SEISMIC CONNECTION FOR STEEL SQUARE HOLLOW BEAM-TO-SQUARE HOLLOW COLUMN JOINT

The thesis submitted in partial fulfilment of requirements for the degree of

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in

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(Specialization: Structural Engineering)

by

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CERTIFICATE

This is to certify that the thesis entitled **"SEISMIC CONNECTION FOR STEEL SQUARE HOLLOW BEAM-TO-SQUARE HOLLOW COLUMN JOINT"** submitted by **Samrat Biswas** to the National Institute of Technology, Rourkela for the award of the degree of Master of Technology is a bonafide record of research work carried out by him under my supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

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ABSTRACT

KEYWORDS: *hollow sections, non linear (pushover) analysis, capacity curve, plastic hinge*

Most of the steel structures in India are made of conventional steel sections (such as angle, channel and beam sections). However, new hollow steel sections (such as square and rectangular hollow sections) are gaining popularity in recent steel constructions due to a number of advantages such as its higher strength to weight ratio, better fire resistance properties, higher radius of gyration, lesser surface area, etc. This type of hollow sections can save cost up to 30 to 50% over the conventional steel sections (Tata Steel brochure, 2012). But unlike the conventional steel sections these hollow sections do not have standard connection details available in design code or in published literature. To overcome this problem the objective of the present study was identified to develop a suitable and economic connection detail between two square hollow sections which should be capable of transmitting forces smoothly and easy to be fabricated.

To achieve the above objective, a square hollow beam to square hollow column connection was selected and modelled in commercial finite element software ABAQUS. This model was analysed for nonlinear static (pushover) analysis considering a number of connection details. Following four alternative scheme of connection details were selected for this study: (i) using end-plate, (ii) using angle section, (iii) using channel sections, and (iv) using collar plates. The base model (rectangular hollow beam welded to one face of the rectangular hollow column) is also studied for reference. The performance of the selected connection details are compared and the best performing connection details is recommended for rectangular hollow beam-to- rectangular hollow column joints.

The result shows that the load carrying capacity of the joint and the maximum deformation capacity is highly sensitive to the type of connection used. Also, the location of formation of plastic hinges in the structure (which can be at joint or at beam) depends highly on the type of connection used.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

Most of the steel structures in India are made of conventional steel sections (such as angle, channel and beam sections) and were designed by conventional working stress methods. However, new hollow steel sections (such as square and rectangular hollow sections) are gaining popularity in recent steel constructions due to a number of advantages. Not only the hollow sections make the entire structure light weighted as they possess high strength to weight ratio, also they have higher efficiencies in resisting forces in comparison toconventional steel sections. The hollow sections also have better fire resistance properties. Higher radius of gyration, lesser surface area of the sections result savings in transportation, fabrication and painting costs. According to a recent study (Tata Steel Hollow section brochure), upto 30 to 50% saving in cost can be achieved by using hollow steel sections over the conventional steel sections.The steel industry in India started producing such hollow sections and making them available to the builders in regular basis.Fig. 1 presents photograph of typical rectangular hollow section (RHS) available in India

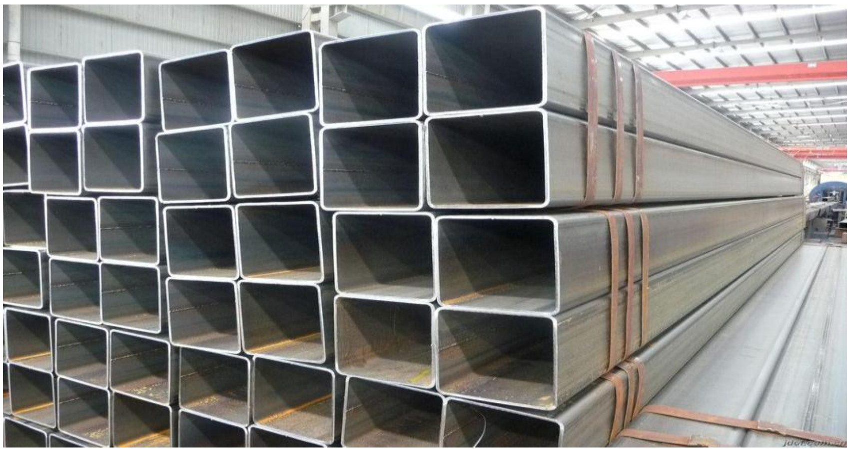


Fig. 1: Typical rectangular hollow section (RHS) available in India

This development has brought attention of researchers to 'the connection design' which is a very important aspect of steel design and construction.Unlike the conventional steel sections these hollow sections do not have standard connection details available in design code or in published literature.The application of hollow sections is not rising up as suitable connection configurations (for shear force and bending moment) have not been developed between the sections.

Sufficient research works have been done on the connections between conventional beams and columns (especially I-beam, I-column), but very little informations are available on the seismic connections between hollow beams to hollow columns. Direct extensions from connections detailing between the conventional sections are also not feasible as there are certain differences in geometry between hollow and conventional sections. As the thickness of the walls in hollow box sections are usually very small (4-8 mm) the possibility of local failure is very high if connection is designed without proper analysis. According to the current practice, full penetration of welds are used in joints involving smaller hollow sections and for large hollow sections diaphragms are inserted in columns at beam flange level. However this is not fully capable of resisting the fracture under seismic loading and also involves a lot of fabrication works resulting the hike in cost of the structure. The use of bolts can be an alternative as it would give considerable tolerances in fabrication but for tightening of nuts the access on the inside for the conventional bolts cannot be provided everywhere.

To overcome these problems a suitable and economic connection detail should be designed between two hollow sections which should be easy to fabricate and should be capable enough to resist loading. This is the primary motivation of the present study.

1.2 REVIEW OF LITERATURE:

Apart from several advantages of hollow sections, some research works are also found on the experimental and analytical investigations on connections between I-beam and box column sections by an extensive literature review. But in most of the works the hollow steel section or RHS is used as column section. So the information is limited. Above all no research work has been previously done in connection joint involving two hollow sections using welding.

White and Fang(1966) have mainly done research work on the performance and behaviour of beam column joint involving I-beam and box column. The experimental model consists of seat angle connections which was used as the main connectors between the I- beam and box column involving both welding and bolting. The results were very much satisfactory. It shows that significant deformation has been occurred at the web connections at the column flange but the deformation is independent of the width to thickness ratio of the column flange.

Ting et al. (1991) have also carried out investigations using finite element analysis of seismic connection between I- beam to box column. In this research the width of the beam was kept less than the column width. They have modelled the connections in three different schemes and have done comparative study to find out the best possible connection detail. The first connection scheme involved externally stiffened triangular plate stiffeners, second one with angle stiffeners and the third with T- stiffeners. After the analysis they have concluded that the best connection scheme among the three is the one with T- stiffeners as it has more capacity to redistribute the stresses and also provides more stiffness to the connection. They also concluded the fact that for the transfer of the stress to the column side walls, the width of beam and the stiffeners should be kept equal. The results also consist the minimum length of the stiffeners for minimum stress levels.

Shanmugam et al. (1991) have presented the results of the experimental investigations on I-beam to box column connections involving cyclic loading. They have stiffened the connection internally with continuity plates and eternally using T- stiffeners and angle stiffeners. Basically they compared the results between the behaviour of T- stiffeners and angle stiffeners as external stiffener involving cyclic loading. Both the connections showed stable load deflection curve and also exhibited satisfactory ductility values (5.1 to 10.7). They also concluded that the plate slenderness of column webs had an important role in the performance of connection under cyclic loading.

Korol et al. (1993) have presented the analytical results of the performance and failure mode of the moment connections between I- beam to box column under monotonic loading. The connections were detailed using extended end plate and high strength blind bolt. The results showed adequate values in ductility, stiffness and moment capacity.

Wheeler et al. (2000) carried out the experimental investigations on the performance of the connection between RHS involving end plate and pre-tensioned bolts. The results showed that the main reason of the failure of the connections were the fracture of bolts due to tension and the large deformation at the end plates.

Rao and Kumar(2012) have carried out the experimental investigations on the behaviour of the connection between a RHS beam and a RHS column involving two channel connectors of uniform thickness under cyclic loading. The connectors were welded to the column face. Rectangular openings were also provided at the beam webs so the bolting between beam flange and connectors can be done. They concluded that the channel connectors transmitted the stress resultants of the beam web to the column flange.

Goswami and Murty(2012) have presented the results of experimental investigations on the performance and failure of seismic connection between I-beam and box column involving different schemes of connection detailing. They have displacement controlled inelastic finite element analysis using ABAQUS software. The results showed that the connection involving externally reinforce inclined rib plate at the column face is the most efficient and economical.

It is clear from the literature review presented above that most of the previous works deal with the connection between I-beam and box column. Works on connection between two box sections involving welding have not been reported in any of the existing literatures.

1.3 OBJECTIVE AND SCOPE

Based on the literature review presented above the main objective of the present study is identified as to develop improved beam-to-column connection detailfor square hollow beam to square hollow column ensuring smooth flow of forces.Followings are the scopes and limitation of the present study.

(i) Only RHS or SHS sections are to be considered in this study for both column and beams.

(ii) Some time hollow columns are filled with concrete or other materials for improving compressive force capacity. This type of concrete filled hollow sections (CFHS) are however kept outside the scope of the present study.

(iii) Only welded connections are to be considered.

1.4 METHODOLOGY

To achieve the above objectives the following methodology have been worked out:

(i) Select the geometry of a square hollow beam to square hollow column connection

(ii) Model the selected connection in commercial finite element software ABAQUS

(iii) Plan for possible alternative connection details for the selected beam-to-column connection

(iv) Analyse the selected beam-to-column connection for nonlinear static analysis considering all the selected connection detail.

