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Modeling and FE analysis of column to beam end-plate bolted connection

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ABSTRACT

This piece of work aims at the modeling and using the finite element method approach (FEM) to analyze the fatigue behavior of bolted beam to column end-plate connection in the structural steel framework subjected to static loading. A detailed three dimensional (3D) simulation model of the bolted beam to column end-plate connection is constructed in PRO-E wildfire and it is analyzed in the ANSYS workbench to obtain its behavior. The bolted end-plate connection is chosen as an important type of beam to column joint. The end-plate connection is chosen for its complexity in the analysis and behavior due to the number of connection components and their inheritable behavior. The solid elements, bonded contact and the bolt pretension are included to obtain the behavior of the structure. The FEA results of the structure with or without bolt pretension are compared with the available literature. At last the fatigue behavior of connections under over tensioning is presented in this work.

1. Introduction

Connections form an important part of any structure and are designed more conservatively than members. This is because, connections are more complex than members to analyze, and the discrepancy between analysis and actual behavior is large. Further, in case of overloading, we prefer the failure confined to an individual member rather than in connections, which could affect many members. Connections account for more than half the cost of structural steelwork and so their design and detailing are of primary importance for the economy of the structure. Combinations of simple fabrication techniques and speedy site erection have made bolted endplate connection is one of the most popular methods of connecting members in structural steelwork frames. Although it is simple in their use, bolted endplates are extremely complex in their analysis and behavior. Bolted joints are

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often the most critical parts with respect to fatigue life of structures. Therefore, it is important to analyze these components and the forces they are subjected to. Bolted connections, especially end-plate types, are being widely used in steel structures. They are often used as moment-resistant connections. They have the advantages of easy quality control and less assembly time than welded connections. These connections include two types: flush end-plate connections and extended end-plate connections with or without stiffeners. The behavior of beam-to-column connection in structural steel frames can be conveniently represented by its flexural behavior which is primarily shown by the moment–rotation (M–ϕ) relationship. This behavior is non-linear even at low load levels. In fact, moment–rotation curves represent the result of a very complex interaction among the elementary parts constituting the connection.

The potential economic implication of connections on frame design is realized by code provisions given in (Faculty of Mechanical Engineering, 2002a, Euro Code 3, 1992). As a result, special design guides for moment resisting connections have been developed as mentioned in (Faculty of Mechanical Engineering, 2002b, Gupta Mohan, 2006, Merton-Williams Metal Company, Product Guide). Since the connection types are highly indeterminate, current design approaches cannot model three-dimensional (3D) systems which are governed by complex combined material and geometrical non-linearity, friction, slippage, contact, bolt–end plate interactions and, eventually, fractures. Hence, the finite element technique has been adopted as a rational supplement to the calibration of design models. The first study into joint behavior using FEM, Bose et al. (1972), related to welded beam-to-column joints, which included: plasticity, strain hardening and buckling. The majority of Finite Element (FE) models found in the literature, however, only analyzes bolted joints, the most relevant of which are discussed next. Krishnamurthy and Graddy (1976) made the first three-dimensional (3D) joint model. Sherbourne and Bahaari (1996) developed a model to investigate the behavior of steel bolted end-plate connections. Bursi and Jaspart (1997a) modeled T-stub connections and isolated extended end-plate connections have been developed by (Bursi and Jaspart, 1997b, 1998). Choi and Chung (1996) presented a 3D model of a double extended endplate, which included the column flange; Bahaari and Sherbourne (2000) developed a detailed 3D model to study 8-bolt unstiffened extended end-plate connections; Sumner et al. (2000) also used 3D models to develop 4 and 8-bolt extended unstiffened end-plate connections; Swanson et al. (2002) used 3D and several two-dimensional (2D) models to study the behavior of T-stub flanges; Citipitioglu et al. (2002) presented different 3D models of bolted connections with angles, following the recommendations of Bursi and Jaspart, (1998) on the FE selection. Gantes and Lemonis (2003) developed a model for bolted T-stub steel connections. Ju et al. (2004) developed a 3D model to study the structural behavior of butt-type steel bolted joints. Tagawa and Gurel (2005) used FE simulations to examine the strength of steel beam-to-column joints stiffened with bolted channels. Abolmaali et al. (2005) developed a 3D model for flush end-plate connections. Maggi et al. (2005) carried out parametric analyses on the behavior of bolted extended end-plate connections using 3D models. Kukreti and Zhou (2006) quantified the impact of semi-rigid connection properties on steel frame behavior, and developed a 3D FE model and computer program to investigate the moment–rotation behavior of eight-bolt stiffened end-plate connections. Jeong Kim et al. (2007) introduced the four kinds of FE model in order to investigate the best modeling technique for the structures with bolted joints. Butterworth (1999) used a combination of full scale testing and materially non-linear three dimensional finite element analysis (FEA) in order to investigate extended end plate beam-to-column connections. Pirmoz et al. (2008) studied the behavior of bolted top-seat angle connections with web angles using several 3D parametric models. Dai et al. (2010) made a simulation study of 10 fire tests on restrained steel beam-column assemblies using five different types of joints: fin plate, flexible endplate, flush end-plate, web cleat and extended end-plate. Diaz et al. (2011) and Shi et al. (2013) presented a full 3D FE model of steel beam to column extended end plate joint to obtain its behavior. A comprehensive review of the literature about the characteristics of the numerical models of bolted joints provides the following three observations:

1. End-plate joints are the most studied;

2. The majority of the models include: bolt contact, pre-tensioning, geometric and material non-linearity.
3. Although computational resources allow for joints to be fully modeled using 3D types, other FE elements have been used:
a. 2D plane stress and shell elements were used for parametric studies and calibration of 3D models;
b. Truss and beam elements were used to model the bolts, columns and beams.

