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EFFECT OF COLUMN PANEL ZONE CHARACTERISTICS ON INSTABILITY OF BEAMS WITH RBS MOMENT RESISTING CONNECTIONS

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SUMMARY

Column panel zone (PZ) ductility significantly affects failure mode of the beams with RBS moment resisting connections. Even though good hysteretic behavior is expected in the connections with strong PZs, their flexural strength considerably deteriorates due to instability in the beams. On the contrary, weak PZs are prone to high shear deformation caused by unbalanced moments in an earthquake. High shear deformation in PZ induces secondary flexural stress adjacent to column face that can finally result in brittle fracture. Experimental observations indicate column panel zones, which lie between two aforementioned limits, can suitably participate with RBS moment resisting connections in energy dissipation. Those specimens developed high inelastic deformations without notable deterioration in flexural strength due to beam instability.

The presented paper aims to investigate how strong PZ must be, so that the positive effect of PZ shear yielding can be utilized on one hand, and the flexural strength deterioration due to beam instability can be properly prevented on the other hand. For this regard, a numerical analysis was conducted on a series of subassemblies with various PZ characteristics by ANSYS finite element software. The results indicate partial shear yielding in PZ can improve hysteretic behavior of specimens by avoiding premature instability in the beams.

INTRODUCTION

The 1994 Northridge earthquake revealed serious damages to the conventional BWWF connection, which was formerly known as a ductile moment connection. Since then, a great deal of research has been conducted on the existing moment connections to find deficiencies and to improve their cyclic behavior (among them: Engelhardt [1], Miller [2], Mahin [3], and Calado [4]). Among various methods proposed to modify cyclic performance of the conventional connections, the RBS moment connection possesses remarkable superiority. Through extensive experimental studies (Engelhardt[5], Engelhardt [6], Uang [7]), it is confirmed that the RBS moment connections can develop high inelastic deformations and attain acceptable plastic rotation. However, there is a major problem with this type of connection that is,

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deterioration of load-carrying capacity owing to lateral and local buckling in beam. Because, reducing the beam flange causes the restraints against out-of-plane deformations to degrade.

One of the parameters, which can considerably influence failure mode of the beams with RBS moment connections, is column panel zone ductility. Popov [8] indicated that the beam-to-column joints with weak PZ encounter high shear deformation, resulting in brittle fracture within the weld connecting beam flange to column face. As a result, in spite of weak PZs ability in dissipating a big amount of energy, using weak joints is not recommended. On the contrary, in the presence of strong PZs, fracture potential is reduced, but the possibility for beam instability rises, especially for RBS connections. Tsai [9] experimentally illustrated that moderately strong PZs show appropriate performance.

This paper aims to develop further justification with respect to the effect of PZ characteristics on cyclic behavior of the beams with RBS moment resisting connections. A numerical analysis was conducted on a series of subassemblies with various PZ characteristics by ANSYS finite element software. The results verify that partial shear yielding in PZ can improve hysteretic behavior of specimens by avoiding premature instability in the beams.

FINITE ELEMENT PARAMETRIC STUDY

In order to verify the validity of this numerical research, a finite element model was prepared for specimen DB3 of experimental study conducted by Engelhardt [6]. This model was analyzed under cyclic displacement control loading using ANSYS finite element software. The obtained moment-rotation hysteretic response is shown in Fig.1. The beam moment was measured at the column face, and the rotation was computed by dividing the total beam tip deflection by the distance from the beam tip to the column face. Some comparable points of these two studies indicate that the numerical results are in an acceptable accordance with those reported by Engelhardt [6]. The main parameters used to evaluate numerical analysis accuracy are: (1) the maximum moment developed at the column face during the cyclic loading: the difference between the maximum moments at the column face resulted from the two studies is less than 4%. (2) the onset of flexural moment deterioration: in the both studies, the moment-rotation loops initiate to deteriorate after a plastic rotation of 0.02 radian (equivalent to 4 in displacement at the beam tip). (3) the total deterioration in the maximum moment developed at the column face: the total deterioration in the maximum beam moment at the end of loading is about 25% in the experimental and numerical studies.

