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Analytical Evaluation of RBS Moment Connection for Indian Profiles Under Seismic Behavior

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Abstract: The effects of Northridge (1994) and Kobe (1995) earthquake, intensive analysis and testing efforts are currently being sought for higher ways of designing seismic resistant steel connections. A connection with RBS (Reduced beam Section) is an acceptable solution to emphasizing a moment connection and can be the most reliable type of connection. Considering the benefits of RBS connection and the lack of information on resources in Indian standards, an analytical study of RBS connections of Indian profiles is performed. This study elaborates on Indian profiles to understand the beam-column connection's overall performance characteristics during the cyclic response. FEA study is carried out to focus on improving the plastic deformation potential of the connection. The moment capacity of the connection is strengthened by the inclusion of RBS region in the beam. The seven specimens which includes Specimen without RBS, Flange cut RBS and Flange hole RBS have been designed and evaluated using the Finite element ABAQUS program. The findings of this study revealed that specimen without RBS exhibits low Seismic activity efficiency. While RBS specimens performed well, in all instances, Flange holes' specimens demonstrate good performance.

Keywords: Reduced beam section connection, plastic hinge, cyclic loading, finite element analysis, hysteresis behavior, PEEQ

1. Introduction

Steel moment-resisting frames (SMRF) are typically utilized in the areas that are probably littered with the seismic activity. In Steel Moment Resisting Frames, beam and column are rigidly connected with zero releases to the connections once analyzed structurally. The various affiliation strategies to accomplish these connections have evolved over the decades. Even though the techniques have improved the bolt and weld strength, the weld is considered as suitable for the moment connections.

The consequences of Northridge's (1994) and Kobe's (1995) earthquake, intensive analysis and testing efforts are current to seek out higher ways to design seismic resistant steel connections. The column and connections are to be strengthened or the beam section should be weakened are the major interpretations realized to produce extremely ductile and lasting performance. This realization reduced the damage of an individual column. To enrich the poor moment connection, the plastic hinges should form either in beams or in respective connections, however not within the column. This perception leads to a Strong Column-Weak Beam frame.

A biased failure at a beam doesn't directly result in a beam collapsing, though if it collapses, it may also be limited to this beam alone. Furthermore, for the entire frame system, a limited failure in an exceedingly column would be

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disastrous. The beam section can be weakened by reducing the use of heavier column sections to attain a stronger column and weaker beam frame. The section of the beam is weakened by cutting or trimming the sections at prescribed locations where the plastic hinges have to form. Though, reinforcing the connection will increase its value, and conjointly new issues will arise because of the necessity for terribly massive welds and better degrees of restraint if excessive reinforcement is employed.

The RBS (Reduced Beam Section) connection is an alternative to strengthening a moment connection that has equal advantages to additional reinforcing, eliminating a variety of drawbacks. Through cutting the part of the beam off out of the column is probably the best approach to alter the performance of the traditional moment connection [1-4]. The collapse is limited to beam because the beam portion is minimized in the plastic hinge area. Several experiments have been carried out to examine key problems concerning the strength of RBS beams. A new criterion for lateral slenderness is well in agreement with the experimental outcomes. RBS beams' periodic activity is often caused by their lateral uncertainty and slenderness ratio [5]. Within the reduced beam section, yielding will be limited as it acts as a ductile fuse [2,6]. Enormous research studies have been carried out to show this solution's efficacy [7-8]. In some situations, the reduced beam section decreases the frame's stability, resulting in marginal beam section improvement.

After the Northridge earthquake, The RBS connection may be the most affordable form of connection [9]. Since the moment connection with RBS is thoroughly examined and utilized in the American and European countries, although the researches and implementation of RBS connection are very limited to Indian sections. Thus, Indian steel design codes don't elaborate on RBS. It can be implemented in India to achieve greater performance in intense and moderate seismic areas. With the benefits of the RBS concept and the unavailability of asset information in Indian Standards, the comparative study of RBS connections for Indian profiles shall be performed. The present study is to obtain numerical modeling results on connections with and without RBS. The main goals include evaluating all types of RBS connections on ductility, testing the impact of RBS connections on elastic and plastic behavior.

2. Design of Specimens

The study focused on the Indian profiles to recognize the overall performance characteristics of the beam-column connection under cyclic behavior. Seven specimen types were analyzed: Section without RBS, Radius cut RBS, Straight cut RBS, Tapered cut RBS, Single row Same holes RBS, Two-row Same holes RBS and Varied Holes RBS. The specimens along with their specifications are noted in Table 1. ISMB 350 has been used as the column and ISMB 300 has been used as a beam for each type. The properties of Indian sections as per IS 800: 2007 [10] are listed in Table 2 and respective drawings with an isometric view of the Column-Beam Section are shown in Figure 1, plan and elevation in Figure 2.

Table 1 - Specifications of specimens

Specimen	Column	Beam	Description
AIWR-01	ISHB350	ISMB300	Section without RBS
AIRR-02	ISHB350	ISMB300	Radius cut RBS
AISR-03	ISHB350	ISMB300	Straight cut RBS
AITR-04	ISHB350	ISMB300	Tapered cut RBS
AISS-05	ISHB350	ISMB300	Single row Same holes RBS
AITS-06	ISHB350	ISMB300	Two rows Same holes RBS
AIVH-07	ISHB350	ISMB300	Varied Holes RBS

Table 2 - Properties of Indian sections

Member	Length mm	d mm	b mm	Flange Thick mm	Web Thick mm
ISHB350-Column	1000	350	250	11.6	10.1
ISMB300-Beam	825	300	140	12.4	7.5
Continuity plate provided as same thick of beam flange					

