NUMERICAL ANALYSIS OF STRESS FORMATION IN REINFORCED CONCRETE BEAM-COLUMN JOINT SUBJECTED TO LATERAL SEISMIC LOAD USING ABAQUS

A Thesis Submitted
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APPROVAL

The thesis titled "NUMERICAL ANALYSIS OF STRESS FORMATION IN REINFORCEDCONCRETE BEAM-COLUMN JOINT SUBJECTED TO LATERAL SEISMIC LOAD USING ABAQUS" submitted by S. A. Al-Rafi, St. No. 115410, Rais-Ibne-Shahid, St. No. 095446 has been found as satisfactory and accepted as partial fulfillment of the requirement for the degree of Bachelor of Science in Civil Engineering.

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DECLARATION OF CANDIDATE

We hereby declare that, undergraduate project work reported in this thesis has been performed by us and this has not been submitted elsewhere for any purpose except for publication.

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DEDICATION

We dedicate this to our parents, teachers, friends and fellow members without whom it was almost impossible for us to complete our thesis work.

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"***In the name of Allah, Most Gracious, Most Merciful***"

All praises are for Almighty Allah (SWT) who has given us the opportunity to work with such a relevant project. Our gratitude and respect to our supervisor **Dr. Hossain Md. Shahin**, Professor, Department of Civil and Environmental Engineering (CEE), Islamic University of Technology (IUT) for his encouragement, erudite advices and extreme patience throughout the course for establishing such a creative project.

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ABSTRACT

Beam-column joint is the most critical structural component in reinforced concrete frame structures. During earthquake high demand for energy dissipation imposed by adjoining structural component may cause failure in structure. So understanding seismic behavior of beam-column joint is crucial to mitigate seismic hazard. Nowadays moment resisting frame is one of the most widely used earthquake resisting structural system. Generally there are three types of moment resisting frames. Among them special moment resisting frame offers the most ductility to endure severe seismic activity. Its stringent reinforcement detailing allows large deformation without significant damage. In this study we have used ABAQUS to investigate stress-strain formation in special moment frame and have compared with gravity load design. The finite element model uses the concrete damaged plasticity approach. For special moment frame the result shows high stress bearing capacity and displacement than gravity load design.

Keywords:

Special moment resisting frame, stress analysis, finite element analysis, ABAQUS.

LIST OF SYMBOLS

u	Displacements at the nodal points.
f	Externally applied forces at the nodal points.
K	The formation of matrix
u_i,f_i	Deflection at thei_{th} node and the force at the j_{th} node
k_{ij}	Global stiffness Matrix
$\dot{\epsilon}$	Total strain rate
$\dot{\varepsilon}^{el}$	Elastic part of the strain rate
$\dot{\varepsilon}^{pl}$	Plastic part of the strain rate
D_0^{el}	Initial elastic stiffness of the material
f_t	Uniaxial tensile strengths of concrete
f_c	Uniaxial compressive strengths of concrete
A_c	Area of the critical perimeter
J_c	polar moment of inertia of the critical perimeter
ɣ	proportion of the unbalanced moment
c	Distance between the centroid and edge of the critical perimeter.
f_c	Concrete strength in MPa.
v_u	The total maximum shear stress acting on the critical perimeter (v_u)

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Chapter 1 Introduction

1.1General

Earthquake is a major challenge for structural engineers. Every year it is estimated that there are 500,000 delectable earthquakes in the world. 100,000 of them can be felt and 100 of can cause damage. These larger earthquakes have wreaked havoc across the world throughout the past causing huge loss of life and property. Many researches have been done to disseminate earthquake resisting knowledge as its importance has piled high. Building design codes have evolved throughout the past century to tackle seismic hazard. Various types of structural systems have borne out of these vigorous endeavors.

Reinforced concrete moment frames are one type of structural system that is widely used to resist seismic forces. The design requirements for these frames have been divided into three categories based on the seismic activity of a building’s location: special moment frames, intermediate moment frames and ordinary moment frames. Ordinary moment frames are located in areas of low seismic activity and follow the standard design practices for flexural members, columns, and members in compression and bending. Meanwhile, special moment frames are used in areas of high seismic activity.

As earthquake resisting techniques are recently adopted many of the existing reinforced concrete buildings are old and designed only for gravity load. They provide less lateral resistance, even less protection against earthquake. Also in many developing and underdeveloped countries proper design practices are not conducted and buildings are designed for gravity load only. These buildings lack necessary lateral resistance, increasing vulnerability in case of an earthquake.

For reinforced concrete frame structures beam-column joint region is the most crucial region and needed proper consideration during designing. Negligence in design and detailing give rise to weak beam-column joint which fail to dissipate large energy imposed by adjoining beams and columns. Such weak beam-column joint may cause the whole structure to collapse even if other components are designed properly. The earthquake at Turkey and Taiwan occurred in 1999 is the perfect example as the resultant catastrophe has been attributed to beam-column joint failure.

Fig 1.1: Structural Damage in 1999 Taiwan earthquake.

Fig 1.2: Beam-column joint failure in a building in Turkey earthquake, 1999

So the buildings designed only for gravity load need to be investigated to better understand their seismic response. **In our thesis we mainly focused on stress formation in beam-column connection in special moment frame and gravity load design frame subjected to seismic loading.**

1.2 Literature Review:

1.2.1 Seismic behavior:

Athanassiadou and Bervanakis (2005) studied the seismic behavior of reinforced concrete building with setbacks designed to capacity design procedure provided by Euro code 8. In their study, two ten story frames with two and four large setbacks in the upper floors respectively, as well as a third one, regular in elevation, have been designed to the provisions of Euro code 8 for the high (H) ductility class and a common peak ground acceleration (PGA) of 0.25 g. All frames were subjected to inelastic dynamic time –history analysis for selected motions. They found that the seismic performance of the studied multistory reinforced as completely satisfactory, not inferior and in some cases even superior of that of the regular ones, even for motions twice as strong as the design earthquake.

Juan Chen, Chengxiang Xu, and Xueping Li (2012) pointed out the seismic behavior of frame connections composed of special-shaped concrete-filled steel tubular columns and steel beam, finite element analyses were performed using ABAQUS compared against experimental data.

Moehle (1984) studied the seismic behavior of four irregular reinforced concrete test structures. These test structures were simplified models of 9-story 3 bay building frames, plane of these structures were introduced by discontinuing the structural wall at various levels based upon measured displacements & distributions of story shears between comprised of moment frames and frame–wall combinations. Irregularities in the vertical frames and walls, it was apparent that the extent of the irregularity could not be gauged solely by comparing the strengths & stiffness of adjacent stories in a structure. Structures having the same stiffness interruption, but occurring in different stories didn’t perform equally.

Pinto and Costa (1995), studied Set-back structures and concluded that the seismic behavior of regular and irregular structures are similar. In their study the amount of discontinuity and the ratio of the base height to the total height were small.

Duan and Chandler (1995) pointed out that both static and modal spectral analyses were inadequate to prevent damage concentration in members near the setback level. This observation support the need for the development of new methods such as the DBD procedure proposed in this work.

Tena-Colunga (2004) studied two irregular (setback and slender) 14-storey RC moment resisting framed buildings, with one or two-bay frames in the slender direction. In this case, structures were designed close to the limiting drift angle of 1.2%, established by the Mexican code. Results obtained through nonlinear dynamic analyses suggested that the slender direction of setback buildings with one-bay frames is vulnerable, contrary to what occurs if a bay is added in the slender direction thanks to the higher redundancy in framed structures. The author concluded that seismic codes should penalize seismic design of buildings with single-bay frames in one direction.

Moehle and Alarcon (1986) presented a combined experimental and analytical study to examine the seismic response behavior of reinforced concrete frame-wall structures. In one of the models, vertical irregularity in the frame-wall system was introduced by interrupting the wall at the first story level. Inelastic dynamic analysis was capable of adequately reproducing measured displacement waveforms, but accurate matches of responses required a trial and error approach to establish the best modeling assumptions. It was observed that in the vicinity of the discontinuity, the elements exhibited a curvature ductility demand 4 to 5 times higher than in the case of the model without any interruption of the wall.

Costa (1990) extended the previous work (Costa et al. (1988)) on seismic behavior of irregular structures. The study was based on twelve, sixteen, and twenty story reinforced concrete building models. They found the following conclusions: the role of a shear wall in a mixed structural system was to distribute the frame ductility uniformly along the height, the interruption of a shear wall in part or for the total height of the structure led to a very irregular distribution of frame ductility, also, significant increase was observed in the first level above the interruption of the shear wall. Below the interruption, the behavior was similar to a regular building. In summary it can be observed that analytical and experimental investigations by previous researchers have identified differences in dynamic response of regular and irregular buildings.

Moehle and Alarcon (1986) carried out an experimental response study on two small scale models of reinforced concrete frame-wall structures subjected to strong base motions by using shake table. One of the test structures, designated as FFW, had two nine-story, three-bay frames and a nine-story, prismatic wall. The other structure, designated as FSW, was identical to FFW except that the wall extended only to the first floor level. Thus the test structures FFW and FSW represent the buildings having "regular" and "irregular" distributions of stiffness and strength in vertical plane respectively. They compared the measured response with that computed by the inelastic dynamic response time-history analysis, inelastic static analysis, elastic modal spectral analysis, and elastic static analysis. Several inelastic response time history analyses were conducted for each test structure. For each analysis, different modeling assumptions were tried in an effort to establish a "best-fit" model. They compared maximum top-floor displacements obtained by the experiments and by different inelastic dynamic and elastic analysis methods.

Ruiz and Diederich (1989) studied the seismic performance of buildings with weak first story in case of single ground motion. They studied the influence of the lateral strength discontinuity on ductility demand at the first story under the action of the acceleration record with largest peak ground acceleration, as obtained on soft soil in Mexico City during the Mexico earthquake of September 19, 1985. A parametric study was carried out for 5- and 12-story buildings with weak first story, and with brittle infill wall in upper stories in some cases and ductile in others. The fundamental periods of these buildings were 0.67 and 1.4 s respectively. They noted that the behavior of weak first story buildings greatly depends on the ratio of the dominant periods of excitation and response, the resistances of upper and first stories, and on the seismic coefficient used for design. The ratio of dominant periods of response and excitation was found to be closely related to the formation of plastic hinges, yielding or failure of infill walls, and to the times of their occurrences.

Valmudsson and Nau (1997) focused on evaluating building code requirements for vertically irregular frames. The earthquake response of 5-, 10-, and 20-story framed structures with uniform mass, stiffness, and strength distributions was evaluated. The structures were modeled as two-dimensional shear buildings. The response calculated from the time-history analysis was compared with that predicted by the ELF procedure as embodied in UBC (1994). Based on this comparison, they evaluated the requirements under which a structure can be considered regular and the ELF provisions are applicable.

1.2.2 Beam-Column Joint:

Paulay (1989) showed, through equilibrium, the internal force distributions of seismically loaded beam-column joints for determining maximum joint shear stresses. He found that the interior cracking. He also showed that beams dilate rather than confine joint cores. Therefore the width 1-7 of the beam relative to the width of the column is irrelevant in terms of joint performance. However, beams that run transverse to the direction of motion provide some beneficial confinement of the joint cores. The performance of interior joints loaded in two orthogonal directions was consistently found to be deficient as compared to the performance of identically reinforced joints in one-way loading.

Zerbe and Durrani (1990) indicated that the slab has a significant effect on the joint stiffness and strength. For joint shear design, it was recommended that the effective slab of twice the beam depth on each side be used. Furthermore the slab participation was found to reduce stiffness degradation and be dependent on the story drift.

