Modelling Techniques for RC Beam-Column Joint

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Abstract:- Unsafe design and detailing of the beam-column joint jeopardizes the entire structure, even if other structural members fit into the design requirements. A study is conducted considering the importance of joint shear failure which influences strength, ductility, and stability of RC moment-resisting frames, an exterior beam-column joint is modeled considering finite element (FE) using ABAQUS software. Four models MODEL 1 as per IS 13920:1993 provisions, MODEL 2 & MODEL 3 as per IS 13920:1993 provisions with a curtailment of bottom longitudinal rebar in the beam at joint face and MODEL 4 with modification of MODEL 2 by providing extra fillet section of 50*50 mm at the bottom face of the beam-column joint are considered. A 3D Finite Element model capable of appropriately modeling the concrete stress-strain behavior, tensile cracking, and compressive damage of concrete and indirect modeling of a steel-concrete bond are used. To define the nonlinear behavior of concrete material, the concrete damage plasticity is applied to the numerical model as a distributed plasticity over the whole geometry. It was concluded that the ultimate load-carrying capacity of MODEL 1 is 7.5% more than MODEL 2, 15% more than MODEL 3, and 3% more than MODEL 4.

***Keywords*:-** *Beam-Column Joint, Abaqus, Concrete Damage Plasticity, Crack Pattern*.

I. INTRODUCTION

Earthquake is one of the extremely hazardous typical activity which is not predictable and whose effect is immediate as a result of most sudden destruction that a severe earthquake can initiate. Investigation of damages obtained in moment resisting reinforced concrete framed structures exposed to former earthquakes indicates that failure may be due to the poor performance of concrete not having enough resistance, soft storey, the beam-column joint breakdown due to fragile reinforcements, or inappropriate anchorage and column failure resulting storey mechanism.

Experimental and analytical studies show that special confinement specimen carries more load-carrying capacity than the control specimen (Daniel et al., 2018). Study on various parameters for monotonically loaded exterior and corner reinforced concrete beam-column joints shows that the behavior of exterior and corner beam-column joints subjected to monotonic loading is different. (Patil and Manekari,2013). The experimental investigation of one-third-scale precast concrete beam-column connections subjected to reverse cyclic loading showed that the ultimate load-carrying capacity of the monolithic specimen was superior to that of both the precast specimens. (Vidjeapriya et al.,2013). A new design approach for beam-column joints was introduced using advanced reinforcement details minimized damage and considerably improved the structural performance of beam-column joints under cyclic load reversals. (Ha and Cho.,2008). A comparison of the numerical and experimental results to validate the simulation showed that the column axial load made the joint more stiff but also introduced stresses in the beam longitudinal reinforcement. (Haach et al.,2008). Shear strength of beam-column joint predicted by the criterion of initial diagonal cracking is highly dependent on the level of axial loads applied on the column; this model gives very good correlations with all the test data in this study. (Kuang and Wong.,2006). The factors impacting the bond transfer within the joint appears to be well related to the level of axial load and the number of transverse reinforcements in the joints. (Uma S. R.,2006). Different parameters like the effect of the material properties, effect of the geometry of connection, the effect of reinforcement, the effect of concrete compressive strength, and joint slenderness parameters are factors influencing the shear strength of different types of connections. (Josef et al.,2004). The calculated results indicated the global behavior of the joint simulated to correlate is well within the experimental observations. The effects of several critical design parameters on the joint behavior can be explored by the means of finite element models. (Li et al.,2003). Exterior RC joint sub assemblages with four details of longitudinal beam bar anchorage and three details of transverse joint reinforcement, all these specimens showed low ductility and poor energy dissipation with excessive shear cracking of the joint core. (Murty et al.,2001).

II. MODELLING

In this study, four models of exterior beam-column joint namely MODEL 1, MODEL 2, MODEL 3, and MODEL 4 are modeled and analyzed using ABAQUS. Out of which MODEL 1 had been carried out by R. Vidjeapriya and K. P. Jaya. MODEL 2 had been modified from MODEL 1 in which the bottom reinforcement of the beam was restricted up to the face of the column. The reinforcement detailing of MODEL 3 is the same as MODEL 2 but with reverse loading condition. MODEL 4 was modified by providing a 50 mm × 50 mm fillet at the bottom part of the beam-column joint face. The cross-section of the column is 100 mm × 100 mm with an overall length of 1200 mm and the beams are of 100 mm × 100 mm with a cantilever length of 550 mm. MODEL 1 was designed according to IS 456 (BIS 2000) and detailed according to IS 13920 (BIS 1993). The material properties of concrete and rebars are given below in Table 1.

