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# Numerical Evaluation of the Extended Endplate Moment Connection Subjected to Cyclic Loading

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**Abstract-** In this paper, the seismic behaviour of extended endplate moment connection is analysed using finite element method (FEM). First, an existing test setup is modelled and analysed using ANSYS computer program. The model is validated by comparing the results from the finite element with the experimental ones. Afterwards, by changing the dimensions of members of the connection, their effect on the overall seismic performance of connection is investigated. The results show that by enlarging the column depth and stiffening the connection, the seismic performance is improved and the thickness of endplate should be chosen in a way that its moment capacity is larger than the plastic moment of beam.

**Keywords-** Extended Endplate; Moment Connection; Finite Element Method; Seismic Behaviour; Cyclic Loading

## I. INTRODUCTION

Current steel design codes tend to incorporate moment resisting frames for the steel structures built in regions with high seismic activity. This is due to the high ductility of these types of structures. It is clear that their seismic performance is highly dependent on the connection layout. The traditional method of assembling consisted mainly of welding the beam to column. After observing the performance of such connections in Northridge and other earthquakes, it was concluded that due to the high incorporation of welding and the natural brittle manner of welds, this type of connection could not provide the required ductility. Therefore, new details for moment resisting connections were proposed. One of these details, which was also used before Northridge, is called extended endplate connection. In this connection setup, a plate of defined size is welded to the end of beams in shop and then bolted to column flange in field. Thus, the uncertainties of welding in the field are omitted and by using the bolts, it can be more ductile and reliable than welds. Considering the size of connecting members, this connection is fabricated using different layouts. Some of them are shown in Fig. 1.

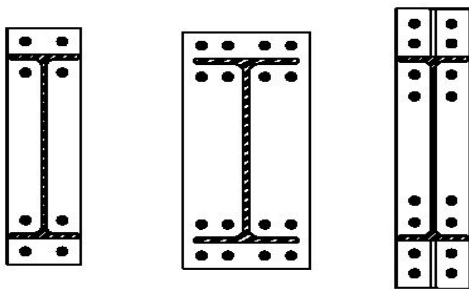


Fig. 1 Different types of extended endplate connection

This type of connection was first used in the United States during the 60's. Krishnamurthy used the finite element method for the analysis and design of the endplate thickness for the first time in 1978 [1]. Ghassemieh *et al.* analysed the behavior of eight bolted stiffened connection under monotonic loading. They used both experimental and finite element analysis and found a good correlation between these two methods [2]. Popov and Tsai investigated the cyclic behavior of this type of connection and found out that the current methods, which used the monotonic results for the design of connection in seismic regions, need to be revised [3]. Bahari and Sherburne modelled this connection in ANSYS computer program and predicted the monotonic behavior of this connection successfully [4]. Adey investigated the effect of beam depth, endplate thickness, and stiffeners on the energy dissipation of the connection. He suggested that increasing the beam depth reduces the energy dissipation and stiffening has a positive effect on this character of the connection [5]. Sumner proposed a unified method for the seismic design of this type of connection [6]. He designed some specimen based on his method and subjected them to the cyclic loading according to SAC loading protocol [7], then this specimens were analysed under monotonic loading using finite element method. Ahuja *et al.* investigated elastic behavior of eight bolted stiffened connection with FEM [8]. Kennedy and Hafez studied moment versus rotation curve for this type of connection and obtained an analytical model. They found good correlation with experimental results [9]. Murray provided a design method for four bolted unstiffened and four bolted extended unstiffened connections [10]. Ghojarah *et al.* studied seismic behavior of this connection. They managed five specimens that in some of them, axial load was also applied on column [11]. Bursi and Jaspart presented an overall investigation on methods of obtaining the moment versus rotation curve [12]. Ryan evaluated the ability of inelastic rotation of extended endplate connection [13]. Mays used the FEM for design of eight bolted extended and unstiffened column connection [14]. Murray and Shoemaker presented recommendations for design of connections with vertical, wind, and low-level earthquake loads [15]. Maggi *et al.* used ANSYS computer program for modelling and they compared the FEM results with experimental results under monotonic loading [16].

Based on the previous studies, extended endplate connection can provide the required strength, ductility, and stiffness for the moment resisting frames in seismic regions.

In addition, finite element method can be used to predict the behavior of the connection under various loading condition. There are so many articles on the analysis and design of this connection, but only a few have investigated the 3D modelling of this connection under cyclic loading considering all of the nonlinearities of the connection. This paper attempts to offset this demand. First, an existing four bolted endplate connection is modelled and analysed under cyclic loadings same as the experimental specimen. The model is full scale and all the major nonlinear properties of the connection are considered. Comparing the results from the model and experimental test has validated the proposed model. Afterwards, by changing the dimensions of members of the connection, their effect on the overall seismic performance of connection is investigated.

II. VALIDATION OF THE FINITE ELEMENT MODEL

In order to verify and validate the FEM model, it is decided that experimental setup which has been assembled and tested previously by Sumner [6], to be modelled and analysed. This setup was subjected to a cyclic loading in accordance with SAC loading protocol [7]. Afterwards, the results from both experimental and FEM are compared with each other and through this process, the model is validated. The typical test setup of model is shown in Fig. 2.

