The 14th World Conference on Earthquake Engineering
October 12-17, 2008, Beijing, China

EXPERIMENTAL EVALUATION OF NONLINEAR REPLACEABLE LINKS IN ECCENTRICALLY BRACED FRAMES AND MOMENT RESISTING FRAMES

Nabil Mansour[1], Yunlu Shen[1], Constantin Christopoulos[2] and Robert Tremblay[3]

[1] *Graduate Student Researcher,* [2] *Associate Professor, Dept. of Civil Engineering, Univ. of Toronto, Canada*
[3] *Professor, Dept. of Civil, Mineral and Geological Engineering, Ecole Polytechnique, Montreal, Canada*
Email: c.christopoulos@utoronto.ca

ABSTRACT :

While steel EBF and MRF structures designed according to the current seismic design specifications can provide life safety during a design level earthquake, they are expected to sustain significant damage through repeated inelastic deformation and localized buckling. The design of the yielding elements is also interlinked with the design of the connecting beam, which often results in a significant over-design. These drawbacks can be mitigated by introducing nonlinear replaceable links at the locations of expected inelastic action. Ten full-scale links were tested at the University of Toronto following incremental cyclic loading protocols for verification of strength and ductility: six shear links for EBFs and four flexural links for MRFs. Two link types were tested for each system: i) back-to-back channels bolted to the web of the connecting beam, ii) W-sections with end plate connections. All end plate links provided good performance and energy dissipation. The channel links exhibited an improved ductility upon improvement of the web connection design.

KEYWORDS: Replaceable Links, EBF, MRF, Seismic Design, Steel Frames, Seismic Connections

1 INTRODUCTION

Steel eccentrically braced frames (EBFs) and moment resisting frames (MRFs) designed according to current seismic design specifications provide good ductility and energy dissipation under severe cyclic loading. Specific portions of the beam are designed and detailed to sustain large inelastic deformations without significant loss of strength. They act as ductile fuses, dissipating energy through stable hysteretic behavior while limiting the forces transmitted to the other components in the structure. As a result, the basic design philosophy for EBFs and MRFs can be summarized as follows: i) size the fuse to provide the required level of frame strength, ii) detail the fuse to provide the required level of ductility, iii) design and detail all other frame members to be stronger than the forces associated with yielding of the fuse and to satisfy the specified drift requirements.

Because the yielding fuse is a part of the beam in current construction, strength design and drift design of the structure are interlinked, often resulting in over-designed structures. In addition, significant damage can result in the beam from repeated inelastic deformation and localized buckling during a design level earthquake. As the cumulative inelastic action of the structure is unknown, it is difficult to assess the extent of damage and the structure's ability to provide adequate level of safety for any subsequent loading. Furthermore, repair of the beam is very difficult, disruptive, and costly.

The replaceable link concept shown in Figure 1.1, specifically addresses these drawbacks by introducing nonlinear links at the locations of expected inelastic action. It allows for independent control of beam stiffness and required strength, resulting in more efficient structures; it allows welding of critical elements to be done in the shop, considerably improving construction quality and reducing erection time; and it allows for quick inspection and replacement of damaged links following a major earthquake, significantly minimizing the disruption time of the structure. This concept has been previously suggested for by Balut & Gioncu (2003) and Stratan & Dubina (2004).

A research program of combined analytical and large-scale experimental studies is conducted at the University of Toronto and Ecole Polytechnique of Montreal to develop, assess, and validate guidelines for the design of replaceable nonlinear links for EBFs and MRFs. This paper first introduces how the concept is applied to the two structural systems, then presents results from ten full-scale quasi-static tests.

