Behavior of steel beam to built-up box column moment connections with vertically stiffened panel zone

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Abstract

Continuity plates (horizontal diaphragms) are conventionally used in steel beam to column moment connections to provide a continuous load path for transferring moments in joints. However, fabricating connections with continuity plates in frames with built-up box columns is usually faced with problems which are mainly due to lack of accessibility. In the present study, the effectiveness of using vertical stiffeners in the panel zone of steel beam to built-up box column connections as an alternative to conventional continuity plates is investigated through a numerical investigation. In this regard, first, the Finite Element (FE) model which is implemented in FE software ABAQUS is validated using the results of tests conducted before on exterior steel beam to box column connections. The analysis results show a good agreement between the analysis and test results in terms of the hysteresis moment-rotation curve and the development of yielding in the connection. Then, the behavior of exterior and interior connections with different configurations of panel zone, including bare connection (with no stiffener in the panel zone), connection with conventional continuity plates, and joints with the proposed vertical stiffener detail, is investigated. The numerical simulations are conducted on models with different beam and column dimensions. According to the analysis results, the connections with unstiffened panel zone usually do not show a satisfactory performance. In these models the premature yielding of column flange is the dominant mode of failure and prevents connections from reaching the beam plastic strength. Oppositely, connections whose panel zones are stiffened using the proposed detail, show an almost similar behavior with those with conventional continuity plates. Furthermore, according to the yield pattern in the models and the hysteresis behavior of the connections, they can be classified as beam to column connections in special moment frames.

Keywords Steel beam to column moment connection · Built-up box column · Continuity plates · Numerical study · Vertical stiffener

1 Introduction

Beam to column connections in steel moment resisting frames, play a crucial role in the seismic behavior of buildings with this structural system. The brittle premature failure of moment connections in the past earthquakes such as Northridge 1994 and Kobe 1995 and other destructive seismic events confirms this statement [1, 2]. Undesirable modes of failure, particularly in welded moment connections, not only weakens the structure against lateral loads, but also adversely affect the global ductility of structures. Accordingly, extensive investigations have been conducted to evaluate the seismic behavior

of different types of moment connections [3–6], and to propose techniques for strengthening or modifying the vulnerable ones [7–9]. Moreover, innovative types of connections have been introduced in recent years [10–12].

Steel beam to column moment connections have different parts which provide a continuous load path with adequate strength to transfer internal actions (moment and shear) through beam to column joints, see Fig. 1. Among these parts, the continuity plates which are also called horizontal diaphragms prevent the column flanges from excessive out-of-plane deformations and local yielding.

Different studies have been performed to assess the role of continuity plates in the seismic behavior of steel beam to column moment connections [13–15]. However, the behavior of beam to built-up box column connections has been investigated in limited studies [16–18]. While placing the continuity plates in the built-up box columns and welding all four

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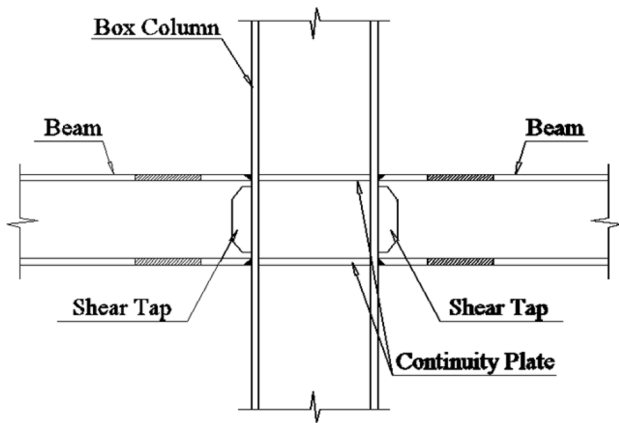 University of Technology, Shiraz, Iran

Fig. 1 Different parts of steel beam to column moment connections

sides of internal diaphragms to the column is usually faced with difficulties which is mainly due to the lack of accessibility. Sometimes electro slag welding is used to overcome this problem. Ozkula et al. [19] investigated the seismic behavior of beam to built-up box column connections with electro slag welding through an experimental study. In another investigation, Chen et al. [20] evaluated the behavior of electroslag welding in box columns through a numerical study. They also conducted a literature review on the previous studies on the seismic behavior beam to box column connections focusing on the performance of electroslag welding. Nonetheless, the

inspection of such welding is not easy in practice. Therefore, some investigations have been conducted on alternatives for continuity plates in beam to box column moment connections. For instance, Goswami and Murty [21] proposed a detail for externally reinforcing the moment connections. They evaluated the seismic behavior of the connection through a numerical study. In another research, Mirghaderi et al. [22] investigated the behavior of beam to built-up box column connections with a vertical through plate detail in an experimental and numerical study. Furthermore, load transferring mechanism in this connection was assessed analytically. Using external diaphragms in box columns was another technique which was suggested by Rezaifar et al. [23]. In this regard, they conducted tests on interior connections in which the panel zone was reinforced with external diaphragms. Yin et al. [24] evaluated the behavior of the panel zone of box columns which were reinforced with external diaphragms. In another research, the behavior of the panel zone of beam to built-up box column connections was assessed through an experimental study by Jahanbakhti et al. [25]. Beside the tests, they also evaluated the failure of column flanges under tensile and compressive forces and proposed a formulation for determining the minimum thickness of column flanges for eliminating the continuity plates. It should be noted that such relations were developed before for columns with wide flange sections. They also conducted a numerical study on the behavior of panel zone in beam to box column connections without continuity plates [26].
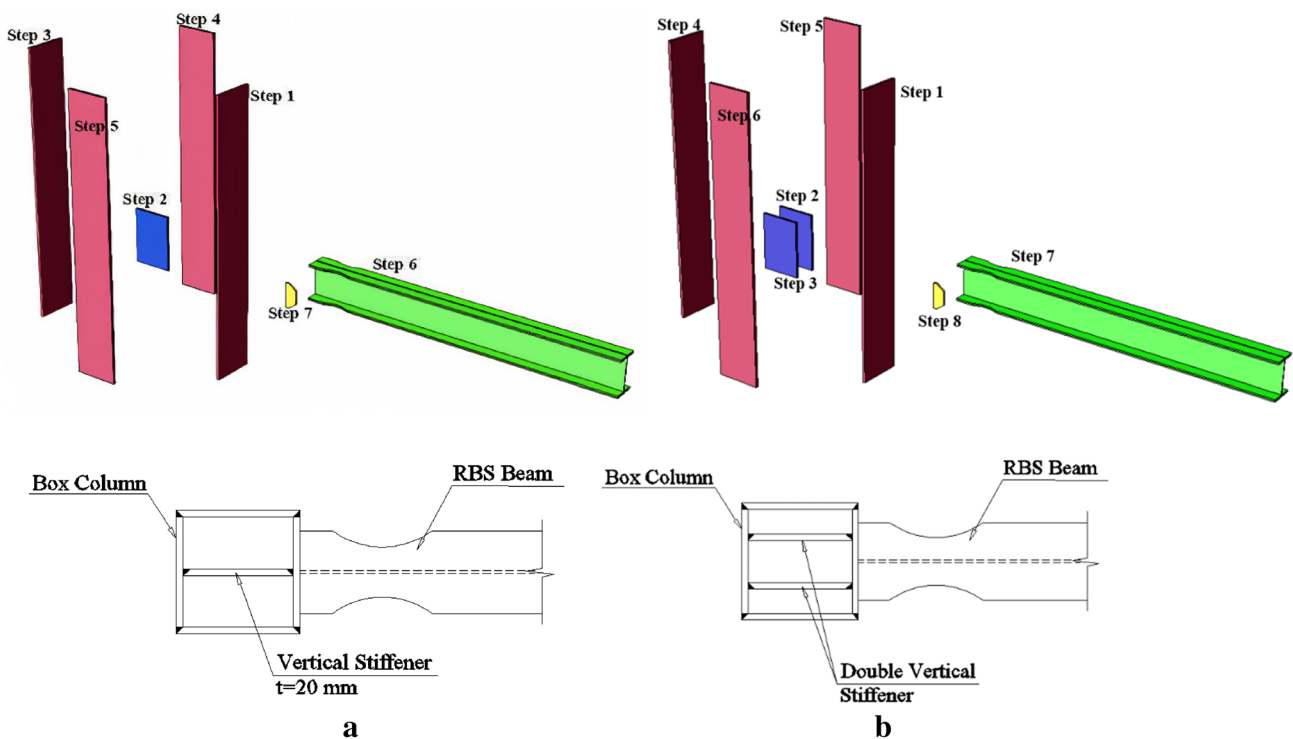

Fig. 2 The fabrication sequence of beam to built-up box column with vertically stiffened panel zone, **a** single stiffener, **b** double stiffeners
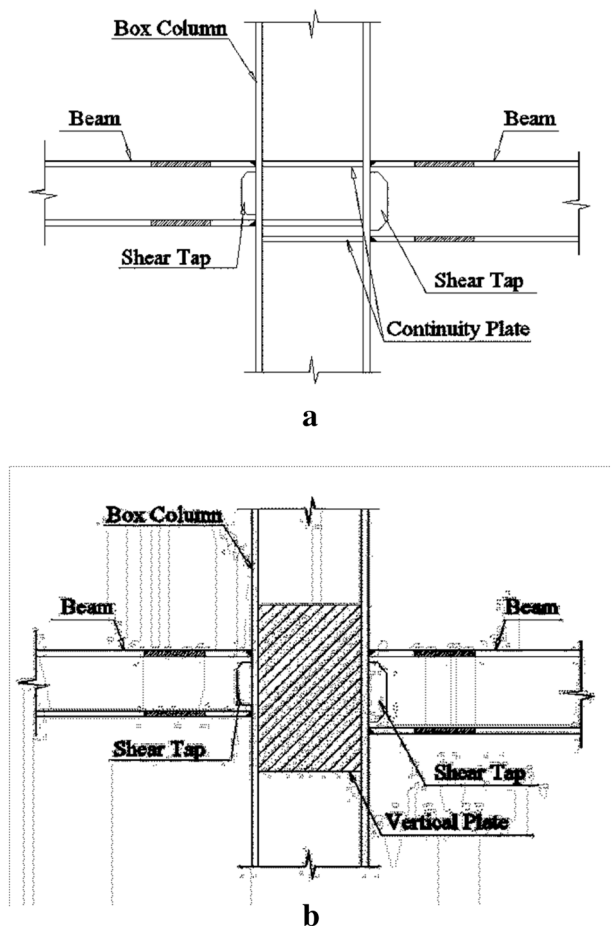
Fig. 3 The panel zone of interior beam to column connections with unequal beam heights with, **a** conventional continuity plates, **b** vertical stiffener

Recently, Najafgholipour et al. [27] have proposed the idea of stiffening the column flanges in the joint region with a vertical stiffener. In this technique, a vertical stiffener is utilized in the panel zone instead of continuity plates. In this way, the box column becomes similar to a boxed wide flange section in the joint region. They evaluated this method through tests and Finite Element (FE) models on exterior connections. This technique was inspired from the need of continuity plates in connections with boxed wide flange column section. According to the AISC 341-16 [28] provisions, under some conditions the continuity plates are not necessary in columns with wide-flange and boxed wide flange sections. In contrast to the common continuity plates, the proposed technique can be easily used in practice with minimum practical limitations. Even in joints which do not satisfy the conditions of eliminating continuity plates which is mainly due to the small thickness of column flanges, two vertical stiffeners can be utilized. Two vertical stiffeners can be also placed in the panel zone of the

joints easily in practice. The fabrication sequence of the box columns with this detail of panel zone is illustrated in Fig. 2.

