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Bolt Pre-tension Effect on Performance of Bolted Extended End-plate Moment Connections under Cyclic Loading

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ABSTRACT

The most important structural part of steel structures is the connections of the building frame. Tests conducted by the American Steel Institute have led to the introduction of prequalified connections, which is a good reference for designing connections in steel structures. In this paper, four bolted stiffened and unstiffened extended end-plate connections have been studied by numerical analysis with ABAQUS software. In this study, to examine the effect of bolt pre-tension rarely studied, a coefficient of pre-tension force, introduced based on the Iranian Steel Structures Design Regulations as well as the US Steel Design Regulations, has been considered. These connections have been modeled cyclically and in a displacement control manner. The cyclic behavior of the connections, dissipated energy, the resisting moment as well as the stress and strain distribution in the connection have been investigated. According to the results, by creating a pre-tension force in the bolts, the resisting moment of the connection would increase. The rate of growth of this resistance in the unstiffened connections was greater than that of the stiffened connections. The maximum increase in resistance was about 27% for the unstiffened connection and about 25% for the stiffened connection. The dissipated energy for the connections also increased with the increment of bolt pre-tension. The energy dissipation incremental rate was enhanced to a maximum of about 31% for the unstiffened connection and up to a maximum of about 24% for the stiffened connection.

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NOMENCLATURE

M	Moment resistance of a joint	K_1	$K_i - K_p$
Mp	Plastic moment capacity	n	Shape parameter
K_i	Initial stiffness	M_0	Reference moment
K_p	Strain-hardening stiffness	φ	Rotation of a joint
C	Empirically coefficient from test data	θ	Rotation of a joint

1. INTRODUCTION

In recent years, the implementation of high-rise steel structures with special steel moment frame systems using prequalified bolted connections has increased significantly. Among these, the main connections utilized are bolted extended end-plate moment connections with and without stiffeners, as well as bolt flange connections due to the high ductility of these connections; considering the limitations mentioned in Regulation AISC-358 [1], they are accepted as prequalified connections.

The bolted extended end-plate moment connections are one of the oldest beam-to-column moment connections, which can only be used with cruciform, box, and I-shaped column sections. Many researchers have examined the behavior of the bolted extended end-plate moment connections.

In 1990, Murray [2] introduced the process of designing four bolted stiffened and unstiffened extended end-plate, and eight bolted stiffened extended end-plate moment connections. Astaneh-Asl [3] explored two 4-bolted unstiffened and stiffened extended end-plate

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specimens subjected to a cyclic test in 1999. In the first specimen, the ductility of the connection, which eventually led to the buckling of the beam flange, was investigated. In the second specimen, he used I-shaped shim plates between the endplate and the flange of the column. According to his results, the use of a shim plate could improve the performance of the connection.

In 1995, Jaspart and Maquoi [4] inspected the effect of bolt pre-tension on bolted connections. In their research, they proposed equations to estimate the stiffness and strength of the connection. Bahaari and Sherbourne [5] presented an analytical formulation to represent the moment-rotation relationship of extended end plate connections in 1997. A multiple–regression analysis procedure has been used to derive the parameters in terms of the connection description.

In 1998, Faella et al. [6] evaluated the effect of bolt pre-tension on the behavior of bolted connection with T-joint under axial loading by an experimental study. In 2000, Adey et al. [7] evaluated the parameters of beam dimensions, bolt arrangement, endplate thickness, and endplate stiffeners in the potential of energy dissipation of the bolted end-plate connection through testing on 15 connection specimens under cyclic loading. They found that the energy dissipated capacity diminished with increasing beam dimensions, while end-plate stiffeners led to higher energy dissipation. In 2003, Sumner [8] experimentally investigated the behavior of 4 bolted unstiffened extended end-plate and 8 bolted stiffened extended end-plate connections. They observed that the 4 bolted unstiffened extended end-plate connection and the 8 bolted stiffened extended end-plate connection can be used for seismic resistance in steel moment frames. Diaz et al. [9] presented a review on the modeling of joint behavior in steel frames in 2011. Various methods of modeling the rotational behavior of joints, consist of experimental testing, empirical models, analytical models, mechanical models and numerical models reviewed and compared together. Comparing the various methods, experimental and numerical models are more accurate.

In 2011, Gerami et al. [10] indicated that the cyclic behavior of bolted connections, including the bolted extended end-plate moment connections and the T-joint, would depend on the horizontal and vertical arrangement of the connection bolts. They found that by changing the arrangement of the bolts, the probability of rupture in the T-joint would increase compared to the connection with the endplate. Accordingly, they suggested the use of bolted extended end-plate moment connections in situations where the probability of a constructional defect in the execution of structures increases. In 2017, Morrison et al. [11] investigated the effect of end-plate stiffener removal and different bolt arrangement on the 8 bolted stiffened extended end-plate connection. They concluded that removing the end-plate stiffener would be economically viable, despite the need for higher thickness for the endplate.

In 2018, Guo et al. [12] used the static loading test, examined the effect of the strength of high-strength bolt materials and different bolt arrangements on the load bearing capacity as well as deformation of the bolted connection with the cover plate. Elsewhere, having studied parameters such as shear force, bolt diameter, endplate thickness, and the use of end-plate stiffener. In 2019, Elsabbgh et al. [13] evaluated the behavior of bolted un-stiffened extended end-plate moment connection under cyclic and monotonic loading. They observed that shear force had a significant effect on the connection stiffness. Lyu et al. [14] experimentally and numerically explored the effect of bolt pre-tension on the load bearing capacity of bolted connections with cover plate in 2021. They indicated that the effect of bolt pre-tension on the final tear-out failure mode was negligible. On the other hand, out-plane confinement would limit the piling-up of plate material in front of the bolt, which would reduce the related bearing resistance.

Recently, the construction of high-rise buildings has increased significantly worldwide. Studies also show that the bolted extended end-plate moment connections can be used in high-rise steel structures as a part of the lateral seismic resistance system of the moment frame. On the other hand, the implementation of such structures demands considerable time, due to administrative and financial issues and problems. Thus, due to the relatively long time of construction, it is important to consider the loads during construction and control the stability of the structure for different stages of construction.

In a complete structure, all connection bolts are fully pre-tensioned in the prequalified moment connections, and the structure will have acceptable seismic performance. However, during the construction of steel structures, the executive groups of steel structures usually fabricate some story of the structure and tighten the connection bolts, which is called the snug-tightened bolt in this paper. As the construction of the structure is completed, the lower stories, in which the connection bolts are snug-tightened, will be pre-tensioned at least to the level of preloading based on design codes, which in this paper is called the pre-tensioned bolt, so the connections are completed, while some of the upper stories have snug-tightened bolts. This process will continue until the construction of the structure is completed. Accordingly, it is demonstrated that the stiffness of the bolted connections changes during construction and the structural properties vary over time. Hence, to cover the gap between the design regulations, which mainly focus on the design of the structure in a full execution state, it is necessary to examine the seismic behavior of the bolted extended end-plate moment connections in both snug-tightened and pre-tensioned bolt with the effects during the construction of high-rise structures while also considering the looseness and tightness of the connection bolts until the construction is completed.

