

INVESTIGATIONS ON THE BEHAVIOUR OF SEMI-RIGID ENDPLATE CONNECTIONS

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ABSTRACT: The present paper describes experimental and numerical evaluation of moment-rotation behaviour of semi-rigid bolted endplate connections. In the experimental programme of research, eight specimens of endplate connections which have end plates extended in the tension side alone are investigated. The column webs are stiffened at the beam flange column interface. Numerical studies are conducted using ABAQUS finite element program for all the eight specimens. The finite element model is validated through a comparison of moment-rotation curves and failure loads obtained from the experimental results. Further, the results of the experimental analysis are used in reassessing the Frye-Morris [4] polynomial model which is recommended in the Indian limit state code for structural steel design, the IS:800 [17].

Keywords: *Semi-rigid connection; moment-rotation behaviour; endplate connection; Frye-Morris model*

1. INTRODUCTION

Connections play an important role in the design of any steel structure. In the past two decades the incorporation of semi-rigid behaviour of steel connection in the design has attracted attention of researchers since the consideration of real behaviour of the connections leads to simpler, reliable designs and better economy in construction. In recent years, endplate connections have gained popularity in steel building construction due to its simplicity of fabrication, and good ductile performance under static and dynamic loadings. A simple endplate connection consists of a rectangular steel plate welded at shop to the beam end, and bolted to the column flange at site, either snug tight or fully pre tensioned. Adjustments can be made to a simple endplate connection to meet the requirements of different situations. For example, stiffeners can be added to maintain the stiffness of a connection while reducing the endplate thickness. A 3D view of an endplate connection is shown in Figure 1. This semi-rigid connection comes in the upper end of the semi-rigid connection moment-rotation behavior spectrum. Even though these connections exhibit high stiffness, the end plate yielding is a major mechanism for strength and yielding.

2. BRIEF REVIEW OF LITERATURE

Extensive research has been directed to the modeling, analysis and design of the end plate connections employing elastic and plastic theories, finite element procedures and full scale testing. Research starting in the early 1950s and continuing to the present, resulted in refined design procedures for end-plate connections. The earlier design methods were based on simple statical assumptions concerning prying forces. These methods resulted in larger end plate thicknesses and larger diameter bolts. Recent studies have used the finite element method and regression analysis to develop design equations. Accurate solutions can be developed using the latter technique.

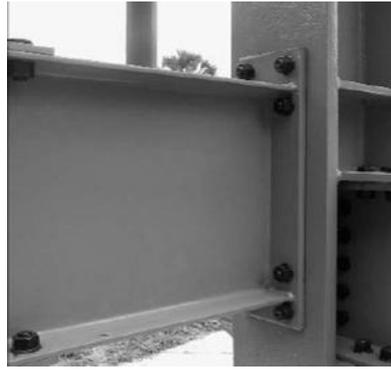


Figure 1. Bolted Extended End-Plate Connection

Prior to 1975, **Doughty** and McGuire [1], Kato and McGuire [2], Nair et al. [3], developed design criteria for the end-plate connections based on the ‘Tee-Stub analogy’. In fact these methods predict a high ‘prying action’ which in turn resulted in large end-plate thicknesses and also large bolt diameters. For seven commonly used connection types, Frye and Morris [4] presented a polynomial model for analysing steel frames with combination of pinned connections, fixed connections, connections with any specified flexibility characteristics. A method is outlined for expressing, in a non dimensional form, the moment-rotation characteristics for all connections of a given type. This method is quite simple to use and it is dependent on geometrical parameter of the connection. Later, Packer and Morris [5] used curved yield lines to predict end-plate capacity with testing directed at the failure of the column flange. Finite element methodology for the analysis of end-plates was first presented by Krishnamurthy et al. [6]. A 2-dimensional/3-dimensional finite element analysis was conducted to determine adequate correlation between results. Krishnamurthy et al. [6] concluded that prying forces do not exist in moment end-plate connections based on this study. Mann and Morris [7] recommended a design procedure for four bolt extended-stiffened end plate connections. The required end-plate thickness was determined using straight yield-line patterns passing through the bolts. The values for end-plate thickness achieved by this design procedure were reported to withstand the plastic moment strength of the beam. Maxwell et al. [8] used simple bending theory, yield line theory, finite difference method and the finite element method to study the endplate and other connections. They concluded that the finite element method was the best feasible alternative for analyzing such a problem, because it will significantly reduce the number of full scale tests normally required to establish behaviour with this type of investigations. Philips and Packer et al. [9] conducted a series of tests to determine the influence of the endplate thickness on moment rotation characteristics and the endplate collapse mechanism. The introduction of yield line theory as the primary analytical tool in predicting end-plate strengths resulted in greater accuracy of predicted strengths, leading to more economical plate design. Zoetemeijer et al. [10] used the yield line theory for analysis of T-stub and end-plate connections with four bolts. In his study, a design method for the tension side of the statically loaded connection is developed. The design methodology is based on two different collapse mechanisms. Tests were performed to ensure that the developed design rules would lead to connections that would satisfy the limit state of deformations. The test results showed a satisfactory agreement with the proposed design rules. Tsai and Popov [11] conducted three tests on four-bolt stiffened endplate connections under cyclic loading. They found that the conventional design procedure for endplate connections subjected to monotonic loading was satisfactory except for bolt sizing. Korol et al. [12] presented seven full scale extended end plate beam to column connections under cyclic loading. Ghobarah et al. [13] carried out five numbers of beam-to-column bolted end plate connections under cyclic loading simulating the earthquake effects on steel moment resisting frame to validate the behaviour of connection as well as the connecting elements. Foley and Vinnakota [14] proposed a three parameter power model for the moment-rotation behaviour of extended end plate connections with

unstiffened columns. More recently, Mofid et al. [15] developed a physically based yield line model to predict the ultimate moment capacity and yield moment of the bolted extended endplate connections, using the connection dimensions as input. The analytical method is proposed for the extended endplate joints having four bolts in tension region, and without any stiffened plate. Comparison is made to a series of test results for a range of bolted endplate moment connections, and good agreement is achieved. Furthermore, this method has been claimed to efficiently serve design engineers in real design condition. Also, Abolmaali et al. [16] developed two types of moment rotation model equations using Finite Element Analysis. The accuracy of the FEM and its results were compared with experimental results. The obtained FEM results were fitted with Romberg-Osgood and Three Parameter Power models.

The Indian Code for general construction in steel IS:800 [17] has proposed the Frye and Morris [4] model for the design of semi rigid connections. In spite of the criticism against this method, as it yields negative derivatives, this method is simple to use and uses the geometrical parameters of the connection. However the exponents of the Frye-Morris [4] model have not been extensively verified by experiments. Various methods may be used for this verification such as empirical, analytical, mechanical, numerical and experimental tests. In the present study it has been proposed to conduct experimental and numerical investigation to assess the Frye and Morris [4] polynomial model.

3. EXPERIMENTAL INVESTIGATION

Eight test cases are identified for experimental investigation on end plate connections by varying the thicknesses of the end plate and the lever arm distance (distance from the centre of top tension bolt to the centre of thickness of the compression flange of the beam). The details of the experimental specimen are shown in Table 1, where ISMB 300 refers to Indian standard medium beam of depth 300mm and the flange width is 170mm. All endplate connections are made using High Strength Friction Grip (HSFG) bolts. The endplate connections are classified with respect to the thickness of endplate and the lever arm distance. The values of the geometric variables for the different endplate connections with HSFG bolt are presented in Figure 2 and Table 2.