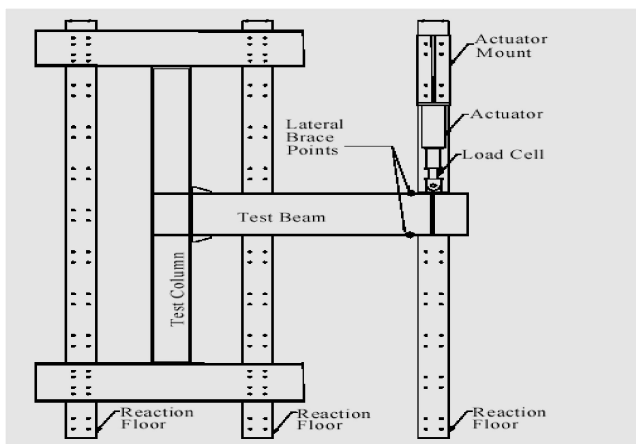


Fig. 2 Test setup configuration [6]

The test setup consists of a cantilever beam that was connected to a column by an endplate connection. This system was moved to a horizontal position and then the column was bolted to the supporting beams and the beams were bolted to the ground, so the column ends are assumed to be pinned. Load is applied to the tip of test beam using an actuator. A roller was placed between test beam and ground in order to eliminate any bending in a plane other than the loading plane. Table 1 summarizes the geometric dimensions of test setup [6].

TABLE I DIMENSIONS OF THE SUMNER TEST SETUP [6]

	Length (in)	Web thickness (in)	Flange thickness (in)	Section depth (in)	Width of flange (in)	$F_y$ (ksi)	$F_u$ (ksi)
Beam	169.75	0.438	0.583	23.875	9.2	53.6	70.7
Column	218.625	0.601	0.933	14.5	14.75	52	70

The beam had enough lateral bracing that lateral-torsional buckling of the beam would occur after the plastic moment capacity. The connection had a four bolted unstiffened endplate setup. Its layout dimensions are shown in Fig. 3 and Table 2. All bolts of connection were pretensioned according to AISC code [17].

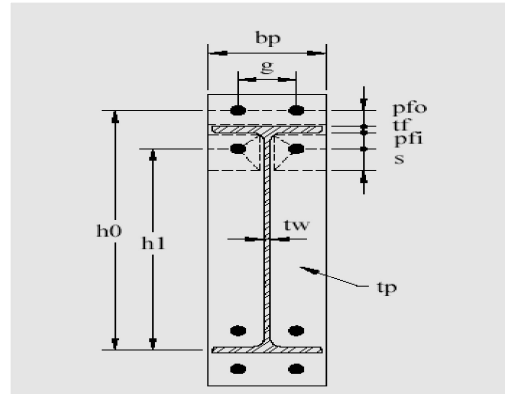


Fig. 3 Four bolted endplate details [6]

TABLE II ENDPLATE DIMENSIONS [6]

$t_p$ (in)	$b_p$ (in)	$l_{pl}$ (in)	$g$ (in)	$p_{flop}$ (in)	$p_{fbot}$ (in)	$F_{y-endplate}$ (ksi)	$F_{u-endplate}$ (ksi)	bolt Gr.	bolt diameter (in)
1.53	10	33.7	5.98	1.71	2.093	38.1	68.8	A490	1.25

In order to model the setup with the FEM, first the geometry is drawn. Endplate, column, and beam are all modelled in the ANSYS computer program and each bolt is modelled by three volumes; head, nut, and the shank that passes through the holes in endplate and the column flange.

It must be noted that due to the symmetry of structure about the vertical plane that passes through the middle of web of beam, only one half of the setup is modelled and because the beam is fully welded to endplate in reality, these parts are modelled continuous to each other. A solid element with eight nodes and 24 degrees of freedom is chosen for modelling the mesh. The element has a capability of modelling the plasticity criterion and large displacements and also accepts thermal loading. Because the bending behavior of endplate is important, the thickness of plate is meshed using three layers of elements. The dimensions of elements grew bigger by moving away from the region and going toward the end of the beam.

Because the endplate is bolted to the column flange, it shows a complex bending behavior. As long as the plate is under compression, it stays in contact with the column flange and force is transferred between these parts. On the other hand, the parts that are under tension, loses contact with each other and therefore no force is transferred between them. Bolts also have the same condition with their surroundings. In order to model this complex contact behavior, which holds shear and pressure transfer and no tension transfer between elements, contact and target elements are used. These elements are mounted on the surfaces of the contact regions (endplate and column flange). The contact elements contain constant factors which

changing each one of those factors would result in changing the overall behavior of model and could result in divergence of analysis before reaching the ultimate capacity of model. In order to avoid this instability phenomenon, one should perform as many as iterative analysis to reach the appropriate values for the specific problem. By attaining the right constant values of the factors, the numerical instability of the model would be a result of excessive plasticity and not the numerical divergence. Contact elements are also on the shank, nut, and head of bolts and on their corresponding target surfaces.

Stress versus strain curves of different steels used in bolts and other parts of the connection have been defined in the program. These curves should account for the plasticity and strain hardening of steel. Two types of stress-strain curves have been proposed in literature [6, 14]. Both curves are illustrated in Figs. 4 and 5.