Another advantage of using vertical stiffeners as alternative to horizontal continuity plates is ease of fabrication of interior joints connecting beams with unequal heights. As illustrated in Fig. 3, when two beams with different heights are jointed to a column, additional continuity plate is needed at the level of beam flanges to provide the load path in the panel zone, while in the connections with vertically stiffened panel zone, the vertical stiffener can provide the load path alone.

In this study, the efficiency of the proposed detail in which the panel zone is stiffened with vertical stiffeners is investigated through a parametric study using FE simulations. Since several connections in a 3D building frame are interior connections and the seismic demand on these joints is usually higher than the exterior ones, the behavior of the interior beam to built-up box connections with a vertical stiffener in the joints panel is also investigated. In this regard, first the FE model is validated using a test conducted before on a connection under cyclic loadings. Then, through a parametric study, the efficiency of using vertical stiffener as an alternative for continuity plates in exterior and interior connections is assessed. Furthermore, in models with relatively thin column flanges where the proposed idea by using a single vertical stiffener in the panel zone may not work, two vertical stiffeners are utilized to stiffen the column flanges. The analysis results are presented in terms of hysteresis moment-rotation curves, plastic strain distributions in whole of the model and along some critical paths, and internal forces induced in different parts of the connections.

2 Verification of numerical modeling

2.1 Experimental program

In order to evaluate the seismic behavior of beam to built-up box column connections, an experimental study was conducted by Najafgholipour et al. [27] on two exterior connections. Here, to validate the accuracy and applicability of the FE method in the analysis of connections, the test specimens (RBS-V and RBS-H) were simulated and analyzed. The dimensions of the test specimens are illustrated in Fig. 4. Accordingly, the connections were made of a built-up box column with section dimensions of 300 mm × 300 mm and thickness of 20 mm. Moreover, hot rolled wide flange profile HEA 280 with a reduced section at a distance of 150 mm from the column face was utilized as beam in the test specimen. In order to stiffen the panel zone, in RBS-V specimens, a vertical stiffener with a thickness of 20 mm as an alternative to conventional continuity plates was utilized. In this way, the column section became similar to a boxed-wide flange section in the joint region. The
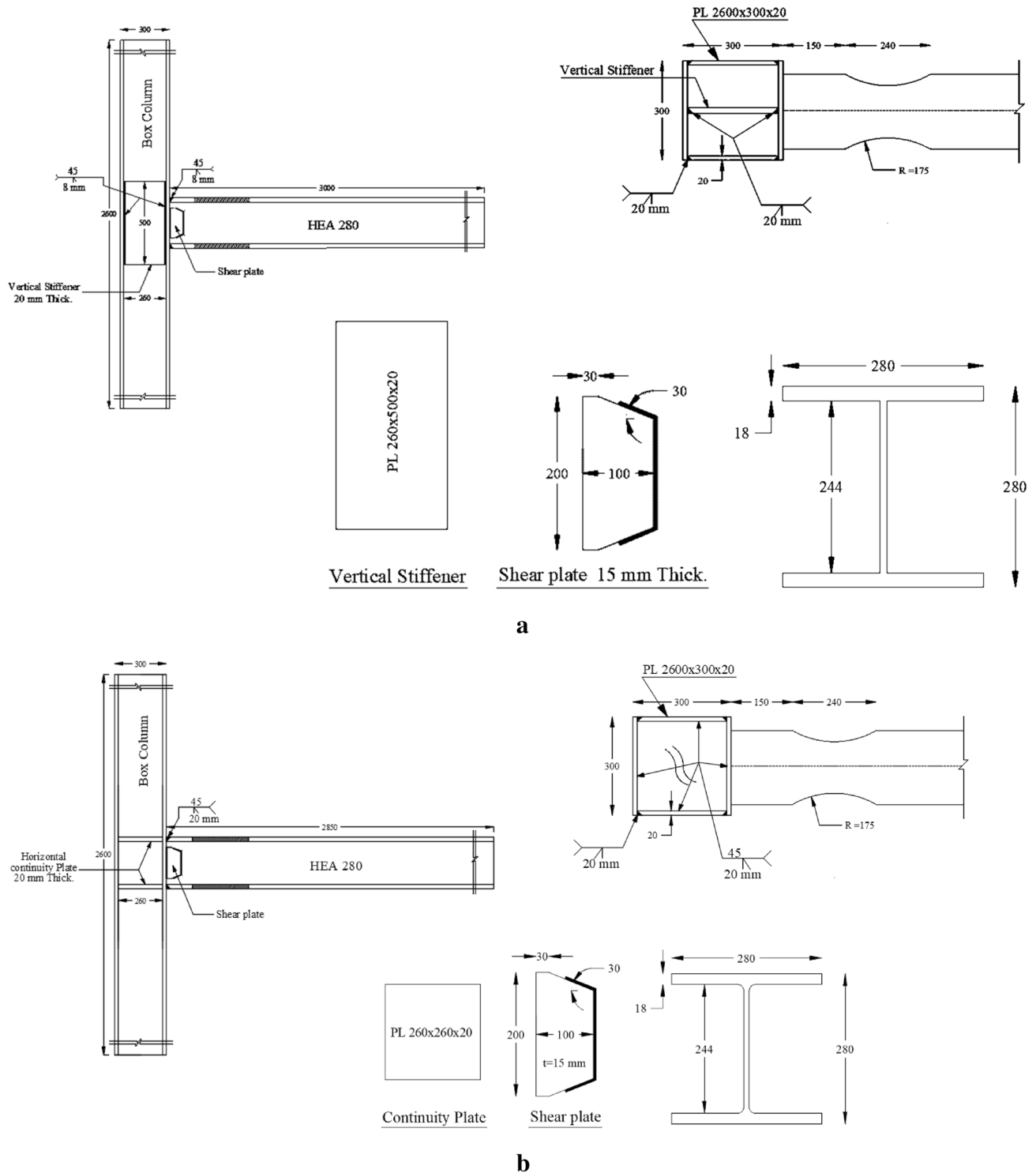
Fig. 4 The configuration and dimensions of the connections, **a** RBS-V connection, **b** RBS-H connection (Units: mm)

panel zone of the other connection (RBS-H) was stiffened with conventional continuity plates. The beam flanges were directly connected to the box column by groove welding.

In order to conduct test on the connections, the specimens were installed in reaction frame by means of two pin supports,

as shown in Fig. 6. The cyclic displacement-controlled loading was inserted at the end of the beam by means of a hydraulic actuator following the loading protocol suggested in AISC 341-16 up to a beam rotation of 0.04 rad, see Fig. 5. Moreover, the mechanical properties of materials used for fabricating the

Fig. 5 Cyclic loading protocol [28]
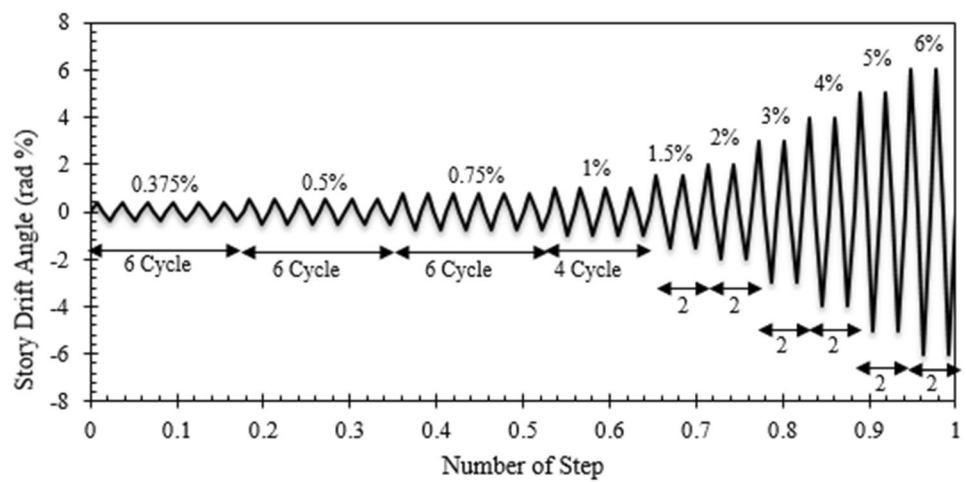
Table 1 Mechanical properties of the steel used in the test specimen

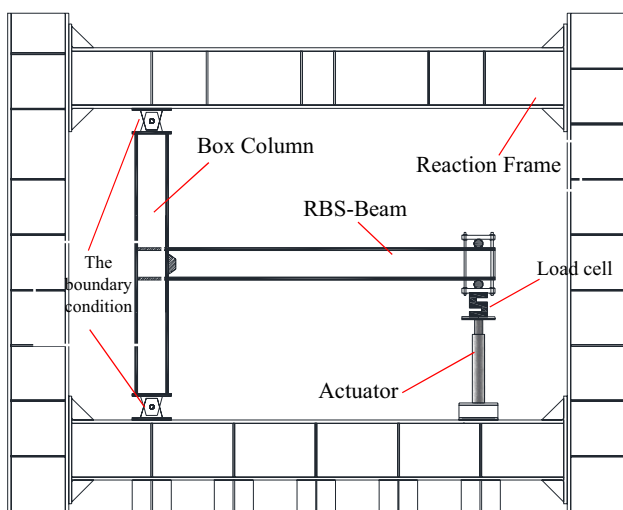| Member	Plate thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Elongation (%)
Beam web plate HEA280	10.05	368.8	521.1	24.8
Beam flange plate HEA280	18	298.1	474.9	28.9
Column plate & continuity plate & vertical stiffener	20	300.7	475.1	24.2
Shear plate	15	292.2	476.7	20.1

Fig. 6 Test set up utilized in the experimental program

Fig. 7 3D FE model of the RBS-V connection and imposed boundary conditions

connection were determined through tests on standard coupons, see Table 1 (Fig. 6).