(v) Arrive at the most suitable connection detail for the selected beam-to-column connection considering flow of the stresses or forces, load carrying capacity of the joint and the associated ductility achieved.

1.5 ORGANIZATION OF THE THESIS

This introductory chapter presents the background, motivation, literature review, objective, scope and the methodology of the present study

Chapter 2 presents the details of the selected geometry of a square hollow beam to square hollow column connection,alternative connection details and structural modelling used in the present study. This Chapter also discusses about the principle of nonlinear static (pushover) analysis.

Chapter 3 presents the results obtained for the selected beam-to-column connection and discusses the relative performances of each of the different connection details used in this study.

Chapter 4 describes the summary of the work presented in this thesis and important conclusions drawn from the results and discussions presented in Chapter 3

CHAPTER 2

MODELLING

2.1 INTRODUCTION

Different methods can be utilised to study the responses of beam-to-column joints. Experimental testing would be the most effective out of all possible methods. However, experimental methods are very costly with respect to time and money. Therefore, finite element analysis technique has been used to analyse the joint in the present study. The finite element analysis has been increased in recent days due to progressive knowledge and capabilities of computer software and hardware. The use of computer software to model components is very much faster and also very cost effective. Acceptable conclusions can be drawn by analysing the structuresthrough finite element analysis.

Modelling is a very important aspect in finite element analysis; accuracy of the results depend on the accuracy in modelling.For the purpose of modelling and analysing a beam column joint involving two box sections, it is necessary to know about the element type and about their features and behaviours. It is also very important to have an idea about the different types of elements being used in FE software ABAQUS. A clear knowledge should be present on which element is better in 2D approximation and which is better for 3D approximation in ABAQUS.Therefore this chapter deals with finite elementmodelling including material modelling used in the present study.

This research is based on nonlinear static (pushover) analysis of selected beam to column joints. This chapter presents a brief discussions on the pushover analysis procedure used in the present study.

2.2 ELEMENTS IN ABAQUS

The wide range of elements in the ABAQUS element library provides flexibility in modelling different geometries and structures. – Each element can be characterized by considering the following-

- Family
- Number of nodes
- Degrees of freedom
- Formulation
- Integration

2.2.1 Family

A family of finite elements is the broadest category used to classify elements. Elements in the same family share many basic features. There are many variations within a family.

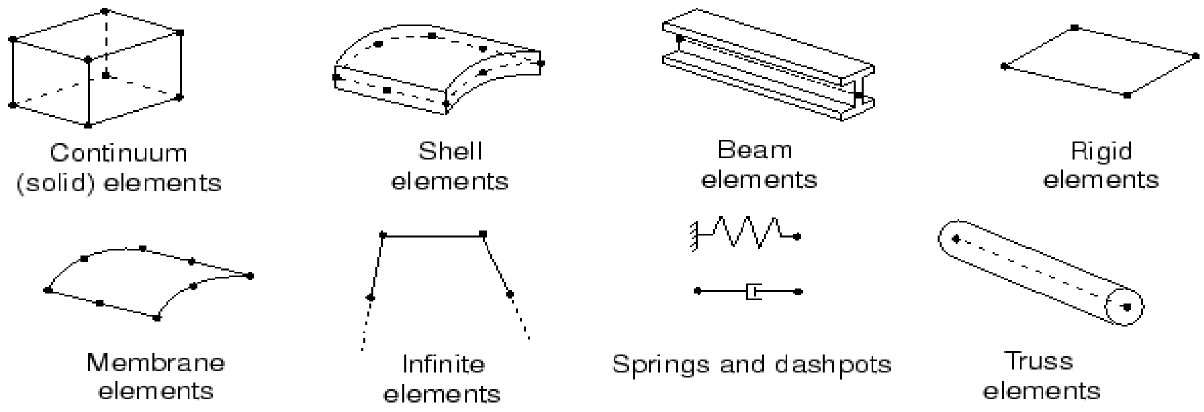


Fig 2.1 Elements Family (manual tutorial ABAQUS 6.12)

2.2.2 Number of nodes (Interpolation):

An element's number of nodes determines how the nodal degrees of freedom will be interpolated over the domain of the element. ABAQUS includes elements with both first and second-order interpolation.

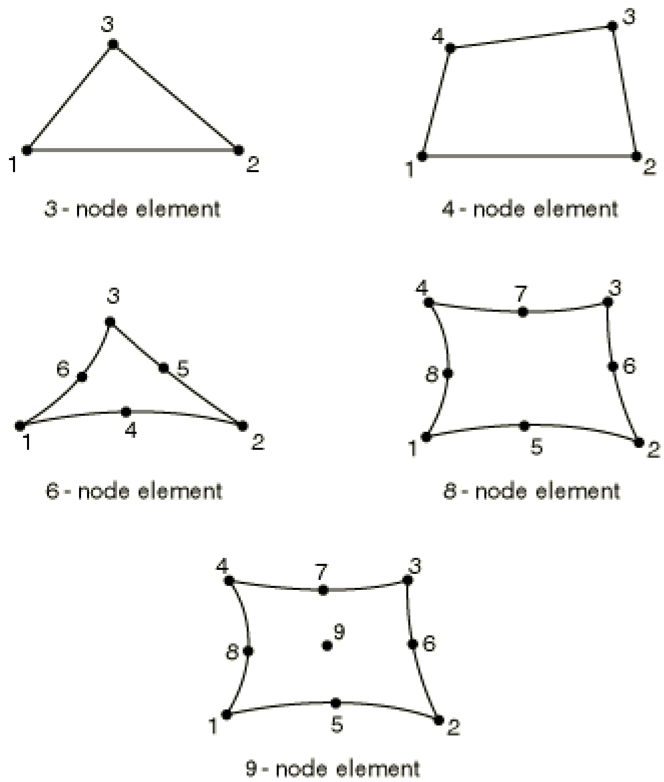


Fig 2.2 Interpolation points – node ordering on elements (manual tutorial ABAQUS 6.12)

2.2.3 Degrees of Freedom

The primary variables that exist at the nodes of an element are the degrees of freedom in the finite element analysis. Examples of degrees of freedom are translation in direction 1, translation in direction 2, translation in direction 3, rotation about the 1-axis, rotation about the 2-axis ,rotation about the 3-axis ,warping in open section beam elements, acoustic pressure, pore pressure, hydrostatic fluid pressure ,electric potential, temperature (or normalized concentration in mass diffusion analysis) for continuum elements or temperature at the first point through the thickness of beams and shells temperature at other points through the thickness of beams and shells. Directions 1, 2, and 3 correspond to the global 1-, 2-, and 3-directions, respectively, unless a local coordinate system has been defined at the nodes.

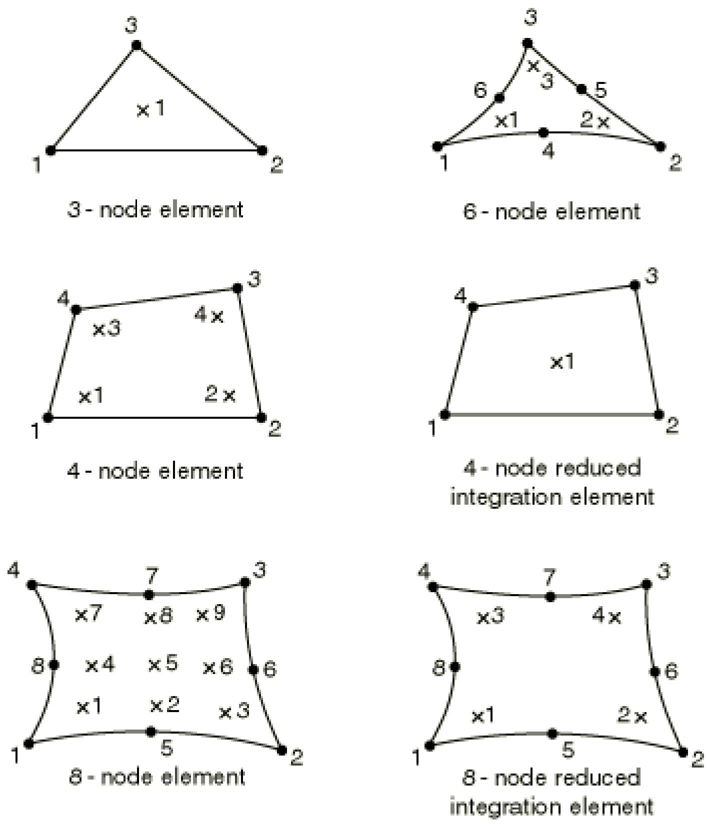


Fig 2.3 Interpolation points – numbering on output (manual tutorial ABAQUS 6.12)

2.2.4 Formulation

The mathematical formulation used to describe the behaviour of an element is another broad category that is used to classify elements. Examples of different element formulations: plane strain, small-strain shells, plane stress, finite-strain shells, hybrid elements, thick shells, incompatible-mode elements, thin shells.

2.2.5 Integration

The stiffness and mass of an element are calculated numerically at sampling points called "integration points" within the element. The numerical algorithm used to integrate these variables influences how an element behaves. The stiffness and mass of an element are calculated numerically at sampling points called "integration points" within the element.

Full integration: The minimum integration order required for exact integration of the strain energy for an undistorted element with linear material properties.

Reduced integration: The integration rule that is one order less than the full integration rule.

Reduced integration uses a lower-order integration to form the element stiffness. The mass matrix and distributed loadings use full integration. Reduced integration reduces running time, especially in three dimensions. For example, element type C3D20 has 27 integration points, while C3D20R has only 8; therefore, element assembly is roughly 3.5 times more costly for C3D20 than for C3D20R (use only with hexahedra elements).

