Full-scale testing of European steel beams with reduced beam section under reversed cyclic loading

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Abstract. Steel beams with reduced beam section (RBS) are often used as part of prequalified connections in seismic regions. However, RBS connections are not as common in Europe. Reasons relate to easiness in fabrication, which requires on-site welding, and the seismic performance qualification. This paper presents preliminary experimental results from a recent testing program that was conducted at the Structures Laboratory at EPFL on full-scale steel beams with RBS. The steel beam featured a European IPE profile. The test specimen discussed herein was subjected to symmetric cyclic lateral loading up to 4 % rad followed by asymmetric lateral loading, which is characteristic of dynamic response prior to structural collapse. The experimental results suggest that inelastic deformations concentrated within the RBS region, as expected. The beam flange-to-steel plate complete joint penetration welds behaved satisfactory throughout the imposed lateral loading history. The test specimen did not experience flexural strength deterioration prior to 4 % rad. Comparisons of the cyclic response of the test specimen with available test data from prior testing programs are also discussed.

Keywords: Reduced beam section, Steel wide flange beam, Full-scale testing, Quasi-static cyclic loading, Fully restrained beam-to-column connections.

1 Introduction

In North America, seismic resistant steel moment-resisting frames (MRFs) often employ fully restrained beam-to-column connections with reduced beam sections (RBS) [1, 2]. Inelastic deformations mostly concentrate within the RBS region, which reduces the inelastic demands on demand-critical welds between the beam flange-to-column flange [3]. The plastic rotation capacity of beams with RBS is primarily controlled by the onset of web local buckling [4–6]. In seismically prone countries in Europe, RBS connections are not as common. Reasons relate to (a) the use of space MRFs that generally favours shallow beam designs for the average building stock, thereby bolted connections may be effective for prefabrication; (b) incomplete information regarding easiness in fabrication; and (c) the lack of formal procedures for the seismic performance pre-qualification in Europe till recently.

This paper presents preliminary experimental results from a testing program that was conducted at the Structures Laboratory at École Polytechnique Fédérale de Lausanne (EPFL) on a full-scale steel beam with RBS. The steel beam featured a European IPE profile. A comparison of the cyclic response of the test specimen with available test data from prior testing programs is also presented.

2 Experimental program

Figure 1 illustrates one of the test specimens. The steel beam featured an IPE550 profile. Both nominal and measured dimensions are summarized in Table 1. The cross-sectional local slenderness of web and flange are h/t_w = 42 (in which h is equal to the depth d after subtracting twice the flange thickness t_f and the k-area fillet radius r of the cross section) and $b_f/2t_f$ = 6.1, respectively. The beam with RBS was welded to a 60 mm thick base plate through complete joint penetration (CJP) groove welds. Referring to Fig. 1, the CJP weld as well as weld access hole was detailed according to [1, 7, 8]. For the weld fabrication, execution class 3 (EXC3) according to the European standard [9] was adopted. The steel material of the beam specimen as well as the base plate was S355J2+M (i.e., nominal yield stress, $f_{y,n}$ = 355 MPa).

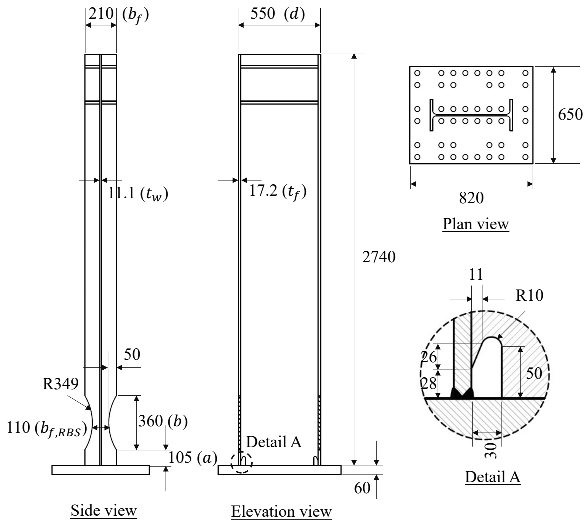


Fig. 1. Test specimen (Units: mm).

