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Research Article

Cyclic Behavior of Bolted Stiffened End-Plate Moment Connections for Different Bolt Pretensioning Levels: An Experimental Study

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Bolted stiffened end-plate moment connections are one of the most usable connections used as a prequalified connection in special steel moment frames. According to the AISC design code, this connection can be considered one of the most important parts of moment resisting frames with enough bolt pretension levels. In this paper, using three full-scale bolted stiffened end-plate moment connections designed according to AISC, the effects of bolt pretension levels have been examined experimentally under SAC cyclic-loading protocol. Bolt pretension level has been defined as α coefficient to show pretensioning level in three specimens. The bolts of the first specimen are not pretensioned, called snug-tightened bolts, and are reference connections. The bolt pretension level of the second and third specimen is created in accordance with the AISC design code and more to Fu of bolts, called pretensioned and fully pretensioned, respectively. The bolt's moment capacity, total energy absorption, initial rotational stiffness, and ductility of connection as well as stress and strain variation have been investigated. According to the results, increase in bolt pretension level would significantly improve the cyclic behavior of connections. Furthermore, an increase in bolt pretension led to the initiation of the inelastic deformation from a smaller rotation, and the ductility of the connection improved. In addition, the rate of growth in moment capacity and energy absorption in the pretensioned specimen was 8% and 9% compared with fully pretensioned, respectively. Note that the connection with bolt pretension level by design regulations in comparison with the reference connection can be considered as a connection of a special moment resisting frames where bolt pretension level higher than the value mentioned in the design code is better but not needed.

1. Introduction

In recent years, the use of special steel moment frame systems with prequalified bolted connections in high-rise steel buildings has expanded significantly. Due to their high ductility, bolted extended end-plate moment connections with and without stiffeners and bolted flange connections are the most often used connections; they are approved as prequalified connections considering the requirements specified in regulation AISC-358 [1].

The bolted extended end-plate moment connections, which can be used with cruciform, box, and I-shaped column sections, are one of the earliest beam-to-column moment connections. The behavior of bolted extended end-plate moment connections has been studied by a number of researchers.

Murray [2] proposed the concept of four-bolted stiffened and unstiffened extended end-plates, as well as eight-bolted stiffened extended end-plate moment connections. Astaneh-Asl [3] investigated two 4-bolted unstiffened and stiffened extended end-plate specimens that were cyclically tested. The ductility of the connection, which finally led to the buckling of the beam flange, was studied in the first specimen. He utilized I-shaped shim plates between the end plate and the column's flange in the second example. According to the findings, using shim plate could improve the connection's performance.

Jaspart and Maquoi [4] investigated the effects of bolt pretensioning on bolted connections. They provided equations to estimate the stiffness and strength of the connection in their research. Bahaari and Sherbourne [5] proposed an analytical model for the moment-rotation relationship of extended end-plate connections. The parameters in terms of the connection description were derived using a multipleregression analysis approach.

An experimental investigation by Faella et al. [6] measured the impact of bolt pretension on the behavior of bolted connections with T-joint under axial loading. Adey et al. tested 15 connection specimens under cyclic loading [7], evaluating the parameters of beam size, bolt arrangement, end-plate thickness, and end-plate stiffeners in the potential energy dissipation of the bolted end-plate connection. They discovered that when beam diameters increased, the energy dissipated capacity decreased; however, end-plate stiffeners increased energy dissipation. Sumner [8] evaluated the behavior of four-bolted unstiffened extended end-plate connections and eight-bolted stiffened extended end-plate connections in an experimental study. They discovered that in steel moment frames, the four-bolted unstiffened extended end-plate connections and the eight-bolted stiffened extended end-plate connections could be used for seismic resistance. Diaz et al. [9] reviewed the modeling of joint behavior in steel frames. Experimental testing, empirical models, analytical models, mechanical models, and numerical models were reviewed and compared in order to model the rotational behavior of joints. Experimental and numerical models were shown to be more accurate when compared with other methods.

According to Gerami et al. [10], the cyclic behavior of bolted connections, such as bolted extended end-plate moment connections and T-joints, is affected by the horizontal and vertical arrangement of connection bolts. They discovered that if the bolts were arranged differently, the risk of a T-joint rupture increased as compared to the end-plate connection. As a result, they suggested using bolted extended end-plate moment connections in situations where the probability of a constructional defect in the execution of structures increases. Abdollahzadeh et al. [11] evaluated the behavior characteristics of I beam-to-concrete filled tube(CFT) column connection using bolted end-plate moment connection. They showed that increase in end-plate thickness has led to increase in moment capacity and stiffness of connection. Liao et al. [12] provided an analytical model for transient excitation of a simple bolted joint beam in the tangential direction. The findings showed that the nonlinear effects shown in suggested transient stimulation experiment matched the numerical simulation results. Morrison et al. [13] investigated the effect of removing the end-plate stiffener and various bolt configurations on eight-bolted stiffened extended end-plate connections. They concluded that removing the end-plate stiffener would be economically viable, despite requiring higher thickness for the end plate. Guo et al. [14] investigated the influence of the strength of high-strength bolt materials and different bolt configurations on the load-bearing capacity and the deformation of the bolted connection with the cover plate using a static loading test in 2018. Elsabbagh et al. [15] analyzed the behavior of a bolted unstiffened extended end-plate moment connection under cyclic and monotonic loading by studying parameters such as shear force, bolt diameter, end-plate thickness, and the use of an end-plate stiffener. They found that the shear force had a significant effect on the stiffness of the connection. Lyu et al. [16] investigated the effect of bolt pretension on the load-bearing capacity of bolted connections with cover plate in an experimental and numerical study. They concluded that bolt pretension had no effect on the ultimate tear-out failure mode. On the other hand, out-of-plane confinement limit the piling-up of plate material in front of the bolt, reducing the related bearing resistance. A finite element model for preload bolt flange connection structures was presented by Sun et al. [17]. They showed that evaluating a preload bolted flange connection structure under impact load proved the efficacy of the suggested method, which can quickly and nondestructively determine loosening using acceleration signals. Noferesti and Gerami [18] investigated the effect of different bolt pretension levels in prequalified bolted end-plate moment connections under cyclic loading by a numerical study. They showed that dissipated energy increases with increase in the bolt pretension levels.

High-rise building construction has expanded in recent years all over the world. Studies also suggest that bolted extended end-plate moment connections could be implemented as part of the moment frame's lateral seismic resistance system in high-rise steel constructions. On the other hand, implementing such structures demands considerable time, due to administrative problems and financial issues. Thus, due to the relatively long construction time, it is important to consider the loads during construction and control structure's stability for different stages of construction.

All connection bolts in a complete structure are pretensioned in prequalified moment connections, ensuring that the structure has appropriate seismic performance. During the building of steel structures, however, the executive groups usually fabricate some stories of the structure and tighten the connection bolts, which is called the snug-tightened bolt in this research. The lower stories, in which the connection bolts are snug-tightened, will be pretensioned to the level of preloading based on design codes, which is called the pretensioned bolt in this paper; so, the connections will be completed, while some of the upper stories will have snug-tightened bolts. This process will be repeated 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 bolted extended end-plate moment connections in both snug-tightened and pretensioned bolts 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.

