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Numerical investigation seismic performance of rigid skewed beam-to-column connection with reduced beam section

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Abstract. Reduced beam section (RBS) moment resisting connections are among the most economical and practical rigid steel connections developed in the aftermath of the 1994 Northridge and the 1995 Kobe earthquakes. Although the performance of RBS connection has been widely studied, this connection has not been subject to in the skewed conditions. In this study, the seismic performance of dogbone connection was investigated at different angles. The Commercial ABAQUS software was used to simulate the samples. The numerical results are first compared with experimental results to verify the accuracy. Nonlinear static analysis with von Mises yield criterion materials and the finite elements method were used to analyze the behavior of the samples The selected Hardening Strain of materials at cyclic loading and monotonic loading were kinematics and isotropic respectively The results show that in addition to reverse twisting of columns, change in beam angle relative to the central axis of the column has little impact on hysteresis response of samples. Any increase in the angle, leads to increased non-elastic resistance. As for Weak panel zone, with increase of the angle between the beam and the column, the initial submission will take place at a later time and at a larger rotation angle in the panel zone and this represents reduced amount of perpendicular force exerted on the column flange. In balanced and strong panel zones, with increase in the angle between the beam and the central axis of the column, the reduced beam section (RBS), reaches the failure limit faster and at a lower rotation angle. In connection of skewed beam, balanced panel zone, due to its good performance in disposition of plasticity process away from connection points and high energy absorption, is the best choice for panel zone. The ratio of maximum moment developed on the column was found to be within 0.84 to 1 plastic anchor point, which shows prevention of brittle fracture in connections.

Keywords: dogbone moment connection; skewed beam; reduced beam section; nonlinear analysis; cyclic loading; monotonic loading

1. Introduction

Since early 1960s, engineers began using steel moment resisting buildings, it was assumed that steel moment resisting frame connections possess appropriate ductility behavior. However, from

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the reports published prior to the Northridge earthquake, numerous cases of dissatisfaction of the connections behavior were mentioned, including the sudden failure in flange welding (Bertero *et al*. 1973, Popov and Bertero 1973), and sudden failure of moment resisting connections after experiencing repeated cyclic loading and entering the plastic deformation (Popov *et al*. 1985). The earthquake occurred on 17 January 1994 with 7.6 on the Richter scale in Los Angles, America challenged the belief of engineers and it became evident for them that the connections of steel moment resisting frames are not as ductile as was expected (FEMA-350 2000). The observation of damages to buildings of 1994 Northridge earthquake, showed that in many cases, prior to initiation of discussed behavior, brittle fracture occurs in very small amounts of plastic deformation and even in few cases, this happens when the structure is still in elastic stage. Therefore, to increase the plasticity and improve the seismic behavior of these connections, a connection with reduced beam section, named dogbone connection was proposed. The idea of this kind of connection was proposed by Engelhardt *et al*. (1998). In this connection, within a small distance from the face of column, a tapered cut is created in the beam section to reduce beam moment capacity in the region. By this operation, the plastic hinge is formed away from the beam to column connection, and occurs in the beam and prevents failure in the welded beam flange to column connection. This connection is favored because of following advantages:

- Reduced mass of structure.
- Reduced economic costs.
- Forcing plastic hinge away from the column face
- Increased strength and ductility compared to other cuts.

Following proposal of dogbone connection, extensive research has been conducted on its details. In the year 1998, investigations were made on the correct use of dogbone connection, in terms of percentage of cut, details of beam to column connection, lateral bracing of column and details of floor slab (Iwankiw and Carter 1998). In other studies, investigation is made on the optimum shape of cut for dogbane connection under cyclic loading and by providing the optimum cut-shape of the beam flange; energy dissipation capacity enhanced significantly (Ohsaki *et al*. 2009). In another study, it is shown that local instability occurs mostly in the beam web, and following it, instability occurs in the beam flange and then lateral-torsional buckling occurs in the beam (Engelhardt *et al*. 2002). In a statistical study for evaluation the stability of steel moment resisting connections, it was found that the response of plastic rotation capacity and the rate of strength reduction and slenderness parameters for buckling are dependent on the slenderness ratio and web local buckling, and are not dependent on the lateral-torsional buckling (Uang and Fan 2001). Studies have also shown that the use of stiffener in the reduced beam section region, delays local buckling of the region and results in reduced ductility under cyclic loading (Li *et al*. 2009). In dogbone connections, to promote cyclic performance, it is recommended to use backing bars in the upper beam flange connection (Uang *et al*. 2000). Also the impact of two types of loading namely, standard loading and near-fault loading are investigated in the cyclic performance of dogbone steel moment resisting connections (Uang *et al*. 2000). Chen and Tu (2004) investigated the RBS connections for large sections and high-strength steel. The results show that this connection can bear plastic rotation angle of about 0.03 rad. To connect the beam flange to the columns flange, full penetration welding is recommended. Yu and Uang (2001) examined the effects of near-fault resistant to moment connection RBS and showed that samples at a lower rotation may be buckling. Also they added a lateral bracing near the RBS area that produced more plastic rotation.

Deylami and MoslehiTabar (2005) studied the beam instability due to its reduced section and the effects of column panel zone ductility, and showed that weak panel zone has a great impact on the energy dissipation, but it undergoes failure. In strong panel zone, due to the lateral and local buckling, reduced moment capacity is observed in the beam. Zhang and Ricles (2006) performed a laboratory study on the RBS connections for columns with large wide-flange sections. The results of this study show that use of floor slab for prevention of beam displacement, would reduce the amount of lateral displacement of the beam's upper and lower flange in the RBS section, as well as column torsion and cause lower resistance during the beam instability in the RBS section. Lee and Kim (2007) experimentally examined the steel dogbone moment resisting connection with bolted beam web in the laboratory and proposed a new design procedure, called critical error design method. Engelhardt and Kim (2007) investigated non-prismatic element of beams for RBS connections in moment-resistant frames. The validity of the proposed method was evaluated by finite elements method. In this study, the effects of RBS connections on elastic stiffness of frames and beams was investigated through a series of parametric studies. The results show that for moment frame buildings with 50 percent reduction of beam flange, the maximum rotation angle of floor is between 6 and 8 percent while for 40 percent reduction in beam flange the maximum rotation angle of floor is between 4.5%-6%. Farokhi *et al*. (2010) studied the effect of the seismic behavior of structural steel moment connections. They suggested two types of failure modes in the moment resisting connection. They concluded that employing this technique, the structural reliability of the moment resisting connections shall be improved. Maliki and Tabbakhha (2012) investigated a combinatory connection named Slotted-Web-Reduced-Flange (SWRF). In this study it was shown that the connection of slotted web with the reduced flange, has better cyclic performance compared to the SBW and RBS connection. It was also shown that connection with the weak panel zone indicate wider hysteresis behavior and the maximum von Mises stress occurs in the center of the panel zone. Whan Han *et al*. (2012) reexamined the bolted web dogbone connection and showed that based on seismic design provisions ANSI/AISC358-05 the use of bolted web dogbone connection is only feasible for Intermediate Moment Frame (IMF). Also Torabian *et al*. (2012) proposed a new approach to design skewed rigid connection.

In this study, the seismic behavior of dogbone moment resisting connection under different the beam to column angles have been studied. The skewed beam connection to the column has a lot of practical problems which are yet to be scrutinized and are not even referred to in valid regulations. This connection is of great importance in steel moment-resistant frames in which connections play an essential role. This type of connection occurs in the buildings according to the following conditions:

- Structure or building is on the intersection of two streets or alleys, so the beam should be implemented skewed.
- Due to the architectural design, we have to locate a column out of the axis, so the corresponding beam is located skewed.

