EXPERIMENTAL STUDY ON THE KEY ISSUES AFFECTING CYCLIC BEHAVIOUR OF REDUCED BEAM SECTION MOMENT CONNECTIONS

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ABSTRACT :
Reduced beam section connections are renowned as ductile moment connections which are able to dissipate a large amount of energy during seismic excitations. In spite of the good hysteretic behaviour, an RBS beam is susceptible to local and lateral instability through which load-carrying capacity of the beam reduces. This paper aims to investigate the RBS connection behaviour, and key issues influencing cyclic behaviour of RBS beams such as panel zone. To this end, an experimental study was conducted on a series of steel subassemblies with RBS connection. The results indicate that all the specimens showed ductile behaviour and were able to dissipate significant energy. Moreover, if the PZ ductility is well selected, the beam buckling severity and its adverse effect on moment capacity reduce.

KEYWORDS:
RBS moment connections, Instability, Cyclic behaviour, PZ ductility

1. INTRODUCTION

The 1994 Northridge earthquake showed that the conventional moment connections are susceptible to unexpected damage (Miller 1998 and Mahin 1998). Consequently, the moment frame, which was formerly recognized as a ductile seismic resistant system, is not capable to reach adequate ductility. The pre- and post-Northridge laboratory observations have also indicated the inherent disability of the conventional moment connections to develop adequate ductility (Engelhardt and Husain 1993, Calado 2000).

Since the Northridge earthquake, a number of various studies have been carried out in order to improve the seismic performance of the conventional welded connections. One of the most promising ways to modify the behaviour of the conventional moment frame is to soften a portion of the beam flanges near the column face (Plumier 1997, Engelhardt et al. 1996, Engelhardt et al. 1998, Yu et al. 1999, Yu and Uang 2001). The connection softening may be accomplished by trimming off circular sectors from the beam flanges near the column. This solution, so-called reduced beam section (RBS) method, leads plastic hinges toward the beam span away from column face, resulting in reduction of stress concentration at the interface of beam and column.

However, as the consequence of reducing the beam section within a sensitive zone, the beam becomes more prone to buckling. Some studies have been conducted to assess key issues influencing the instability of RBS beams (Uang and Fan 1999, Uang and Fan 2001, Moslehi Tabar and Deylami 2005). The local and lateral slenderness ratios were the parameters which first drew attention of researchers. Uang and Fan (1999,2001) has showed that the cyclic behaviour of RBS beams is predominantly governed by the web local buckling. However, Moslehi Tabar and Deylami (2005) defined a new lateral slenderness parameter, which is in good agreement with the experimental data. According to this definition, the cyclic behaviour of RBS beams is mostly affected by their lateral instability and the beam depth-to-length ratio. Column panel zone (PZ) flexibility is another concern affecting the behaviour of RBS connections. Tsai and Chen (2002) and Jones et al. (2002) experimentally illustrated that RBS moment connections with moderately strong PZs show appropriate performance.

This paper is to present the results of testing carried out on three subassemblies with and without RBS moment connections. The main objectives are: (1) to study the effect of RBS moment connection on improving cyclic performance of the beams with less depth-to-length ratio; and (2) to examine the influence of PZ flexibility on reducing the buckling severity of the RBS beams.

2. EXPERIMENTAL PROGRAM

2.1. General specifications

Three full-scale steel subassemblies were tested in this laboratory program. The conventional moment connection was used in one of the specimens, whereas the other specimens had RBS moment connection. All the specimens were chosen as exterior subassemblies (Figure 1).

In the aforementioned figure, H, L_b and L_o denote the column height, the beam length and the unsupported length of beam, respectively. General specifications of the specimens are given in Table 1.

The specimen with conventional moment connection is designated by REF and taken as a benchmark to the performance of the other specimens. The test on this specimen was conducted to verify the effect of the RBS moment connection. The two other specimens with RBS connection are designated by RBS2-S and RBS2-B. In the specimen RBS2-S the PZ was strengthened using two doubler plates. No doubler plate was employed in the specimen RBS2-B. The specimens REF and RBS2-S are the same with exception that the RBS moment connection was not employed in the specimen REF.