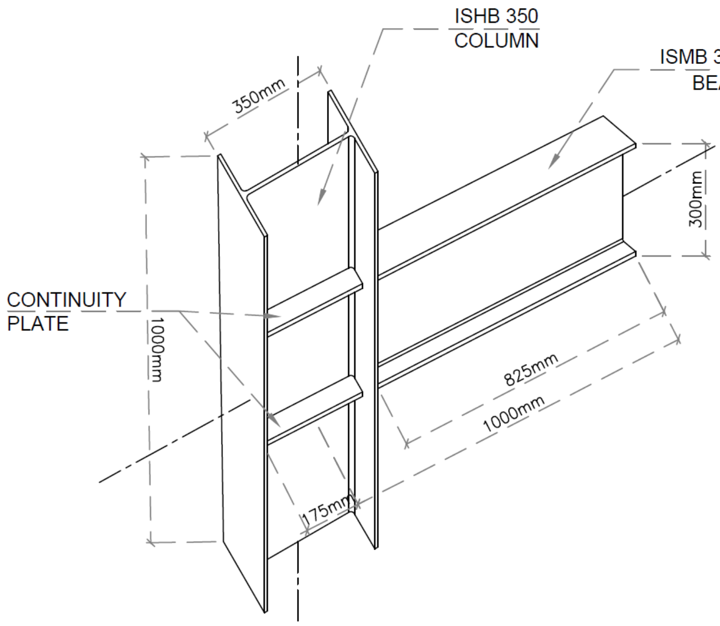


Fig. 1 - Isometric view of column- beam section

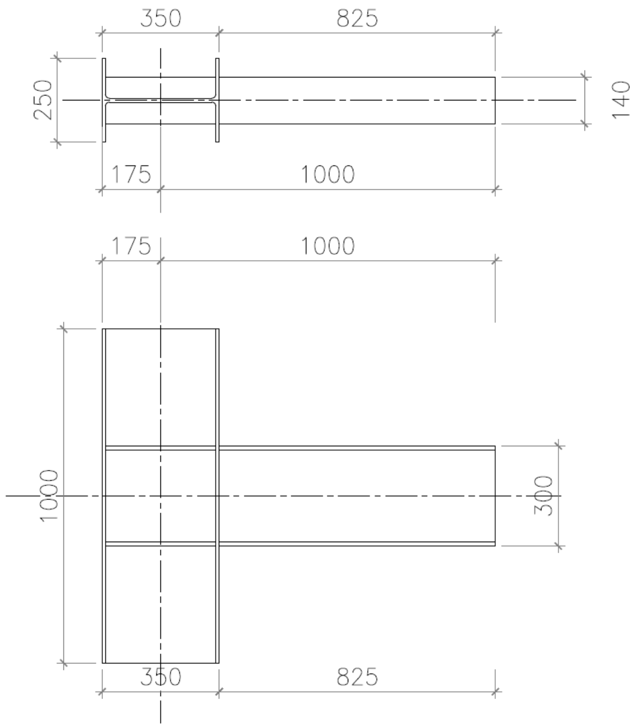


Fig. 2 - Plan and elevation of column- beam section

The dimensions of RBS have supported the recommendations projected by FEMA 350 [11-13] and calculated as

$$0.5\,b_{bf} \leq a \leq 0.75b_{bf}$$
$$0.65\,d_b \leq b \leq 0.85d_b$$
$$0.1\,b_{bf} \leq c \leq 0.25b_{bf}$$

where

b_{bf}= Beam Flange width

d_b = Beam depth

$$R = \frac{4c^2 + b^2}{8c}, \quad \text{where R = Radius of RBS cut}$$

The main parameters were a, the location of RBS trim from the column, b, the size of RBS, and c, the depth of RBS at its lowest. The cut diameter R can be identified with the respective dimensions of parameters b, parameter c predicted via arc curvature. The dimensions of RBS are tabulated in Table 3.

Table 3 - RBS dimensions

Specimen	a (mm)	b (mm)	c (mm)	R (mm)	Beam Reduction %
AIWR-01	-	-	-	-	100
AIRR-02	80	225	30	226	73.12
AISR-03	80	225	30	226	73.12
AITR-04	80	225	30	226	73.12
AISS-05	80	225	30 mm dia holes – 6 nos on each side (Single row)		73.12
AITS-06	80	225	15 mm dia holes – 11 nos per row – 22 nos on each side (Two rows)		73.12
AIVH-07	80	225	Dia of holes varied from 10 mm to 30 mm with 5 mm increment on each side		73.12

Nevertheless, the dimension a should be sufficient to allow strain in reduced beam portion to distributed evenly all around the face of the column across the flange width. Likewise, the dimension b ought to be adequate to prevent exorbitant inelastic strains inside the RBS zone. The c value regulates the optimum moment produced within the RBS, thereby regulating the maximum moment created in the column face. In order to achieve an acceptable design, many iterations in the RBS dimensions can be required. Based on the RBS profile sizing, Section without RBS is shown in Figure 3, Radius Cut RBS is displayed in Figure 4, Straight Cut RBS in Figure 5, Tapered Cut RBS is displayed in Figure 6, Single row Same holes RBS is displayed in Figure 7, Two rows Same holes RBS is displayed in Figure 8 and Varied holes RBS is displayed in Figure 9.