The effect of bolt pretension levels on the behavior of bolted stiffened extended end-plate moment connections has been studied in a few research studies. This paper investigated the effect of different levels of bolt pretension on the behavior of bolted stiffened extended end-plate beam-to-column connections, taking into account the long construction time of high-rise steel structures and the fact that their stability is dependent on the effect of bolt pretension on the behavior of bolted end-plate moment connections.

The behavior of bolted extended end-plate moment connections could be evaluated using theoretical, computational, and experimental investigations. Experimental research is much more accurate than the other approaches suggested. This research investigates the effects of pretension level on the behavior parameters of bolted stiffened end-plate beam-to-column connections. During construction, the gravity load is low, but on the other hand, the system of incomplete and bearing capacity of structures is still less. In this case, few loads are important for these incomplete structures. As a result, the researchers of this article decided to investigate different amounts of pretension applied to the bolt at different stages of the construction to clearly determine the effects of this parameter on the capacity of tall structures.

2. Test Specimen

In this research, three laboratory specimens including 3 BSEP (bolted stiffened extended end-plate) connections have been tested and designed based on AISC [1, 19]. Based on the infrastructure provided in FEMA350 [20], the beam-to-column connection in all samples indicates the corner connection. The cross section of the beam in all samples is IPB160 with a length of 1.45 meters. The cross section of the column has been made using the plates whose dimensions are shown in Figure 1, where the length of the column is 2 meters. The connection of the beam to the end-plate in the connection of BSEP to the column has been carried out based on AISC-358 [1] and Design Guide 4 [19] using CJP (complete joint penetration) groove welds and Co2 welding technique. All CJP welds have been tested using the ultrasonic method to ensure the health of the weld. Welding details and welding tests are presented in Figure 1.

In order to determine the inelastic behavior in the beam flange, a white plastic paint layer that could fall in large strains has been used. Meanwhile, in all specimens, the beam and the column have been meshed approximately 50 cm and 60 cm long, respectively, with the dimensions being 2 cm for each rectangle grid. Figure 2 displays how to mesh the beam and column using a marker.

Different levels of pretension bolt loads have been determined based on α times. 70% of the minimum tensile strength of bolts based on AISC360 [21] and RCSC [22] ($\alpha \cdot 0.7F_u A_e$ where Fu and Ae represent the ultimate material stress and effective bolt cross section, respectively) are reported in Table 1 for different specimens. In order to study the effect of bolt pretension levels on the inelastic behavior of BSEP connections, three different levels of pretensioning have been considered. As can be seen in Table 1, the specimen test name is also based on the α coefficient. In the specimen of SEP-0.00(R), the amount of bolt pretensioning is zero, so-calledsnug-tightened, and is a reference specimen.

In SEP-1.00, the minimum levels of bolt pretension have been provided based on AISC360 [21] and RCSC [22], so-called pretensioned. In SEP-1.43, the minimum tensile strength of bolts has been provided, called fully pretensioned.

Table 2 outlines actual material properties used in different components of the joint, including bolts, beams, columns, and end plates, as obtained from the tensile test.

3. Test Setup

The complete test setup and equipment are shown in Figure 3. The column is horizontal, and the beam is vertical, as indicated. The setup consists of a reaction frame, a floor beam on which the column is installed, and a strong floor. A one-sided beam-to-column connection is used to simulate the test setup.

Two pinned supports have been used to connect the column to the rigid floor of the laboratory, as indicated in Figure 3. A two-way50-ton hydraulic jack was used to provide cyclic loading based on FEMA350 [20] by the SAC cycle-loading protocol (Table 3). To prevent lateral buckling, out-of-plane displacement of the beam has been constrained using lateral support at the middle of the height of the beam, as shown in Figure 3. The displacement at the end of the beam is measured using the LVDT, installed at the beam tip, which could record two-way displacement. At the end of the column, the LVDT is installed to ensure no transition displacement would occur in the column. To record the strain at different points of connection, including the beam flange near the end plate, beam web, doubler plate in the panel zone, end plate and bolts, and strain gauges have been attached, as displayed in Figure 1.

Meanwhile, strain gauges have been placed at bolt shanks by making a cut in the bolt head to record the strain changes in the interior and exterior bolts during cyclic loading, as shown in Figure 4.

4. Bolt Installation and Pretensioning Procedure

Prior to pretensioning, the snug-tightening procedure is required to bring the plies into firm contact. Bolts have been placed in all holes with washers positioned as required by RCSC [22] and nuts threaded to complete the assembly. The snug-tightened condition is the tightness attained with the full effort of an ironworker using an ordinary spud wrench to bring the plies into firm contact under RCSC [22]. For specimen SEP-0.00, all bolts are in snug-tightened conditions, but for SEP-1.00 and SEP-1.43, all bolts must be tightened first and then be pretensioned and fully pretensioned, respectively, as mentioned in Table 1.

In order to ensure that the bolts are pretensioned as required according to Table 1, two methods, including turn-of-nut pretensioning and calibrated wrench pretensioning,

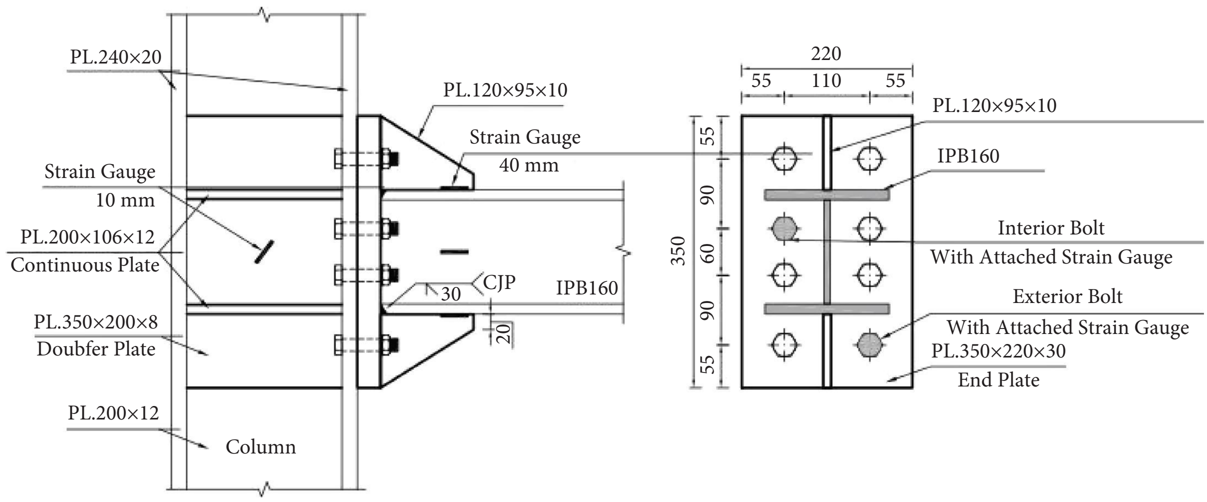


Figure 1: Properties of specimen-fabricating parameters and location of strain gauges.