TABLE 1 MATERIAL PROPERTIES

S.N.	**Material**	**Grade**
1.	Concrete	M20
2.	Longitudinal Reinforcement	Fe415
3.	Stirrups	Fe250

A. Reinforcement Detailing of MODEL 1

The main reinforcement provided in the beam is 2 No's - 10 mm diameter bars at the top and bottom. The two-legged stirrups are of 3 mm diameter spaced at 25 mm c/c for a distance of 2d, i.e. 200 mm from the face of the column and at 50 mm c/c for the remaining length of the beam. The longitudinal reinforcement provided in the column was 4 No's of 10 mm diameter bars equally distributed along four sides of the column. The column confinements are of 4 mm diameter bars spaced at 25 mm c/c for a distance of 100 mm from the face of the column and 50 mm c/c for the remaining length of the column.

B. Reinforcement Detailing of MODEL 2, 3 & 4

The bottom bar of MODEL 2, 3, & 4 is restricted to the face of the column as shown in Fig. 2 keeping other detailing the same as that of MODEL 1 as shown in Fig. 1.

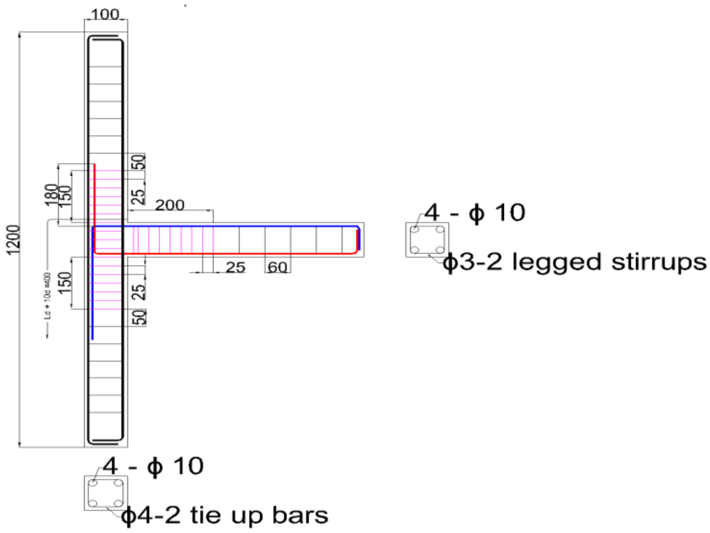


Fig. 1 Reinforcement Detailing of the MODEL 1
(Source: R. Vidjeapriya1 and K. P. Jaya2 2013)

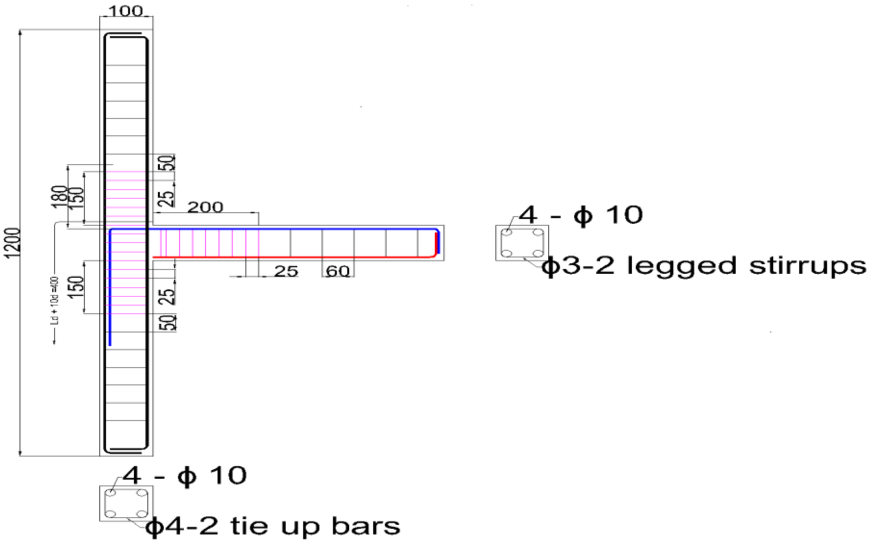


Fig. 2. Reinforcement Detailing of the MODELs2,3 & 4
(Modified MODEL 1 of R. Vidjeapriya et.al)

