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Analytical Behavior of Steel Pre-tensioned Bolted Connections with Flushed and Extended End Plates under Bending

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Abstract: The connection between beam and column is repeatedly used in steel structures. Only rigid and pin connections are considered in many steel codes such as Egyptian code (ECP).Floor beams with high reactions need so large number of bolts to be connected with other elements and then these connections do as semi rigid connections. Semi rigid connection can resist some of internal bending moment in steel beam according to connection stiffness and then beam can be redesigned with saving in its weight. The stiffness of semi rigid connection is affected by many parameters as; type of connection end plate (flushed or extended), plate thickness, bolt diameter and stiffening of column panel zone. The analytical investigation utilizes nonlinear finite element modeling techniques using ANSYS program, considering both geometric and material nonlinearity. The functions of ANSYS are used to simulate the pretension force in bolts, as well as the interface between each of end plate, column and bolts accurately. The results of the finite element models were verified and they were found closed with those of experimental and analytical models. The main purpose of this paper is to study the effect of all effective parameters on the stiffness of semi rigid connections according to the basis of the moment-rotation curves. Recommendations are presented to the Egyptian Code of Practice, ECP.

Key words: End plate connection · Finite element analysis · Joints · Pre-tensioned bolts · Semi-rigid connections

INTRODUCTION

End-plate connections consist of two main types, flush and extended end-plate connections are used widely in steel structures, [1-3]. For a given steel building the connection behavior should be predicted accurately to estimate the building structural behavior and to predict the exact force distribution among steel building elements connected using such as connections. In steel frame analysis, two types of connections are considered: rigid (fixed) or pinned (hinged). Ideally rigid connections prevent relative rotations between connected parts. In contrast, ideally pinned connections do not resist moments. In practice, rigid connection issimulated by the extended end plate and the pinned one issimulated by the flushed end plate. Extended end plate connections do not completely prevent the relative rotation between connected members. Also, flushed end plate connections can resist some moments. This fact leads to third type of connections called semi-rigid. Semi-rigid connections allow relative rotations and resist some moments depending on the connection stiffness. Connection stiffness is affected by many parameters such as, the end plate is flush or extended, the end plate extends beyond one or both ends of the beam flanges, the length of this extension, the diameter of the bolt, the number of bolt rows, the vertical and horizontal spacing of the bolts, the grade of the bolts, the end-plate thickness, the stiffening of the end plate or column panel zone, the bolt pretension force, the dimensions of the beam and column, the yield strength of the steel and the coefficient of friction between the contact surfaces etc [4]. Because of these large numerous parameters, it was too hard to study the behavior of these connections comprehensively except by physical experimental tests which have a high cost. Nowadays and due to computer programs development, finite element modeling, FEM, gives accurate values and helps in the study of such types of connections.

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Eurocode 3 gives a mathematical method to represent the semi-rigid connection behavior based on the moment-rotation, (M-θ), characteristics of the joint. This method is called component method.

The experimental works became limited in the last years. Nevertheless the results obtained from experiments are still most reliable. In 1981, Phillips and Packer [5] conducted a series of experimental tests to determine the influence of the end-plate thickness on moment-rotation characteristics and the end-plate collapse mechanism. They also studied end-plate connections with two rows of tension bolts in order to study the influence of the second row on the stiffness of the connection. They suggested two failure mechanisms for end-plates with two rows of bolts, by which they determine the required thickness of the end-plate. They concluded that flush end-plates with two rows of bolts in the tension region are suitable for semi-rigid construction and that the second row of tension bolts is effective but to a much extent lesser than the first row. In 1983, Srouji *et al*. [6] conducted several experimental testing and yield line analyses on flushed end-plate connections with one and two rows of bolts below the tension flange. In this study yield line analysis closely predicted the strength of the connection. In 1997, Seradj [7], who carried experimental tests on eight-bolt stiffened extended beam-to-column end-plate connections under cyclic loading came to the conclusion that this type of connection might act as either a fully rigid or a semi-rigid connection depending mainly on the thickness of the end-plate and the bolt diameter. Connections with thinner end-plates and thicker bolts behave more like semi-rigid connections. In 2004, Luís Simões da Silva *et al*. [8] conducted an experimental investigation of eight statically loaded extended end plate moment connections to provide insight into the behavior of this joint type up to collapse. The specimens were designed to confine failure to the end plate and/or bolts without development of the full plastic moment capacity of the beam. The parameters investigated were the end plate thickness and steel grade. The results show that the increase in end plate thickness results in an increase in the connection flexural strength with stiffness and a decrease in rotation capacity. Similar conclusions were drawn for the effect of the end plate steel grade, though no major variations in the initial stiffness were observed. In 2007, Shi *et al*. [9] conducted eight experiments to study the behavior of flushed and extended end plate friction type connections and, also, to verify a proposed analytical model derived by them. The parameters of this study were end plate thickness, bolt diameter, column stiffener and end plate stiffener. In this paper the stiffened extended end plate connections have been recommended for end plate moment connections in multi-story steel frames. Also, the results showed that increasing the end plate thickness and bolt diameter lead to an increase in the ultimate moment capacity and stiffness of connection whereas rotation capacity decreases. As mentioned before the experimental studies are too hard due to the numerous parameters affecting the connection behavior and, also cost. Many researchers tend to FEA as it is more effective to study the connections widely with low costs. In 1991, Safar [10] developed three dimensional finite element models to simulate common types of steel bolted corner and base connections. The finite element results were then verified by comparison to some available experimental test data. The results show that the extended end plate connections using high strength bolts possess some flexibility which may result in the reduction of the weight of steel frames. Also, the adequate connection flexibility can be obtained by the reduction of the bolts pretension. In 1994, Bahari and Sherbourne [11, 12] used the three dimensional finite element analyses to study the structural behavior of end plate bolted connection to stiffen columns. The study took all major influencing factors into account on the behavior of connection such as, column, beam, bolt components, material plasticity with strain hardening and the contact effect between the end plate and column. All analyses were applied under incremental pure bending moment. The parameters of this study were bolt size, thickness of end plate and thickness of column flange. The results of study showed that thick end plate distributed the beam flange force symmetrically around the beam flange in tension zone. Also, the study showed that the increase of bolt size or decrease end plate thickness lead to increase the prying force. Finally, the study found that the pretension of the bolt did not affect the force distribution of bolts and the prying action at the ultimate capacity of the connection. In 1997, Biahwanath *et al*. [13] presented a finite element analysis of unstiffened flushed end plate bolted connection. The study took nonlinear material properties with strain hardening and contact effect into account. The study supposed insignificant contribution of welds, bolt head and column fillets in moment rotation relationship; therefore, it neglected them in simulation of modeling. The comparison indicates a good correlation between the modeling and experimental results. In 1998, Xiao and Pernetti [14] introduced nonlinear simulation of extended end plate connections. The study investigated the effect of thickness of end plate in both cases of stiffened and