Table 1. Endplate Connection Types and Sections Used

No.	Test ID.	Column	Beam	Endplate		Bolt type
				Thickness in (mm)	Length in (mm)	
1	EP-1-8	ISMB 300	ISMB 300	8	400	High Strength Friction Grip bolts of Class 8.8, 16Ø
2	EP-2-8			8	420	
3	EP-3-10			10	440	
4	EP-4-10			10	420	
5	EP-5-10			10	400	
6	EP-6-12			12	400	
7	EP-7-12			12	420	
8	EP-8-12			12	440	

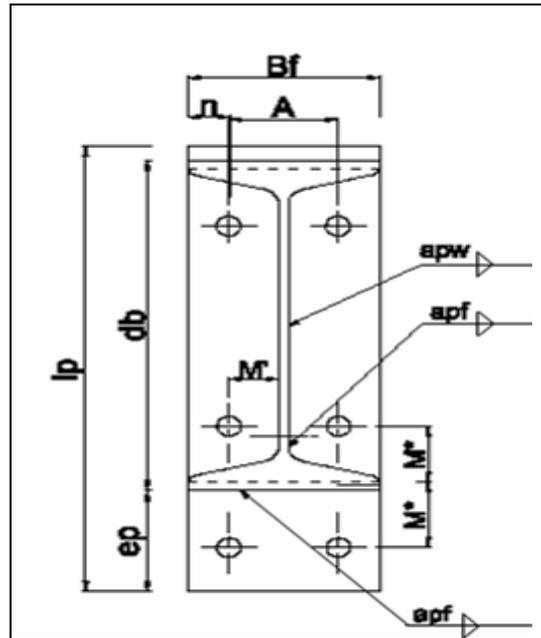


Figure 2. Notations used in the Endplate Connections

Table 2. Geometric Variables of Different Endplate Connections

S. No.	TEST ID.	B_f	t_p	A	M'	N	M^*	d_b	e_p	l_p	d_{bolt}	a_{pw}	a_{pf}
1	EP-1-8	140	8	80	36.25	30	55	300	88.6	400	16	6	8
2	EP- 2-8	140	8	80	36.25	30	75	300	108.6	420	16	6	8
3	EP-3-10	140	10	80	36.25	30	55	300	88.6	400	16	6	8
4	EP- 4-10	140	10	80	36.25	30	75	300	108.6	420	16	6	8
5	EP-5-10	140	10	80	36.25	30	95	300	128.6	440	16	6	8
6	EP-6-12	140	12	80	36.25	30	55	300	88.6	400	16	6	8
7	EP-7-12	140	12	80	36.25	30	75	300	108.6	420	16	6	8
8	EP-8-12	140	12	80	36.25	30	95	300	128.6	440	16	6	8

3.1 Test Setup and Instrumentation

The present experimental test set up consists of two beams connected to a stub column by two endplates connected on either side of the stub column as shown in Figure 3 and Figure 4. All the beams and stub columns used in the test was ISMB300 section of FE 250 steel whose nominal yield strength is 250 MPa. Each beam of length 1150mm is used for all endplate connections. The beam assembly is supported on a roller and hinge assembly. Roller and hinge supports were made up of mild steel. The length and diameter of the roller was 200 mm and 90 mm respectively. The hinge was designed in such way that the support allowed the maximum rotation of 28.03°. Endplates of grade Fe250 were used. By varying its thickness 8mm, 10mm and 12mm, using the combinations with 400mm, 420mm and 440mm length of plate i.e varying only the distance of

tension side bolts from the centre of beam flange by 55mm, 75mm and 95mm. The stub column flanges are stiffened by 8 mm thick stiffeners fillet welded (6mm) to the column web and flanges. This is done to avoid the yielding of column flange, so that the column can be repeatedly used for other specimens also. The beam which is assumed rigid is welded to the endplate specimens on its both sides, so as to use the beam by reverting and then studying a different connection parameter. The stub column of height 650 mm is used. The span of the beam column joint assembly is 2.10 m. A 60 T capacity hydraulic jack is used for applying load over the specimen. The jack is supported by the cross beam of reaction frame.

Load transfer is achieved by pumping highly viscous oil over the piston. Jack is controlled by instrumentation on an electrical panel board. Mild steel plate of size 190mm x 350mm was kept over the specimen to distribute the load evenly over the stub column. The advantage of the present setup is that since the column does not rotate and displaces only up and down, the relative rotation between the column and beam is due to the contribution of the connection deformation alone; hence the relative rotation θ_r can be easily measured using an inclinometer. During testing the verticality of the stub column is checked using plumb-bob.