Figure 2 shows the test apparatus. The test was conducted in a cantilever fashion. Referring to Fig. 2, the specimen was mounted to a spreader beam, which was connected to the strong floor. The top of the specimen was connected to an actuator through clamping plates with pre-tensioning rods. The specimen was braced laterally by using running beams, which were placed at approximately two-thirds of the height of the specimen (see Fig. 2). The cantilever length of the specimen was 2540 mm.

Table 1. Nominal and measured cross-sectional properties of beam with RBS (Unit: mm) (see Fig. 1 for definitions of each symbol).

	d	b_f	t_w	t_f	$b_{f,RBS}$	b	a
Nominal	550	210	11.1	17.2	110	360	105
Measured	552	208	11.1	16.8	110	356	108

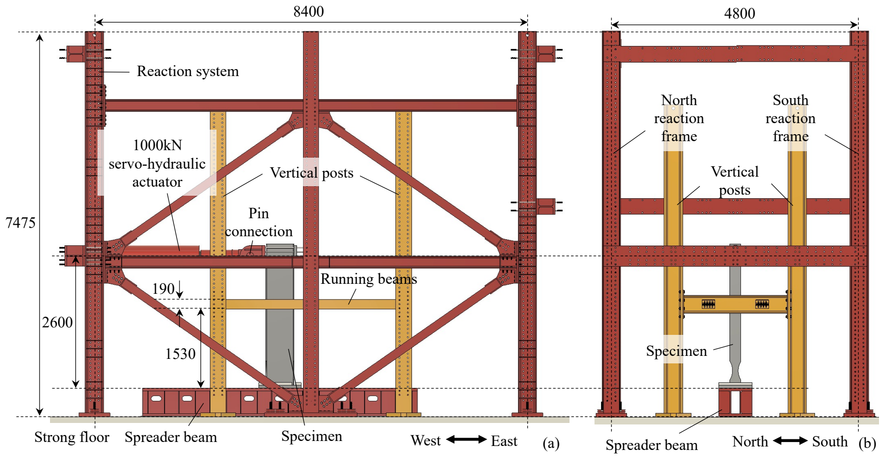


Fig. 2. Test setup with nominal dimension (Unit: mm): (a) elevation view and (b) side view.

The imposed loading protocol, which is shown in Fig. 3, was applied in displacement control. The test was controlled based on the chord rotation of the specimen, which was defined as the effective lateral displacement of the beam tip over the cantilever length. Deformations due to slip and/or rotation of the respective connections and the spreader beam were measured and excluded from the chord rotation of the beam. Referring to Fig. 3, a modified symmetric loading protocol from the AISC pre-qualification protocol [10] was considered up to 4 % rad. A ratcheting protocol was then adopted to represent the response of structural components in MRFs prior to structural collapse [11].

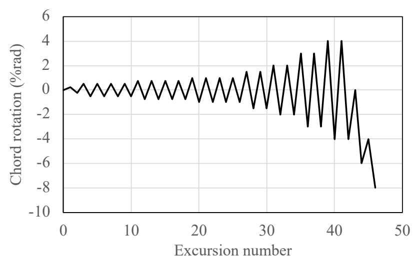
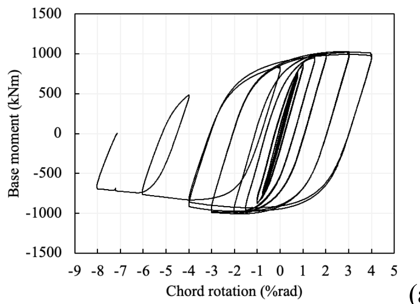


Fig. 3. Loading protocol adopted in the test.