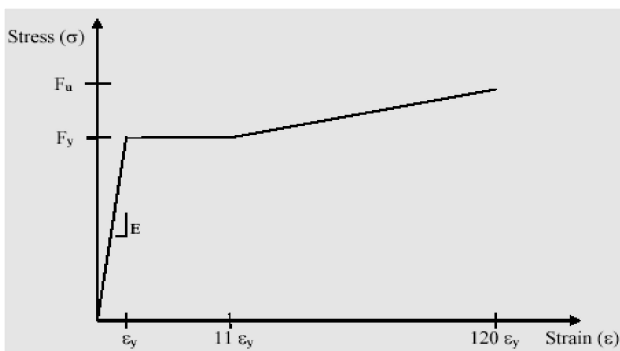


Fig. 4 Stress-strain curve of endplate, beam, and column [14]

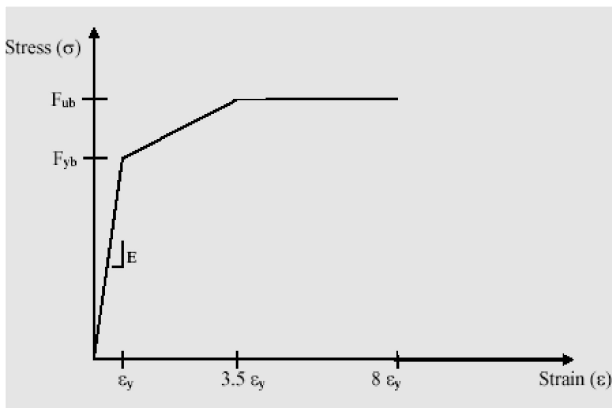
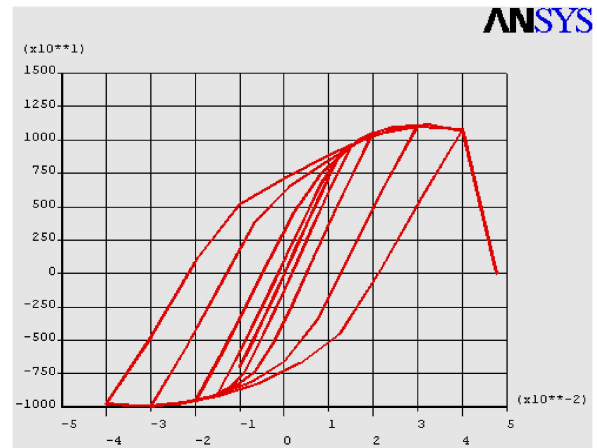


Fig. 5 Stress-strain curve of bolts [6]

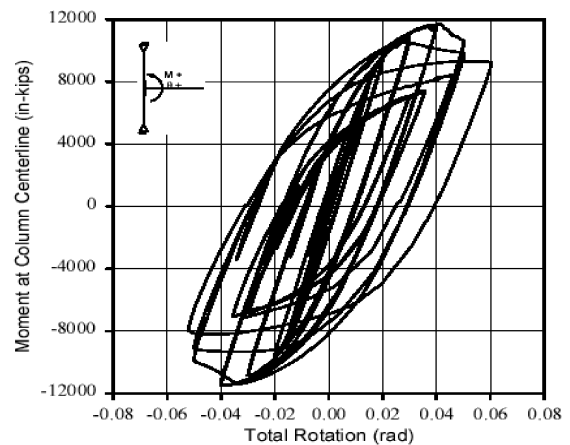
Bolts are more brittle and could bear a lower strain before failure. Elastic modulus and Poisson's ratio of all steels used are 29800 ksi and 0.3 respectively. Loading is applied in two different phases on the FEM model. The first step is the pretensioning of bolts which is done by applying a negative temperature in the shank of bolts. The thermal expansion factor for bolts is defined as orthotropic and only applied in the axes of shank of bolts, which is  $1.2 \times 10^{-5}$ . This is done intentionally in order to prevent the reduction of shank diameter due to the thermal loads. Therefore, the temperature reduction results the stress in the shank of bolts to be equal to the stress of pretensioning. Next phase of loading is the cyclic loading, which is applied in accordance with the SAC loading protocol. In the ANSYS program, this is done by applying a displacement on the tip of beam in a

way that the resulting drift angle would be equal to the values referred in the protocol.

Because the test beam in laboratory had enough lateral bracing to prevent the lateral-torsional buckling and to avoid dealing with the further unknown complicated parameters in model, this phenomenon is not considered in the model. Each displacement loading step value is applied only one cycle on the model. Moment rotation hysteresis loops obtained by Sumner [6] and by FEM program are shown in Fig. 6. Moment is a function of total inter-story drift angle.



(a)



(b)

Fig. 6 Moment-rotation hysteresis loops for FEM (a) and experimental (b) models

Moment is equal to the multiplication of shear at tip of beam to the distance between tip of beam and column centerline. The results for FEM and experimental models are also compared in Table 3.

TABLE III COMPARISON OF THE RESULTS OBTAINED FROM HYSTERESIS LOOP FOR FEM AND EXPERIMENTAL MODEL

	$M_y$ (k.in)	$\theta_y$ (rad)	$M_{max}$ (k.in)	$\theta_u$ (rad)
FEM model	8000	0.012	11300	0.028
Experiment	8600	0.014	11703	0.038

In this table,  $M_y$  is the moment that the first plasticity is observed in model,  $\theta_y$  is the corresponding drift angle,  $M_{max}$  is the maximum applied moment which is not necessarily corresponding to  $\theta_u$ , and  $\theta_u$  is the ultimate

drift angle.

According to the Table 3, it is obvious that the model has a fair accuracy for all the results except maximum rotation ( $\theta_u$ ). The reason for this inaccuracy is that; after the test setup has reached the maximum moment (which corresponds to the initiation of local buckling of members), test specimen begins to rotate more and simultaneously bears a lower moment, but this phenomenon is not modelled in the finite element model. Therefore, when the finite element model reaches the buckling moment, numerical instability happens due to the large deflections and as a result, the calculations diverge. In the other words, the model becomes unstable because of large plastic deformations in the panel zone and beam flanges but the test specimen is failed due to the lateral-torsional and local buckling of members, specially beam.