C. Element Type and Mesh

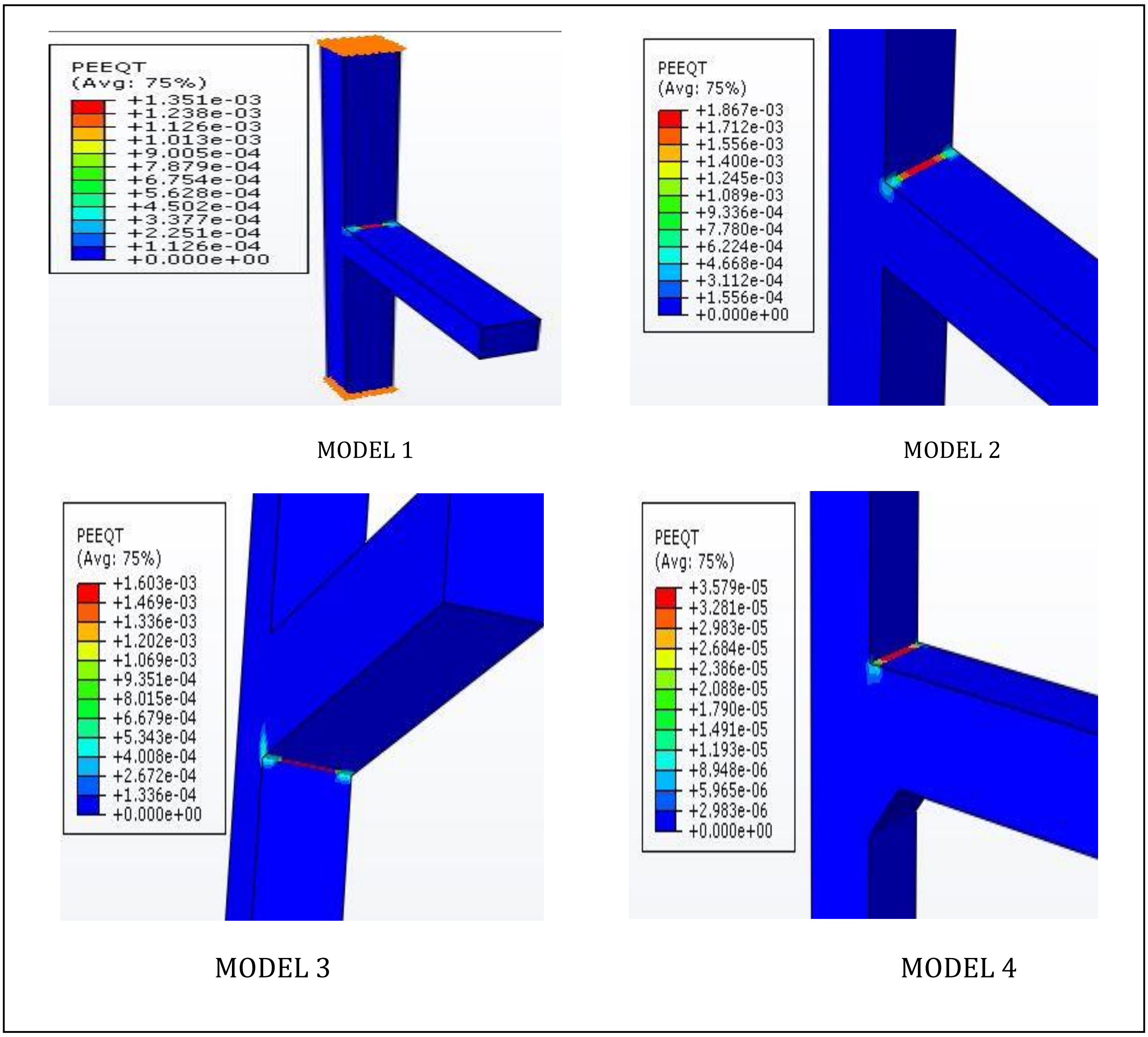
The elements used for the study are C3D8R (8-node linear brick) and T3D2 (2-node linear 3-D truss). The concrete beam-column joint is modeled in 3-D assigned with C3C8R elements and reinforcement in a longitudinal direction, and shear reinforcement bar (Stirrups) is modeled in 2-D was assigned with T3D2 element. Mesh convergence studies were conducted to determine the best balance between accuracy and computational cost. Three element sizes, namely 10 mm, 20 mm, and 30 mm, were considered and a uniform element size of 10 mm was finally selected.