An IPE450 beam is connected to an IPB300 in the specimens. The column and the beam are 2400 mm and 2750 mm long, respectively. The column ends were hinged to the support and the load was applied on the beam tip. The beam was laterally supported at a distance of 2250 mm from the column face.

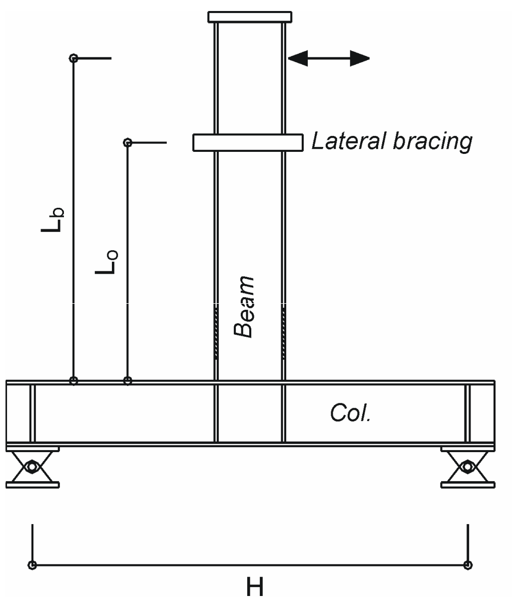


Figure 1 General configuration of exterior subassemblies

Table 1 General specifications of specimens

Specimens	Beam	Column	L_b (mm)	H (mm)	L_o (mm)	Doubler plate th. (mm)
RBS1-S	IPE450	IPB300	2750	2400	2250	6+6
RBS1-B	IPE450	IPB300	2750	2400	2250	-
REF	IPE450	IPB300	2750	2400	2250	6+6

2.1. RBS geometry

The radius cut as shown in Figure 2 was employed for the RBS region. The RBS connections were designed according to the recommendations proposed by Engelhardt et al. (1998). Resulting dimensions for the RBS region are noted in Table 2.

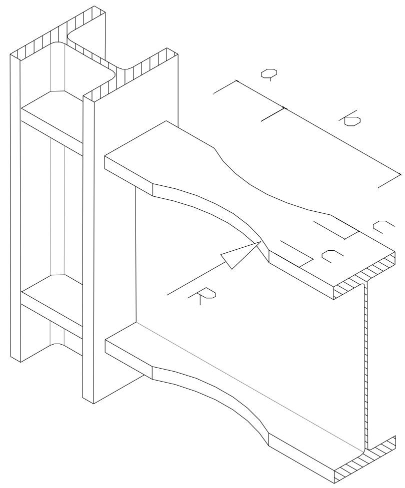


Figure 2 RBS general geometry

Table 2 RBS region dimensions

a(mm)	b(mm)	c(mm)	R(mm)
100	350	45	362.8

2.2. Loading procedure

Each subassembly was loaded on its beam tip by applying quasi-static cyclic displacement according to the SAC (1997), except that the number of the first few elastic cycles was reduced from six to three (see Figure 3). As illustrated in Figure 3, the loading is based on the total beam rotation. The total beam rotation is computed by dividing the total beam tip displacement by the distance to the column face.

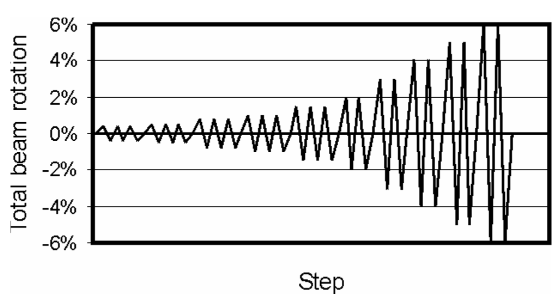


Figure 3 Loading history

2.3. Test set-up

The specimens were installed horizontally on a strong floor. The column ends were hinged to the strong floor. The specimens were located between two reaction frames supporting opposite hydrolic actuators (Figure 4). As one of the actuators applies load, another one was made out of work in order not to resist of the beam tip movement. A lateral support with the details shown in Figure 4, was provided for the specimens beam. The laterally unsupported length of the beam (about 2250mm) was chosen as the maximum unsupported length specified in AISC Seismic Provisions (2005).