Von Mises stress distribution around the connection region and the bolt is shown in Fig. 7 for the last step of loading.

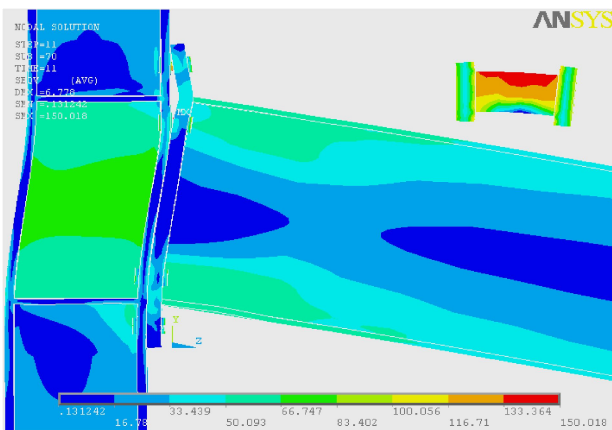


Fig. 7 Stress distribution in 14 in column connection

The followings can be concluded from Fig. 7:

1- Unlike the web, flanges of beam in the area adjacent to the endplate have not yielded, which shows that most of the yielding is due to the moment transfer in this area. By moving away from the connection region, it can be observed that yielding is intensified in web. In addition, it seems that plastic hinge location is in the section 12 in away from the flange of column ( $d_{beam}/2$ ) since the direction of stress is changed in this section. This conclusion is in accordance with studies of Sumner [6].

2- The panel zone of connection has shown a minor plastic behavior, which is not a desirable circumstance. Therefore, this part of connection is relatively weak and needs to be improved.

3- Bolts have no rupture.

4- On top and bottom of endplate in the extended region, there is little stress in both ends, which shows a little force is transferred in this region.

5- The extended part of the endplate outside the exterior bolt is in contact with the column flange, but it is separated from the column flange between the exterior and interior bolts. Thus, the prying action has begun and it may cause the bolts to rupture. This phenomenon will be studied more

by observing Fig. 8.

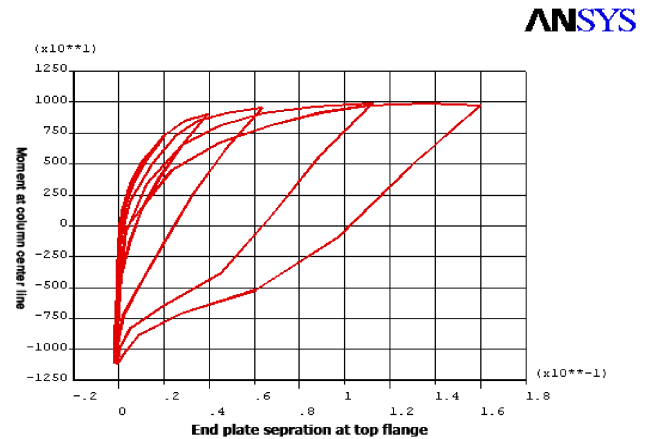


Fig. 8 Applied moment versus endplate separation at top flange

Fig. 8 illustrates that yield moment of endplate is 8000 k.in and since this value is greater than the same value from the hysteresis loop of the structure, endplate is not the first yielding member of the structure. Therefore, as shown in Fig. 7, the plastic hinge location is far away from the face of column. According to FEMA considerations, this is a desirable condition in designing moment resisting frames [18]. In addition, since the separation has a continuous trend, the bolts of this part have not ruptured. Endplate is contributed in the energy dissipation.

By collecting all the information from the previous figures, the reason for failure of the connection in the finite element model is excessive plasticity in beam or column with no rupture of bolts or endplate, which is in accordance with the experimental observations. This should be noted that no report from the column behavior is presented in experimental reports.

### III. VERIFICATION OF THE EFFECTIVE PARAMETERS IN THE BEHAVIOR OF CONNECTION

The behavior of a connection changes due to many parameters. In this section of the paper, effective parameters are discussed.

#### A. Effect of the Column Depth

As discussed in previous sections, the panel zone of original connection has shown a minor yielding which is not a desirable condition. In order to overcome this flaw, the panel zone should be improved. There are two solutions for this problem:

- The desirable depth for column should be chosen.

- The desirable thickness of the column web should be chosen.

According to AISC, the following equation should be considered in designing the panel zone [17]:

$$V_{pz} < V_y \quad (1)$$

where  $V_{pz}$  is the design shear in column centerline and  $V_y$  is the ultimate shear capacity of panel zone. According to this code,  $V_y$  and  $V_{pz}$  can be calculated from the following equations [2]:

$$V_y = 0.6F_{yc}d_c t_{wc} \left\{ 1 + \frac{3b_c t_{cf}^2}{d_b d_c t_{wc}} \right\} \quad (2)$$

$$V_{pz} = 0.8 \frac{M_{pb}}{d_b} \quad (3)$$

in which  $M_{pb}$  and  $d_b$  are plastic moment and depth of beam, respectively.  $F_{yc}$ ,  $d_c$ ,  $t_{wc}$ ,  $b_c$ , and  $t_{cf}$  are yield stress, depth, web thickness, flange width, and flange thickness of column, respectively.

$M_p$  could be calculated using the following classic equation:

$$M_p = Z.F_y \quad (4)$$

where  $F_y$  and  $Z$  are the yield stress and the plastic modulus of beam, respectively.

However, by considering the stress hardening effects of steel, the most accurate equation is presented in FEMA-350 for calculating  $M_p$  [18]:

$$M_p = 1.1 \left[ \frac{F_y + F_u}{2} \right] Z \quad (5)$$

where  $F_u$  is the ultimate stress of steel.