D. Boundary Conditions, Constraints, and Loading

An additional constraint (i.e. EMBEDDED REGION) was employed to tie the degrees of freedom of the truss elements simulating the embedded reinforcing bars to the degrees of freedom of the brick elements of the surrounding concrete. Boundary conditions were restrained to both concrete sections at the top and bottom ends of the column. A monotonic loading in the form of a prescribing displacement history was imposed at the beam end.

III. RESULTS AND DISCUSSION

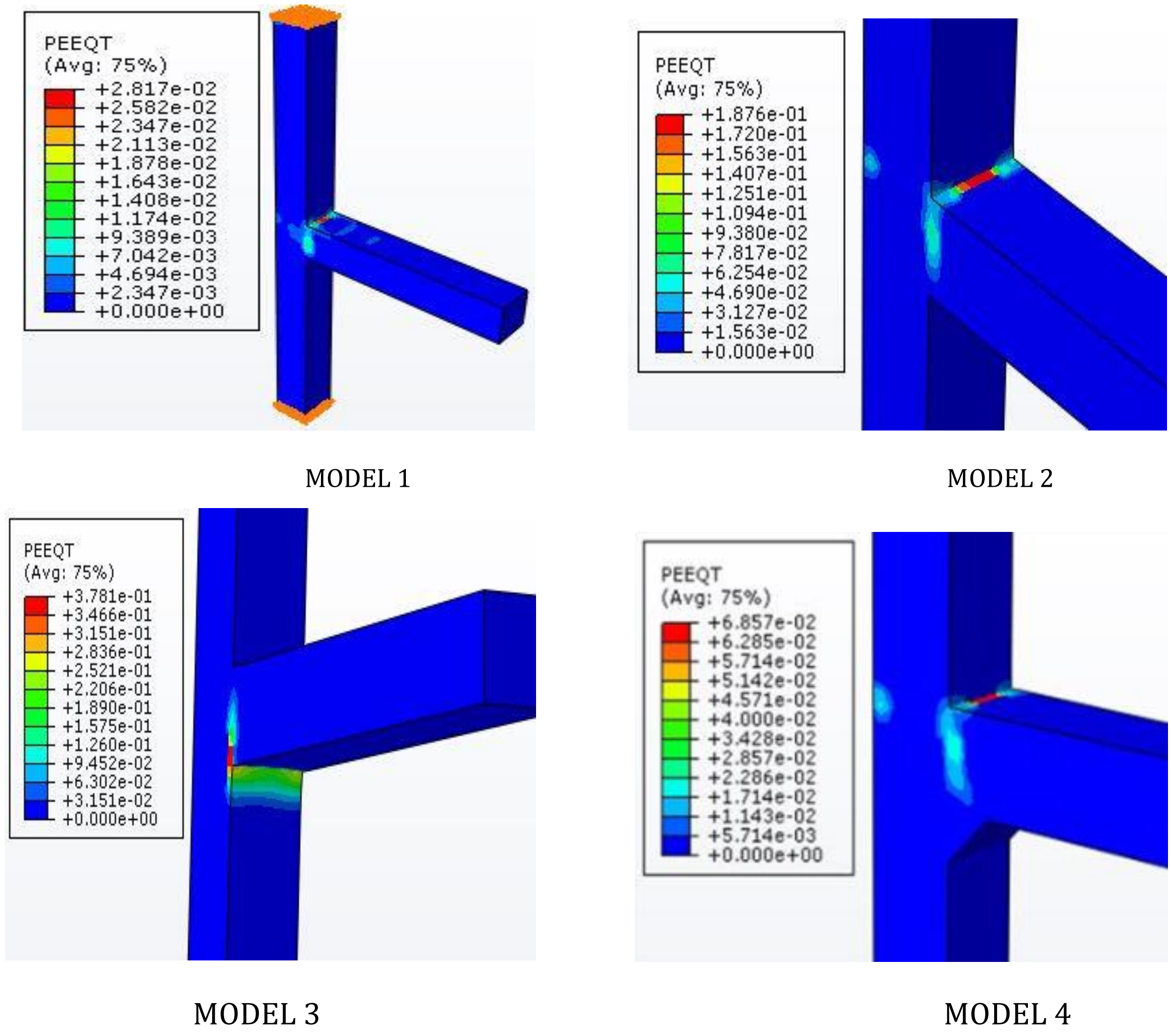
The exterior beam-column joint is studied with different parameters like i.e. Development of crack in the concrete, First crack load, Ultimate load, Load Deflection behavior, Moment Rotation behavior, Concrete Compressive Damage, Concrete Tensile Damage subjected to monotonic loading.The first crack of MODEL 1, MODEL 2, MODEL 3 and MODEL 4 was witnessed at the load of 5.29 kN, 4.58 kN, 3.9 kN, and 5.77 kN respectively when the value of the minimum principal strain reached the limits of concrete compressive strain i.e 0.00135, 0.001867, 0.0016and0.001603