A few studies have dealt with the effect of bolt pre-tension on the behavior of bolted extended end-plate moment connections. Considering the significant construction period of high-rise steel structures and their stability depends on bolt pre-tension effect on the behavior of bolted end plate moment connections, this paper has inspected the effect of different levels of bolt pre-tension on the behavior of bolted extended end-plate moment connections.

2. ANALYTICAL FORMULATION

Analytical models use the basic concepts of structural analysis: equilibrium, compatibility and material constitutive relations, to obtain the mathematical formulation leading to computing the rotational stiffness and moment resistance of a joint due to its geometric and mechanical properties.

In 1986, Yee and Melchers [15] proposed a mathematical model that could predict the moment-rotation relationships of bolted extended end plate connections, using the connection dimensions. The model represents a physically based approach to predict the moment-rotation curves, taking into account the possible failure modes and the deformation characteristics of the connection elements. The mathematical formulation to predict the M-θ relationship is given by Equation (1):

$$M = M_p \left\{ 1 - \exp\left[\frac{-(K_i - K_p + C\theta)\theta}{M_p} \right] \right\} + K_p \theta \qquad (1)$$

To use the expression in Equation (1), the parameters M_p, K_i and K_p must be predicted analytically, where are plastic moment capacity, initial stiffness and strain-hardening stiffness, respectively. The value of C is empirically determined from test data. Bahaari and Sherbourne [5] introduced Equation (2) to account for moment-rotation behavior of bolted extended end plate connections based on the stiffener properties of the connection elements in 1997:

$$M = \frac{K_1 \phi}{\left(1 + \left| \frac{K_1 \phi}{M_0} \right|^n \right)^{(1+n)}} + K_p \phi \qquad (2)$$

Where $K_1 = K_i - K_p$; K_i and K_p represent the initial and strain hardening stiffness of the connection, respectively; M_0 is referred to as reference moment; and n is the shape parameter of the M- ϕ curve.

Since the analytical formulations have limitations to account for the local effects on rotational behavior of joints, numerical modeling and experimental testing are proposed to predict the moment-rotation performance of joints with sufficient accuracy [9].

3. FINITE ELEMENT MODELLING

Since the numerical simulation could be account for the important local effects that are difficult to measure with sufficient accuracy, e.g. prying and contact forces between the bolt and the connection component, is used to assess the rotational behavior of bolted extended plate connections. In this paper, the ABAQUS finite element program was used to numerically model the bolted extended end-plate moment connection. To verify the results of the program, one of the experimental bolted connections, studied by Saberi et al. [16], was numerically modeled. Experimental and numerical results will be compared to investigate the verification of numerical results of bolted connections, modeled using ABAQUS.

3. 1. Verification The connection test model was used as an experimental study conducted by Saberi et al. [16] called (EP-R) for verification. The specimen represents a corner connection according to the substructure proposed in FEMA350 [17]. Table 1 reports the geometric specifications of the connection. Test setup and boundary conditions are very important for cyclic loading tests of connections extra page charges are settled. The other opportunity is for an author to reduce the size of the manuscript before final submission. Reduction after processing is not possible.

The test setup and boundary conditions are displayed in Figure 1. As it can be seen, the column is in the horizontal situation and the beam is in a vertical position in this setup. Two pinned supports are used to connect the column to the rigid floor of the laboratory. According to FEMA350 [17], the standard SAC loading is applied by two one-way 50 ton hydraulic jacks to beam tip until specimen failure. To prevent lateral buckling, out-of-plane displacement of beam is constrained using lateral support installed in the load applying point. Strain gauges are placed on beam flange and web, column web, endplate and bolts. Mechanical material properties of various parts of the connection are mentioned in Table 2.

TABLE 1. Geometric specifications of connection components [16]

member	Specification
beam	IPB140
column	IPB200
Number of bolts (material category)	8(A490)
bolt diameter (mm)	M20
Endplate thickness (mm)	25

TABLE 2. Mechanical properties of the used materials [16]

Used part	Materials	Yield Stress (Mpa)	Ultimate stress (Mpa)
Beam, column, end plate	ST37	245	372
bolt	A490	-	1048

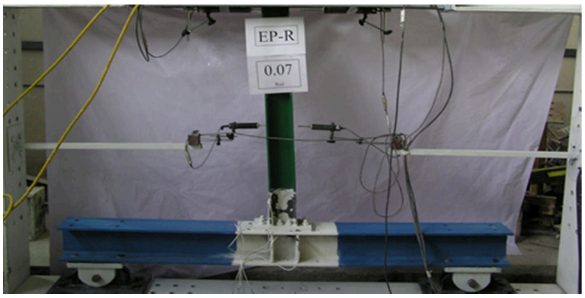


Figure 1. Test setup [16]

In numerical modeling, the dimensions and geometry of the beam, column, and other connection components have been considered similar to the test specimen. As the end plate and beam would be connected through fully penetration groove weld, these two parts were continuously modeled in the finite element model. The connection was subjected to cyclic loading based on SAC loading protocol according to FEMA350 [17] using a two one-way 50 ton hydraulic jack to beam tip until specimen failure, as shown in Figure 2.

3. 2. Comparison of Numerical and Experimental Results

At the end of loading, the formation of the plastic hinge occurred on the flange of the beam at a distance of 7cm from the end plate (Figure 3). On the other hand, as shown in Figure 4, after applying a cyclic load to the numerical model as in the experimental, the plastic hinge has been formed at 7cm away from the endplate.

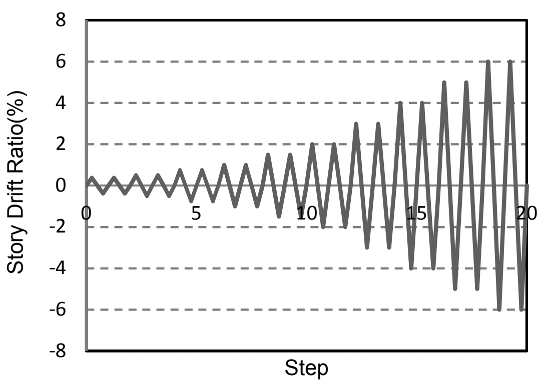


Figure 2. FEMA/SAC2000 loading protocol in accordance of FEMA350 [17]

Figure 5 also reveals the hysteretic moment at the column centerline versus total rotation obtained from the numerical study. The moment capacity in the bolted extended end-plate moment connection at 0.07 rad is 108.3 KN.m for the test specimen and 113.5 KN.m for the numerical model. Comparison of the results of the numerical and experimental study indicates a difference of 4.8%. Thus, the results of the numerical and experimental study are acceptable.