Pessiki, Conley, Gergely, and White (1990) tested several LRC beam-column joints with typical reinforcement details found in the Central and Eastern United States which include: (i) lightly confined lap splices in columns just above story level; (ii) discontinuous beam reinforcement in the beam-column joints; (iii) little or no joint confining steel; (iv) construction joints located above and below beam-column joints. However, the specimens did not include transverse beams or slabs, the importances of which were previously discussed. It was observed that pull-out of the discontinuous beam reinforcement was the mode of failure in this joint and the column lap splice (and construction joint) location was not critically damaged. The recorded joint shears with discontinuous beam reinforcement were about 20% smaller than the shears with continuous beam reinforcement.

Bindhu K.R, Sreekumar K.J (2011) experimented on seismic resistance of exterior beam column joint with diagonal collar stirrups. Based on the experimental investigation conducted on exterior beam-column joint under static reverse cyclic loading they found that second specimen having additional beam reinforcements and diagonal collar stirrups at joints exhibits a better performance than the others.

Romanbabu M. Oinam, Choudhury. A. M, and Laskar AI (2012) made an experimental study of beam-column joint with Fibers under Cyclic Loading. Based on the interpretation of result they found that the addition of fibers plays an important role for arresting, delaying and propagating of cracks. There was remarkable increase in load carrying capacity due to addition of fiber. The initial stiffness for fibers specimen increased tremendously. The energy dissipation increased considerably for fibers specimens. The ductility increased tremendous for fibers specimens.

S. S. Patil, S. S. Manekari (2013) studied various parameters for monotonically loaded exterior and corner reinforced concrete beam column joint. Various graphs like load vs. displacement, Maximum stress, Stiffness variations i.e. joint ratios of beam-column joints were plotted. They found that as load increases displacement, minimum stress and maximum stress also increases. For fixed support condition for corner and exterior joint the displacement, minimum stress and maximum stress values are minimum as compare to hinge support condition. The behavior of corner beam column joint is different than that of the exterior beam column joint. As stiffness of the structure changes the displacement, minimum stress and maximum stress changes Non-linearly.

1.2.3 Finite Element Analysis:

NilanjanMitra and Laura N. Lowes (2007) developed and evaluated a model for use in simulating the response of reinforced concrete interior beam–column joints using an extensive experimental data set. The model was built on previous work by

Lowes and Altoontash in 2003, modifying the previously proposed model to improve prediction of response and extend the range of applicability. First, a new element formulation was proposed to improve simulation of joint response mechanisms. Second, a new method for simulating the shear stress-strain response of the joint core was developed. The method assumed that joint shear is transferred through a confined concrete strut and simulates strength loss due to load history and joint damage following yielding of beam longitudinal steel. Third, modifications are made to enable better simulation of anchorage zone response. Comparison of simulated and observed response histories indicated that the new model represents well stiffness and strength response parameters for joints with a wide range of design parameters.

Bing Li, Cao Thanh Ngoc Tran and Tso-Chien Pan (2009) carried out experimental and analytical investigations on lightly reinforced concrete beam-column joints subjected to seismic loading. Five 3/4-scale reinforced concrete beam-column joints were tested to investigate the seismic behavior of the joints. The variables in the tested specimens include column orientations and the presence of slabs on the top of beams. The specimens were subjected to quasi-static load reversals to simulate earthquake loadings. Performance of the test specimens was found to be satisfactory in terms of strength and stiffness up to a DR of 2.0%.

S.H. LUK and J.S. KUANG (2012) investigated the seismic behavior of reinforce concrete exterior wide beam-column connections through computational simulations using ABAQUS, focusing on the load transfer paths and different performances of the joints with conventional and wide beams. They found that wide beam-column joints have lower strengths and stiffness as compared to the conventional beam-column connection. Also found that lesser crack opening occurs in wide beam-column connections. The beam width has significant effect on the load transfer paths in wide beam and joint core. The results also indicated that joint shear stress in wide beam-column connections is higher than that of conventional beam-column ones.

JirawatJunruang, and ViroteBoonyapinyo (2014) performed an incremental dynamic analysis on seismic performance of gravity load design (GLD), intermediate ductile frames (IDF) and special moment frames (SDF). The analytical models they used consider all type of failure mode of column (i.e. shear failure, flexural to shear failure and flexural failure); beam-column joint connection, infill wall and flexural foundation. This study found that the lateral load capacity of GLD, IDF, and SDF building was 19.25, 27.87, and 25.92 %W (W= total building weight) respectively, and roof displacement was 0.89, 1.24, and 1.49 %H (H = total building height), respectively. The average response spectrum at the collapse state for GLD, IDF, and SDF are 0.75 g, 1.19 g, and 1.33 g, respectively. The results show that SDF is more ductile than IDF and the initial strength of SDF is close to IDF. The results indicate that all of frames are able to resistant a design earthquake.

1.2.4 Identification of parameters of CDP model:

Tomasz JANKOWIAK, Tomasz LODYGOWSKI (2005) presents a method and requirements of the material parameters identification for concrete damage plasticity constitutive model. The laboratory tests, which are necessary to identify constitutive parameters of this model, have been presented. Two standard applications have been shown that test the constitutive model of the concrete. The first one is the analysis of the three-point bending single-edge notched concrete beam specimen. The second presents the four-point bending single-edge notched concrete beam specimen under static loadings.

1.3 Observation of Literature review:

Many researches have been done on moment resisting frame and on various types beam-column joint (corner, interior etc.). Seismic behavior of gravity load design is also investigated. Very few research studies have been carried out to evaluate numerically the effects of seismic load on these frames. There have also been detailed studies on real beam-column joints that failed during earthquakes. Shear failure, bond failure, core failures are common type of failures in these studies. Studies containing finite element analysis aimed to predict the behavior of beam-column joints are small in number compared to the studies done by practical examination. Nevertheless, in recent years research activity in this field has been growing. Researchers have a lot of studies perfecting the numerical modeling of reinforced concrete. These studies allow easy time and cost saving analysis of seismic behavior of RCC structures. Though many finite element software are used, seismic response analysis using ABAQUS is very less.

1.4 Objectives & Study:

The objective of this study:

- To perform a numerical analysis, using finite element method, of a beam-column connection subjected to seismic load.
- To understand the stress formation and distribution at the joint.

To achieve the aforementioned objectives, the scope of this research includes:

- To model a reinforce cement concrete (RCC) beam-column joint using a finite element software.
- To study the flexural behavior of the joint subjected to seismic load.
- To compare special moment frame with gravity load design.

Chapter: 2 Numerical Methods:

2.1 Introduction:

Generally, engineering analysis can be classified into two types: Analytical method and Numerical method. Analytical solutions, also called "closed form solutions," are more intellectually satisfying, particularly if they apply to a wide class of problems, so that particular instances may be obtained by substituting the values of free parameters. But they tend to be restricted to regular geometries and simple boundary conditions. Most problems faced by the engineers either do not yield to analytical treatment or doing so would require a disproportionate amount of effort. The practical way out is numerical simulation. For which finite element methods were introduced.

Finite Element Analysis (FEA) was first developed in 1943 by R. Courant, who utilized the Ritz method of numerical analysis and minimization of vibrational calculus to obtain approximate solutions to vibration systems. Numerical solutions to even very complicated stress problems can now be obtained routinely using FEA, and the method is so important that even introductory treatments of Mechanics of Materials - such as these modules - should outline its principal features. The term *Finite Element Method* actually identifies a broad spectrum of techniques that share common features. Since its emergence in the framework of the Direct Stiffness Method (DSM) over 1956–1964, FEM has expanded very fast surging from its origins in aerospace structures to cover a wide range of nonstructural applications, notably thermo-mechanics, fluid dynamics, and electromagnetics.

2.2 Finite Element Method:

Finite element analysis is a numerical method for solving partial differential equation as well as integral equations generated from complex structure. It starts the analysis by dividing the interested object into a number of non-uniform regions (finite elements) that are connected to associated nodes as shown in Fig

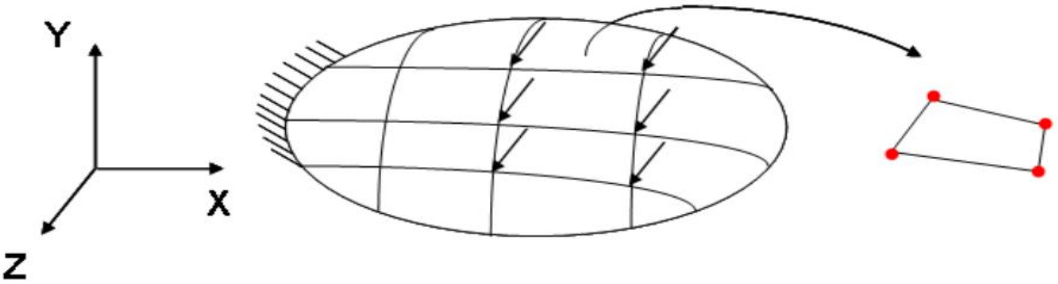

Fig2.1: Finite element

For each typical element, there exit dependent variables at the nodes such as displacement. An interpolation function is defined relative to the values of the dependent variables at the nodes associated with the element. Therefore, for one element, the equation including these variables can be expressed as follows:

$$[K]_e\{U\}_e = \{F\}_e$$

$[K]_e$ is the elementary stiffness matrix, which is determined by geometry, material property and element property.

$\{U\}_e$ is the elementary displacement vector, which describe the motion of nodes under force.

$\{F\}_e$ is the elementary force vector, which describe the force applied on element.

The functions of all the elements are assembled into global matrix equation (governing algebraic equations) to represent the object we study as shown in Fig

$$[K]\{U\}= \{F\}$$

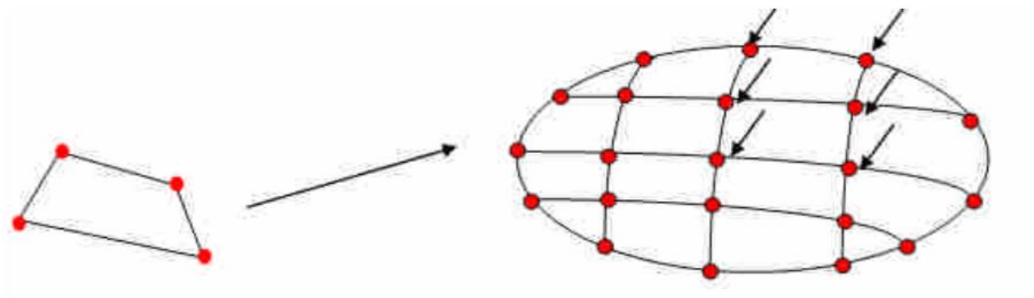

Fig2.2: Assembly of global element

After applying boundary condition, the governing algebraic equation can be solved for the dependent variable at each node.

2.2.1 Matrix analysis of trusses

Pin-jointed trusses provide a good way to introduce FEA concepts. The static analysis of trusses can be carried out exactly, and the equations of even complicated trusses can be assembled in a matrix form amenable to numerical solution.

Matrix analysis of trusses operates by considering the stiffness of each truss element one at a time, and then using these stiff-nesses to determine the forces that are set up in the truss elements by the displacements of the joints, usually called nodes in finite element analysis.