Numerical methods together with ABAQUS finite element software were used to study this type of connection. For doing so, first a laboratory sample was simulated in this software and the results were compared with experimental samples. After the verification of simulation stages, other samples were simulated and analyzed. In this study, the main goal was to examine the effect of skewed beam angle on the formability, strength and stiffness of dogbone connection.

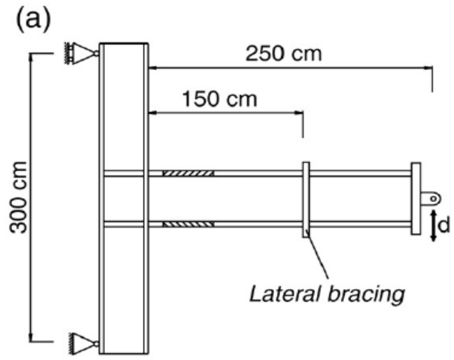


Fig. 1 Adopted dimensions for the subassembly used in the specimens (Deylami and MoslehiTabar 2005)

2. The analytical study of specimens

In this study, dogbone connection in steel moment resisting frames and under various angles by applying the effects of thickness of panel zone was studied. This study aimed to obtain the nonlinear cyclic behavior of a connection and find the moment-rotation curve as well as connection monotonic curve. In addition, the state of stress distribution and the critical stress location were investigated. In this study the nonlinear behavior of the material is considered for all analyzes. Details of subassemblies design are explained in the following.

2.1 Specimens design

For analysis, a subassembly with overall position, demonstrated in Fig. 1, is adopted (Deylami and MoslehiTabar 2005). In all specimens the central axis of the beam is taken 250 cm. The column height is also 300 cm. All subassemblies are specified with IPE300 for beam and IPB200 for column. The column is restrained by a hinge support at the base, while the other end is restrained by a roller support. Lateral bracing of beam flanges is placed at a distance of 150 cm from the face of the column. The distance satisfies seismic requirement of AISC-2010 Specifications (AISC/ANSI 341-10 2010). The adopted angles for specimens were selected as 0, 10, 15, 20 and 30°. Fifteen specimens were analyzed, each beginning with RBS1 at the start of their name. The letter after the hyphen in the naming of the specimens indicates the kind of panel zone; that W indicates weak panel zone, B indicates balanced panel zone and S shows strong panel zone. Number written at the end specifies the distance of the angle between the beam central axis and the column axis.

2.2 Panel zone variation

To evaluate the effects of the thickness of panel zone, at any angle, three type of panel zones are studied: weak (*W*), balanced (*B*) and strong (*S*). The criterion for determining the type of panel zone is based on the previous studies (Deylami and MoslehiTabar 2005). According to this criterion if $0.7<V_r/V_y<0.8$, the panel zone is balanced and less than 0.7 is weak and more than 0.8, the panel zone is strong. Accordingly a panel zone without doubler plates is called weak. To provide balanced and strong panel zones, the panel region is strengthened with doubler plates of 6

Table 1 Characteristics determining the panel zone

Specemens	Column section	Beam section	Douler Plate Thickness (mm)	V_y (KN)	V_r (KN)	V_r/V_y
RBS1-W	IPB200	IPE300	0	324	405.1	1.25
RBS1-B	IPB200	IPE300	6	496.8	405.1	0.82
RBS1-S	IPB200	IPE300	10	612	405.1	0.66

and 10 mm thicknesses. To determine the ultimate shear strength of panel zone (V_y) and required shear strength (V_r), the recommendation of seismic specifications AISC-2010 is used (AISC/ANSI 341-10 2010). Both these values are calculated from the following relationships

$$V_y = 0.6F_y d_c t_{pz}\left[1+\frac{3b_c t_{cf}^{\ 2}}{d_b d_c t_{pz}}\right] \tag{1}$$

$$V_r = \beta_E M_P\left[\frac{1}{0.95d_b} - \frac{L_b + d_c/2}{L_b}.\frac{1}{H}\right] \tag{2}$$

Where F_y=yield stress of panel zone materials, M_P=plastic moment capacity of the beam section, L_b=beam length from the face of column to the tip of the beam, H=Height of the column. Dimensions t_{cf},t_{pz},b_c,d_b,d_c correspond to column depth, the beam depth, column flange width, panel zone thickness and column flange thickness, respectively. In Eq. (2) $\beta_E M_P$ is the flexural demand imposed at the column face. The suggested value of β_E is between 0.85 and 1. Here, β_E=0.85 is used for all specimens. Characteristics of the specimens, ultimate shear strength and required shear strength are all presented in Table 1.

2.3 Continuity plate variation

For all specimens, continuity plate is used. The plate thickness is equal to the beam flange thickness and its width is equal to the width of the beam flange in the region of flange beam to column flange connection (flange width of the beam when it is metered) is considered. Also its length is considered equals to the length of column web.

2.4 Design of the RBS region

Radius-cut properties are shown in Fig. 2 for the reduced region. Details of RBS connection were designed according to the proposed recommendations of Engelhardt (Engelhardt *et al*. 1998). For all specimens, the reduction in beam flange (RBS) is equal to 40 percent. Other details are as follows

$$a = (0.5 to 0.75)b_f \Rightarrow a = 0.6b_f \tag{3}$$

$$b = (0.65 to 0.85)d \Rightarrow b = 0.75d \tag{4}$$

$$c = (0.1 to 0.25)b_f \Rightarrow c = 0.2b_f \tag{5}$$

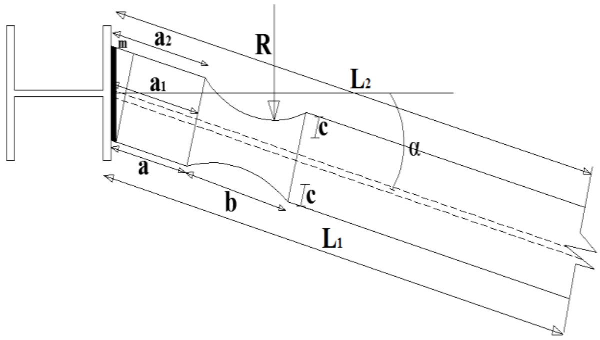


Fig. 2 Properties of cuts in the case of skewed beam

$$R = \frac{4c^2 + b^2}{8c} \tag{6}$$

Therefore, according to equations 3 to 6 and Fig. 2, the values are as follows:
bf=0.15 m, *d*=0.3, *a*=0.09 m, *b*=0.225 m, *c*=0.03 m, *R*=0.226 m.

As is observed in Fig. 2, minimum distance of cut start from the column face is considered equal to a. Using the geometrical relationships, a_1, a_2 are calculated as follows

$$\Rightarrow a_1 = a + \frac{b_f}{2}\tan\alpha \tag{7}$$

$$\Rightarrow a_2 = a + b_f \tan\alpha \tag{8}$$

In these relations b_f is beam flange width and, α represents the angle between the central axis of column and central axis of beam. This angle might range between zero and 90 degrees ($0 \leq \alpha < 90$). Other parameters are shown in Fig. 2. Considering above equations, the dimensions used are provided in the Tables 2.

3. Finite element analysis

Finite element software, ABAQUS is used to simulate specimens (ABAQUS -V10 2010). This software is appropriate for nonlinear analysis, considering large deformations. In this study, two types of analyses are performed on the specimens; the first analysis is under cyclic loading and the second one is under one-way or monotonic loading. Other details of simulation are given in the following.