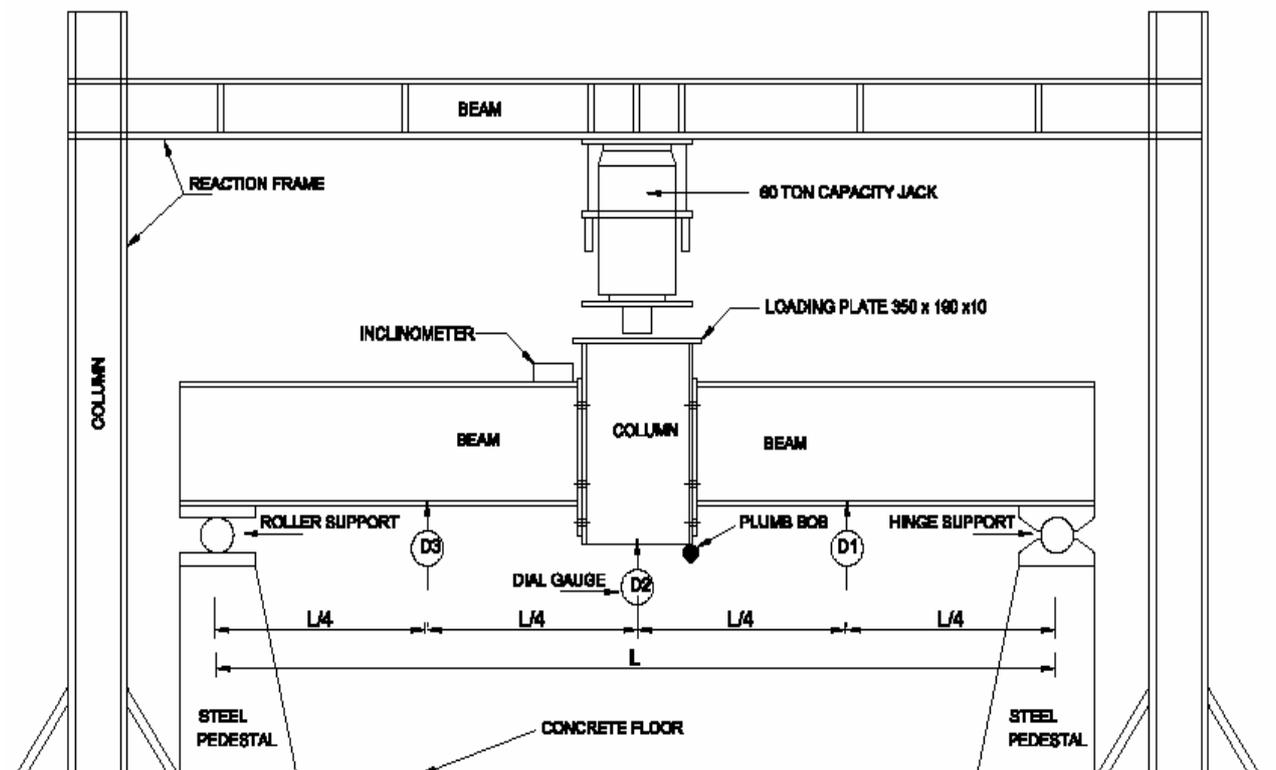


Figure 3. Test Setup and Instrumentation

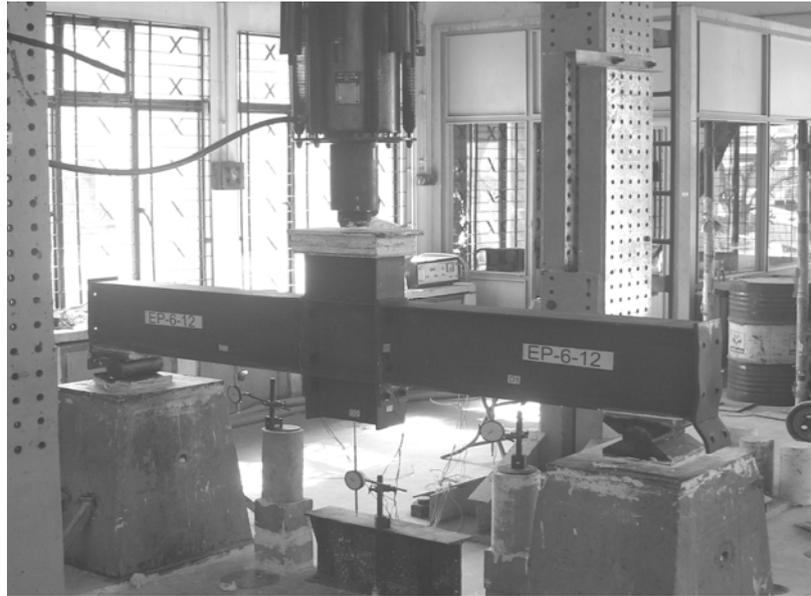


Figure 4. Photographic View of Test Set-Up

The least count of the dial gauge used is 0.01mm. The maximum deflection allowed in one rotation of dial is 25mm. The number of strain gauges is fixed based on the yield line pattern that will develop on the endplate (Mofid et al. [15], Mann and Morris [7]). The major yield lines are as follows (1) along the bottommost tension bolt row, (2) between the top of tension flange weld and the endplate, (3) adjacent to the web weld and the plate, (4) Inclined line from the centre of the bolt of the mid tension bolt row to the mid of the top tension flange weld and (5) inclined line from the centre of the bolt of the mid tension bolt row to the centre of the compression flange of beam. All specimens showed very stiff linear behaviour up to a minimum load of 7.5 tonnes. The contact opening of the endplates with respect to the column flange on the tension side is symmetric on both sides for all specimens and throughout the tests. Initially the neutral axis lies at centre (i.e., between the compression bolt row and the tension bolt row) due to pre-tension and it slowly goes to the compression flange of the beam as the load increases. It can be seen as an opening between the compression bolt row and mid tension bolt row (Figure 5). Deflection is measured at quarter spans from each end and at mid span. Dial gauge located near the roller support is named as D1, dial gauge located near the hinge support is named as D3 and dial gauge located middle span is named as D2. Deflection patterns of D1 and D3 are same. The general observations of the failure mode in all the specimens are that the second tension bolt rows undergo bending along with axial tension.



Figure 5. Contact Opening of Endplate Connection

4. EXPERIMENTAL RESULTS AND DISCUSSIONS

Seven strain gauges are used to measure strain variation; out of which five are fixed in the endplates and two are fixed in the stub column at the tension zone near the stiffener. The strain variation with the applied load for the specimen EP-1-8 is shown in Figure 6. The strains observed in the stub column are very less; hence the effect of buckling of the column flange can be completely neglected.

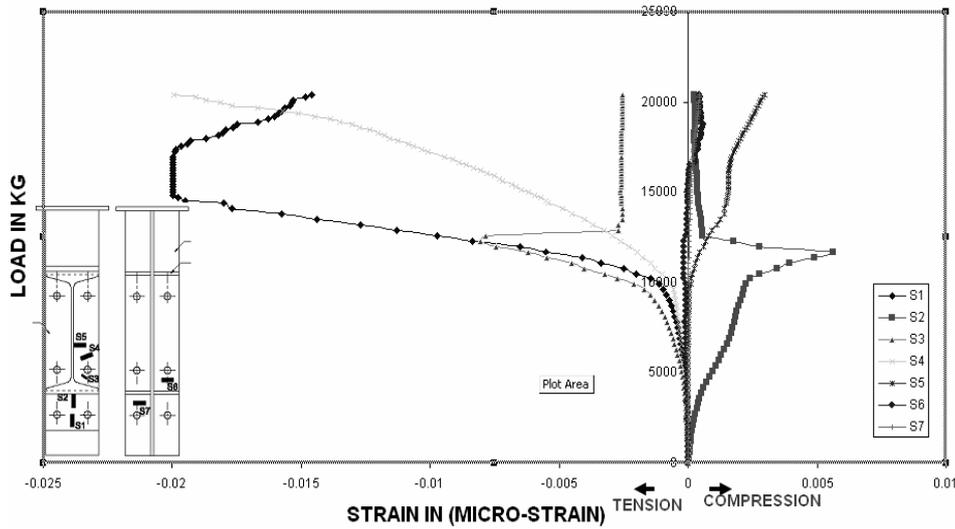


Figure 6. Load vs Microstrain for EP-1-8

The combined moment-rotation and load-deflection curves of all the specimens are shown in Figure 7 and Figure 8 respectively.

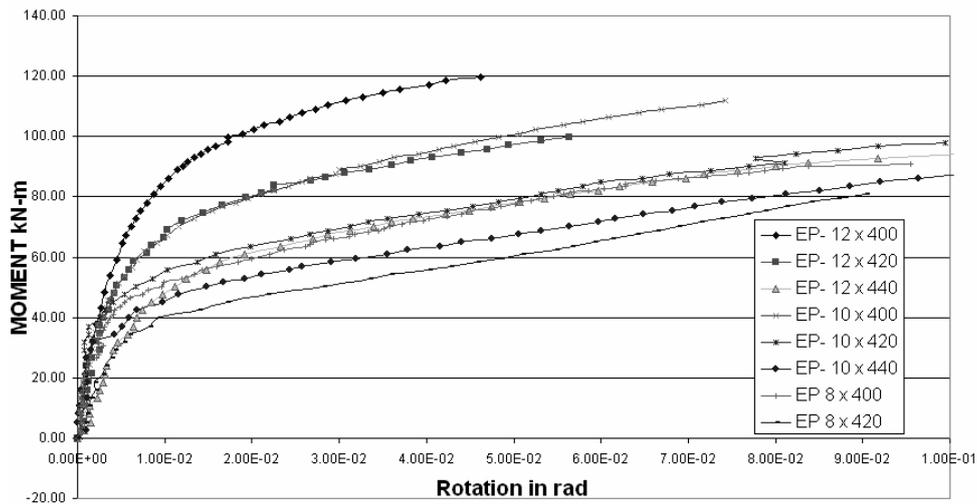


Figure 7. M-θr Relationship of Different Endplate Specimens

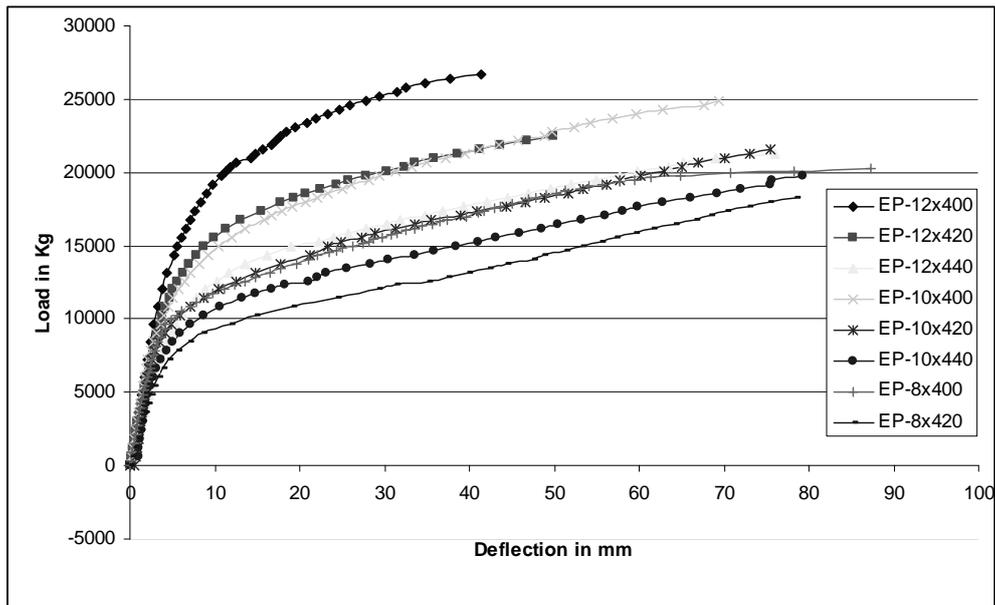


Figure 8. Load-Deflection Behaviour of Different Endplate Specimens

Focusing first on the end plate behavior, the relative displacements between the endplate and the column flange for all the specimens are presented in Figures 9 - 11. The maximum relative displacement of the specimen EP-1-8 is 24mm and for the specimen EP-2-8 is 30mm. The maximum relative displacement of the specimen EP-3-10 is 45mm, for the specimen EP-4-10 is 28mm and for the specimen EP-5-10 is 24mm. The maximum relative displacement of the specimen EP-6-12 is 37mm, for the specimen EP-7-12 is 42mm and for the specimen EP-8-12 is 28mm.