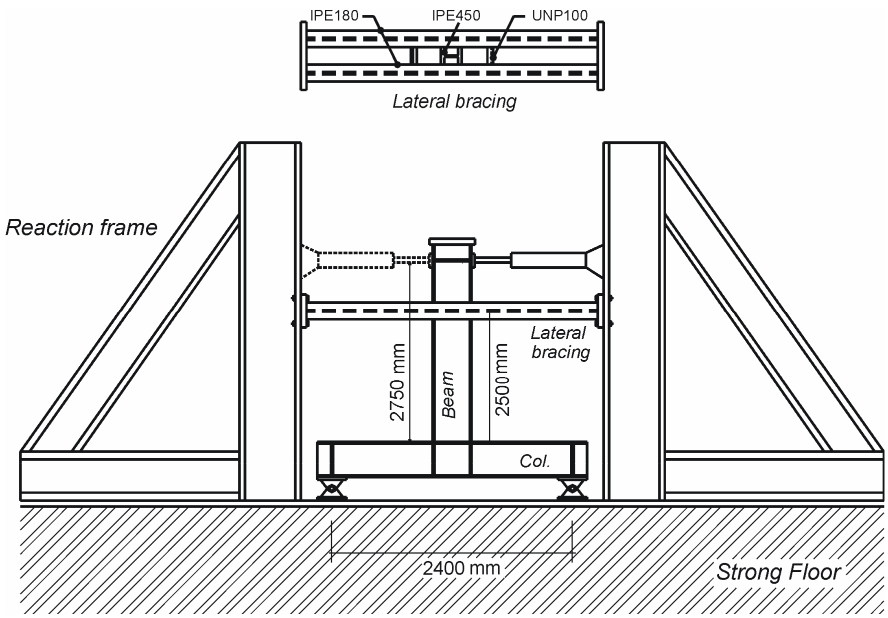


Figure 4 Test set-up

2. PERFORMANCE OF SPECIMENS

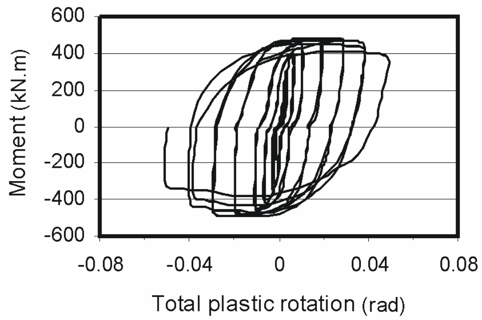
Beam moment-plastic rotation hysteretic response of the specimens are shown in Figure 5. The beam moment was measured at the column face. The plastic rotation was computed by dividing the plastic deflection of the loading point by the beam length for the exterior subassemblies and by the column height for the interior subassemblies.

The specimen RBS2-S possesses stable and increasing hysteretic response up to a plastic rotation of 0.03 rad (Figure 5a). The maximum moment developed at the column face is 485 kN.m. Increasing the displacement amplitude, the beam buckled locally and laterally, resulting in degradation of moment capacity. The moment decreased to 325 kN.m as the plastic rotation reached 0.05 rad. At the end of the testing, the beam flange ruptured due to its severe local buckling.

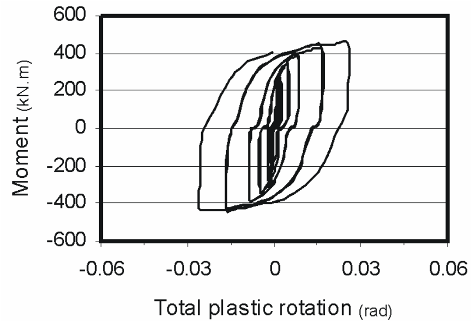
The specimen RBS2-B has increasing and stable hysteretic response (see Figure 5b). The maximum moment developed at the column face is 450 kN.m. No obvious buckling occurred in this specimen. Owing to high shear deformation of the PZ, a severe stress concentration took place at the conjunction of the beam bottom flange and column face, resulting in the fracture of the corresponding weld. The fracture happened at the end of first cycle of 0.03-rad plastic rotation. The specimen RBS2-B underwent less cyclic loading as compared with RBS2-S.