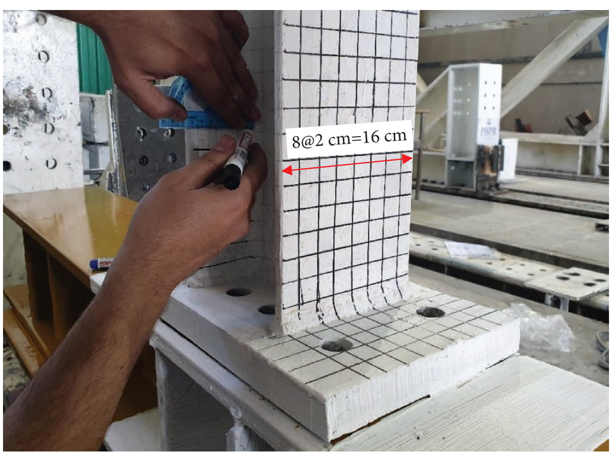


Figure 2: Grid dimensions.

have been used (Figure 5). Match-marking of the nut and protruding end of the bolt after snug tightening can be helpful in the subsequent installation process called pretensioning and is certainly an aid to inspection, as depicted in Figure 5. As shown in this figure, to measure the amount of nut rotation to pretension of bolts, 360-degree calibrated cellophane has been used to show the angle of nut rotation.

5. Test Result

The summary of the test results is presented in Tables 4 and 5. The results are described in two separate parts. In the first part, the cyclic behavior of the specimens is described, and in the second part, the cyclic behavior of the specimens is compared.

5.1. Cyclic Behavior of the Specimens. In this section, the behavior of each specimen has been described under cyclic loading separately. Different parameters have been studied, such as the moment capacity, absorbed energy, initial and secondary stiffness, corresponding rotation of inelastic deformations, failure modes, and strain variation history of the interior and exterior bolts of BSEP beam-to-column connections.

5.1.1. Specimen SEP-0.00-R. The connection has been designed according to the AISC [20] requirements code with the end plate connected to the beam flange via full penetration welding. In this connection, the bolts are snug-tightened, and α is zero.

The plastic deformation by the white plastically paint layer falling and the formation of meshing generated near the connection is detected. The plastic formation has been initiated at 0.028 rad 13 cm away from the connection location on the beam flange's exterior side. Then, the plastic area is extended on the exterior and interior sides of the beam flange after L_{st} (L_{st} is the length of end-plate stiffener) with the plastic hinge formed in a rotation of 0.04 rad.

Then, the rotational stiffness of the connection is reduced by 82%. The strain variation in the beam flange during the loading process is shown in Figure 6. Upon increasing the load amplitude, plastic strain in the beam flange and web at the mentioned region is increased; hence, a little gap opening (relative displacement between the column flange and the end plate referred to as "gap opening" in this paper) is observed. The lack of pretensioning in the bolts leads to an increase in the maximum gap opening by about 3 mms, between the end plate and column at the end of the loading process, which corresponds to the rotation of 0.07 rad. No noticeable plastic area is observed in the connection components and column at the test end (Figure 7). The moment at the column centerline versus the total rotation curve of the specimen is shown in Figure 8. Finally, the moment capacity of the connection reached 1.03 Mp (Mp is the beam plastic moment). Since the plastic area formation is initiated at the rotation of 0.03 rad and the plastic hinge is formed at the rotation of 0.038 rad, the energy absorption curve slope has increased after the rotation of 0.03 rad as shown in Figure 9. At the end of the loading process, allowable failure mode occurs, and using the beam plastic capacity, 67.7 kJ energy was absorbed. Note that, due to the elastic behavior of the

Table 1: Details of the test specimens.

Group	Specimen test name	α parameter	Bolt layout	End-plate thickness (mm)	Weld type	Bolt diameter (mm)
	SEP-0.00-R	0.00	Snug-tightened	30	CJP	24
BSEP	SEP-1.00	1.00	Pretensioned	30	CJP	24
	SEP-1.43	1.43	Fully pretensioned	30	CJP	24

Table 2: Mechanical properties of the used material.

Material	Application	Measured average yield strength (MPa)	Measured average ultimate modulus (MPa)	Measured average elastic modulus (MPa)
ST37	Beam	243	371	195.164
ST37	Column, end plate	238	365	187.183
A490 (Gr.10.9)	Bolts	943	1073	—

connection components, such as the end plate, column, and panel zone, the mentioned components did not contribute to the energy absorption of the specimen.

Since the bolts are snug-tightened, the bolted connection behavior is like a shear-bearing joint. As illustrated in Figure 7, tension and shear interaction during cyclic loading in bolts have led to deformation in the length of bolts.

In this connection, at the start of loading, the strain of bolts is zero, and the strain will increase during the cyclic test. The bolt strain variation history in this specimen is not relevant to pretensioning. During the loading process, the strain started to grow but not more than inelastic strains (see Figure 10).

5.1.2. Specimen SEP-1.00. In this connection, all bolts have been pretensioned by AISC and RCSC [21, 22] design regulations, called $\alpha = 1.0$, as defined in Table 1. Up to rotation 0.01 rad during the loading process, no inelastic deformation and fall was observed in the white plastically paint layer color and deformation in the meshing generated. Plastic deformation occurred after the rotation of 0.018 rad on the exterior sides and extended to beam's interior sides. In rotation of 3.5%, the plastic hinge occurred after the end-plate stiffener and at a distance of 13 cm from the end-plate connection to the column location (Figures 7 and 8). The moment of yielding M_y is 72 kN·m. Finally, the moment capacity of the connection is 1.245 Mp, as shown in Figure 8 (hysteresis moment at column centerline normalized to plastic moment versus the total rotation). The increase in the moment capacity can be due to the bolt pretension effect, leading to enhanced rigidity of the end plate-to-column connection where the prying action will be negligible.

Due to the bolt pretension level by design code regulations, the initial stiffness of the connection has increased significantly. Still, after the initiation of the inelastic deformations, it has diminished by 84%. As displayed in Figure 9, at the end of cyclic loading, the total energy dissipation occurred by 86 kJ. As shown, the slope of the diagram of total energy dissipated has increased after rotation of 0.028 rad, due to initiation of inelastic deformation and enhanced ductility. Note that, during and at the end of the loading process, different components of the connection, such as the end plate, column, and panel zone, were elastic and had a minor effect on the energy absorption of the connection after which the total energy dissipation is mainly due to inelastic deformation occurring in the beam flange.

Furthermore, during cyclic loading, the bolt pretension stress is reduced, and at the end of loading, it completely disappears (Figure 10). The behavior of bolts during the test shows that the exterior bolts have more participation in load-carrying capacity than the interior ones. This can be attributed to the rigidity of the end plate and its elastic behavior during the loading process.

According to the behavior of the specimen discussed above, the connection can be considered as a connection of a special steel moment frame.

As it is known, the intended loop is related to the final loading step, which, unlike other samples, has continued up to 0.08 radians. This issue can be due to the reduction of stiffness in the connection and, as a result, the nonreturn of plastic deformations in the corresponding strain gauge in the beam flange.