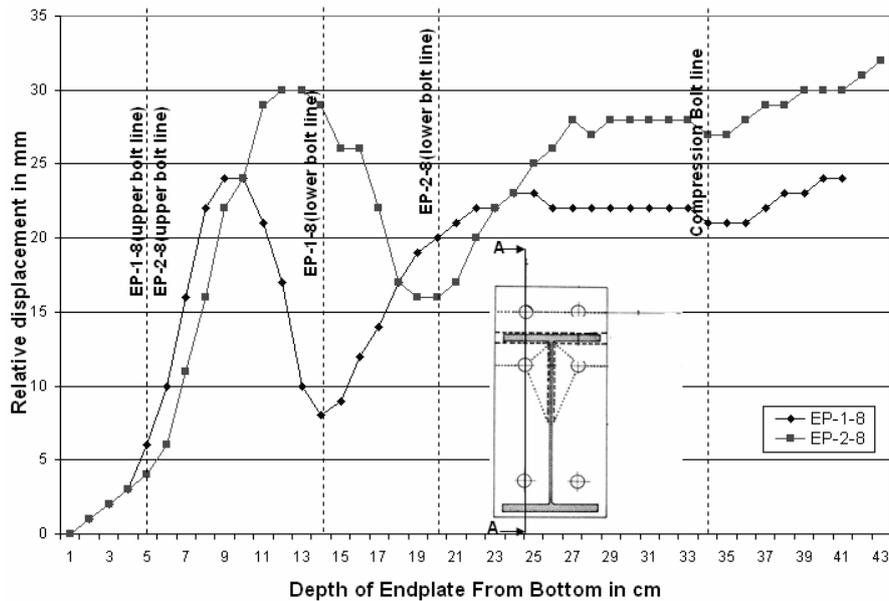


Figure 9. Endplate Relative Displacements of 8mm Thick Endplates

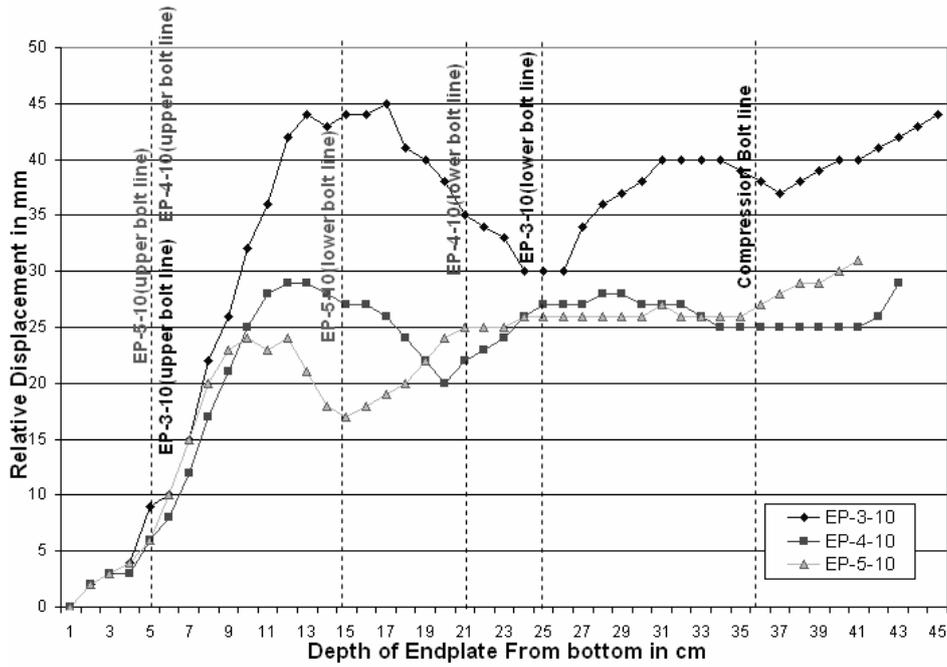


Figure 10. Relative Displacement of 10mm Thick Endplates

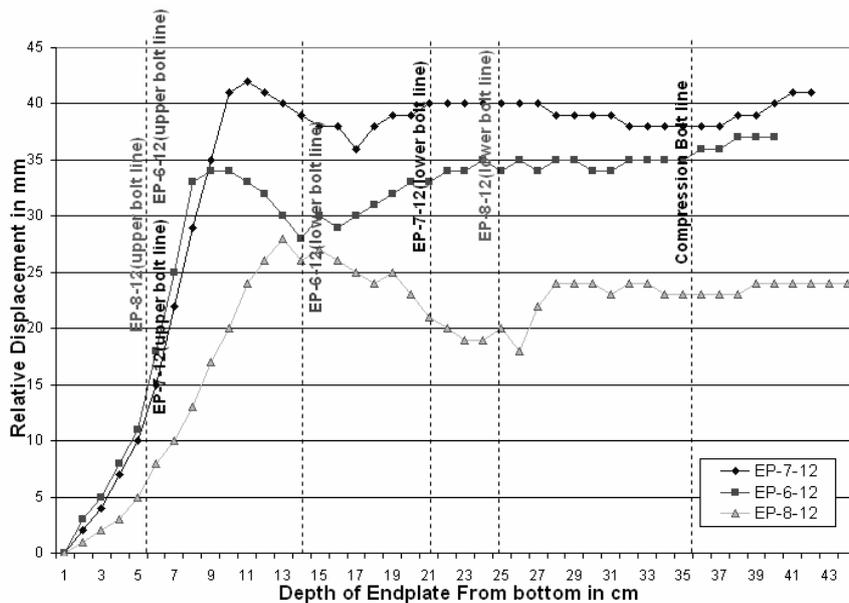


Figure 11. Relative Displacement of 12mm Thick Endplates

5. NUMERICAL INVESTIGATIONS ON END PLATE CONNECTIONS

5.1 FINITE ELEMENT MODELING

Finite element analysis of all the eight test specimens was carried out using ABAQUS software [Hibbit et al. [18]] to assess the design procedure. All the connecting members including the bolts were modeled using the element C3D8R which refers to continuum three dimensional 8-noded brick element with reduced order integration. This element has three degrees of freedom at each node, translations in the nodal x, y, & z direction. As the solid elements do not have rotational degree of freedom, the end-plate and the column flange are discretized across the thickness also to

consider the effect of bending. Small sliding surface-to-surface contact was considered for all the contacts in the connection which assumes relatively small sliding, but can undergo arbitrary rotation of the bodies. The connection between the beam and the end-plate was assumed to be rigid. Frictional (value $\mu = 0.3$) contact using penalty stiffness formulation was considered for the tangential contact between the end-plate and the column flange. The normal contact between the end-plate and the column flange was considered as hard using augmented Lagrange formulation. Hard constraint is used for the connection of bolt head/nut to the beam/column.

The yield and ultimate tensile stresses for the beams, column and end-plate were considered as 250 and 420 MPa respectively. The ultimate strain considered was 0.23. High strength friction grip bolts (8.8 grade) were used in this study. The yield and ultimate tensile stresses considered for the bolts were 640 and 800 MPa respectively. The ultimate strain considered was 0.12. A pretension force of about 0.7 times the bolt yield stress [Maggi et al. [19]] was applied at the center section of the bolt using the pretension option in ABAQUS. Hexagonal bolt heads and nuts are idealized as circular bolt heads and nuts to simplify the model. Washers are not modeled. Bolt holes are assumed to be 1.5mm larger than the bolt size ($D=17.5\text{mm}$). One end of the beam-column assembly is supported by a hinge and the other by a roller to simulate simply supported condition. The fillets in the angles are not modeled in order to reduce the complexity of the model. Final arrangement of the finite element mesh is decided based upon the computer time, convergence of solution and by comparison with the experiment results. Mesh refinement is carried out for corners, holes and the angles, where stress concentration is expected to occur. The sizes of meshes are controlled between the components of connection to enable surface to- surface contacts and easy convergence.