2.2.6 Hourglass

Hour glassing can be a problem with first-order, reduced-integration elements (CPS4R, CAX4R, C3D8R, etc.) in stress/displacement analyses. Since the elements have only one integration point, it is possible for them to distort in such a way that the strains calculated at the integration point are all zero, which, in turn, leads to uncontrolled distortion of the mesh. Countermeasure: use finer mesh.

2.2.7 Shear and Volumetric Locking

Shear locking occurs in first-order, fully integrated elements (CPS4, CPE4, C3D8, etc.) that are subjected to bending. The numerical formulation of the elements gives rise to shear strains that do not really exist—the so-called parasitic shear (elements too stiff in bending) Countermeasure: use finer mesh through the thickness of the section.

2.3 ELEMENTS USED FOR MODELLING

For modelling purpose of RC beams, mainly beam elements, bar elements and shell elements are used**.** The element library in ABAQUS contains several types of beam elements. A “beam” is an element in which assumptions are made so that the problem is reduced to one dimension mathematically: the primary solution variables are functions of position along the beam axis only (as bar element). A beam must be a continuum in which we can define an axis such that the shortest distance from the axis to any point in the continuum is small compared to typical lengths along the axis. The simplest approach to beam theory is the classical Euler-Bernoulli assumption, that plane cross-sections initially normal to the beam's axis remain plane, normal to the beam axis, and undistorted (called B23, B33). The beam elements in ABAQUS CAE allow “transverse shear strain” (Timoshenko beam theory); the cross-section may not necessarily remain normal to the beam axis. This extension is generally considered useful for thicker beams, whose shear flexibility may be important (called B21, B22, B31, B32).

2 node linear beam element in plane (designation B21 in ABAQUS): It is an element used for plane stress analysis. It has three degrees of freedom at each node. Each element has two nodes.

2 node linear beam in space (designation B31): It is an element used for simple stress-strain analysis. It has two nodes and six degrees of freedom at each node at space.

3 node quadratic beam in plane-(designation B22): It is quadratic beam element used for stress analysis. Each element has three nodes. Each nodes carries three degree of freedom.

3 node quadratic beam in space-(designation B32): It is a quadratic element in space which has three nodes, six degrees of freedom at each node. It is used for stress concentration and for analysis of beam in space or frame in space.

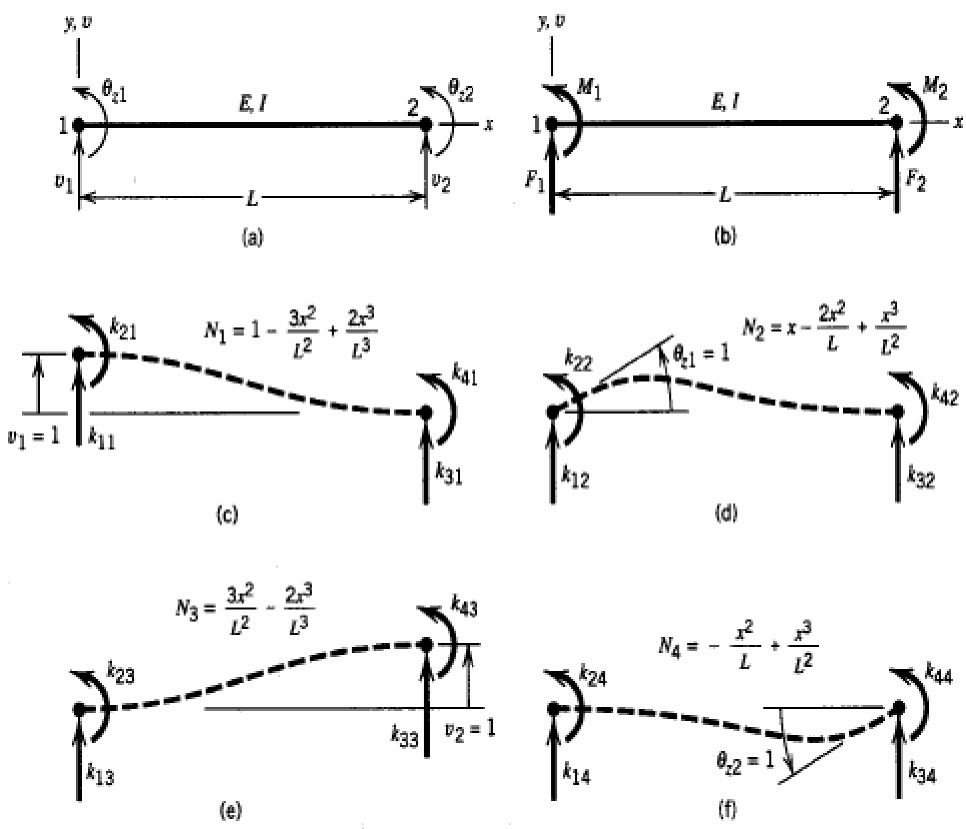
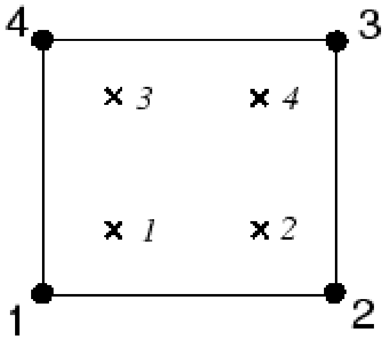


Fig 2.4: Shape functions, beam defined in ABAQUS CAE has linear or quadratic interpolation function (element B21, B22, B31 and B32) (manual tutorial ABAQUS 6.12)

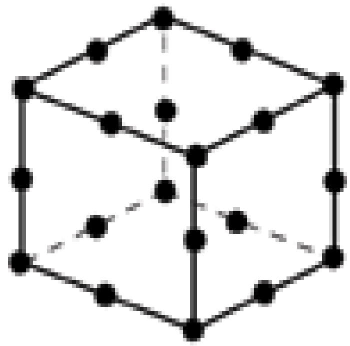
The solid element library includes isoparametric elements: quadrilaterals in two dimensions and **“bricks” (hexahedra)** in three dimensions. These isoparametric elements are generally preferred for most cases because they are usually the more cost-effective of the elements that are provided in ABAQUS. They are offered with first- and second-order interpolation. Standard first-order elements are essentially constant strain elements: the isoparametric forms can provide more than constant strain response, but the higher order content of the solutions they give is generally not accurate and, thus, of little value. The second-order elements are capable of representing all possible linear strain fields. Thus, in the case of many problems (elasticity, heat conduction, and acoustics) much higher solution accuracy per degree of freedom is usually available with the higher-order elements. Therefore, it is generally recommended that the highest-order elements available be used for such cases: in ABAQUS this means second order elements.

8 node linear brick element reduced integration-(designation C3D8R): It is an 8 node 3D brick element and it used for higher order beam analysis. Displacements, rotations, temperatures, and the other degrees of freedom mentioned in the section are calculated only at the nodes of the element. At any other point in the element, the displacements are obtained by interpolating from the nodal displacements. Usually the interpolation order is determined by the number of nodes used in the element. Used for 3D models.

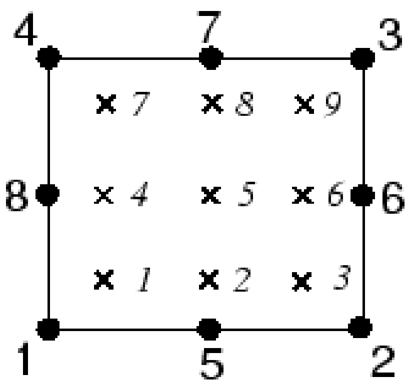
20 node quadratic brick element, reduced integration-(designation C3D20R): It is required for three dimensional model analysis. Displacements, rotations, temperatures, and the other degrees of freedom mentioned in the previous section are calculated only at the nodes of the element. At any other point in the element, the displacements are obtained by interpolating from the nodal displacements. Usually the interpolation order is determined by the number of nodes used in the element. Elements with mid side nodes, such as the 20-node brick shown in Figure 19 use quadratic interpolation and are often called quadratic elements or second-order elements.



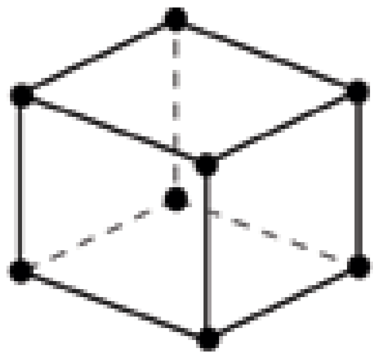
(a)



(b)



(c)



(d)

Fig 2.5: Finite elements-(a) linear element; B21,B31 (b) quadratic element; B22,B32

(c) C3D8R (d) C3D20R (manual tutorial ABAQUS 6.12)

2.4 GEOMETRIC MODELLING

As the elements required for modelling in the FE software have already been discussed in the above, now another important thing required to proceed in this research topic is the modelling of the geometry of the problem. As it has been already stated that the main objective of this work is to design a suitable connection between two box sections, the modelling part should contain box sections and other suitable sections (like angle, plate) to design a beam column connection. This part consists of the different types of proposed connection details modelled with the help of ABAQUS, their physical and FE characteristics, Boundary conditions.

2.4.1 Basic Connection

Two exactly similar square hollow sections are modelled with the help of ABAQUS and the connection is done by tie. No welding or bolting is done in this connection. The dimension of both the sections are 200×200×8 mm. Length of both the sections are 3000 mm. Boundary conditions are kept at the column ends as both sides hinged. The connection is shown in the below.