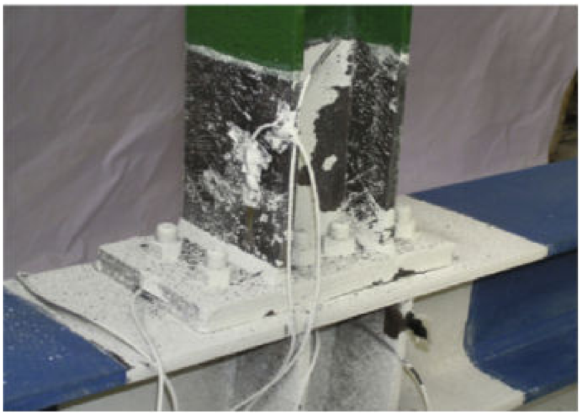


Figure 3. Beam flange deformation at the end of the loading [16]

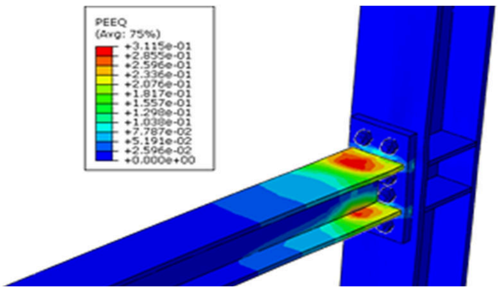


Figure 4. Formation of plastic hinge in beam flange in numerical modeling at the end of loading

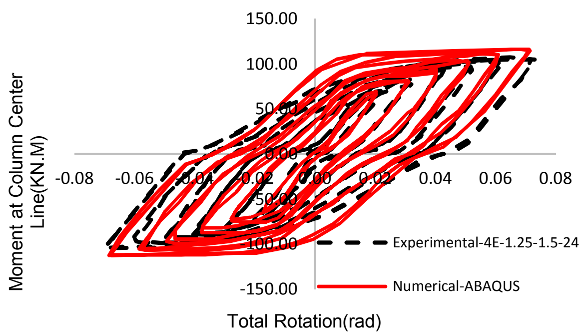


Figure 5. The hysteretic moment at column centerline versus total rotation

4. NUMERICAL STUDY OF THE EFFECT OF BOLT-PRETENSION ON CONNECTION BEHAVIOR

After verification of the numerical model in the ABAQUS program, this section studies the effect of bolt pre-tension on the prequalified connection behavior of the bolted endplate. To numerically examine the effect of pre-tension on the connection behavior, instead of modeling the entire frame, the connection can be separated from the inflection points, where the moment is zero; by placing pinned supports to resist the shear on these points, the substructure will be modeled and analyzed. The substructure represents a beam-to-column connection on the corner of the structure. Figure 6 shows the substructure schematically. The specimen studied in this research has been modeled accordingly. In the numerical study, the length of the beam and column has been considered 3 m. The geometric characteristics of the connection components have been outlined in Table 3. The cyclic loading based on SAC protocol has been applied to the specimen following FEMA350 [17]. Table 3 lists the steel and bolt material properties, which is the criterion for defining the material properties in numerical modeling.

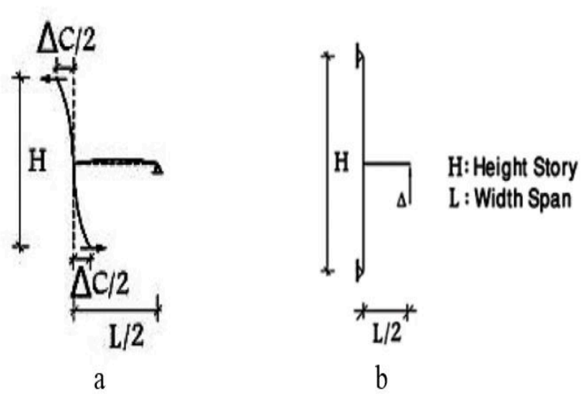


Figure 6. Flexural frame under lateral load: a) external connection isolated from inflection points, b) substructure used in numerical studies of specimens [17]

TABLE 3. Mechanical properties of the used materials [16]

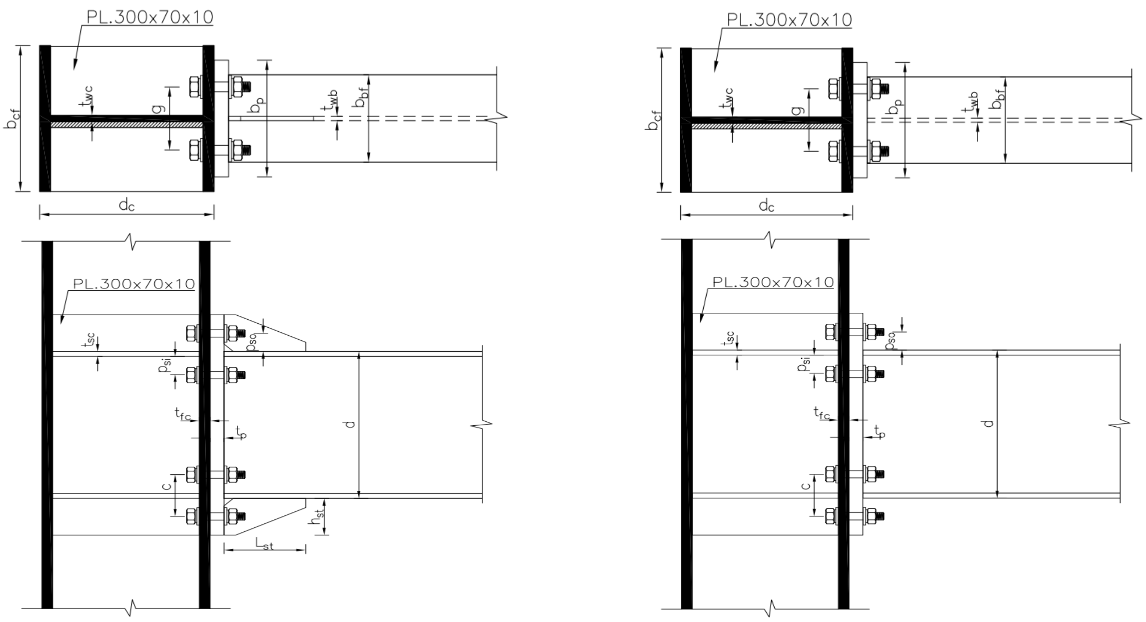
Material	Part	Strain	Stress (Mpa)
ST37	Beam, column, endplate	0.001143	240
		0.02	240
		0.18	360
		0.2	370
		0.35	370
A490	bolt	0.00386	794
		0.0135	1035
		0.0309	1035
		0.2	1048

Figure 7 illustrates the characteristics of the connection components, including beams, columns, bolts, doubler plate, and connection plates, along with the connection geometric parameters reported in Table 4. Figure 8 presents the connection meshing in the program.