This piece of work aims to present a full 3D simulation model in Pro-E V5.0 (PTC Software System, 2010) and the same FE model is imported in the ANSYS WORKBENCH V14.0 (Swanson Analysis System, 2012) to obtain the behavior of structural steel beam-to-column bolted endplate joints. This model includes: contact and sliding between different elements; bolt pre-tension; geometric and material nonlinearity. The results from the FE Analysis (FEA) are verified by comparing the obtained load-deformation and load-stress (Von mises) curve with those from experimental results found in the literature (Butterworth 1999) and with the results obtained using the model proposed with the help of dimension and properties of rolled steel I-section (Faculty of Mechanical Engineering, 2002b, Gupta Mohan, 2006) with pretension and without pretension are compared. At last the behavior of connections under over tensioning, results in fatigue failure which results in strain hardening and the failure of the bolt takes place is shown in this work.

2. Modeling of beam to column bolted structure

In order to analyze the beam to column end-plate connection under static load, a finite element model is presented using the software package PRO-E version 5.0. Then the model is analyzed in the software package ANSYS WORKBENCH version 14.0 to investigate the results of the referred model by (Butterworth 1999), for structural steelwork beam-column endplate connection.

2.1. Joint Configuration

Fig.1 shows the typical 2D joint configuration which consisted of a rectangular end-plate welded to the beam cross-section which is shown below in Fig.2 and fixed to the column Flange by the two rows of bolts of diameter 16mm, M16, Hex Bolt & Nut, IS: 2389, IS: 1364 given in Faculty of Mechanical Engineering, 2002a (one row is above and the other is below the tension beam flange). The dimensions of the bolt used in the structural joint are shown in Fig.3 below:

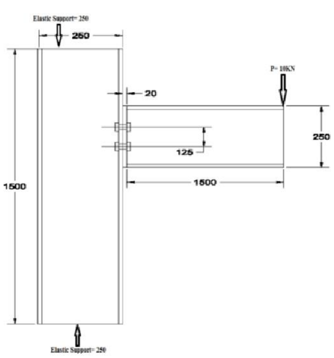


Fig. 1. Beam-Column bolted Structural steel connection. (Dimensions are in mm)

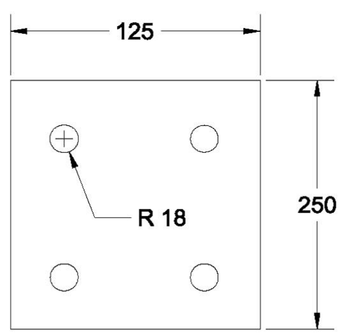


Fig. 2. End-plate used in the Connection

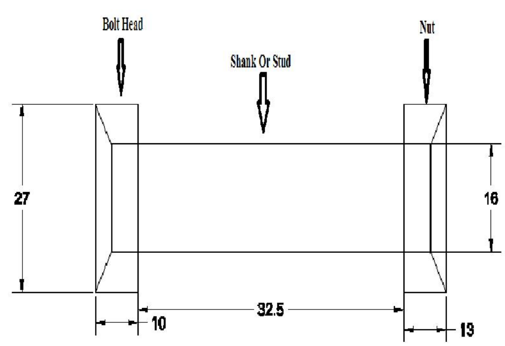


Fig. 3. Solid Bolt used in the connection

2.2 Dimension of Column and Beam

All the dimensions shown below in the Table 1 are referred from the PSG design data Book (Faculty of Mechanical Engineering, 2002b) and the other additional sectional dimensions are referred from the paper of Gupta Mohan et al. (2006) and also the value of R (root radius) is taken from the same as per the consideration of dimensions for the column and the beam. The dimensions of the end-plate are assumed accordingly as per the dimensions of the two joining members. The dimensions of the validation assembly is referred from the Product Guide of the Merton-Williams Metal Company, UK which are given below in Table 2 and Fig.4 showing the notations for the beam and column section is given below:

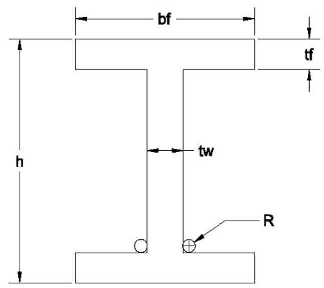


Fig. 4. Notations for the dimensions of column and beam

Table 1
Dimensions of the proposed model in mm

Column (ISMB 250)	Beam (ISMB 250)	End Plate (W*Th*L)
250*250*1500	250*125*1500	160*20*320

Table 2
Dimensions of the validation model in mm

Column (Grade 355)	Beam (Grade 355)	End Plate (W*Th*L)
354*127*33UB	254UC73	200*20*460

2.3 Material properties

The stress–strain curves are taken as elastic-strain hardening. This is acceptable since strain hardening is paired with excessive yielding in large areas and a large deflection criterion governs the ultimate strength design. However, in end-plate connections excessive strain is mostly local and besides considerable shear stresses occur in the region between the top bolts and the beam tension flange which necessitates considering strain hardening. In this finite element model, the adopted stress-strain relationships of structural steel are isotropic elastic-perfectly plastic. All the data of material properties are referred from the ANSYS V14.0 User's manual, Material Library (Swanson Analysis Systems, 2012) and are given below in Table 3 as follows,

Table 3
Material properties of Structural Steel

Density (kg mm^{-3})	Coefficient of Thermal expansion (C^{-1})	Specific heat (mJ kg^{-1} C^{-1})	Thermal conductivity (W mm^{-1} C^{-1})	Ultimate strength (MPa)	
				Compressive	Tensile
7.85E-06	1.20E-05	4.34E+05	6.05E-02	0	460

Yield strength (MPa)		young's modulus (MPa)	Bulk modulus (MPa)	Shear modulus (MPa)	Poisson's ratio
Compressive	Tensile				
250	250	2.00E+05	1.67E+05	76923	0.3

2.4 Loading System

For the studied connection, a point load and the spring stiffness force is used. The elastic support with the stiffness of 250N/mm^3 is applied axially on both bottom and upper faces of the column which helps the column to resist the bending force caused due to the application of the load on the beam member. The increasing point load P acts on the edge of the beam to generate an increasing bending moment during the loading as shown in the Fig.1 above.