2.2 FE modeling

In this study, the FE simulations were implemented in ABAQUS FE software. In this regard, first, the geometry of the connection 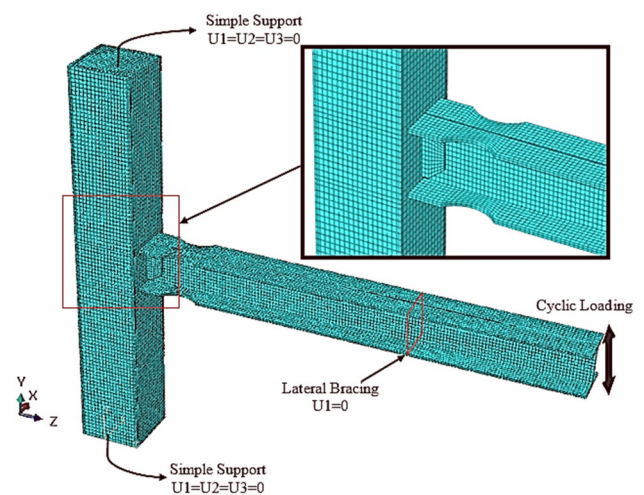(RBS-V) was made according to the dimensions illustrated in Fig. 4. In order to analyze the models, 20-node solid elements (C3D20R) with three translational degrees of freedom were generated with approximate size of 20 mm, see Fig. 7. In total, around 25,000 elements and 130,000 nodes were generated for the analysis of the connection. Since, no weld fracture was observed during the tests, different parts of the model were jointed together with tie constraint. In order

Fig. 8 Hysteresis moment rotation diagrams obtained from experimental study and numerical simulation, **a** RBS-V connection, **b** RBS-H connection
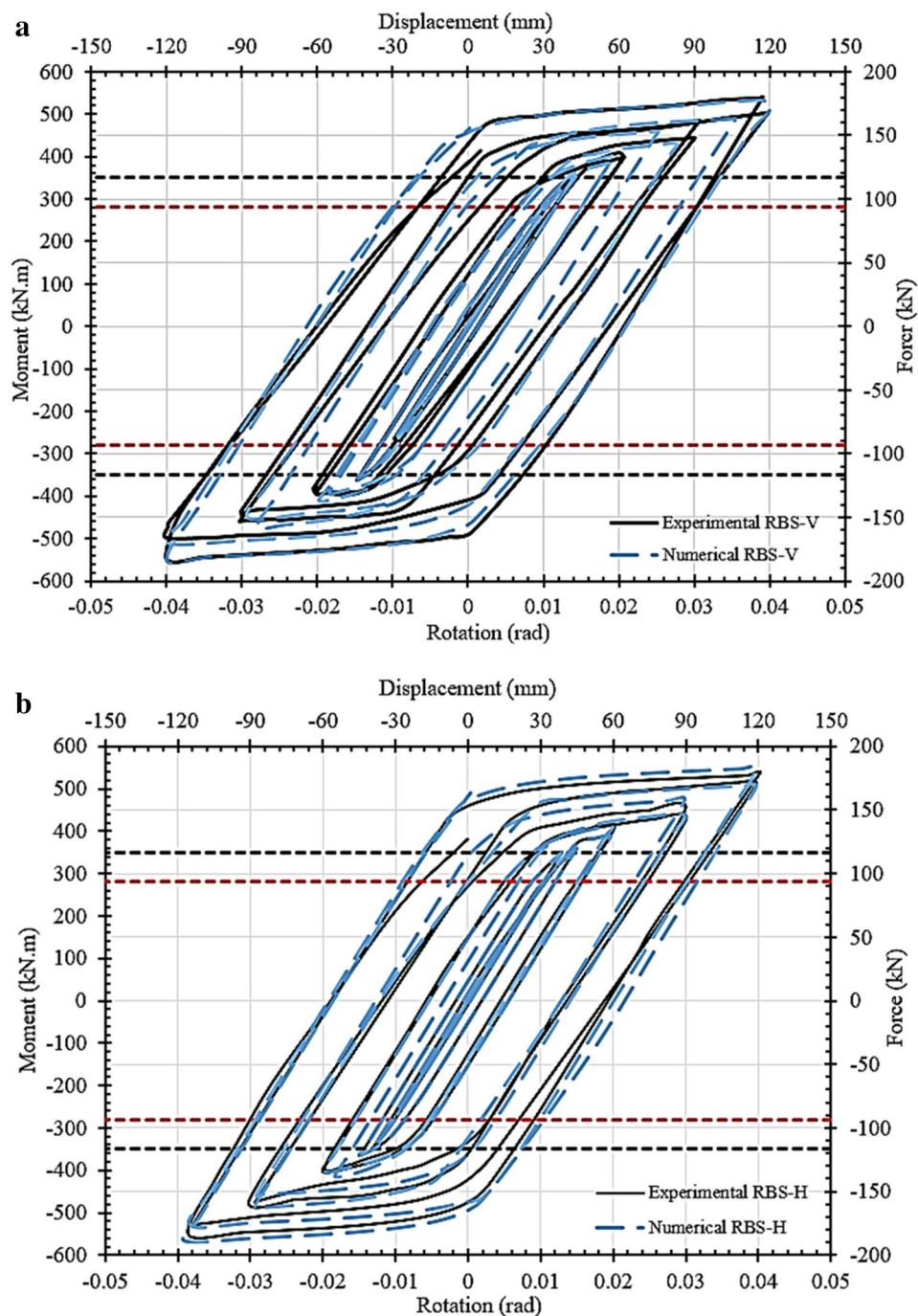
to analyze the model, static/general solution algorithm was utilized.

According to the test set-up, the top and bottom of column were mounted on the reaction frame with a pair of hinge supports. Therefore, the boundary conditions implemented to the numerical model let the top and bottom ends of column rotate freely. For this purpose, the translational DOFs along the centerline of columns, perpendicular to the specimen

plane, were restrained. In addition, the nodes at the coordinates of the lateral support on beam were restrained against out-of-plane translation, see Fig. 7. The displacement-controlled loading was inserted to the end of the beam following the loading protocol shown in Fig. 5.

The mechanical properties of steel used for fabrication of the test specimen, listed in Table 1, were utilized in the FE
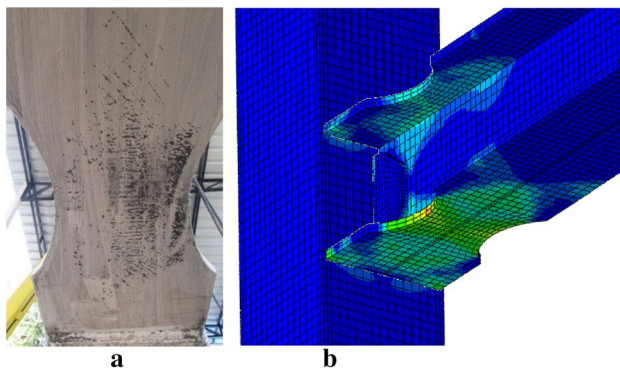
Fig. 9 The concentration of plastic strain in beam, **a** test result, **b** FE model
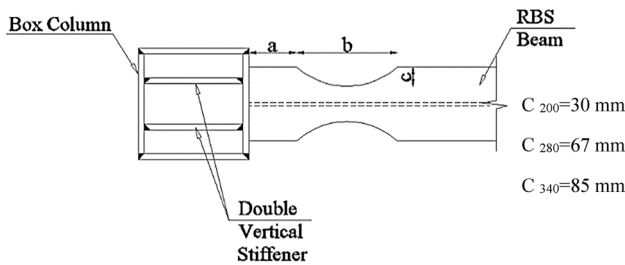

Fig. 10 The dimensions of RBS in the models

model. Bilinear stress–strain model with strain hardening was employed for modeling the nonlinear behavior of steel. Moreover, the Poisson's ratio and Young's modulus of steel were assumed 0.3 and 210 GPa, respectively. It should be mentioned that the Von-Mises yield criterion was used to model the plastic behavior of material.

In addition, although no local buckling was observed in the experimental test up to the last cycle of the loading protocol, an initial imperfection of 0.001 L was defined for the numerical models to recognize any local buckling in the numerical model.

2.3 The analysis results

The hysteresis moment-rotation curves obtained from the FE study and the tests are compared in Fig. 8. Accordingly, the results obtained from the numerical study are in good agreement with those reported in the tests. For instance, the ultimate strength of the connections (RBS-V) obtained from the test and FE model were 550 kN-m and 530 KN-m, respectively. The difference between initial stiffness of the connection obtained from the numerical study and the test was less than 2%.

As reported in the test program, the behavior of the connections was almost linear up to a rotation of 0.01 rad. During the cycles of 0.015 rad and 0.02 rad, the initiation of yielding and

Table 2 The geometrical details of exterior connections investigated the numerical study

Connections	Column size, b_{cf} (mm × mm)	Column flange and web thickness, t_{cf} (mm)	Beam flange width, b_{bf} (mm)	Stiffeners thickness, t_s (mm)	Detail of column stiffener
E-B-300-200	300×300	20	200	0	None
E-H-300-200				20	Continuity plate
E-V-300-200				20	Vertical stiffener
E-B-300-280	300×300	20	280	0	None
E-H-300-280				20	Continuity plate
E-V-300-280				20	Vertical stiffener
E-B-400-200	400×400	25	200	0	None
E-H-400-200				20	Continuity plate
E-V-400-200				20	Vertical stiffener
E-B-400-280	400×400	25	280	0	None
E-H-400-280				20	Continuity plate
E-V-400-280				20	Vertical stiffener
E-B-400-340	400×400	25	340	0	None
E-B-400-340				20	Continuity plate
E-H-400-340				20	Vertical stiffener
E-B-500-200	500×500	30	200	0	None
E-H-500-200				20	Continuity plate
E-V-500-200				20	Vertical stiffener
E-B-500-280	500×500	30	280	0	None
E-H-500-280				20	Continuity plate
E-V-500-280				20	Vertical stiffener
E-B-500-340	500×500	30	340	0	None
E-H-500-340				20	Continuity plate
E-V-500-340				20	Vertical stiffener

Table 3 The geometrical details of Interior connections investigated the numerical study

Connections	Column size, b_{cf} (mm × mm)	Column flange and web thickness, t_{cf} (mm)	Beam flange width, b_{bf} (mm)	Stiffeners thickness, t_s (mm)	Detail of column stiffener
I-B-300-200	300×300	20	200	0	None
I-H-300-200				20	Continuity plate
I-V-300-200				20	Vertical stiffener
I-B-300-280	300×300	20	280	0	None
I-H-300-280				20	Continuity plate
I-V-300-280				20	Vertical stiffener
I-B-400-200	400×400	25	200	0	None
I-H-400-200				20	Continuity plate
I-V-400-200				20	Vertical stiffener
I-B-400-280	400×400	25	280	0	None
I-H-400-280				20	Continuity plate
I-V-400-280				20	Vertical stiffener
I-B-400-340	400×400	25	340	0	None
I-B-400-340				20	Continuity plate
I-H-400-340				20	Vertical stiffener
I-B-500-200	500×500	30	200	0	None
I-H-500-200				20	Continuity plate
I-V-500-200				20	Vertical stiffener
I-B-500-280	500×500	30	280	0	None
I-H-500-280				20	Continuity plate
I-V-500-280				20	Vertical stiffener
I-B-500-340	500×500	30	340	0	None
I-H-500-340				20	continuity plate
I-V-500-340				20	Vertical stiffener

nonlinear behavior in RBS regions of the beam was observed. In beam rotation of 0.03 to 0.04 rad, extended nonlinear behavior in reduced beam section region of the test specimen was obvious. Such behavior was also observed in the FE analysis results. As illustrated in Fig. 9, the initiation and development of plasticity was in the RBS region of the beam. Furthermore, at the end of the test, the concertation of plasticity was mainly in the beam (RBS region) and a distance from the column face.
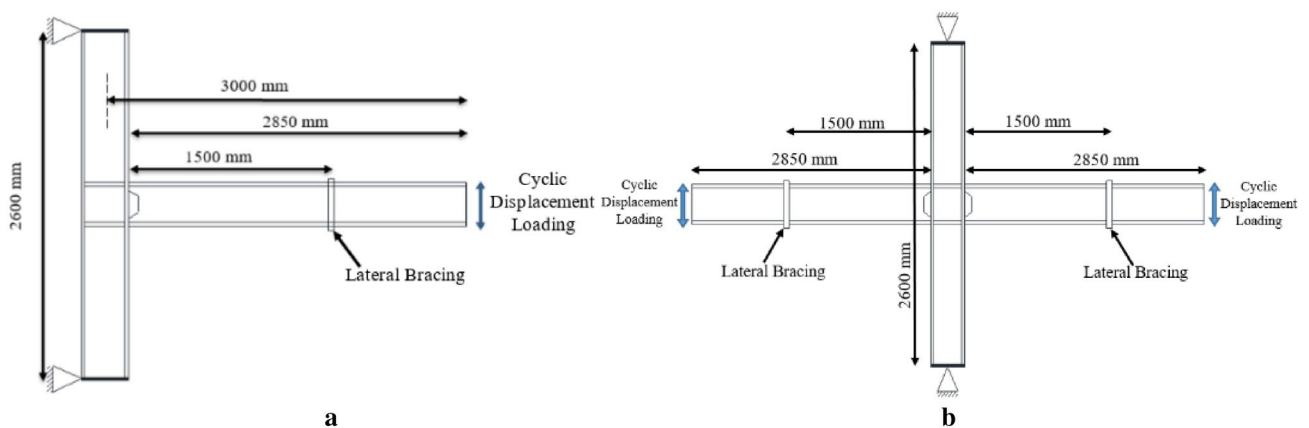