unstiffened column under applying pure bending moment. The results of the study indicated that increasing of the end plate thickness increases the stiffness and strength of the connection. In addition, connection rigidity improved using column web stiffeners. These decreased the stress at compression and tension zones in column web and changed the mode of failure for whole connection. In 2004, Maggi *et al*. [15] performed a parametric study of steel bolted end-plate connection using three dimensional finite element modeling. The study took the material and geometric nonlinearities into account. Temperature gradients were used in bolts to impose pretension forces. The study generated the bending moment in the connection by displacement load at tip of beam. The parameters of this study were bolt size and thickness of end plate. The results of finite element modeling were compared against the experimental data. The results of the study showed that the created gap between the end –plate and the column flange is at tension zone. It was found that the thick end-plate leads to high stiffness of connection. It was also found that force distribution of bolts depends on the relative stiffness between the end-plate and the bolts. In 2005, Abolmaali *et al*. [16] developed a three-dimensional nonlinear finite element model by using the FE software package ANSYS version 6.0 to study flushed end plate connections with one row of bolts below the tension flange, which included material, geometric and contact nonlinearities. The connection M–θ data for these FEM analyses were curve fitted to Ramberg–Osgood and three-parameter power model equations to obtain parameters defining these equations. It was shown that both models predicted the M–θ plots closely, with the more accurate model being the Three-Parameter Power model. In 2008, Gang Shi *et al*. [4, 17] described the development of a finite element numerical model with the ability to simulate and analyze the mechanical behavior of different types of beam–column end-plate connections in which all of the bolts are pre-tensioned. The general purpose ANSYS software formed the basis of the modeling and its new functions are used to simulate the interface between the end plate and the column flange, as well as the pretension force in the bolts. The finite element models were compared with experimental test results, which verified that the numerical procedure can simulate and analyze the overall and detailed behavior of a number of types of bolt-pretensioned end-plate connections and components accurately. In 2012, Roxana Balc *et al*. [18] presented the analysis with finite elements of a steel joint with end plate and prestressed bolts, using finite element methods. The results obtained were compared with the experimental data in order to validate the model. This study offers the possibility to simulate the actual behavior of the connection by finite elements at low costs in comparison with experimental models.

The main purposes of this paper is to accurately simulate the flushed and extended end plate connections including the effect of end plate thickness, bolt diameter and stiffening of column panel zone on the behavior of each type.

Finite Element Analysis: The most comprehensive version of ANSYS [19] provides many new functions that can simulate and analyze the mechanical behavior of bolted end-plate connections accurately. These functions are used to simulate the chosen end plate connections as well as to draw the moment-rotation curves for them. Actually, drawing moment-rotation curves which represent the result of very complex interaction between connection elements requires the consideration of various parameters. Some of these parameters were considered in this study such as; geometrical and material nonlinearities of the elementary parts of the connection, bolt pretension force and its response under general stress distribution, contacts between bolts and plate components, i.e. bolt shank and hole, bolt head or nut contacts and compressive interface stresses and friction. In contrast, the effect of slip due to bolt to hole clearance, welds, imperfections (i.e. residual stresses) and representation of the environment (i.e., temperature) are not considered in the FE modeling.

Geometric Details of Connections: To conduct the parametric study, it was decided to vary the geometric variables of all bolted end plate connections within the practical ranges. The types of connections studied in this study are flushed end plate, FEP and extended end plate, EEP, as shown in Fig. 1. Twenty models were studied; ten for FEP connections from FEP1 to FEP 10 and the others for EEP from EEP1 to EEP10. Two experimental test specimens were selected from Gang Shi *et al*. [17] to verify the finite element models and were regarded as reference models. These two models are indicated as FEP1 and EEP1for flushed and extended end plate connections, respectively and all the other connections differ from them in only one geometric parameter, e.g. the thickness of the end-plate, bolt diameter and whether the joint has a column panel zone stiffener or not. Details of all connection specimens used in FEA are shown in Table 1 and Fig. 1. The beams and columns for all models in this