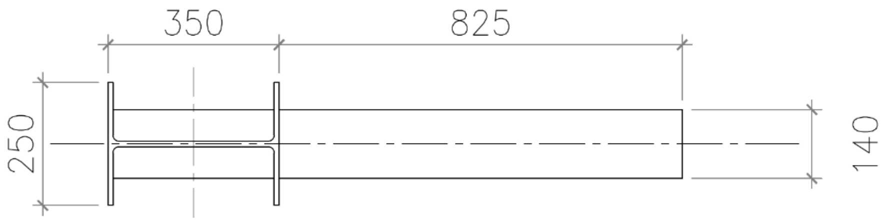


Fig. 3 - Section without RBS

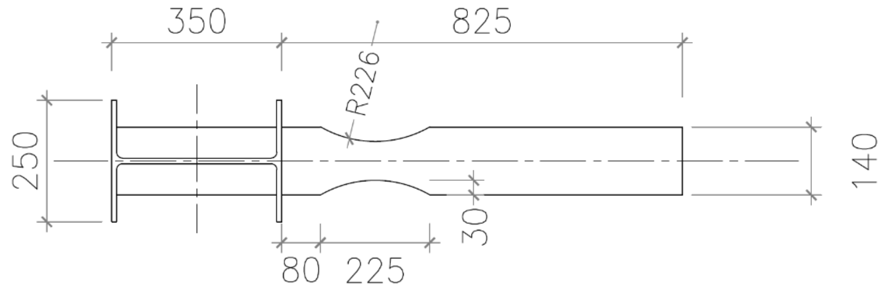


Fig. 4 - Radius cut RBS

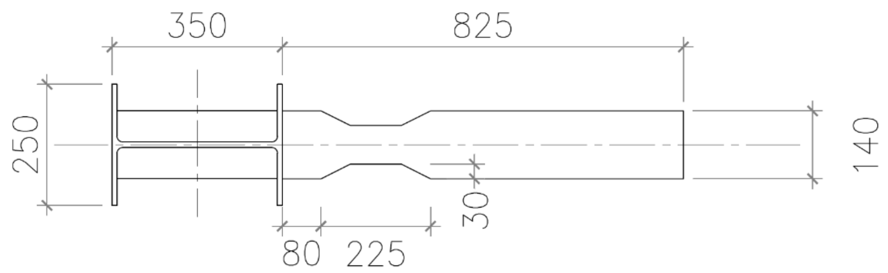


Fig. 5 - Straight cut RBS

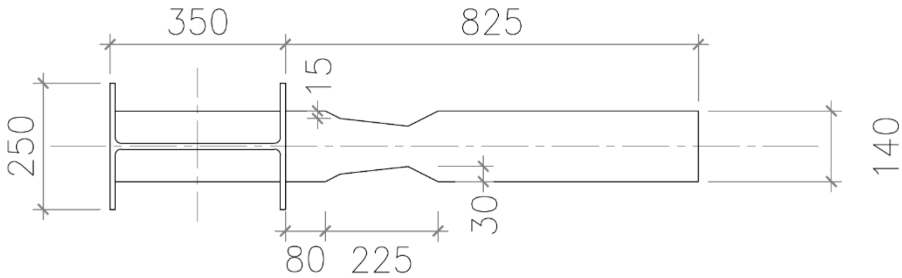


Fig. 6 - Tapered cut RBS

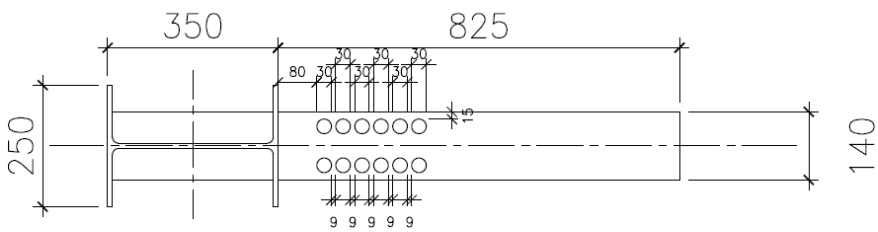


Fig. 7 - Single row same holes RBS

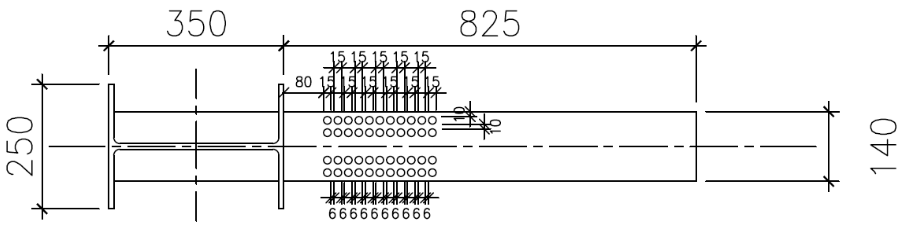


Fig. 8 - Two rows same holes RBS

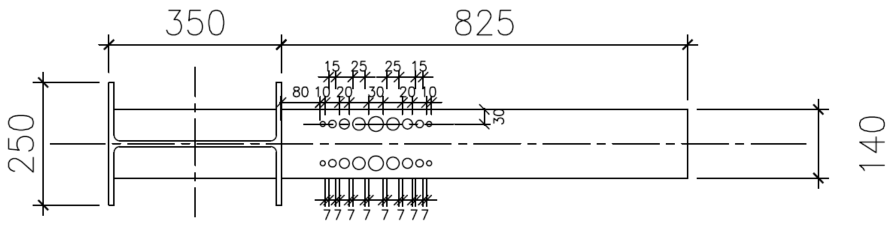


Fig. 9 - Varied holes RBS

The general purpose of scaling the RBS is to confine the maximum moment of the beam which can develop on the column, values within the variety of approximately between 85% and 100% of the actual plastic or yield moment of the beam [14-15]. The Column's maximum moment (M_f) against the beam's plastic(yield) moment (M_{pe}) was calculated as follows and the results are tabulated in Table 4.

$$\frac{M_f}{\Phi_d M_{pe}} \leq 1$$

where M_{pe} = Plastic Moment of Beam

M_f = Column's Maximum Moment

Table 4 - RBS moment calculation

Specimen	M_{pe} Nmm	M_f Nmm	$\frac{M_f}{\phi_d M_{pe}}$
AIWR-01	-	-	-
AIRR-02	215.10 X 10^6	202.19 X 10^6	0.94
AISR-03	215.10 X 10^6	202.19 X 10^6	0.94
AITR-04	215.10 X 10^6	202.19 X 10^6	0.94
AISS-05	215.10 X 10^6	202.19 X 10^6	0.94
AITS-06	215.10 X 10^6	202.19 X 10^6	0.94
AIVH-07	215.10 X 10^6	202.19 X 10^6	0.94

The section of the beam and column should be plastic or compact depending on the constraint laid down in IS 800: 2007 [10]. The beam and column section used for the connections should fulfill the following relation

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} \geq 1.2$$

Where ΣM_{pc} = Sum of moment capacity in column

ΣM_{pb} = Sum of moment capacity in beam

The above relation was utilized to test stronger column -weaker beam requirements, the result values are shown in Table 5.

Table 5 - Strong column-weak beam requirements

Specimen	ΣM_{pc} Nmm	ΣM_{pb} Nmm	$\frac{\Sigma M_{pc}}{\Sigma M_{pb}}$
AIWR-01, AIRR-02, AISR-03, AITR-04, AISS-05, AITS-06, AIVH-07	317.17 X 10^6	232.05 X 10^6	1.367

In columns, the thickness of Continuity plates selected to be adequate the beam's flange thickness, so it may turn out to be a robust panel zone, causing the plastic hinge to appear within the weakening zone. Through the values calculated from the below relation as per IS 800:2007 [10], it may be ascertained that double web plates weren't needed.

$$t \geq (d_p + b_p)/90$$

where t = Thickness of double plate

d_p = Depth of panel zone

b_p = Width of panel zone

3. Numerical Modeling

The empirical analysis has been carried out to determine the impact of various parameters on the behavior of connections. The 3-D model was designed with four-node shell elements by using the finite element software ABAQUS [16]. The model consists of subassemblies of a column and a beam linked by an end plate welded on the beam. The beam length from the column flange was taken as 825 mm and therefore the column length was 1000 mm for the FE model.

The boundary conditions were given as fixed at each end of the column for the computation in this paper. The seven specimen types that include Section without RBS, Radius cut RBS, straight cut RBS, tapered cut RBS, Single row Same holes RBS, two rows Same holes RBS and Varied Holes RBS have been modeled. Several analyses have been carried out for Radius cut RBS and Tapered cut RBS, particularly for European profiles [17-19]. Welds were not designed in the subassemblies, as the use of reduced beam sections lowers the stress requirements and weld tolerance, thus avoiding the need for in-depth modeling of the welding behavior. Instead of combining the instances for connection in the ABAQUS models, a tie constraint was used.

The nonlinear material properties are derived from the coupon test carried out for this research for the statistical analysis. The values Young's modulus, Poisson's ratio and Yield stress are $E = 2 \times 10^5$ MPa, $\nu = 0.3$ and $f_y = 250$ N/mm^2 used respectively. For model plastic deformations of the joint elements, a hybrid (isotropic/linear kinematic) strengthening law with a criterion of Von Mises yielding was implemented. For modeling, the connection zone and the region of column and beam in the region near the connecting zone, the smoother mesh was provided. Figure 10 shows the view of the finite element model with typical meshing.