Fig. 11 The dimensions of the investigated connections, **a** exterior connections, **b** interior connections
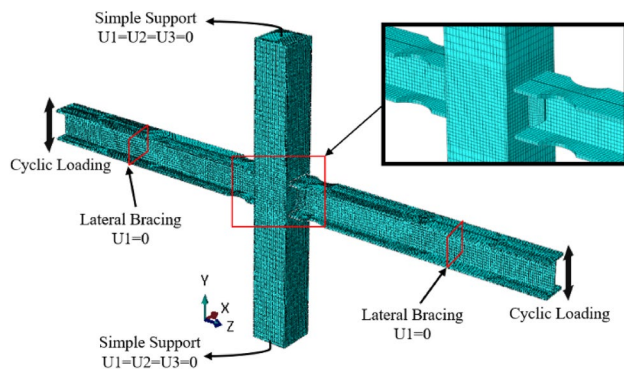
Fig. 12 3D FE model of the interior connections and imposed boundary conditions

3 Parametric study

In order to investigate the efficiency of the proposed detail for stiffening the panel zone of connections, a parametric study was conducted on exterior and interior joints following the validated FE modeling technique. The investigated parameters in the models were the dimensions of column section (width and thickness of box section), the dimensions of beam (flange width and section height), and the type of stiffener utilized in the panel zone (conventional continuity plates, vertical stiffener, and bare connection with no stiffener). In this regard, columns with three section dimensions of 300 mm×300 mm, 400 mm×400 mm, and 500 mm×500 mm were considered. The thickness of the column section was chosen based on the seismic compact section criterion provided in AISC 341-16 [27] (20 mm, 25 mm and 30 mm for columns with different section widths). However, in exterior joints with column section size of 400 mm, models with thinner column flanges (t_{cf}=20 mm), which did not follow the seismic compact section criterion, were considered. In these models, the idea of using two vertical stiffeners (instead of single one) in the panel zone was examined. Since the ratio of beam to column flange widths (b_{bf}/b_{cf}) is an influential parameter on the need of a joint panel zone to continuity plates, beams with three widths of 200 mm, 280 mm and 340 mm, were considered in the investigated models. It should be noted that the RBS dimensions were tuned to achieve beams with almost the same flexural nominal strength, see Fig. 10.

The dimensions of the exterior and interior connections including the column height, beam length, and the distance of lateral support from the column face are illustrated in Fig. 11. In exterior connections, the configuration of the models, loading and boundary conditions are the same as those presented in the verification section. In interior models, the vertical displacement was applied at the ends of the beams on each side of column simultaneously and in opposite directions. The history of the applied rotations was based

on the loading protocol provided in AISC 341-16, see Fig. 5. The properties of the investigated connections (48 connections) are also listed in Tables 2 and 3. Accordingly, the exterior and interior models can be categorized into three groups according to the column size (small, medium, and large sections). It should be noted that for each connection three cases were investigated. A model with conventional horizontal continuity plates as reference detail, a model with the alternative detail with vertical stiffeners in the panel zone, and a bare connection in which no stiffener was placed.

The FE models of the interior connections with the generated mesh and the imposed BCs are illustrated in Fig. 12. The details of FE modeling including the element type, material properties, and material constitutive models utilized in the parametric study, are the same as those described in the verification section.

3.1 Analysis results and discussion

The results of the numerical study corresponding to exterior and interior joint are presented in two separate sections. The results are presented in terms of the hysteresis moment-rotation curves, plastic strain distribution in the models, and plastic strain on the beam flanges.

3.1.1 The exterior connections

The hysteresis moment-rotation curves of the exterior connections in different categories are plotted in Figs. 13, 14, 15. The moments and rotations in the diagrams are calculated with respect to the column axis. The plastic strength of the beams was transferred on the column axis and was specified on the diagrams. The distribution of plastic strain corresponding to the rotation of 0.04 rad is also illustrated for exterior models in Fig. 16. Generally, among three different configurations of panel zone in every connection, the bare connections had the least strength. In most of the bare models, the strength of the connections did not reach the plastic strength of the beam. In a few connections such as E-B-300-280, E-B-400-340, and E-B-500-340, the moment-rotation curves reached hardly to the plastic strength of the beam. It should be noted that among bare connections with similar column dimensions, those with greater b_{cf}/b_{bf}, have smaller strength. For instance, in models with column dimensions of 500 mm×500 mm, the ultimate flexural strengths of connections with beam flanges widths of 200 mm, 280 mm, and 340 mm, are 333.7 kN-m, 396.6 kN-m, and 409.4 kN-m, respectively. By considering the plastic strain distribution in the bare connections, it is inferred that in these models, the local yielding in the flange of box column was the dominant mode of failure. In other words, before reaching the beam to its plastic moment, the plasticity developed in the column. This phenomenon confirms the need of appropriate

Fig. 13 Hysteresis moment-rotation curves of exterior columns with column size of 300 mm × 300 mm, **a** b_{bf} = 200 mm, **b** b_{bf} = 280 mm
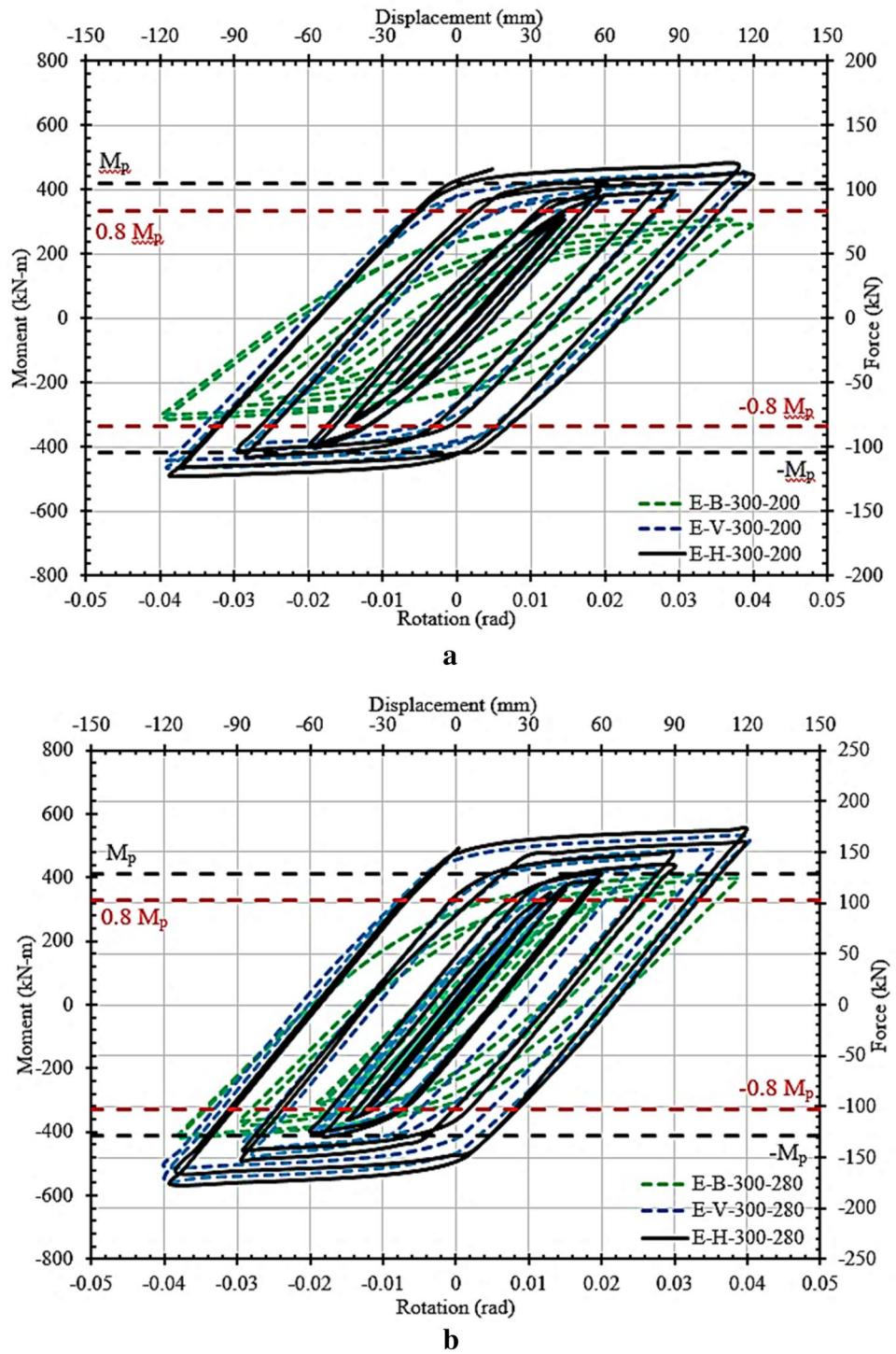
reinforcing detail in the panel zone of beam to box column connections. It should be noted that using thicker plates as column flange can also reduce the deformations and avoid
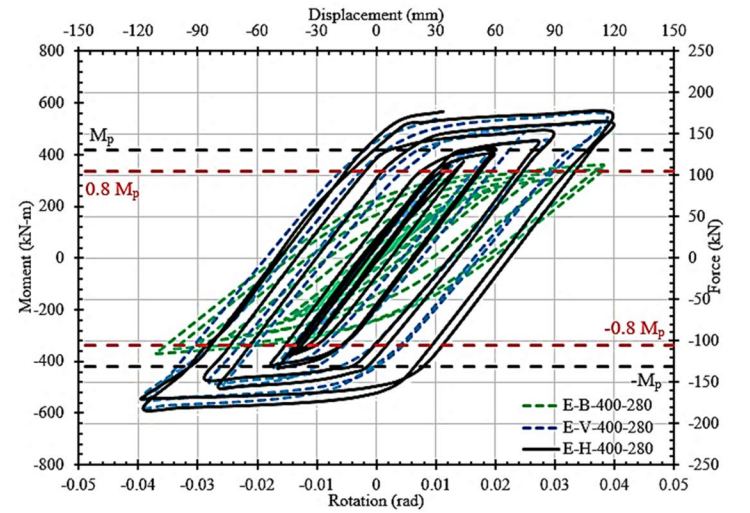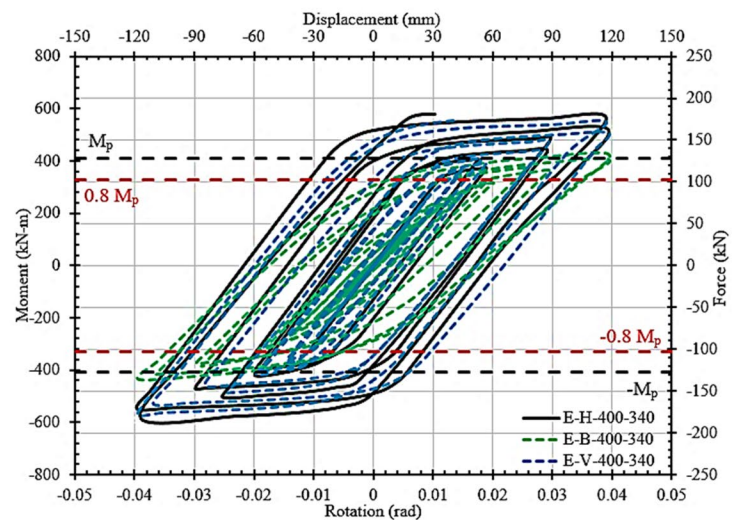
yielding in column. However, usually using stiffeners in panel zone is more cost effective.