These equations are conveniently written in matrix form, which gives the method its name:

$$\begin{bmatrix} K_{11} & K_{12} & \cdots & K_{1n} \\ K_{21} & K_{22} & \cdots & K_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ K_{n1} & K_{n2} & \cdots & K_{nn} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ \vdots \\ u_n \end{Bmatrix} = \begin{Bmatrix} f_1 \\ f_2 \\ \vdots \\ f_n \end{Bmatrix} \quad \text{................(ii)}$$

Here $u_i$and f_i indicate the deflection at thei_{th} node and the force at the j_{th} node (these would actually be vector quantities, with subcomponents along each coordinate axis). The k_{ij} coefficient array is called the global stiffness matrix, with the ij component being physically the influence of the j_{th} displacement on the i_{th} force. The matrix equations can be abbreviated as using either subscripts or boldface to indicate vector and matrix quantities.

$$K_{ij}u_j = f_i \text{Or} \quad \text{Ku=f} \text{.....................(iii)}$$

Either the force externally applied or the displacement is known at the outset for each node, and it is impossible to specify simultaneously both an arbitrary displacement and a force on a given node. These prescribed nodal forces and displacements are the boundary conditions of the problem.

2.2.2 The Eight-Node Hexahedral ‘Brick’ Element:

Another important element for finite element analysis is hexahedral brick element. Once a part or structure has been designed, it must be ensure that it will perform according to specifications in the real world. The design is turned into a mesh of finite elements. FEA software then tests each finite element for how it responds to such phenomena as stress, heat, fluid flow or electrostatics. For meshing a structure there are three basic approaches to FEA: the h, p and h-p methods. With the method (when the mesh size decreases to zero), the element order (p) (when the element order is increased to infinity) is kept constant. But the mesh is refined infinitely by making the element size (h) smaller. With the p method, the element size (h) is kept constant and the element order (p) is increased. With the h-p method, the h is made smaller as the p is increased to create higher order h elements. Either reducing the element size or increasing the element order will reduce the error in the FEA approximation.

There are different element types like eight-node hexahedrons, four-node tetrahedrons and ten-node tetrahedrons, but eight-node hexahedrons, which part and die designers call “bricks,” lead to more reliable FEA solutions. There are many reasons for the eight-node hexahedral element produces giving more accurate results:

The eight-node hexahedral element is linear (p = 1), with a linear strain variation displacement mode. Tetrahedral elements are also linear, but can have more discretization error because they have a constant strain.

Besides being more accurate, the hexahedral element presents other advantages in FEA model building. Meshes comprised of hexahedrons are easier to visualize than meshes comprised of tetrahedrons.

In addition, the reaction of hexahedral elements to the application of body loads more precisely corresponds to loads under real world conditions.

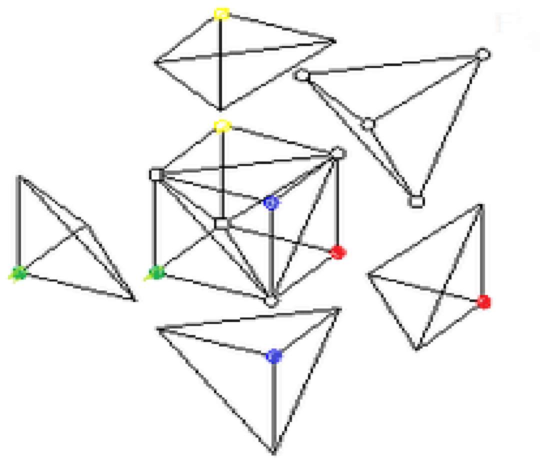

Fig 2.3: Five 4-node tetrahedral elements comprise a single 8-node brick element

2.3 Material Model:

2.3.1 Concrete Damaged Plasticity:

In this method, Concrete displays the characteristics of both a plastic material and a damaging material, it is appropriate to develop models that in corporate both mechanisms of response.

Concrete acts in a brittle manner and the failure mechanisms are cracking in tension and crushing in compression under low confining pressure. The damage in quasi-brittle materials can be defined by evaluating the dissipated fracture energy required generate micro cracks. The main elements of the concrete damaged plasticity model are as under:

Strain rate decomposition is assumed for the rate-independent model as

$$\varepsilon = \varepsilon^{el}\varepsilon^{pl} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(iv)}$$

Where $\dot{\epsilon}$ is the total strain rate, $\dot{\varepsilon}^{el}$ is the elastic part of the strain rate, and $\dot{\varepsilon}^{pl}$ is the plastic part of the strain rate. The stress-strain relations are governed by scalar damaged elasticity:

$$\sigma = (1\text{-d})\, D_{\circ}^{el} : (\varepsilon - \varepsilon^{pl}) = D^{el} : (\varepsilon - \varepsilon^{pl}) \ldots\ldots\ldots \text{(v)}$$

Where $D_{\circ}^{el}$ is the initial (undamaged) elastic stiffness of the material; is the $D^{el} = (1 - d)\, D_{\circ}^{el}$ degraded elastic stiffness; and d is the scalar stiffness degradation variable, which can take values in the range from zero (undamaged material) to one (fully damaged material). Damage associated with the failure mechanisms of the concrete (cracking and crushing) therefore results in a reduction in the elastic stiffness. The usual notions of continuum damage mechanics, the effective stress is defined as:

$$\bar{\sigma} \stackrel{\text{def}}{=} D_{\circ}^{el} : (\varepsilon - \varepsilon^{pl}) \text{ ....(vi)}$$

The Cauchy stress is related to the effective stress through the scalar degradation relation:

$$\sigma = (1\text{-}d)\, \bar{\sigma} \text{ ....(vii)}$$

The fundamental group of the constitutive parameters consists of four values, which identify the shape of the flow potential surface and the yield surface. In this model for the flow potential G, the Drucker-Prager hyperbolic function is accepted in the form:

$$G = \sqrt{(f_c - m.f_t.\tan ß)^2 + q^{-2}} - \bar{p}.\tan ß - \sigma \text{.......(viii)}$$

Where f_t and f_c are the uniaxial tensile and compressive strengths of concrete, respectively ß is the dilation angle measured in the p-q plane at high confining pressure, while m is an eccentricity of the plastic potential surface. The flow potential surface is defined in the p-q plane, where $\bar{p} = \frac{1}{3}\bar{\sigma}.l$ is the effective hydrostatic stress $\bar{q} = \sqrt{\frac{3}{2}\bar{S}.\bar{S}}$ the Mises equivalent effective stress, while S is the deviatory part of the effective stress tensor Ǭ

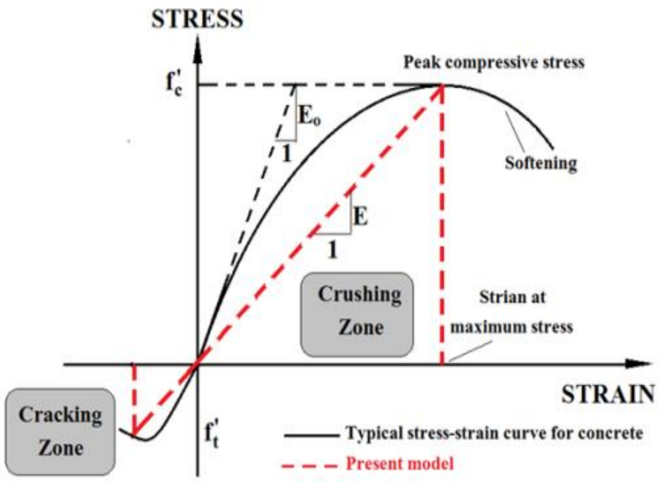

Fig 2.4: Stress-Strain curve in compression for CDP Model

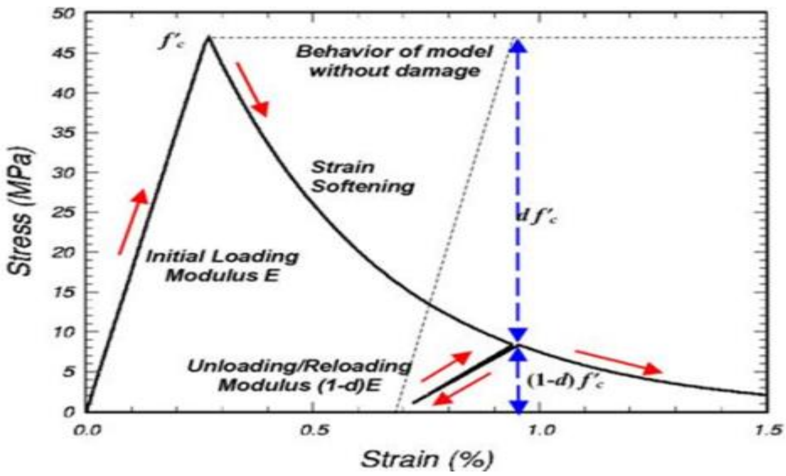

Fig 2.5: Stress-strain curve in tension for CDP Model

2.3.2 Elastic Material Model:

Steel of the reinforcing bars has an approximately linear elastic behavior when the steel stiffness introduced by the Young's or elastic modulus keeps constant at low strain magnitudes. Once the stress in the steel exceeds the yield stress, permanent (plastic) deformation begins to occur. The stiffness of the steel decreases once the material yields. The plastic deformation of the steel material increases its yield stress for subsequent loadings. There is no dependence on the rate of loading or straining.

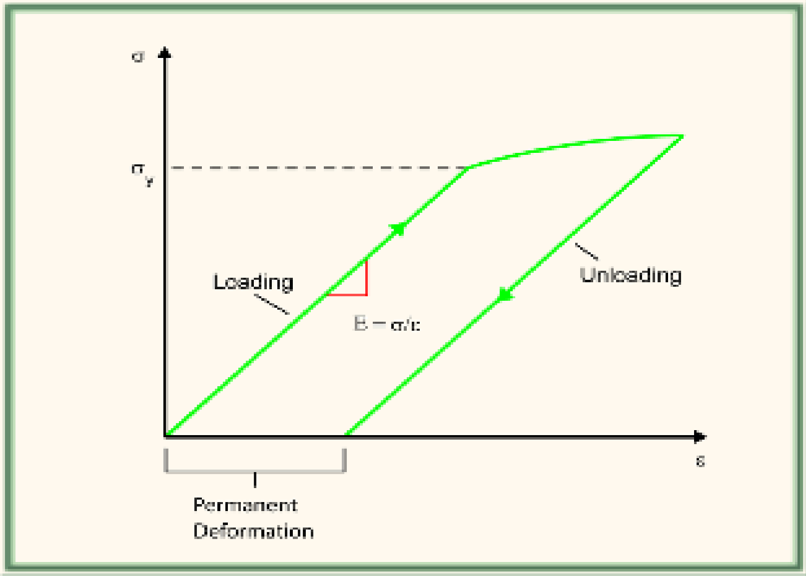

Fig 2.6: Linear Elastic model

2.4 Contact/Interaction:

2.4.1 Boundary Condition:

A key strength of the FEM is the ease and elegance with which it handles arbitrary boundary and interface conditions. Essential boundary conditions in mechanical problems involve displacements (but not strain-type displacement derivatives). Support conditions for a building or bridge problem furnish a particularly simple example. But there are more general boundary conditions that occur in practice:

1. Ground or support constraints:

Directly restraint the structure against rigid body motions.

2. Symmetry conditions:

To impose symmetry or anti-symmetry restraints at certain points, lines or planes of structural symmetry. This allows the discretization to proceed only over part of the structure with a consequent savings in modeling effort and number of equations to be solved.

3. Ignorable freedoms:

To suppress displacements like rotational degrees of freedom normal to smooth shell surfaces.