MODEL 1 MODEL 2

MODEL 3 MODEL 4

Fig. 3 Initial crack pattern at joint



MODEL 1 MODEL 2

MODEL 3 MODEL 4

Fig. 4 Yield stage crack pattern at Joint respectively.

As the load level was increased, further cracks were developed in other portions of the beam in the specimens. The crack patterns in the joint of the specimens are shown in Fig. 3 & 4.

The ultimate load carrying capacity is 12.94 kN, 11.97 kN, 10.96 kN, & 12.47 kN and deflection is 8..46 mm, 11.87 mm, 17.4 mm & 8.92 mm respectively for MODEL 1, MODEL 2, MODEL 3, & MODEL 4. As the load level was increased, further deflection increases up to the ultimate load and then after decreases in all those specimens. From load v/s deflection curve in Fig. 5, it is clear that load carrying capacity is more in MODEL 1 than all other specimens. The Ultimate load-carrying capacity of MODEL 1 is observed to be 7.5% more than MODEL 2, 15% more than MODEL 3, and 3% more than MODEL 4.

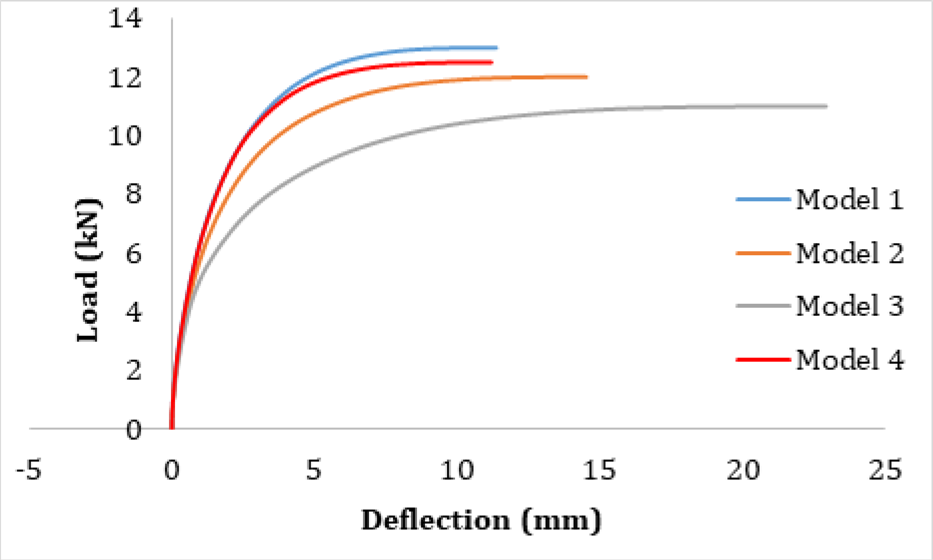


Fig 5. Load - deflection behavior

The ultimate Moment carrying capacity of the MODEL 1, MODEL 2, MODEL 3, and MODEL 4 are 5.2 kN-m, 4.8 kN-m, 4.4 kN-m & 5 kN-m with rotations 0.0008 rad, 0.0016 rad, 0.0019 rad & 0.0012 rad respectively. With the increase of load, the moment developed at the beam-column joint increases up to the ultimate load thereafter decreases in all the models. From Fig. 6, it can be predicted that moment carrying capacity is more in MODEL 1. The Ultimate moment carrying capacity of MODEL 1 is 7% more than that of MODEL 2 and 15% more than that of MODEL 3 and 4% more than that of MODEL 4.