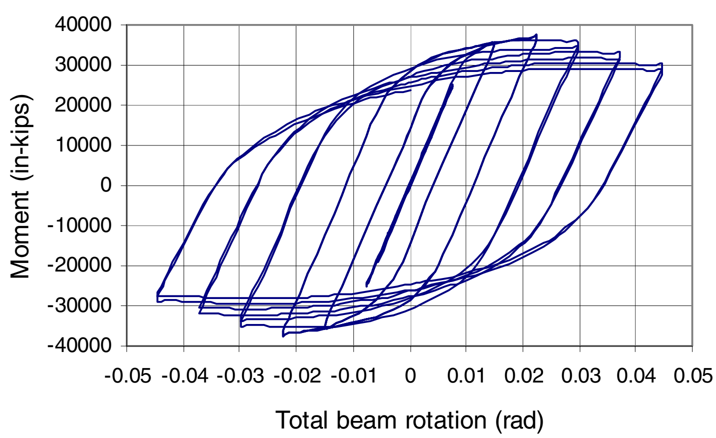


Fig.1. Analytical hysteretic response of specimen DB3

After being assured about numerical analysis accuracy, six other subassemblies with the general configuration shown in Fig.2 (a) were chosen. The column was supported at the base by a hinge, while a vertical roller was used at another end. The beam was laterally braced at a distance of 150 cm from the column face. This distance coincides with UBC [10] provisions. The design of these specimens is described in the next section.

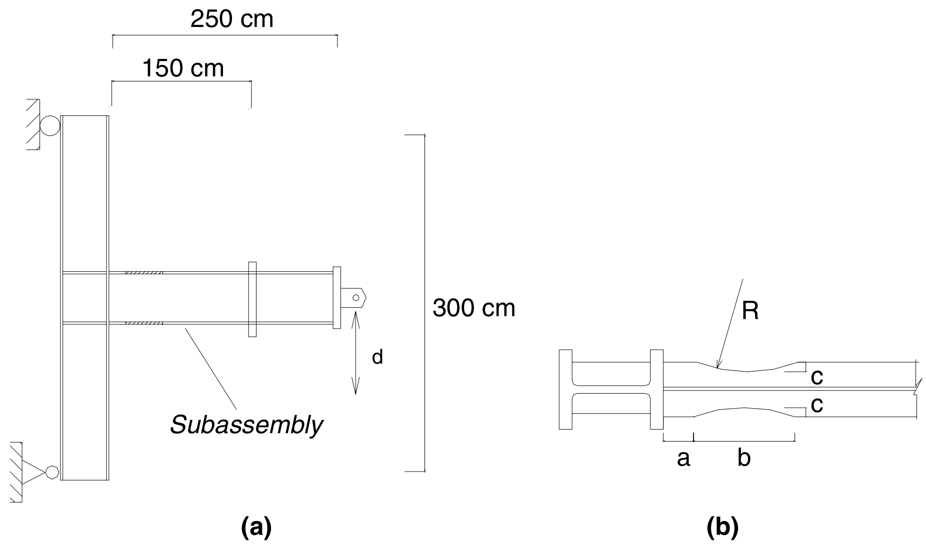


Fig.2. Configuration of (a) subassembly; (b) RBS connection

Design of specimens

The first three specimens, designated by RBS1, consist of a IPE300 beam and a IPB200 column, and in the remaining specimens, designated by RBS2, a IPE450 beam is framed into a IPB300 column. The Beams and columns were pre-selected so that they could produce weak column panel zones at the intersection of beam and column. Afterwards, in each set of specimens, RBS1 and RBS2, column panel zone was enhanced by a 6 mm and a 10 mm thick plate in order to provide balanced and strong PZ, respectively. That whether PZ is strong or not is judged by comparing the design shear demand, V_r, with the ultimate shear strength of column panel zone, V_y, recommended by UBC [10]. They both can be computed as follows:

$$V_y = 0.55 F_y d_c t_{cw} (1 + 3 b_c t_{cf}^2 / d_b d_c t_{cw}) \tag{1}$$

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

in which F_y =yield stress of PZ material, M_P =plastic moment capacity of beam section, L_b =beam length from column face to beam tip, and H =column height. d_c, d_b, b_c, t_{cw}, and t_{cf} are column depth, beam depth, column flange width, PZ thickness, and column flange thickness, respectively. In Eq.2, $\beta_E M_P$ is the flexural demand imposed at the column face. It is suggested that β_E is limited between 0.85 and 1. β_E =0.85 was taken as an initial assumption.