In contrary to the bare connections, the behavior of remaining models in which stiffeners were utilized in the panel zone

Fig. 14 Hysteresis moment-rotation curves of exterior columns with column size of 400 mm×400 mm, **a** $b_{bf}=200$ mm, **b** $b_{bf}=280$ mm, **c** $b_{bf}=340$ mm
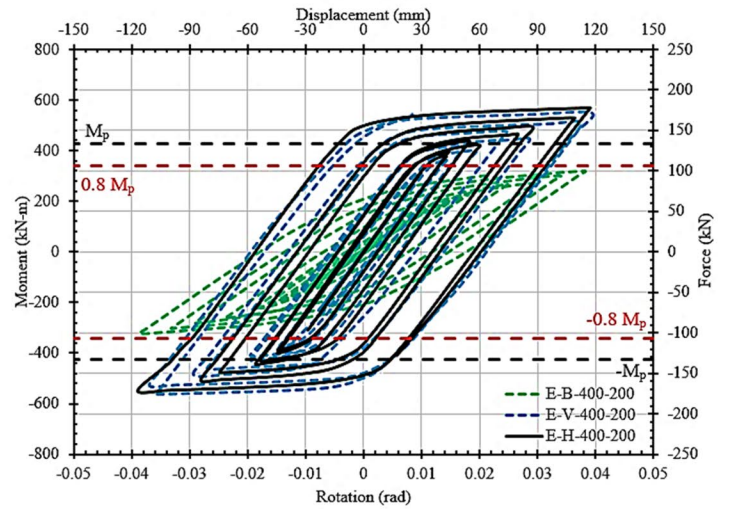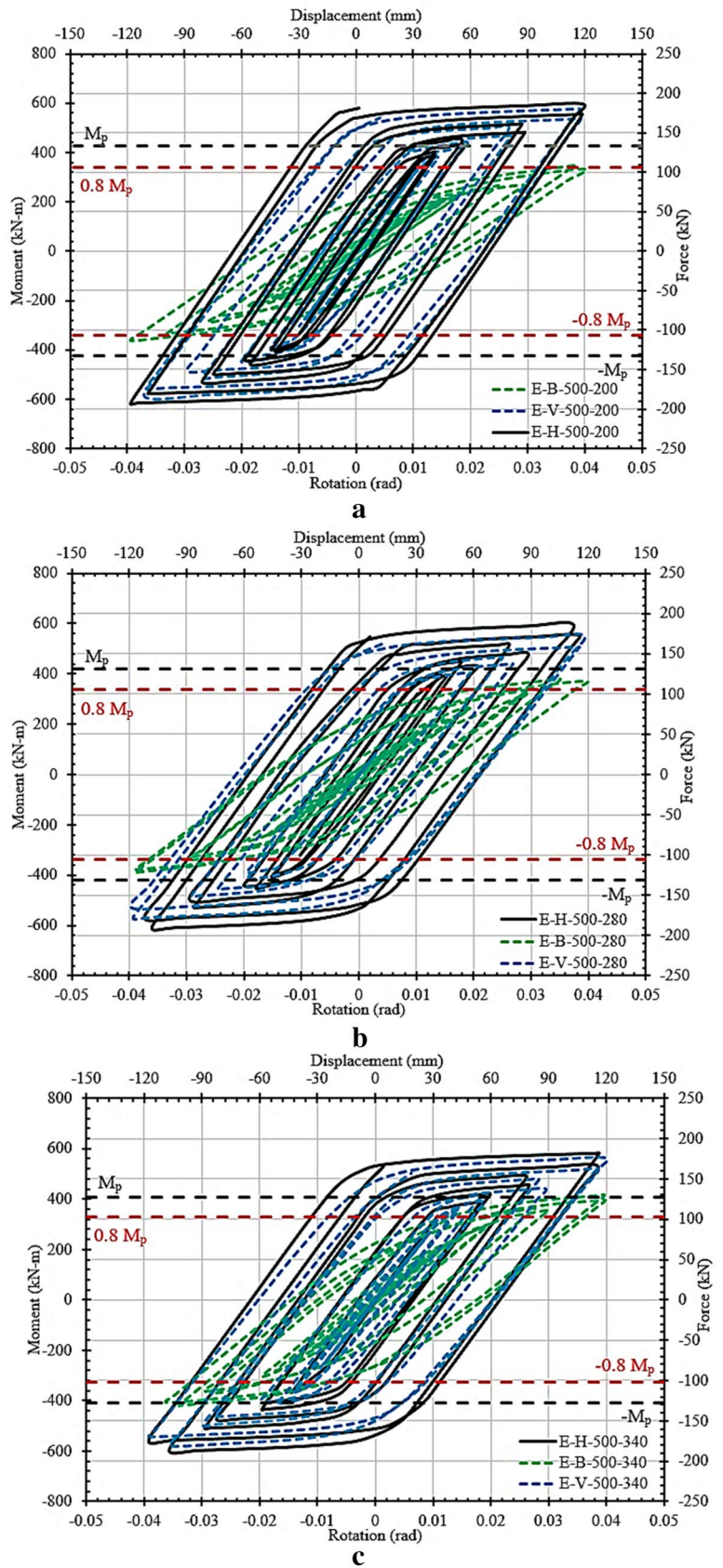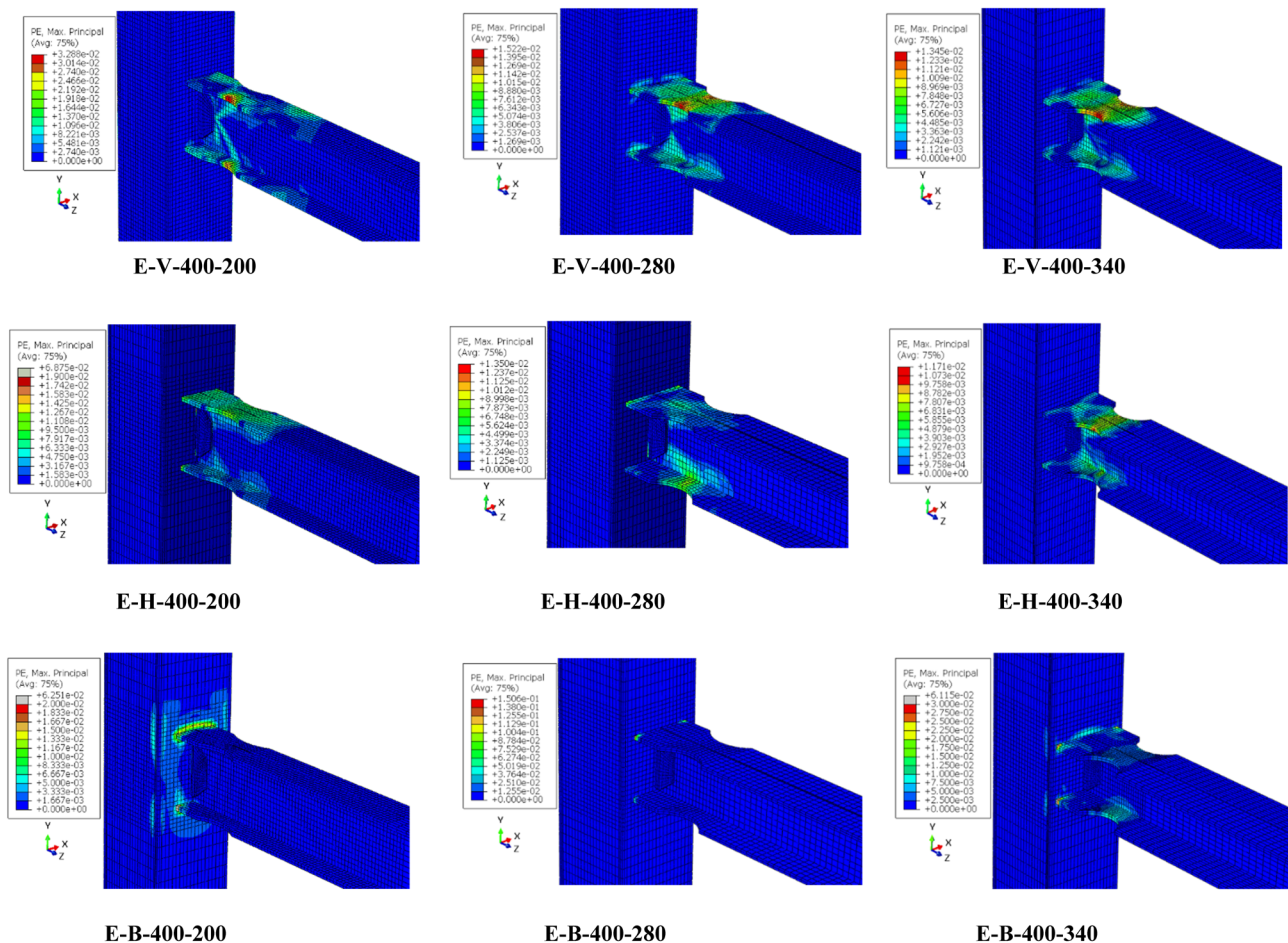

a

b

c

Fig. 15 Hysteresis moment-rotation curves of exterior columns with column size of 500 mm × 500 mm, **a** b_{bf} = 200 mm, **b** b_{bf} = 280 mm, **c** b_{bf} = 340 mm

Fig. 16 Distribution of plastic strain in models with column size of 400 mm × 400 mm
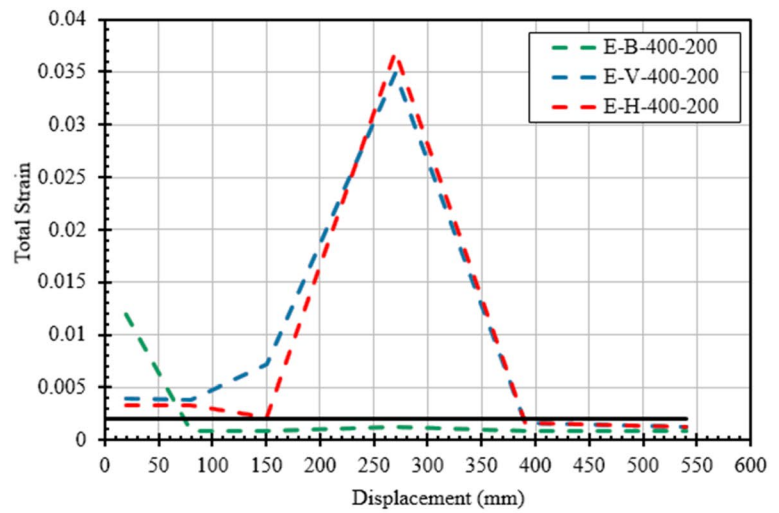
(continuity plates and vertical stiffeners) was completely different. In the connections with horizontal continuity plates, as reference models, the hysteresis moment-rotation curves reached the beam plastic strength. Moreover, the plasticity was mostly concentrated in the beam and RBS region. Such desirable behavior was also observed in models in which a vertical stiffener was utilized in the joint panel. In these models the strength of connections was higher than the plastic strength of the beam and no significant strength degradation was observed up to the rotation of 0.04 rad. Therefore, according to the criteria defined in AISC 341-16 [28], the connections with proposed detail can be utilized as moment-connections in special moment resisting frames. Furthermore, the plastic strain concentration was in the beam with a certain distance from the column face. In addition, the stiffness of the connections with stiffener was significantly higher than the bare models. This is the result of excessive out-of-plane deformations of column flange in the bare connections at early stages of loadings which reduces the rigidity of connections with no stiffener.