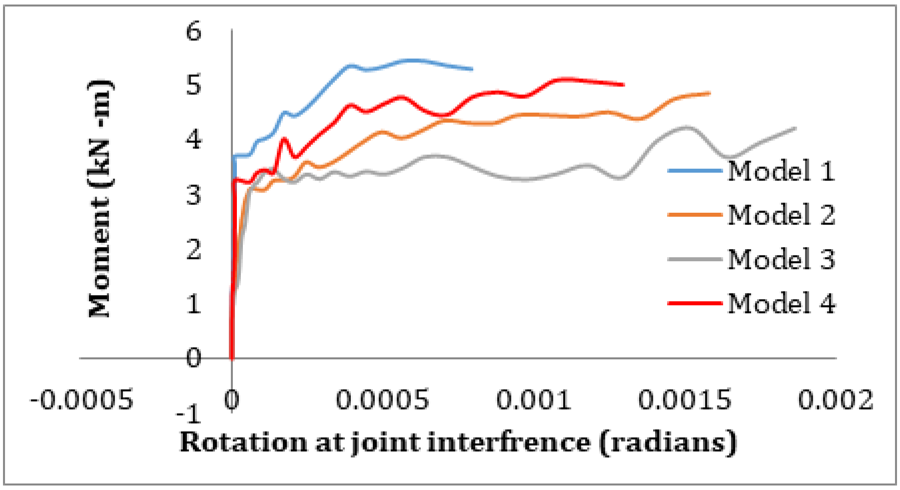
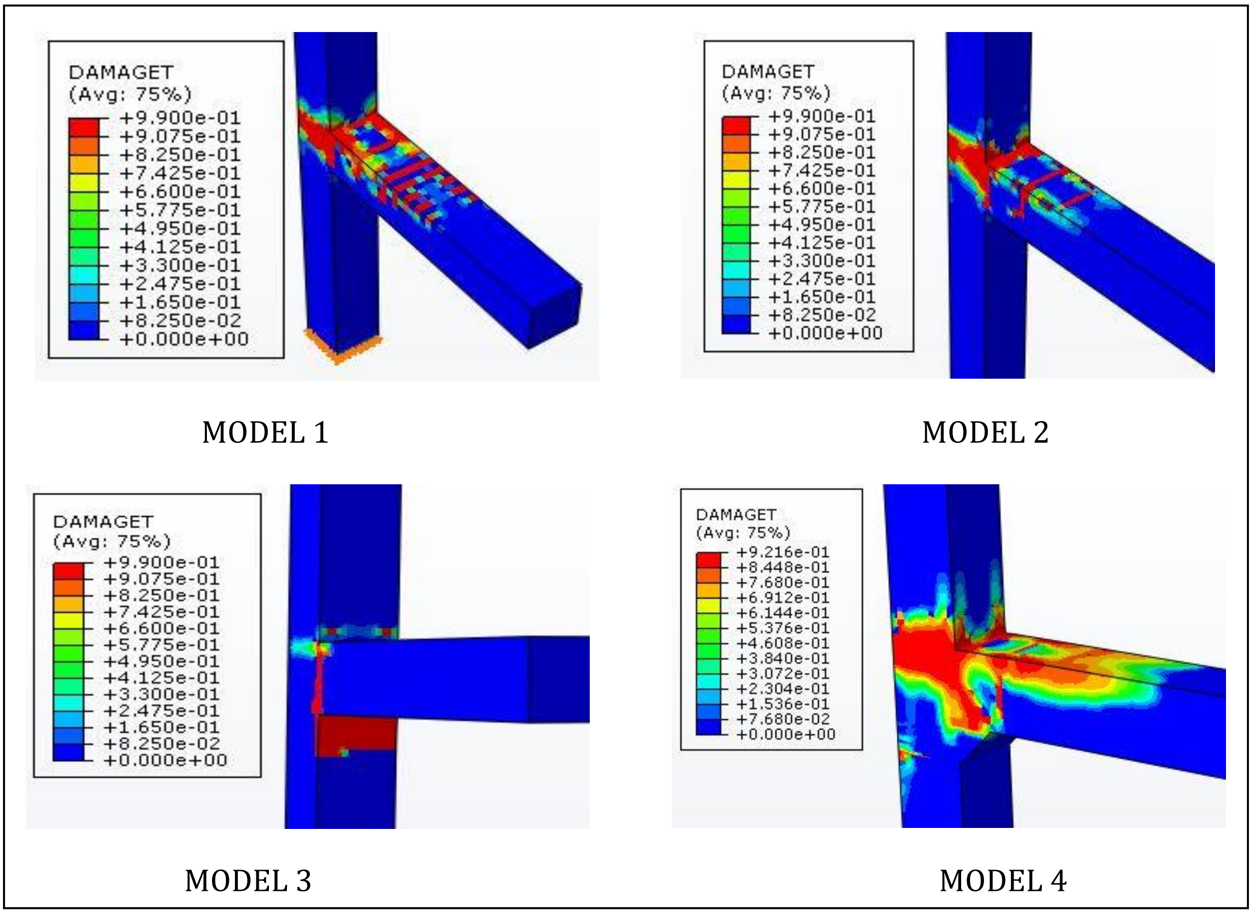