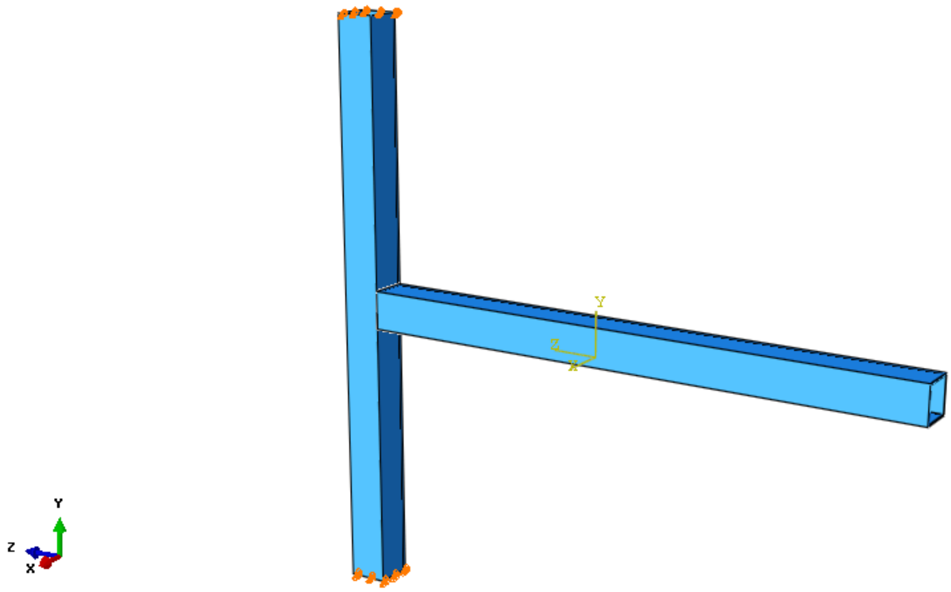


Fig 2.6: Basic connection detail

2.4.2 Connection Detail using End-plate

An end plate is connected at the beam column joint of the basic connection which is shown above. The thickness of the end plate is 10 mm. The mesh size of the end plate is kept 40mm square. The dimension of the plate is 200×1000×8.

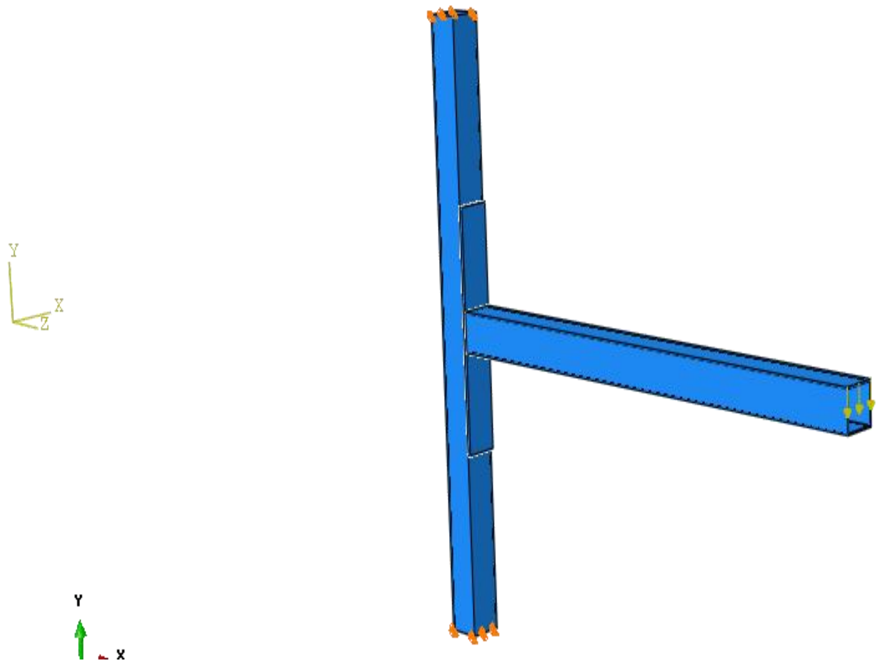


Fig 2.7: Connection detail using end plate

2.4.3 Connection Detail using Angle Section

In this type of connection detail four angle sections are connected with the box sections at the beam column junction. The mesh size of the angle sections are kept as 45 mm square. The dimension of each angle section is 100×100×10 mm.

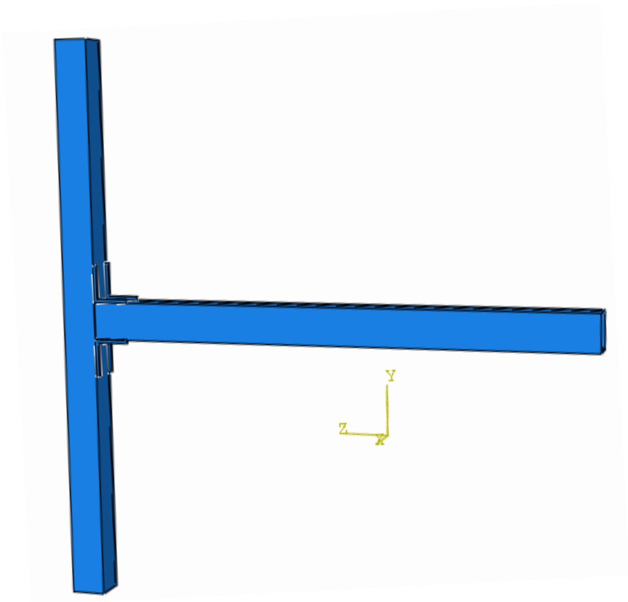


Fig 2.8: Connection detail using angle sections

2.4.4 Connection Detail using Channel Sections

Two channel sections are used in this type of connection detailing. They are basically used for jacketing the column section at beam column junction. The dimension of each channel section is 200×500×6 mm. The mesh size of the channel section is 40 mm square.

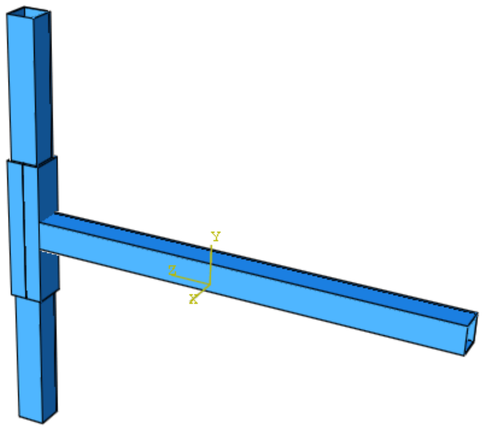


Fig 2.9: Connection detail using channel sections

2.4.5 Connection Details using Collar Plate:

This type of connection consists of two collar plates and twelve triangular plates. The collar plate is made by connecting two ‘C’ shaped plate of thickness 10 mm welding face to face.

The collar plates thus formed is then welded at the column at the upper and lower part of the beam column junction. The triangular plates are the welded between the column face and the collar plates to make a rigid connection. The dimension of the triangular plates are taken as 200×100×8 mm. The mesh size is taken as 40 mm square.

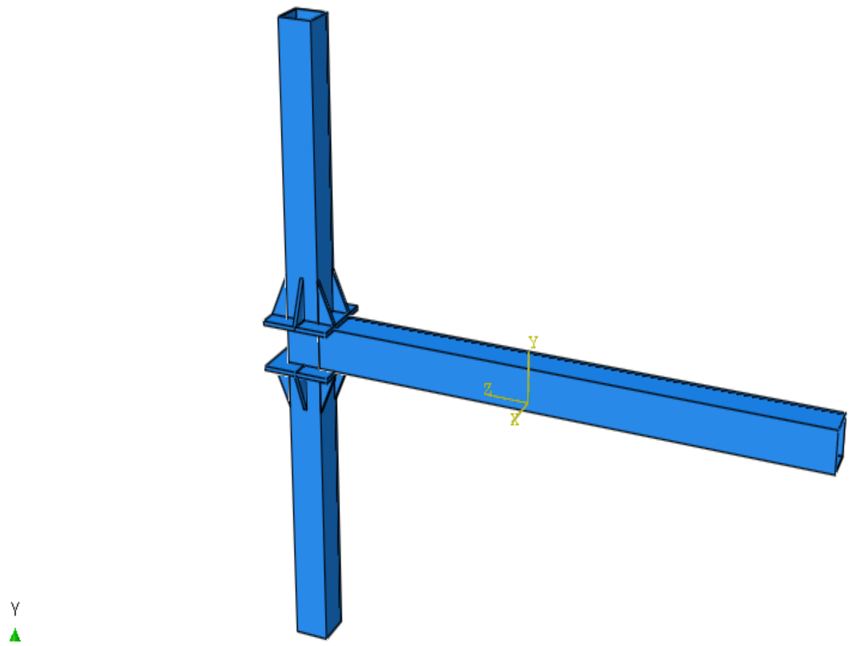


Fig 2.10: Connection detail using collar plates

2.5 MATERIAL MODELLING

Linear elastic analysis does not reflect the true behaviour of structure under ultimate loads, and it becomes necessary to model the material non-linearity when the structure is subjected to large deformations (due to forces like earthquake). To model the material nonlinearity in the present study stress strain curve of steel is considered as per. Fig. 2.11 presents the stress-strain relation of mild steel used in the present study. The characteristic strength of steel in tension and compression is assumed to be same as 250 MPa. The slope of the strain hardening portion of the curve is after removal of elastic curvature component. This stress strain relationship curve of mild steel has also been used in the research work titled “Use of external t-stiffeners in box-column to I-beam connections” by Ting and Shanmugam in the year 1998.

