is about 22.6 kN at a displacement of 23.3 mm while in the numerical analysis it is found to be 24.38 kN at a displacement of about 22.48 mm respectively (Figure 4.1). Thus, an overall deviation of about 7.8% in the ultimate load from the numerical analysis is observed, which is very less. Though the deviation in ultimate load is quite less, it is also understood that FEM based models are always believed to show conservative type of results. The initial behavior of the BCJ obtained from numerical analysis differs from the experimental study considerably. which can again be attributed to the high initial stiffness of the material models considered in the FE model.

To gain more insight into the modelling of the beam – column joint, comparison is also done with the numerical results obtained from ATENA (Varinder and Bansal (2014)). Both the load – displacement plots from the numerical modelling in Abaqus as well as Atena show a good tally. The ultimate load for control specimen is achieved at a displacement of 18.5 mm of about 24.85 kN. Whereas the ultimate load for the control specimen is of 24.34 kN at a displacement of about 23 mm. Thus, there is almost no deviation in ultimate loads obtained. Further, the initial elastic behavior is also almost comparable. Therefore, the control BCJ model as modelled in Atena (Varinder and Bansal (2014)) is chosen as the basic model, which is further used in the study for retrofitted BCJ specimens.

4.1.2 Stress resultant in concrete and steel

Stress resultants are simplified representations of the stress state in structural elements such as beams, plates, or shells. The von Mises stress is not a real stress. It is a stress convention, that provides a measure of the shear, or distortional stress in the material. The input given for defining the material concrete and steel can be seen to correlate with the visual stress levels given by Abaqus after analysis. The Figure 4.2 show the stresses (Von Mises) in concrete and steel, respectively at an ultimate load of 24.38 kN. Further, the location of maximum stress (indicated by the circles in the figure) can be located to be near the joint region for both the materials of the BCJ, the maximum stress reaches to 17.4 N/mm^2 in concrete Figure 4.2(a) and for reinforcing steel the maximum stress of 601 N/mm^2 is observed Figure 4.2(b), for ultimate load.



(a) **(b)**

Figure 4.2. Von Mises stresses in the beam-column joint obtained from FE analysis (a) Concrete, (b) Reinforcing steel

4.1.3 Damage flow patterns

The damage in the model is predicted by the software based on the relationship between strain and damage parameter for input values given in the material definition of concrete i.e. given in CDP model in Chapter 3, that ranges between 0 to 1, where 0 represents undamaged state of the element and 1 predicts a fully damaged element. The damage due to tension and compression in concrete for the beam-column joint at and ultimate load of 24.38 kN. It can be clearly observed that the damage in tension is more than the damage in compression, since concrete is strong in compression and weak in tension.it is observed that the joint region of the considered structure shows the location of maximum damage, and thus, it would be beneficial to retrofit the area near the joint of the damaged structure, so as to restore or even increase the load carrying capacity/strength of the beam column joint assembly.

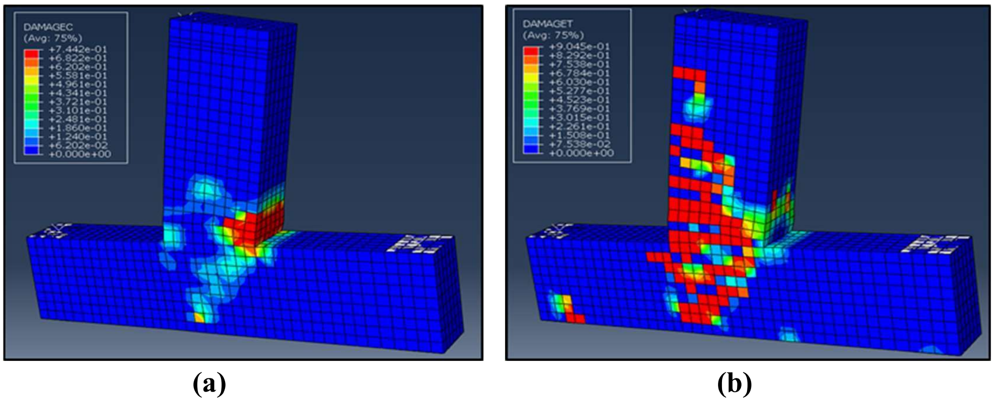


Figure 4.3. Damage in the concrete of the beam column joint due to (a)compression, (b) tension obtained from the numerical analysis

4.1.4 Stiffness degradation

Abaqus gives a unique visualization for the stiffness degradation in the model. The elements that are stressed to maximum level and do not contribute significantly to the stiffness of the structure assembly can be located. Figure 4.4 shows the stiffness degradation for the elements of the assembly

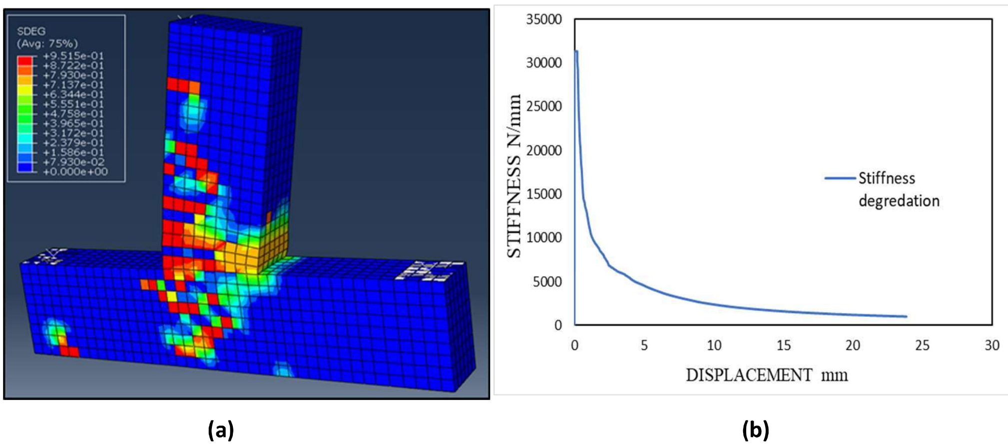
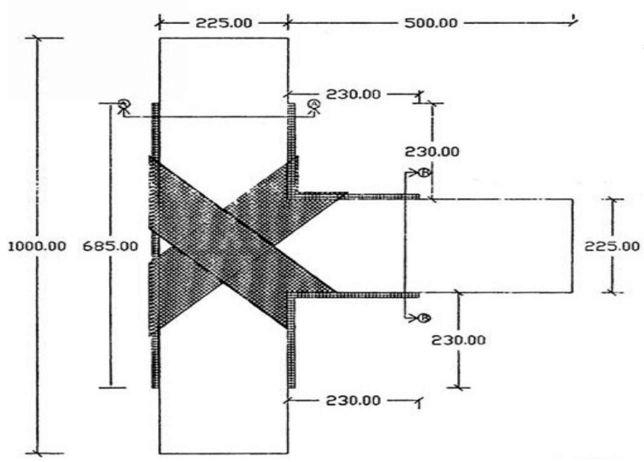


Figure 4.4. Stiffness Degradation in the beam column joint under nonlinear static loading (a) in the numerical model (b) with respect to displacement

As can be taken from the figures (4.2 to 4.4) above, the stiffness degradation takes place mostly in the joint region due to cracking and crushing of the concrete in the joint region. Therefore, again the location for the retrofitting of the Beam-Column joint is to be the joint region of the structural element assembly.