4. Connection constraints:

To provide connectivity to adjoining structures or substructures, or to specify relations between degrees of freedom. Many conditions of this type can be subsumed under the label *multipoint constraints* or *multi-freedom constraints*.

2.4.2 Embedded Element:

The embedded element technique is used to specify that an element or group of elements is embedded in "host" elements. It is mainly used for reinforcement modeling. In this simulation reinforcements are embedded in the host element "concrete".

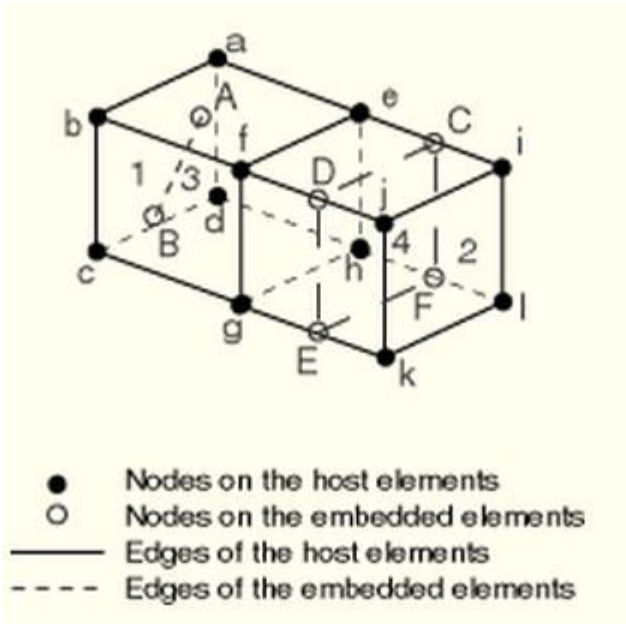

Fig 2.7: Elements lie embedded in host elements

If a node of an embedded element lies within a host element, the translational degrees of freedom at the node are eliminated and the node becomes an "embedded node.” The translational degrees of freedom of the embedded node are constrained to the interpolated values of the corresponding degrees of freedom of the host element. Embedded elements are allowed to have rotational degrees of freedom, but these rotations are not constrained by the embedding.

There is some limitations of using embedded elements:

- Elements with rotational degrees of freedom (except axisymmetric elements with twist) cannot be used as host elements.
- Rotational, temperature, pore pressure, acoustic pressure, and electrical potential degrees of freedom at an embedded node are not constrained.
- Host elements cannot be embedded themselves.
- The material defined for the host element is not replaced by the material defined for the embedded element at the same location of the integration point.

2.5 ABAQUS:

Abaqus FEA is a software suite for finite element analysis and computer aided engineering, originally released in 1978. The name and logo of this software are based on the abacus calculation tool. Abaqus is used in the automotive and aerospace, and industrial products industries. The product is popular with academic and research institutions due to the wide material modeling capability, and the program's ability to be customized. Abaqus also provides a good collection of multi-physics capabilities, such as coupled acoustic-structural, piezoelectric, and structural-pore capabilities, making it attractive for production-level simulations where multiple fields need to be coupled.

Abaqus was initially designed to address non-linear physical behavior; as a result, the package has an extensive range of material models such as elastomeric (rubberlike) material capabilities.

Chapter 3: Methodology

3.1 General

In this thesis two cases have been considered. In first case reinforcement are detailed less stringently and in second case reinforcement is detailed according to special moment frame design. In both cases similar loading and boundary condition is applied.

3.2 Geometry

3.2.1 Beam-Column Joint Modeling

Beam-column joint model has been drawn. Beam dimension is 400*200 and column dimension is 400*400. Concrete damage plasticity type model has been used here. The Beam-column joint acted as the host element for the reinforcement provided.

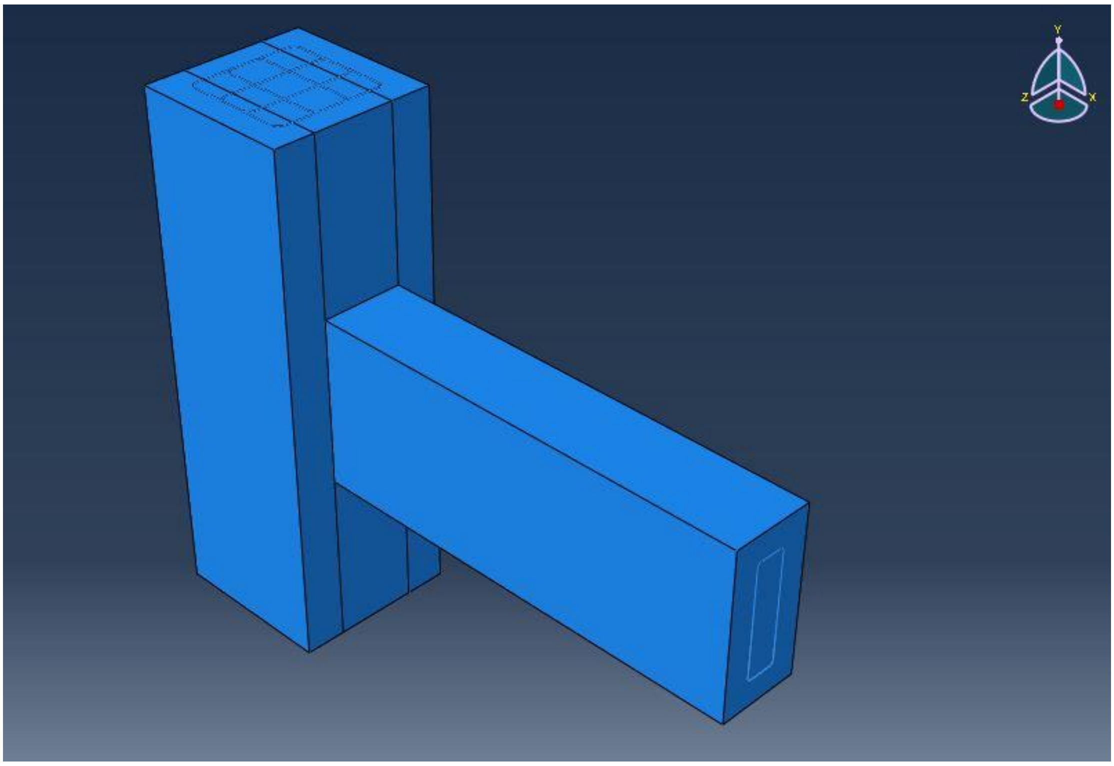

Fig 3.1: Beam-Column joint Concrete Block

3.2.2 Reinforcement Modeling

Reinforcement Modeling has been done. One model represents special moment frame and the other gravity load design. Linear elastic type material has been used. Co-ordinate system has been used to draw the frame. Section assignment has been done for the reinforcement. The reinforcement used as embedded element here.