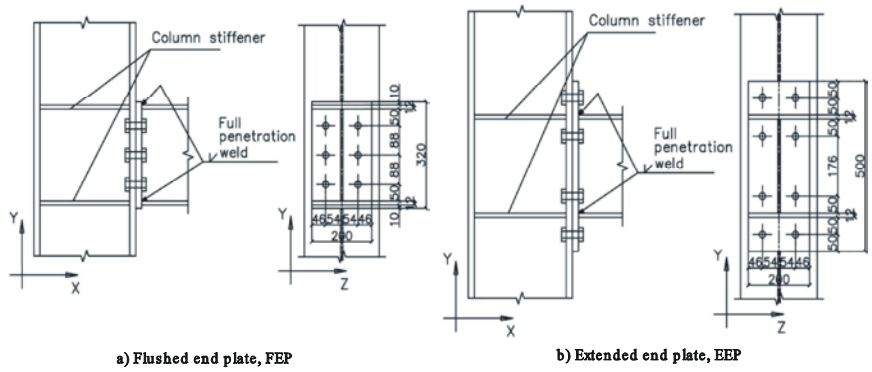


Fig. 1: Details of FEP and EEP connections considered in this study (dimensions in mm).

Table 1: Types and details of specimens.

Specimen number	End plate thickness (mm)	Bolt diameter, d (mm)	Number of bolts	Column stiffeners	Specimen number	End plate thickness (mm)	Bolt diameter, d (mm)	Number of bolts	Column stiffeners
FEP 1	20	20	6	Yes	EEP 1	20	20	8	Yes
FEP 2	12	20	6	Yes	EEP 2	12	20	8	Yes
FEP 3	16	20	6	Yes	EEP 3	16	20	8	Yes
FEP 4	25	20	6	Yes	EEP 4	25	20	8	Yes
FEP 5	30	20	6	Yes	EEP 5	30	20	8	Yes
FEP 6	20	12	6	Yes	EEP 6	20	12	8	Yes
FEP 7	20	16	6	Yes	EEP 7	20	16	8	Yes
FEP 8	20	24	6	Yes	EEP 8	20	24	8	Yes
FEP 9	20	27	6	Yes	EEP 9	20	27	8	Yes
FEP 10	20	20	6	No	EEP 10	20	20	8	No

FEP refers to flushed end plate and EEP refers to extended end plate.

Table 2: Dimensions of all beams and columns cross-sections (mm).

Item	Section depth	Web thickness	Flange width	Flange thickness
Beam	300	8	200	12
Column	300	8	250	20

Table 3: Material properties for high strength bolts

Stress (MPa)	Strain (%)
0	0
990	0.483
1160	13.6
1160	15

Table 4: Minimum pretension forces in bolts according to AISC-LRFD.

Bolt diameter, d (mm)	Pretension force (kN)
12	69
16	114
20	179
24	257
27	334

study are taken with the same dimensions, as detailed in Table 2. The thickness of the column panel zone stiffener is 10 mm. These stiffeners were aligned with beam flanges to have maximum effect of them in enhancing the resistance of column flange in bending and the column web in tension zone and compression zone as well.

Finite Element Modeling: In the modeling herein, all elements of the beams, columns, end-plates, stiffeners were meshed by the 8-node solid structural elements SOLID45. The stress–strain relationship for the steel plates was taken as elastically–perfect plastic with a Poisson's ratio of 0.3. The yield strength and elastic modulus of steel plates thicker than 16 mm were taken as 363 MPa and 204, 227 MPa, respectively; but for plates thinner than 16 mm were taken 391 MPa and 190,707 MPa. The high strength bolts were also, meshed by the 8-node solid structural elements SOLID45. The stress–strain relationship for the high strength bolts (including the bolt heads, shanks and nuts) was taken as tri-linear. The points used to define the stress–strain relationship for the bolts are taken as given in Table 3. The PSMESH command was used to define pretension sections in the middle of the bolt shanks and to generate the pretension elements PRETS179 through which the pretension forces in the bolts are applied by the command SLOAD. Pretension force values are taken according to AISC LRFD represented in Table 4. The interface between the end-plate and the column flange, bolt shank and hole and bolt head/nuts and end plate/column flange were simulated by creating contact pairs with the 3D target

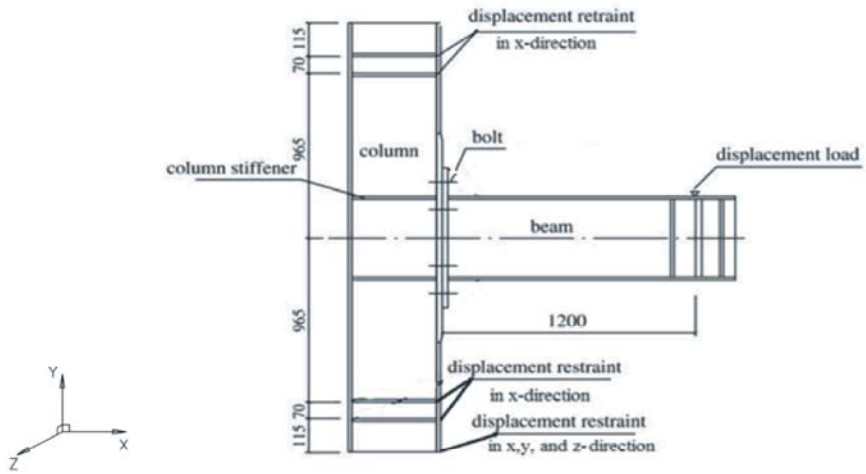


Fig. 2: Finite element models dimensions and loading arrangement with the same values as which considered in the experimental test by Gang Shi *et al.* [17] (all dimensions in mm).