5.1.3. Specimen SEP-1.43. In connection with Sep-1.43, the pretension stress was created to Fu level, as mentioned in Table 1, and called fully pretensioned. As presented in Figures 7 and 8, after rotation of 0.016 rad, inelastic deformation and strains initiated a fall in the white plastic color layer and started to grow on both the exterior and interior sides of the beam. The plastic hinge was created after the end-plate stiffener at the rotation of 0.03 rad. In addition, the initial stiffness of the connection increased with the increase in bolt pretension stress and end-plate rigidity (see Figures 8 and 11). Note that, the rotational stiffness after plastic deformation's initiation decreased by 84%. The loading process continued to a rotation of 0.08 rad, and at the end of cyclic loading, a crack was observed in the weld of end-plate stiffener to beam flange, as shown in Figure 7. Then, the behavior of the connection can be accepted up to a rotation of 0.07 rad showing the desired ductility as a beam-to-column connection. As displayed in Figures 8 and 11, the moment capacity has increased

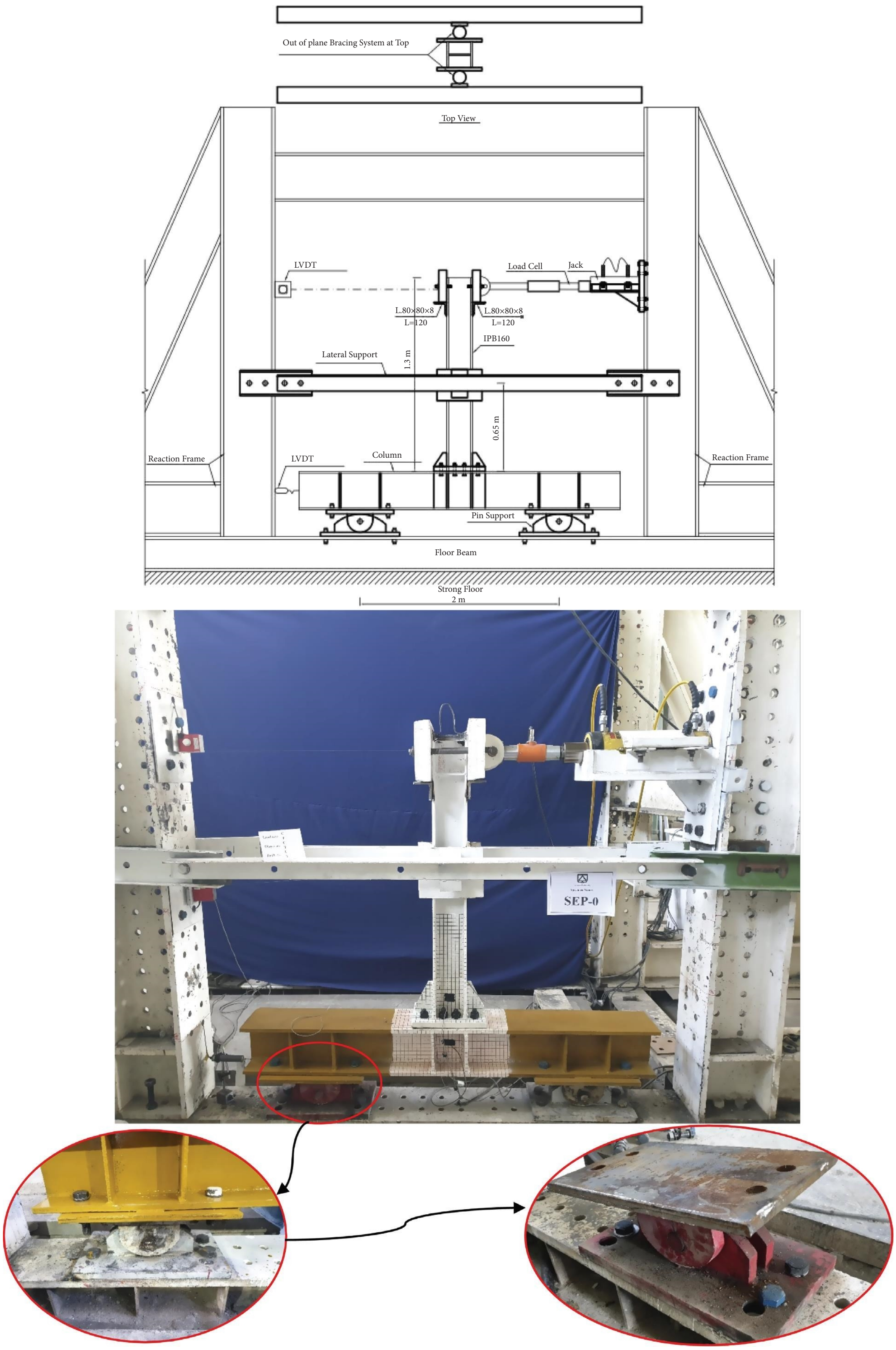


Figure 3: Test setup.

Table 3: FEMA/SAC2000 loading protocol in accordance with FEMA350 [20].

Load step number	Story drift angle (rad)	Number of loading cycles
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2

Continue with increments in drift angle of 0.01 and perform two cycles at each step.

to 1.33 Mp due to full pretension in bolts of connections, an increase in end-plate rigidity, and disappeared prying action in the end plate.

At the end of the loading process, the maximum dissipated energy for $\alpha = 1.43$ is about 92.4 kJ. At a rotation of 0.02 rad, the slope of total energy dissipated versus rotation was increased, with the energy absorption increasing due to inelastic deformation of the sides of the beam and increased connection ductility (Figure 9). Strain variations of fully pretensioned bolts show that at the end of the loading process, the pretension stress of bolts has not completely disappeared. On the other hand, the strain variation pattern shows that the reduction of pretension of interior bolts is greater than the exterior ones. The exterior bolts have a more effective role in the connection behavior (Figure 10).

With respect to the behavior of specimen SEP-1.43, as mentioned above, this connection can be considered as a connection of a special steel moment frame.

5.2. Comparison of Cyclic Behavior of Specimens. As mentioned earlier, one of the most important purposes of this research is to investigate the effect of bolt pretension levels in the behavior of BSEP connections for experimental studies on three bolted stiffened end-plate beam-to-column connections. Various parameters such as moment capacity, energy absorption, plastic deformation, initial stiffness, and strain variation history of bolts were examined. In SEP-0.00-R with snug-tightened bolts, the M-θ hysteresis curve indicated the moment capacity diminished in comparison with other specimens (Figure 8).

An increase in bolt pretension in specimen SEP-1.00 has led to an increase in moment capacity by 21% compared with SEP-0.00-R. Meanwhile, an increase in bolt pretension stress in SEP-1.43 has led to a 29% rise in moment capacity compared with SEP-0.00-R and only 6.8% compared with SEP-1.00 (Figures 8 and 11). Envelope of hysteresis moment at column centerline-total rotation curve (Figure 11) shows that an increase in bolt pretension level significantly increases the moment capacity. Furthermore, it can be seen that although the bolt pretension in SEP-1.43 increased to Fu level, the moment capacity increased by only 6.8% compared with SEP-1.00. A comparison of the M-θ curve and envelope of hysteresis moment-total rotation shows that the SEP-0.00-R specimen can be considered as a semirigid beam-to-column connection.