3.1 Elements and meshing

A four node shell element is used for subassemblies. The type of element used in this research is S4R, where S represents the element family and means shell, 4 is the number of nodes, and R represents reduced integration in problem solving. One of the main reasons for using shell element in simulation of steel moment resisting connections is its ability to model the distortion and local

Table 2 Specifications of specimens

Specimens	Column	Beam	α (Deg)	a_1 (m)	a_2 (m)	m (m)	L_1 (m)	L_2 (m)
RBS1-W0	IPB200	IPE300	0	0.09	0.09	0	2.5	2.5
RBS1-W10	IPB200	IPE300	10	0.103	0.116	0.026	2.487	2.513
RBS1-W15	IPB200	IPE300	15	0.11	0.13	0.040	2.48	2.52
RBS1-W20	IPB200	IPE300	20	0.117	0.145	0.055	2.473	2.527
RBS1-W30	IPB200	IPE300	30	0.133	0.177	0.087	2.457	2.543
RBS1-B0	IPB200	IPE300	0	0.09	0.09	0	2.5	2.5
RBS1-B10	IPB200	IPE300	10	0.103	0.116	0.026	2.487	2.513
RBS1-B15	IPB200	IPE300	15	0.11	0.13	0.040	2.48	2.52
RBS1-B20	IPB200	IPE300	20	0.117	0.145	0.055	2.473	2.527
RBS1-B30	IPB200	IPE300	30	0.133	0.177	0.087	2.457	2.543
RBS1-S0	IPB200	IPE300	0	0.09	0.09	0	2.5	2.5
RBS1-S10	IPB200	IPE300	10	0.103	0.116	0.026	2.487	2.513
RBS1-S15	IPB200	IPE300	15	0.11	0.13	0.040	2.48	2.52
RBS1-S20	IPB200	IPE300	20	0.117	0.145	0.055	2.473	2.527
RBS1-S30	IPB200	IPE300	30	0.133	0.177	0.087	2.457	2.543

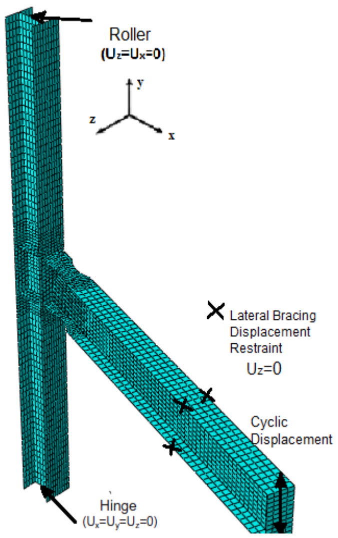


Fig. 3 Finite element mesh in the RBS region

buckling, because one major factor for failure in connections is the localized buckling. Fig. 3, shows the type of the finite element mesh used in this study. For regions adjacent to the reduced beam section, and in panel zone, denser mesh is used because the plasticity occurs mainly in this region and needs higher accuracy.

3.2 Modeling of the materials

Nonlinear material is used for steel in the analysis of the specimens. Material hardening is kinematic in cyclic loading and is isotropic in monotonic loading. In all analyzes Von Mises yield criterion is adopted for materials. In this study, structural mild steel, ST37 is used according to DIN German standards, which is equivalent to A36 in ASTM standard. Steel has yield strength of 240 MPa, the ultimate strength is 370 MPa and Poisson's ratio is taken 0.3 and Young's modulus 2.1×105 MPa. By assuming good quality full penetration welding of beam to column connection, modeling of the weld is neglected and beam-to-column connection is considered direct.

3.3 Loading protocol

To investigate the connection behavior, cyclic and monotonic loadings are used. Cyclic loading is according to the standard loading of SAC97 (Clark 1997). That are also recommended in seismic provisions of AISC-2010 (AISC/ANSI 341-10 2010). This loading is shown in Fig. 4 and its amplitude and other specifications are given in the Table 3. Common seismic criteria for special moment resisting frames state that seismic connections should be able to rotate up to 0.04 Rad. without significant loss in the capacity. Accordingly, monotonic loading is applied on these specimens. While applying loading (increasing displacement) to the free end of the beam, in order to avoid stress concentration at the tip of the beam, a 4 cm thick plate was used.

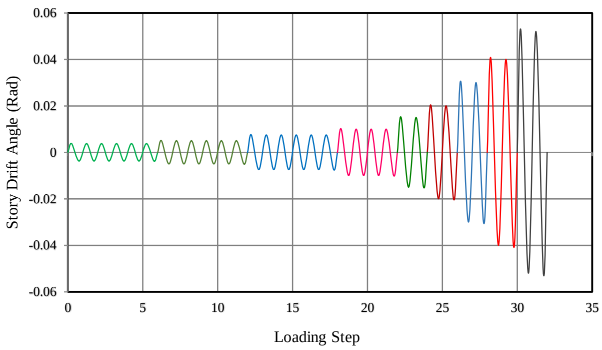


Fig. 4 The standard loading amplitudes according to SAC97

Table 3 Details of cyclic loading applied to the specimens (AISC/ANSI 341-10 2010)

loading step	Rotation angle (Rad)	Number of cycles	Displacement applied to the end the beam (cm)
1	0.00375	6	0.95
2	0.005	6	1.26
3	0.0075	6	1.89
4	0.01	4	2.53
5	0.015	2	3.79
6	0.02	2	5.05
7	0.03	2	7.58
8	0.04	2	10.1
9	0.05	2	12.63

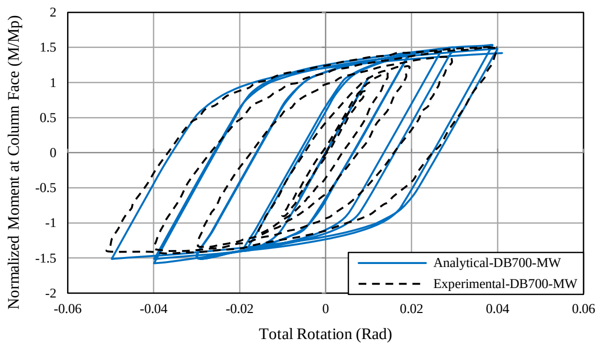


Fig. 5 Verification of analytical hysteresis diagram for specimen DB700-MW

3.4 Verification of simulation stages

To verify the simulation stages, first a number of connection specimens that had undergone tests were simulated. The purpose was to obtain a correct procedure for simulation of other specimens to confirm the accuracy of adopted element type, assigning elements, type of loading and so on in the ABAQUS software, to use this method for simulation of other specimens. For example, the moment-rotation diagram of laboratory specimen DB700-MW, used by Lee *et al*. (2005) is shown in Fig. 5, together with simulated specimens in the software.

In this laboratory sample, the beam has perfect rotational capacity and is unfractured. But increase tension is observable in the beam's reduced section and in the panel zone. Increased tension is also observable in the beam flange of the sample which is simulated by software and this tension continues along the beam web. Increased tension is also observable from the panel zone center towards its corners.