In order to evaluate the concentration of plastic strain in the models, the distribution of total strain is plotted along the beam
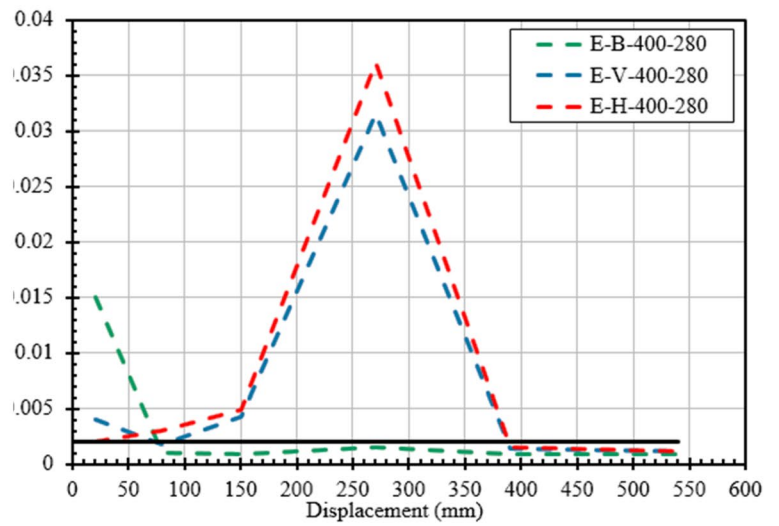
flanges in Fig. 16. Accordingly, in connections with stiffened panel zone (either with continuity plates or vertical stiffener) the plastic strain concentration is clearly located at the RBS region in the beam. While, in bare connections not only the strain in the beam is much smaller than those measured in the stiffened connections, but also the maximum strain occurred at the vicinity of column face (Fig. 17).

In order to evaluate the load transferring in the joint panel, the shear force induced in the vertical stiffener (V_s) and the column webs (V_{cw}) are compared. Accordingly, regardless of the dimensions of the beams and columns, the proportion of shear force transferred from the vertical stiffener mainly depends on the ratio of beam flange to the column flange (b_{bf}/b_{cf}), see Fig. 18. Accordingly, by increasing the b_{bf}/b_{cf} the shear force proportion in the vertical stiffener decreases following an almost linear trend. For instance, in the model E-V-500-200 (with $b_{bf}/b_{cf}=0.4$), 85% of the shear force transferred from the joint panel was induced in the vertical stiffener, while this proportion is less than 50% in model E-V-300-280 (with $b_{bf}/b_{cf}=0.93$).
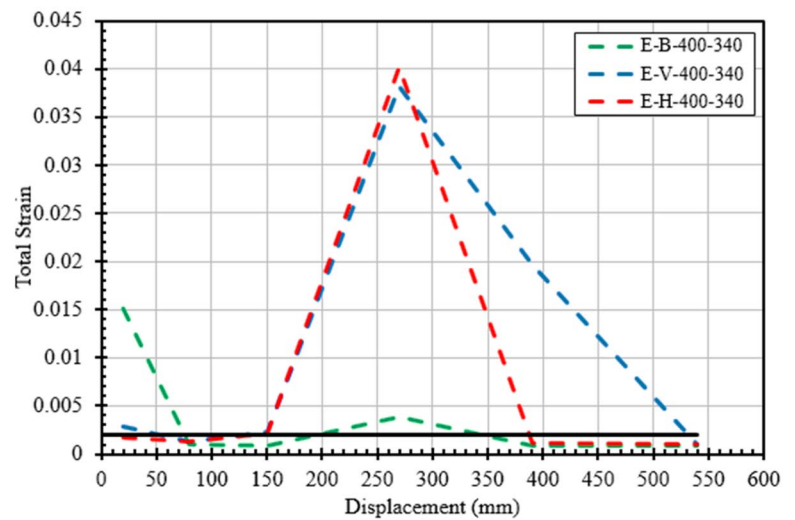
Fig. 17 Distribution of plastic strain on beam flange in models with column size of 400 mm×400 mm and beam flange width of, **a** 200 mm, **b** 280 mm, **c** 340 mm

a

b
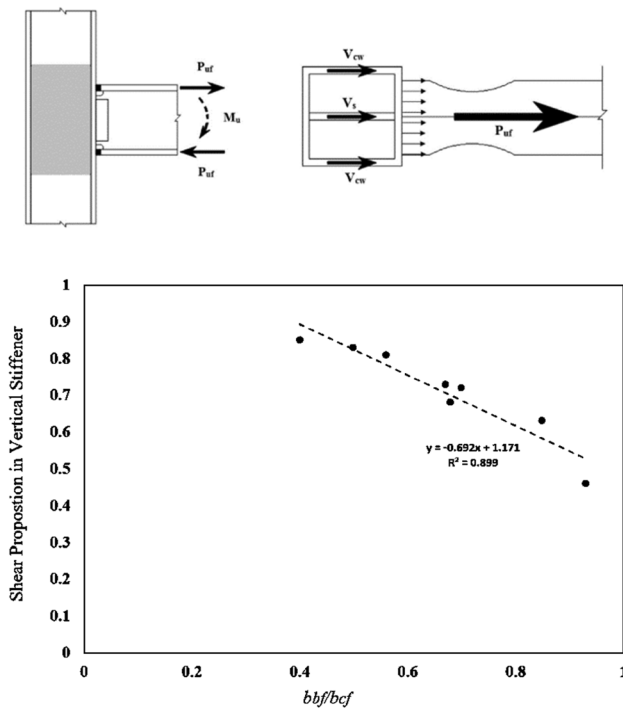
c

Fig. 19 Hysteresis moment-rotation curves of exterior columns with column size of 400 mm × 400 mm and double vertical stiffeners

Fig. 18 The variation of the shear force proportion in the vertical stiffener with b_{bf}/b_{cf}

If the b_{bf}/t_{cf} does not meet the conditions of eliminating the continuity plates, using a single vertical stiffener may lead to development of plastic strain in the column flange. In other words, the detail may fail to work appropriately. In such cases, double vertical stiffeners can be used in the panel zone, see Fig. 2b. In order to examine the effectiveness of this technique, the behavior of the model with column flange width and thickness of 400 mm and 20 mm, respectively, whose panel zone was stiffened with two vertical stiffeners was investigated (E-DV-400-280), see Fig. 19. Accordingly, the connection in which two vertical stiffeners were utilized in joint panel, show a satisfactory performance. Oppositely, in model reinforced with a single stiffener (E-SV-400-280) plasticity developed in the column, particularly at the intersection of web and flange of the box section, see Fig. 20.

Idealized bilinear curves have been found for the envelope of the hysteresis moment-rotation curves of the connections. The effective yield rotation of the connections obtained from the bilinear curves have been presented in Table 4. Accordingly, the yield rotations of the bare connections are smaller than those of models with stiffened panel zones.

3.1.2 The interior connections

Usually, in moment-frames subjected to lateral loads, the demand on the panel zone of interior connections is greater than the ex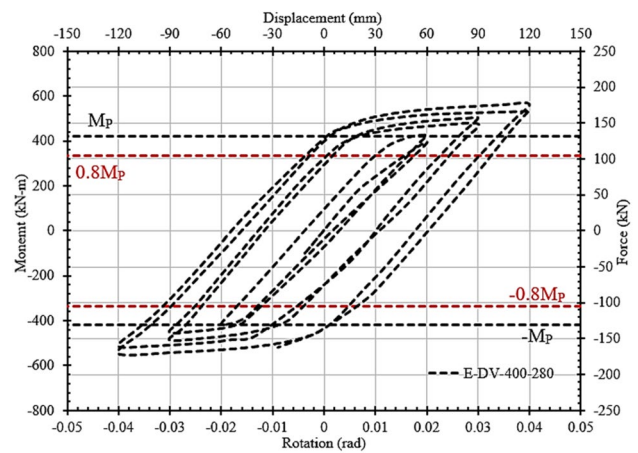terior ones. Therefore, the behavior of the panel zone in interior connections is of great importance. The hysteresis moment-rotation curves of interior connections with different stiffening details of panel zone are plotted in Figs. 21, 22, 23. The moment in these curves is that transferred from one of the beams on each side of the column and was calculated with respect to the column axis.

Similar to the exterior models, in compare to the joints with stiffened panel zone (with either vertical stiffener or continuity plates), the bare connections had the least strength and stiffness. For instance, in model I-B-300-200, the ultimate strength of the connection was around 78% of the beam plastic strength. In these models, the plasticity was concentrated in the column flange and at the intersection of column web and flange, see Fig. 24. The development of plasticity in these regions is mainly the result of the excessive out-of-plane deformations of the column flanges.