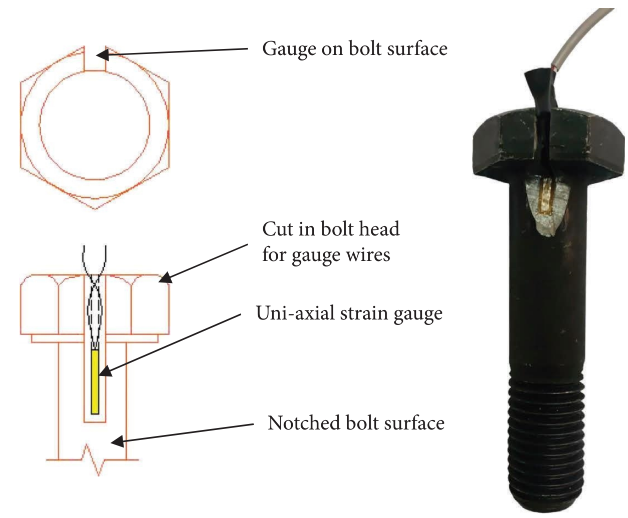
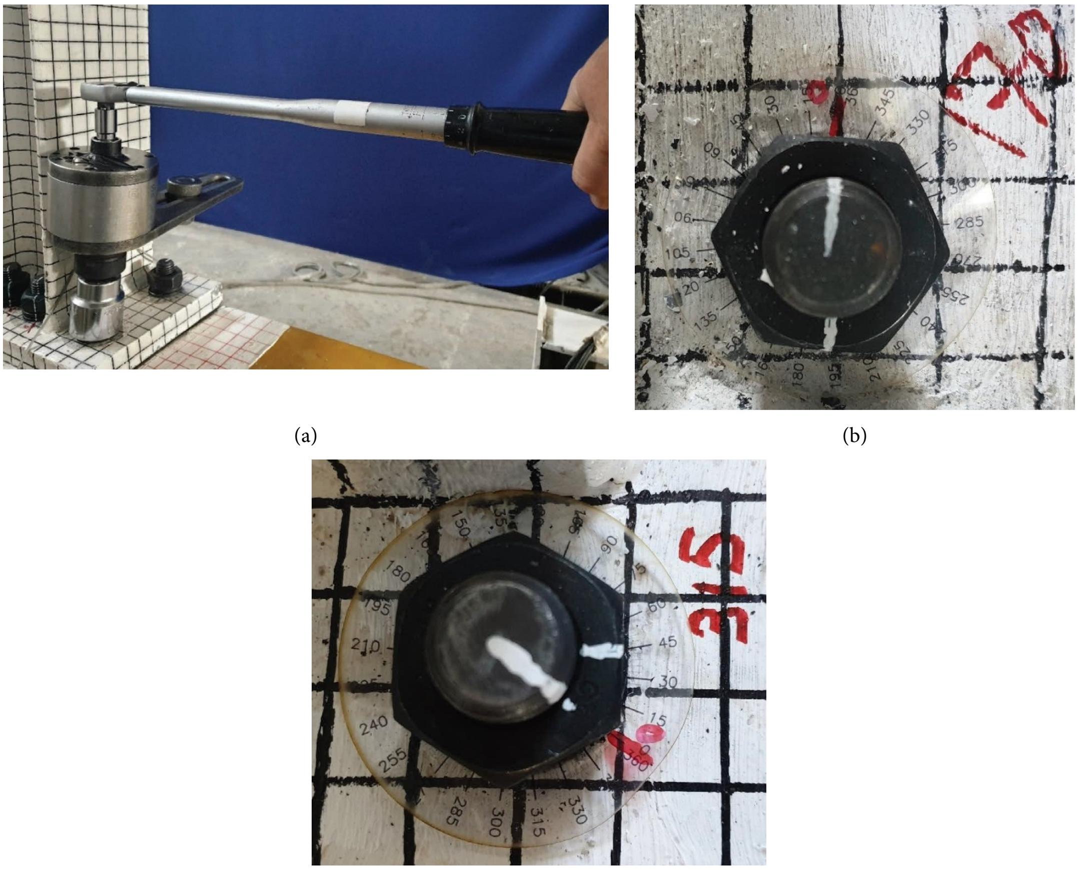


Figure 4: Installation of strain gauges on bolt shanks.

In specimen SEP-1.43, a moment of yielding (M_y) 76 kN·m is occurring at the rotation of 0.016 rad (θ_y). Furthermore, θ_y and M_y in SEP-1.00 are 0.018 rad and 72 kN·m, respectively. On the other hand, the plastic deformation initiated at the rotation of 0.028 rad and M_y is 61 kN·m in SEP-0.00-R. As seen in SEP-0.00-R θ_y and M_y, due to lack of pretension stress in bolts, contact forces between the end plate and column decreased, and the connection stiffness is significantly lower than those of SEP-1.00 and SEP-1.43. So, during cyclic loading in SEP-0.00-R, θ_y increased by 1.55 and 1.75 times θ_y of SEP-1.00 and SEP-1.43, respectively; however, M_y in SEP1.00 and SEP-1.43 increased by 1.18 and 1.25 times of My in SEP-0.00-R, respectively. An increase in the yielding moment of connection by only 5.5% in SEP-1.43 compared with SEP1.00 (Table 4) indicates that enhancement of bolt pretension level beyond that of the value mentioned in design regulations has had little effect on the increase in elastic rotational stiffness of connection (E_{in}). According to the points mentioned above, E_{in} in SEP-1.43 and SEP-1.00 increased by 2.18 and 1.84 times SEP-0.00-R, respectively (Figures 8 and 11). In SEP-0.00-R, the rotation of initiation of plastic deformation is greater than SEP1.00 and SEP-1.43. This shows at the end of the loading process (rotation of 0.07 rad), inelastic deformations have had less contribution to total plastic deformations. Thus, in SEP0.00-R, connection ductility has declined compared to pretensioned and fully pretensioned specimens.

The total dissipated energy in SEP-1.43 and SEP1.00 has increased by 36% and 27%, respectively, compared with SEP-0.00-R, showing an increase in bolt pretension level that has led to more ductility and inelastic deformations. As indicated in Table 4, the total energy dissipated in SEP-1.43 has increased by only 9% more than that of SEP-1.00, suggesting the bolt pretension level by design regulations seems to be enough. Desired energy absorption and moment capacity in pretensioned BSEP beam-to-column connections can be considered a special steel moment frame connection.



(a) (b) (c)

Figure 5: Bolt pretensioning procedure for SEP-1.00 and SEP-1.43. (a) Calibrated wrench. (b) Match-marking turn-of-nut, SEP-1.00. (c) Match-marking turn-of-nut, SEP-1.43.

Table 4: Summary of test results.

Specimens	$M_{\max}$ (kN·m)	$M_{\max}/M_{pb}$ [a]	M_y (kN·m)	θ_y (rad)	$R_{in} = M_y/\theta_y$ (kN·m/rad)	E (kJ) [b]	θ_u (rad)
SEP-0.00-R	87.6	1.03	61	0.028	2178	67.7	0.07
SEP-1.00	105.9	1.245	72	0.018	4000	86	0.07
SEP-1.43	113.2	1.33	76	0.016	4750	92.4	0.07

[a] Plastic moment of beam. [b] Total energy dissipated (kJ).

Table 5: Comparison of test results.

Group	Specimens	$M_{\max}/M_{\max-\mathrm{Ref}}$ [a]	$R_{in}/R_{in-\mathrm{Ref}}$ [b]	E/E_{Ref} [c]	Failure mode
	SEP-0.00-R	1.00	1.00	1.00	Plastic hinge in the beam
BSEP	SEP-1.00	1.21	1.84	1.27	Plastic hinge in the beam
	SEP-1.43	1.29	2.18	1.36	Plastic hinge in the beam

[a] The ratio of the plastic moment of the samples to the plastic moment of the reference sample. [b] The ratio of the initial stiffness of the connection to the initial stiffness of the reference connection. [c] The ratio of the dissipation energy of the sample to the dissipation energy of the reference sample.