The member sizes and doubler plate thickness of the specimens as well as their ultimate shear strength and design shear demand are given in Table 1. In this study, the PZs without doubler plate are known as weak panels. On the contrary, the relatively thick PZs, whose the ultimate shear strength is notably more than the design shear demand, are introduced as strong panels. The balanced panel zones lie between two aforementioned limits.

Table 1. Specimens specifications

Spec.	Col.	Beam	Doubler Pl. th. (mm)	V_y (ton)	V_r (ton)
RBS1-W	IPB200	IPE300	0	29.70	33.01
RBS1-B	IPB200	IPE300	6	45.54	33.01
RBS1-S	IPB200	IPE300	10	56.10	33.01
RBS2-W	IPB300	IPE450	0	55.30	68.55
RBS2-B	IPB300	IPE450	6	80.05	68.55
RBS2-S	IPB300	IPE450	10	96.55	68.55

Design of RBS region

The radius cut with essential parameters shown in Fig.2 (b) was employed for the RBS region, and designed according to the following recommendations proposed by Engelhardt [6],

$$\frac{L_b}{L_b - a - b/2} M_{RBS} \leq M_F \tag{3}$$

$$M_F = \beta_E M_p \quad , \ \beta_E = 1 - 0.85 \tag{4}$$

$$a = (0.5 - 0.75)\, b_f \tag{5}$$

$$b = (0.65 - 0.85)\, d_b \tag{6}$$

$$c \geq \frac{Z_p}{2\, t_f (d_b - t_f)} \left[1 - \frac{\beta_E (L_b - a - 0.5\, b)}{\alpha\, L_b} \right] \quad , \ 2\, c \leq 0.5\, b_f \tag{7}$$

in which M_{RBS} and M_F = the maximum flexural moments at the most reduced section and at the column face, respectively, b_f = beam flange width, t_f = beam flange thickness, Z_P = plastic section modulus of beam, and α = a strain hardening factor of 1.15.

By making use of Eq. (3) to Eq. (7), the RBS region dimensions were determined as the values given in Table 2.

Table 2. RBS region dimensions

Beam	a (cm)	b (cm)	c (cm)	$b_{f\ RBS}$* (cm)	R (cm)
IPE300	8	20	3.2	8.6	17.225
IPE450	12	35	4.5	10	38.528

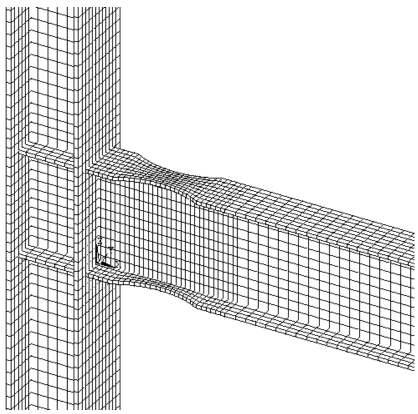
* $b_{f\ RBS}$ = the most reduced width of beam flange

FINITE ELEMENT ANALYSIS

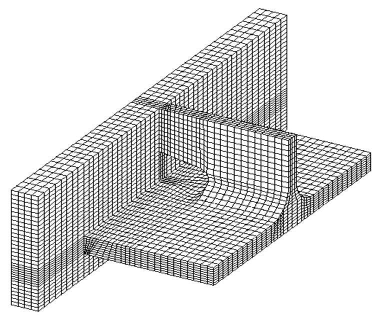
Modeling and analysis

The ANSYS finite element software [11] was utilized to model the specimens for large-deformation nonlinear analyses. The analyses were primarily intended to investigate the overall cyclic behavior of the subassemblies with an emphasis on the influence of PZ characteristics. For this purpose, the subassemblies were modeled using a quadrilateral 4-node shell element (element SHELL43 in ANSYS) with possibility to employ material nonlinearity. Fig.3 (a) shows a typical subassembly finite element meshing used in this study. As observed in Fig.3 (a), a more refined mesh was applied for the regions near the RBS. Since it was expected nonlinear deformations were mostly accommodated around the beam-to-column joint, nonlinear material was assigned to the elements within those portions, whereas for the reminder, the material was assumed to behave elastically. The plasticity model was based on the von Mises yielding criteria and its associated flow rule. The fundamental assumptions made to idealize steel mechanical properties were including: Young's modulus $= 2.1 \times 10^6\, kg/cm^2$, Poisson's ratio =0.3 , yield stress = $2500\,\mathrm{kg/cm^2}$, ultimate tensile strength = $3700\,\mathrm{kg/cm^2}$, and tangent modulus = Young's modulus / 100.