4.2 NONLINEAR ANALYSIS OF THE RETROFITTED BEAM COLUMN JOINT WITH CFRP

As can be evaluated from the Figures (4.2 to 4.4) for damage in tension, compression and overall stiffness degradation of the system, the joint region of the beam column joint is most damaged area up on loading. Further this area is not only considered crucial for the monotonic loading but also for the cyclic loading acting on the structure. There has been a wide research by many prominent researchers around the world on various techniques for retrofitting of beam column joint to regain or increase the ultimate load carrying capacity and repair the damage occurred in the assembled structural elements. This study deals with an attempt to model the retrofitting of the beam-column joint in a FEM based software Abaqus with CFRP (discussed in chapter 1) material. Experimental study on the retrofitting of the beam-column joint using CFRP by Bansal and Kumar (2014) is considered in the present study. The initial control specimen is loaded up to ultimate load and subsequent retrofitting is done using two layers of the CFRP material (Figure 4.5). Two layers of CFRP are bonded to the surface of the concrete using a layer of cohesive epoxy material. Three different damage levels of the control specimen were considered after which the retrofitting is done: 50%, 85% and 100% of the ultimate load carrying capacity of the control specimen. The pattern for adhering the CFRP on the joint specimen has been evaluated based on the most damaged region in the assembly as discussed earlier.



(All dimensions are in mm)

Figure 4.5. CFRP retrofitting of the beam-column joint (Bansal and Kumar 2014)

4.2.1 Numerical implementation in Abaqus

The present study adopts two different modelling approach for the retrofitting of the beam column joint, each has their own limitations. The first way of modelling retrofitting of damaged specimens is the method followed by most of the researchers i.e. changing the material properties of the control specimen to account for degradation in the strength of the properties of the specimen after failure. Though this method is iterative and the user needs the already formulated results to match the extend of decrease of the material properties, required to match the results with the experimental results. Which in turn takes away all the purpose of doing an analysis numerically i.e. to save time, material, and cost. Another way of modelling in Abaqus is using model change option available in the Abaqus, this option is quite easy, flexible and is a two-step process. Basically, the geometry of the retrofitting material is suppressed in the step 1 of the analysis, till the control beam is damaged up to the ultimate load carrying capacity of the assembly and then in step 2 the retrofitting assembly is activated and allowed to take stresses for further loading in this step which is quite what is expected to happen in real life. Though the limitation of this step is that the Abaqus software, does not accept the adhesion of CFRP to the damaged surface created on the control specimen due to step 1 loading. This problem could be solved by not giving the damage parameters in the material definition which in this case is concrete but that takes away all the initial stiffness degradation from the assembly and the user is unable to know how the stiffness of the system decreased and further is not able to get the damage visualization for the assembly. This has been explained in detail in the coming parts of this study.

4.2.1.1 *Retrofitting using model change approach*

As discussed in section 4.6.1, the model change method for modeling the retrofitting of any structural assembly is a realistic approach towards the problem**.** Although it difficult to capture the degradation in initial stiffness of the assembly.

But it's a non-iterative process and makes the use of the basic model once fixed for the analysis of the control specimen. Further, it saves time and effort of the user while giving the considerable ultimate load carrying capacity for the retrofitted sample. The various steps involved in defining the model change procedure for retrofitting in ABAQUS are discussed below:

STEP 1 Fixing parameters for the control specimen

This step involves fixing the parameters involved in the modeling of the control specimen (material properties, pinned end BC's etc.), further in this step the CFRP element is kept attached to the control specimen using a cohesive layer.

STEP 2 Defining step's in the analysis process

This step involves the defining of the steps in which the analysis will be carried throughout the time interval. So how this works is that the user defines two analysis steps for the whole model, the first analysis step is defined such that the specimen is loaded up to the ultimate load carrying capacity of the control specimen, with the only difference in the interaction behavior of the control sample with the CFRP attached layers. In the interaction property for the first step the model change interaction is activated with a condition that the CFRP layers get suppressed or deactivated in the first step of the analysis. Thus, no contribution is given by the CFRP layers in the first step of the analysis process and the control specimen behavior is obtained for the system. Further, the model is stressed to the requires level with damages in both concrete and reinforcing steel.

The second step is defined like the step 1 of the analysis process, the difference being in the model change interaction. In second step of the analysis process the CFRP layers are activated and the interaction with concrete using a cohesive layer is defined for a given time and this arrangement is again loaded up to the ultimate load carrying capacity of this system of assembly is achieved. Which gives us the ultimate load carrying capacity of the retrofitted specimen.

STEP 3 Creating and submitting job for the analysis process

This step involves the final creation of the job for the analysis process and submitting the same for the analysis to proceed.

After the completion of the analysis for the assembly, the load vs displacement results for the retrofitted samples are obtained for various levels of damage on the control specimen and the results are compared with that of the experimental results of the retrofitted samples. Figure 4.6 below shows retrofitted model of the beam- column joint in ABAQUS.

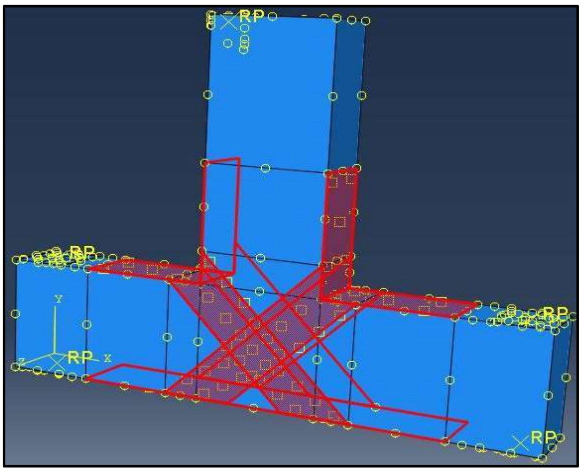


Figure 4.6. Retrofitted model of the beam column joint with CFRP layers at the joint region

4.2.1.2 *Nonlinear load - displacement behavior using model change approach*

Figure 4.7, shows the comparison of force displacement for the considered beam column joint retrofitted after attainment of 100% damage level using the model change technique with that of experimental study. The Model change based retrofitting method in Abaqus can predict only the ultimate load with minor variations when compared to the experimental data. The initial stiffness degradation of the specimen could not be traced with the considered modelling strategy. This is because the damage parameters for the main material concrete couldn't be used in the analysis process, a detailed reason for which is discussed in section 4.6.1. Therefore, it can be said that only the ultimate load behavior of the retrofitted specimen can be calculated by the model change method used for modelling CFRP retrofit in Abaqus. However, the initial behavior of the tested BCJ specimen is significance, to understand the effect of retrofitting on the damaged control specimen. Hence, the model change method of modelling the retrofit is discarded for the further analysis of the joint. It is noteworthy to mention that, this method of analysis is relatively easy, the computational time required is less.