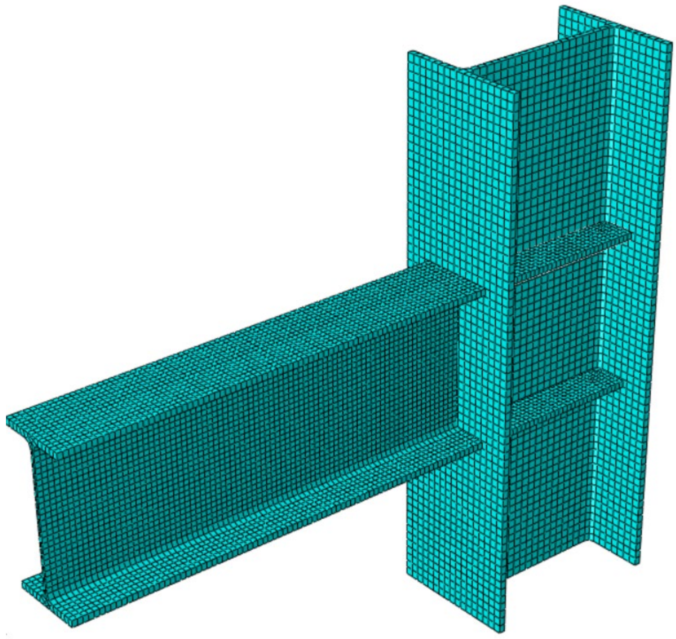


Fig. 10 - View of the FE model

The study was carried out using cyclic differential amplitude displacement at the beam tip. The loading procedure followed as per the recommendation provided in AISC Seismic provisions [14,15] and SAC Loading history [20]. The loading assessment used for this study is shown in Table 6 and Figure 11.

Table 6 - Loading assessment

Step	Number of Cycles (n)	Deformation δ_y (radians)
1	6	0.00375
2	6	0.005
3	6	0.0075
4	4	0.01
5	2	0.015
6	2	0.02
7	2	0.03
8	2	0.04
9	2	0.05

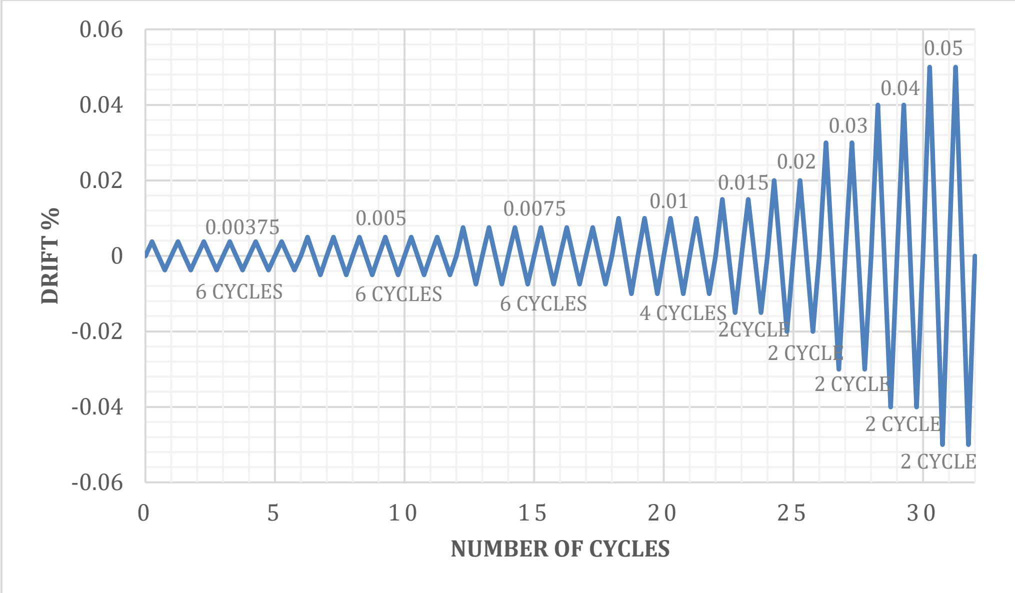


Fig. 11 - Loading protocol

4. Analytical Study Observations

4.1 Von Misses Stress

The Von misses stress evaluates the yield failure thresholds by ascertaining the material yields. The Figure 12 shows von misses stress for AIWR-01 (Section without RBS), Figure 13 for AIRR-02(Radius cut RBS), Figure 14 for AISR-03(Straight Cut RBS), Figure 15 for AITR-04 (Tapered cut RBS), Figure 16 for AISS-05 (Single row Same holes RBS), Figure 17 for AITS-06 (Two rows Same holes RBS) and Figure 18 for AIVH-07 (Varied Holes RBS) respectively.