Since brittle fracture is likely to occur within the groove weld connecting the tensile beam flange to column face, a more detailed sub-model was made in order to evaluate fracture potential in each specimen. The element type SOLID 45 was used to create the sub-model which included weld access hole and beam flange to column face groove weld. Furthermore, it was assumed the beam flange was directly attached to the column flange by a groove weld that was also included in the sub-model. The yield stress and ultimate tensile strength of $2500\,\mathrm{kg/cm^2}$ and $2500\,\mathrm{kg/cm^2}$, respectively, were considered for the weld metal.



(a) Global model



(b) Sub-model

Fig.3. Three dimensional finite element model

Loading and analysis

Each subassembly was loaded on its beam tip by imposing cyclic displacement loading according to the standard SAC loading protocol [12] (see Fig.4). Cyclic nonlinear analyses of the subassemblies were performed using Riks method. In this method, buckling mode shapes of the model, computed in a separate buckling analysis, are implemented to perturb the original perfect geometry of the model. Then, the obtained imperfect model is analyzed to take local and lateral buckling into account.

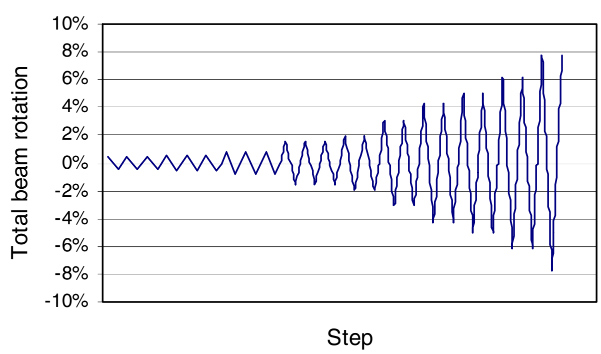


Fig.4. Loading history

In addition to cyclic analyses of the subassemblies, the sub-model created for each subassembly was monotonically loaded and analyzed under the deformations resulted from a plastic beam tip displacement of about 0.03 radian.

ANALYSIS RESULTS

Hysteretic response

Moment-beam rotation hysteretic response of the subassemblies resulted from the finite element analyses are shown in Fig.5. As previously defined, the beam moment was measured at the column face, and the rotation was computed by dividing the total beam tip deflection by the distance from the beam tip to the column face. In addition, in order to find out the influence of PZs on the cyclic behavior of the specimens, moment-PZ rotation hysteretic loops are illustrated in Fig.6. The PZ rotation is defined as:

$$\gamma_{PZ} = \frac{(a^2 + b^2)}{2\,a\,b}(\delta_1 - \delta_2) \qquad (8)$$

where $a\,, b$ = initial length and width of PZ, and δ_1, δ_2 = changes in length PZ diagonals.