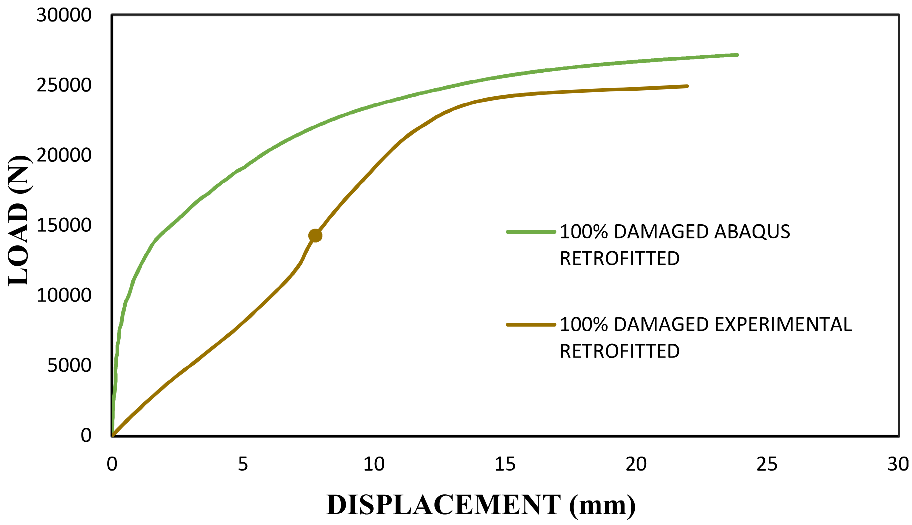


Figure 4.7. Comparison of force – displacement relation for 100% damage level obtained using experimental and numerical methods

A general comparison of all the retrofitted models of the control sample for various levels of damage retrofitted experimentally (Bansal and Kumar 2014) and numerically in ABAQUS is shown in figure 4.8. The convergence of the retrofitted samples is close to each other for the ultimate loads in both the cases of retrofitting. Though no initial loss of stiffness can be seen in the numerical behavior of the retrofitted samples for any of the damage level which has been discussed in detail in section 4.6.1.

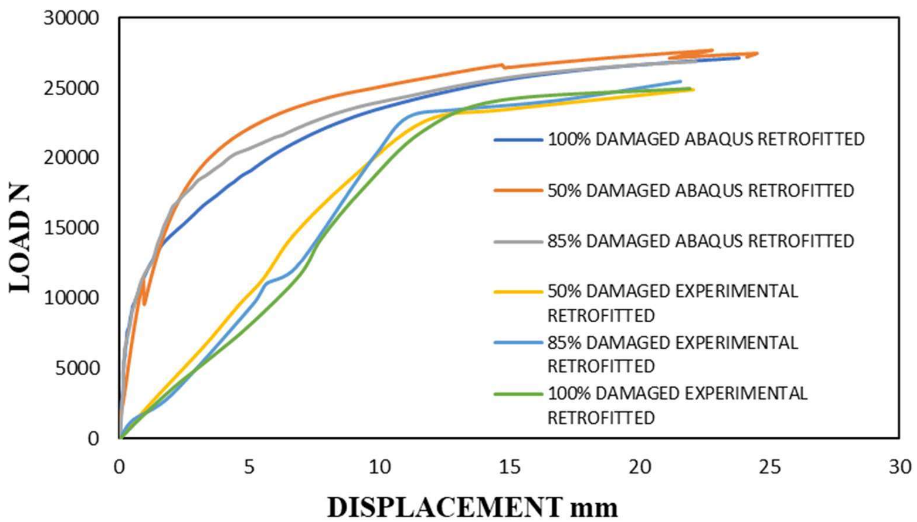


Figure 4.8. Comparison of force – displacement relation for different damage level obtained using experimental and numerical methods

Table 4.1 Comparative experimental and numerical load-displacement data for retrofitted BCJ samples for various levels of damage and standard deviation.

Damage level (%)	Ultimate load (kN)		Standard deviation (%)
	Experimental	Numerical (model change)	
50	24.8	27.3	+9
85	25.48	26.8	+5
100	24.9	27	+7.4

4.2.1.3 *Retrofitting by changing the material properties*

As discussed in section 4.2.1, this is widely used method of modelling a damaged specimen and its retrofitting in numerical analysis. This method involves changing the material properties of the original control specimen to replicate the damaged section properties and then retrofitted with the retrofitting material which in this study is the CFRP. The basic idea is to decrease the strength of the materials used to define a chosen damaged state. This is achieved by decreasing the compressive strength of concrete used, which in turn leads to a change in the elastic modulus of the concrete. Also, the yield and ultimate stress level in the steel is reduced to represent the decreased strength in the assembly due to damage in it. Table 4.2 shows the various change in the material properties (both concrete and steel) obtained to validate with the experimental outcomes. This involves a lot of iterations to match the behavior of the numerical model of the retrofitted damaged section with the experimentally specimen.

The way this method of analysis has been used in this study, is that an approximate decrement in the properties of the material is taken from the results of the experimentally performed retrofitting, and then the material properties of the numerical control specimen are decreased by this approximate amount to account for the damage, which is then retrofitted using CFRP. Although, it is an iterative technique, it shows a considerable match not only with the ultimate load carrying capacity but also with the initial elastic behavior of the assembly. Further, it directly gives an idea about the amount of decrease in the values of the material properties when loaded to different amounts of damage.

Although experimentally the two layers of CFRP has been effectively used for the retrofitting of the damaged beam column joint, the present study also investigates the effectiveness of a single layer of CFRP for the retrofitting of the damaged specimen. However, the bonding between the concrete and CFRP wrap is considered to be intact throughout the loading process in both single and double layer, i.e., there was no loss of bond between the materials. Various results obtained by this modelling strategy for the retrofitting of the damaged beam column joint at different damage levels, and their comparison with the experimental study are discussed in the next section

Table 4.2 Change in the material properties for various levels of damage

Damage level	**Modulus of elasticity (*Ec*)**		**Compressive strength (*fck*)**		**Yield tensile strength of steel (*fy*)**	
%	**MPa**	**% reduction**	**MPa**	**% reduction**	**MPa**	**/ % reduction**
0	200000	0	20	0	500	0
50	20000	0	18	10	500	0
60	18000	10	16	20	410	18
70	16000	20	14	30	350	30
85	14000	30	12	40	250	50
100	10000	50	10	50	250	50

4.2.1.4 *Nonlinear load - displacement behavior by change in material properties*

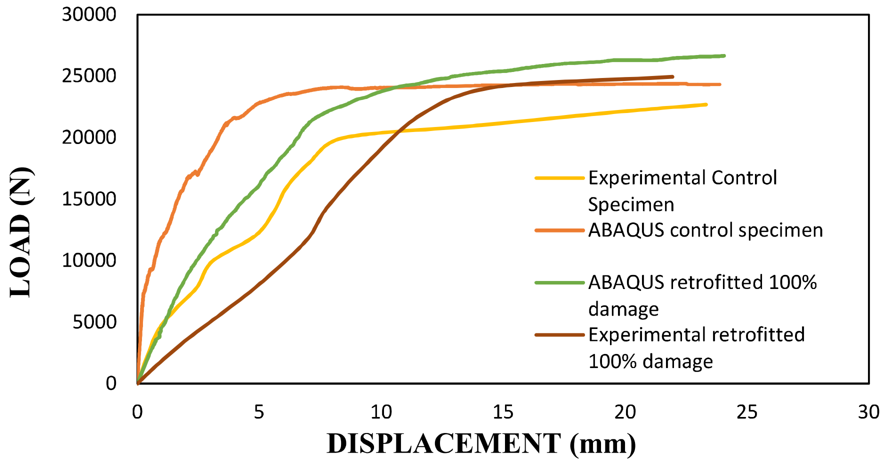


Figure 4.9. Comparison of experimental and numerical load – displacement behavior of control and retrofitted BCJ with 2 layers of CFRP for 100% damage.

A general comparison of numerical control BCJ and all the retrofitted models of the control sample for various levels of damage retrofitted numerically in Abaqus are shown in Figure 4.10, the convergence of the retrofitted samples is close to each other for the ultimate loads. Further, initial loss of stiffness can be seen in the numerical behavior of the retrofitted samples for all the damage levels.