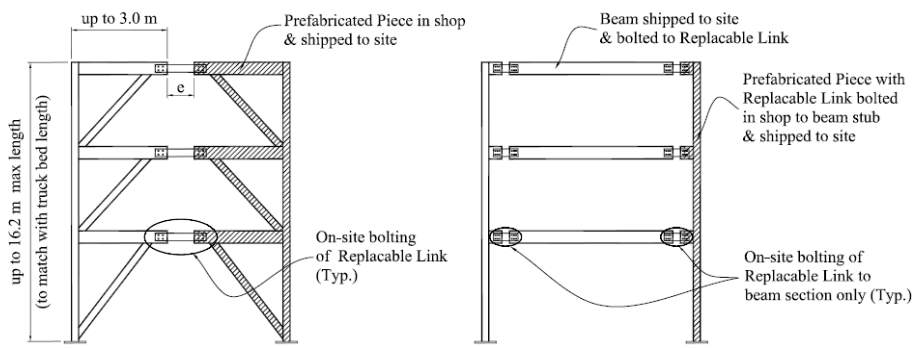


Figure 1.1 Replaceable link concept for: (a) EBFs, (b) MRFs

1.1 EBF design using replaceable links

In well-designed EBFs, the strength and ductility of the frame are directly related to the strength and ductility of the links. Currently EBFs are designed such that the yielding link element is part of the same floor beam. Accordingly, the floor beam must be designed to yield in shear in the link region, but resist the forces due to the strain-hardened link in the portions of the floor beam outside the link. Balancing these two requirements is a lengthy exacting process, most often resulting in oversized link elements, which result in larger forces that must be resisted by all the other EBF members, including the braces, columns, floor slabs, connections and foundations, which ultimately leads to an increase of the overall cost of the structure. Therefore, de-coupling the link by using replaceable links, allows an independent control of strength, stiffness and ductility of the overall behavior of the EBF system. With the proposed replaceable link, the designer has greater flexibility to choose a section (for the yielding link) that best meets the required strength, without automatically changing the floor beam section. Furthermore, built-up sections with relatively thin webs and thick flanges can be used to optimize the design EBFs, and if deemed beneficial, links made of different grades of steel can also be used.

1.2 MRF design using replaceable links

During the 1994 Northridge Earthquake, premature brittle fractures developed at the welded joints of many steel MRFs. In these pre-Northridge moment connections, plastic hinge develops at the column face, causing high strain concentrations near the beam flange groove welds. In the subsequent years, a variety of new beam-to-column connection concepts have been developed. Of these concepts, the reduced beam section (RBS) connection has been most extensively investigated. In a RBS moment connection, the beam flanges are trimmed at a short distance away from the column face, forcing the plastic hinge to form within the reduced beam section, thus limiting the forces experienced at the beam-to-column welds. The replaceable link concept also employs the weakening strategy. Instead of weakening the beam flanges, however, nonlinear links with smaller flexural capacity than the beam are introduced.

1.3 Link Connections

In order for the links to achieve satisfactory performance, the link-to-beam connections must be capable of transferring the forces generated by the fully yielded and strain-hardened links. The depth of the floor beam must also account for erection tolerances to accommodate the bolting operation and to allow for the replaceability of the links after an earthquake. For this study, two link types with different bolted connections were tested for each structural system: i) W-sections with end plate connections, and ii) back-to-back channels with eccentrically loaded web connections. The latter is deemed more economical because of material savings and elimination of complete joint penetration welds. Furthermore, the web connection provides easier access to bolts and is more suitable for upgrade or retrofit of existing systems.

2 EBF AND MRF TEST PROGRAM

Tables 2.1 and 2.2 provide a list of the replaceable link test specimens. The dimensions of the specimens were designed to represent the first storey links of a 5-storey EBF and MRF frames designed according to the 2005 NBCC (NRCC, 2005) specified seismic loads, representative of a high seismic areas in British Columbia, Canada. Sets of two identical specimens were tested for each link type, to assess the replaceability of the links and the performance of the replaced link in comparison to the original. A total of 6 EBF link specimens with comparable shear yield strengths were fabricated. They comprised of two sets of the double channel links with reinforced top and bottom flanges (EBF-1 and 2), and one set of the welded end-plate W-section (EBF-3). Four MRF link specimens with comparable flexural yield strengths were fabricated: two double channel links (MRF-1) and two welded end plate links (MRF-2).