As displayed in Figure 10, in SEP-0.00-R, strain in bolts of connection at the beginning of cyclic loading was zero. During the loading process, tension and compression strain occurred in the interior and exterior bolts of SEP-0.00-R. Bolts carry tension loads; however, compression loads are carried using the contribution of bolts and the contact area between the end plate and column. In addition, due to the rigidity and elastic behavior of the end plate, during cyclic loading, prying action and end-plate deformation were not observed, which can lead to increased tension strains compared with compression strains in exterior bolts in comparison with interior ones (Figure 10). In SEP-1.00, at the start of the loading process, the bolts have had pretension stress.

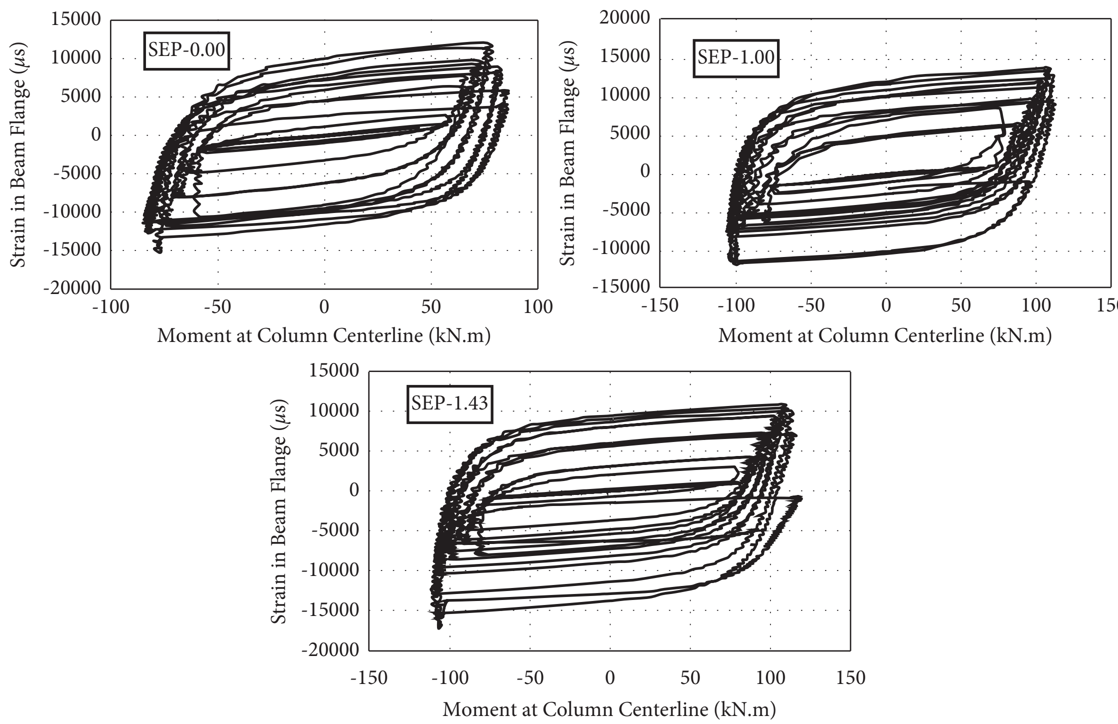
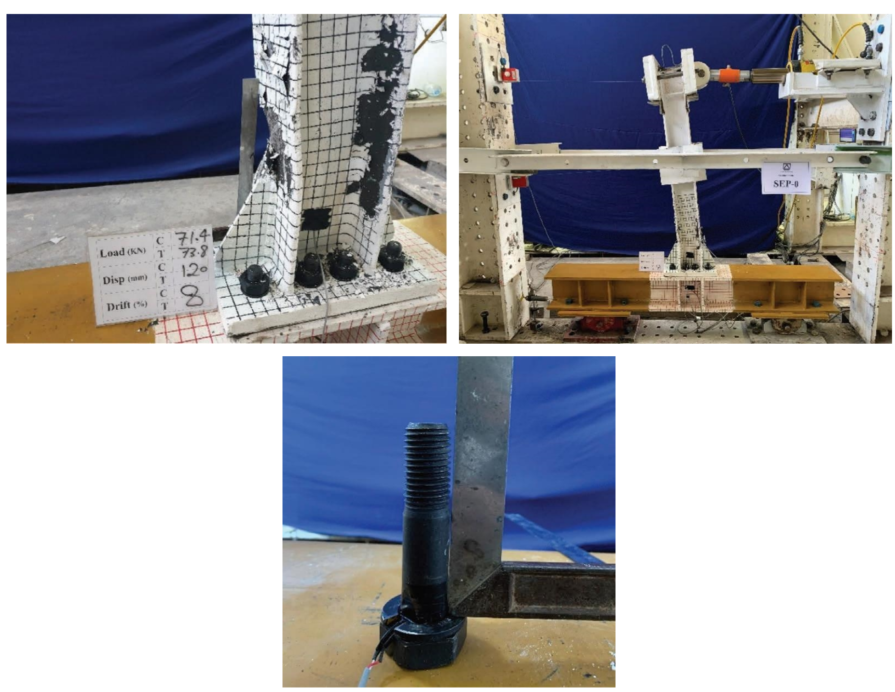
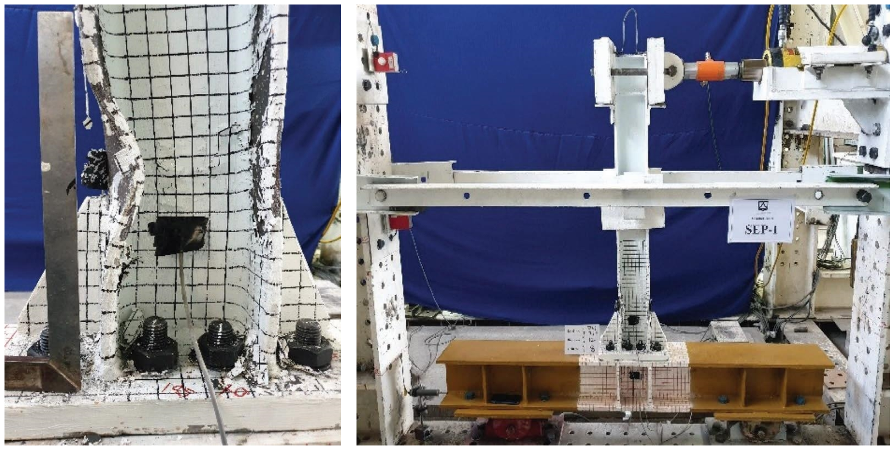


Figure 6: Strain variation history in beam flange.