2.5 Generation of the Finite Element Model

In order to define the joint with the finite element, following simplifications were assumed:

1. Both the bolt flange and nut is modeled in a single part with a diameter of 16mm and calculated from the mass property.
2. There is a clearance gap of 2mm provided between the bolt and the hole.
3. Both side flanges of the bolt are butted with the surface of the end plate and the column flange, respectively.
4. Washers were not included in order to reduce the number of contact regions.
5. Instead of weld there are bonded contact were provided between the surface of the end-plate and the contacting face of the beam with it.
6. The structure shown below in Fig.5 is modeled with 3142 elements and 14551 nodes.

The simplifications which are discussed above are shown below in the Fig.6:

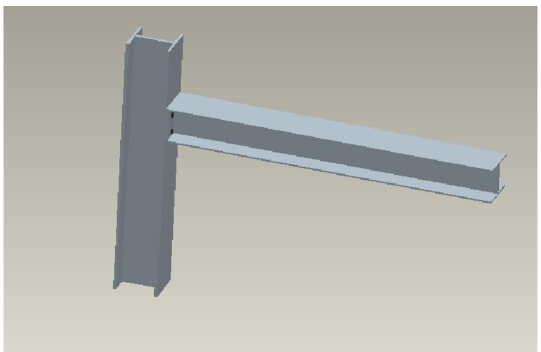


Fig. 5. Numerical model of beam to column end plate connection

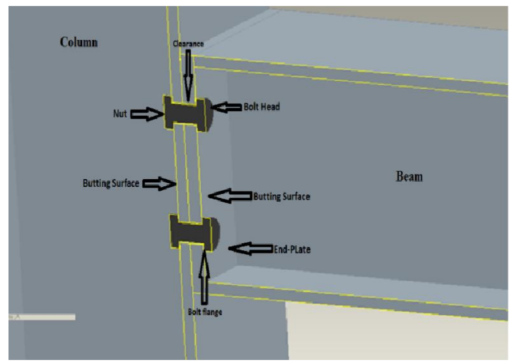


Fig. 6. Cut Section of the Beam-Column connection model

2.5.1 Selection of Elements

Different finite elements types in the ANSYS workbench 14.0 [28] are used to define the column, beam, endplate and bolts. These elements are SOLID186, which is used to define the column, beam and end-plate. SOLID187 is used to define the solid bolt. Since bolt head and nut will remain in contact with the connecting plate and the column flange through all the load steps, they are defined continuous with both bolt column flange and the end plate nodes, respectively. The interface between the column flange and the end-plate is simulated by creating contact pairs with the 3D target surface elements TARGE 170 and 3D surface to surface (SURF 154) Bonded contact elements CONTA 174. The PSMESH command is used to define the pretension section in the middle of the bolt shank or stud and generate the pretension elements PRETS 179 through which the pretension force of the bolts are applied by using the command CLOAD201 and COMBIN 14. Meshed finite element model of the connection is shown below in Fig.7, 15030 nodes and 3315 elements have been created.

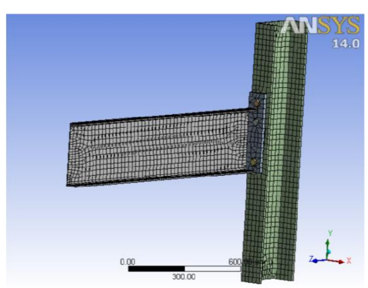


Fig.7. Proposed Meshed Model

2.5.2 Contact Regions

The bonded contacts are considered between the following three regions, which are given below:

1. Surface of the end-plate and the contacting face of the beam with the plate.
2. Bolt flange and the surface of the end-plate.
3. Inner surface of the nut and the column flange.

2.5.3 Boundary Conditions

The applied boundary conditions are:

1. The web of both column and beam and also the bottom and the upper face of the column are prevented with any movement along the X-axis that is it has the value of degree of freedom along the X-axis or the horizontal plane is zero.
2. The elastic support with the stiffness force of 250N/mm^3 is also provided on the upper and the bottom face of the column which restricts the movement of the column along the Y-axis as well as in the Z-axis.
3. The bottom face of the end-plate is provided with the zero degree of freedom along the Y-axis.

All the boundary conditions mentioned above are shown below in the Fig.8:

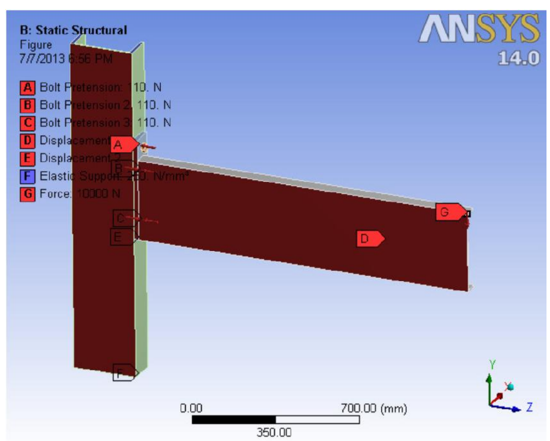


Fig. 8. Boundary Conditions

2.5.4 Applied loads

The finite element model is used to calculate the behavior of the connections under without pretension, pre-tension and the stage of over tensioning. This was attained by the following loading stages:

Without Pretension:

- Apply the point load P incrementally on the edge of the beam as shown in the Fig.1 until the bending moment occurs, which doesnot affect our case to the great extent.

Pretension:

- Apply the bolt pre-load given by the Eq.1, given below:

$$\mathbf{f_p = 0.8\, f_{y \cdot b}\, x\, A_s} \quad (1)$$

Where, $f_{y.b}$ = Yield strength of the bolt and A_s = Tensile stress area.

- Also apply the point load P incrementally on the edge of the beam as shown in the Fig.1 until the bending occurs in any member of the structure.

Over tensioning:

- Apply the bolt pre-load incrementally until the fatigue failure in the bolt is seen.
- Also apply the point load P incrementally on the edge of the beam as shown in the Fig.1 until the strain hardening and bolt failure occurs.