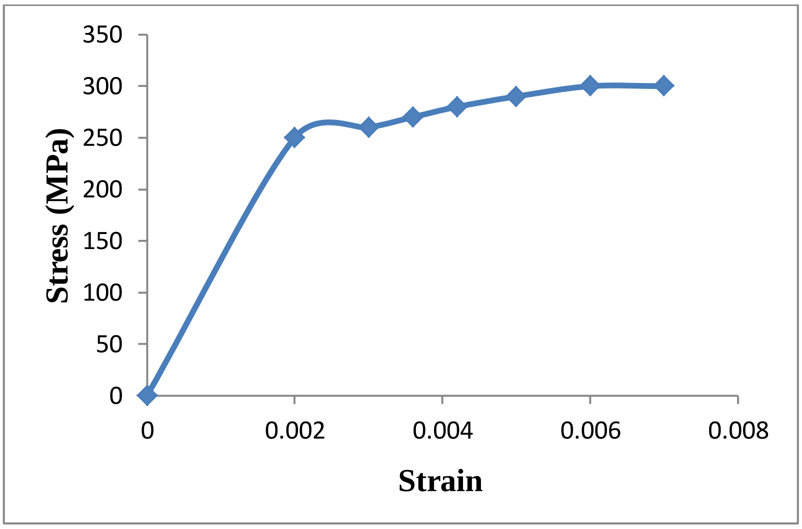


Fig 2.11: Stress strain curve for mild steel used in present study

2.6 NON LINEAR PUSH OVER ANALYSIS

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading (or displacement) is incrementally increased in accordance with a certain predefined pattern. In the present study a point load is applied at the tip-end of the beam and the load is incrementally increase using displacement controlled approach. With the increase in the magnitude of the displacement, weak links and failure modes of the beam-to-column joints are found. Local nonlinear effects are modelled through specifying nonlinear stress-strain behaviour and the tip end of the beam is pushed until collapse mechanism is formed. At each step, the total shear force reaction at the fixed end of the beam and the displacement of the free tip-end of the beam are plotted (Capacity Curve). Fig. 2.12 presents a sketch showing the pushover analysis procedure (as per ATC-40)

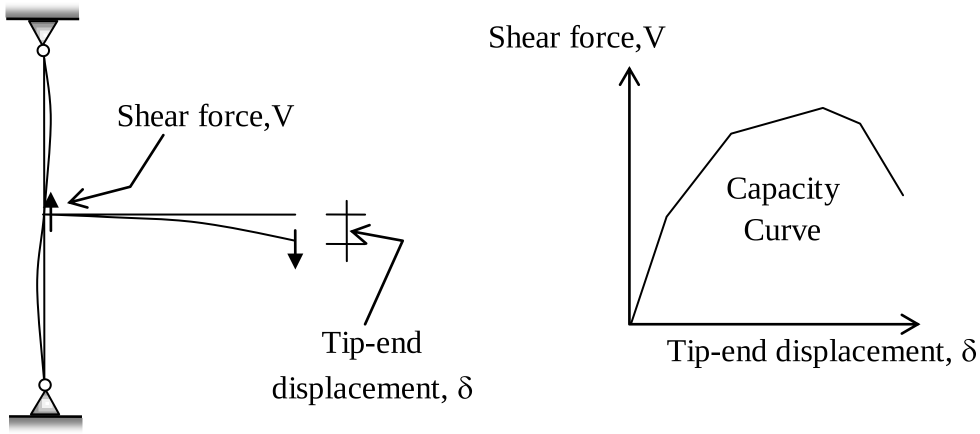


Fig. 2.12: Pushover analysis (ATC 40)

CHAPTER 3

RESULTS AND DISCUSSIONS

3.1 INTRODUCTION

As discussed in the previous chapter, the main objective of this research work is to develop a suitable connection between two box sections considering welding connective. To achieve this objective, the first thing to be considered is the parameters required to accomplish this work. From the extensive literature review which has been done for this research work, some inherent difficulties have been found out while designing a suitable connection between two hollow sections. The main difficulty is that the works which have done previously are basically between one hollow section and one conventional section. So direct extensions from that connection details is not feasible due to geometric differences. For thissome new parameters should also be considered. So in this chapter firstly a Fe analysis is done for the model consisting two Square Hollow sections with the help of ABAQUS software up to failure. Then analysing the results the main parameters like flow of forces, location of the formation of plastic hinge are sorted out and then some proposed connection details have been modelled fulfilling the criteria. Then a thorough comparative analysis has been done among the proposed connection details to select the best connection detail for the problem in all aspects.

So this chapter consists of modelling and analysis of the proposed connection details fulfilling the criteria of designing a suitable connection detail and selection of the best possible connection detail among the proposed ones.

3.2 BASIC CONNECTION DETAIL

To proceed in the current research work two suitable hollow sections have been chosen from the Indian Steel table. In this case two square hollow sections are chosen which are identical. Dimensions of the sections are 200×200×8 mm. The length of both the sections is taken as 3000mm. The modelling part and the mesh sizes have been stated in the previous chapter. Then the sections are assembled as abeam column structure. The assembled structure has been shown in figure 3.1. Boundary conditions at both the ends of the column are kept as hinged and load is applied at the tip of the beam.

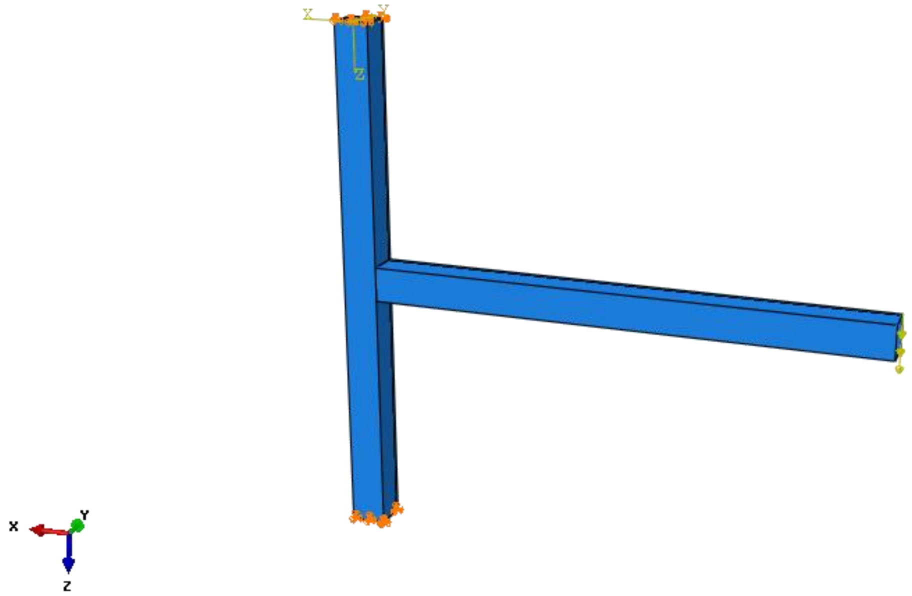


Fig 3.1 Basic connection detail

Applying the load at the tip of beam the structure is analysed with the help of ABAQUS software till the failure occurs. After the analysis it is seen that the major stress concentration is found at the beam column joint of the structure which has been shown in figure 3.2 which means that plastic hinge is formed at the column which is not good for a seismic connection. For a seismic connection the formation of plastic hinge should be away from the beam column joint. The other basic observation made form the analysis is that the flow of maximum principle force is from the beam centreline to the column web.

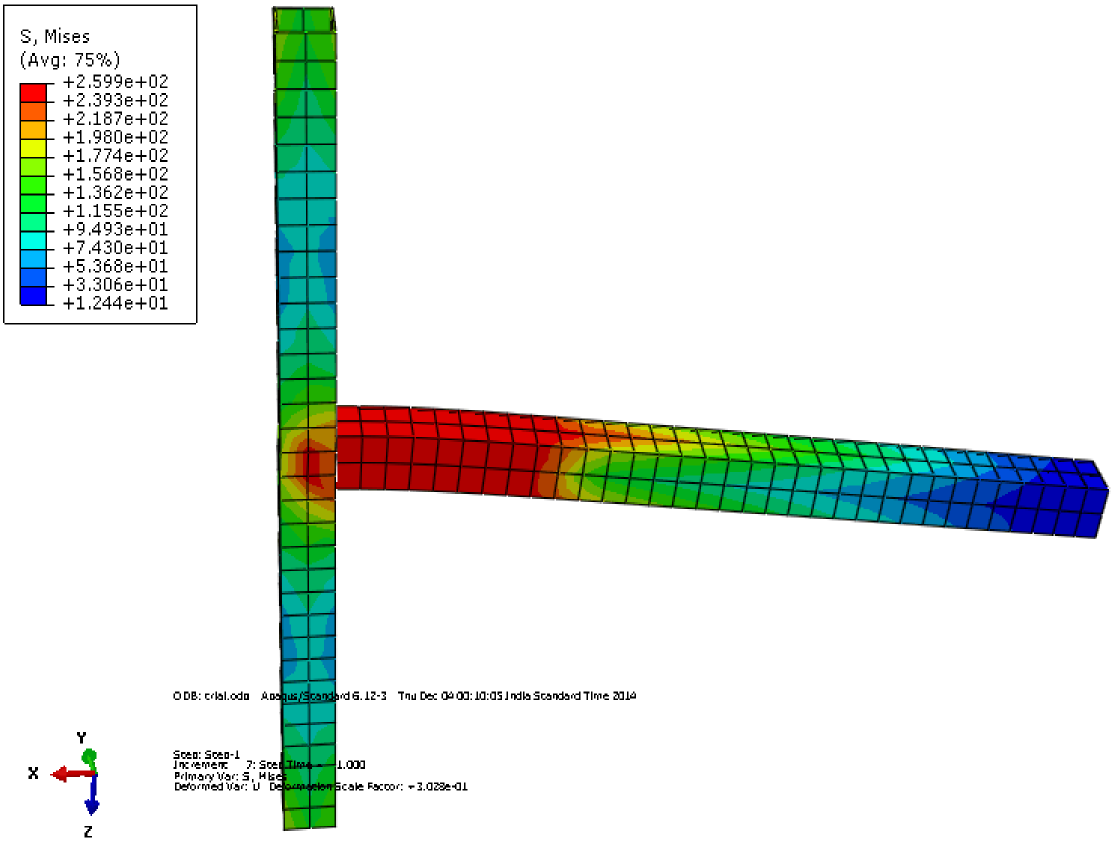


Fig 3.2 Stress distribution at failure

3.3 PROPOSED CONNECTION DETAILS

As the observations from the analysis of the basic connection detail show that formation of plastic hinge and the path of the force flow is not good for a suitable seismic connection, some new connection details between the existing structures have been proposed to overcome the above mentioned difficulties. The basis of selections of the appropriate connection details are as follows