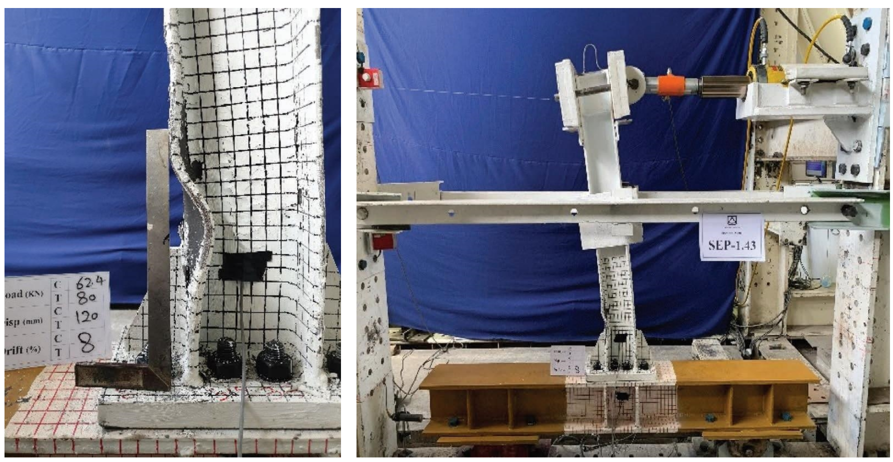


(a)

Figure 7: Continued.



(b)



(c)

FIGURE 7: Specimens at the end of loading. (a) SEP-0.00. (b) SEP-1.00. (c) SEP-1.43.

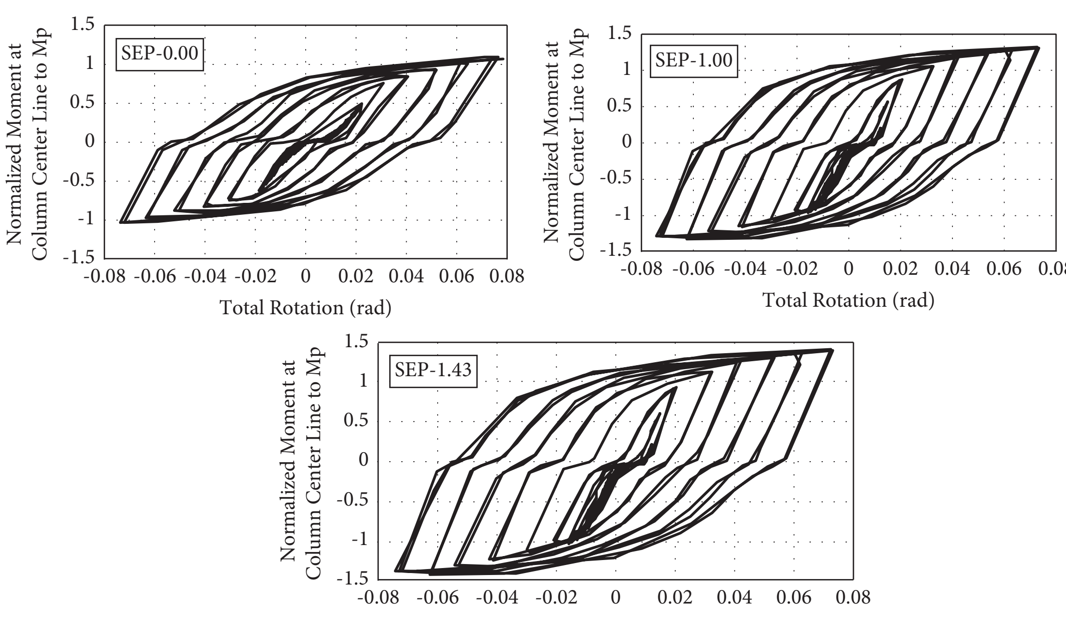


FIGURE 8: Hysteretic moment at the column centerline versus total rotation for all specimens.

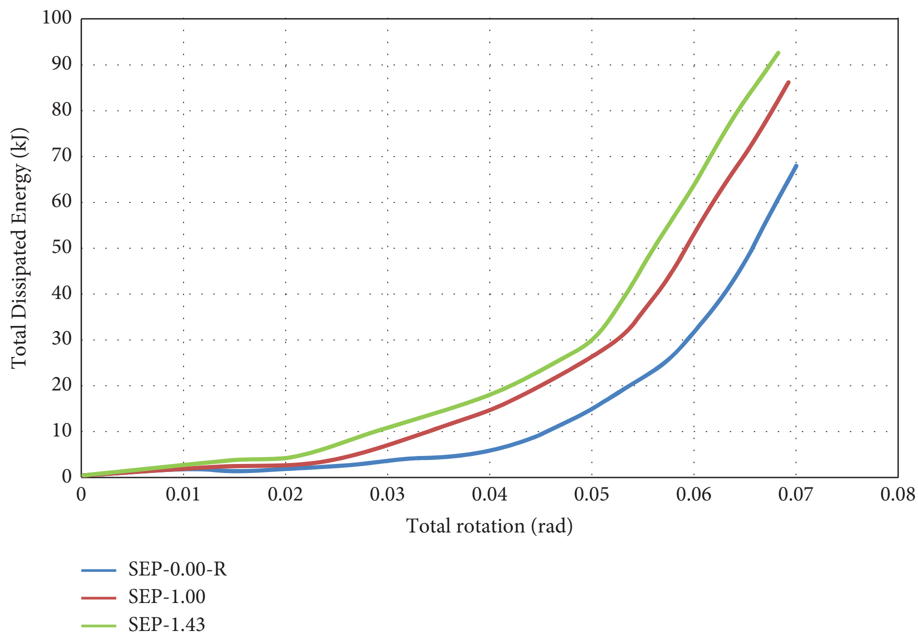


FIGURE 9: Total dissipated energy of the specimens during the loading process.