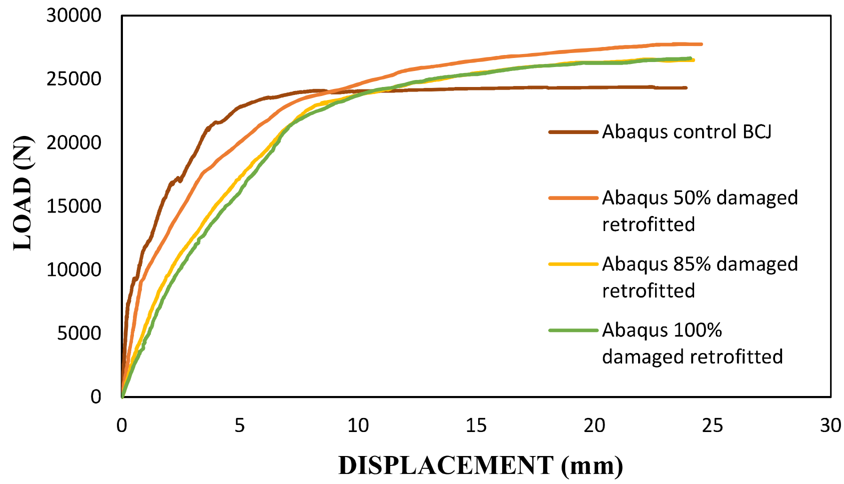


Figure 4.10. Comparison of numerical load – displacement behavior of control and retrofitted BCJ with 2-layers of CFRP for different damage levels.

For all the damage levels (50%, 85% and 100%) the material properties assumed for the analysis are given in Table 4.2. Figure 4.10, shows the load vs displacement plot for the all retrofitted sample with different damage levels. Further, the comparison is concluded with the results obtained from the numerical analysis for the control BCJ. Retrofitting of damaged BCJ with 2 layers of CFRP efficiently restores the strength of the damaged specimens and goes beyond that of a control BCJ. Table 4.3 gives a comparison for the ultimate load capacity of all the retrofitted models with the numerical control specimen of BCJ.

Table 4.3 Comparative numerical load-displacement data for control and 2-layer CFRP retrofitted BCJ samples for various levels of damage and standard deviation.

Damage level (%)	Ultimate load (kN) for the BCJ		Standard deviation (%)
	Control	Retrofitted 2-layer	
50	24.3	27.7	+12.27
85		26.7	+8.99
100		25.5	+4.7

As can be concluded from the results in Table 4.3, numerical retrofitting of the damaged BCJ shows positive results of standard deviation for ultimate load carrying capacity, when compared to the numerical control BCJ. Which in turn, shows the efficiency of the wrapping for various damage levels in the BCJ corresponding to the ultimate load of the control specimen. Therefore, it can be concluded that the numerically retrofitted samples with 2 layers of CFRP, for 50%, 85% and 100% damage levels show +12.27%, +8.99% and +4.7% increase respectively for ultimate load than control specimen.

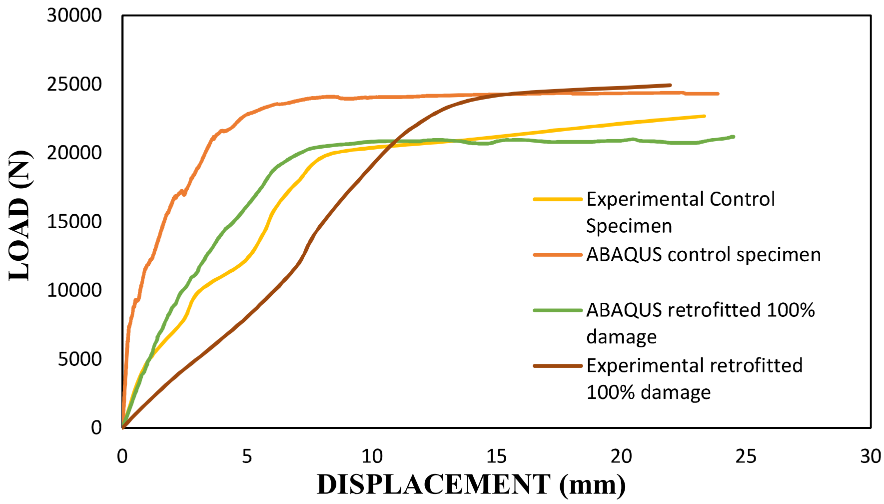


Figure 4.11. Comparison of experimental and numerical load–displacement behavior of control and retrofitted BCJ with 1-layer of CFRP for 100% damage.

A general comparison of numerical control BCJ and all the retrofitted models of the control sample for various levels of damage retrofitted numerically in Abaqus are shown in Figure 4.12, the convergence of the retrofitted samples is close to each other for the ultimate loads. Further, initial loss of stiffness can be seen in the numerical behavior of the retrofitted samples for all the damage levels.

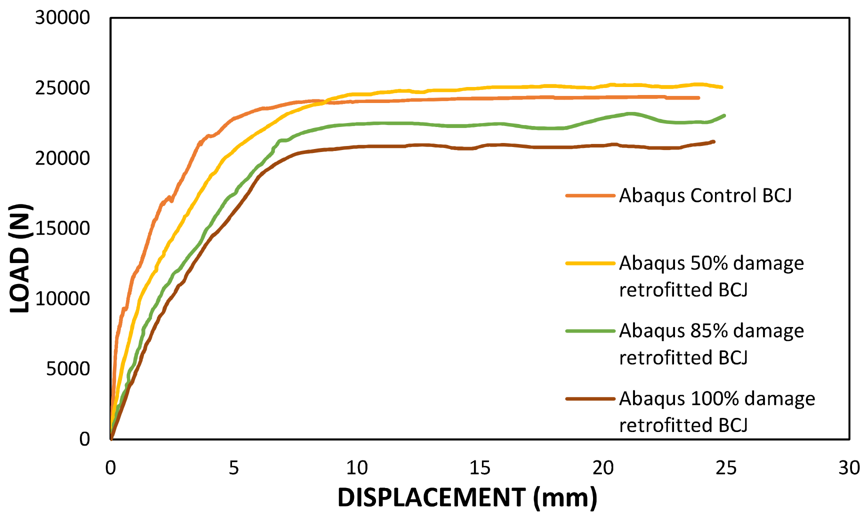


Figure 4.12. Comparison of numerical load – displacement behavior of control and retrofitted BCJ with 1-layer of CFRP for different damage levels.

Table 4.4 Comparative numerical load-displacement data for control and 1 layer CFRP retrofitted BCJ samples for various levels of damage and standard deviation.

Damage level (%)	Numerical Ultimate load (kN)		Standard deviation (%)
	Control BCJ	Retrofitted 1 layer CFRP BCJ	
50	24.3	25.2	+1.58
85	24.3	23.18	-9.57
100	24.3	20.9	-19.2

As can be concluded from the results in Table 4.4, numerical retrofitting of the damaged BCJ shows positive results of standard deviation for ultimate load carrying capacity up to a damage level of 50% i.e. +1.58% when compared to the numerical control BCJ. Which in turn, shows that retrofitting of 50% damaged BCJ, with 1 layer of CFRP is efficient enough to restore the full strength of the damaged BCJ up to the numerical control BCJ. Although, after that for higher levels of damage to the control specimen 1 layer of CFRP retrofitting is not efficient enough to restore the ultimate strength of the damaged specimens up to the control specimen mark. There is a difference of -9.57% and -19.2% in ultimate load of retrofitted samples from control BCJ for damage levels of 85% and 100% respectively.