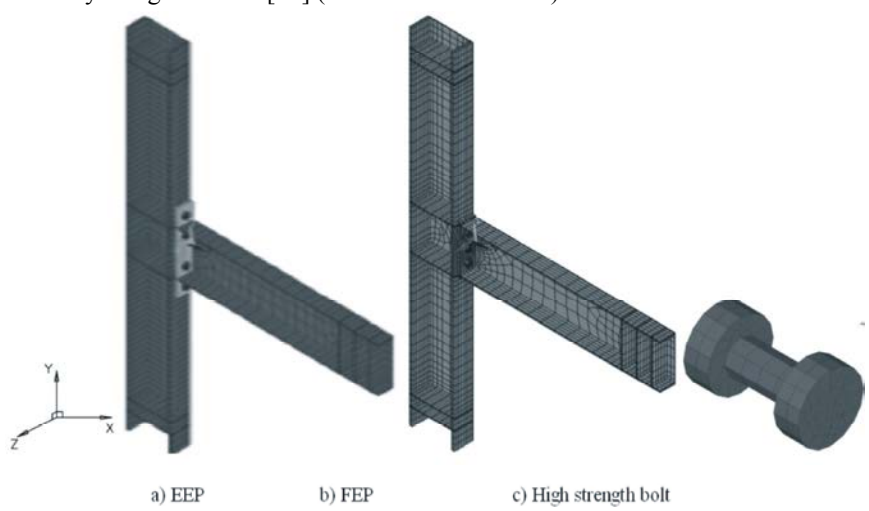


Fig. 3: Typical finite element model of connections.

surface elements TARGE170 and the 3-D 8-node surface-to-surface contact elements CONTA174. The element CONTA174 is capable of representing contact and sliding between 3-D target surfaces, which are TARGE170 element in this simulation environment and a deformable surface, defined by this element. The coefficient of friction for all contact surfaces is assumed to be uniformly distributed and was taken equal 0.44 as the same value considered in the experimental test of Gang Shi *et al*. [17].

Because of the connection geometry is symmetric about the central plane passing parallel through the beam and column webs, only one half of each of the connection specimens was modeled for the FEA in order to reduce computation time. All degrees of freedom that occurred out of symmetric plane will be restrained against moving, so the nodes along (X and Y) plane at line passing through connection centerline were restrained against the translation in Z-direction and against the rotation around the Y and X-directions. Similar to experimental test configuration, the column was restrained near the ends against the translation in the X-direction and restrained against translation in the X, Y and Z-directions at one tip (Fig. 2). Finite element models dimensions and loading arrangement of the two connection types are taken with the same values as considered in the experimental test by Gang Shi *et al*. [17] as shown in Fig. 3.

RESULTS AND DISCUSSION

Definition of the Moment-rotation Curve: The following verification and parametric study are based on studying the moment-rotation, (M-θ), curves of each connection. The joint moment, M_j, was taken as the product of the

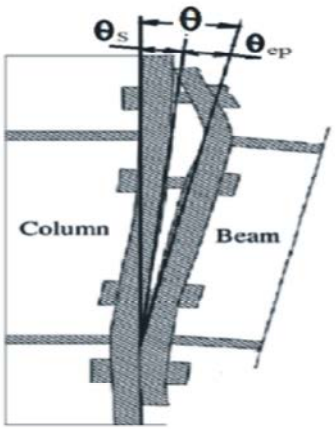


Fig. 4: Definition of joint rotation, θ.

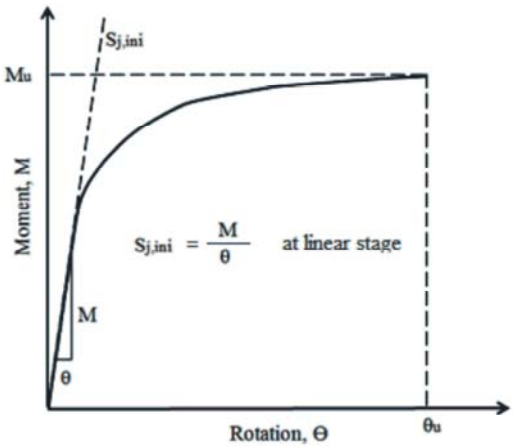


Fig. 5: Definition of joint initial rotational stiffness, $S_{j,ini}$.