The load application has been effected in two load steps. In the first step, pre tensioning forces are applied to all the bolts. In the second step, uniform pressure load is applied over the stub column (Figure 12). Non linearity arises from large displacement effects, material non linearity and boundary nonlinearity due to contact and friction. In the present study, material and boundary nonlinearities are considered. The highly non linear nature of this problem, leads to non-convergence of the solution. It is overcome by using the nonlinear analysis option of ABAQUS. Figure 13 shows the finite element model and the deformed configuration of a typical end-plate connection. The deformations and rotations were observed to be higher than that observed from the test near to the failure.

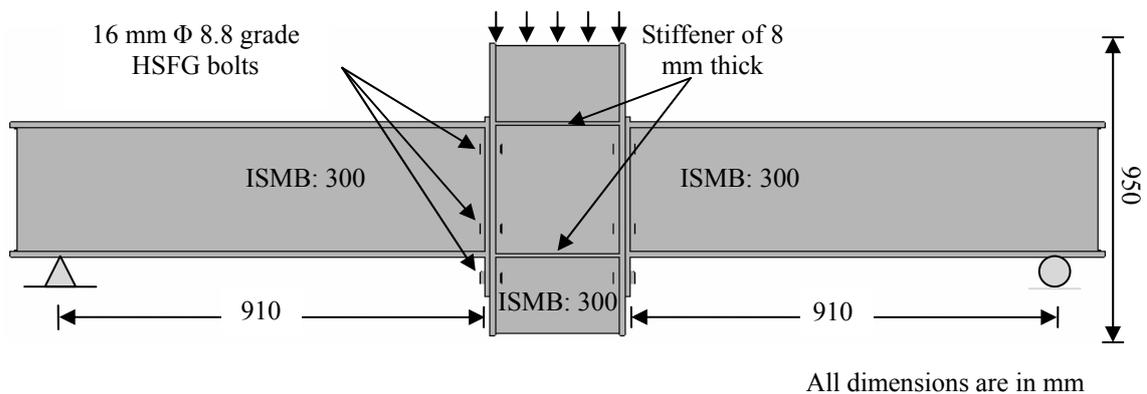


Figure 12. Connection Details

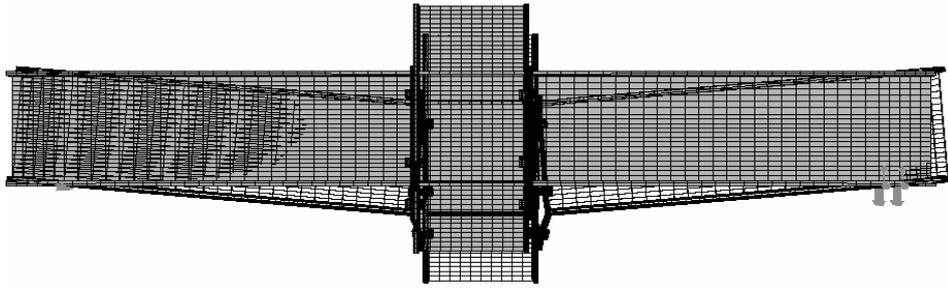


Figure 13. FE Model and the Deformed Configuration of a Typical End-Plate Connected Beam Assembly Model

5.2 COMPARISON OF ABAQUS RESULT WITH EXPERIMENTS

Comparison of Moment-Rotation curves

Figures 14-16. show the comparison of moment-rotation curves obtained from the finite element analysis with those obtained from the tests, and predictions by Frye-Morris [4] model. For EP-1-8 specimen, the experimental $M-\theta_r$ curve is linear and stiff upto 105 kN and beyond that the flexibility of connection starts. The Frye-Morris model traces more or less the experimental curve. The experimental $M-\theta_r$ curve show that the EP-2-8 specimen is linear and stiff upto 72 kN and beyond that the flexibility of connection starts. This connection behaviour is more flexible than the previous specimen. The Frye-Morris model is stiffer than the experimental curve. Similar to that of 8mm thick plate with 400mm plate length, the behaviour of EP-5-10 specimen matches with experimental value. The experimental $M-\theta_r$ curve is linear and stiff upto 144 kN and beyond that the ductility of connection starts. For EP-4-10 specimen, the experimental $M-\theta_r$ curve is linear and stiff upto 90 kN and beyond that the ductility of connection starts. This connection behavior is more flexible and similar to that of EP-2-8 specimen. The Frye-Morris model curve shows a stiffer behaviour than the experimental curve. EP-3-10 specimen behaves stiffer than the previous specimen 10 x 420 and the Frye-Morris model curve is conservative for this specimen. The curve is linear upto 78kN and the ductility of the connection starts beyond that load. The behaviour of experimental moment rotation curve of EP-6-12 specimen is similar to that of the 8mm & 10mm thick endplate. The Frye-Morris model curves are unconservative when compared to the experimental ones. The experimental $M-\theta_r$ curve is linear and stiff upto 174 kN and beyond that the ductility of connection starts. The experimental $M-\theta_r$ curve of EP-7-12 specimen is linear and stiff upto 132 kN and beyond that the ductility of connection starts. The connection behavior is more flexible and similar to that of 8mm x420mm. The Frye-Morris model curve is slightly stiffer than the experimental curve. The moment rotation is so stiff than the previous specimen 10 x 420 and the Frye-Morris model is much conservative for EP-8-12 specimen. The curve is linear upto 114kN and the ductility of the connection start beyond that load.

A good correlation of moment rotation curves obtained from the FEM with that obtained from the tests can be seen up to a specimen deflection of $L/100$ which is well beyond the allowable deflection for serviceability ($L/180$) as prescribed in the IS: 800 draft [17] for 10 mm and 12 mm thick plates. The moment rotation curve for 8 mm thick plate slightly deviates from the test curve after a deflection of $L/525$, but the initial and final stiffness correlate well with the test results. It is also noticed that the stiffness of the connection increases with the increase in the thickness of the end-plate and reduces as the lever arm increases both in tests as well as FEM.