Further, a comparison is drawn between the use of 2 layers and 1 layer of CFRP, for retrofitting of the damaged BCJ. For this study the damage levels considered are 50%, 60%, 70%, 85% and 100%.

Table 4.5 Comparison of effectiveness of numerical 2 and 1-layer retrofitting of damaged BCJ

Damage Level	Ultimate Load		Effectiveness of 2 layer CFRP model from 1 layer CFRP model
	FOR 2 layer	FOR 1 layer	
%	kN	kN	%
50	27.8	25.06	9.85
60	27.367	24.7	9.74
70	27.278	24.14	11.5
85	26.77	23.28	13.037
100	26.61	21.18	20.41

The important point to notice here is that as the damage level increases in the control specimen the more effectiveness of having more layers of the retrofitting material is observed as can be seen for the 100%(complete damage) from the Table 4.5. Though this change in the behavior is only for the ultimate load carrying capacity obtained for the damaged samples, the initial behavior of both the samples i.e. one retrofitted with 2 layers of CFRP and other with 1 layer of CFRP retrofitting is same.

4.2.1.5 *Damage flow pattern of the retrofitted specimens using 1 and 2 layers of CFRP*

Orientation of the CFRP fibers, used in retrofitted specimens is important because CFRP is an orthotropic material i.e. the strength properties are aligned all along one direction only. In this study this direction is axis one along which the major strength of the material is assumed to be, based on the effectiveness in restoring the strength of the damaged specimen this orientation of the fibers of CFRP is considered. The same can be seen in Figure 4.13.

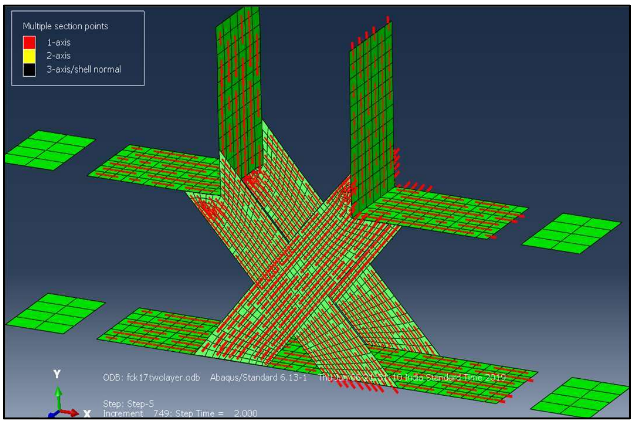
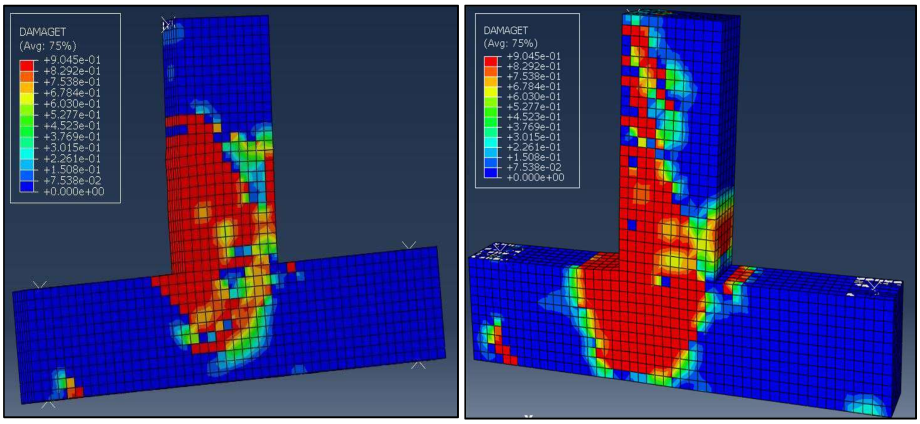


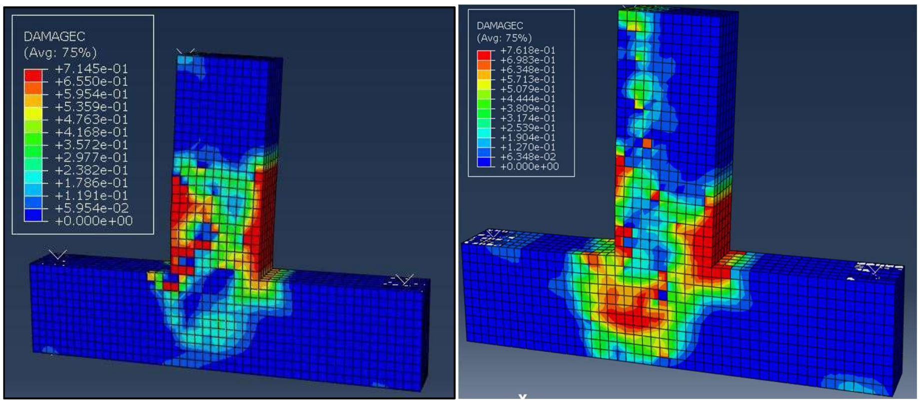
Figure 4.13. CFRP fibre orientation for numerical retrofitted BCJ



(a) **(b)**

Figure 4.14. Damage in the concrete of the retrofitted beam column joint due to tension a) 2-layer CFRP retrofitting b) 1-layer CFRP retrofitting

The damage in the BCJ specimen retrofitted using 1 layer of CFRP Figure 4.14(b) can be seen to be far spread then the BCJ retrofitted using 2 Layers of CFRP Figure 4.14(a). Further, the effectiveness of CFRP in resisting damage can be concluded from the Figure 4.14. The retrofitted specimen with 2-layer of CFRP shows a much lesser damage in the joint region at the ultimate load, compared to the damaged BCJ retrofitted with just 1-layer of CFRP. Further, the beam shows a lesser damage in case of 2-layer retrofitting predicting the effectiveness of CFRP as a retrofitting material.

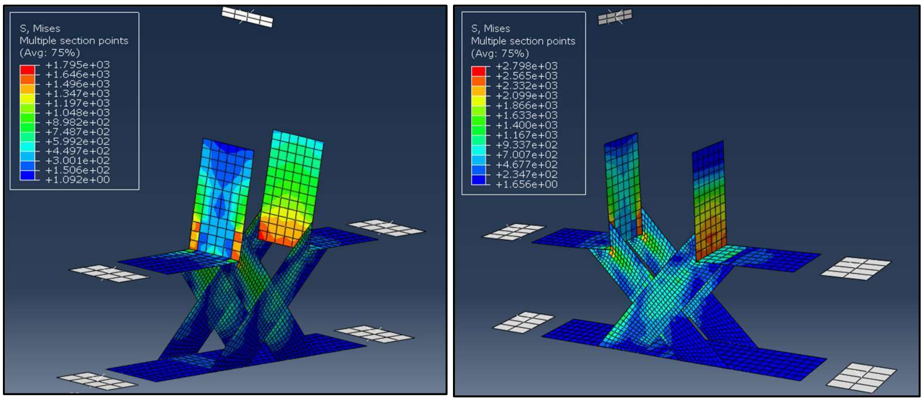


(a) **(b)**

Figure 4.15. Damage in the concrete of the retrofitted beam column joint due to compression a) 2 layer CFRP retrofitting b) 1 layer CFRP retrofitting

The damage in the BCJ specimen retrofitted using 1 layer of CFRP Figure 4.14(b) and Figure 4.15(b) for damage in tension and compression, respectively can be visualized to be far spread then the BCJ retrofitted using 2 Layers of CFRP Figure 4.14(a) and Figure 4.15(a). Further, the effectiveness of CFRP in resisting damage can be concluded from the Figure 4.13 and Figure 4.14.