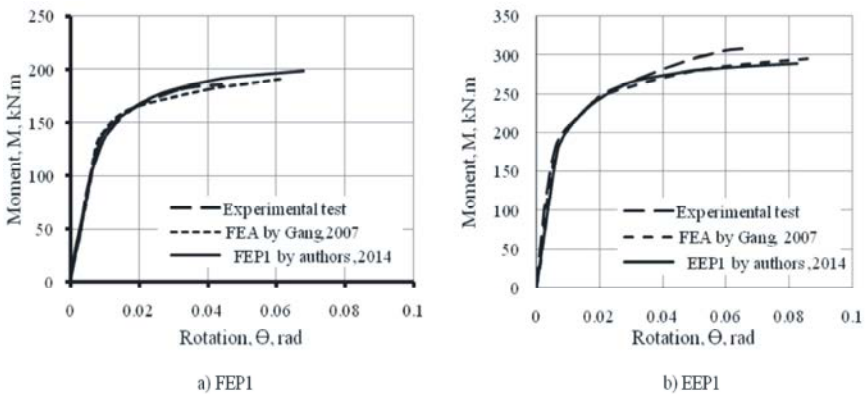


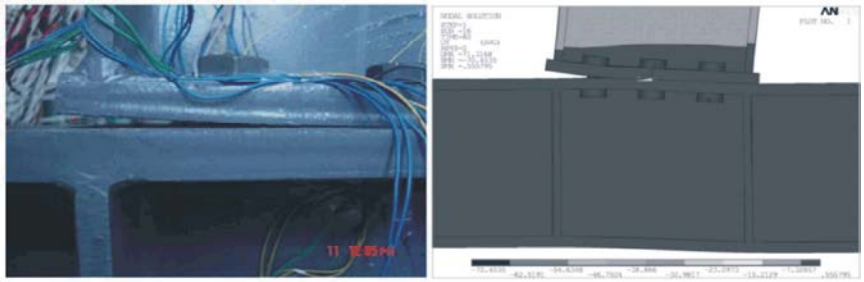
Fig. 6: Comparison between moment-rotation curves of the experimental test, FEA by Gang 2007 and FEA by the authors 2014

Table 5: Comparison of loading capacities (kN) between results of FEA and experimental tests

Specimen type	Experimental test	FEA by Gang, 2007	FEA by authors, 2014	Current FEA / Experimental test	Current FEA /Previous FEA
FEP	155.3	156.2	161.23	1.04	1.03
EEP	256.9	244.2	242.68	0.95	0.99

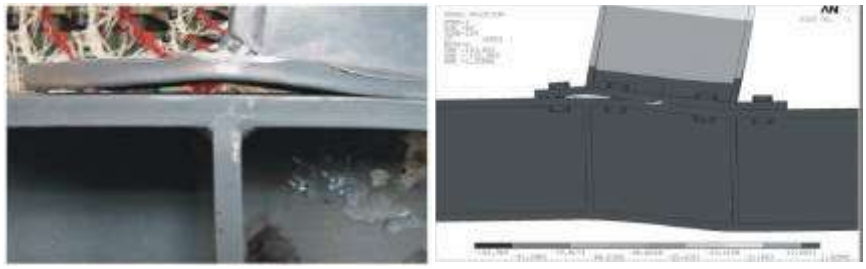
load and its lever arm of 1200 mm (which is the distance from the loading point to the column flange, as shown in Fig. 2. The joint rotation, θ_j, of the beam to column end plate connection is defined as the relative rotation between the center lines of the beam top and bottom flanges at the beam end and it usually includes two parts: the shearing rotation θ_s, contributed by the panel zone of the column and the end gap rotation θ_{ep}, caused by the relative deformation between the end plate and the column flange, including the bending deformation of the end plate and column flange as well as the extension of bolts (Fig. 4). The initial rotational stiffness, $S_{j,ini}$, is defined as the slope of curve at initial linear stage (Fig. 5).

Verification of Finite Element Results: For the purpose of verification and check of modeling accuracy, the moment-rotation curves have been plotted for both analytical and available experimental test data obtained from Gang Shi *et al*. [17]. These curves were then compared with the curves obtained from the current finite element modeling. Fig. 6 shows the comparisons between moment-rotation curves of the current study and the available experimental test and previous FEA. Table 5 shows the loading capacities of analytical models and experimental test results. Figs. 7 and 8 show the end plate deformation for the flushed and extended end plate connections, respectively. When comparing the moment-rotation curves of this study with FEA and physical



a) Experimental test by Gang, 2007 b) FEA by authors, 2014

Fig. 7: Comparison of ultimate failure mode of FEP connection.



a) Experimental test by Gang, 2007 b) FEA by authors, 2014

Fig. 8: Comparison of ultimate failure mode of EEP connection.

experimental tests done by Gang, it can be seen that the initial stages of loading for the two connections are linear and that the agreement between results of FEA and experimental tests are extremely close. In the nonlinear range, the agreement is very close with small discrepancies between the FEA results and the physical experimental tests. The small discrepancies between the FEA results and the physical experimental tests need some discussion. Firstly, the stress–strain relationship for the steel plates used in the FEA was elastic–perfectly plastic and so strain hardening was neglected. This idealization is reflected in the FEA results for connections whose capacities are governed by steel plates, including the panel zone in shear and the end-plate in bending, where the discrepancies are larger owing to the post-yield steel strength; the discrepancies of specimen EEP1 are attributable to the neglect of the strength of the end-plate in bending after yielding. The load capacities of the connection FEP1 are controlled by the bolts and the error is much smaller. Secondly, the values of the applied pretension force for all of the bolts were determined by the design values given in Table 4. In conducting physical experimental tests it is very difficult to induce a predetermined bolt force by pretensioning techniques because the bolt forces are very high [20] and, as a consequence, this has been identified as a reason for discrepancies between theoretical predictions and experimental test results [21, 22]. Thirdly, fabrication errors in the experimental test specimens can lead to a geometric deviation between the physical and numerical results. Fourthly, mesh sensitivity or the types of used elements may contribute to these small discrepancies also.