2.5.5 Analysis Program

During the analysis and solution of the finite element model, the half section of the Structure is considered due to the symmetry of the structure along the vertical axis (y-axis) and also because of less time consumption by the software in analyzing the proposed structure, on which the displacement loads were applied on the loading point as shown in Fig.1. Also the value of the displacement load of each test reference is referred to the test data (Butterworth 1999). The analysis type is "Static Displacement" which is elastic-plastic analysis. The material yield criterion is Von-Mises yield criterion which is used to predict the onset of the yielding. The behavior upon further yielding is predicted by the "flow rule" and the "hardening law".

3. Results and Discussion

The graph showing the developed maximum equivalent (Von-mises) stress in the compression flange of the reference, validation and proposed FEA model with respect to the incresing test load with and without pretension is given below in the Fig.9.(a) and 9.(b) and also the percentage difference between the Von mises stress developed in the reference, validation and proposed FEA model with and without pretension is shown below in the Tables 4 to 7, respectively.

Firstly, the Von mises stress developed in the compression flange of the beam with and without pretension is validated with the reference model available in the literature. The further Von mises stress analysis is carried out with the proposed FEA model in this work in the same manner which shows quite difference between the stress pattern in the validation model and the proposed model, shown in the Fig.9.(a) and 9.(b).The linear equations and regression (R^2) values for the models considering pretension and without pretenesion, are also also given below:

The flange stresses developed in the validation model is found to be quite low as compared to the reference model in both the cases. At the load of 10kN there is a stress development of 321.72MPa in the compression flange of the validation model (considering bolt pretension), in comparison to the 370MPa of the flange stress seen in the reference model. While increasing the load to 29kN, the stress developed is 777.5MPa in comparing it with the flange stress of 815MPa in reference model. Similarly at 10kN the flange stress developed in the validation model without the consideration of bolt pretension is 312.68MPa in comparison to the stress of 350MPa in the reference model. On increasing the load to 29kN, there is the flange stress seen is 759.45MPa in the validation model, in comparison of the flange stress of 799.98MPa seen in the reference model. Whereas the FEA results of the proposed model shown the Von mises stress at the compression flange is of the order of 398.06MPa at the load of 10kN on considering bolt pretension. On increasing the load to 29kN the stresses in the compression flange becomes 925MPa. The Von mises stress is developed in the beam flange wihtout considering bolt pretension is 372.19MPa at the load of 10kN and on increasing the load to 29kN it becomes 887.66MPa which is seen in the web and plate connection of the beam.

Table 4

Von-mises stress pecentage difference between Ref. and validation model with pretension

Test Load (kN)	Stress in the Reference model with Pretension(MPa)	Stress in the Validation model with Pretension(MPa)	% of difference between Reference and Validation model
10	370	321.72	-13.04
21.5	607	576.68	-4.99
22	617	589.83	-4.4
29	815	777.5	-4.6

Table 5

Von-mises stress perecntage difference between validation and FEA model with pretension

Test Load (kN)	Stress in the Validation model with Pretension(MPa)	Stress in the FEA(Proposed model) with Pretension(MPa)	% of difference between FE and Validation model
10	321.72	398.06	19.17
21.5	576.68	673.79	14.41
22	589.83	684.52	13.83
29	777.5	925.63	16

Table 6

Von-mises stress perecentage difference between Ref. and validation model without pretension

Test Load (kN)	Stress in the Reference model without Pretension	Stress in the Validation model without Pretension	% of difference between Reference and Validation model
10	350	312.68	-8.09
21.5	598	552.56	-7.59
22	605	572.38	-5.39
29	799.98	759.45	-5.06

Table 7

Von-mises stress percentage difference between validation and FEA model without pretension

Test Load (kN)	Stress in the Validation model without Pretension	Stress in the FEA (Proposed model) without Pretension	% of difference between FEA and Validation model
10	312.68	372.19	15.98
21.5	552.56	626.01	11.73
22	572.38	651.21	12.1
29	759.45	887.66	14.44

Linear equations for the models considering pretension:

$$\begin{aligned} y &= 134.5x + 131.5 && \text{(Reference model)} \\ y &= 138.05x + 83.261 && \text{(Validation model)} \\ y &= 160.97x + 106.02 && \text{(FEA or proposed model)} \end{aligned} \quad (2)$$

Regression (R^2) values for the models considering pretension:

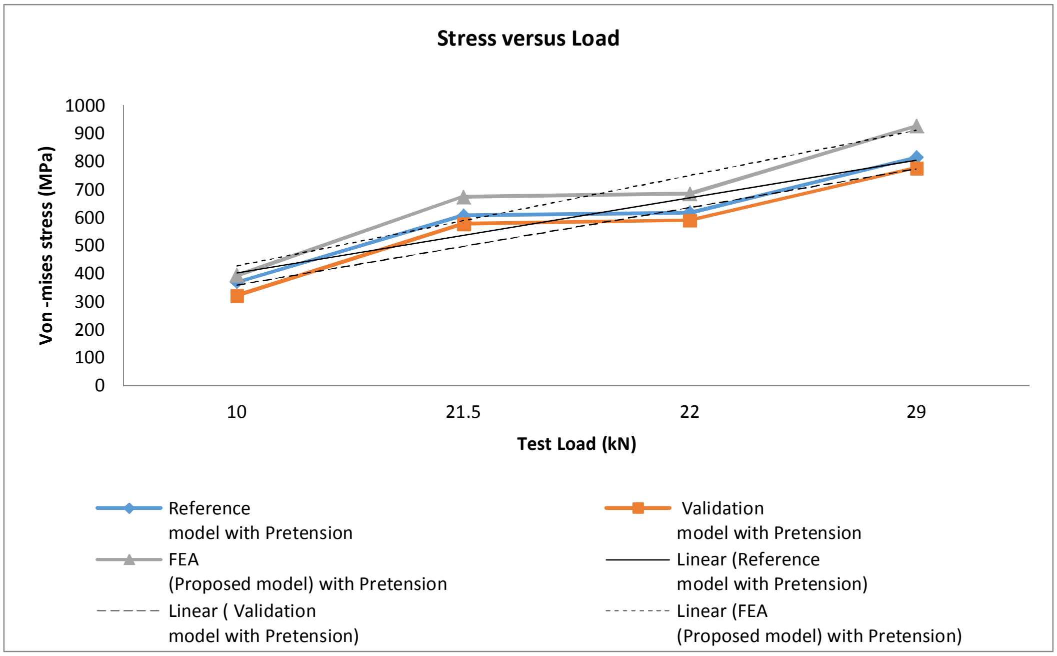
$$\begin{aligned} R^2 &= 0.9096 && \text{(Reference model)} \\ R^2 &= 0.9068 && \text{(Validation model)} \\ R^2 &= 0.9093 && \text{(FEA or proposed model)} \end{aligned} \quad (3)$$