4.2.1.6 *Stress resultant of the retrofitted specimens*



(a) **(b)**

Figure 4.16. Stress resultant in the CFRP of the retrofitted beam column joint at ultimate load for complete damage a) 2-layer CFRP retrofitting b) 1-layer CFRP retrofitting

The stress levels in the CFRP of the two retrofitting schemes show the expected variation in the stress levels. The 2 layers of CFRP used for retrofitting damaged BCJ sows a resultant stress of 1795 N/mm^2, while the specimen retrofitted with 1-layer of

CFRP shows a resultant stress level of 2798 N/mm^2. Further, which concludes that the CFRP is highly stressed and is efficiently used in the latter case i.e. 1-layer CFRP retrofitting of damaged BCJ.

4.3 STRENGTHENING OF CONTROL BCJ WITH 1 AND 2-LAYERS OF CFRP

Basically, in this study the behavior of using two and one layer of CFRP for retrofitting has been observed. This section deals with the study of CFRP as a strengthening material if used on the control specimen to check the effect on the ultimate load carrying capacity and the initial behavior of the strengthened specimens. Where the control specimen is strengthened using 2 layers of CFRP and 1 layer of CFRP. The figures below show the behavior obtained for strengthening of control specimen using CFRP.

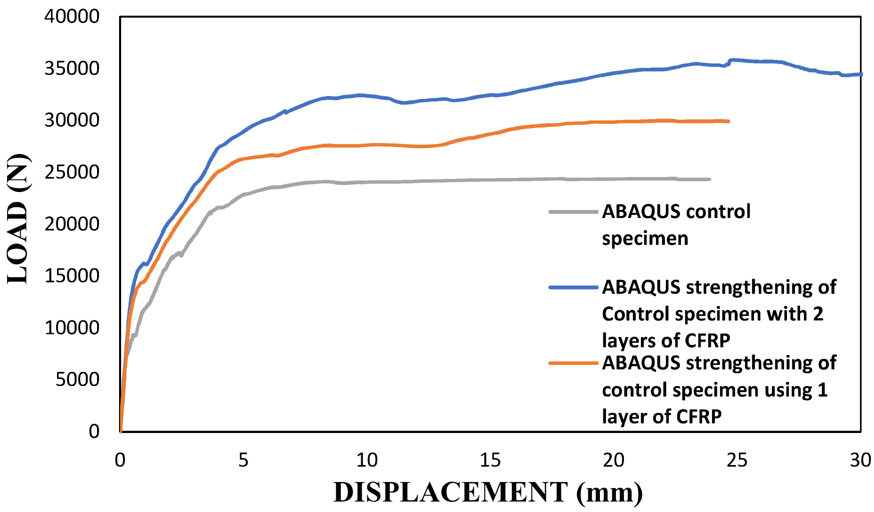


Figure 4.17. Comparison of load-displacement behavior after strengthening control BCJ using 1 and 2- layer of CFRP

The strengthening results propose a viable option of strengthening the older structures that are yet to be damaged by any loading activity leading to reversal of moments at the joint region of the joint. Figure 4.16, shows such a comparison between the effectiveness of using multi-layers of a retrofitting material like CFRP tested in this study. As can be concluded from the results of Figure 4.16, CFRP is highly efficient in strengthening of the control specimens. The efficiency can be measured by comparing the ultimate load of the strengthened and control specimen. The ultimate load capacity of the sample strengthened by 2-layers of CFRP is 35.79 kN and that from the control specimen is of about 24.35 kN. So, it can be taken out from the two observations that there is an increase of about 32%. Which is quite a significant increase. The ultimate load capacity of the sample strengthened by 1-layer of CFRP is 30 kN and that from the control specimen is of about 24.35 kN. So, it can be taken out from the two observations that there is an increase of about 18.4%. Further, depending on number of layers, CFRP proves to be more efficient, as can be concluded from Figure 4.16, the ultimate load for a 1-layer CFRP strengthened specimen is of 30 kN, while for a 2-layer strengthened specimen is 35.79 kN, which is about 17% more than the 1-layer model. Further the bonding will be more for more layers of the CFRP though this would also be up to a certain limit for number of the layers of CFRP.

The study of using 1-layer CFRP instead of 2-layers as done experimentally by Bansal and Kumar (2014), has shown that 1-layer of CFRP is effective for restoring the ultimate load carrying capacity of the section of beam-column joint up to an appreciable level. This can further be understood from the below bar chart representing various ultimate load attained after retrofitting specimens with various damage levels.

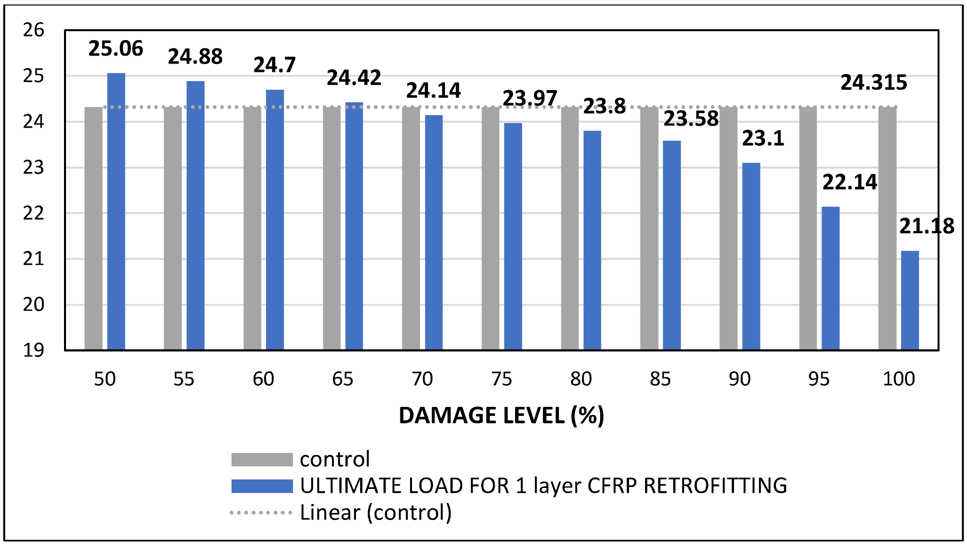


Figure 4.18. Ultimate load for 1 layer of CFRP retrofitted vs Control Specimen

As can be taken from the above figure the effectiveness of 1 Layer of CFRP in restoring the ultimate load of the damaged specimen for various levels of damage. Further as can be seen that 1 layer of CFRP is effective in restoring the ultimate load carrying capacity of the beam-column section up to a damage level of approximately 70%, considering the cost of retrofitting the whole structure with CFRP this much amount of retrofit can be considered sufficient for some cases, and thus a major part from the cost of retrofit can be reduced to half. Further what can be concluded from the above figure is that for increase in damage level of the joint the retrofit of the structure with 1 layer of CFRP is still an effective solution provided some other procedure is incorporated with it say providing and angle plate at the bottom of the junction or a steel plate with bolted connection to the beam of the structure.

4.4 CORRELATION ANALYSIS

Though the analysis of retrofitting, damaged BCJ is efficiently performed numerically in Abaqus using material change procedure. The method is still cumbersome due to the iterations needed to match the experimental results and further the time required to get the output for each iteration takes away the benefits of using numerical procedure for analysis.