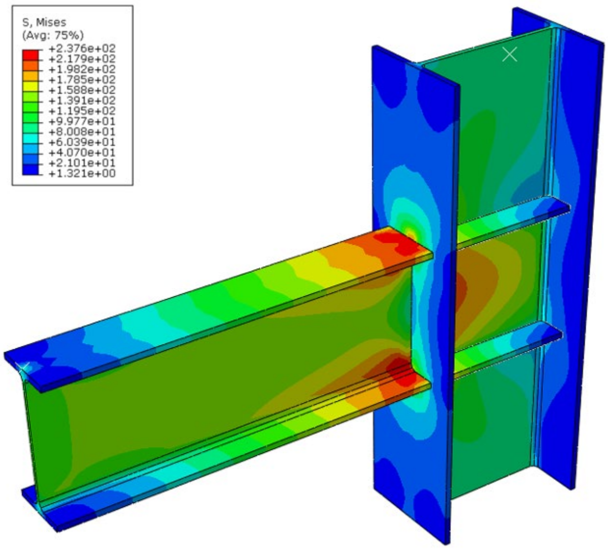


Fig. 12 - Von misses - AIWR-01

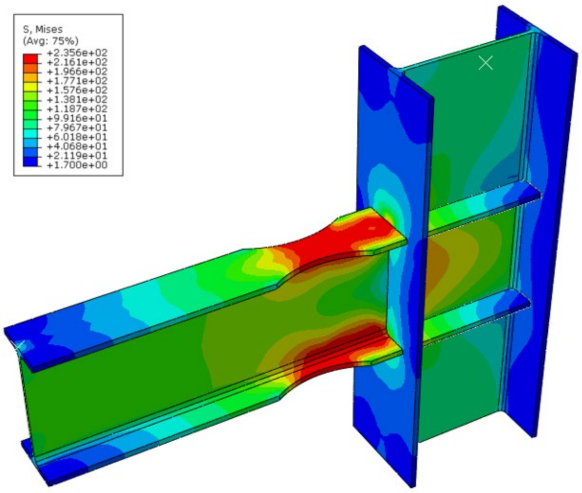


Fig. 13 - Von misses - AIRR-02

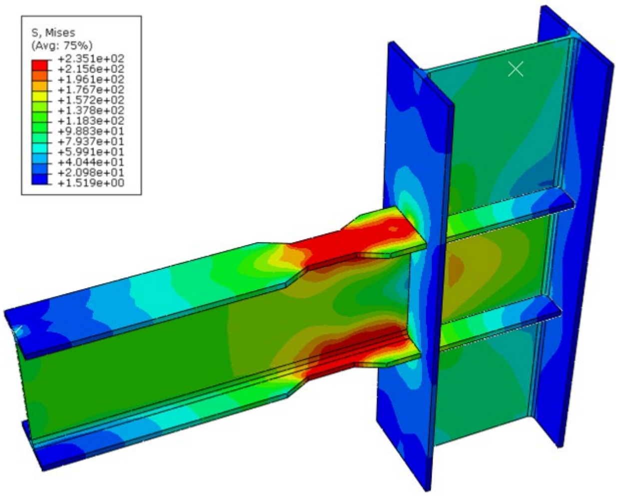


Fig. 14 - Von misses - AISR-03

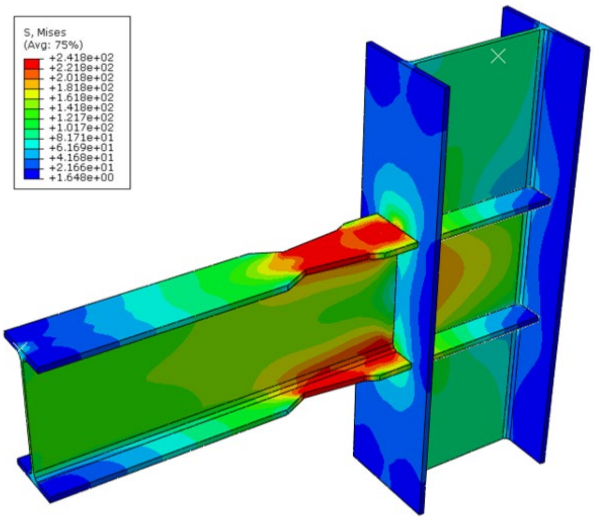


Fig. 15 - Von misses - AITR-04

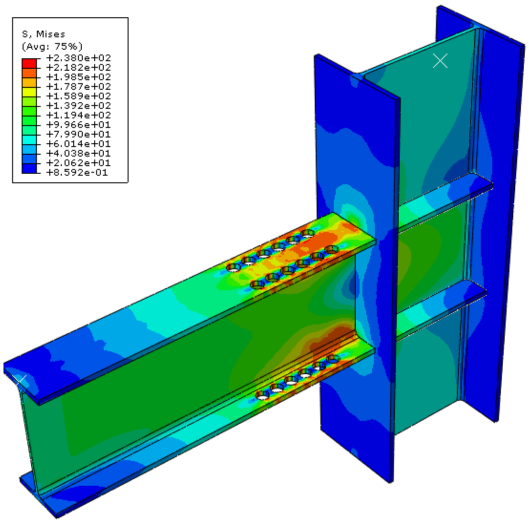


Fig. 16 - Von misses - AISS-05

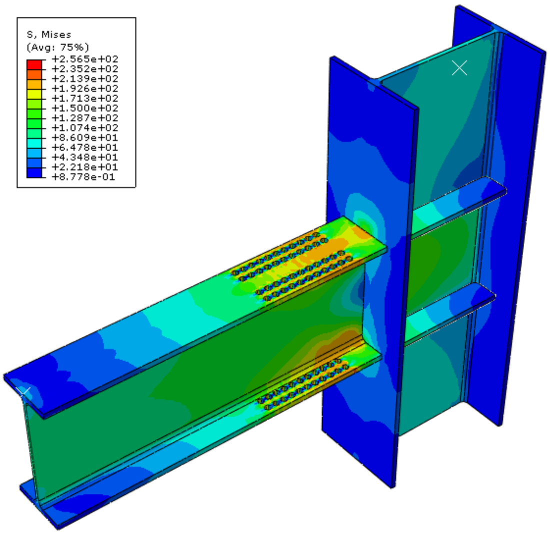


Fig. 17 - Von misses - AITS-06

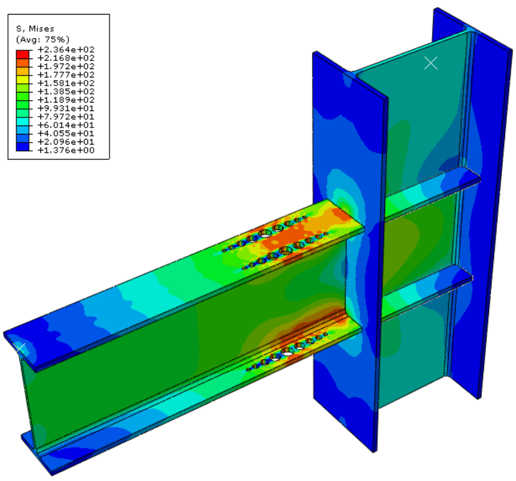


Fig. 18 - Von misses - AIVH-07

For AIRR-02, AISR-03, AITR-04 model specimens, stress occurred within the beam itself specifically in the RBS zone at 0.05 radians. Likewise, the highest stress is observable in and around the holes for the specimens AISS-05, AITS-06 and AIVH-07. In AIWR-01, stresses occurred in connection near the column face. As a result, the fracture may happen in the weld connection for the AIWR-01 model specimen. Column face also has a significant stress distribution in the section without RBS than Section with RBS. The peak stress contour is evident in the panel zone for section without RBS, whereas the stress distribution is lower in all sections with RBS. The moment potential is enhanced to decrease the stress intensity at the connection to prevent initial stage fracture inadequacies of the connection. The outcome of FEA simulation offers complete information that supports the statement.

4.2 PEEQ

The PEEQ (equivalent plastic strain) index is known to be the localized failure index that contribute significantly to the redesign of the connection under seismic activity. In the specimen AIWR-01, the PEEQ is observed to be higher in connections leading to the localized fracturing at the welding joints. For the specimens AIRR-02, AISR-03 and AITR-04, moderate to high PEEQ occurred within the RBS area, while substantial PEEQ was also seen in the connections. Whereas in the specimens with flange holes, the highest PEEQ is clearly identifiable near the holes and much less PEEQ is found in the connection areas. It is conclusive that the plastic hinge is generated in the RBS region instead of the connection zone. AISS-05 and AITS-06 showed successful findings in equivalent plastic strain. Figure 19 for AIWR-01(Section without RBS), Figure 20 for AIRR-02 (Radius cut RBS), Figure 21 for AISR-03(Straight Cut RBS), Figure 22 for AITR-04 (Tapered cut RBS), Figure 23 for AISS-05 (Single row Same holes RBS), Figure 24 for AITS-06 (Two rows Same holes RBS) and Figure 25 for AIVH-07 (Varied Holes RBS) respectively.