Linear equations for the models without considering pretension:

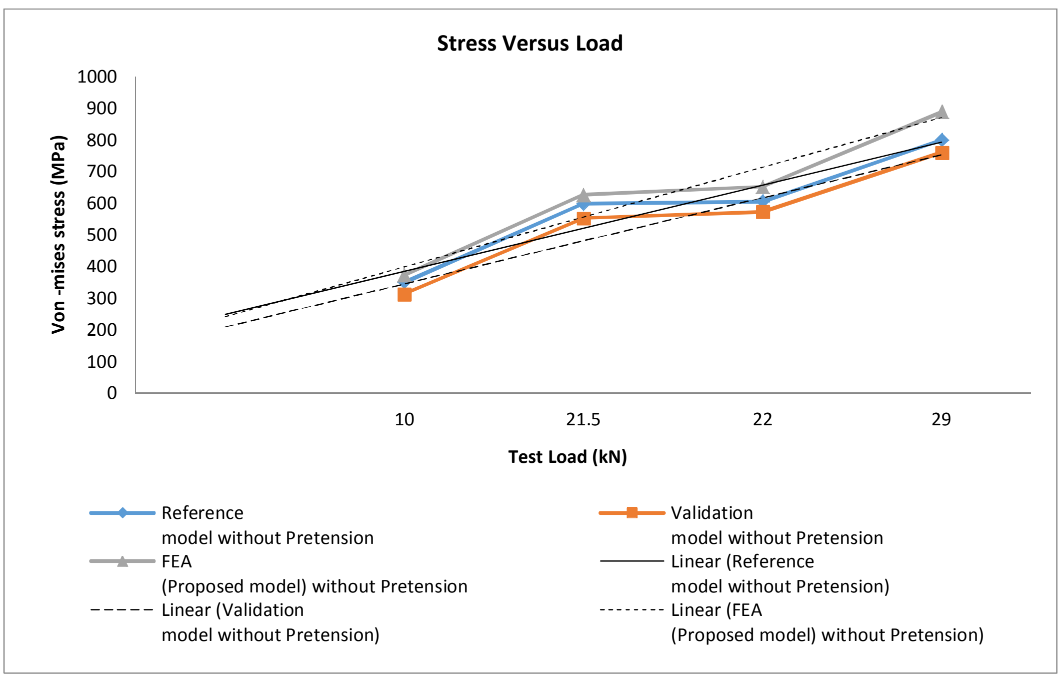
$$y = 135.69x + 113.32 \quad \text{(Reference model)}$$
$$y = 136.01x + 73.222 \quad \text{(Validation model)} \qquad (4)$$
$$y = 157.16x + 84.204 \quad \text{(FEA or proposed model)}$$

Regression (R^2) values for the models without considering pretension:

$$R^2 = 0.9029 \quad \text{(Reference model)}$$
$$R^2 = 0.9186 \quad \text{(Validation model)} \qquad (5)$$
$$R^2 = 0.9268 \quad \text{(FEA or proposed model)}$$



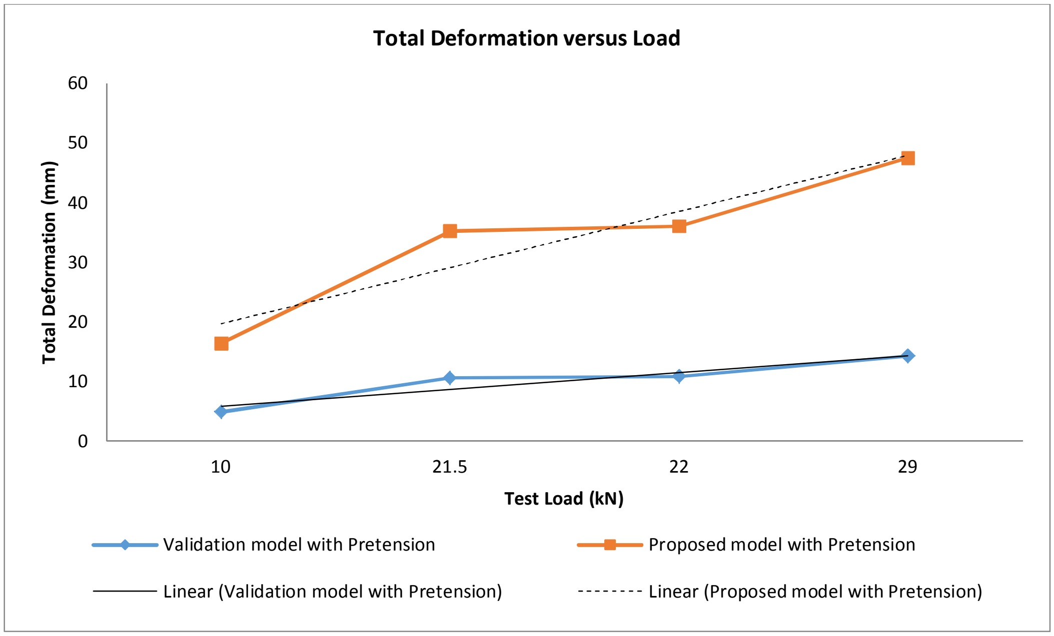
(a)