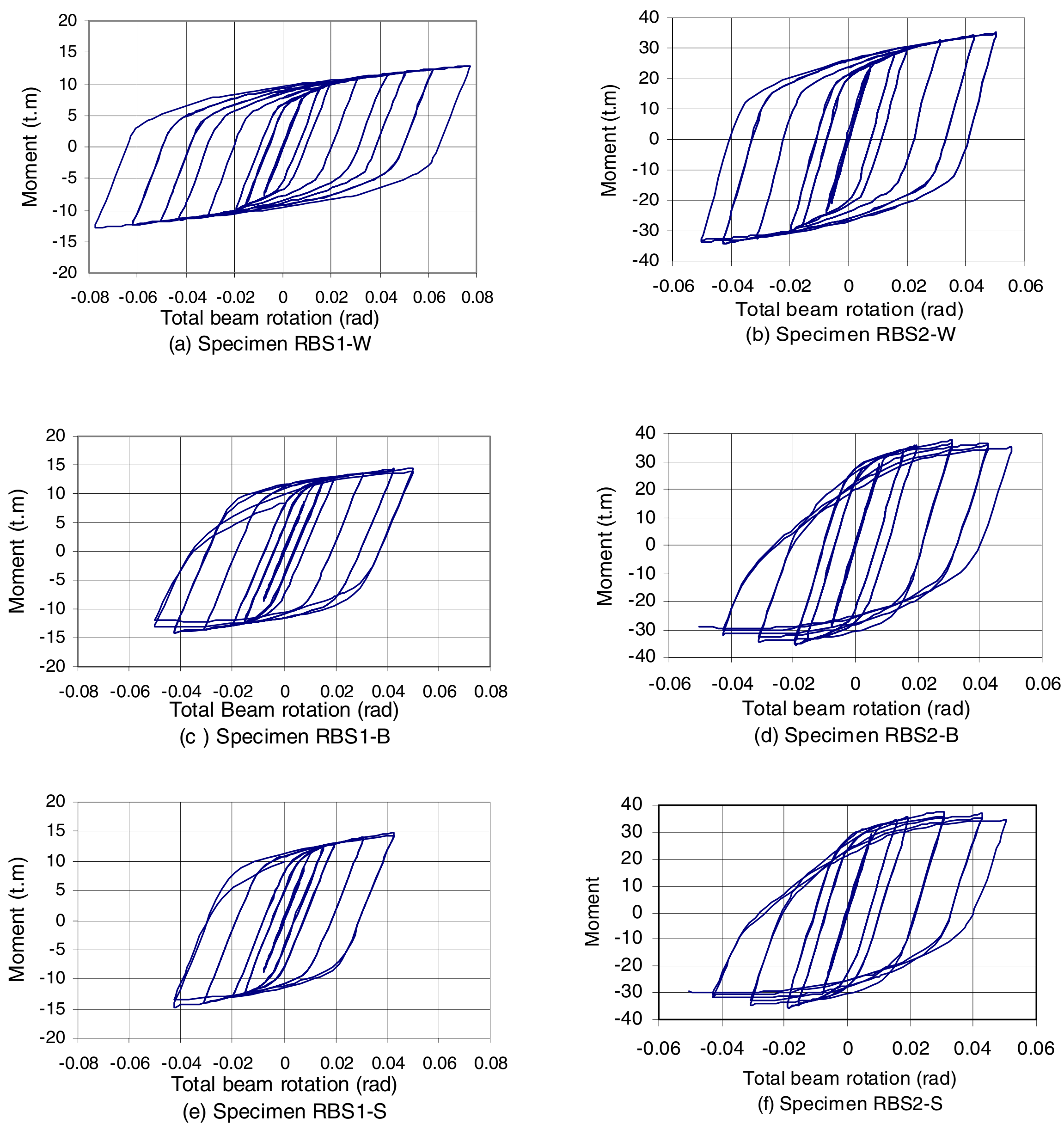


Fig.5. Hysteretic response of the specimens

In the specimens with weak PZ, RBS1-W and RBS2-W, no reduction is observed in the beam moment capacity. They showed, indeed, expanding and stable hysteretic behavior. The panel zones underwent high inelastic shear deformation and thoroughly yielded. In these specimens, the PZs played the main role in the energy dissipation. This can be apparently interfered by comparing Fig.5 (a) and (b) with Fig.6 (a) and (b), respectively.