A simplified procedure to determine the material properties for each damaged level is required for the efficient working. Therefore, use of simplified equations based on the available data using regression can be generated to predict the results for different damage levels. Though this procedure effectively predicts the ultimate load for the system that has been retrofitted with 1-layer of CFRP, there is still some error for the prediction of the ultimate load for the 2-layer CFRP system, this can be attributed to the fact that the orientation of the CFRP changes for 2-layers of the CFRP retrofitting. Various equations types have been tried to satisfy the data for prediction of the ultimate load for the specimen, with minimum error. Two curve fit equations where basically selected that could be used to predict the behavior of the damaged specimens for the ultimate loading:

1) Polynomial equation $(y = ax^2 + bx + c)$.
2) Linear equation $(y = ax + c)$.

Although polynomial curve fit gives a better R^2 value for the data fitting curve but, its unable to predict the earlier pre-yield region for the retrofitted 1-layer CFRP, this is further discussed in the coming sections. Taking this limitation into account the linear fit is selected as its able to predict the pre-yield behavior well and further can predict the ultimate load for various damage levels in the model.

Table 4.6 Ultimate load for various retrofitting's of CFRP on damaged samples and Control specimen from Abaqus

Damage level		Retrofitted samples	
Load kN	**% of Ultimate load**	**ULT – 2 Layers kN**	**ULT – 1 Layers kN**
12.16	50	27.80	25.06
13.37	55	27.58	24.88
14.59	60	27.37	24.70
15.80	65	27.32	24.42
17.02	70	27.28	24.14
18.24	75	27.224	23.97
19.45	80	27.17	23.8
20.67	85	26.815	23.58
21.88	90	26.77	23.1
23.10	95	26.69	22.14
24.32	100	26.61	21.18

1) Polynomial fit

The figure below shows the polynomial equation fitted for the curves of ultimate loads obtained for various levels of damage and for different retrofitting schemes.

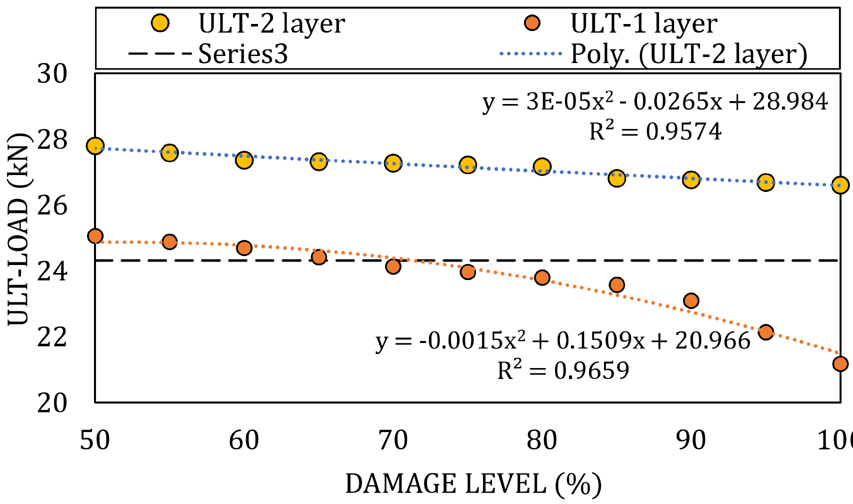


Figure 4.19. Polynomial curve fit for Ultimate load of retrofitted samples and control specimen

As can be concluded from the polynomial curve fitting of the ultimate loads of the various damage levels with different retrofitting schemes, the R^2 value for the generated equations is close to 1 for both the retrofitting schemes depicting less errors and more fit. The two equations for different retrofitting schemes are:

1. **For 2-layer CFRP retrofitting scheme**

$$y = 3\times10^{-5}x^2 - 0.0265x + 28.984 \qquad R^2 = 0.957 \qquad \text{(Eq 4.1)}$$

2. **For 1-layer CFRP retrofitting scheme**

$$y = -0.0015x^2 + 0.1509x + 20.966 \qquad R^2 = 0.9659 \qquad \text{(Eq 4.2)}$$

In equations(4.1 and 4.2), x stands for the damage level.

y stands for the ultimate load.

From the polynomial fit equation, the data predicted for 1 layer of CFRP is not able to determine the pre-yield behavior of the specimen for ultimate load and thus can't be a perfect fit for the purpose.

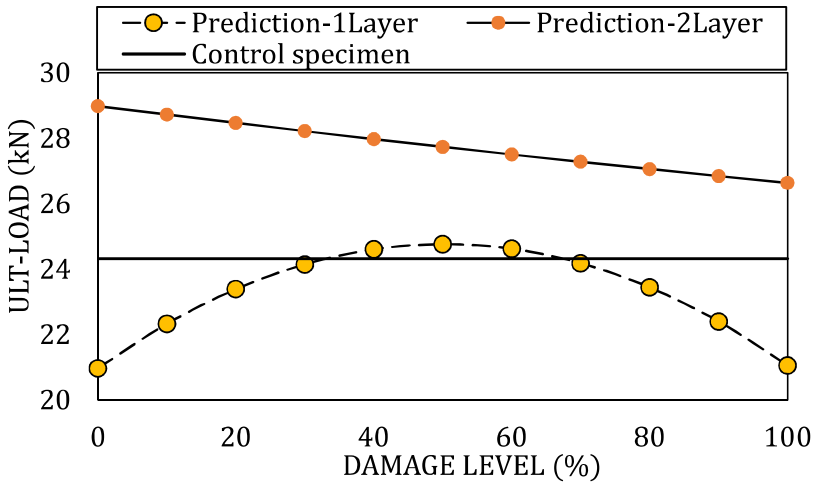


Figure 4.20. Predicted values for different retrofitted schemes by polynomial curve fit

2) Linear fit

The figure below shows the linear equation fitted for the curves of ultimate loads obtained for various levels of damage and for different retrofitting schemes. As can be concluded from the Linear curve fitting of the ultimate loads of the various damage levels with different retrofitting schemes.

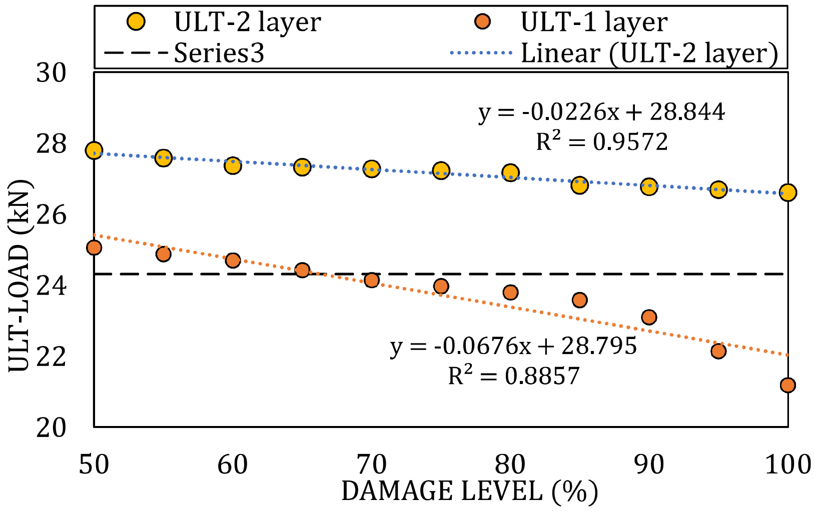


Figure 4.21. Linear curve fit for ultimate load of retrofitted samples and control specimen

The R^2 value for the generated equations is close to 1 for 2 layer CFRP retrofitting schemes depicting less errors and more fit. Though for 1 layer CFRP retrofitting scheme the value of R^2 is about at 0.89. The two equations for different retrofitting schemes are:

1. For 2-layer CFRP retrofitting scheme

$$y = -0.0226x + 28.984 \qquad (R^2 = 0.957) \qquad \text{(Eq 4.3)}$$

2. For 1-layer CFRP retrofitting scheme

$$y = -0.0676x + 28.795 \qquad (R^2 = 0.8857) \qquad \text{(Eq 4.4)}$$

In equations (4.3 and 4.4), x stands for the damage level.
y stands for the ultimate load.