4. Results of analysis and discussion

4.1 Hysteresis response of specimens

Moment-rotation hysteresis response of the beam in subassemblies obtained from finite element analysis for all specimens is shown in Fig. 6. These curves are significantly affected by the type of materials and structural system adopted. In these curves, moment and rotation are calculated with respect to the face of the column. The moments and rotation in all specimens are obtained based on the beam central axis which has a length of 2.50 meters. Vertical axis is the normalized moment (M/M_P) and is derived, dividing moments obtained through analysis at the face of column (M) by beam plastic moment (M_P). In graphs, the horizontal line drawn is line 0.8 which according to the provisions of FEMA-350 don't represent acceptable critical conditions to investigate the strength and ductility of connection in moment frames with special ductility (FEMA-350 2000). Also this provision is given in the seismic specifications AISC-2010, which states that in moment resisting frames with special ductility, the moment at the column face and in rotation angle of 0.04 Rad , should not be less than 80% of beam plastic moment and also the

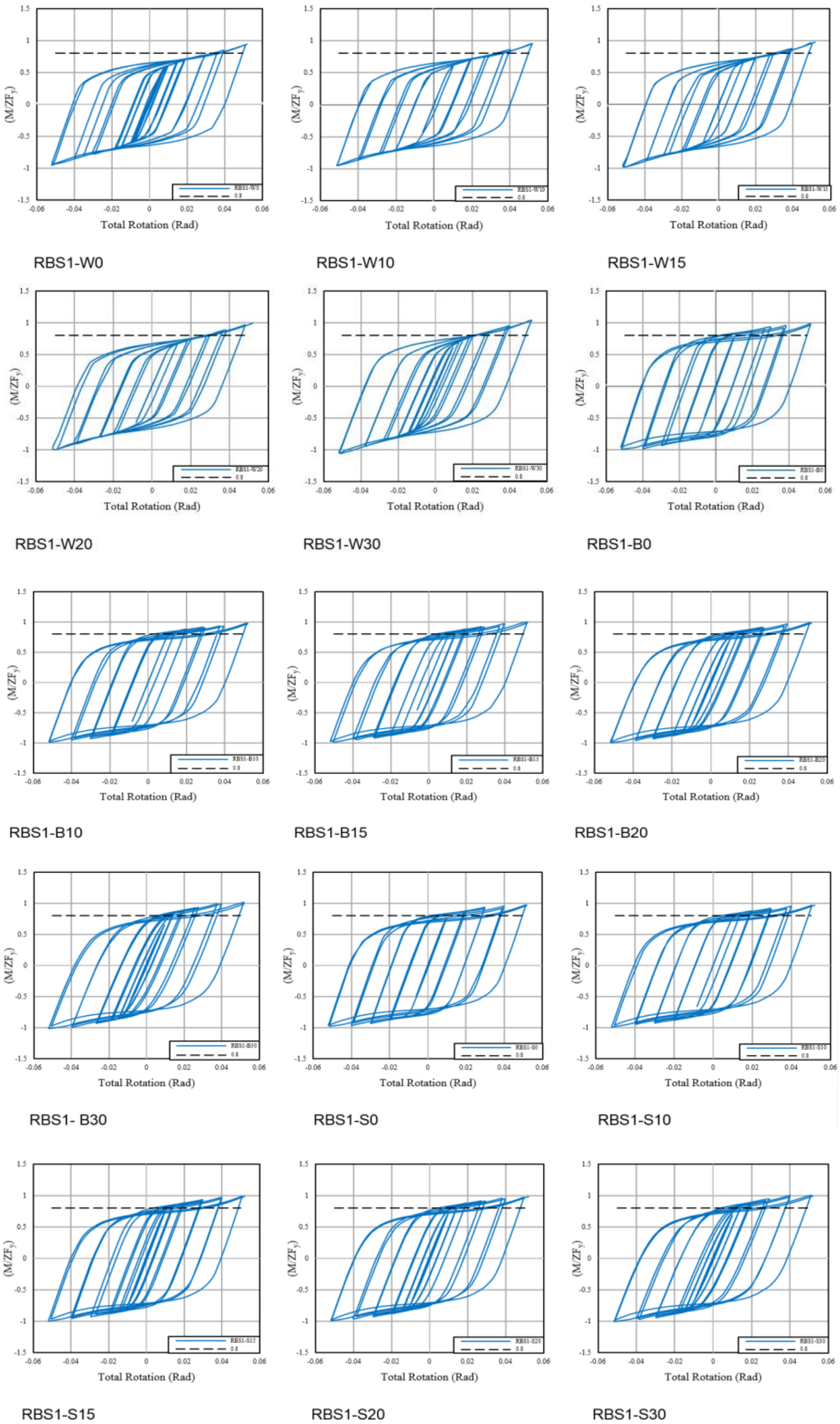


Fig. 6 Hysteresis response of the specimens

graph don't exhibit strength decline (AISC/ANSI 341-10 2010). In other words, the following should hold

$$M_{0.04} \geq 0.8M_P \tag{9}$$

As shown in Fig. 6, in specimens RBS1-W0, RBS1-W10, RBS1-W15, RBS1-W20 and RBS1-W30 in which the panel zone is weak, the moment at the column face and for the rotation angle 0.04 Rad, exceeds 80 percent of beam plastic moment and the diagram does not exhibit strength decline.

In specimens RBS1-B0, RBS1-B10, RBS1-B15, RBS1-B20, RBS1-B30, RBS1-S0, RBS1-S10, RBS1-S15, RBS1-S20 and RBS1-S30 where the panel zone is balanced and strong, the moment at the face of column and for rotation angle 0.04 Rad, exceeds 80 percent of beam plastic moment, but buckling occurs in the beam and thus diagram shows strength decline, which is negligible. Drop in the diagram and strength decline, increases by increase in angle. Deterioration in the steel structures is mainly due to localized buckling and connection brittle fracture. Roughly one can say that these specimens have the ability to be used in special steel moment resisting frames and provisions of the seismic specifications AISC-2010 concerning special steel moment resisting frames are established (AISC/ANSI 341-10 2010). However, the results show that the specimens exhibit stable and wide hysteresis behavior, and the variation in beam angle with respect to the central axis of column has little effect on hysteresis response of the specimens.

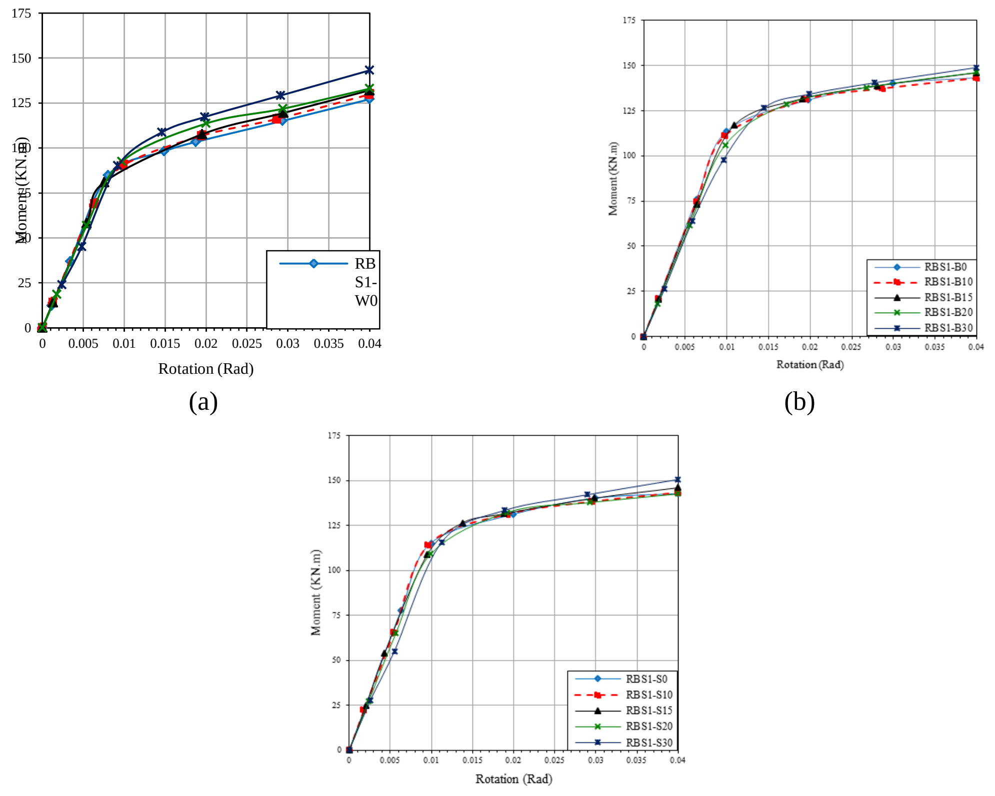
The results corresponding to moments induced in beams, and for each specimen, are provided in Table 4. Also the plastic moment of each specimen for whole section and the most reduced beam section (center of the reduced area) is given in the same table. The results for maximum moment induced in each specimen, at the face of column, and at the center of the reduced section, are compared. The ratio of maximum moment at the face of column to the plastic moment of the section is between 0.84 and 1 (commonly this value is appropriate for $0.85 \leq \alpha \leq 1$), while for Pre-Northridge earthquake connections, it has been more than 1.05. These values indicate the performance of dogbone connections in controlling extraordinary stresses at the face of column and thus preventing brittle fracture of the connection. Also the ratio of maximum moment at the reduced section to the plastic moment of this region for the specimens is between 1.07 and 1.33, which indicates formation of plastic hinges and significant energy dissipation, in these regions. Therefore respecting the above descriptions, using dogbone connection in the case of skewed beam causes a slight decrease in stiffness, but in contrast increases the reliability and ductility of the connections.