- The connection detail should reduce stress intensity at the column face and push the location of energy dissipating plastic hinge away from the column face

- It should allow a smooth transfer of beam shear to column webs
- It should be capable of resisting larger deformations without fracture
- Strength and stiffness are to be sufficiently large.

Keeping the above points in mind some alternative connection details have been proposed. The descriptions, analysis and results of those proposed connection details are given in the below.

3.3.1 Scheme 1: Using End-plate:

In this type of connection detail a plate of thickness 8 mm is welded at the beam column junction of the structure as shown in figure 3.3. The dimension of the plate provided is 200×1000×8 mm. The main objective of providing the plate between the beam section and the column section is to reduce stress intensity at the column face and push the location of energy dissipating plastic hinge away from the column face.

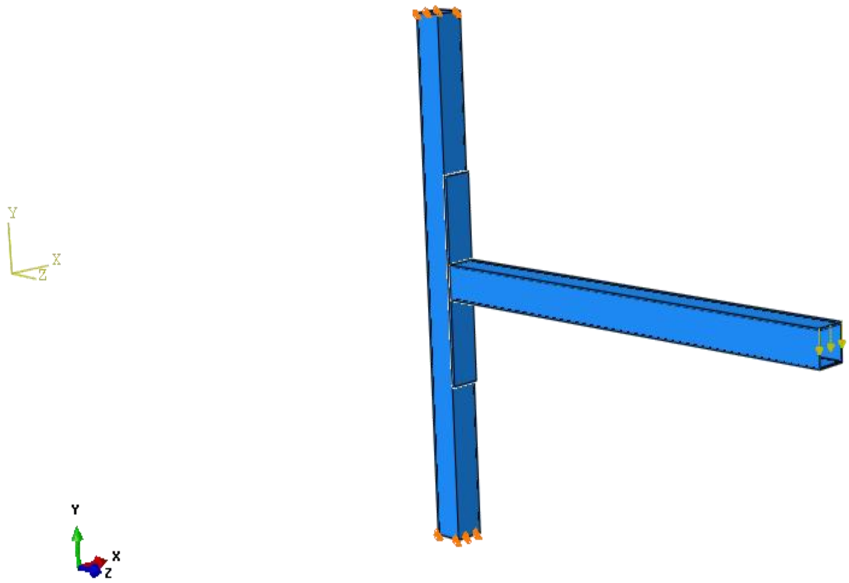


Fig 3.3 connection detail using end plate

The structure is then analysed by FE software ABAQUS after applying downward deflection at the tip of the beam. Deflection at the tip is gradually increased till it fails. The stress distribution of the structure at failure is shown in figure 3.4.

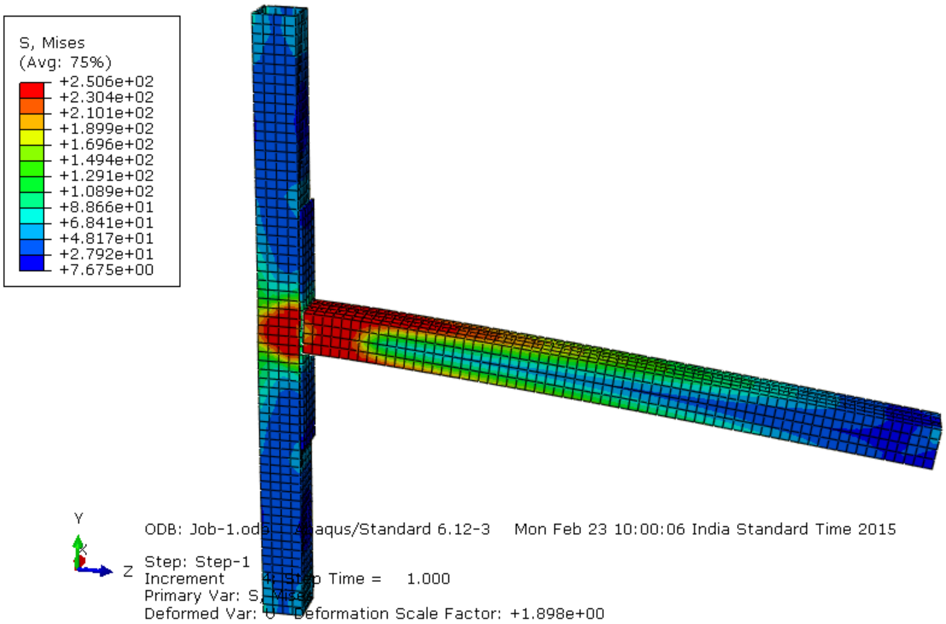


Fig 3.4 Stress distribution at failure

The results show that ultimate load carrying capacity of this connection is moderate but the displacement ductility is not up to the mark. The stress distribution also shows that maximum stress is concentrated at column face of the beam column junction. So, the purpose of trying to shift the formation plastic hinge away from the column face is also not satisfied.

3.3.2 Scheme 2: Using Angle Sections:

This type of connection consists of four equal angle sections welded at four corners of the beam column joint. The dimension of each angle section is 100×100×10 mm. The main purpose of providing the angle sections at the joint is to reduce the stress intensity at the column face and to achieve a moderate ductility. The connection detail is shown in figure 3.5.

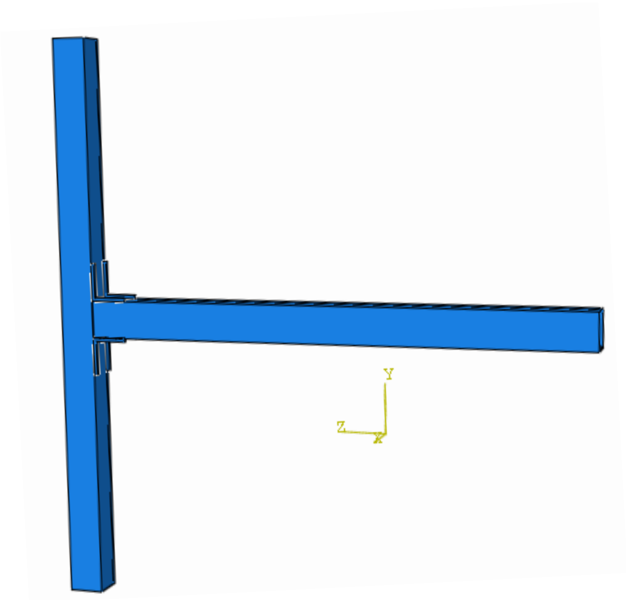


Fig 3.5: Connection detail using angle sections

The structure is then analysed by FE software ABAQUS after applying downward deflection at the tip of the beam. Deflection at the tip is gradually increased till it fails. The stress distribution of the structure at failure is shown in figure 3.6.

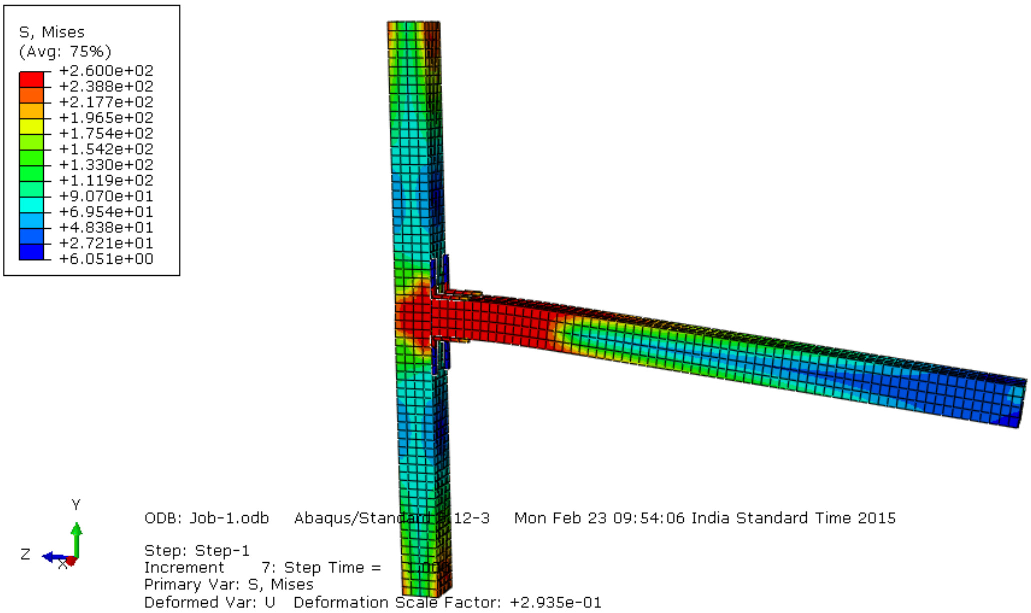


Fig 3.6 Stress distribution at failure

The result shows that the ultimate carrying capacity of this type connection is greater than the previous connection detail and the ductility is also better. But this connection is also not capable enough to reduce the stress intensity at the column face as it is shown in figure 3.6.