To inspect the effect of bolt pre-tension on the cyclic behavior of the bolted extended end-plate moment connections, two types of bolted stiffened and unstiffened extended end-plate moment connections have been studied. Table 5 presents the different levels of pre-tension bolt loads examined. Parameter α in the table indicates the pre-tension coefficient of the bolt relative to the bolt pre-tension following the Iranian national Design Code of Steel Structures [18]. In the model, where α is considered equal to one, the bolt pre-tension of the code is applied to the bolts. In numerical modeling, specimen meshing is done with a 3D stress model and in a structural form. This element uses reduced integration where a node is used at each edge intersection. The use of reduced integration leads to enhanced analysis speed and accuracy. On the other hand, to be able to observe the interaction in the contact surfaces of the connection, the contact element has been used tangentially with a coefficient of friction of 0.3 while the vertical interaction has been used as Hard Contact. This interaction prevents the nodes of the elements from collapsing at the point of contact. Cyclic displacement has been applied as a loading protocol at the end of the beam. This loading is in the form of control displacement. To provide the conditions for lateral restraint of the beam, the connection has been restricted at three points of the beam against lateral displacement. This constraint has been applied by restraining the out-of-plane displacement. Loading has been conducted in two steps. In the first step, pre-tensioning of the bolts has been done and then at the end of the beam, the displacement is applied as a cyclic load. The tie constraint has been utilized to connect the continuous plates, the column flange, and the web to each other. In this way, the nodes of both elements are connected and the relative displacement between them is constrained. This can be represented by welded plates to each other. There are two supports at the top and bottom of the connection. Using the MPC constraint, the top and bottom of the column are connected to a reference point.This point is represented as the restrained plane. The displacement and rotation of these reference points are then constrained. The near areas of connection have meshed finer. Since there is considerable stress concentration in these areas, the elements have been chosen smaller. Larger elements have been selected in the column as there is no expectation of large deformations.

5. DISCUSSION OF RESULTS

After applying the cyclic loading to all the specimens in Table 5, the M-θ hysteresis curves have been presented



4 Bolted stiffened extended end-plate connection 4 Bolted unstiffened extended end-plate connection

Figure 7. Geometries of the bolted extended end-plate moment connections

TABLE 4. Connection Parameters

Connection type	Beam				Column				Bolt arrangement						End-plate stiffener	
	d	bbf	tfb	twb	dc	bcf	tfc	twc	pfi	pfo	pSi	pso	c	g	Lst	hst
Unstiffened connection	400	180	18.5	8.6	360	300	22.5	12.5	50	50	50	50	113.5	130	-	-
Stiffened connection	400	180	18.5	8.6	360	300	22.5	12.5	50	50	50	50	113.5	130	175	100

TABLE 5. Specimens with various levels of bolt pre-tension

Type of connection	Specimen No.	Specimen name	Type of bolt	$(\boldsymbol{\alpha} * \mathbf{0.55}\boldsymbol{F_u})$[1]
	1	EP-00	A490	0
	2	EP-06	A490	0.6
BUEEP[2]	3	EP-10	A490	1.0
	4	EP-16	A490	1.6
	5	EP-18	A490	1.8
	6	SEP-00	A490	0
	7	SEP-06	A490	0.6
BSEEP[3]	8	SEP-10	A490	1.0
	9	SEP -16	A490	1.6
	10	SEP -18	A490	1.8

[1] $0.55F_u$ is the bolt pretension level based on Iranian Steel Code

[2] Bolted Un-Stiffened Extended End-Plate

[3] Bolted Stiffened Extended End-Plate

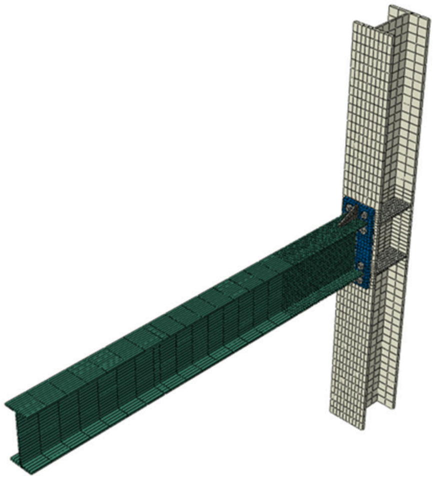


Figure 8. Connection mesh in finite element program

for all specimens related to the 4 Bolted unstiffened extended end-plate moment connection in Figure 9. Figure 10 also shows the hysteresis curves for the 4 bolted stiffened extended end-plate moment connections.

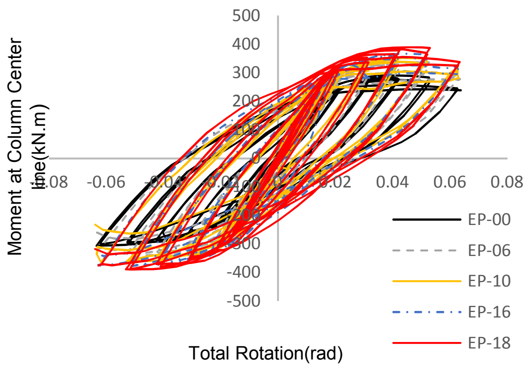


Figure 9. Hysteresis moment curve for different levels of α in BUEEP connection

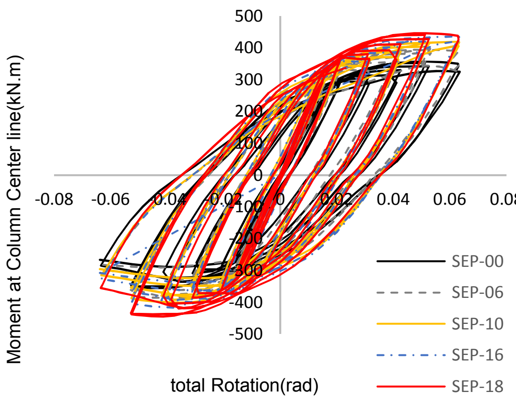


Figure 10. Hysteresis moment curve for different levels of α in BSEEP connection

As displayed in Figures 9 and 10, in the specimens EP-00 and SEP-00, α is 0 and represents the state without pre-tension, called a snug-tightened bolt. The hysteresis moment curve of the snug-tightened specimen has less moment resistance than the specimens with pre-tensioned bolts. The hysteresis curves are stable and have good energy absorption as well as dissipation. An important observation in the hysteresis curves of the snug-tightened specimens is that after the maximum resistance a softening behavior is observed in the specimen. This effect may be due to the local buckling of the beam flange under compression, which will be shown in this research. It can be obtained that the Lack of prying and contact force between bolts, end plate and column flange in snug-tightened specimens leads to more flexibility of the joint. Compared to snug-tightened, in pre-tensioned specimens, the prying and contact force increases due to pre-tensioning which leads to improve maximum moment resistance and more rigidity of the joint. The maximum resistance values of each specimen have been provided in Table 6.

It can be seen in both types of connections, increasing the bolt pre-tension would augment the resistance moment of the connection. It seems that in the unstiffened connection, increasing the bolt pre-tension has had a greater effect on enhancing the resistance. The amount of growth in resistance in the pre-tensioned bolt and for the case where the value of α = 1.8 is about 27% for the unstiffened connection and about 25% for the stiffened connection. Also, the increase in resistance in the unstiffened connection in the pre-tensioned bolt specimens, where α = 1.0, is about 19% and for the stiffened connection is about 17%. Figure 11 indicates the values in Table 6 for the unstiffened connection and Figure 12 for the stiffened connection.