4.2 Envelope curve for specimens' hysteresis plots

For comparison of specimens together, the envelope curve for hysteresis plots up to 0.04 Rad, are plotted alongside each other and shown in Fig. 7. These envelopes are plotted according to FEMA-440 guideline (FEMA-440 2005). Regarding criteria in AISC-2010 specifications for evaluation of special moment resisting frames, there should be no strength decline in diagram for rotations up to 0.04 Rad. This assumption requires that the yield rotation be about 0.01 Rad. and for the nonlinear (plastic) region be about 0.03 Rad (AISC/ANSI 341-10 2010). According to Fig. 7, the yield rotation is about 0.01 Rad. For all specimens, Variation of angle had not any effect on the linear behavior of the system but causes increase in the strength of inelastic region. In other words, with the increase in angle, strength increases. The area under hysteresis plots shows the amount of absorbed energy in the connections and indicates that energy absorption of all

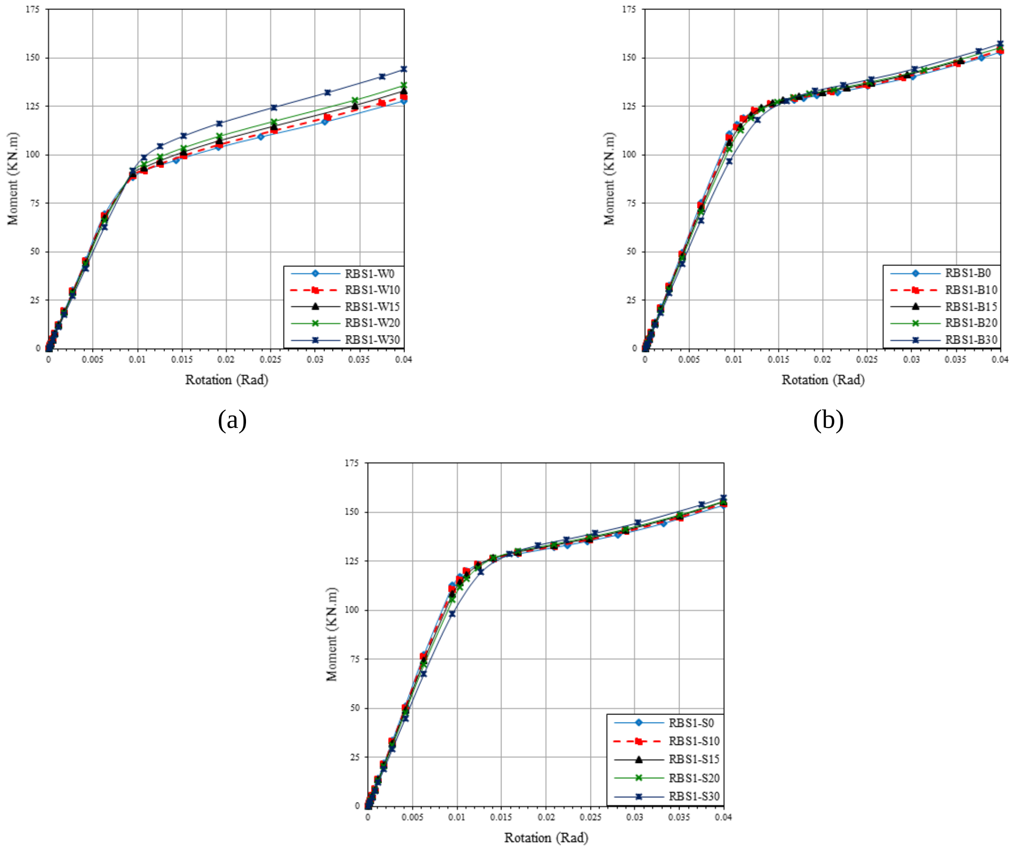
Table 4 Summary of analysis results of specimens with dogbone connection

Specimens	Beam	$M_P(KN)$	$M_{max(0.04Rad)}(KN)$	$M_{P-RBS}(KN)$	$M_{max-RBS}(KN)$	$\frac{M_{max}}{M_P}$	$\frac{M_{max-RBS}}{M_{P-RBS}}$
RBS1-W0	IPE300	150.72	127.13	108.78	116.83	0.84	**1.07**
RBS1-W10	IPE300	150.72	129.84	108.78	118.63	0.86	**1.09**
RBS1-W15	IPE300	150.72	132.08	108.78	120.32	0.88	**1.11**
RBS1-W20	IPE300	150.72	133.01	108.78	120.78	0.88	**1.11**
RBS1-W30	IPE300	150.72	143.17	108.78	129.10	0.95	**1.19**
RBS1-B0	IPE300	150.72	143.33	108.78	131.73	0.95	**1.21**
RBS1-B10	IPE300	150.72	143.25	108.78	130.88	0.95	**1.20**
RBS1-B15	IPE300	150.72	146.10	108.78	133.10	0.97	**1.22**
RBS1-B20	IPE300	150.72	146.28	108.78	132.83	0.97	**1.22**
RBS1-B30	IPE300	150.72	148.98	108.78	134.33	0.99	**1.23**
RBS1-S0	IPE300	150.72	142.86	108.78	131.29	0.95	**1.21**
RBS1-S10	IPE300	150.72	143.37	108.78	131	0.95	**1.20**
RBS1-S15	IPE300	150.72	146.10	108.78	133.09	0.97	**1.22**
RBS1-S20	IPE300	150.72	142.67	108.78	129.56	0.95	**1.19**
RBS1-S30	IPE300	150.72	150.72	108.78	135.90	1	**1.25**



(a) (b) (c)

Fig. 7 Envelope curves of hysteresis plots: (a) Specimens with weak *PZ*; (b) Specimens with balanced *PZ*; (c) Specimens with strong *PZ*



(a) (b)

(c)

Fig. 8 Response of specimens under monotonic loading: (a) weak *PZ*; (b) balanced *PZ*; (c) strong *PZ*

specimens with reduced beam flange section in the case of skewed beam is appropriate for weak, balanced and strong panel zones. Drops observed in the plots are due to buckling in the beam.