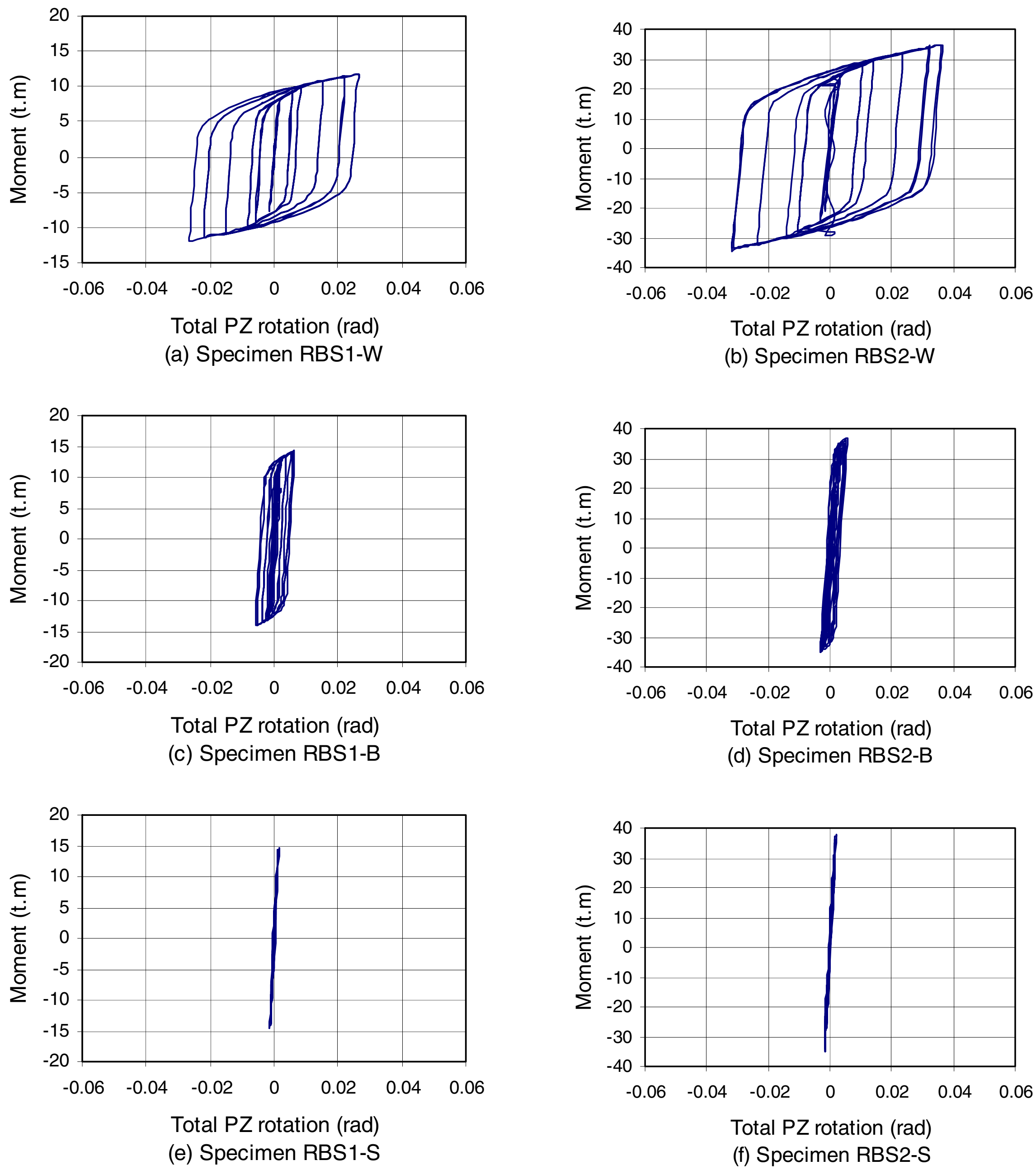


Fig.6. Hysteretic response of the column panel zones

In the specimens of strong PZ, the lateral and local buckling caused the flexural moment capacity of beam to be reduced. The amount of moment capacity reduction depends on the buckling mode which predominantly influences cyclic behavior of specimen. For instance, in the specimen RBS1-S, which possesses relatively slender beam, lateral-torsional buckling dominated hysteretic behavior. Indeed, the secondary tensile stress arisen from out-of-plane deformations due to lateral buckling was added to the tensile in-plane bending stress. As a result, before a significant reduction took place in the beam moment capacity, approximately uni-axial stress within the RBS region had exceeded the ultimate tensile strength of steel material, and the cyclic loading was therefore stopped. On the contrary, the lateral buckling was of less importance in the specimen RBS2-S, instead the local web buckling became a remarkable concern. In this situation, load-carrying capacity of specimen gradually deteriorated until the uni-axial stress in the RBS reached the ultimate tensile strength. The gradual deterioration of load-carrying capacity in the latter case arises from less out-of-plane deformations and less secondary tensile stress caused by lateral buckling. Uang [7] and Uang [13] have indicated in their valuable research that the web local buckling is the most important buckling mode in beams with RBS moment connections. However, according to this study, it seems the lateral buckling may be more destructive than the web local buckling, especially for laterally slender beams.