Fig. 6. Moment - rotation curves

A. *Concrete damage*

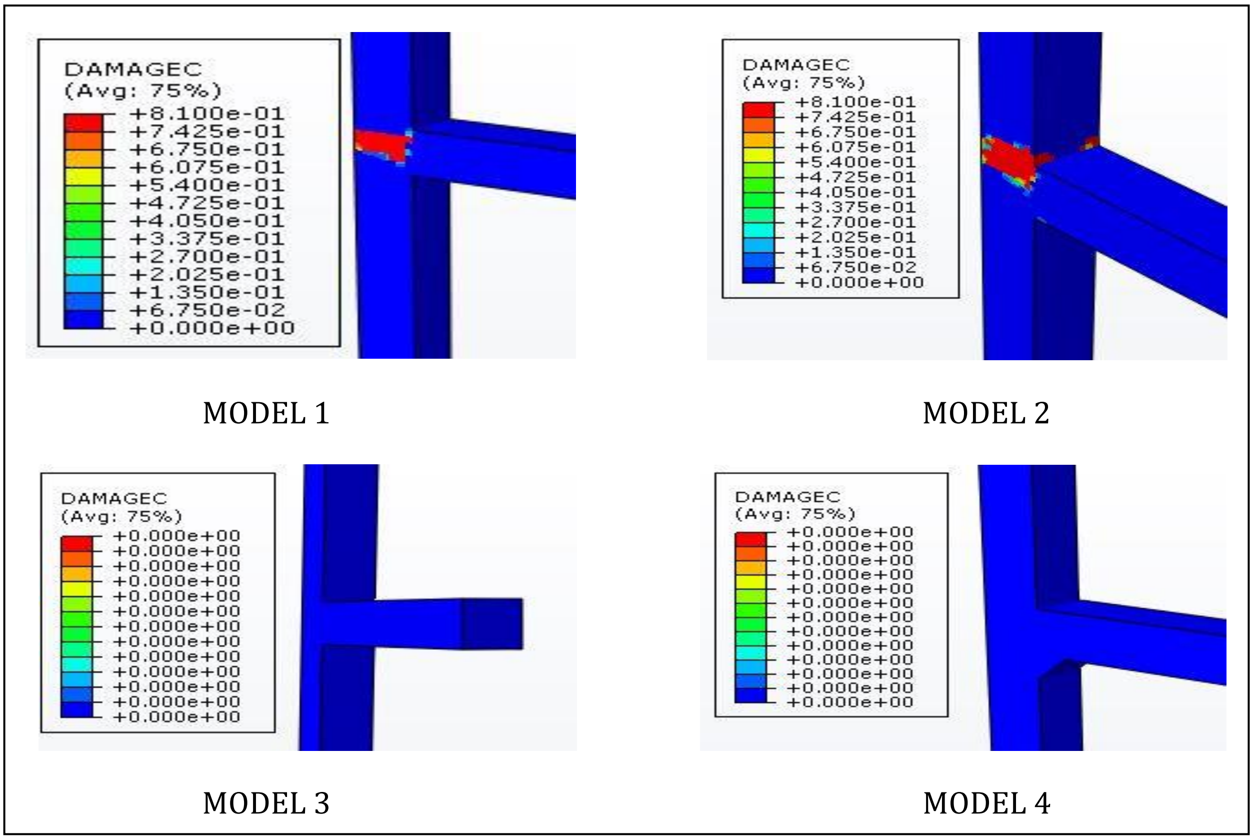
Concrete compressive and tensile damage pattern at the joint is shown in Fig.7 and 8. It is observed that MODEL 1 is failed due to concrete tensile damage and concrete compressive damage in the joint core at the ultimate load. And nearly half of the beam section is damaged starting from the joint face only due to concrete tensile damage. And MODEL 2 is failed due to concrete tensile damage and concrete compressive damage in the joint core at the ultimate load stage, more seriously affected joint core was observed than MODEL 1 and 25% of the length of the beam specimen is only affected due to concrete tensile damage. But Due to reverse loading conditions in MODEL 3 the core of joint is less affected as compared to MODELs 1 & 2 at the ultimate load-carrying capacity. No part of the beam portion is affected due to concrete tension and compressive damage. In this case, when the load is reached to the ultimate stage, the face of the beam-column joint is less damaged in concrete tension only at the joint core. And MODEL 4 is failed only due to concrete tensile damage at the joint when the load reaches to the ultimate stage. This model specimen is safe in concrete compressive damage because the area of the cross-section of the joint face is increased by provided concrete fillet 50mm x 50mm. Therefore the joint resist the ultimate load from compressive damage.