The specimens were designed according to CAN/CSA-S16 (CSA, 2001). The eccentrically loaded web connection was designed using the method of instantaneous center of rotation with the load-deformation relationship developed by Kulak et al. (1987). The MRF welded end plate connections were designed in consultation with Moment Connections for Seismic Applications (CISC, 2005) and Design of Reduced Beam Section Moment Frame Connections (Moore et al. 1999), as well as research by Sumner & Murray (2003). All W-sections were CSA G40.21-350W, the C-sections were CSA G40.21-300W and the plates were CSA G40.21-300W grade steel. The link sections used have Class 1 webs and flanges. All bolted connections used 1" A490 bolts.

Table 2.1 Summary of EBF replaceable link specimens

Specimen No.	Link Section	Connection Type	Connection Reinforcement	V_p (kN)	e (mm)	$e/(M_p/V_p)$
EBF-1A	C310x31	Eccentrically Bolted Web	1/4" web doubler plate	726	680	1.04
EBF-1B			1/4" web doubler plate			
EBF-2A	cut W310x39	Eccentrically Bolted Web	n/a	692	680	1.03
EBF-2B			1/4" web doubler plate			
EBF-3A	W360x101	Bolted End Plate	n/a	722	960	1.38
EBF-3B						

Table 2.2 Summary of MRF replaceable link specimens

Specimen No.	Link Section	Connection Type	Connection Reinforcement	M_p (kN.m)	a / b_{beam}	s / d_{link}
MRF-1A	cut W460x89	Eccentrically Bolted Web	3/8" web doubler plate	896	0.62	1.17
MRF-1B			1/4" web doubler plate			
MRF-2A	W460x128	Bolted End Plate	n/a	1068	0.68	0.69
MRF-2A						

2.1 EBF Test Specimens and Set-up

The back-to-back channel link sections had top and bottom reinforcing plates to increase the flexural capacity. These links were also provided with angles welded to the top and bottom flanges. This was deemed necessary so that the two back-to-back channel sections, with a gap in-between equal to the floor beam web, would act as one unit, reducing the possibility of lateral torsional buckling. To prevent excessive bolt-hole ovalization at the connection due to bolt bearing, the web of the link specimens were reinforced with a 6 mm plate, cut to fit in-between the top and bottom k-lines. The W-section with end plate link connection design and weld detail is similar to that of the moment connection used for the MRF links.

The subassemblage chosen for the experimental program models a portion of a typical chevron bracing EBF system. The test setup was devised to reproduce the force distribution and deformation experienced by the link in an EBF frame. The replaceable link elements are connected at both ends to floor beam sections. The links are subjected to a constant shear force along their length, equal reverse curvature bending moment at the ends, and no axial force. The replaceable shear link specimens qualify if they achieve 0.08 radian plastic link rotation without fracture or severe strength degradation during a standard cyclic loading test. The loading protocol specified in Appendix S6.3 of the 2005 AISC Seismic Provisions for Structural Steel Buildings (AISC, 2005) was used. The tangential deviation between the ends of the connecting beams was used to calculate the link rotation and to control the incremental loading protocol. Details of the link specimens and the test setup can be found in Mansour et al. (2006).

2.2 MRF Test Specimens

Specimens MRF-1A and 1B are double channel links manufactured from W460x89 sections. The W-sections are used to obtain thinner flanges and thicker webs for a given section depth, thereby ensuring that flexural yielding occurs before shear yielding. To form the channels, the flanges on one side of the W-section are cut flush with the web, retaining the full web thickness. End stiffeners are added to the channels to constrain the top and bottom flanges. The back-to-back channels are connected to each other with 5/8’’ bolts that fasten the channel webs, preventing the webs from buckling outwards. Backing plates with a total thickness of 38mm, equal to the thickness of the beam web plus its doubler plates, are inserted between the two channels to prevent inward buckling. To minimize bolt hole deformations and hence improve the connection performance of the links, doubler plates are fillet welded to the connection region. The doubler plates in specimen 1B are welded on all four sides, terminating 5mm away from the plate corners and leaving the corners free. Based on the performance of specimen 1B, doubler plates in specimen 1A were welded all around. Welds are first placed around the plate corners, starting and terminating 50mm from the tip. Straight welds are then placed on all sides of the plates.