3.3.3 Scheme 3: Using Channel Section:

In this type of connection steel jacketing is used. Two equal channel sections are used to jacket the column at the beam column junction. The dimension of each channel section is taken as 200×500×6 mm. This type of connection detail is used to achieve a higher ultimate load and to reduce the stress at the column face by thickening the column. The connection detail is shown in figure 3.7.

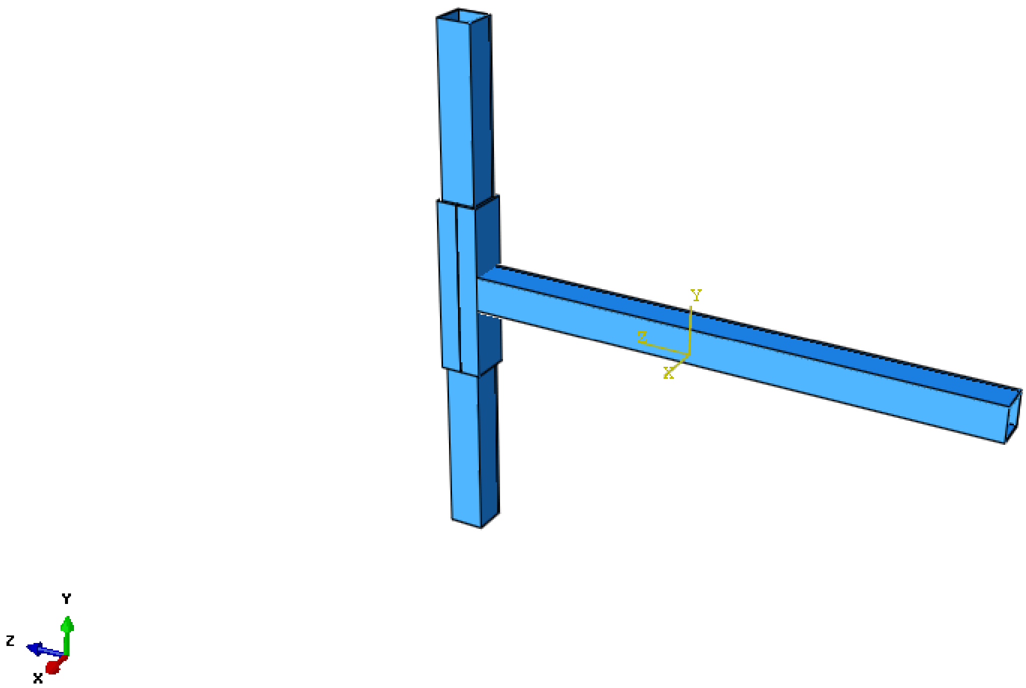


Fig 3.7: Connection detail using channel sections

The structure is then analysed by FE software ABAQUS after applying downward deflection at the tip of the beam. Deflection at the tip is gradually increased till it fails. The stress distribution of the structure at failure is shown in figure 3.8.

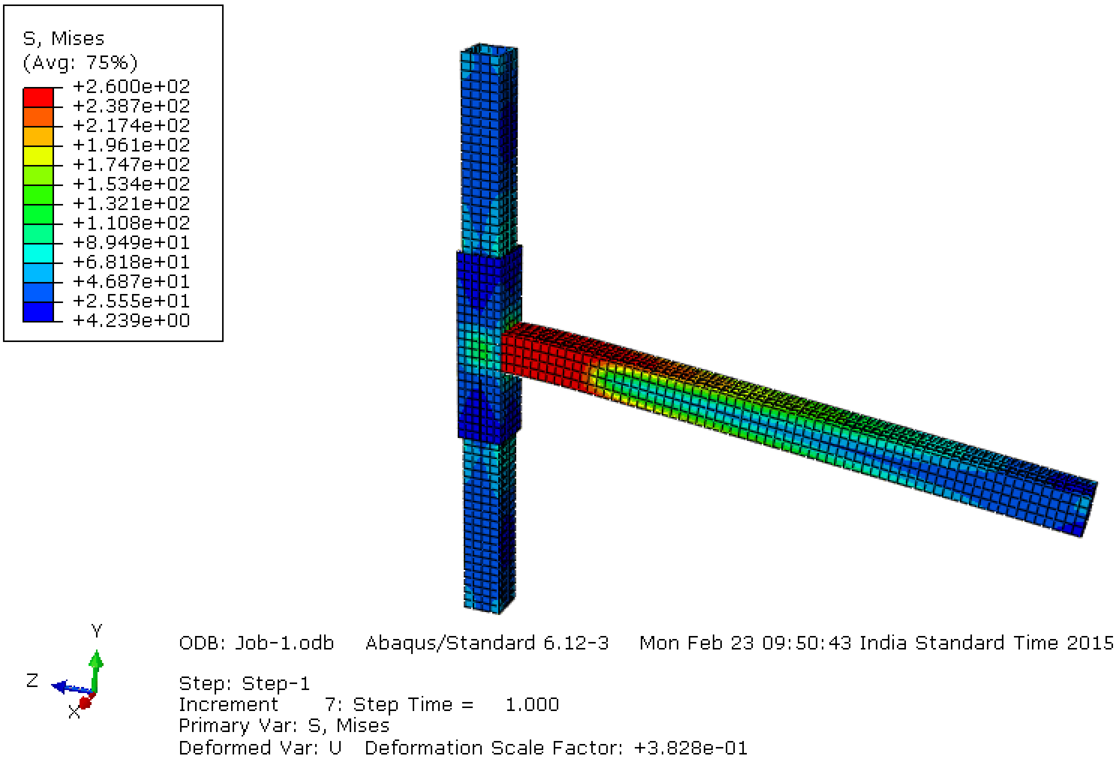


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Fig 3.8 Stress distribution at failure

The result shows that the ultimate load carrying capacity is very much higher than the previous two connections. From the figure 3.8 it can be also seen that at failure the stress intensity at the column face is very less. So the objective of reducing the stress intensity at the column is achieved. But the main drawback of this type of connection is displacement ductility is not that much high.

3.3.4 Scheme 4: Using Collar Plate:

This type of connection consists of two collar plates and twelve triangular plates. The collar plate is made by connecting two 'C' shaped plate of thickness 10 mm welding face to face.

The collar plates thus formed is then welded at the column at the upper and lower part of the beam column junction. The triangular plates are the welded between the column face and the collar plates to make a rigid connection. The dimension of the triangular plates are taken as 200×100×8 mm. The connection detail is shown is figure 3.9. The main objective for designing this type connection is to achieve a higher ductility value and a greater ultimate load at failure.

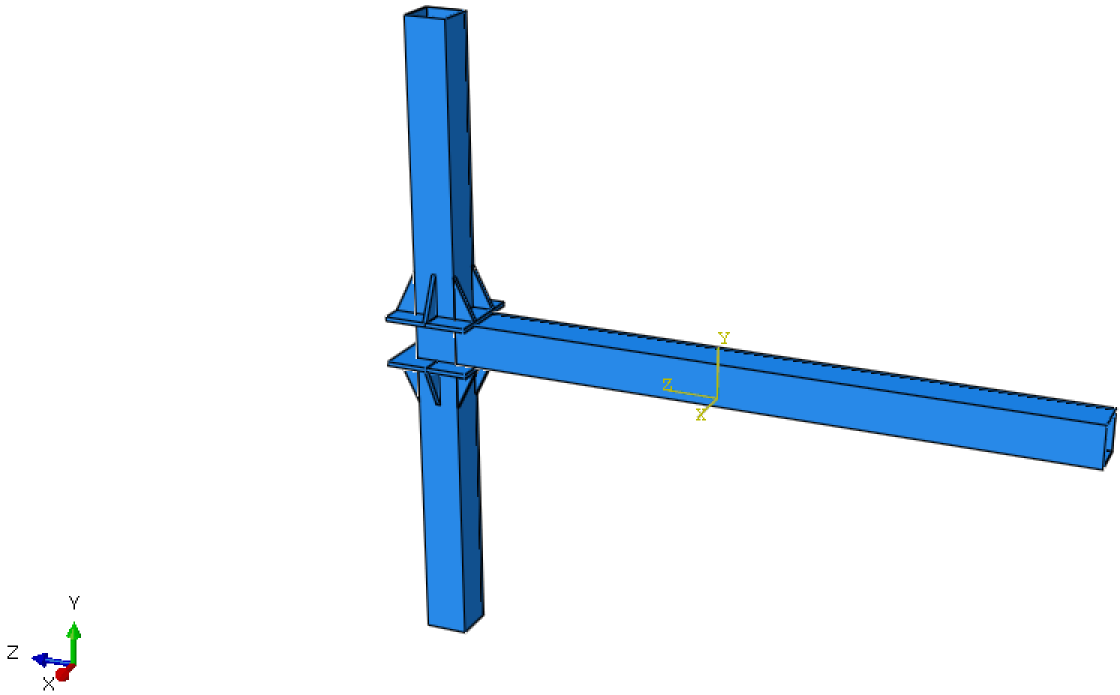

Fig 3.9: Connection detail using collar plates

The structure is then analysed by FE software ABAQUS after applying downward deflection at the tip of the beam. Deflection at the tip is gradually increased till it fails. The stress distribution of the structure at failure is shown in figure 3.10.