If the properties of the current structure are applied to Eq. 2, shear capacity of the connection would be 301.11 kip. If Eq. 4 is considered for the plastic moment and the result is used for the existing shear in Eq. 3, the result for shear design ( $V_{pz1}$ ) would be 314.7 kip. On the other hand, if Eq. 5 is used instead of Eq. 4, the result for shear design ( $V_{pz2}$ ) would be 403.7 kip. It is clear that in both conditions, the shear capacity of panel zone is lower than the shear design. In order to retrofit the connection, the desired column depth is designed using each of the equations separately. The results are:

$$\begin{aligned} V_{pz1} = 314.7 &\Rightarrow d_{c1} = 15.3 \text{ in} \Rightarrow d_{c1} = 16 \text{ in} \\ V_{pz2} = 403.7 &\Rightarrow d_{c2} = 20.3 \text{ in} \Rightarrow d_{c2} = 21 \text{ in} \end{aligned} \quad (6)$$

In order to verify the effect of column depth in overall performance of the connection, two new models with only the difference in column depth and one additional 18 in have been modelled and analysed in the finite element program. Hysteresis loops are presented in Fig. 9, and Table 4, these results are compared to each other.

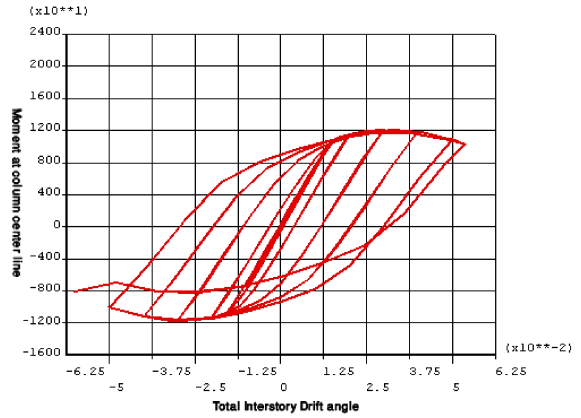


Fig. 9 Moment-rotation hysteresis loops for connection of 16 in (a) and 21 in (b) column depth

TABLE IV COMPARISON OF THE RESULTS OBTAINED FROM HYSTERESIS LOOP FOR CONNECTIONS WITH DIFFERENT COLUMN DEPTH

Column depth (in)	$\theta_y$ (rad)	$M_y$ (k.in)	$\theta_p$ (rad)	$M_{max}$ (k.in)
16	0.013	9400	0.037	11300
18	0.014	10800	0.046	12000
21	0.014	10800	0.046	12000

It is clear from Table 4 that, stiffness and plastic rotation capacity of connection have been improved by increasing the column depth.

Von Mises stress distribution in the last step of loading in both connections is shown in Figs. 10 and 11.

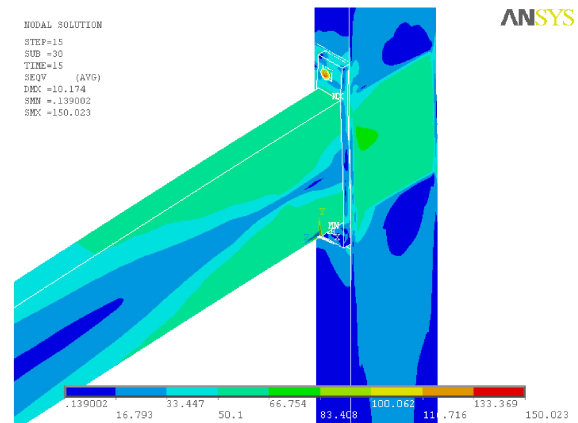


Fig. 10 Von Mises stress distribution for 16 in column depth connection

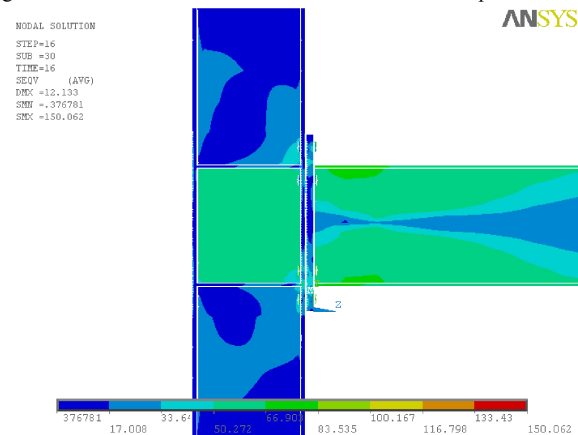
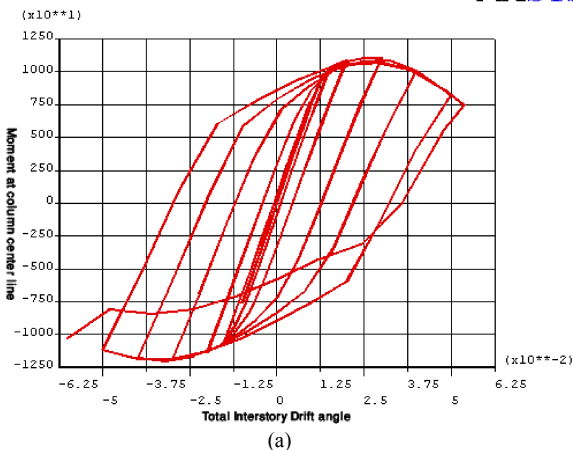


Fig. 11 Von Mises stress distribution for 21 in column depth connection





Comparing these two figures leads to the following conclusions:

The stress in the panel zone of 16 in column connection is greater than that of beam near this region. So yielding in column starts before beam, but this trend is reversed for the 21 in column setup. Therefore, the ideal behavior is achieved by 21 in column depth. As a result, it is advisable to calculate the plastic moment by Eq. 5.