TABLE 6. Maximum moment resistance

Group	Specimen	α	M_{max}(kN.m)	$\frac{M_{max}-M_{ref}}{M_{ref}}$ (%)
BUEEP	EP-00(Ref)	0	306.75	-
	EP-06	0.6	342.04	11.50
	EP-10	1.0	364.63	18.87
	EP-16	1.6	380.80	24.14
	EP-18	1.8	389.65	27.02
BSEEP	SEP-00(Ref)	0	356.89	-
	SEP -P-06	0.6	397.14	11.28
	SEP -P-10	1.0	419.03	17.41
	SEP -P-16	1.6	435.72	22.09
	SEP -P-18	1.8	445.99	24.97

As outlined in Figures 11 and 12, the ascending trend of flexural strength is almost linear. It can also be seen that the difference between the moment resistance for the specimens with α = 1.8 and α =1.6 in both BUEEP and BSEEP connections is less than for other specimens. Accordingly, increasing the bolt pre-tension beyond the value mentioned in the national code of steel structures will lead to increase in a bit amount of moment resistance.

Figures 13 and 14 reveal the envelope curve resulting from the hysteresis curve. Based on the envelope curves, it is observed that the initial stiffness of the snug-tightened specimens is less than the stiffness of the pre-tensioned specimens. This can be due to the enhanced stiffness of the connection and low rotation of the endplate than the column. Also, the resistance of pre-tensioned specimens is higher than that of the snug-tightened. In the specimens without stiffeners after a maximum resistance, softening occurred, which can be due to the local buckling of the flange and plastic strains distribution in the beam flange.

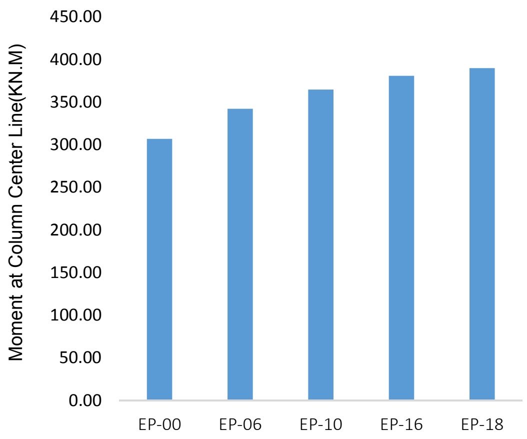


Figure 11. Resisting moment values– BUEEP

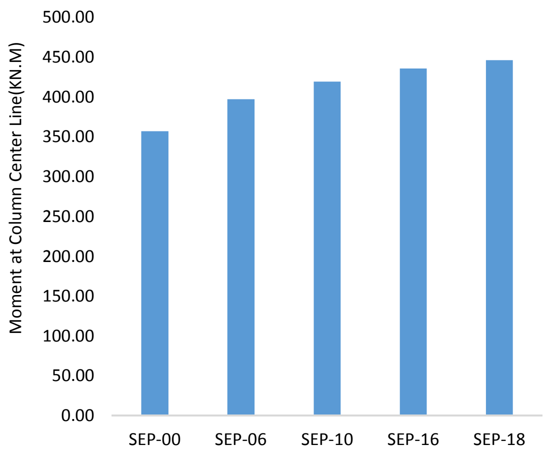


Figure 12. Resisting moment values - BSEEP

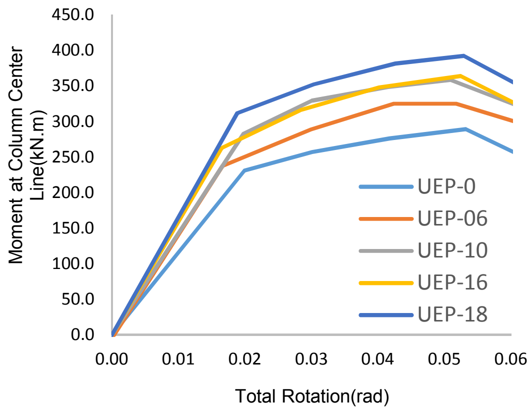


Figure 13. Moment versus total rotation hysteresis envelope of the specimens-BUEEP

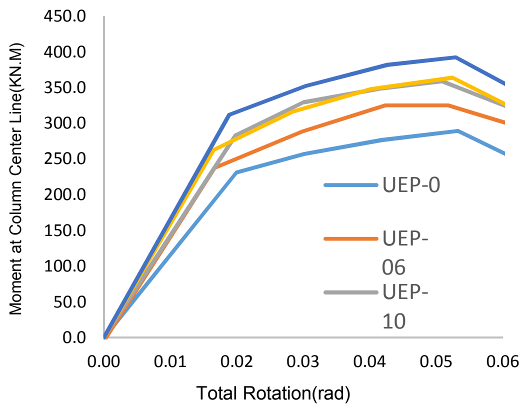


Figure 14. Moment versus total rotation hysteresis envelope of the specimens-BSEEP

Figures 15 to 17 demonstrate the yielded area for BUEEP connections. As can be seen, local buckling is evident in the flange at the end of the cyclic loading. The plastic hinge has occurred near the connection and there is no significant stress in the column. The local buckling of the beam flange, as well as the formation of the plastic hinge near the column, indicate that the cyclic behavior of the column is suitable for seismic loads. In both types of connections, the stress distribution is almost the same formation, but different plastic strains and yield areas can be observed. Figures 18 to 20 reveal the yield areas in the stiffened connection. As can be seen, for the snug-tightened connection, the yielding is more severe as compared to pre-tension. Also, in the pre-tensioned specimen with α = 1.0, a significant amount of beam flange and web near the connection has been yielded. In the pre-tensioned connection with α = 1.8, yield distribution in the beam flange and web decreased but was more severe. As a result, it can be expected that the dissipated energy would also be larger in these connections. In the stiffened connections, the plastic hinge has been formed in the near part of the end-plate stiffener.