4.3 Responses of specimens under monotonic loading

Current seismic provisions state that seismic connections should have the ability to rotate to 0.04 Rad. Without significant loss in their capacity (AISC/ANSI 341-10 2010). Therefore applied loadings are in both cyclic and monotonic patterns. The monotonic loading is applied on the subassembly till it rotates up to 0.04 Rad, so a displacement of 10 cm magnitude is applied to the free end of the beam. Depending on the type of loading (monotonic), material hardening in elastic and plastic region is taken isotropic. Results of monotonic loading are shown in Fig. 8. As is observed, these diagrams are similar to hysteresis plots which indicate the accuracy of simulation procedures and finite element analyses. For the analysis of specimens under monotonic loading, the criterion of special moment resisting frame is met and response of specimens in the case of skewed beam are evaluated to be suitable for the weak, balanced and strong panel zones.

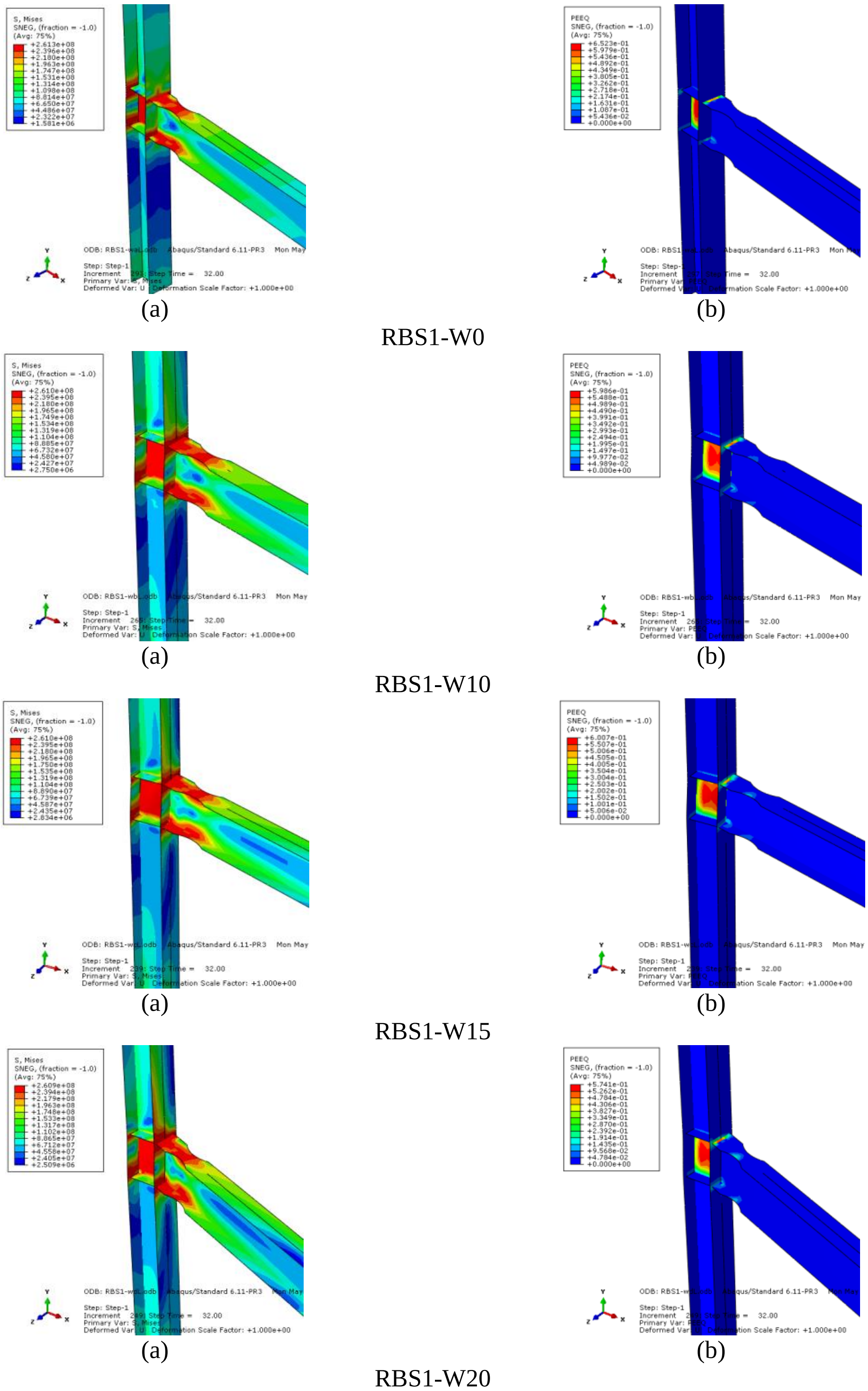
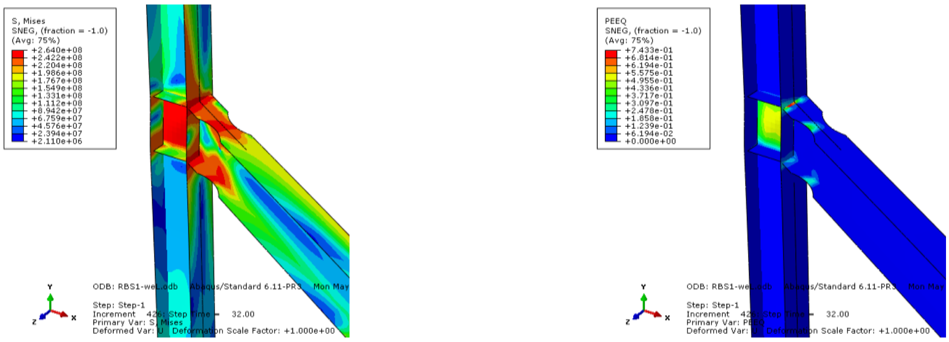


Fig. 9 Distribution of stress and strain in last cycle of loading for specimens with weak panel zone: (a) von Mises stress; (b) equivalent plastic strain



(a) (b)

RBS1-W30

Fig. 9 Continued

4.4 Stress distribution

In special steel moment resisting frames, beams are considered as seismic energy absorbing members that achieve this task by nonlinear behavior in the plastic hinge. In all specimens, except for specimens with weak panel zone, plastic hinge in the connection, locates within reduced beam section and in a desired distance away from the face of column. This means, it is unlikely that sudden brittle fracture occurs in the connection and also exhibits appropriate behavior, in the case of skewed beam. In the Figs. 9 to 11 Von Mises stress contours and equivalent plastic strain, are shown for the last cycle of loading of the specimens under study. The reason for adopting Von Mises criterion is that it clearly shows the yield and ultimate fracture of soft materials such as mild steel .In continuation, different types of panel zone are described.

4.4.1 Weak panel zone

Von Mises stress distribution and equivalent plastic strain for weak panel zone are shown in Fig. 9. Due to stress expansion within weak panel zone, it was observed that the initial yield occurs in the center of panel zone and the yield expands toward the corners, and panel zone deforms to a parallelogram. Then, the whole the panel zone yields completely and the initial yield occurs in the beam and at a region adjacent to beam flange to column connection, where the penetration weld is located. Thus the beam never reaches its flexural capacity, in other words connection fails before failure of the beam. For specimens with angles of 10, 15, 20 and 30 degrees, the initial yield in beam occurs simultaneously with penetration welding and within reduced section. If the design is such that the panel zone dissipates all the incoming force of a large earthquake, It should be able to bear very large inelastic shear deformations and this leads not in the undesirable behavior in the panel zone, but creates many difficulties in the welded parts, because very large shear deformation induce high level strains in the corner of the panel zone and also in the region of beam flange to column flange connection. It is observed that in using weak panel zone despite high energy dissipation and ductility, due to the high shear deformation and plasticity in the panel zone, the risk of failure in the welding of beam flange to column connection, is very high. This issue has already been seen in (Deylami and MoslehiTabar 2005, Lee *et al*. 2005, Lee *et al*. 2004) and here is confirmed for the skewed beam as well.