In contrast, the hysteresis moment-rotation curves of the joints with stiffened panel zone (either with conventional continuity plates or the proposed vertical stiffener) reach the plastic strength of the beams. Furthermore, except minor plastic strain developed in the panel zone of the connections with continuity plates that can be eliminated by using doubler plates or thicker web plates, the plastic strain is concentrated in the beams and RBS region with a distance from the column face, see Fig. 24. In other words, the proposed detail (using vertical stiffener) for reinforcing the panel zone can eliminate the excessive deformations in the column flange as well as plasticity in this region, appropriately. In addition, the hysteresis curves of the connections with the proposed detail of panel zone do not show significant strength degradation up to the rotation of 0.04 rad. The comparison of moment-rotation curves of connections with vertically stiffened panel zone is almost the same as those of models with conventional continuity plates. The effective yield rotation of the connections have been determined using idealized bilinear curves and are listed in Table 4.
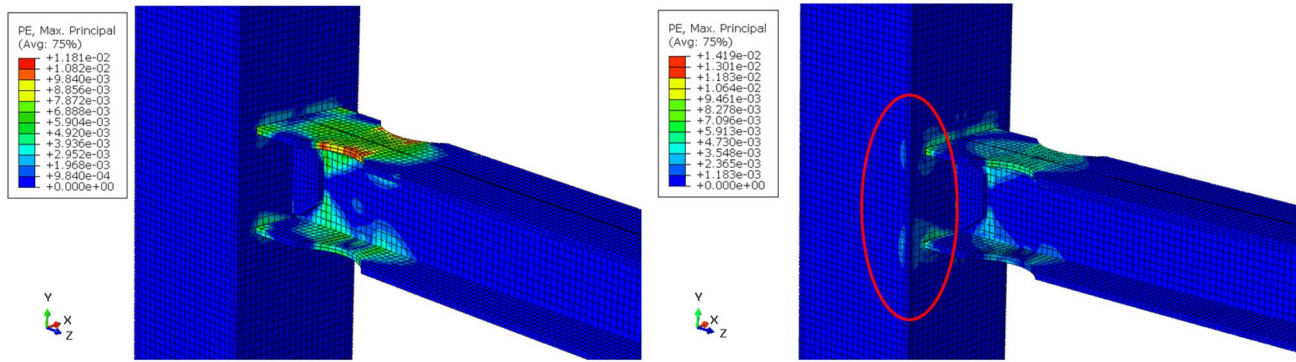
Fig. 20 Plastic strain distribution in models, **a** E-DV-400–280, **b** E-E-SV-400-280

4 Design procedure

Based on the analysis results and the AISC 341-16 [28] provisions, a simple design procedure is presented in Table 5. Accordingly, first, the minimum thicknesses of the vertical stiffener (t_s) and the column flanges (t_{cf}) are determined based on the code provisions for the panel zone of the beam to column connections with boxed-wide flange column section. Then, the proportion of the shear force in the panel zone which is transferred through the vertical stiffeners is determined. For this purpose, the diagram presented in Fig. 18 can be utilized. Finally, the thickness of the vertical stiffener is controlled for the induced shear demand.
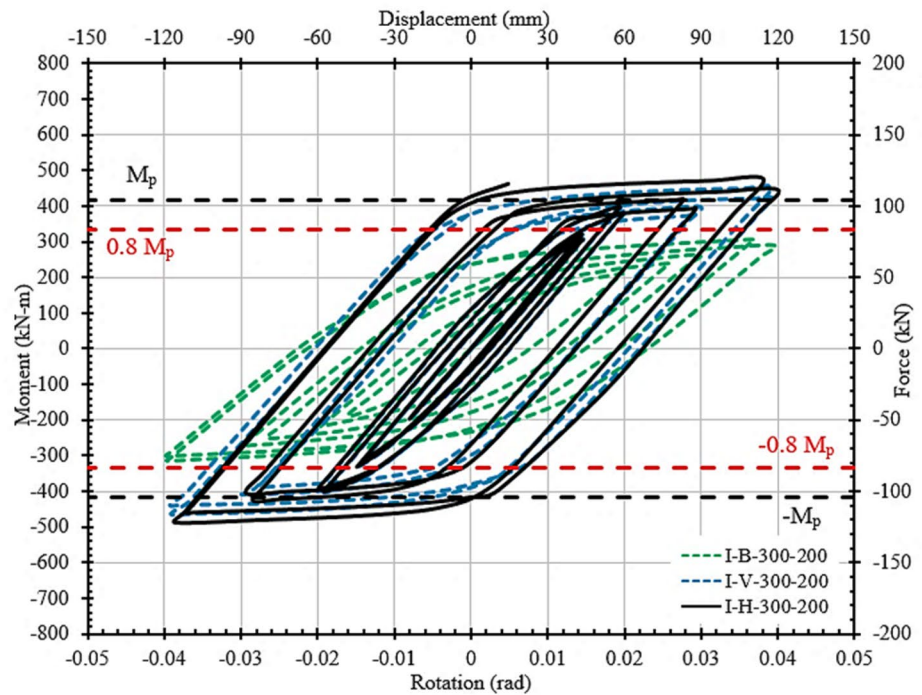
5 Conclusions

In order to evaluate the effectiveness of using vertical stiffener(s) in the panel zone of steel beam to built-up box column connections as an alternative to continuity plates, a numerical study was conducted in FE software ABAQUS. In this regard, first, the FE model was validated using two cyclic tests on an exterior connection tested before on the seismic behavior of beam to column connections. Then, the behavior of exterior and interior connections (50 models) with the proposed detail was assessed through FE analysis of models under cyclic loading. In addition, the behavior of connections with conventional continuity plates as well as those without any stiffening detail were assessed through the numerical study. Finally, a procedure for designing the panel zone of connections with the proposed detail was suggested.

The major findings of this study are as follows:

- Among models with different configurations of panel zone (exterior and interior connections), the bare connections had the least strength and stiffness. This behavior is the result of excessive out-of-plane deformations of column flange and premature local yielding of this region. Such undesirable behavior confirms the need of appropriate technique for stiffening the panel zone of connections.
- The exterior and interior connections with different beam and column dimensions, in which vertical stiffeners were utilized as alternative to the conventional continuity plates, exhibit a satisfactory behavior in terms of the hysteresis moment-rotation curves and plastic strain distribution.

Table 4 The effective yield rotation of connections

Connection	Yield rotation (rad)	Connection	Yield rotation (rad)
E-H-300-200	0.0147	I-H-300-200	0.0149
E-V-300-200	0.0149	I-V-300-200	0.0151
E-B-300-200	0.0111	I-B-300-200	0.0110
E-H-300-280	0.0146	I-H-300-280	0.0148
E-V-300-280	0.0151	I-V-300-280	0.0153
E-B-300-280	0.0105	I-B-300-280	0.0112
E-H-400-200	0.0142	I-H-400-200	0.0145
E-V-400-200	0.0161	I-V-400-200	0.0158
E-B-400-200	0.0101	I-B-400-200	0.0104
E-H-400-280	0.0145	I-H-400-280	0.0157
E-V-400-280	0.0145	I-V-400-280	0.0162
E-B-400-280	0.0099	I-B-400-280	0.0107
E-H-400-340	0.0141	I-H-400-340	0.0147
E-V-400-340	0.0148	I-V-400-340	0.0148
E-B-400-340	0.0116	I-B-400-340	0.0112
E-H-500-200	0.0139	I-H-500-200	0.0146
E-V-500-200	0.0142	I-V-500-200	0.0155
E-B-500-200	0.0115	I-B-500-200	0.0111
E-H-500-280	0.0149	I-H-500-280	0.0156
E-V-500-280	0.0201	I-V-500-280	0.0196
E-B-500-280	0.0101	I-B-500-280	0.0103
E-H-500-340	0.0190	I-H-500-340	0.0201
E-V-500-340	0.0194	I-V-500-340	0.0204
E-B-500-340	0.0113	I-B-500-340	0.0107

Fig. 21 Hysteresis moment-rotation curves of interior connections with column size of 300 mm × 300 mm, **a** b_{bf} = 200 mm, **b** b_{bf} = 280 mm


a
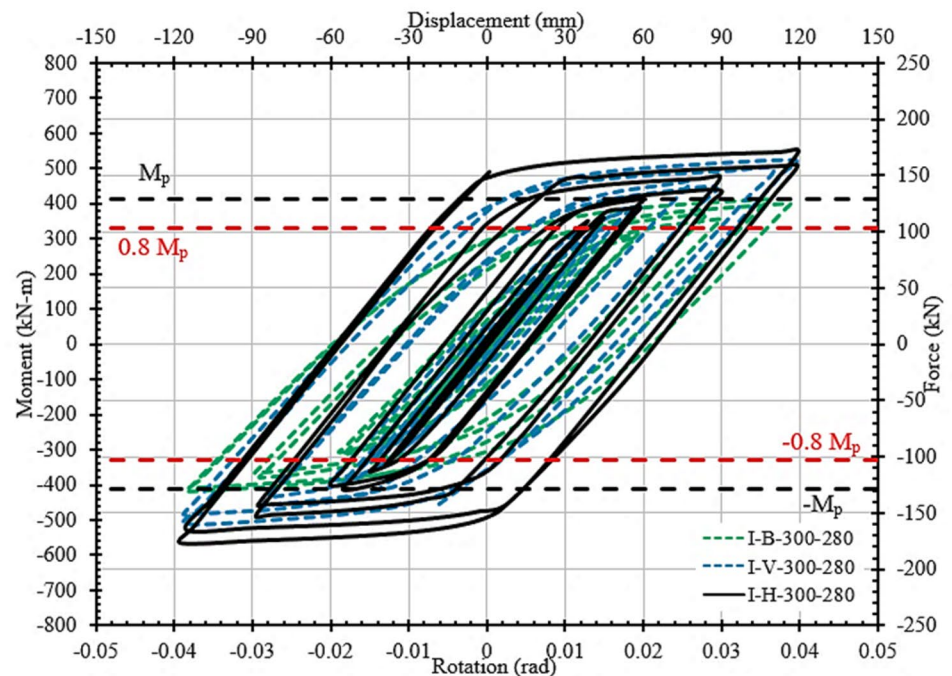
b

The moment-rotation curves of these models did not show strength degradation up to a rotation of 4.0%. Therefore, the connections with the proposed detail can be classified as special steel beam to column moment connections. Furthermore, in the joints with vertically stiffened panel zone,

the plastic hinge is concentrated in the RBS region and at a certain distance from the column face.