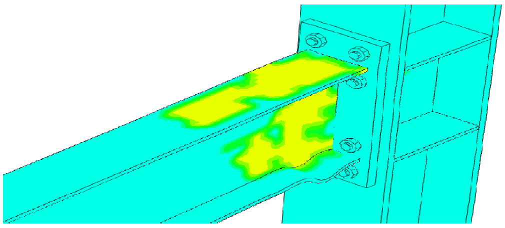


Figure 15. Yielding distribution of material - EP-00

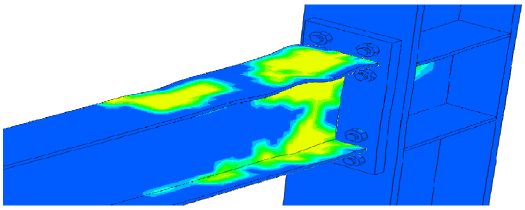


Figure 16. Yielding distribution of material - EP-10

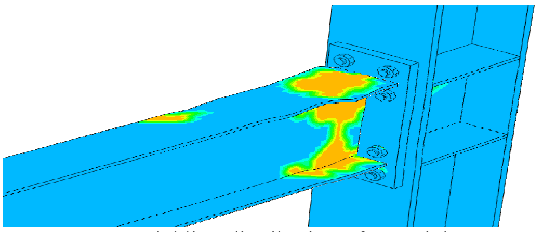


Figure 17. Yielding distribution of material - EP-10

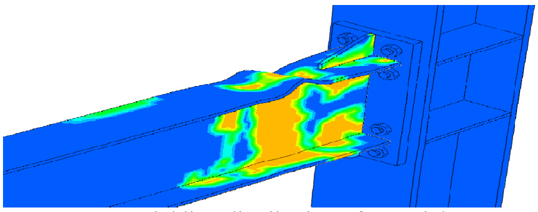


Figure 18. Yielding distribution of material - EP-00

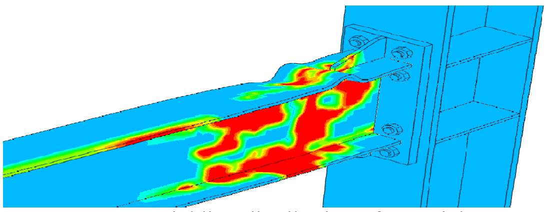


Figure 19. Yielding distribution of material - EP-10

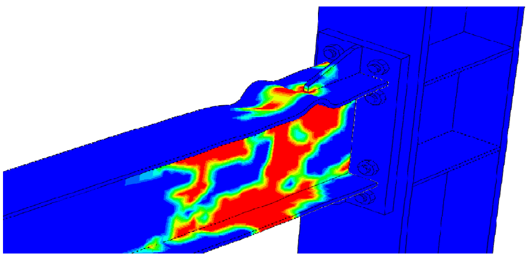


Figure 20. Yielding distribution of material - SEP-18

Major local deformations and buckling in the beam flange occurred after the end-plate stiffener. Also, the yielding areas in the specimen with pre-tension are larger than those of snug-tightened specimen.

Figure 21 shows the energy dissipation curve for the unstiffened specimens and Figure 22 indicates the energy dissipation curves for the stiffened connections. As can be seen in the connections with pre-tension, the energy dissipation has increased. This may be due to the enhanced connection resisting moment and more plastic distribution of deformation, leading to more ductility. Accordingly, more areas are yielded in these connections. Table 7 reports the maximum energy dissipated in different specimens as well as the increase in energy dissipated compared to the snug-tightened.

In Table 7, the energy dissipation of bolted connections with snug-tightened bolts called reference specimen, has been mentioned as E_{ref}. According to Table 7, it can be seen that the energy dissipation has grown with enhancement of α in both types of connections. This is due to more yielding deformation of materials in the specimens with enhanced alpha, resulting in greater ductility and energy dissipation. On the other hand, it is indicated that energy dissipation of pre-tensioned connections to E_{ref} for the unstiffened connection with $\alpha = 0.6$ is about 18%, for $\alpha = 1.0$ about 24%, for $\alpha = 1.6$ about 29%, and for $\alpha = 1.8$ about 31%.

It is also about 13% for the stiffened connection with $\alpha = 0.6$, about 16% for $\alpha = 1.0$, about 21% for $\alpha = 1.6$,

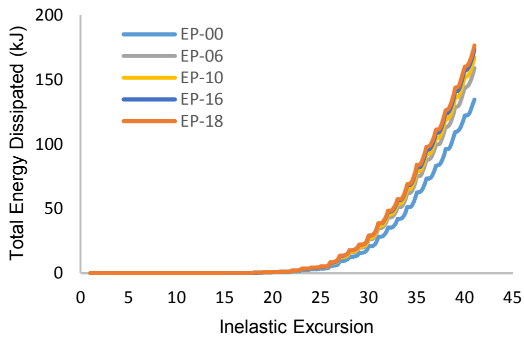


Figure 21. Total energy dissipated –BUEEP connections

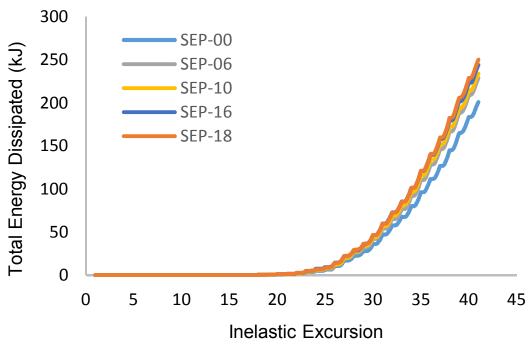


Figure 22. Total energy dissipated –BSEEP connections

and about 24% for α = 1.8. it can be found that enhancement of bolt pre-tension beyond that of proposed by the national design code has not significantly affected the energy dissipation enhancement. This suggests that if the bolt pre-tension is provided in accordance with the design code, the expected energy dissipation is fulfilled.

Energy dissipation for different levels of bolt pre-tension (α) at both end-plate connections with and without stiffeners have been exhibited in Figures 23 and 24.

Bolt stress variation during cyclic loading is a key parameter to be investigated. Figure 25 indicates the bolt axial stress variations with load steps for the BUEEP connections. It can be found that for snug-tightened specimens, the axial stress bolt is initially zero and then grows with load step enhancement during the loading up to cycle 17, where the stress after unloading reaches zero. However, after this cycle, some residual stress will occur. Regarding α = 1.0, the bolt stress, called pre-tension stress, starts from about 550 MPa and diminishes slightly with an increment of load steps. At the end of the load cycle, the stress dropped to about 175 MPa. Concerning α = 1.8, the bolt pre-tension stress is about 925 MPa and fall with increase of the load cycle. For the last state, the bolt stress would decrease to 411 MPa. It seems that reduction of bolt stress is due to reciprocating of cyclic loading, especially in the higher cycles, as well as the bolts loosening.

TABLE 7. Total Energy Dissipation

Group	Specimen	α	E (kJ)	$\frac{E-E_{ref}}{E_{ref}}$ (%)
BUEEP	EP-00(Ref)	0	134.62	-
	EP-06	0.6	159.09	18.18
	EP-10	1.0	167.47	24.41
	EP-16	1.6	173.61	28.97
	EP-18	1.8	176.70	31.26
BSEEP	SEP-P-00(Ref)	0	201.09	-
	SEP -P-06	0.6	228.68	13.72
	SEP -P-10	1.0	234.06	16.40
	SEP -P-16	1.6	244.03	21.35
	SEP -P-18	1.8	250.20	24.42