Parametric Study: In order to carry out a parametric study on the flushed and extended end plate connections, three parameters were chosen to study the connection behavior. These parameters are the thickness of the end-plate, bolt diameter and whether the joint has column panel zone stiffeners or not. Twenty FE models with the geometry shown in table1were modeled using ANSYS program and then the results were plotted and discussed to get the effect of each parameter. The (M-θ) curves of all connections considered in this study are shown in Fig. 9. Moment capacity, initial rotational stiffness and ultimate rotation values are shown in Table 6 for all types of connections. According to Fig. 9 and Table 6, the moment capacity and initial rotational stiffness of FEP and EEP connections increased, while the ultimate rotations of FEP connections decreased due to the increase of end plate thickness and bolt diameter. This could be explained as in the case of FEP connections where the end plate behaves as a cantilever with fixation at the upper bolt row. So, in case of connection with large bolt diameter as FEP9 (d=27 mm) the cantilever length of end plate is shorter than in the case of connection with small bolt diameter as FEP6 (d=12 mm) (Fig. 10).

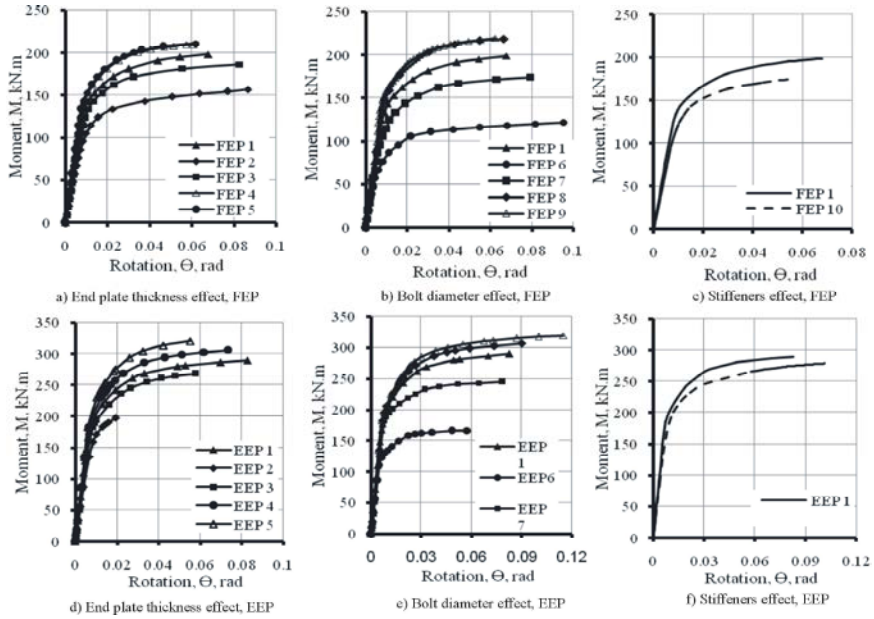
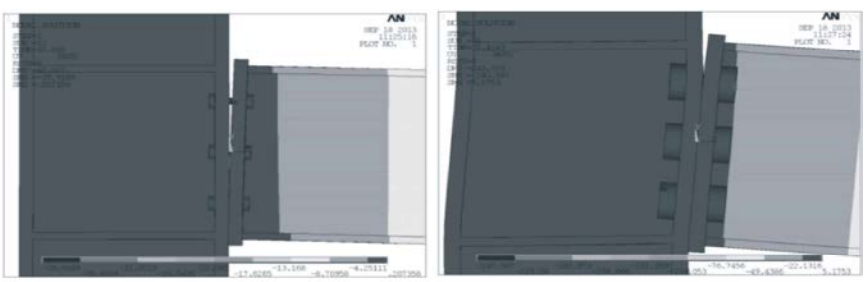


Fig. 9: Comparison of moment-rotation, (M-O), curves for all connections.



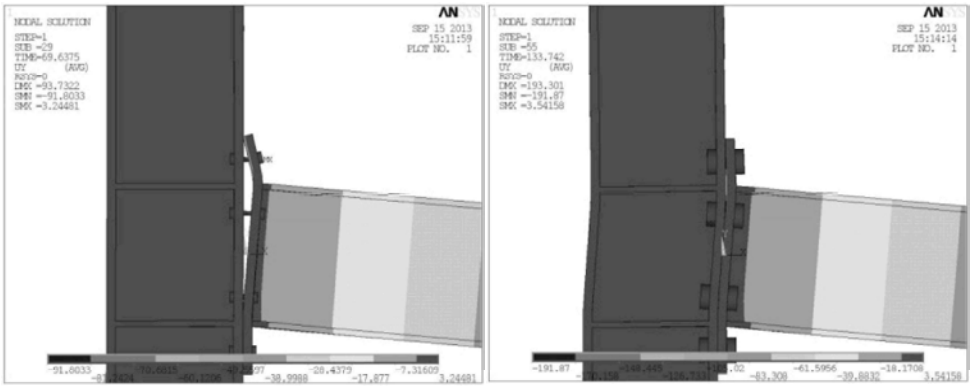
a) FEP6 with d = 12 mm b) FEP9 with d = 27 mm

Fig. 10: Behavior of FEP connections with various bolt diameters, d.