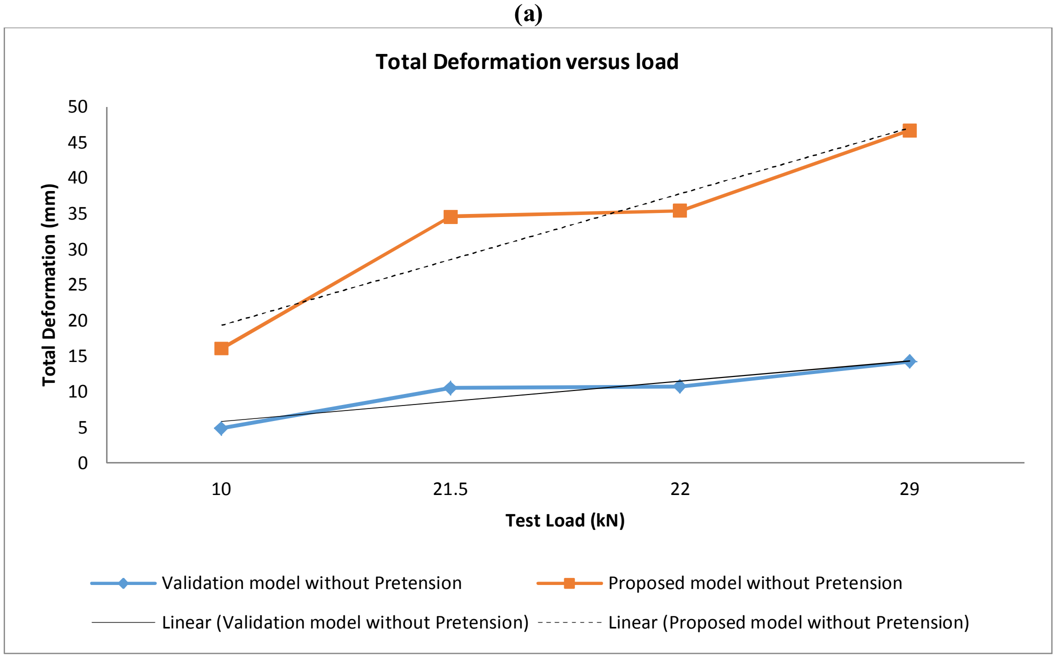
(b)

Fig. 9. (a) Graph showing the developed Von mises stress in the compression flange of Ref., validation and proposed model with pretension. (b) Graph showing the developed Von mises stress in the compression flange of Ref., validation and proposed model without pretension

The total deformation- test load graph between the validation and proposed model is shown below in Fig.10. The linear equations and the regression values (R^2) for the load- deformation values of both models analyzed in this work are also given below:



(a)



(b)

Fig. 10. (a) Graph showing the total deformation in the compression flange of validation and proposed model with pretension. (b) Graph showing the developed total deformation in the compression flange of validation and proposed model without pretension

Linear equations for the models considering pretension:

$$y = 2.8284x + 3.0743 \quad \text{(Validation model)}$$
$$y = 9.4209x + 10.24 \quad \text{(FEA or proposed model)} \qquad (6)$$

Regression (R^2) values for the models considering pretension:

$$R^2 = 0.8902 \quad \text{(Validation model)}$$
$$R^2 = 0.8903 \quad \text{(FEA or proposed model)} \tag{7}$$

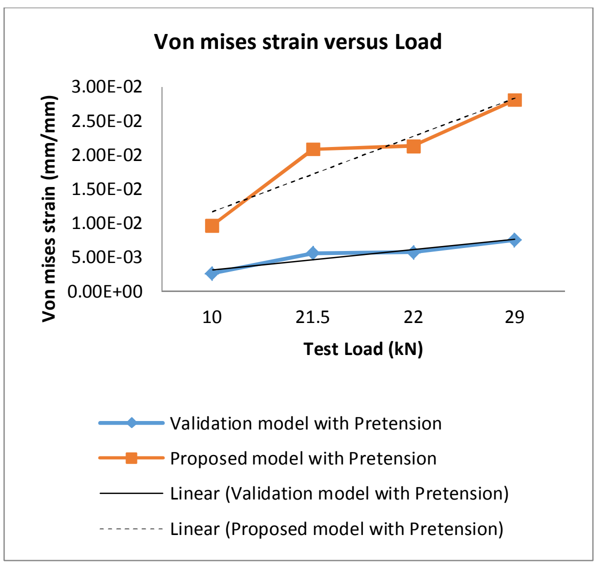
Linear equations for the models without considering pretension:

$$y = 2.8183x + 3.0633 \quad \text{(Validation model)}$$
$$y = 9.2569x + 10.062 \quad \text{(FEA or proposed model)} \tag{8}$$

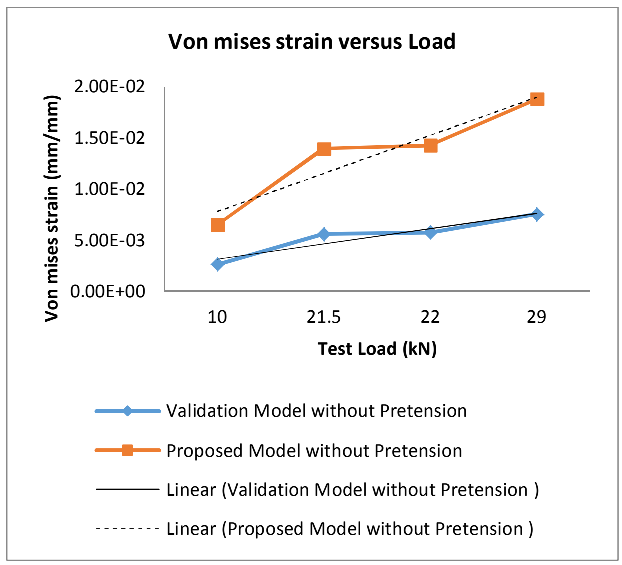
Regression (R^2) values for the models without considering pretension:

$$R^2 = 0.8903 \quad \text{(Validation model)}$$
$$R^2 = 0.8903 \quad \text{(FEA or proposed model)} \tag{9}$$