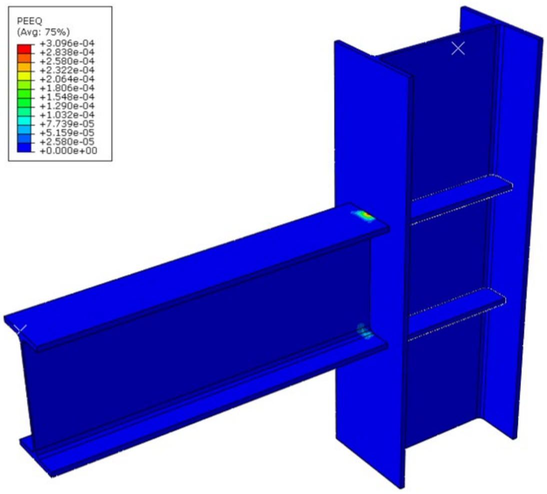


Fig. 19 - PEEQ index - AIWR-01

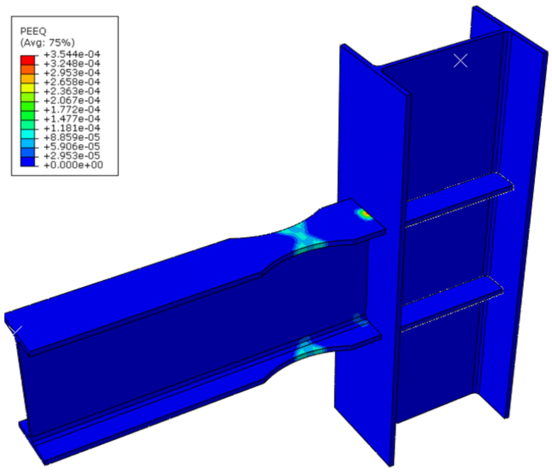


Fig. 20 - PEEQ index - AIRR-02

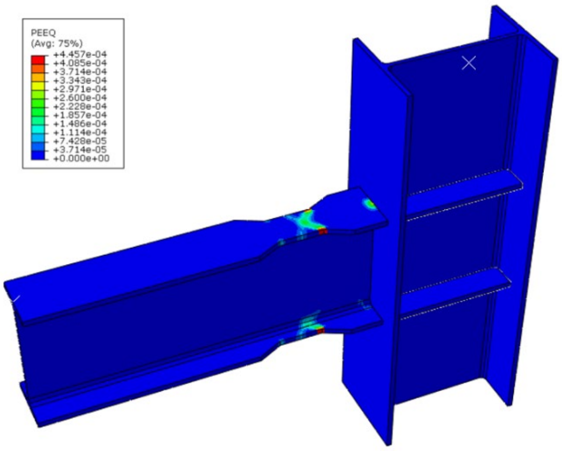


Fig. 21 - PEEQ index - AISR-03

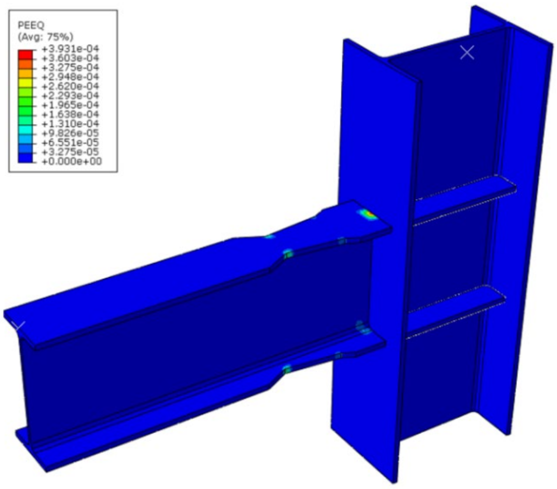


Fig. 22 - PEEQ index - AITR-04

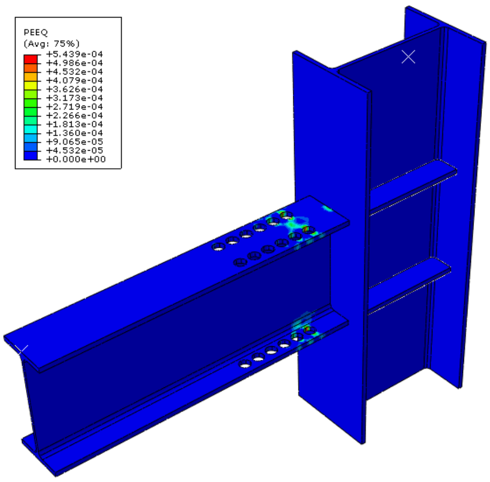


Fig. 23 - PEEQ index - AISS-05

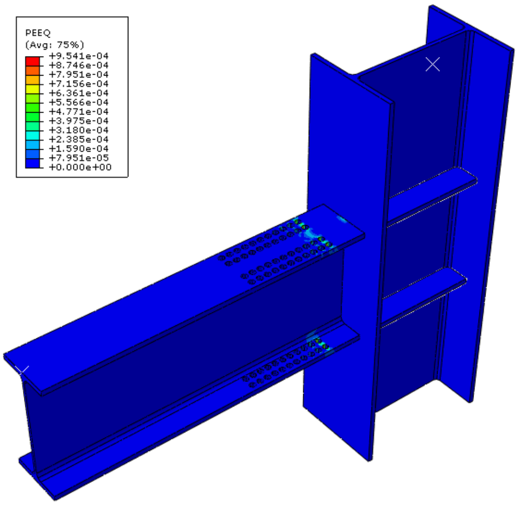


Fig. 24 - PEEQ index - AITS-06

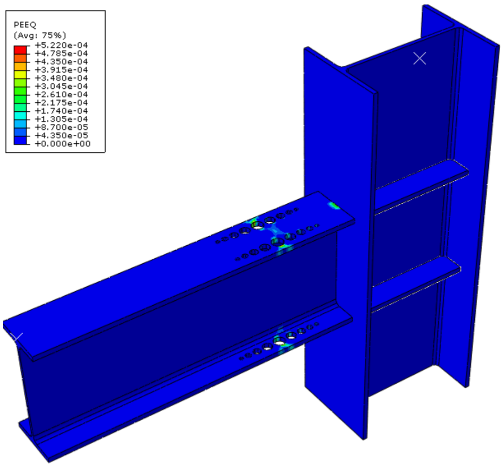
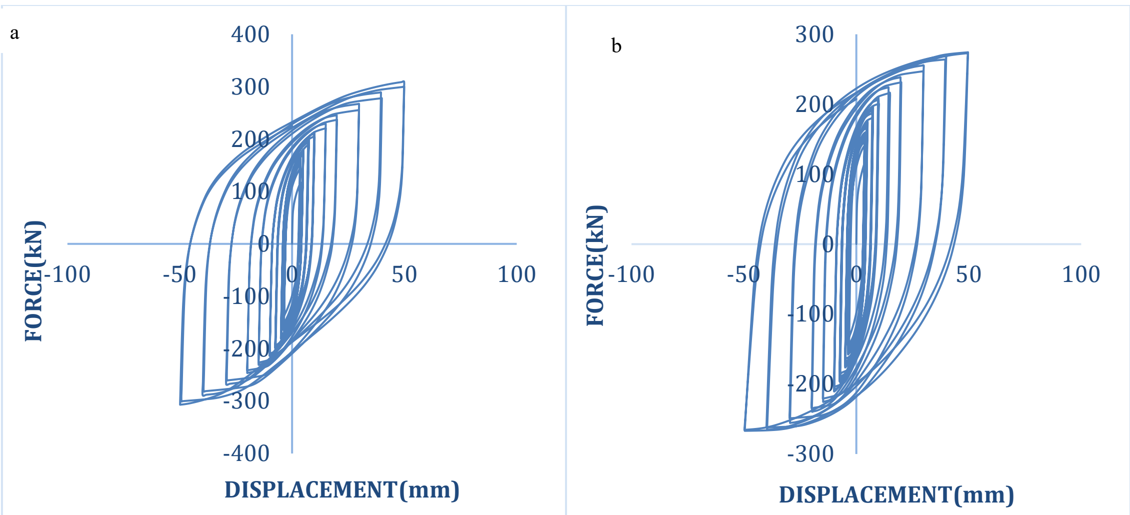


Fig. 25 - PEEQ index - AIVH-07

4.3 Loading Response

The cyclic response was effectively determined prior to the breakdown of the beam welding, which decreases the rigidity and strength of the connection. For all models with and without RBS, Figure 26 indicates hysteresis activity between force and displacement. All models have similarities in curves with slight variations in hysteresis behavior. Noticeable force deterioration occurs in Section with RBS, which is greater than 85% of plastic moment strength, which is allowed under Special moment frame systems seismic provision of AISC:2005[14,15].