Table 6: Types and details of connections

Specimen number	The moment resistance, $M_{j.Rd}$, (kN.m)	Variation in $M_{j.Rd}$ (%)	Rotational stiffness, S_J, (kN.m/rad)	Variation in S_j (%)	Ultimate rotation capacity, O_u (rad.)	Variation in O_u (%)
FEP 1	198.62	-	16535	-	0.068	-
FEP 2	156.71	- 21.10	13276	- 19.71	0.087	+ 27.94
FEP 3	191.04	- 3.82	14973	- 9.45	0.082	+ 20.59
FEP 4	210.14	+ 5.80	17578	+ 6.31	0.061	- 10.29
FEP 5	210.74	+ 6.10	18059	+ 9.22	0.062	- 8.82
FEP 6	121.94	- 38.61	13440	- 18.72	0.095	+ 39.71
FEP 7	174.32	- 12.23	14988	- 9.36	0.079	+ 16.18
FEP 8	210.14	+ 5.80	17578	+ 6.31	0.061	- 10.29
FEP 9	210.74	+ 6.10	18059	+ 9.22	0.062	- 8.82
FEP 10	174.53	- 12.13	13935	- 15.72	0.054	- 20.59
EEP 1	289.6	-	27416	-	0.083	-
EEP 2	197.2	- 31.91	9347	- 65.91	0.019	- 77.11
EEP 3	268.6	- 7.25	25771	- 6.00	0.058	- 30.12
EEP 4	306.0	+ 5.66	29100	+ 6.14	0.073	- 12.05
EEP 5	320.2	+ 10.57	29926	+ 9.16	0.055	- 33.73
EEP 6	166.3	- 42.58	23983	- 12.52	0.057	- 31.33
EEP 7	244.4	- 15.61	25955	- 5.33	0.079	- 4.82
EEP 8	306.8	+ 5.94	28520	+ 4.03	0.090	+ 8.43
EEP 9	319.4	+ 10.29	29292	+ 6.84	0.115	+ 38.55
EEP 10	278.8	- 3.73	22958	- 16.26	0.101	+ 21.69

Variation in values is based on comparing with FEP 1 for all FEP connections and with EEP 1 for all EEP connections.
(-ve) sign refers to decrease and (+ve) sign refers to increase.



a) EEP6 with d = 12 mm b) EEP 9 with d = 27 mm

Fig. 11: Behavior of EEP connections with various bolt diameters, d.

On the contrary, the ultimate rotations of EEP connections increased due to the increase of the bolt diameter when the end plate thickness is up to 20 mm. It is found that in case of connection EEP6 with d=12 mm the failure occurred in bolts (brittle failure with limited rotation capacity) and in case of connection EEP9 with d=27 mm the failure occurred in end plate in tension zone (ductile failure with large rotation capacity). When the end plate thickness increased to 25 mm or 30 mm, the EEP connections failure was in the bolts which fail in a brittle manner and hence the ultimate rotation capacity decreased. Also, it is noticed that the increase in the moment capacity of FEP connections type is more efficient due to the increase of bolt diameter of the connection than the increase of the thickness of the end plate. In case of bolt diameter increased from 12 mm (FEP6) to 27 mm (FEP9), it was found that the moment capacity increased by 72.82%, but when the end plate thickness increased from 12 mm (FEP2) to 30 mm (FEP5), the moment capacity increased by 34.48%.The initial stiffness in case of FEP connections increased with the same rate in the case when end plate thickness or bolt diameter increased. It is expected that a FEP connection with end plate thickness of 12 mm and bolt diameter of 12 mm will perform in a ductile manner than the FEP modeled connections with smaller moment resistance. In EEP9, the end plate tends to be fixed-fixed which results in increasing the end plate capacity and then increases the connection capacity. In this case, all joint rotations come from the shearing deformation of the panel zone of the column (Fig. 11). Also, it is found that the rotational stiffness of EEP connections highly depends on the end plate thickness than the bolt diameter as when the bolt diameter is increased from 12 mm (EEP6) to 27 mm (EEP9), the rotational stiffness increased by 22.14%. When the end plate thickness is increased from 12 mm (FEP2) to 30 mm (FEP5), the rotational stiffness increased by 337% (approximately three times). The moment resistance of EEP connection is highly influenced by increasing the bolt diameter than increasing the end plate thickness as when the bolt diameter is increased from 12 mm (EEP6) to 27 mm (EEP9), the moment resistance increased by 92%. When the end plate thickness is increased from 12 mm (FEP2) to 30 mm (FEP5), the moment resistance increased by 62%.

It is found that the ultimate moment resistance and initial rotational stiffness of FEP and EEP connections decrease when the horizontal column panel stiffeners were removed. As shown in Figs. 9-c and 9-f, the ultimate moment resistance and initial rotational stiffness for FEP10 and EEP10 are lower than that in case of FEP1 and EEP1, respectively. In FEP connections the maximum rotation depends mainly on moment rotation of the end plate plus a small value of the shearing rotation of the column web panel. In case of FEP10 connection, the failure happens in the column web at compression zone before the end plate reaches its maximum rotation and so the value of the shearing rotation of the column web panel decreases which explains decreasing of the maximum connection rotation by 21%. In contrast, the maximum rotation of EEP connections depends mainly on the shearing rotation of the column web panel plus a small value of the moment rotation of the end plate. In case of EEP10 connection, the value of the shearing rotation of the column web panel increased by 22% before the connection reaches its ultimate moment resistance.

Fig. 12 shows a comparison between the effects rate of the studied parameters on the both FEP and EEP connections with respect to initial stiffness and Fig. 13 shows this comparison with respect to moment capacity. For each curve, the difference between curve slopes represents the parameter effect rate between the two connections types; i.e. FEP and EEP. It is seen that small

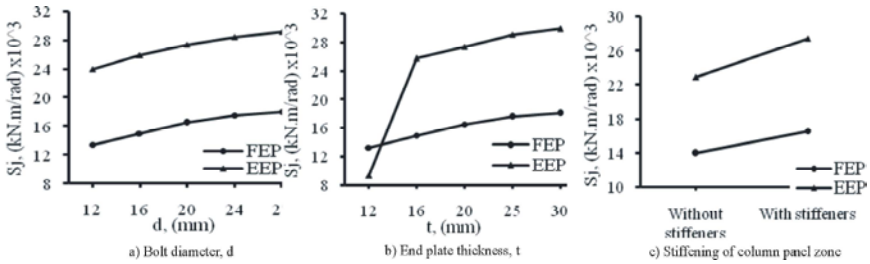


Fig. 12: Comparison between the effect of the studied parameters on the initial stiffness of FEP and EEP connections.