For both setups, the plastic hinge location in beam is in a distance of  $d_{beam}/2$  from column flange. Therefore, it seems that this plastic hinge in this connection is developed far from the flange of column that is a desirable seismic behavior, and column depth has no effect on the location of plastic hinge.

**B. Effect of Endplate Thickness**

In order to analyse the effect of endplate thickness on the overall behavior of connection, the required endplate thickness is calculated using the equation based on yield line theory presented by Srouji [19]:

$$M_{pl} = F_{yp} \cdot t_p^2 \left\{ \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_f} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_{f0}} - \frac{1}{2} \right) + \frac{2}{g} [h_1 (p_{fi} + s)] \right] \right\} \tag{7}$$

where variables  $h_1$ ,  $h_0$ ,  $p_{fi}$ ,  $p_{f0}$ ,  $b_p$ ,  $s$ , and  $g$  are illustrated in Fig. 12.

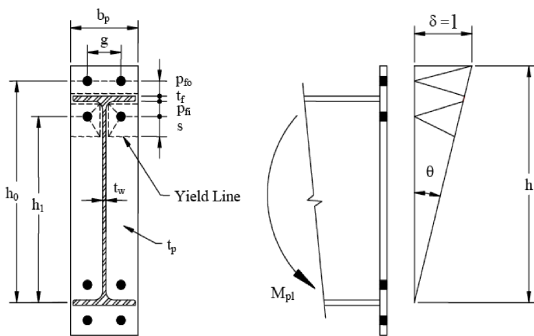


Fig. 12 Controlling mechanism and design parameters for a four bolted endplate [19]

$t_p$  and  $F_{yp}$  are the thickness of endplate and the yield stress of steel, respectively. The parameter  $s$  is calculated from the following equation:

$$s = 0.5 \sqrt{b_p} \cdot g \tag{8}$$

If original dimensions of connection are used in Eq. 7, the result for  $M_{pl}$  would be 16362 k.in.

Clearly, the ultimate moment of endplate is greater than the plastic moment of beam. Therefore, the thickness of endplate is sufficient for this connection, and as shown, this connection failed due to the local buckling of beam and excessive plasticity in panel zone without any rupture in endplate.

According to FEMA considerations, for design of endplate thickness in Eq. 7,  $M_{pl}$  is chosen equal to plastic

moment of beam, the plastic moment could be calculated by any of the two methods. As a result, there would be two different calculated values for endplate thickness, 1.43 and 1.21 in. In order to investigate this effect clearly, three values of 1, 1.25, and 1.5 in are chosen for endplate thickness. Therefore, two new analyses should be performed. The corresponding moment for 1 in thickness plate would be 7272 k.in. This value is less than the plastic moment of beam. Hysteresis loops for both of these connections are shown in Figs. 13 and 14.

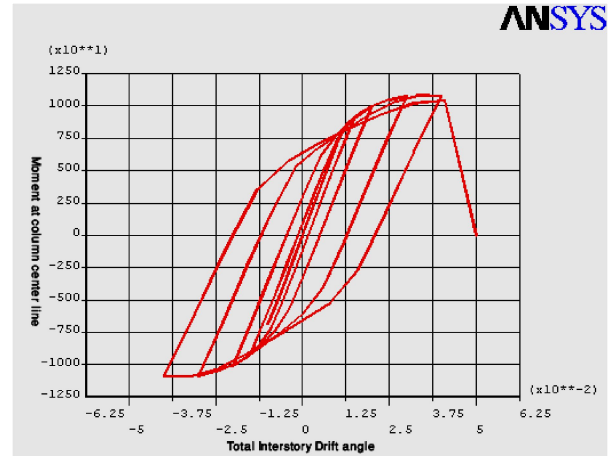


Fig. 13. Moment rotation hysteresis loops for connection with 1.25 in endplate

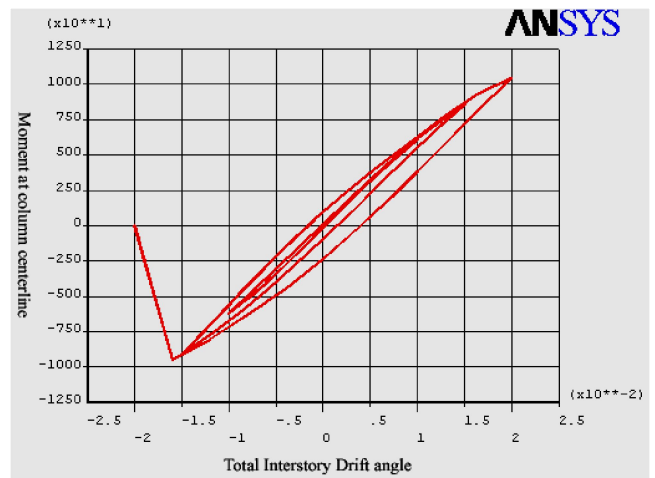


Fig. 14 Moment rotation hysteresis loops for connection with 1 in endplate

The results are presented in Table 5:

TABLE V COMPARISON OF THE RESULTS OBTAINED FROM HYSTERESIS LOOP FOR UNSTIFFENED CONNECTIONS WITH DIFFERENT ENDPLATE THICKNESS

Endplate thickness $s$ (in)	$\theta_y$ (rad)	$M_y$ (k.in)	$\theta_p$ (rad)	$M_{max}$ (k.in)
1.25	0.011	7600	0.029	11000
1	0.01	6250	0.01	10100

Comparing these values with the same values from original connection would result in:

1- Plastic rotation in 1 in plate has become severely decreased, so the connection is an ordinary moment

resisting frame. On the other hand, the 1.25 in plate is still in special moment resisting frame classification.