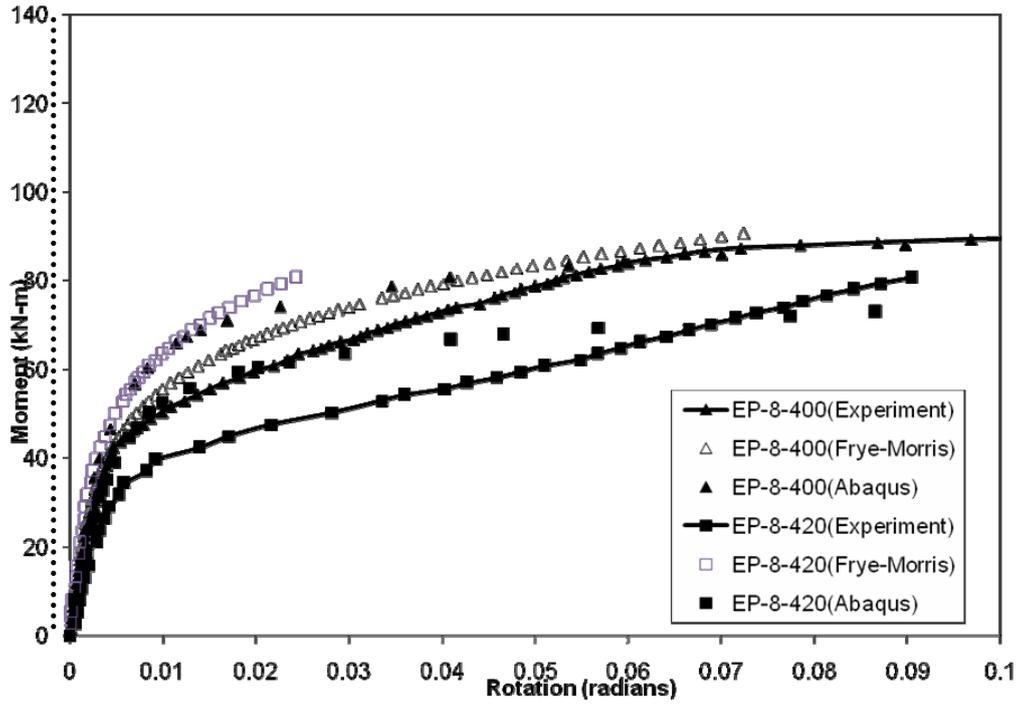


Figure 14. Comparison of Moment Rotation Curves for All 8mm Endplates

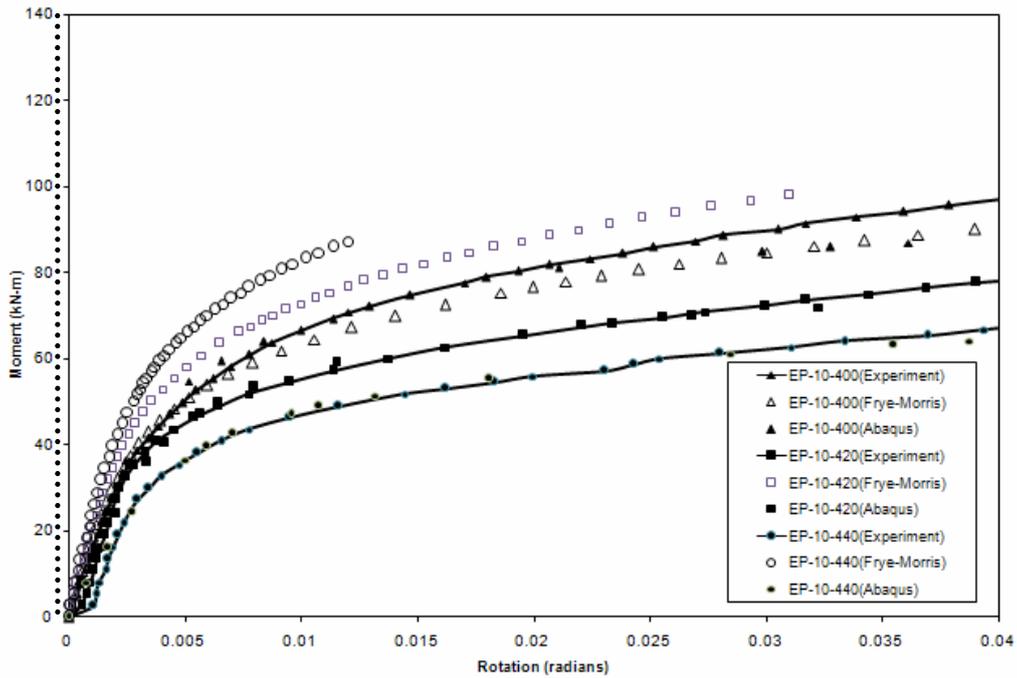


Figure 15. Comparison of Moment Rotation Curves for All 10mm Endplates

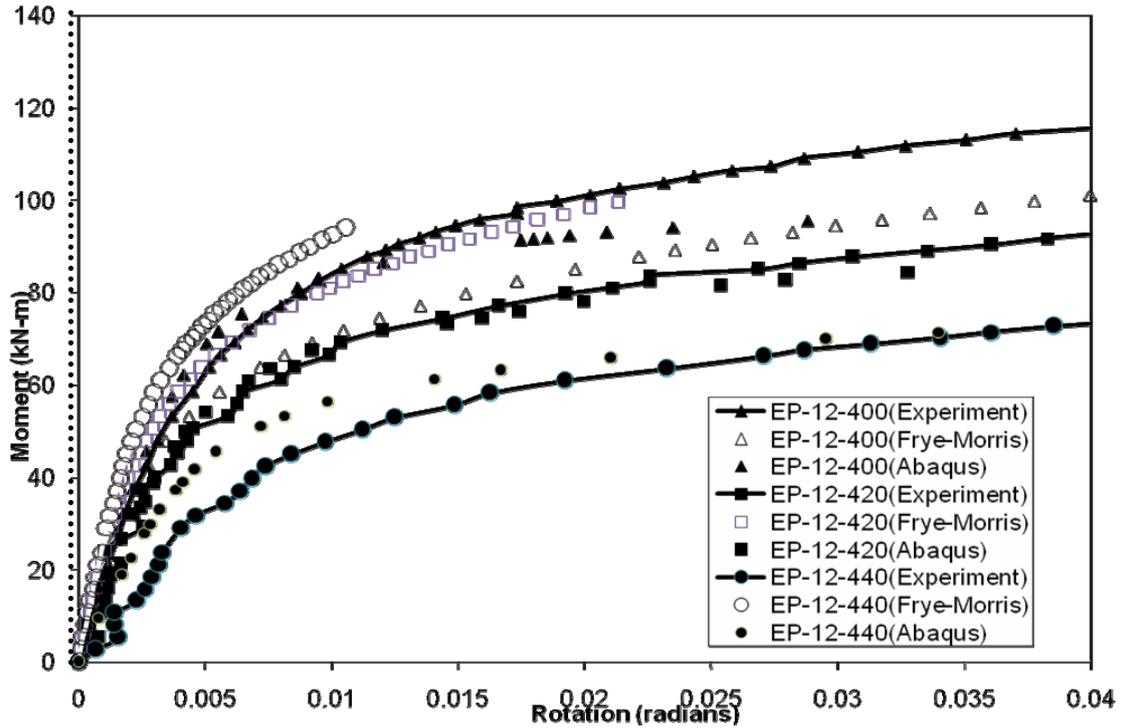


Figure 16. Comparison of Moment Rotation Curves for All 12mm Endplates

Comparison of Failure modes and Loads

Figure 17. shows the stresses in all the parts of the connection at failure. Two types of failures (i) end-plate rupture and (ii) bolt rupture were observed in the tests as well as in FEM analysis (Figure 18). Table 3 shows comparison of failure modes and loads obtained from FEM with that obtained from the tests. The specimen with 8 mm thick and 400 mm depth end plate failed due to tearing of end-plate. As the lever arm increases, the failure of the specimens is by fracture of the bottom row of tension bolts. For plate with 10 mm thick and 400 and 420 mm depth, the failure is by tearing of end-plate by finite element analysis against fracture of the middle row of bolts observed in tests. In both the cases EP-3-10 and EP-4-10, these two limit states such as tearing of endplate and fracture of bolts were close to each other. Since finite element analysis is based on ideal assumptions, in which the fracture of the bolt occurred before the fracture of endplate. However in the experiments the endplate fractured before the fracture of the bolt. For larger lever arm, failure is by fracture of bottom row of tension bolts in finite element analysis and tests. For plate with 12 mm thick and 400 and 420mm, the failure is by fracture of bottom row of bolts by finite element analysis against the fracture of the middle row of bolts observed in tests. For larger lever arm, failure is by fracture of bottom row of tension bolts by both finite element analysis and tests. The failure loads obtained from FEM analysis compare within 12% of the loads obtained from the tests.

Table 3. Failure Modes and Loads

Specimen Designation	Failure load (kN)		Mode of failure	
	Experiment	FE analysis	Experiment	FE analysis
EP-1-8	204	203.5	Tearing of end-plate	
EP-2-8	185	174	Fracture of bottom row tension bolt	
EP-3-10	252	221.3	Tearing of end-plate	Fracture of middle row tension bolt
EP-4-10	219	193.3		
EP-5-10	201	188.5	Fracture of bottom row tension bolt	
EP-6-12	270	246.2	Fracture of bottom row tension bolt	Fracture of middle row tension bolt
EP-7-12	225	214.9		Fracture of middle row tension bolt
EP-8-12	210	196.5		Fracture of bottom row tension bolt