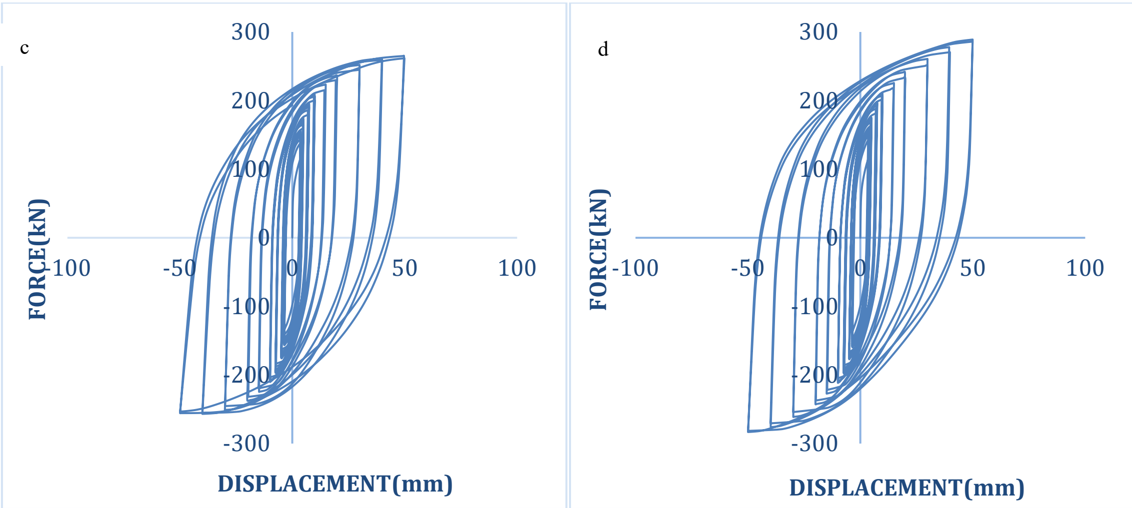


Fig. 26 - Hysteresis behaviour a) AIWR-01; b) AIRR-02; c) AISR-03; d) AITR-04

4.4 Rupture Index

The Rupture Index (RI) was evaluated to assess the impact of ductile failure capacity for all the models using FEA outputs. The ductile failure of the connection is directly proportional to the RI value. The Rupture Index value is derived by using

$$RI = \frac{PEEQ/\varepsilon_f}{\exp(1.5\frac{p}{q})}, \qquad \text{where p = hydrostatic pressure; q = von misses stress}$$

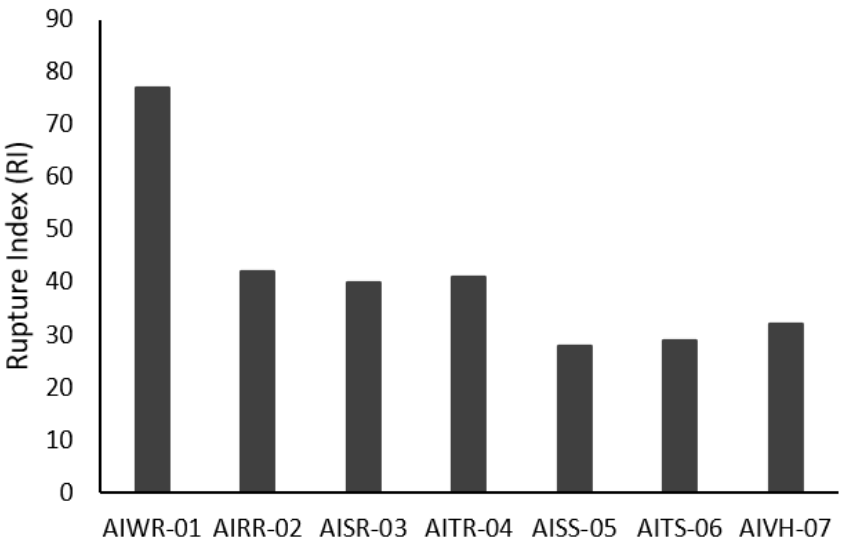


Fig. 27 - Rupture Index for all specimens

Figure 27 reveals that the RI value of the section without RBS is greater than that of other specimens. The lesser plastic strain produced at the beam and column junction results in lower RI and the minimal probability of failure for the specimens AISS-05 and AITS-06.

4.5 Connection Zone Inferences

Stress and strain are the essential criteria from which failure and efficiency of the connection can be examined. As seen in Figure 28, Line A is positioned between the beam flange and the column face along the joint which is perceived to be the critical region.

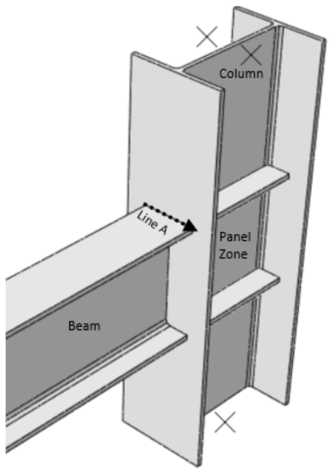


Fig. 28 - Typical view of critical connection zone

It is obviously seen from the stress distribution contour in Figure 29 along line A that the specimen without RBS has a higher stress value and the stress distribution is much broader than the other specimens. Flange-Holes specimens such as AISS-05 and AITS-06 have relatively lower stress values.

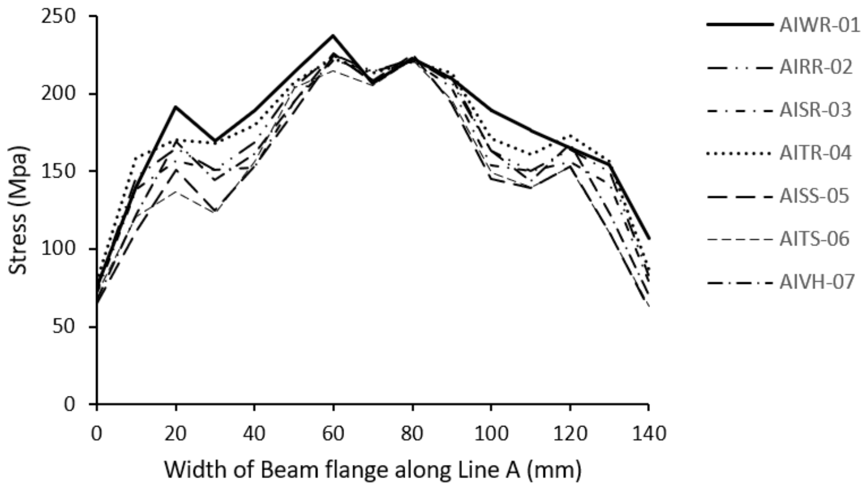


Fig. 29 - Stress distribution in Line A

The PEEQ index value varied along the line running through the beam flange width from null to maximum. For the specimen without RBS, the PEEQ index value is optimum relative to the specimen with RBS in the critical connection zone as shown in Figure 30. The plastic hinge is thus developed for the Specimen AIWR-01 in the connection region.