- In connections with thinner column flanges that the conditions of eliminating continuity plates are not satisfied, double vertical stiffeners can be used. The connections with double vertical stiffeners can be also fabricated prac-
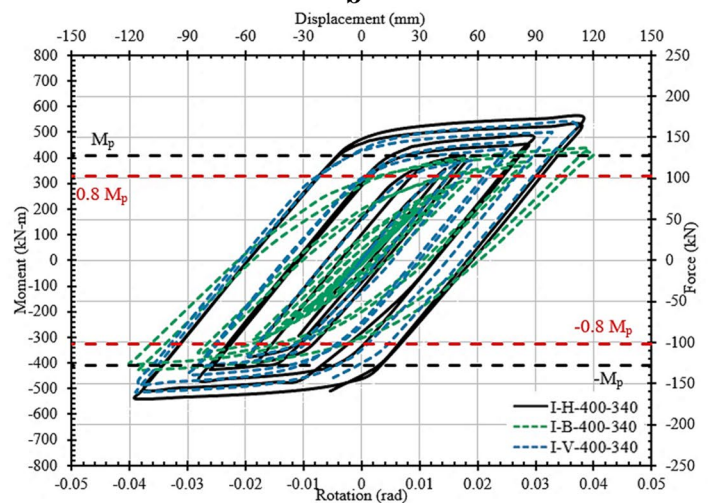
Fig. 22 Hysteresis moment-rotation curves of interior connections with column size of 400 mm×400 mm, **a** b_{bf}=200 mm, **b** b_{bf}=280 mm, **c** b_{bf}=340 mm
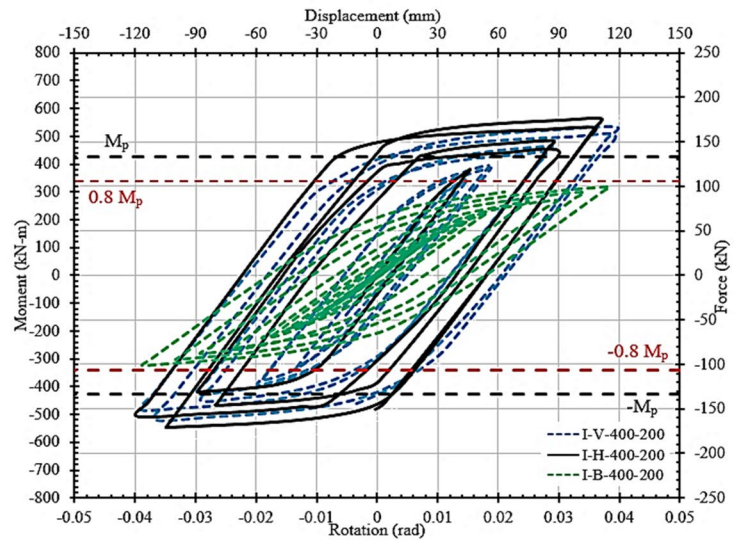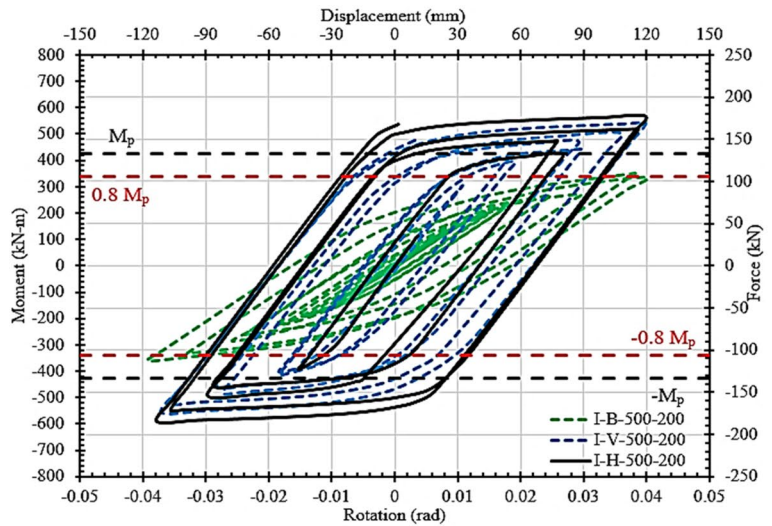

a
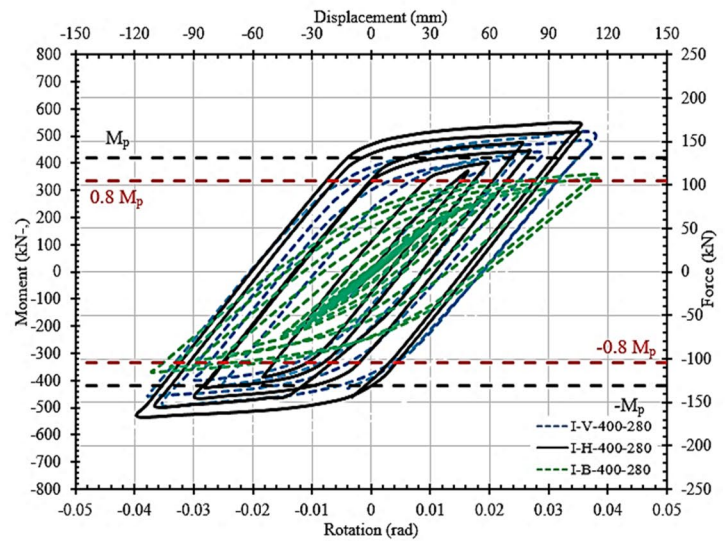
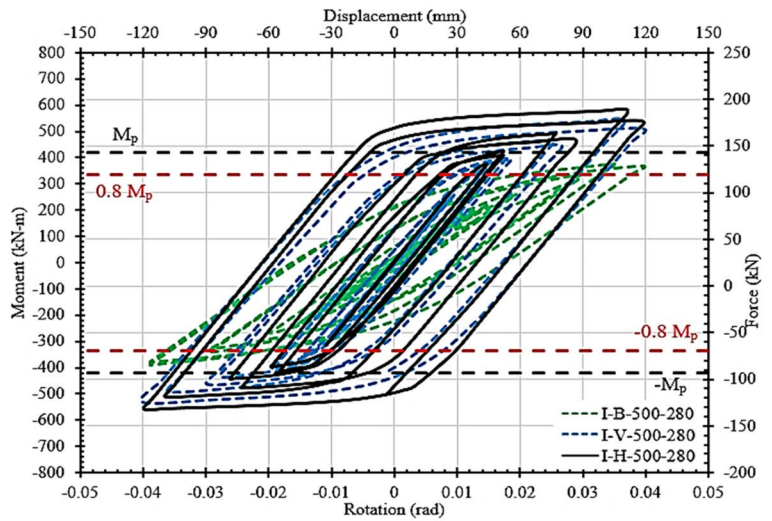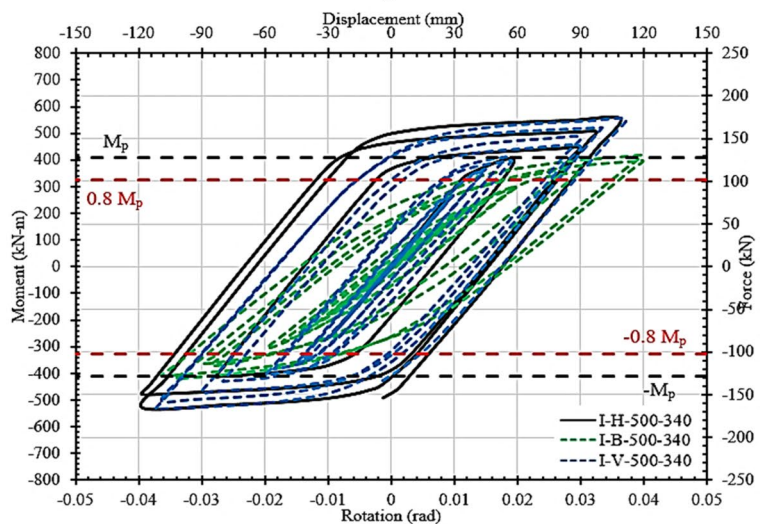

b

c

Fig. 23 Hysteresis moment-rotation curves of interior connections with column size of 500 mm×500 mm, **a** b_{bf} = 200 mm, **b** b_{bf} = 280 mm, **c** b_{bf} = 340 mm
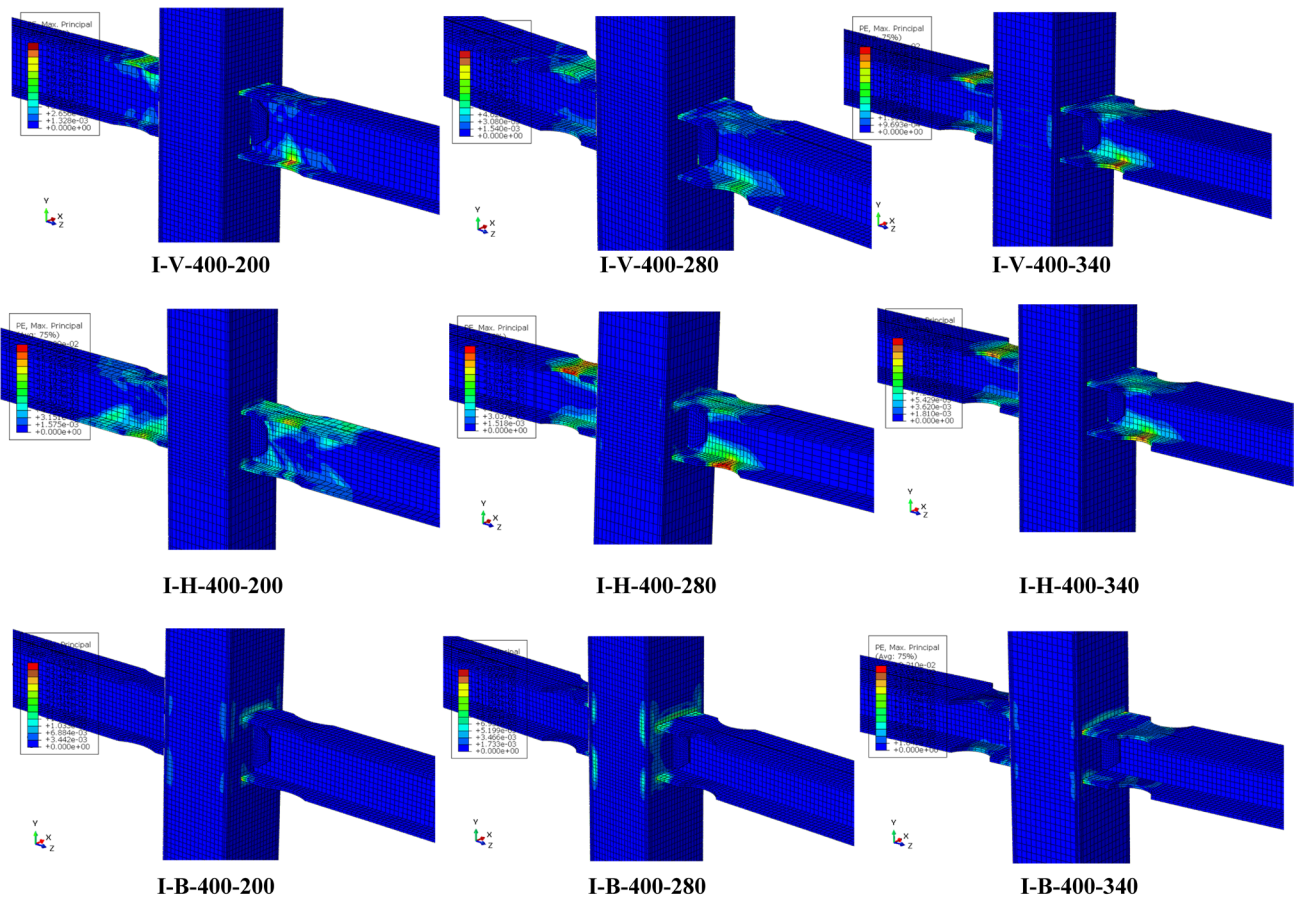

a

b

c

Fig. 24 Distribution of plastic strain in interior models with column size of 400 mm × 400 mm

Table 5 Design steps for the panel zone of connections with vertical stiffener

Design step	Action	Procedure
1	Finding the minimum thicknesses of the column flange (t_{cfmin}) and the vertical stiffener based on the formulations and procedures recommended in code for the boxed wide flange sections with no need of continuity plates	AISCE 341-16- section 6f
2	Determining the proportion of the shear force in the panel zone which is transferred through the vertical stiffener	Figure 18
3	Design the vertical stiffener for the shear demand	–

tically. Based on the results of analyses on such models, connections with double vertical stiffeners in their panel zone have acceptable seismic behavior, even with thinner column flanges.

- The proportion of the shear force induced in the vertical stiffener mainly depends on the b_{bf}/b_{cf} such that by increas-

ing this ratio, the proportion of shear force in the vertical stiffener reduces following an almost linear trend.

Declarations

Competing interests The authors declare no competing interests.

Conflict of interest The authors declare that they have no conflict of interest.

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