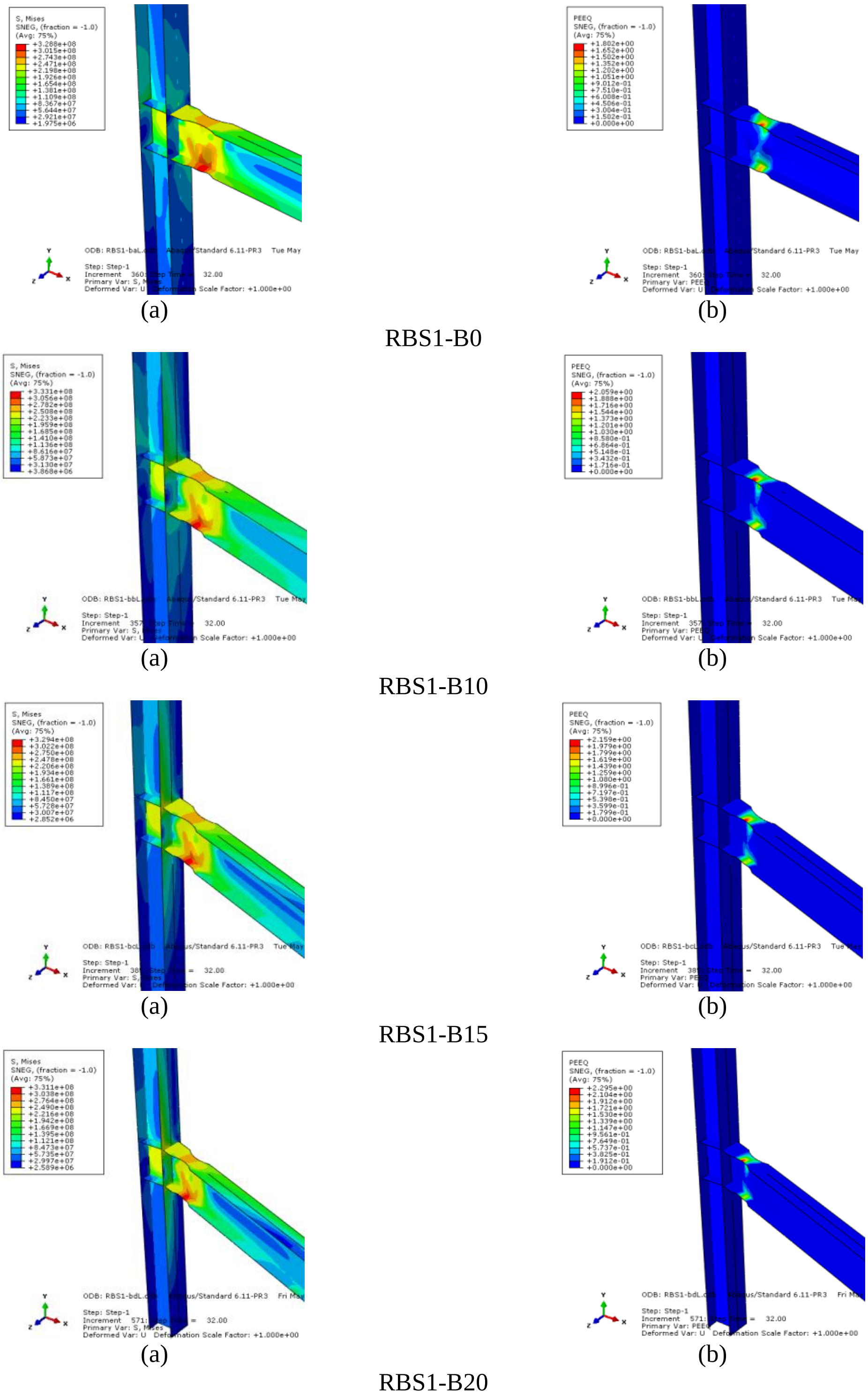
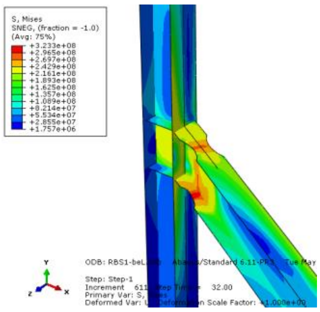
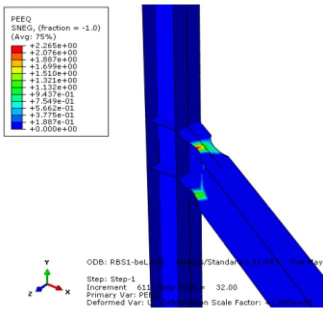


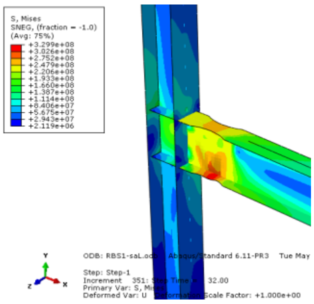
Fig. 10 Distribution of stress and strain in last cycle of loading for specimens with balanced panel zone: (a) von Mises stress; (b) equivalent plastic strain

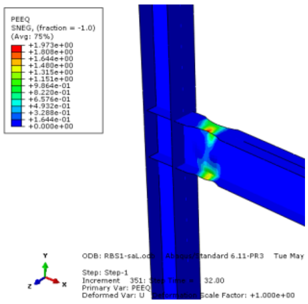

(a)


(b)

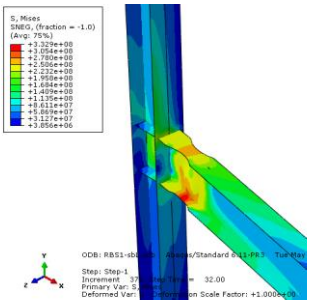
RBS1-B30

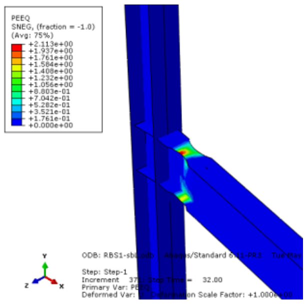
Fig. 10 Continued


(a)

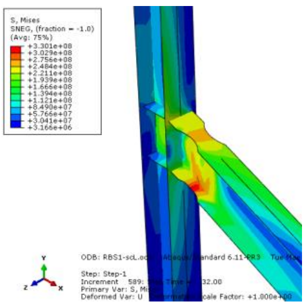

(b)

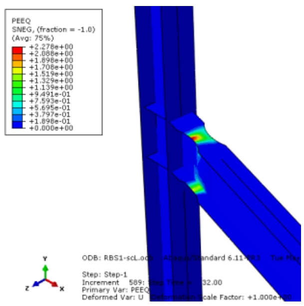
RBS1-S0


(a)


(b)