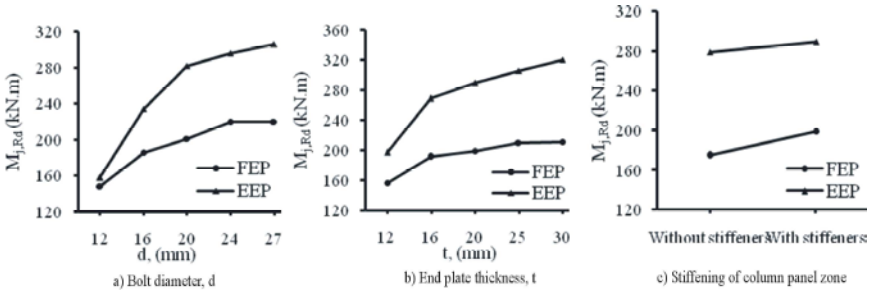
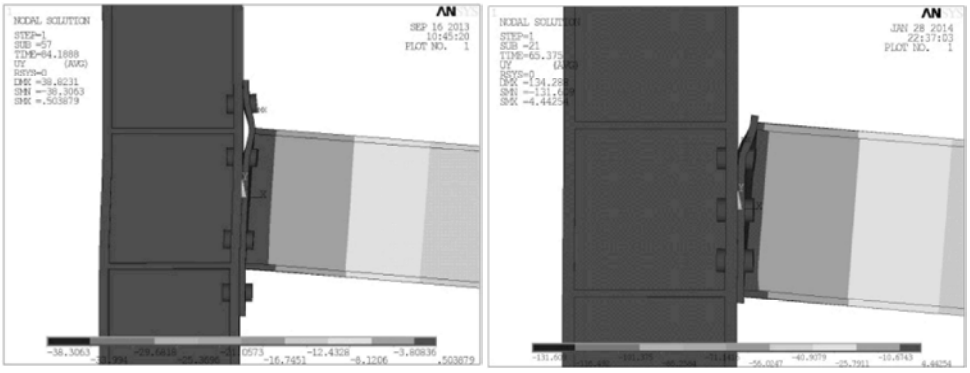


Fig. 13: Comparison between the effect of the studied parameters on the moment resistance, $M_{j,Rd}$, of FEP and EEP connections.



a) EEP b) FEP

Fig. 14: End plate behavior of FEP and EEP connections at end plate thickness, t= 12mm.

values of end plate thickness and bolt diameter have a significant influence on the behavior of EEP connections than FEP connections. This may be explained as the behavior in case of FEP connections depend on the end plate length measured from the beam tension flange and first bolt row. In contrast, the initial stiffness in case of EEP depends on the distance between the two tensioned bolt rows. The distance between the two tensioned bolt rows of EEP is greater than the length measured from the beam tension flange and the first bolt row in FEP connection (Fig. 14). The influence of other parameters on the rotational stiffness is almost similar on the two types of connections. The moment capacity of EEP connections is affected by bolt diameter and end plate thickness more than the FEP connections.

CONCLUSIONS

To study the behavior of steel end plate connections, two types of connections were studied; flushed end plate, FEP and extended end plate, EEP. Twenty specimens were modeled using ANSYS program; ten models for FEP connections and the others for EEP connections. Two models only were verified with experimental tests done by Gang Shi *et al*. [17] and values found to be close. The aim of this research is to study the effect of type of connection end plate (flushed or extended), plate thickness, bolt diameter and stiffening of column panel zone on the moment resistance, initial stiffness and ultimate rotation of the connection. It is concluded that the moment capacity and initial rotational stiffness of FEP

and EEP connections increase with increasing the end plate thickness and bolt diameter. The ultimate rotations of FEP connections decrease with increasing the bolt diameters and end plate thickness. In contrast to FEP connections, the ultimate rotations of EEP connections increase with increasing the bolt diameters and the end plate thickness up to t=20 mm. When the end plate thickness increased to 25 or 30 mm, the EEP connections failure was in the bolts which fail in a brittle manner causing a decrease in the ultimate rotation capacity. Also, it is noticed that increasing the bolt diameter of FEP connections is more efficient in moment resistance than increasing the end plate thickness. It is expected that a FEP connection with end plate thickness of 12 mm and bolt diameter of 12 mm will perform in a ductile manner than the FEP modeled connections with smaller moment resistance. The EEP connection rotation is governed by the shearing rotation of the column web and the FEP connections is governed by the end plate stiffness. The ultimate moment resistance and initial rotational stiffness of FEP and EEP connections decrease when column panel zones unstiffened. It is seen that for small values of end plate thickness and bolt diameter, EEP connections have a significant influence on their behavior than FEP connections. The influence of other parameters on the rotational stiffness is almost similar on the two types of connections. The moment capacity of EEP connections is affected by bolt diameter and end plate thickness more than the FEP connections.

RECOMMENDATIONS

To reduce the total structure weight; it is recommended that ECP should allow considering the effect of the flushed end plate connections rigidity in the analysis and design of steel structures.

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