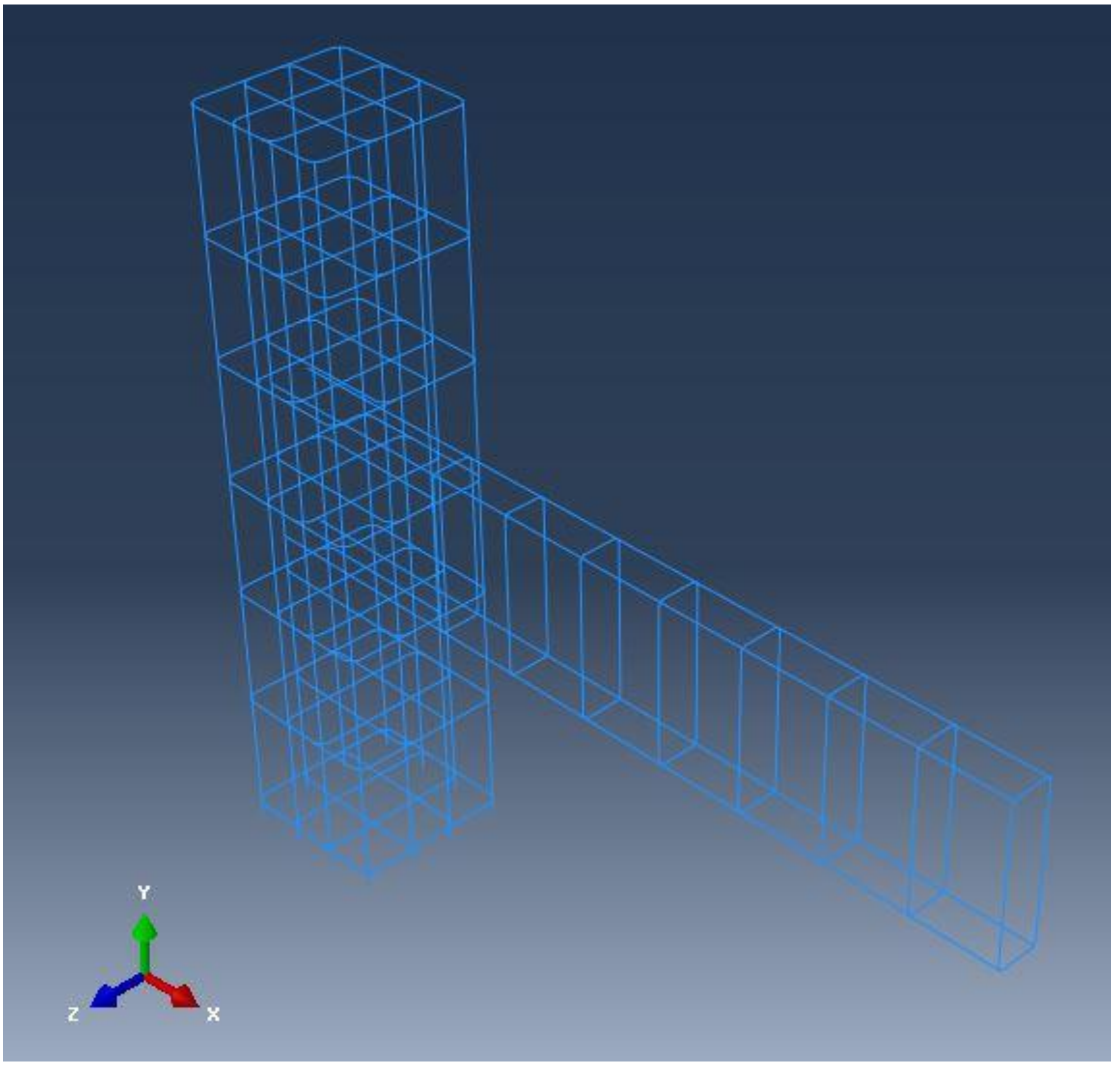

Fig 3.2: Gravity load design Reinforcement Modeling

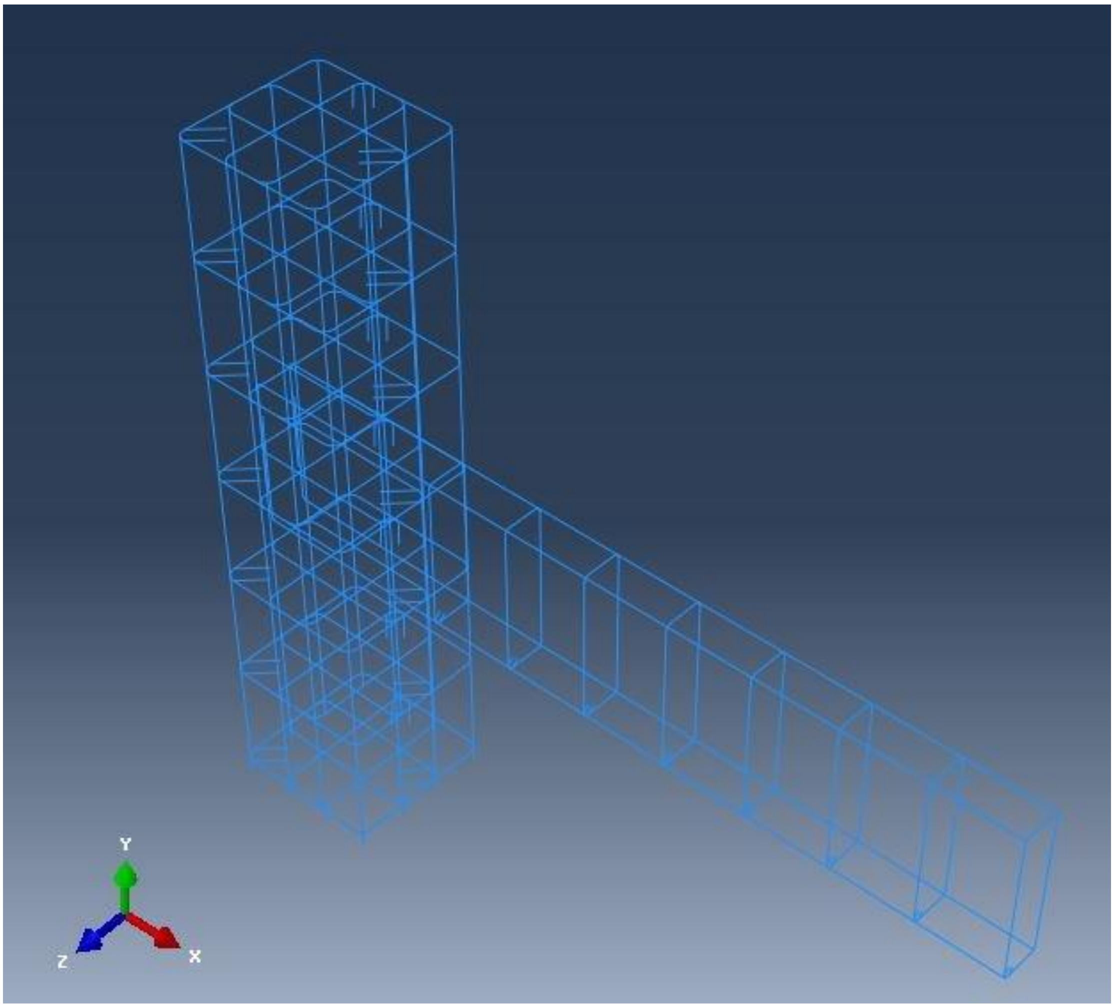

Fig 3.3: special moment frame reinforcement modeling

3.3 Material Modeling

For each model a beam size is placed as 400mm*200mm. And the column size is placed as 400mm*400mm. And the height of the column is placed as 800mm.

Four no10 rebar are used in both column and beam. Shear reinforcement are detailed differently for each model. For both models no 4 bar is used for beam and column stirrups. For special moment frame stirrups have their each end closed off with crossties having 135 degree bends. For gravity load design the crossties are bended 90 degree at the opposite end. Beam stirrups are provided at distance of 100 mm and column stirrups are provided at 100 mm distance.

3.4 Loading and Boundary Condition

For both models same loading and boundary condition is applied. Seismic load is applied at the beam and displacement\rotation type boundary condition is applied at beam and column ends.

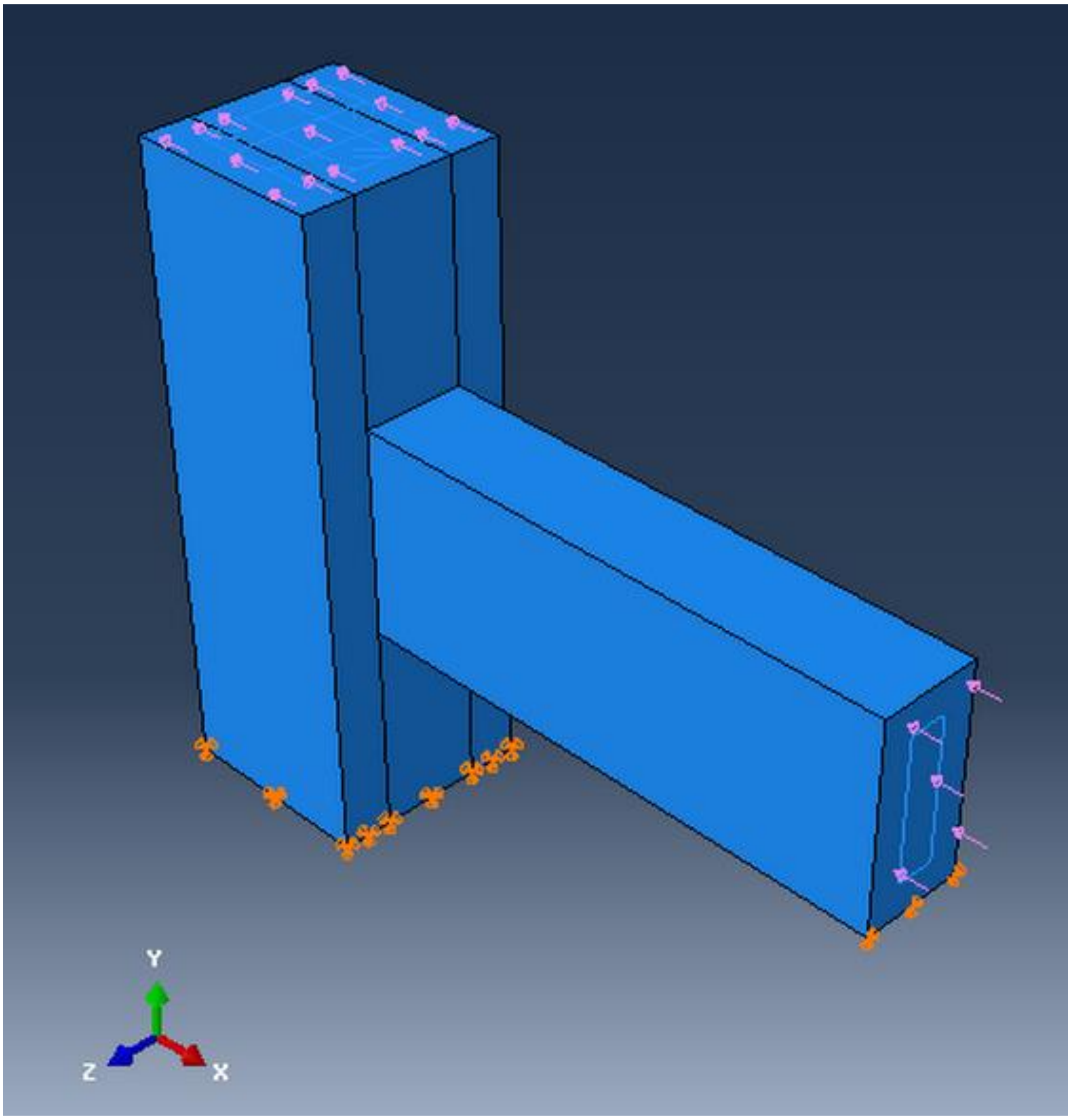

Fig 3.4: Seismic loading and Boundary condition

Seismic loading is applied by using amplitude table. Data for seismic loading is collected from digitized record of acceleration vs time graph.

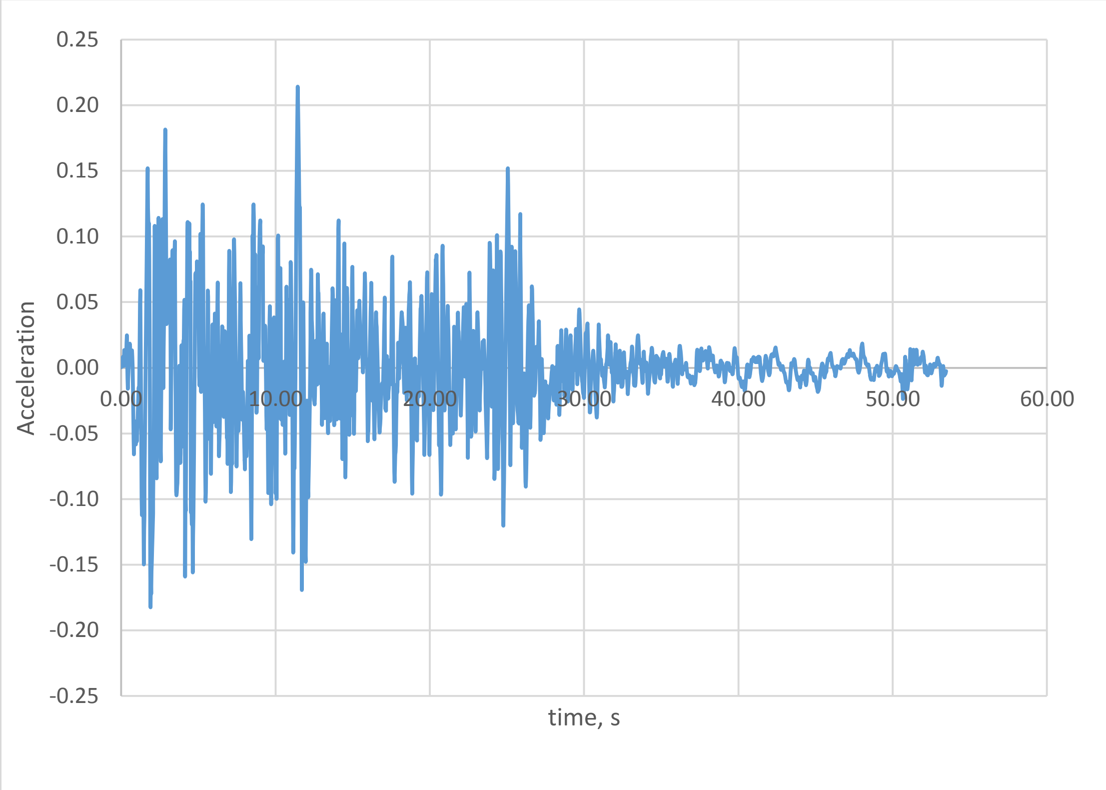

Fig 3.5: Acceleration vs. Time Graph Used for Seismic loading

3.5 Material Properties of Concrete Beam& Column

Table 3.1: Properties of solid slab-column concrete

Density (kg/ mm^3)	Young Modulus (MPa)	Poison's Ratio	Dilation Angle	Eccentricity	FB_o $/FC_o$	K	Viscosity Parameter	Yield Strength (Compressive)	Yield Strength (Tensile)
2.4e^-6	32000	0.23	38	0.1	1.16	.67	8.5e^-5	33	3.3

3.6 Material Properties for Reinforcement Bar

Rebar's density is 7.75E-6 kg/mm^3 And Young modulus 19947.95MPa. Poison ratio was taken 0.3. T3D2 element has been used. Truss elements are rods that can carry only tensile or compressive loads. They have no resistance to bending; therefore it can be modeled as a truss.

3.7 Meshing

In order to obtain accurate results from the FE model, all the elements in the model were purposely assigned the same mesh size to ensure that each two different materials share the same node. The type of mesh selected in the model is structured. The mesh element for concrete is linear 8-node 3D solid which is called C3D8R and for the rebar it is 2-node 3D truss which is called T3D2. Total number of element is 13,245 for SFM and 13,001 for GLD. Total number of node is 15,268 for SFM and 15,020 for GLD.