From the Linear fit equation, the data predicted for 1 layer of CFRP can determine the pre-yield behavior of the specimen for ultimate load and thus can be taken as a perfect fit for the purpose.

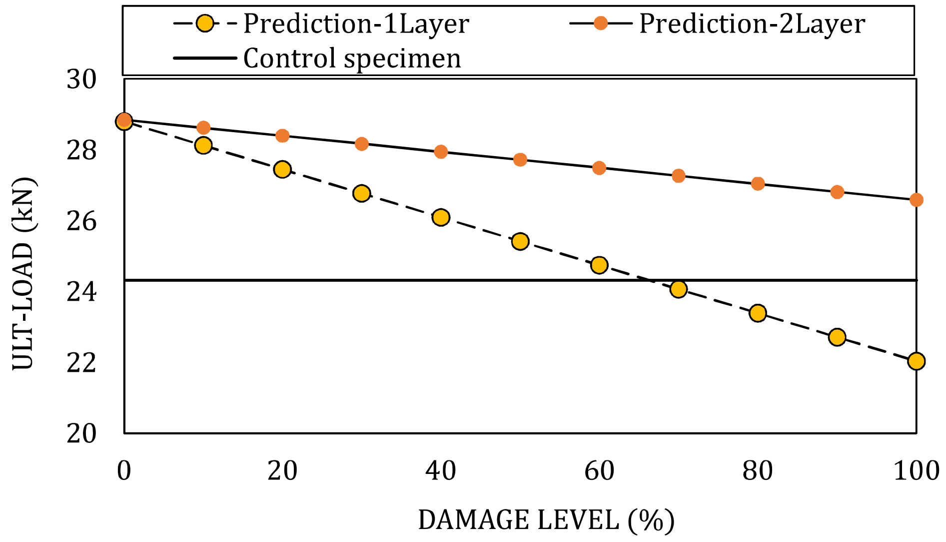


Figure 4.22. Predicted values for different retrofitted schemes by linear curve fit

The prediction of ultimate load from the linear equation for 1 layer of CFRP retrofitting can be verified using the strengthening data available in section 4.3 i.e. the behavior of the strengthened sample with 1 layer of CFRP. A general comparison can be drawn for the ultimate load carrying capacity of the 0% damaged model from the prediction equation and the 1 layer CFRP strengthening of the control sample. The ultimate load in both the cases are exactly same i.e. 29 kN, and thus it can be said that the prediction model prepared for the ultimate load of 1 layer of CFRP retrofit is quite accurate. Though this cannot be said for the ultimate load of the 2 Layer CFRP retrofit equation as the prediction results have a deviation of 17% i.e. for the ultimate load of the strengthened sample as 35kN for control specimen and an ultimate load predicted by the equation 4.3 of the linear fit for the 2 Layer CFRP retrofitting of the damaged sample.

5.0 GENERAL

The present study aims at the numerical investigation of RC beam column joint designed for gravity loading. The capacity of the joint is analyzed through incremental nonlinear monotonic loading up to the ultimate load. The overall significance of several influencing factors in the strength and deformation capacity of the RC beam column joint are highlighted. Attempt has also been made to understand the behavior of the beam column joint retrofitted with CFRP wrapping before and after damage is incurred in the beam column joint due to different levels of loading. The chapter highlights the key points and achievements of the study, limitations, and possible recommendation for future.

5.1 SUMMARY

An exterior RC beam column joint (BCJ) has been numerically modelled in Abaqus and its behavior under nonlinear static loading is studied. The results of control BCJ are validated with those of experimental work (Bansal and Kumar 2014). The BCJ is subjected to various damage levels based up on its ultimate load carrying capacity of the control. The damaged section of BCJ, are then retrofitted using CFRP wrapping sheets and analyzed numerically to predict the damaged behavior of the BCJ and effectiveness of retrofitting strategy. Different procedures for numerical modelling of retrofitted specimens were implemented, however, each has their own advantages and limitations. Further, the effectiveness of different number of layers of CFRP wrapping on the damaged BCJ is also studied, prediction models are proposed for retrofit of BCJ damaged to different intensities.

5.2 CONCLUSION

The conclusions from the present study are summarized as below:

1. ***The control beam column joint***

   The joint region of the control beam-column joint is found to be most critical, under nonlinear monotonic lateral loading. This is clearly observed from the damage flow patterns, both in tension and compression, obtained in the present numerical study for the BCJ. The beam and the column members experienced moderate to minor damage, when loaded to its ultimate strength.

A maximum deviation of +7.3% in the ultimate load of the BCJ is obtained as compared to experimental work.

2. ***Retrofitting using layers of CFRP sheet***

Retrofitting with 2-layers of CFRP, for the damage level of 50%, 85% and 100% shows a good relation with the experimental retrofitted sample, with the deviation in ultimate load carrying capacity as +9.8%, +4.2% and +2.3% respectively.

Retrofitting with 1-layer of CFRP, for the various damage levels (50%, 85% and 100%) the ultimate load capacity of the section was found to deviate from experimental retrofitted samples by an amount of +1.58%, -9.57% and -19.2% respectively.

Further retrofitting with 1 layer of CFRP was found to be effective to restore the ultimate load carrying capacity of the specimens that were damaged only up to 70% of the ultimate load of the control specimen, and thus pose as an effective solution for structures where cost of retrofitting is a major constraint with a material like CFRP.

3. ***Prediction model for damage and ultimate load***

A prediction model is developed that would relate the damage levels with the ultimate load for the beam column joint assembly. Though the model was found to effectively predict the ultimate load corresponding to damage for the single layer of the CFRP wrapping, for 2 layers of CFRP retrofitted samples the equation predicts an ultimate load with 17% deviation from the obtained results.

5.3 LIMITATIONS OF THE PRESENT STUDY

The results presented in the thesis are subjected to following limitations:

1. The present study is based on only nonlinear monotonic analysis of the beam column joint using numerical methods.
2. Due to the constraints of computational power of the machines available, a larger mesh size is considered for the elements of the numerical BCJ which provides a conservative result for the study.
3. The linear equation developed that relates the damage in the specimen and the

ultimate load carrying capacity of the section, requires a larger database of data for more accuracy.

5.4 FUTURE SCOPE

Based on the present results and limitations of the study, the following further study can be carried out.

1. Different configurations of reinforcement details and section size can be implemented to check their influence on the overall performance of the beam and further its retrofitting. Moreover, retrofitted joint at different stress levels can be tested with other kinds of FRP wrapping.
2. Dynamic examination of retrofitted joint at higher deflections can likewise be fused in the investigation.
3. The models to predict the ultimate load from the damage level in the sample can be refined by taking more data points that can reduce the errors and deviation.

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