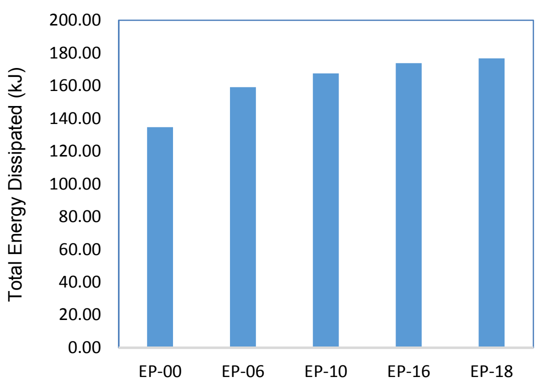


Figure 23. Maximum energy dissipation - BUEEP

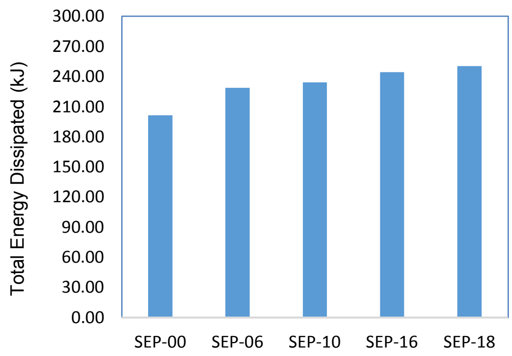


Figure 24. Maximum energy dissipation - BSEEP

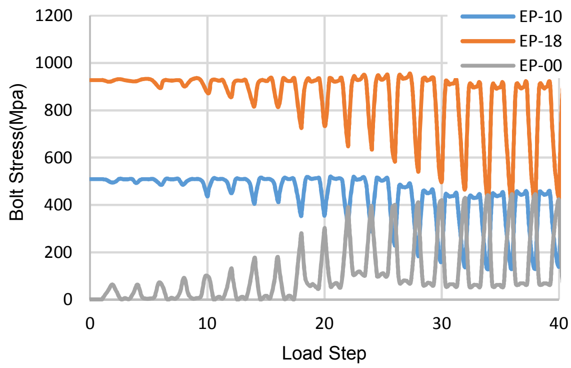


Figure 25. The stress created inside the bolts due to cyclic loading

The end-plate gap and deformation, which indicates the distance of the endplate and side of the column in which will be provided during cyclic loading, is also studied. Figure 26 illustrates the path of the endplate where the gap was studied along. Figure 27 shows the gap and deformation along the endplate of BUEEP connections and Figure 28 reveals it for BSEEP connections.

In the BUEEP connection, when the upper bolts are subjected to tension, the end-plate gap profile is almost similar for all cases including snug-tightened and pre-tensioned bolted connections, though the amount of end-plate gap will change along. The end-plate deformation will result in the maximum amount of gap for snug-tightened (EP-00) specimen by about 6 mm, while the minimum amount is about 4.5 mm for in EP-18. The end-plate gap distribution is slightly different when the bottom bolts are subjected to tension. In this case, more length of the endplate will be affected and deformed. This may be due to the local buckling of the beam flange under compression, which affects the deformation of the endplate. As displayed in Figure 28, the end-plate gap profile in the BSEEP connection is almost linear along with the plate. This indicates that there is not much deformation in the endplate. This can be due to the end-plate stiffener causing more rigid endplate in BSEEP connections as compared to BUEEP connections. As shown in Figure 28, the amount of end-plate gap in the snug-tightened specimen is significantly greater than that in pre-tension specimens. Comparing Figures 27 and 28, it can be found that in BUEEP connections and for the EP-10 and EP-18 specimens, the end-plate gap is about 9% and 20% less than that for EP-00 specimens,

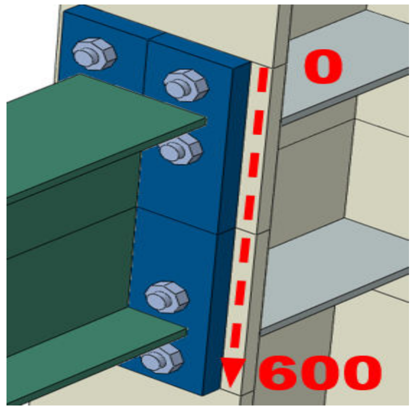
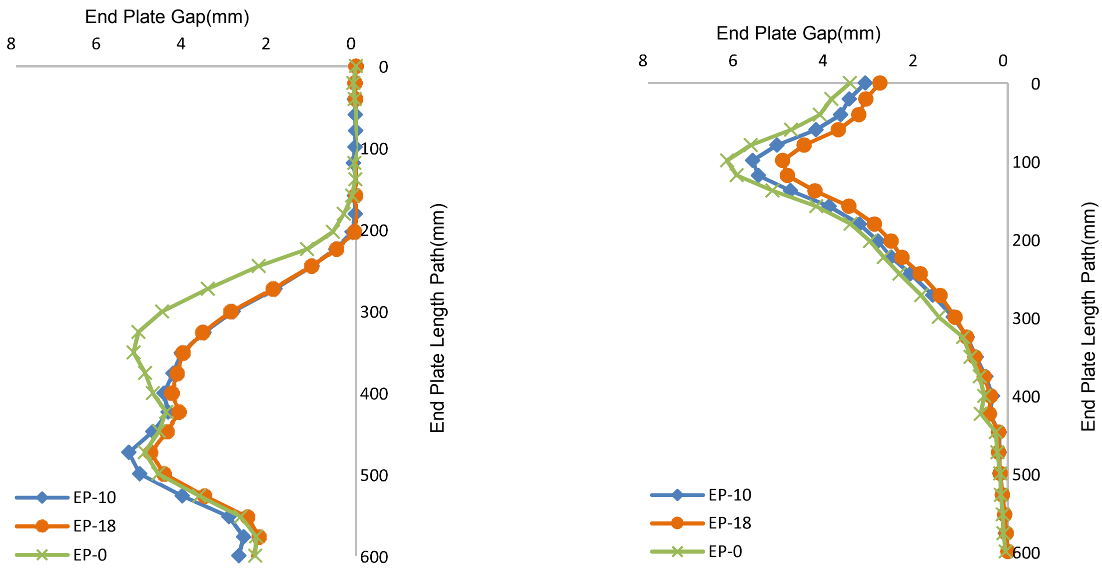
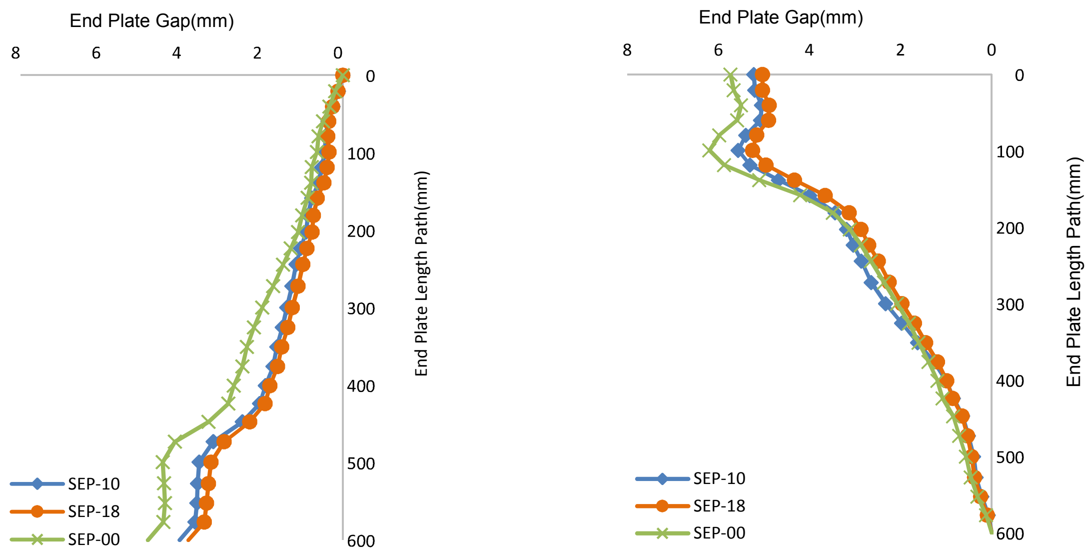