The maximum Von Mises strain- load comparison graph are shown below in Fig.11 (a) and 11 (b). As we can see from the graphs that there is not much difference in the Von-Mises strain values with and without pretension in the validation model, whereas there is a big difference in the values of maximum strain in the proposed model which implies that the maximum strain occurs in the proposed model in comparison to that of the validation model. Due to which the strain hardening of the bolts on increasing the pretension and load on the beam can be nicely seen in the proposed model than that of the validation model. The linear equations and the regression values (R^2) for the Von-Mises strain- test load values of both models analyzed in this work are also given below:



(a)



(b)

Fig. 11. (a) Graph showing the developed Von mises strain in the structure of the validation and the proposed model with pretension. (b) Graph showing the developed Von mises strain in the structure of the validation and the proposed model without pretension

Linear equations for the models considering pretension:

$$y = 0.0015x + 0.0016 \quad \text{(Validation model)}$$
$$y = 0.0056x + 0.006 \quad \text{(FEA or Proposed model)} \tag{10}$$

Regression (R^2) values for the models considering pretension:

$$R^2 = 0.8903 \quad \text{(Validation model)}$$
$$R^2 = 0.8903 \quad \text{(FEA or Proposed model)} \tag{11}$$

Linear equations for the models without considering pretension:

$$y = 0.0015x + 0.0016 \quad \text{(Validation model)}$$
$$y = 0.0037x + 0.004 \quad \text{(FEA or Proposed model)} \tag{12}$$

Regression (R^2) values for the models considering pretension:

$$R^2 = 0.8903 \quad \text{(Validation model)}$$
$$R^2 = 0.8903 \quad \text{(FEA or Proposed model)} \tag{13}$$

The moment (M) - rotation (ϕ) comparison graph are shown below in Fig. 12. The moment-rotation graph shows the flexural behavior of the connections in the structure. The shown moment-rotation graph gives the outcome that the connections, in the validation model, have some kind of stiffness in them to give the best results as compared to the proposed model. The linear equations and the regression values (R^2) for the moment- rotation values of both models analyzed in this work are also given below:

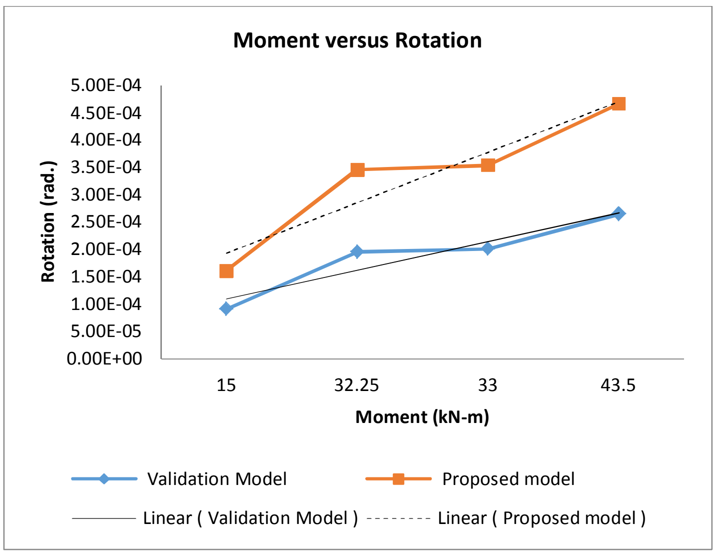


Fig. 12. Moment (M_t) – Rotation (ϕ) Graph

Linear equations for both models considering pretension:

$$y = 5\text{E-}05x + 6\text{E-}05 \quad \text{(Validation model)}$$
$$y = 9\text{E-}05x + 0.0001 \quad \text{(FEA or proposed model)} \tag{14}$$

Regression (R^2) values for the models considering pretension:

$$R^2 = 0.8925 \quad \text{(Validation model)}$$
$$R^2 = 0.8903 \quad \text{(FEA or proposed model)} \tag{15}$$

The fatigue analysis of the bolts in the proposed model is shown below in the Fig.13. As we can see from the analysis below that as the increase in the load takes place, there is a decrease in the life of the bolt and an increment in the fatigue damage of the bolt. The maximum value of the fatigue life occurs in the bottom bolt and whereas the damage occurs at the bottom bolt of the structure as shown in the Fig. 13(g) and 13(h).