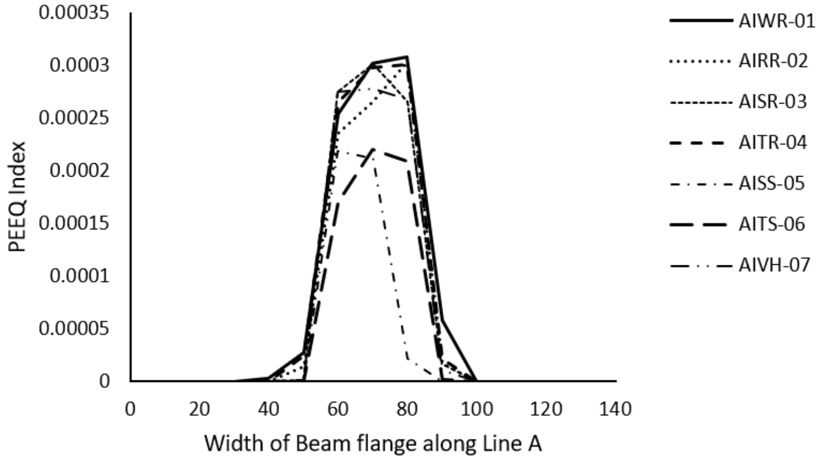


Fig. 30 - Equivalent plastic strain indices in Line A

4.6 Panel Zone

The ductility formed in the panel zone before the failure of connection is known to be a crucial quality indicator for seismic activities. Column and moment ratio, if less than 0.9 is known to be a poor panel zone. Most plastic story drift occurs only in the panel zone area. The plastic strain of the strong panel zone should be drastically smaller compared to the weak panel zone connection. On the basis of the calculations and results referred to in Table 5, it was concluded that all model specimens had an appropriate moment ratio that could be interpreted as a robust panel zone. As seen in Figure 31, design performance stresses in the panel area taken along the beam axis, the model specimen AIWR-01 is significantly higher than the other model specimens. Likewise, AISS-05 and AITS-06 have lower stress value in the panel zone.

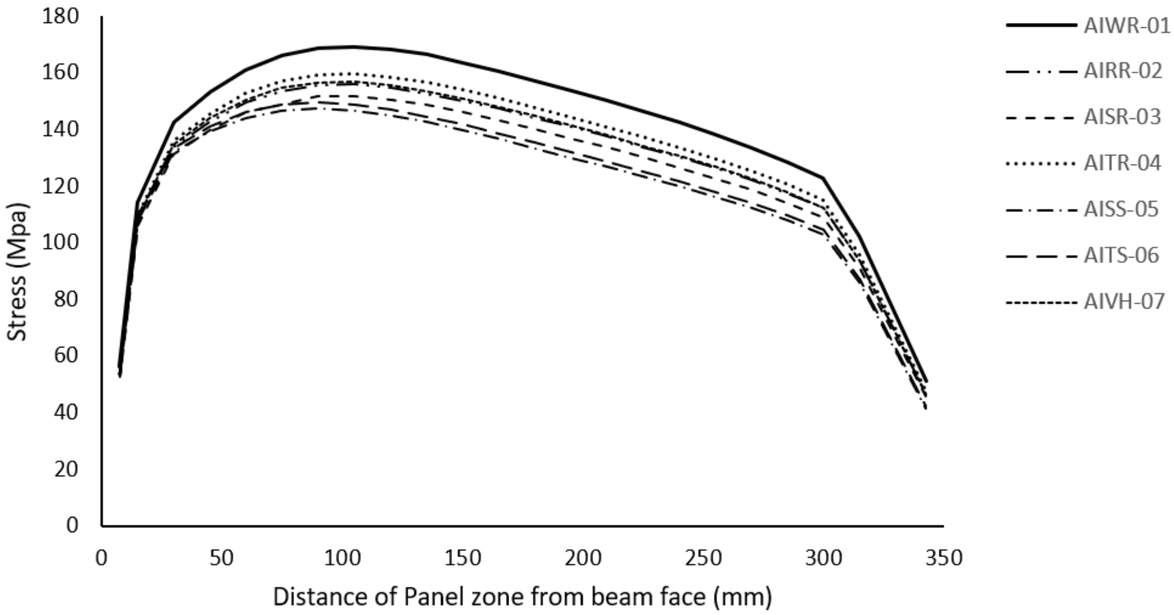


Fig. 31 - Stress distribution in panel zone

5. Conclusions

The following conclusions are drawn from a thorough discussion and analysis of parameters concerning the cyclic behavior of steel moment-resisting frames. Specimens have been developed for a strong panel zone that increases plastic rotation capability and decreases beam distortion due to inelastic strain concentration in the RBS region. The cyclic nature of the beam of the connection is extremely good when plastic hinges are formed in the RBS region. Depending on the hysteresis curve results, the connection is called to be an optimal strength connection and can be ideal for special moment frame structures.

At 0.05 radian plastic hinge occurs in connections for AIWR-01, while Peak plastic hinge occurs in the RBS zone for specimens with RBS. Stress is distributed evenly in the RBS region, resulting in the prevention of fracture for the sections with RBS. AIWR-01 indicates the concentration of stress in the column face while AIRR-02, AISR-03, and AITR-04 indicate the concentration of stress in RBS areas. The maximum PEEQ is readily apparent near the holes in the Flange- Holes specimens and there is much less PEEQ in the connection region. It is conclusive that the plastic hinge is generated in the RBS region instead of the connection zone.

Specimens AISS-05 and AITS-06 were found to have significantly reduced stress concentration at the connections, which actually improved the plastic deformation capability of the connection. AISS-05 and AITS-06 showed successful findings in equivalent plastic strain. The lesser plastic strain produced at the beam and column junction results in lower RI and the minimal probability of failure for the specimens AISS-05 and AITS-06. In comparison with other specimens, AISS-05 and AITS-06 have lower stress concentrations in the panel zone. It is inferred from the above results of FEA investigation that the Specimen with Single row Same hole RBS and Two row Same holes RBS exhibited better seismic activity efficiency. Instead of using heavy columns, it is possible to choose an RBS connection that is the most cost-effective without additional load for seismically enhanced structure. Reconstruction of RBS design parameters formulated by FEMA350 to ensure the safe application of Indian profiles.

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