The hysteretic response of the specimen REF, treated as a benchmark to the specimen RBS2-S, denotes that the specimen has stable cyclic behavior up to 0.03-rad plastic rotation (see Figure 5c). The maximum moment developed at the column face is 630 kN.m. Following this step, the specimen buckled severely, leading to a significant reduction of the moment capacity. The plastic rotation capacity of the specimen reached 0.028 rad which is less than that of its coupon specimen, RBS2-S, by 0.01 rad, approximately. This implies the superior promotion caused by RBS moment connection. This indicates RBS connection promotes the rotation capacity of the specimen up to 25%.

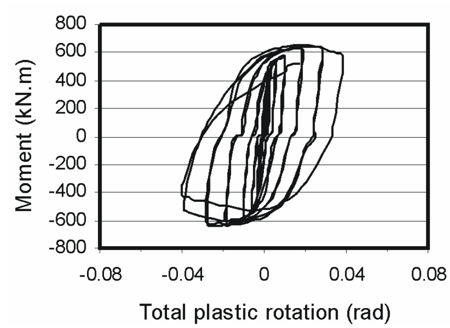
The response of the specimens PZ may be inferred considering the hysteretic loops shown in Figure 6. The figure illustrates the PZ plastic rotation during the cyclic loading. As observed, the PZ of the specimen RBS2-B had the most ductility. The PZ of the specimens RBS2-S and REF did not contribute to energy dissipation, approximately. The extra PZ ductility in the RBS2-B caused high shear deformation in the PZ, resulting in the fracture of the beam-to-column connection.



(a) Specimen RBS2-S

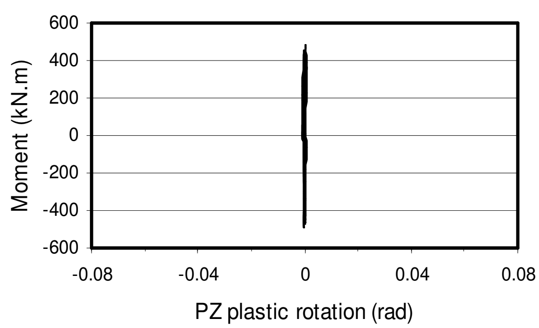


(b) Specimen RBS2-B

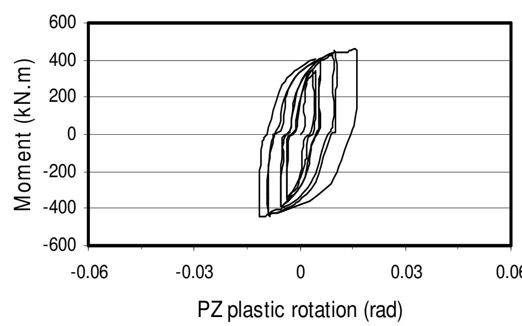


(c) Specimen REF

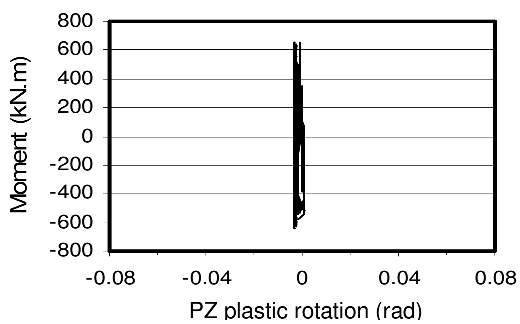
Figure 5 Hystertic response of the specimens



(a) Specimen RBS2-S



(b) Specimen RBS2-B



(c) Specimen REF

Figure 6 Hystertic response of the specimens

A summary of the key parameters denoting the performance of the specimens are noted in Table 3. The table illustrates that the specimens RBS2-B and REF were not able to fulfill the minimum plastic rotation of 0.03 rad specified in AISC Seismic provisions (2005).