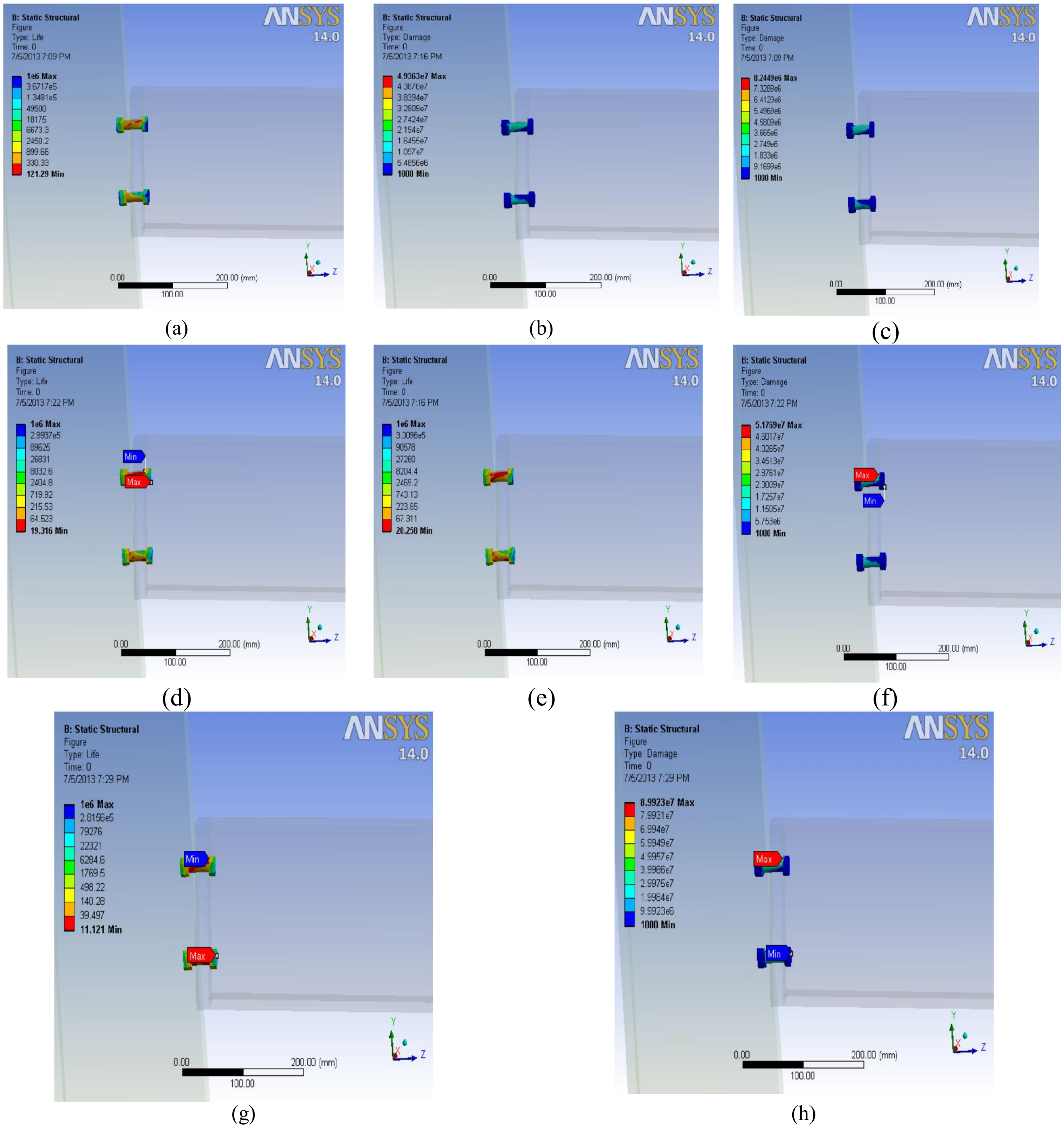
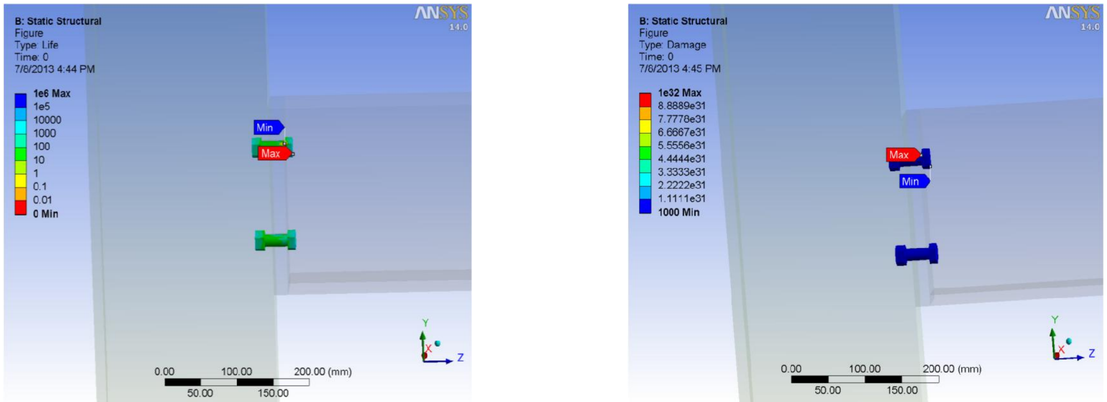


Fig. 13: Fatigue analysis of the bolt in the proposed model, (a) (c) (e) (g) shows the life of the bolts with pretension of 110N at 10, 21.5, 22, 29 kN applied load, respectively, (b) (d) (f) (h) shows the damage in the bolts with the pretension of 110N at the 10, 21.5, 22, 29kN load, respectively

The results of over tensioning in bolts are shown in Fig.14. The minimum ultimate tensile strength (UTS) of the M16 bolt is 1300MPa and its minimum tensile strength is 204kN which is given in the literature (Deepak fasteners LTD, UNBRAKO Engineering Guide). If we apply the pretension more than the upper limit of tensile strength of the bolt, it shows the character which is shown in Fig. 14 under over tensioning which resulted in the fatigue failure with the pre-strain which resulted in the strain hardening of the bolt and ultimately the bolt failure took place.



(a) **(b)**

Fig. 14. (a) Showing the life of the bolts under over tensioning, (b) showing the damage in the bolts under over tensioning

4. Conclusion

In both FEA results it was consistently found that the design theory of the structure underestimated the bolt pretension in the connections. The bolt pretension plays the most important role in the analysis of any bolted structure due to which the desired results can be achieved with high efficiency. FEA can simulate, analyze and compute the mechanical behavior of bolted end-plate connections appropriately. The explicit definition of the joint rotation of the beam to column bolted end-plate connection has been proposed. In many cases, end-plate connections are semi rigid, and the influence of joint stiffness on the behavior of the structure should be considered. Its detailed rotational behavior and design method are to be studied further.

For flush end-plate connections, the moment rotation mainly comes from the relative deformation between the end-plate and the beam flange. For extended end-plate connections, the moment rotation almost comes from the deformation of connection zone of beam to column completely. According to this analysis it can be explained that the flush end-plate connection is more result oriented than the extended end plate connection. Thus it can be used efficiently and effectively in the connection of members in the bridge and shade structures.

In the end it is analyzed that the over tensioning of bolts in the structure can results in the strain hardening and fatigue failure in the bolts which reduces its life and can damage the structure.

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