3 Test results

Figure 4a shows the deduced base moment – chord rotation relation of the test specimen. The base moment is defined as the lateral load multiplied by the cantilever length to the top surface of the base plate. The elastic flexural stiffness, which was deduced from the first elastic loading cycle, is 1.09e5 kNm/rad. Flexural yielding occurred during the 0.75 % rad. The hysteretic response of the steel beam was fairly stable up until the 3 % rad lateral drift excursion. Local buckling of the RBS region became evident at the same loading excursion. Figure 5 depicts the onset of local buckling within the RBS region. The maximum base moment (1030 kNm) occurred during the first excursion of the 3 % rad loading cycle. In the subsequent loading cycles, the flexural strength of the beam gradually deteriorated due to the progression of local buckling within web of the RBS region. The experimental results suggest that the specimen met the prequalification limits as per [1] at the second cycle of the 4 % rad lateral drift amplitude.



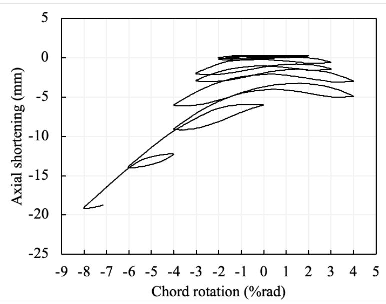


Fig. 4. Test results: (a) Base moment – chord rotation relation and (b) axial shortening – chord rotation relation of the specimen.

In order to assess the behaviour of the test specimen at large deformations associated with structural collapse, an asymmetric loading protocol was adopted, and the specimen was pushed laterally up to 8 % rad. While lateral bracing was installed as shown in Figure 2, the test ended due to appreciable beam twisting. Flexural yielding and local buckling within the RBS region reduced the torsional resistance of the respective cross section. The flexural strength degraded to about 60 % of the maximum attained base moment as shown in Fig. 4a. Overall, the test results suggest that the beam with RBS exhibited very ductile hysteretic behaviour. No visible sign of cracks was observed within the dissipative zone of the test specimen. Moreover, the adopted weld details along with the design and manufacturing process of the beam-to-base plate connection were found to be adequate.

Figure 4b shows the measured beam axial shortening – chord rotation relation. Axial shortening was mainly attributed to local buckling of the RBS web. However, it should be stated that in real steel MRF buildings, beam shortening is likely to be much less because of the framing action and the presence of the slab [12]. In that respect, the experimental findings are deemed to be on the conservative side due to the overly simplified boundary conditions.

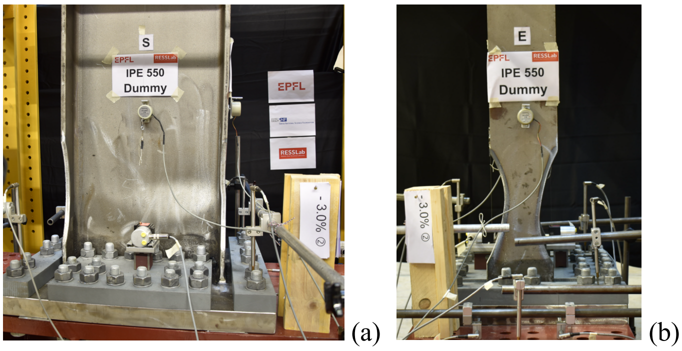


Fig. 5. Deformation of the RBS region at the 2nd excursion of the 3 %rad drift cycle: (a) web; and (b) flange.

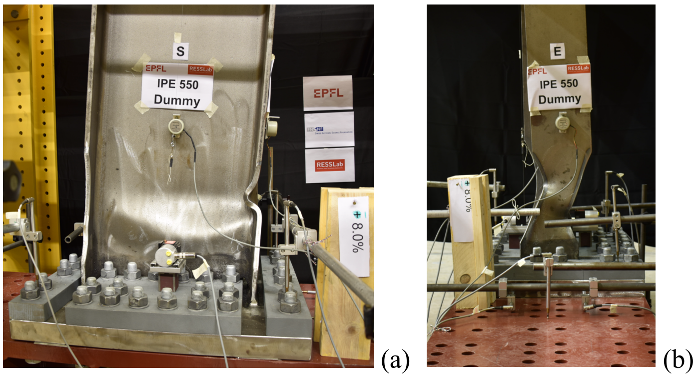


Fig. 6. Deformation of the RBS region observed at the end of testing: (a) web; and (b) flange.