2- Severe decrease in plastic rotation suggests that second connection would probably have a brittle failure and the first point of plastic deflection is the endplate, which is not desirable situation.

It could be noted that the full plastic moment capacity of beam is not reached before the failure of structure and the plastic hinge is going to be 6 in away from the column flange. Therefore, the plastic hinge has shifted towards the column flange, which is not a desirable situation.

Large plasticity is clear in so many regions of plate and in some areas the stress is near the failure stress (55 ksi) but the maximum stress is about 40 ksi in original connection. In the other words, stiffness of beam and column are relatively higher than the same value of plate, this phenomenon causes a large stress to take place in the tension flange of beam and induces prying action in endplate, and a large force in bolts. Thus, the failure is more brittle. This effect is illustrated in Fig. 15. In this figure, the bottom flange is under tension.

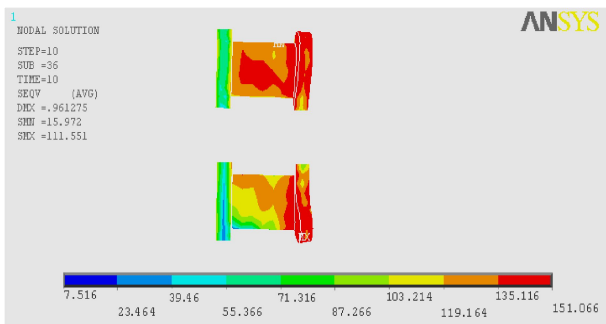


Fig. 15 Von Mises stress distribution in bolts of connection with 1.25 in endplate

It is clear from the Fig. 15 that the internal bolt bears a large tension and this is an effect of prying action due to the reduction of thickness of endplate. Therefore, the reduction of thickness induces more force to bolts in addition to the increasing of the stress of endplate. In order to analyse this effect further, separation of endplate from column flange versus the applied moment in column centerline is shown in Fig. 16.

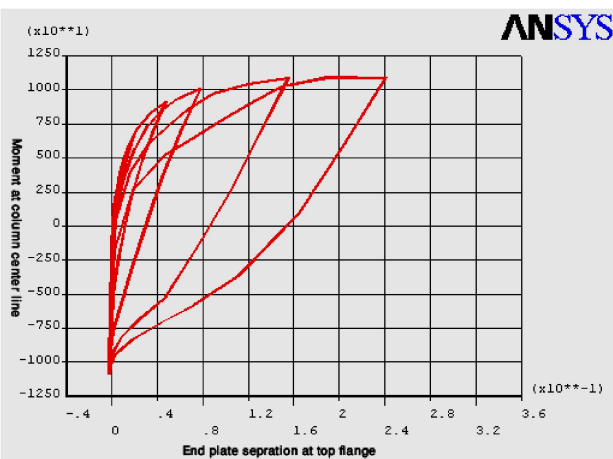


Fig. 16 Applied moment versus endplate separation for 1.25 in plate

The most important point in Fig. 16 is that the separation has increased greatly than the original connection in all load steps. For example in last loading step, this value is 0.24 in, which is 1.5 times of the corresponding value of the original setup (0.15 in). This shows a huge energy dissipation and stress in this part of connection.

Same inspections are done for the 1 in thickness of endplate. It is observed that the failure has become more brittle and the condition of failure is the rupture of endplate and bolts with no plastic hinge in beam. This classifies the connection as ordinary moment resisting frame according to AISC code. This is expected, since the failure moment of endplate is less than the plastic moment of beam. Therefore, it is recommended to design the endplate using a moment greater than plastic moment of beam.

C. Effect of exterior stiffener

This stiffener includes two steel plates, which are welded on beam flanges to the extended part of endplate. Thicknesses of plates are chosen equal to thickness of beam web and their height ( $h_{st}$ ) is equal to extended part of endplate [6, 20]. Their length ( $L_{st}$ ) is chosen assuming a 30 degree angle for tensile load distribution.

The stiffener is modelled on the original structure and the analyses have been done again. Resulting hysteresis loop is shown in Fig. 17.

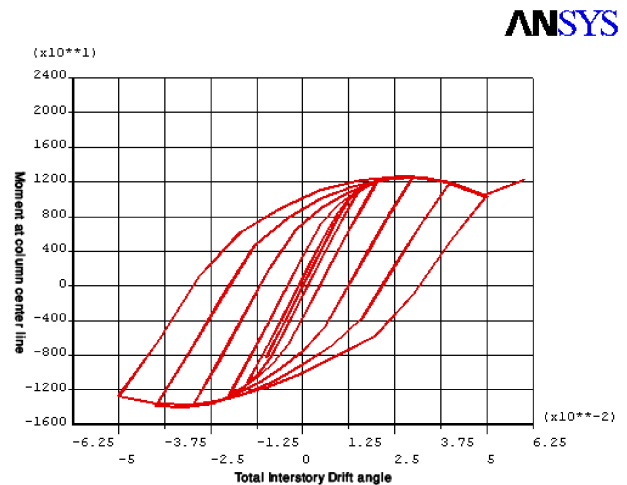


Fig. 17 Moment-rotation hysteresis loops for stiffened connection with 1.5 in endplate

In order to analyse this effect further, the stiffener is also modelled on the connection with 1.25 in plate. The results from these two setups have been summarized in Table 6.