Figure 26. The path along the endplate, the gap studied



Bottom bolts under maximum tension at the end of cyclic load Upper bolts under maximum tension at the end of cyclic load
Figure 27. End-plate deformation of BUEEP



Bottom bolts under maximum tension at the end of cyclic load Upper bolts under maximum tension at the end of cyclic load
Figure 28. End-plate deformation of BSEEP

respectively. However, in BSEEP connections and for the SEP-10 and SEP-18 specimens, the end-plate gap has been reduced by 10 and 15%, respectively, compared with the SEP-00 specimen. Generally, the end-plate deformation and gap study in BUEEP and BSEEP connections suggest that in the snug-tightened specimen, the prying action has had a greater effect, leading to a larger amount of end-plate gap than in the pre-tensioned specimens.

6. CONCLUSION

This study dealt with sensitivity analysis of bolt pre-tension effect on bolted extended end-plate moment connection under cyclic loading for BUEEP and BSEEP connections. α was considered as a coefficient of bolt pre-tension to the bolt pre-tension mentioned in the national Design Code of Steel Structures. The hysteresis, total dissipated energy, maximum moment resistance and stress as well as strain distribution at the connections were studied. The results could be summarized as follows:

a. In BUEEP and BSEEP connections, as the bolt pre-tension increased, so did the connection moment resistance. Increased moment resistance in BUEEP connections including EP-18 and EP-10 specimens, compared with EP-00 specimen, was about 27% and 19%, respectively. However, in BSEEP connection including SEP-18 and SEP-10 specimens, the increase of moment resistance compared with SEP-00 specimen was about 25% and 17%, respectively. Thus, it can be deduced that by increasing the bolt pre-tension by 80% beyond the national Design Code of Steel Structures, the moment resistance of the BUEEP and BSEEP connections would only be increased by about 8%. This suggests that if the bolt pre-tension is provided according to the design codes, the cyclic performance of the connection would be sufficiently improved.

b. The increase in dissipated energy for unstiffened connections, including EP-06, EP-10, EP-16, and EP-18 was about 18%, 24%, 29% and 31% respectively. For stiffened connections including SEP-06, SEP-10, SEP-16, and SEP-18, the increased dissipated energy was also about 13%, 16%, 21%, and 24%, respectively. Thus, the bolt pre-tension following the design code, which is equivalent to $\alpha = 1.0$, was associated with an increase in dissipated energy by about 24% compared with the specimens, with α more than 1, which will not significantly increase the dissipated energy.

c. The strain and yielding distribution of connections indicated that inter-story drift was about 6% under cyclic loading and plastic deformations occurred on the beam flange. On the other hand, the column and panel zone would be elastic.

d. In hysteresis moment of specimens with snug-tightened bolt, softening occurred after the maximum resistance, which is due to inelastic deformation and buckling of the beam flange. However, this effect was less pronounced in pre-tensioned bolt connections.

e. In pre-tensioned bolted connections, the bolt pre-tension diminished with increasing load cycles, while the bolt load increased in snug-tightened bolted connections. This may be due to the effect of heightening the cyclic load on the loosening of the bolts.

f. The end-plate deformation and gap were studied. It was observed that in the BSEEP connections, the end-plate stiffener led to linear growth of the gap along with the plate, where the amount of deformation would be insignificant. However, due to the lower stiffness of the endplate in BUEEP connections, a significant amount of gap and deformation would occur along with the plate compared with BSEEP connections. On the other hand, in pre-tensioned bolted connections, an increase of bolt pre-tension would lead to reduced end-plate gap compared with the snug-tightened bolted connections and performance of connection was improved.

g. Studies of various mentioned parameters indicated the bolt pre-tension in accordance with design code (α=1.0) will lead to a desirable performance of bolted connections under cyclic loading. In other words, the local study of bolt pre-tension effect shows the importance of pre-tension level in seismic behavior of steel moment resistance frames with bolted extended end plate moment connections, which must be pre-tensioned more than minimum mentioned in regulations.

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Persian Abstract

چکیده

در سازه‌های فولادی مهم‌ترین قسمت سازه‌ای اتصالات قاب ساختمانی می‌باشد. آزمایشات انجام شده توسط موسسه فولاد آمریکا منجر به ارائه اتصالات پیش پذیرفته شده است که مرجع مناسبی برای طراحی واحد اتصالات در سازه‌های فولادی می‌باشد. در این مقاله، اتصالات با ورق انتهایی ٤ پیچی با و بدون سخت‌کننده به صورت تحلیل عددی با نرم‌افزار آباکوس مورد مطالعه قرار گرفته‌اند. در این تحقیق، به منظور بررسی میزان تاثیر پیش‌تنیدگی که به ندرت بررسی گردیده، ضریبی از نیروی پیش‌تنیدگی معرفی شده توسط آئین‌نامه طراحی سازه‌های فولادی ایران و همچنین آئین‌نامه طراحی فولاد آمریکا، در نظر گرفته شده است. این اتصالات به صورت چرخه‌ای و جابجایی کنترل مدل‌سازی شده‌اند. رفتار چرخه‌ای اتصالات، انرژی مستهلک شده، لنگر مقاوم و همچنین توزیع تنش و کرنش در اتصال مورد بررسی قرار می‌گیرد. بر اساس نتایج، با ایجاد نیروی پیش‌تنیدگی در پیچ‌ها، مقدار لنگر مقاوم اتصال افزایش می‌یابد. میزان افزایش این مقاومت در اتصالات بدون سخت‌کننده بیش‌تر از اتصالات با سخت‌کننده می‌باشد. مقدار افزایش مقاومت حداکثر برای اتصال بدون سخت‌کننده حدود ۲۷ درصد و برای اتصال با سخت‌کننده حدود ۲۵ درصد است. همچنین انرژی مستلهک شده برای اتصالات با افزایش نیروی پیش‌تنیدگی افزایش می‌یابد. مقدار افزایش انرژی مستهلک شده برای اتصال بدون سخت‌کننده حداکثر حدود ۳۱ درصد و برای اتصال با سخت‌کننده حداکثر تا حدود ٢٤ درصد افزایش می‌یابد.