The balanced PZ specimens had the cyclic behavior the same as that of the strong PZ ones. In these subassemblies, the PZs properly participated with the RBS moment connections in energy dissipation (compare Fig.5 (c) and (d) with Fig.6 (c) and (d), respectively). Using a moderately thick doubler plate caused the buckling occurrence to be postponed and more plastic rotation was therefore achieved. The advantage of using balanced PZ can be readily observed by comparing Fig.5 (c) and Fig.5 (e). There is in appearance no tangible difference between hysteretic response of the specimens RBS2-B and RBS2-S (see Fig.5 (d) and Fig.5 (f)). But, it is worthy to note that at the end of loading, the maximum tensile stress of the RBS region in the specimen RBS2-S exceeded the ultimate tensile strength, contrary to the corresponding stress in the specimen RBS2-B, which was still under its ultimate limit. Consequently, the specimen RBS2-B showed rather better performance.

Fracture potential

In order to compare the different connections for fracture potential, a rupture index was computed at different locations across the tensile beam flange width. The rupture index (RI) which has been used by others (Ricles [14]) is defined as:

$$\mathrm{RI} = \frac{\varepsilon_p / \varepsilon_y}{\exp\left(-1.5\dfrac{\sigma_m}{\sigma_{eff}}\right)} \tag{9}$$

where ε_p, ε_p, ε_p, and ε_p are the equivalent plastic strain, yield strain, hydrostatic stress, and equivalent stress, respectively. The rupture indices across flange width for all specimens are illustrated in Fig.7.

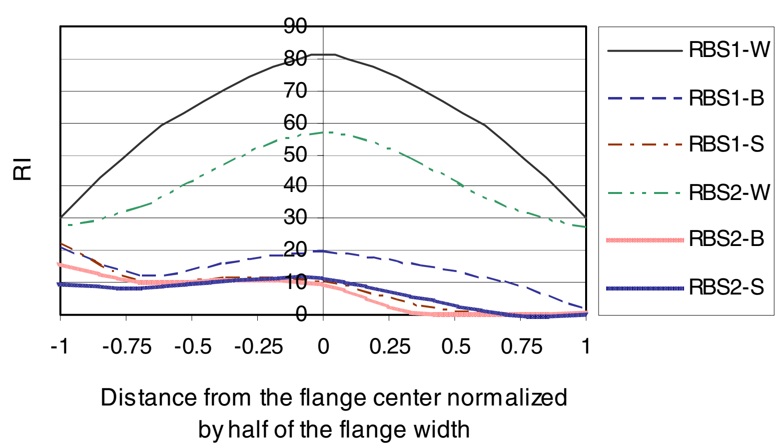


Fig.7. Rupture index across flange width

If the rupture indices are compared with that obtained for the weld material under uni-axial tensile stress (with a RI of about 30), it can be concluded that the specimens of weak PZ are highly prone to fracture. This fact denotes that although the specimens with weak PZ showed stable hysteretic response, the beam-to-column connection might fracture before attaining the required plastic beam rotation.

CONCLUSIONS

In order to evaluate the effect of PZ characteristics on instability of the beams with RBS moment connections a numerical parametric study was conducted on a set of subassemblies with various PZ configurations. In the following, the main results are pointed out:

1. Even though specimens with weak PZ indicate stable hysteretic response, beam-to-column connection may fracture before attaining the required plastic beam rotation.
2. In specimens of strong PZ, the lateral and local buckling cause the flexural moment capacity of beam to be reduced. Depending on the beam slenderness, the amount of moment reduction may vary.
3. In laterally slender beams, the lateral-torsional buckling dominates the hysteretic response of specimen, and it may be much more destructive than web local buckling.
4. In balanced PZ specimens, the PZ properly participates with the RBS moment connection in energy dissipation. Using a moderately thick doubler plate causes the buckling occurrence to be postponed and more plastic rotation is therefore achieved.
5. Because of limitation in the standard thickness of plates, there is sometimes no way except using a doubler plate thicker than that is needed. In such cases, it is suggested to locate a lateral brace at a distance nearer than that computed according to the current rules (such as UBC [10]).

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