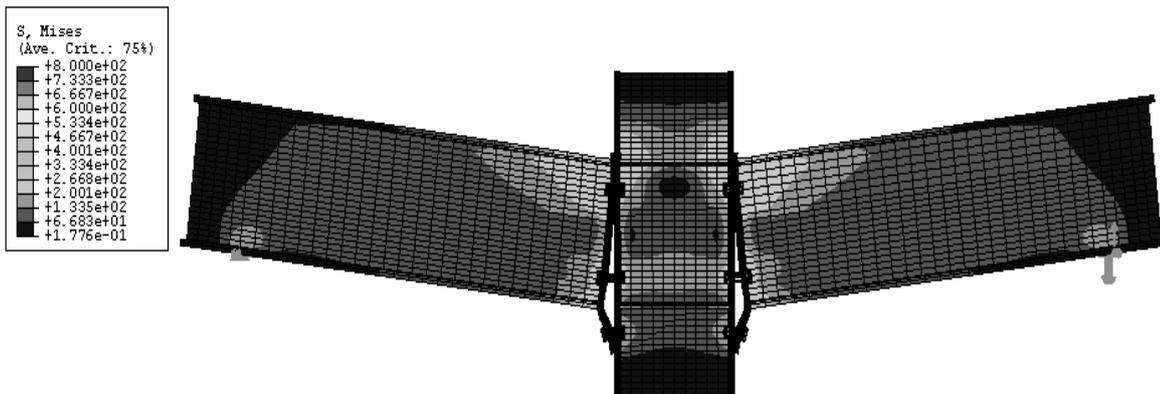


Figure 17. Stress contour at failure

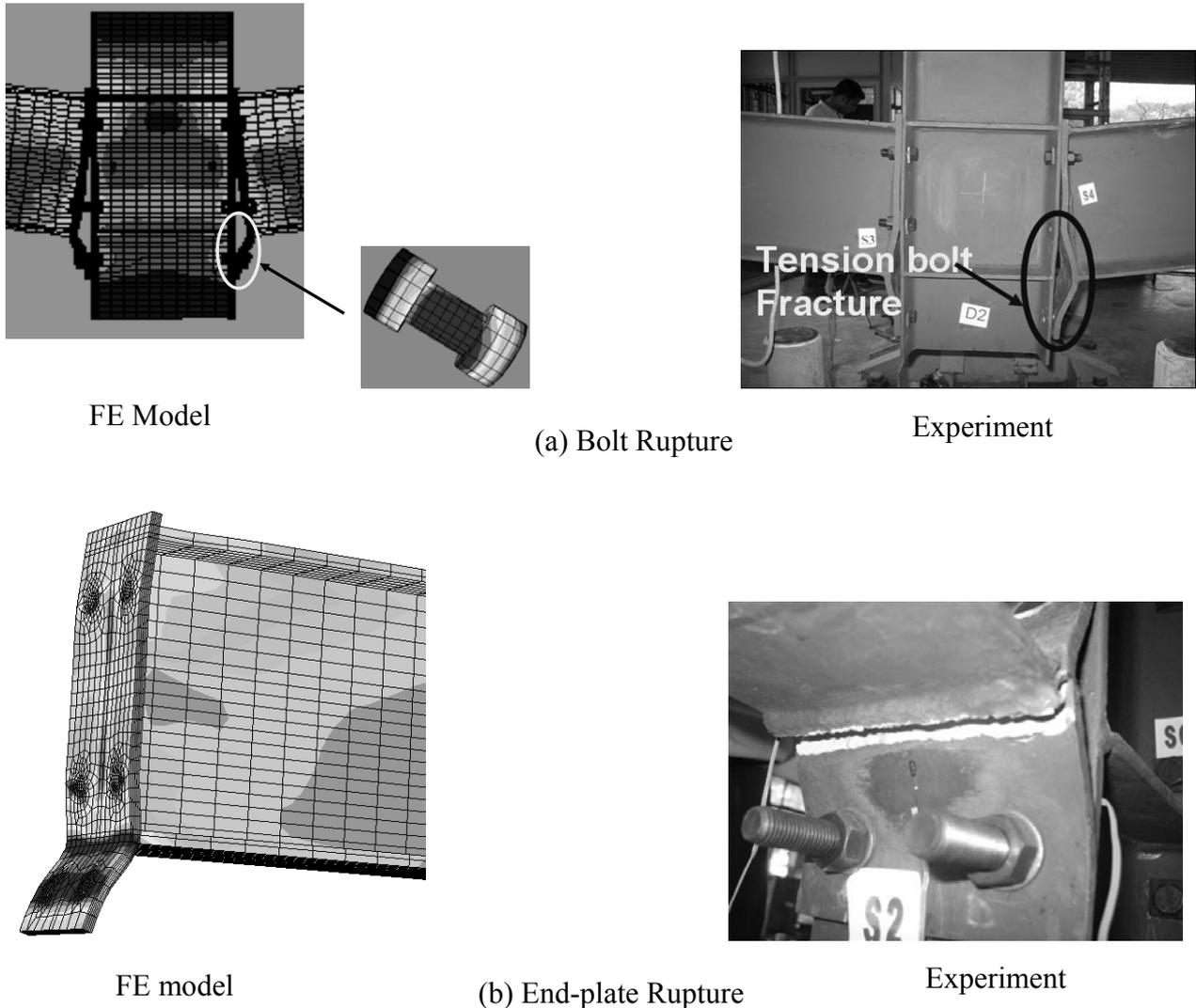


Figure 18. Failure Modes

6. VALIDATION OF FRYE-MORRIS MODEL

As discussed earlier Frye-Morris [4] model, a non-dimensional polynomial model for moment-rotation characterization of the connections, has been incorporated in IS:800 [17]. This model has not been evaluated for Indian construction practices. The polynomial model for the moment-rotation behaviour of the endplate connections is reassessed based on the present experimental and numerical results.

The original Frye-Morris [4] equation for endplate connection is given by

$$\theta_r = 2.06 \times 10^{-1} (KM) + 5.36 \times 10^{-2} (KM)^3 + 1.31 \times 10^{-7} (KM)^5 \quad (1)$$

(Moment kN-m, Parameters in cm) and the standardization factor for the endplate connection is given as

$$K = t_p^{-0.6} \cdot d_g^{-2.4} \quad (2)$$

The above equations are based on limited experimental results numbering 15, which are reported in the literature between 1948 - 1970. However the construction methods themselves have undergone a sea change world over, resulting in very flexible endplate connections. This flexibility is not reflected in the original equations which were based on tests with very stiff connection characteristics. The size parameters for the endplate connections with the column stiffeners are thickness of plate (t_p) and the lever arm distance (d_g). The moment-rotation curves are generated by systematically varying the size parameters. After finding exponents of all size parameters, a standardized moment – rotation curve is plotted for all specimens. Finally, the curve fitting constants C_1 , C_2 , C_3 of standardized moment – rotation relationship is derived by using the least squares curve fitting procedure.

The revised standardization factor K obtained from the present study based on Frye-Morris [4] procedure is

$$K = t_p^{-1.04} \cdot d_g^{4.20} \quad (3)$$

The curve fitting constants C_1 , C_2 , C_3 of standardized $M-\theta_r$ relationship are listed in Table 4.

Table 4. Curve Fitting Constants for All Endplate Connection Specimens

TEST - ID	C1	C2	C3
EP-1-8x400	-1.00E-11	2.11E-27	6.40E-45
EP-2-8x420	-3.24E-11	3.90E-27	-1.34E-44
EP-5-10x400	6.37E-12	9.90E-28	1.49E-44
EP-4-10x420	-3.48E-11	2.62E-27	2.85E-46
EP-3-10x440	-3.81E-11	2.58E-27	-3.88E-45
EP-6-12x400	4.02E-11	-1.16E-27	3.61E-44
EP-7-12x420	2.49E-11	-3.87E-28	1.20E-44
EP-8-12x440	2.06E-11	9.80E-28	1.03E-44

Therefore the average value of $C_1 = -2.91 \times 10^{-12}$, $C_2 = 1.45 \times 10^{-27}$, $C_3 = 7.83 \times 10^{-45}$ and the Eq. 1 becomes,

$$\theta_r = -2.91 \times 10^{-12} (KM) + 1.45 \times 10^{-27} (KM)^3 + 7.83 \times 10^{-45} (KM)^5 \quad (4)$$

This proposed model is checked with the experimental values and with the original Frye-Morris model for different thicknesses of the endplate varying with their bolt spacing. They are given in Figures 19-21. From the comparative study it is found that the original Frye-Morris model prediction overestimates the stiffness for all specimens except for EP-6-12 and shows closer results to specimens EP-1-8, Ep-5-10. The proposed Frye-Morris model equation presents a best fit to all ranges of endplate connections.