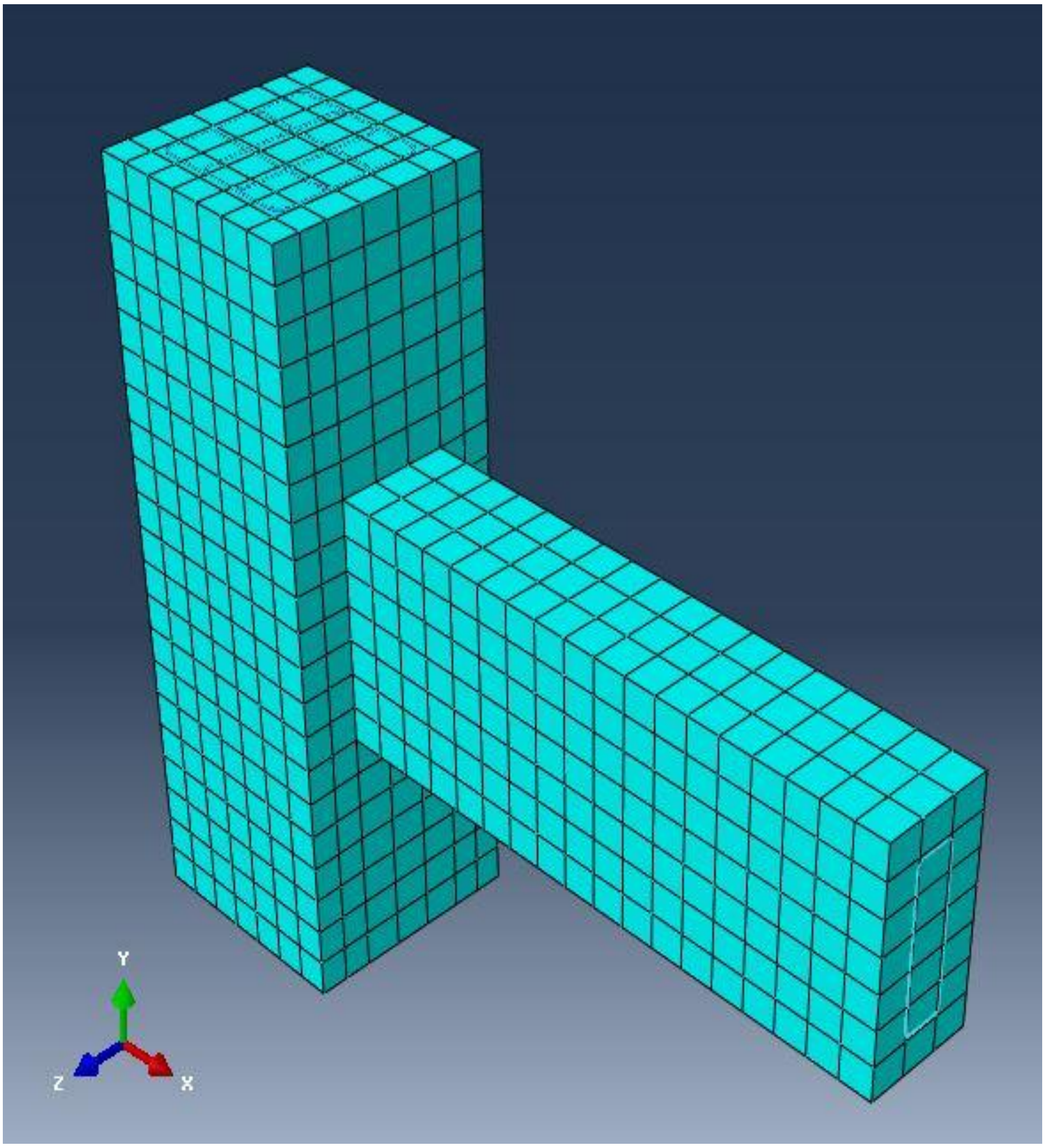

Fig 3.6: Meshing of the Beam-column joint

Chapter 4: Result & Discussion

4.1 General

Comparative studies between special moment frame (SFM) and gravity load design (GLD) have been done here. Von-mises stress, Tresca stress, Max. principal stress, Min. principal stress and displacement has been considered as the point of analysis.

4.2 Von-Mises Stress Distribution

For both models Von-mises stress distribution analysis is done for an element situated near the joint.