MODEL 1 MODEL 2

MODEL 3 MODEL 4

Fig. 7. Concrete compressive damage pattern at joint



MODEL 1 MODEL 2

MODEL 3 MODEL 4

Fig. 8. Concrete Tensile damage pattern at joint

IV. CONCLUSIONS AND RECOMMENDATION

The study presented herein describes the nonlinear behavior of reinforced concrete exterior beam-column connections when subjected to quasi-static load. A 3D finite element (FE) simulation was introduced for modeling of MODEL 1, MODEL 2, MODEL 3, and MODEL 4 respectively. A concrete damage plasticity material model is applied to the numerical procedure as a distributed plasticity over the whole geometry of the specimens to appropriately simulate material nonlinearity.

Main conclusions from Finite Element (FE) validation results are summarized as follows:

- Constructing the FE MODEL with 3D surface interaction in ABAQUS to simulate the steel-concrete bond is very easy and quick for 3D MODELing comparing to any other methods because it does not involve any additional interface part or elements to represent the bond.
- The first crack load of the MODEL 1 Specimen is 13% more than MODEL 2 and 26% more than MODEL 3. The first crack load of the MODEL 1 Specimen is 9% less than MODEL 4.
- The Ultimate load-carrying capacity of MODEL 1 is 7.5% more than MODEL 2, 15% more than MODEL 3, and 3% more than MODEL 4.
- The Ultimate moment carrying capacity of MODEL 1 is 7% more than that of MODEL 2 and 15% more than that of MODEL 3 and 4% more than that of MODEL 4.
- Concrete damage plasticity model for concrete in ABAQUS is found to be acceptable in MODELling the behavior of the Reinforced Concrete (RC) beams-column joint subjected to monotonic loading.
- The proposed FE MODEL of the beams-column joint can be used to estimate the failure load as well as the failure modes (flexural, shear, or bond failure) with reasonable accuracy and can, therefore, serve as an acceptable numerical tool to investigate the effect of different parameters on the behavior of joint.
- MODEL 1 can be adopted to earthquake-prone zones.
- MODEL 2 and MODEL 4 can be adopted for the lower seismic zones.
- Grade of concrete directly governs the CDP and CDP internally responsible for influencing the crack pattern.
- Any desired MODEL and its behavior can be analyzed and studied in ABAQUS without actual experimental testing which will reduce the time and cost.

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