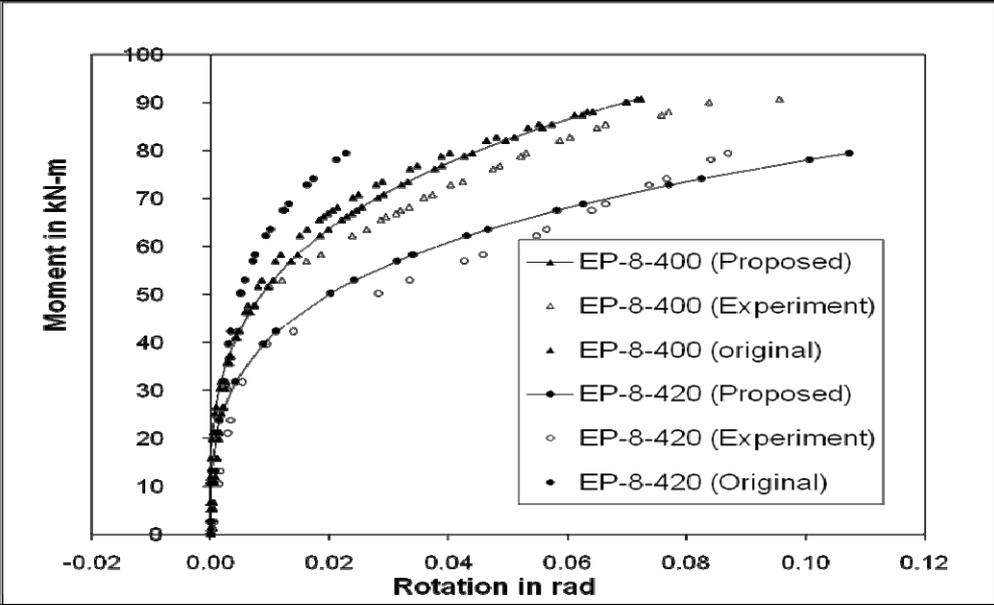


Figure 19. Comparison of $M-\theta_r$ Curves of Proposed Model with Experiment and Original Model for All 8mm Specimens

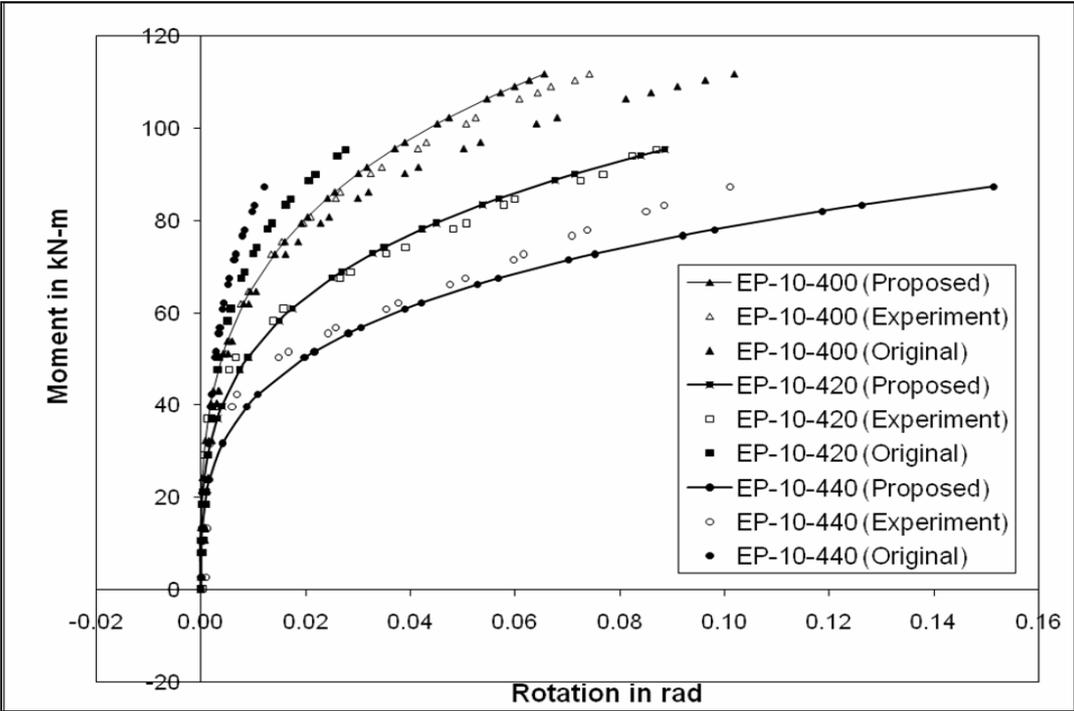


Figure 20. Comparison of $M-\theta_r$ Curves of Proposed Model with Experiment and Original Model for All 10 mm Specimens

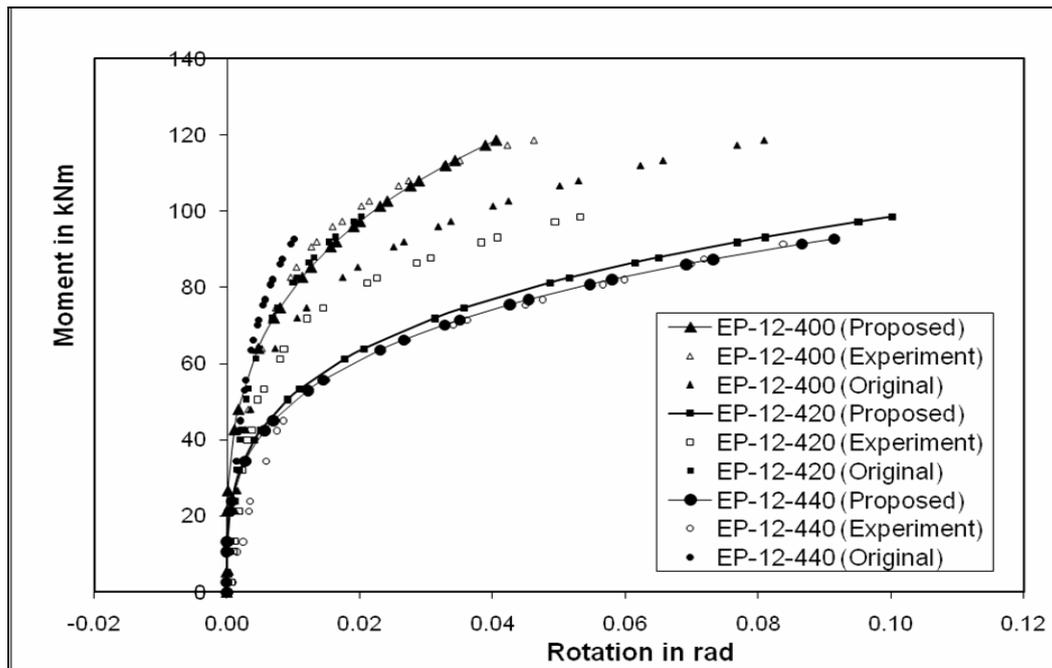


Figure 21. Comparison of $M-\theta_r$ Curves of Proposed Model with Experiment and Original Model for All 12 mm Specimens

7. CONCLUSION

From the experimental study the following observations are made.

1. In the case of endplate connections for a particular thickness, it is observed that the initial stiffness reduces with the increase in bolt spacing. It is also observed that the contact opening increases with the increase in bolt spacing.
2. For the same thickness, the load carrying capacity decreases with the increase in bolt spacing and this is due to the increased prying action on the bolts due to increased prying length.
3. For the same bolt spacing, the load carrying capacity increases with the increase in endplate thickness and this increase is provided by the flexural yielding strength of the endplate. As the thickness of the endplate increases, the endplate becomes stiffer and the failure is observed to be by fracture of tension bolt. This is due to the fact that as the thickness of the plate is increased, the tension is directly transferred to the bolts.

Finite element analysis of eight test specimens are carried out using the FE software, ABAQUS and the results are compared with the tests results. The following conclusions are drawn from the numerical study

1. Moment rotation curves obtained from ABAQUS compare reasonably well with those obtained from the tests.
2. Two modes of failure (i) end-plate rupture and (ii) bolt rupture of the connections are observed in the tests as well as in finite element analysis
3. The failure loads obtained from FEM analysis compare within 12% of the loads obtained from the tests

It is observed that the results obtained from the tests compare well with those obtained using the original Frye-Morris model for EP-1-8 and EP-5-10; whereas for all the other specimens, the Frye-Morris model predicts higher stiffness compared with that obtained from the tests. Hence it is concluded that the Frye-Morris model does not represent the different stiffness ranges in the endplate connection. A modified Frye and Morris model has been developed for the moment-rotation behaviour of endplate connection based on the test results. The revised Frye-Morris model is proposed for incorporation in the next revision of IS:800 [17].

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