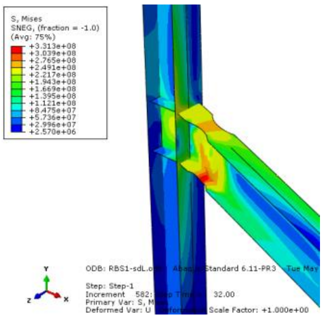
RBS1-S10

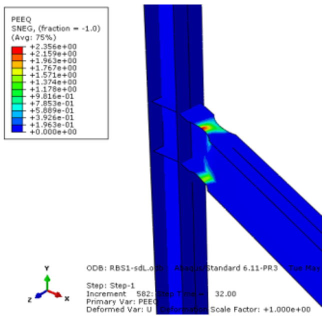

(a)


(b)

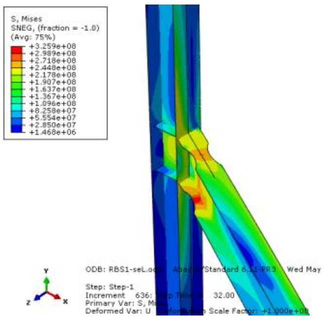
RBS1-S15

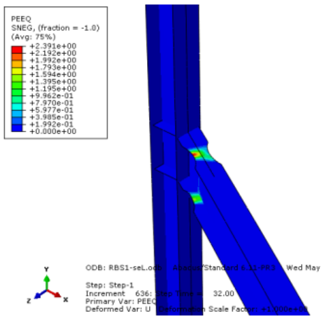
Fig. 11 Distribution of stress and strain in the last cycle of loading for specimens with strong panel zone: (a) von Mises stress; (b) equivalent plastic strain


(a)


(b)

RBS1-S20


(a)


(b)

RBS1-S30

Fig. 11 Continued

4.4.2 Balanced panel zone

Von Mises stress and equivalent plastic strain for balanced panel zone are shown in Fig. 10. Due to stress expansion within specimens of a balanced panel zone, the initial yield took place in beam and in reduced beam section (RBS). No yield is observed in the panel zone of these specimens, and it remained in elastic state, so columns web stiffener has great impact on the rigidity of RBS. Previous studies have also revealed that the best connection behavior is when the capacity of the panel zone is almost identical with the beam moment capacity, or at least the capacity of panel zone is not less than beam plastic moment capacity (Deylami and MoslehiTabar 2005).

4.4.3 Strong panel zone

For all specimens with strong panel zone also, the initial yield took place in beam and in the reduced beam section (RBS). In these specimens too, no yield was observed in panel zone and panel zone remained in elastic state. In strong panel zone, the probability of fracture decreases and the possibility of instability increases. In other words, beam undergoes failure sooner. Therefore the difference between theses specimens with balanced panel zone, is that due to more rigidity and less ductility of strong panel zone, the beam undergoes failure in lower levels of rotations.

The remarkable point observed in the skewed beam connection is that the yield in reduced section starts from one side, while in direct moment resisting connection of beam to column under zero angle, the yield starts from both sides simultaneously.

As described above, for specimens with weak panel zone, the initial failure occurs in the center

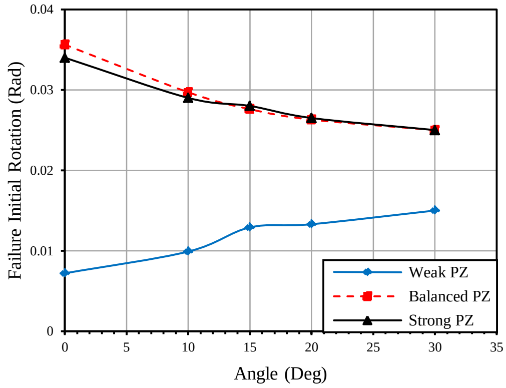


Fig. 12 Rotation angle corresponding to initial failure in the specimens

of panel zone and for balanced and strong panel zones; the initial failure occurs in beam and in reduced cross section. Accordingly, diagram which shows the relationship of angle between beam and central axis of column, and the level of rotation corresponding to initial failure, is presented in Fig. 12. As can be seen, for weak panel zone, the more the angle between the beam and column increases, the initial failure occurs later and in higher level of rotation angle within the panel zone, which indicates that by increasing the angle between beam and column, the acting force decreases in the direction perpendicular to the column flange and resolves according to the beam angle. However, in balanced and strong panel zones, by increasing the angle between beam and central axis of column, the beam failures more rapidly in the reduced area and in lower rotation angle, so according to the above description as well as basic assumptions in design based on capacity, as shown in Fig. 12, if the panel zone is balanced, failure strength of panel zone and beam enhances, also participates adequately in energy decapitation and plastic rotation, along with it prevents severe stress concentration at the face of column, and welding of beam flange-to-column connection, therefore in the case of skewed beam connection, balanced panel zone has the most appropriate performance.

With increasing angle, higher stresses are induced in the left part of column flange, which indicates twist occurrence in the column. Due to continuity plates within column section at the level of beam flanges, beam stresses which are not significant, are not developed inside the column (away from the panel zone), but are directly transferred to the panel zone through the continuity plates. As a result columns outside the panel zone exhibit elastic behavior.

5. Conclusions

In this article, the impact of the angle between beam and column in the horizontal plane is investigated for the dogbone moment resistant connection behavior, and under both cyclic and monotonic loadings. Also the impact of thickness of panel zone for the case of skewed beam connection is investigated and results of various types of panel zones are compared. Considering studies done on the dogbone connection in the case of skewed beams, the most important results are presented here:

- For dogbone skewed beam-to-column connection, acceptable critical conditions provisions, according to seismic specifications AISC-2010 for connection ductility in special moment resisting frames, are established and variation in angle between beam and central axis of column have negligible impact on the hysteresis response of the specimens.
- For skewed beam to column connection, the most appropriate ratio of maximum moment at the face of column to the plastic moment of section $0.85 \leq \alpha \leq 1$. This value for the specimens was between 0.84 and 1, which indicates proper performance of dogbone connections in controlling excessive stresses at the face of column, and thus preventing brittle fracture of the connection. Also the ratio between maximum moment in the reduced section to the plastic moment of this region for the specimens ranges between 1.07 and 1.33, which indicates formation of plastic hinge in these regions and significant energy dissipation.
- Variation in the angle has no impact on linear behavior of system, but causes increase in strength in the inelastic region; this is established for both cyclic and monotonic loadings.
- In the case of skewed beam, specimens with weak panel zone exhibit stable and wide hysteresis response, but due to large shear deformations in the panel zone, extensive strains are induced in the beam flange connection region, which despite energy dissipation and high levels of ductility, due to large shear deformations and high plasticity in the panel zone, the risk of weld fracture of beam to column connection is high.
- In specimens with balanced and strong panel zone, the place of plastic hinge formation is within reduced section of beam and in a desirable distance from the face of the column. It means that it is unlikely that sudden brittle fracture occurs in the connection, and for skewed case, the connection also exhibits proper behavior. In case of skewed beam connection, balanced panel zone, is the best kind of panel zone, due to its highly acceptable performance in forcing away plasticity from the connection region and high levels of energy absorption.
- In skewed beam connection, failure initiates from one side in the reduced section, RBS. While for beam to column moment resisting connection, under zero angles, yield initiates from both sides and simultaneously.
- For weak panel zone, by increasing the angle between beam and column, the initial yield occurs later and in larger rotation angle within the panel zone, which indicates that with increasing the angle between beam and column, the magnitude of acting force perpendicular to column flange decreases. In balanced and strong panel zones, with increase in angle between beam and central axis of column, the reduced section of beam yields faster and in smaller rotation angle.
- In columns, within beam flange-to-column connection region, higher levels of stresses are induced in the left column flange, which indicates twist creation in the column.

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