TABLE VI COMPARISON OF THE RESULTS OBTAINED FROM HYSTERESIS LOOP FOR STIFFENED CONNECTIONS WITH DIFFERENT ENDPLATE THICKNESS

Endplate thickness (in)	$\theta_y$ (rad)	$M_y$ (k.in)	$\theta_p$ (rad)	$M_{max}$ (k.in)
1.5	0.013	10000	0.037	12950
1.25	0.012	8000	0.018	10500

By comparing Tables 5 and 6, it is clear that stiffening the connection would result in a better stiffness and plastic



rotation. Therefore, the overall behavior of connection has been improved and stiffener has no effect on delaying the yielding in connection. These results suggest that by stiffening, a lower thickness could be chosen for the plate.

Von Mises stress distribution is shown in the last step of loading for a stiffened connection in Fig. 18.

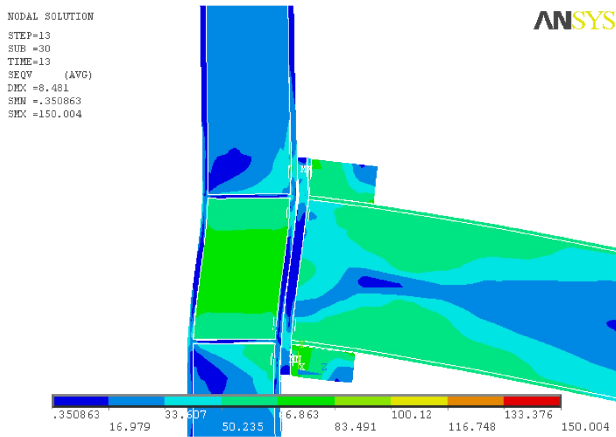


Fig. 18 Von Mises stress distribution for stiffened connection with 1.5 in endplate

It is evident from the figure that plastic hinge location is  $0.25d$  after edge of stiffener. This is general conclusion for every stiffened connection. According to FEMA guidelines, the plastic hinge location should be taken right after the edge of stiffening plate, which is a conservative assumption. Moreover, the stress in beam flange in area adjacent to the stiffener is reduced; this shows that the plastic hinge location is moved toward the beam tip. As a result, the overall seismic behavior of connection is improved by stiffening the endplate.

In order to further analyse this effect, the curve of separation of endplate from column flange versus the applied moment for the current setup is shown in Fig. 19.

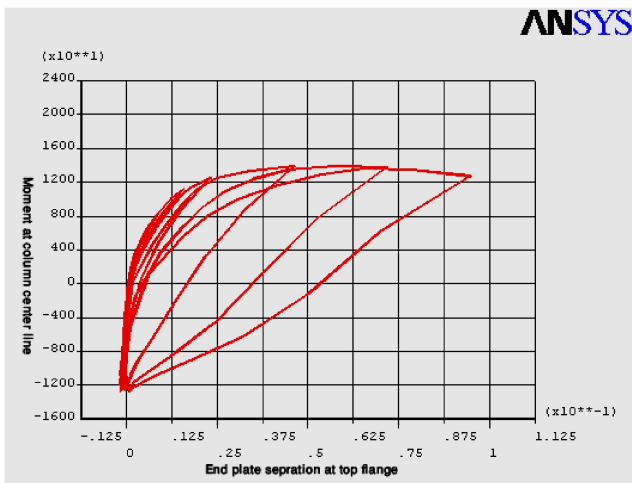


Fig. 19 Applied moment versus endplate separation for stiffened connection with 1.5 in endplate

It is evident from Fig. 19 that for each loading step, the corresponding separation of endplate from column flange is significantly reduced. For example, for  $\theta = 0.04 \text{ rad}$  this value is  $0.07 \text{ in}$  which is lower than the corresponding value

from the original endplate which is  $0.16 \text{ in}$ . This proves that the stiffened connection would have a lower prying action and therefore, a lower force is induced in bolts.

Last considerable tip is that some regions of stiffener have a little stress. As a result, the best shape for stiffener is like Fig. 20. Using this layout would result in an easier welding process for stiffener.

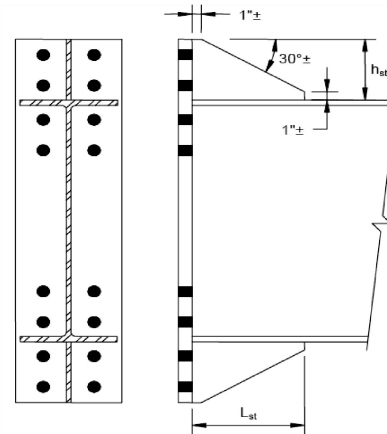


Fig. 20 Stiffener plate details

#### IV. CONCLUSIONS

The seismic behavior of endplate connection was investigated numerically using the finite element method of the analysis in this paper. The investigation showed that column depth has no effect on the location of plastic hinge, however it is advisable to design the panel zone using the equation from FEMA for the plastic moment of beam. Also it proved that the endplate thickness should be chosen in a way that the ultimate moment capacity of it is greater than the plastic moment of beam. If this connection is designed using the mentioned guideline, the plastic hinge location will probably take place in a distance of half depth of beam from the endplate. This is in accordance with FEMA considerations, and is a desirable seismic design. In addition, presence of an exterior stiffener at the connection would improve its overall seismic behavior.

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