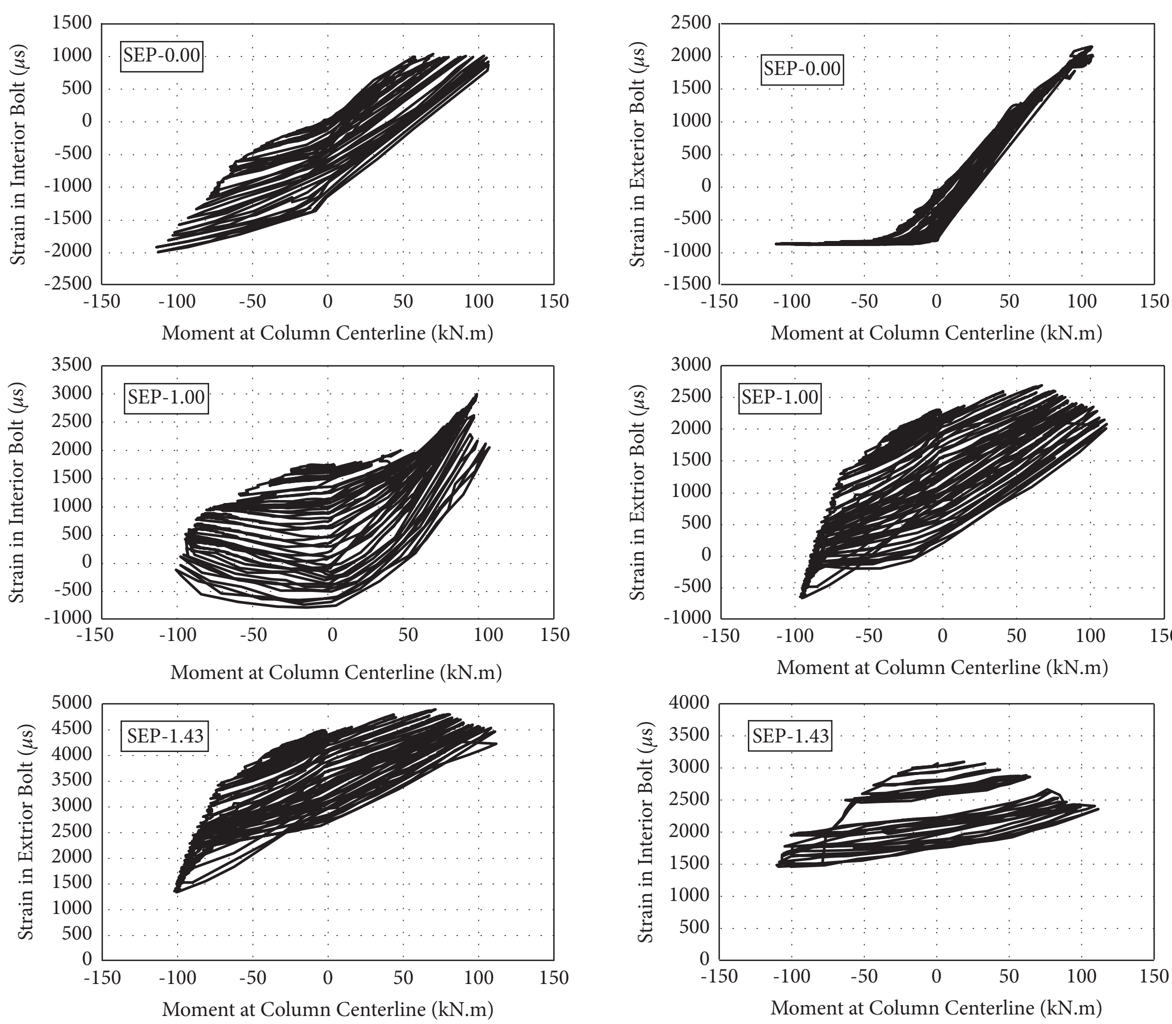


FIGURE 10: Strain variation history for exterior or interior bolt of the specimens during the loading process.

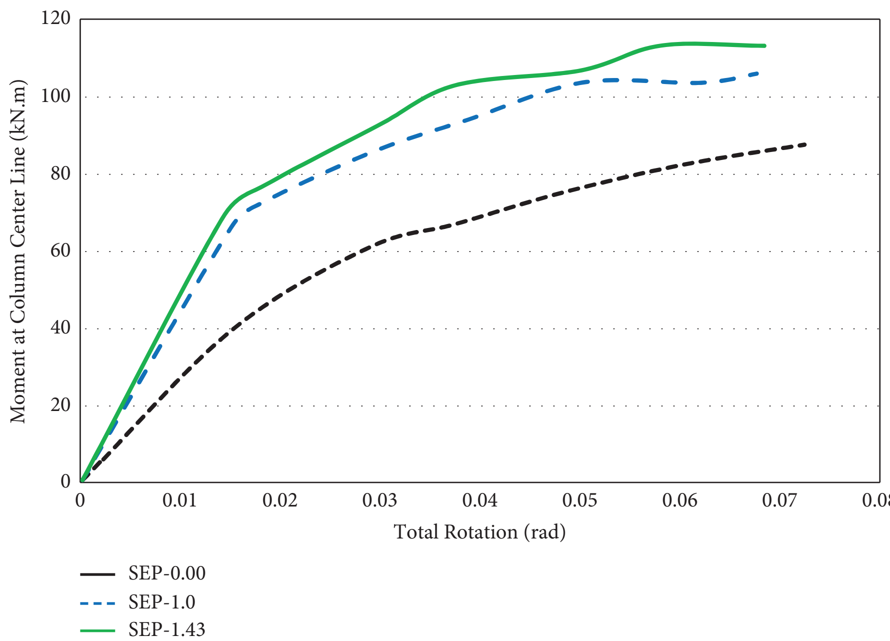


Figure 11: Envelope of moment-rotation hysteresis curve of the specimens.

As shown in Figure 10, during the test, due to cyclic loading, the pretension stress started to decline gradually, and at the end of the test, bolt pretensioning disappeared. As can be seen, due to the contribution of the end plate and column to exerting compression stress with bolts during the test, the compression stress at exterior bolts is less than at the interior ones. In SEP-1.43 compared with SEP-1.00, the bolt pretension level has been far greater, and at the end of the loading process, pretension stress does not disappear completely. An increase in pretension in the connection leads to a decrease in the degree of rotation at the beginning of yielding in the flange of the beam; by reducing the yield rotation, the range of plastic deformations (difference between θ_u and θ_y) increases, and the ductility of the connection increases. This issue is clearly seen in Figure 11.

6. Conclusion

In this paper, three BSEP beam-to-column connections were tested to investigate the bolt pretension effect under cyclic loading. Three full-scale specimens representing a corner connection of a moment frame were loaded under cyclic loading according to the standard SAC loading protocol. All specimens were designed according to the AISC regulation code. In the first specimen, the bolts were snug-tightened, and in the pretensioned and fully pretensioned specimens, the bolts were pretensioned according to AISC and up to Fu level, respectively. The specimens were tested, and the following results were obtained:

(a) In SEP connections, bolt pretensioning led to increased moment capacity. The moment capacity of pretensioned and fully pretensioned specimens increased to 21% and 29%, respectively, compared with the snug-tightened specimen. According to design regulations, an increase in bolt pretension level seems to be enough.

(b) Total energy dissipated increased significantly in SEP-1.00 and SEP-1.43. For SEP-1.00, the total energy dissipated was 27% more than in SEP-0.00-R. On the other hand, in SEP-1.43, the total energy absorption increased by only 9% more than in SEP-1.00. This indicates that despite a significant increase in SEP-1.43, it led to a slight decrease in dissipated energy.

(c) Since the increase in bolt pretension level led to increased elastic stiffness and contact forces between the end plate and column, during and at the end of the loading process, bolt pretension stress started to decline and disappear. On the other hand, the exterior bolts were more effective in carrying out the tension and compression load. This indicates that the bolt pretension level in exterior bolts had a greater effect on the behavior of BSEP moment connection than the interior ones.

(d) Increase in bolt pretension could lead to initiation of inelastic deformation from a smaller rotation. This can be due to increased contact forces between the end plate and column, leading to higher elastic rotational stiffness.

(e) In pretensioned specimens, most deformations lay within the inelastic range. It should be noted that in SEP-0.00-R, the range inelastic deformation was less than the rotation of 0.04 rad, while in SEP-1.00 and SEP1.43, it was more than the rotation of 0.052 and 0.054 rad, respectively.

(f) Experimental studies on the cyclic behavior of BSEP connections, including SEP-0.00-R, SEP-1.00, and SEP-1.43 specimen, showed that the minimum bolt pretension level by design regulations was enough to improve the seismic performance of special steel moment frame and led to increasing in moment capacity, ductility of connection, energy absorption, elastic rotational stiffness, and improved behavior of bolts of connections.

Data Availability

The data used to support the findings of this study are included within the article. Additional data are available from the corresponding author upon request.

Disclosure

The research was carried out as part of academic and educational research.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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