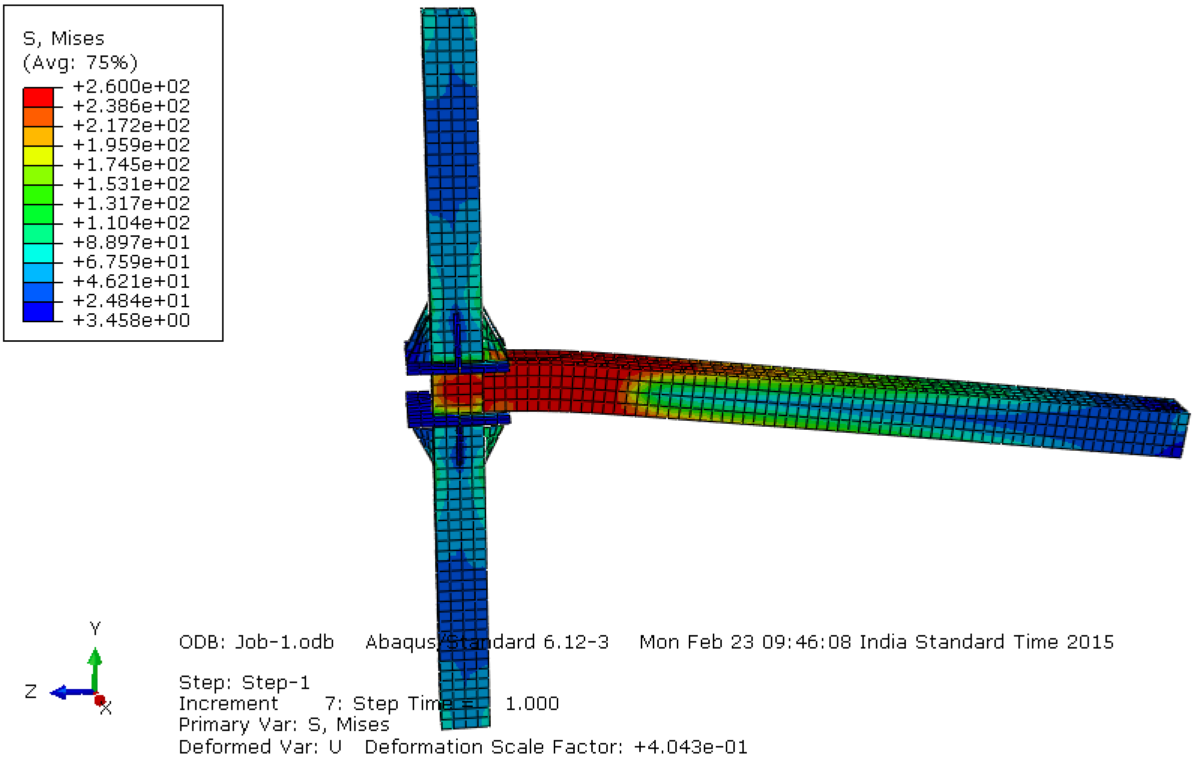

Fig 3.10 Stress distribution at failure

The result shows that though the ductility value is satisfactory but ultimate load carrying capacity is not greater the previous connection detail. From figure 3.10 it is also seen that the connection is not capable to reduce the stress at the column face of the beam column joint.

3.4 CAPACITY CURVES

This section presents the comparison of the capacity curves obtained from pushover analysis of the joint for different schemes of connection details. Fig. 3.11 compares the performance of selected connection details through the resulting capacity curves of the joint. The important characteristics of these curves are presented in Table 3.1. This figure (and the table) shows that the load carrying capacity of the joint and the maximum deformation capacity is highly sensitive to the type of connection used. The table shows that ultimate shear force capacity of the joint may vary 267 kN in basic model (welded) to 506 kN in

Scheme 3 (an increase of almost 90%). Similarly, the deformation at collapse is varying from 306 mm in basic model to 627 mm in Scheme 3 (an increase of about 105%).

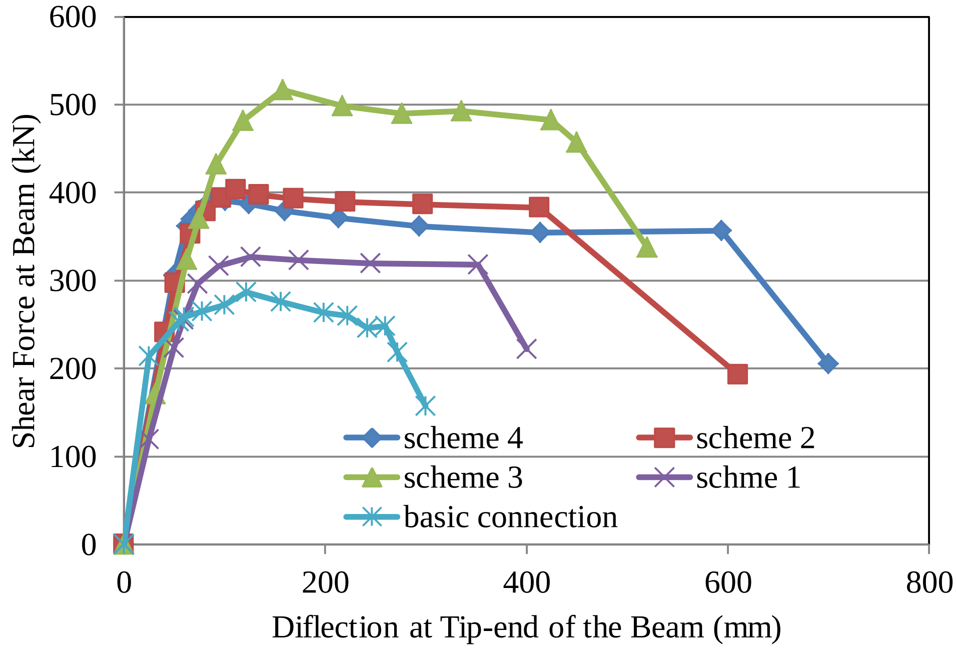

Fig. 3.11: Comparison of capacity curves for different scheme of connections

Table 3.1: Pushover analysis results of the joint for different scheme of connections

Connection Scheme	Maximum Strength (kN)	Yield Deformation (mm)	Ultimate Deformation (mm)	Ductility Factor	Formation of Plastic Hinge at
Basic	267	76	306	3.98	Beam-to-Column Joint
Scheme-1	315	104	413	3.97	Beam-to-Column Joint
Scheme-2	404	100	592	5.92	Beam-to-Column Joint
Scheme-3	**506**	**152**	**627**	**4.20**	**Beam**
Scheme-4	398	98	587	5.99	Beam-to-Column Joint

The results presented in Table 3.1 clearly shows that the performance of the joint with connection details of Scheme 3 performs best among others with respect to the ultimate load and deformation at collapse. Table 3.1 also presents the location of the formation of plastic hinge during the inelastic deformation. This data confirms the effectiveness of Scheme 3 as the plastic hinge form in this scheme in the beam-end away from beam-to-column joint whereas in all other cases the formation of plastic hinges occur in the beam-to-column joint.

CHAPTER 4

SUMMARY AND CONCLUSIONS

4.1 SUMMARY

New hollow steel sections (such as square and rectangular hollow sections) are gaining popularity in recent steel constructions in India due to a number of advantages (such as: high strength to weight ratio, higher efficiencies in resisting forces, better fire resistance properties, Higher radius of gyration, lesser surface area). Unlike the conventional steel sections these hollow sections do not have standard connection details available in design code or in published literature. To overcome this problem the objective of the present study was identified to develop a suitable and economic connection detail between two hollow sections which should be capable of transmitting forces smoothly and easy to be fabricated.

To achieve the above objective, a square hollow beam to square hollow column connection was selected and modelled in commercial finite element software ABAQUS. This model was analysed for nonlinear static (pushover) analysis considering a number of connection details. Following four alternative scheme of connection details were selected for this study: (i) using end-plate, (ii) using angle section, (iii) using channel sections, and (iv) using collar plates. The base model (rectangular hollow beam welded to one face of the rectangular hollow column)

The performance of the selected connection details are compared and the best performing connection details is recommended for rectangular hollow beam-to- rectangular hollow column joints.

4.2 CONCLUSIONS

The important conclusions drawn from the present study are listed as follows:

i) Load carrying capacity of the joint and the maximum deformation capacity is highly sensitive to the type of connection used.

ii) Ultimate shear force capacity of the joint found to vary from 267 kN in basic model (welded) to 506 kN in Scheme 3 (using channel) with an increase of almost 90%. Similarly, the deformation at collapse is varying from 306 mm in basic model to 627 mm in Scheme 3 (an increase of about 105%).

iii) The formation of the plastic hinge is usually found to occur at the beam-to-column joint for all the different schemes of connection details except Scheme 3. Scheme 3 results the plastic hinge in the beam end away from joint.

iv) Performance of the joint with connection details of Scheme 3 (columns jacketed with two channel sections and connected with beam by welding) performs best among others with respect to the ultimate load, deformation at collapse and formation of plastic hinges.

4.3 Scope for future research

This study can be further extended as follows:

1) Present study is based on exterior beam-to-column joints. Similar study can be executed for interior beam-to column joints
2) This study considers equal beam and column sections in the selected joints. It will be interesting to study the responses of the joint with varying dimensions of beam and column sections.
3) Results in this study are based on nonlinear static (pushover analysis). This can be extended to include nonlinear dynamic analyses.

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