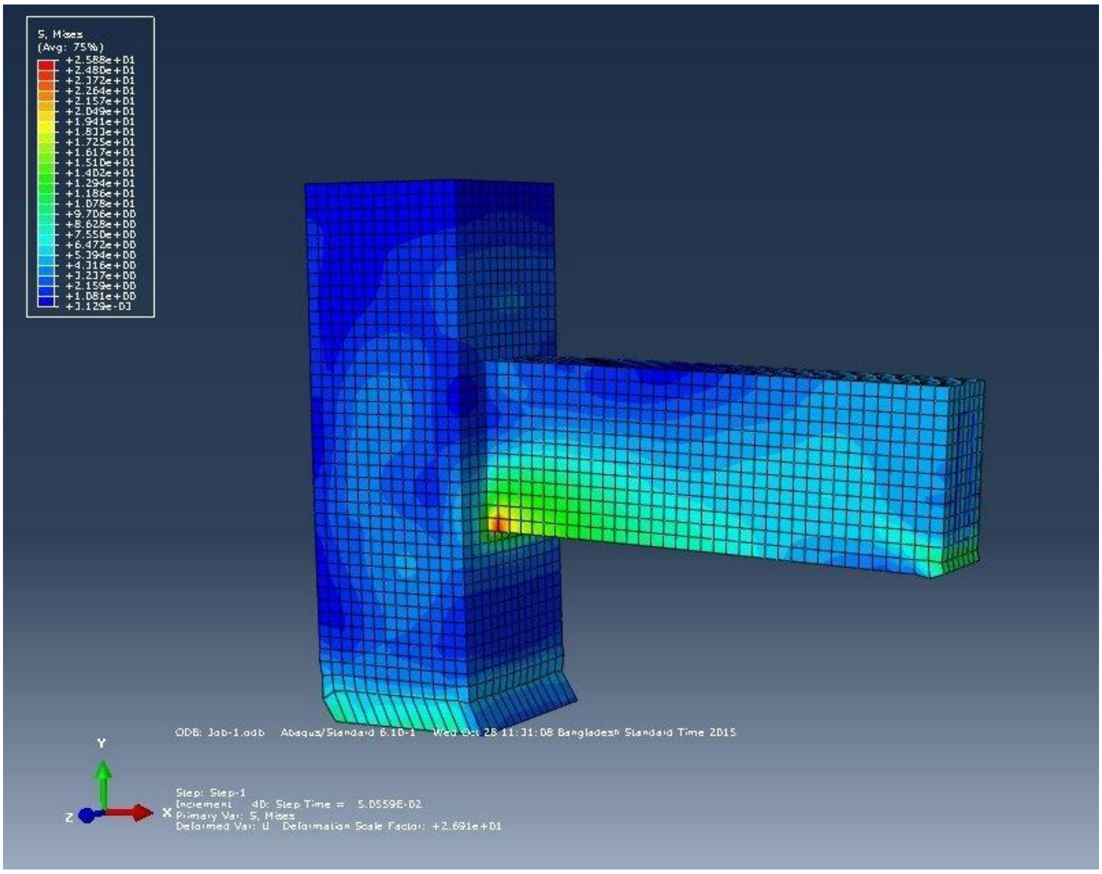

Figure 4.1.1: Von-Mises stress distribution in for GLD

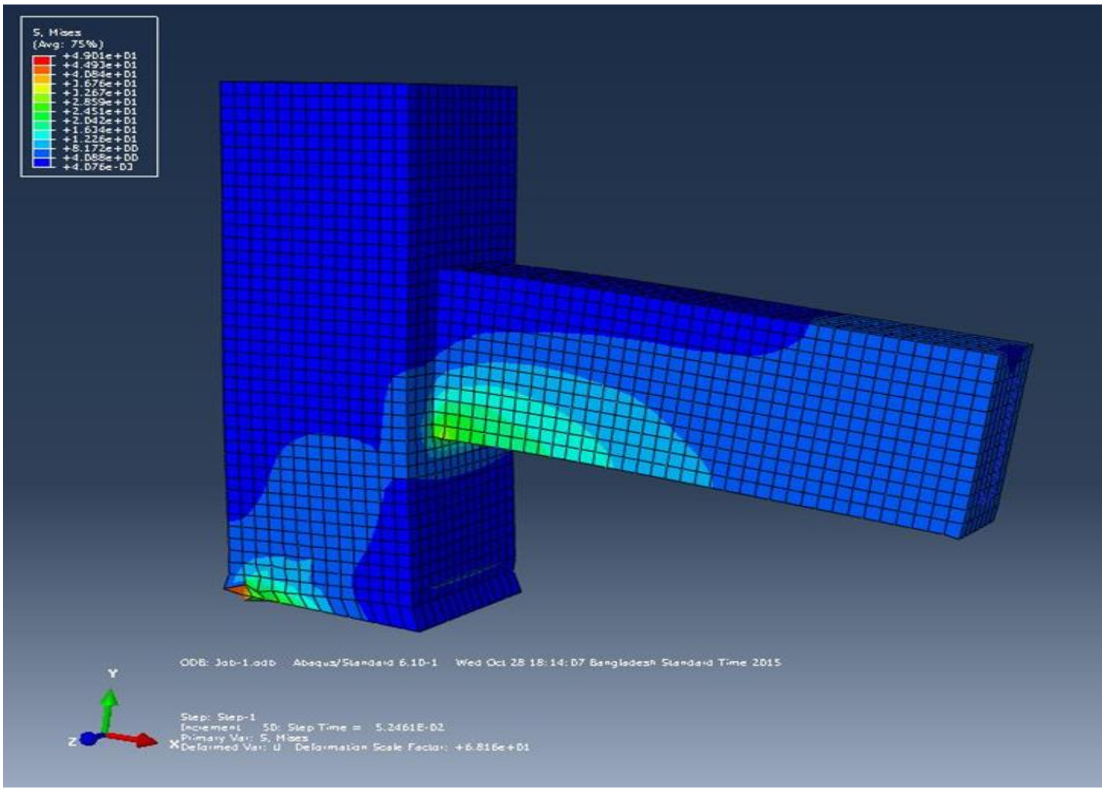

Figure 4.1.2: Von-Mises stress distribution in for SMF

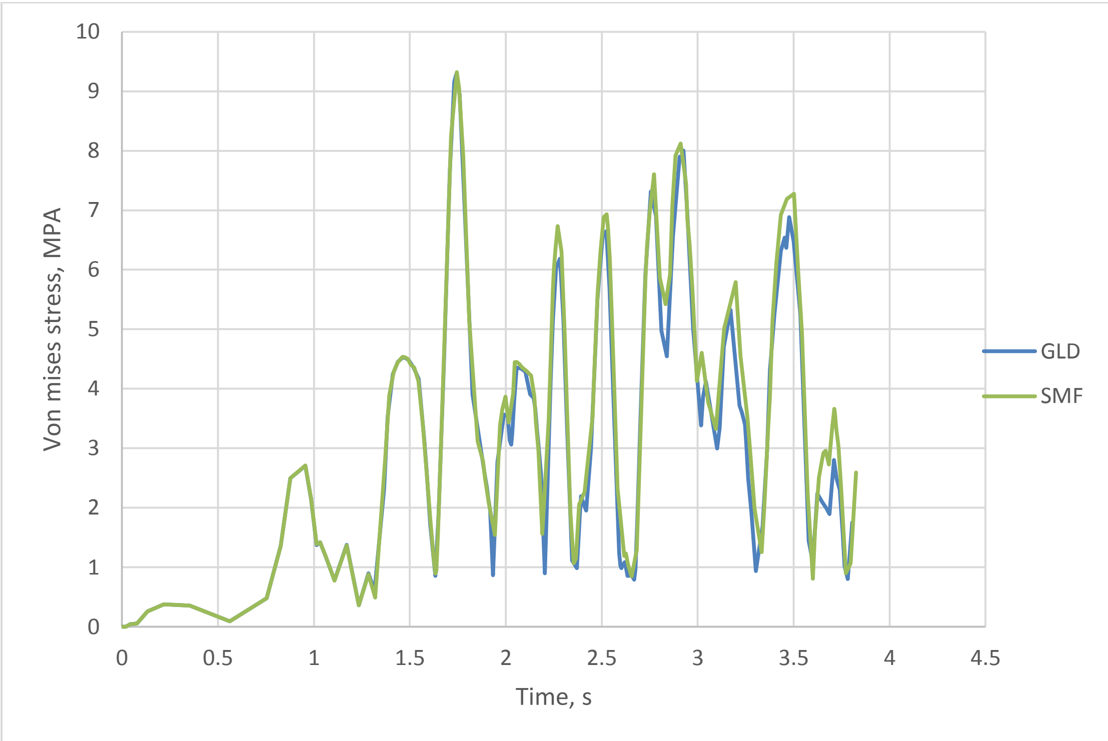

Figure 4.1.3: Von-Mises Stress vs. Time Graph

Initially for both SMF and GLD stress behavior is found same but with time the performance of SMF is found better than GLD.

4.3 Tresca Stress Distribution

For both models Tresca stress distribution analysis is done for an element situated near the joint.

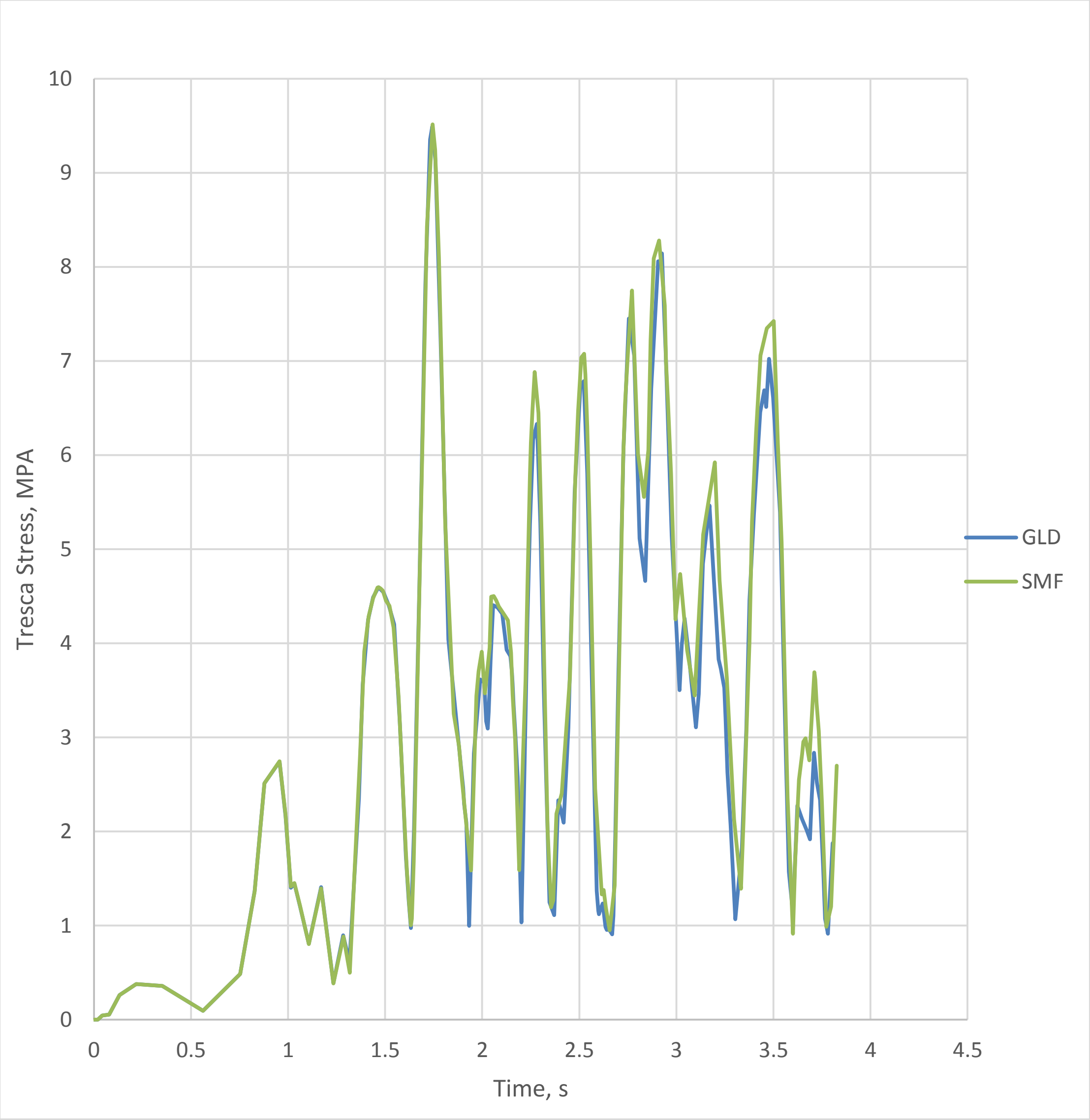

Figure 4.2: Tresca Stress vs. Time Graph

Initially for both SMF and GLD stress behavior is found same but with time the performance of SMF is found better than GLD

4.4 Max. Principal Stress

Max. principal stress is found for both special moment frame and gravity load design and plotted in the same graph for comparison.

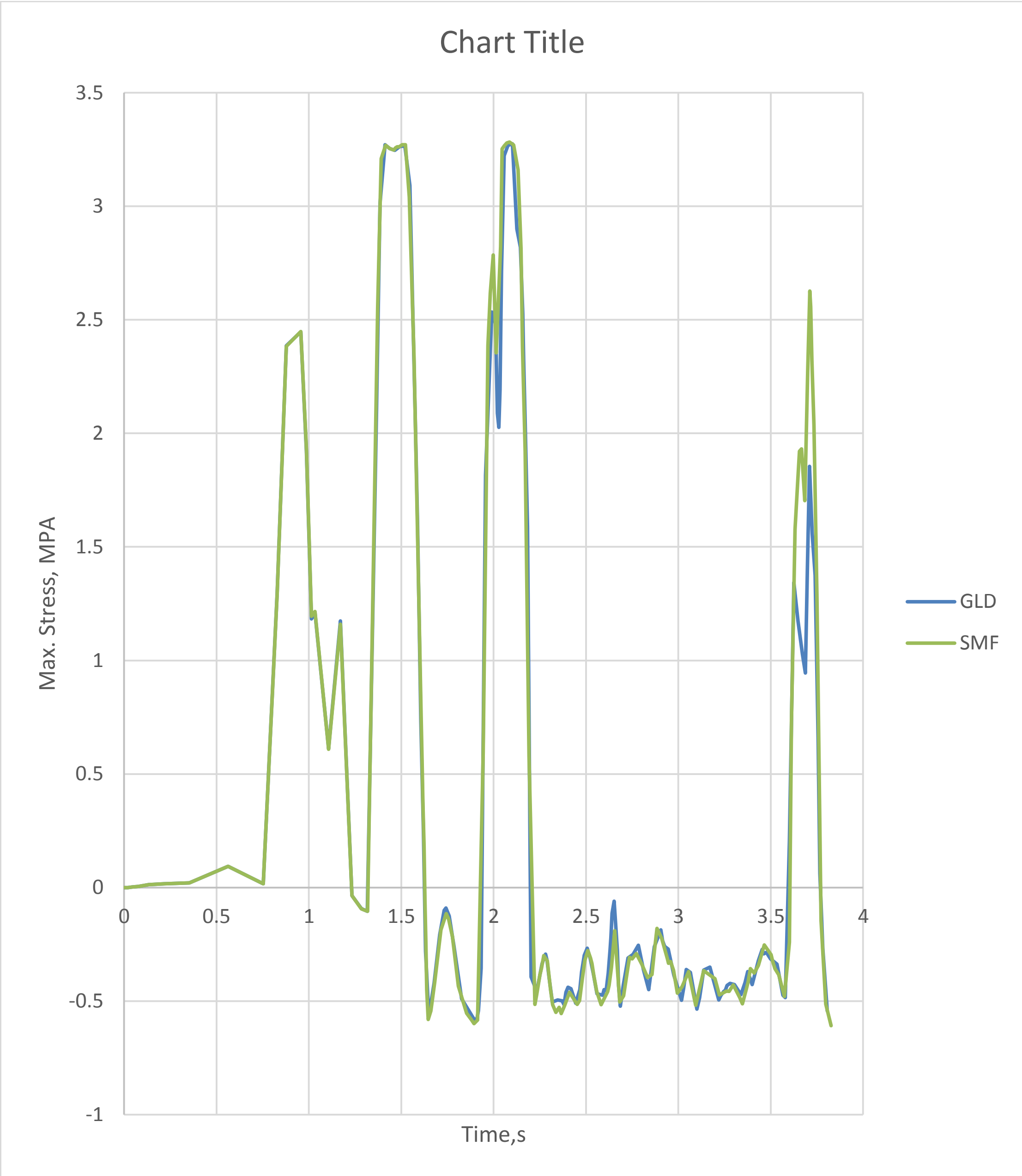

Figure 4.3: Max. Principal Stress vs. Time Graph

Initially for both SMF and GLD stress behavior is found same but with time the performance of SMF is found better than GLD.

4.5 Min. Principal stress

Min. principal stress data for both models is collected and plotted.