Figure 7 compares the hysteretic behaviour of the test specimen with prior test data from a specimen that featured a similar beam with RBS. Particularly, Specimen 'R2' (in the original literature) featured a H-500x200x10x16 cross section that was made of SN400B steel ($f_{y,n}$ = 235 MPa) [13, 14]. This specimen was subjected to a symmetric cyclic loading protocol. The key geometric properties of both specimens are summarized in Table 2. For comparison purposes, the base moment for each specimen was normalized with respect to the respective maximum attained moment as shown in Fig. 7. The comparison suggests that the onset of local buckling within the RBS region is somewhat different although the cross-sectional properties and loading protocols up to the peak strength are similar. Particularly, Specimen 'R2' experienced local buckling at about 5 % rad chord rotation contrary to the 3 % rad of the European IPE550 specimen. This difference is attributed to the difference in steel material grade. The steel material with lower grade tends to have a larger deformation capacity up to the peak response [2, 4]. Other possible reasons relate to the initial geometric imperfections of the respective cross section [15]. Imperfection limits are somewhat different depending on the manufacturing limits per continent [7, 16–19].

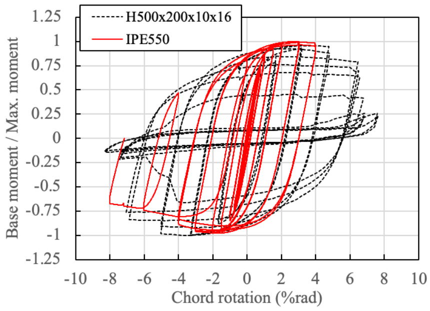


Fig. 7. Comparison of normalized moment – chord rotation relations with test data from prior experiments (test data from [13, 14]).

Table 2. Key features of the specimen 'R2' from [13, 14] and the IPE550 specimen.

	Material	d [mm]	b_f [mm]	t_w [mm]	t_f [mm]	r [mm]	$b_{f,RBS}$ [mm]	b [mm]	a [mm]	h/t_w	$b_f/2t_f$	$b_{f,RBS}/2t_f$
IPE550	S355J2+M	550	210	11.1	17.2	24	110	360	105	42.1	6.1	3.2
R2	SN400B	500	200	10	16	13	110	400	110	44.2	6.3	3.4

4 Summary

This paper summarized in brief the preliminary experimental results of a full-scale steel beam with a reduced beam section (RBS). The steel beam featured an IPE550 cross section. The specimen, which was fabricated in Switzerland based on current European fabrication practice, was subjected to quasi-static cyclic loading. The test specimen exhibited a satisfactory hysteretic behaviour throughout the imposed lateral loading history. More specifically, the specimen did not experience strength deterioration up until a chord rotation of 3 % rad. Moreover, the pre-qualification limits as per AISC-358-16 [1] were satisfied. The behaviour at incipient collapse was also assessed. At a chord rotation of 8 % rad the test specimen reached to about 60 % of its peak strength due to stabilization of the local buckling wave within the RBS region. No evident sign of cracks was observed. Comparisons with past experimental data on steel beams with similar cross-sectional characteristics suggests that while the differences up to the peak response are fairly minor in both specimens, the chord rotations at the onset of strength deterioration are different. This is attributed to the respective steel materials that were used, and the initial geometric imperfections between the two cross sections.

Acknowledgement

This study is based on work supported by the Swiss National Science Foundation (Award No. 200021_169248). The financial support is gratefully acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of sponsors. The authors would like to sincerely thank the personnel of the structural laboratory GIS in EPFL (Mr. Gérald Rouge and Mr. Gilles Guignet), Dr. Albano de Castro e Sousa (postdoc of EPFL), Mr. Hammad El Jisr (PhD student of EPFL), Mr. Nitesh Karmacharya (former visiting engineer of EPFL), Mr. Elias Merhi (master student of EPFL), Mr. Cesar Ramirez (former summer student at EPFL from the university of Texas at El Paso) for their invaluable assistance in conducting the experiment series.

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