Table 3 Beam moment and plastic rotation of specimens

Specimen	Beam moment (kN.m)		Plastic rot. (rad)
	Max.	End of test	
RBS1-S	485	325	0.035
RBS1-B	450	450	0.027
REF	630	430	0.028

3. CONCLUSIONS

In this paper, the results obtained from testing on three steel subassemblies with and without RBS moment connection were noted. The main goals were investigation of cyclic behaviour of the beams with RBS connection and the effect of PZ ductility. Some of the main results are pointed out as below:

1. The plastic rotation of the specimens with strong PZ exceeded 0.035 rad without weld fracture. However, degradation of the moment capacity was significant in these specimens.
2. The balanced PZ showed unexpected high shear deformation. In the specimens with balanced PZ, the high shear deformation caused the beam-to-column weld to fracture.
3. The specimen without RBS connection showed less ductility in comparison with the corresponding specimen having RBS connection. Using RBS connection, the rotation capacity of the specimen was promoted up to 25%. It is worthy to note that the specimen without RBS could not reach the minimum plastic rotation of 0.03 rad.

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REFERENCES

American Institute of Steel Construction (2005). Seismic provisions for structural steel buildings, Chicago.

Calado L. (2000). Cyclic behaviour of beam to column bare steel connection: Influence of column size. In: Mazzolani, editor. Moment resistant connections of steel frames in seismic areas. London: E & FN SPON.

Engelhardt M.D. and Husain A.S. (1993). Cyclic-loading performance of welded flange-bolted web connections. *J Struct Eng* **119(12)**, 3537-3550.

Engelhardt M.D., Winneberger T., Zekany A.J. and Potyraj T.J. (1996) The dogbone connection: Part II. *Modern Steel Construction*, 46-55.

Engelhardt M.D., Winneberger T., Zekany A.J. and Potyraj T.J. (1998). Experimental investigation of dogbone moment connections. *Eng J* **Forth Quarter**, 128-138.

Jones S.L., Fry G.T. and Engelhardt M.D. (2002). Experimental evaluation of cyclically loaded reduced beam section moment connections. *J Struct Eng* **128(4)**, 441-451.

Miller D.K. (1998). Lessons learned from the Northridge earthquake. *Eng Struct* **20(4-6)**, 249-260.

Mahin S.A. (1998). Lessons from damage to steel buildings during the Northridge earthquake. *Eng Struct* **20(4-6)**, 261-270.

Moslehi Tabar A. and Deylami A. (2005). Instability of beams with reduced beam section moment connections emphasizing the effect of column panel zone ductility. *J Construct Steel Res* **61**, 1475-1491.
Plumier A. (1997). The dogbone: back to the future. *Eng J* **Second Quarter**, 61-67.
SAC. (1997). Interim guidelines advisory No.1. *FEMA-267A*, SAC Joint Venture, Sacramento, CA.
Tsai K.C. and Chen W.Z. (2002). Seismic response of steel reduced beam section to weak panel zone moment joints. In: Mazzolani, Tremblay, editors. Behaviour of steel structures in seismic areas. Rotterdam: Balkema.
Uang C.M. and Fan C.C. (1999). Cyclic instability of steel moment connections with reduced beam sections. *Report No. SSRP 99-21*, Department of Structural Engineering, University of California, San Diego, CA.
Uang C.M. and Fan C.C. (2001). Cyclic stability criteria for steel moment connections with reduced beam section. *J Struct Eng* **127(9)**, 1021-1027.
Yu Q.S., Gilton C. and Uang C.M. (1999). Cyclic response of RBS moment connections: Loading sequence and lateral bracing effects. *Report No. SSRP 99-13*, Department of Structural Engineering, University of California, San Diego, CA.
Yu Q.S. and Uang C.M. (2001). Effects of near-fault loading and lateral bracing on the behaviour of RBS moment connections. *Steel & composite Struct* **1(1)**, 145-158.