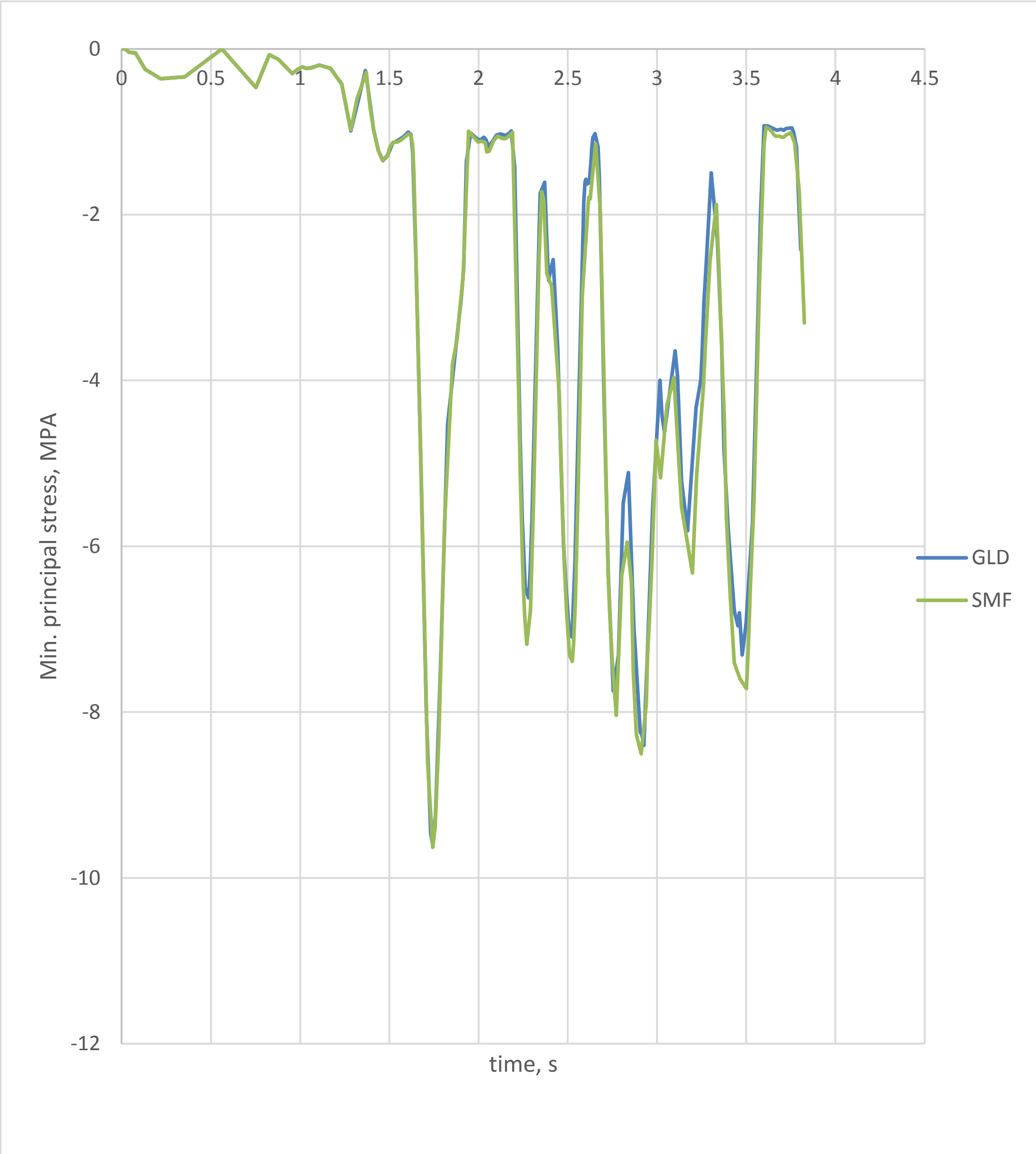

Figure 4.4: Min. Principal Stress vs. Time Graph

Initially for both SMF and GLD stress behavior is found same but with time the performance of SMF is found better than GLD

4.6 Displacement analysis

For both models displacement with time data for a node at the joint is collected and plotted.

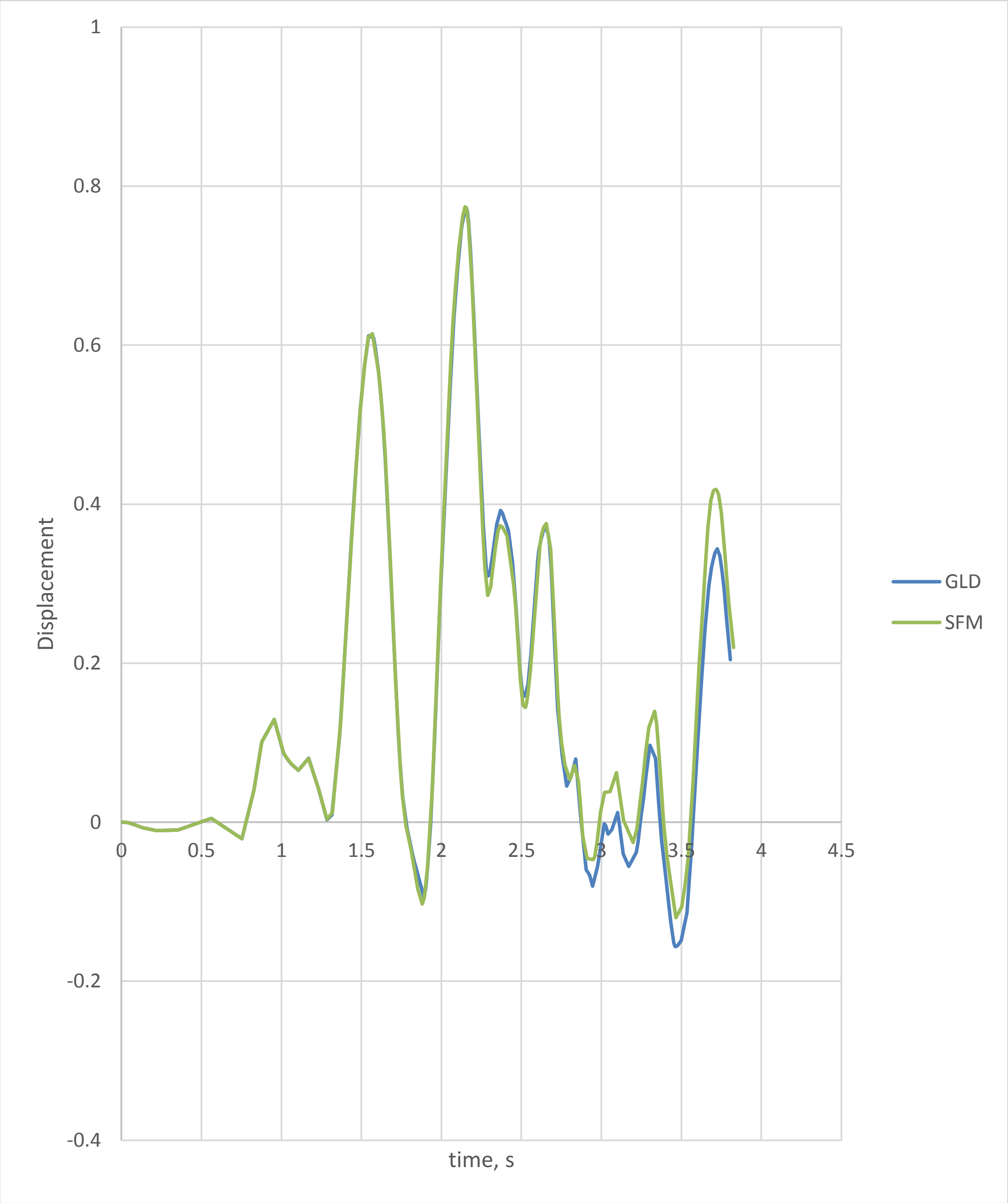

Figure 4.5: Displacement vs. Time Graph

With time the performance of SMF is found better than GLD. The distinction becomes clearer with increase in time. SMF has shown higher displacement than GLD.

4.7 Stress vs Strain analysis

For both model stress vs strain data is analyzed and plotted.

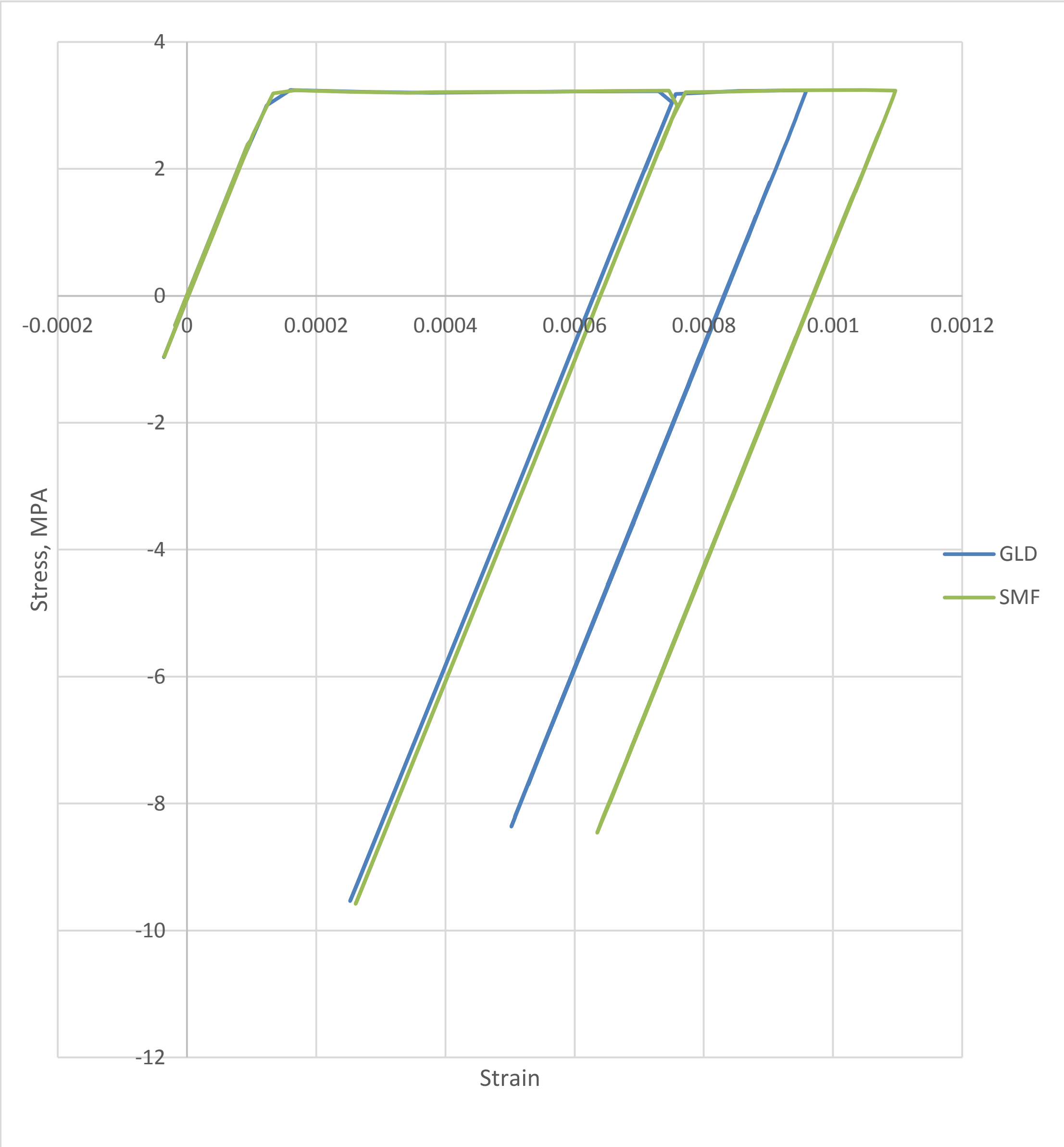

Figure 4.6: Stress vs Strain graph

Like the above results the same is found also true in this case. Stress strain ratio is higher for SMF.

CHAPTER 5:CONCLUSION

In this study stress formation in beam-column joint subjected to seismic load detailed according to special moment frame and gravity load design has been analyzed using ABAQUS. It is found that as the load increases with time special moment frame performs considerably better than gravity load design.

But for small seismic load they almost perform same. So for low risk areas gravity load design is okay but for high seismic zones special moment frame is undoubtedly better. In this simulation the most notable difference between two models are presence of 135° bends on crossties and beam anchorage. So the result may be attributed to them.

Using of the finite element analysis software is an easy solution to determine the behavior of a beam-column connection subjected to seismic loading rather than setting up a practical experiment because it needs more time & money.

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