Specimens MRF-2A and 2B were fabricated from W460x128 sections. End plates with length and width equal to the outer beam dimensions are connected to the W-sections using complete joint penetration welds.

Important characteristics of the links are summarized in Table 2.2 and detailed illustrations of the links are provided in Figure 2.1.

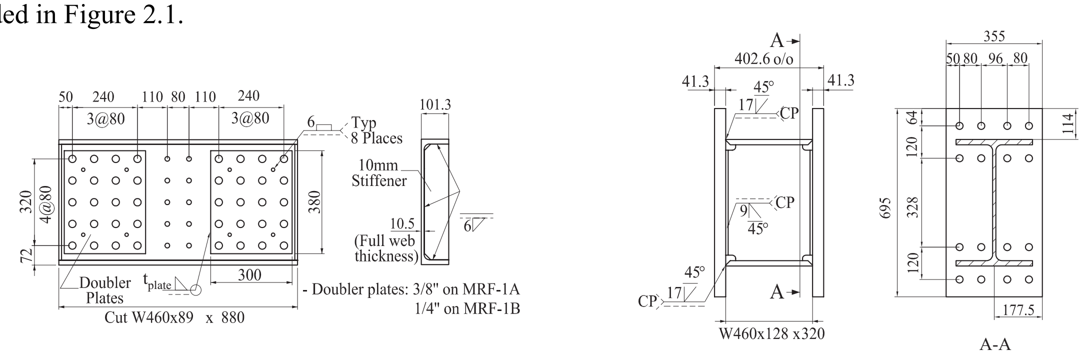


Figure 2.1 MRF replaceable link details

2.3 MRF Test Set-up

The test set-up, shown in Figure 2.3, simulates a typical first floor exterior beam-to-column joint of a five storey building located in Victoria, BC. The assumed inflection points are at the center of the beam, the center of the column above, and $0.7h_s$ from the ground on the column below. Inter-storey drift is imposed on the beam by a 2650 kN Mobile Testing Machine (MTS), 3.51m away from the column centerline. Due to limitations of the strong floor bolt pattern, the distance from the lateral bracing columns to the end of the link is longer than recommended. The same beam and column were used for all four tests. Each end of the beam is designed to connect to one type of link and the column has two welded beam stubs. After the tests were completed for one type of link, the column and beam were inverted for tests of the other link type. Beam-to-column joint details are shown in Figure 2.2.

The specimens were considered satisfactory 0.04 radian storey drift could be achieved without fracture or severe strength degradation during a standard cyclic loading test. The loading protocol specified in Appendix S6.2 of the 2005 AISC Seismic Provisions for Structural Steel Buildings was used. Loading was continued until limitation in the test set-up was reached. The specimens were then cycled at the largest rotation until a severe fracture was observed or until specimen strength degraded below 60% of its peak load.

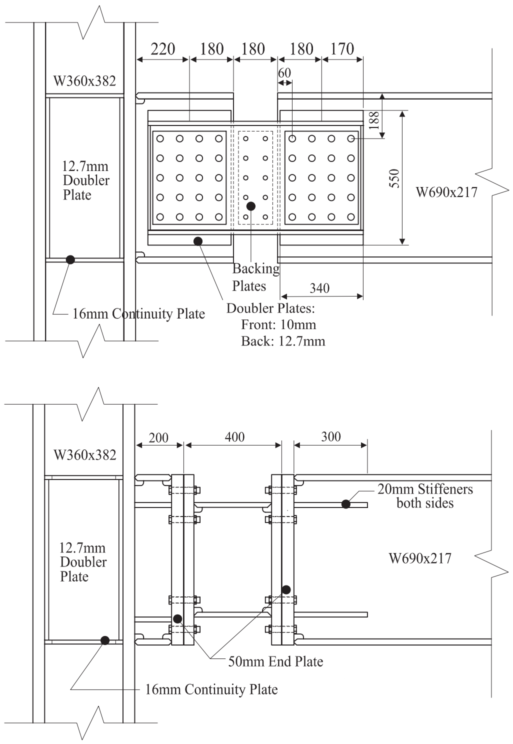


Figure 2.2 MRF beam-to-column joint details

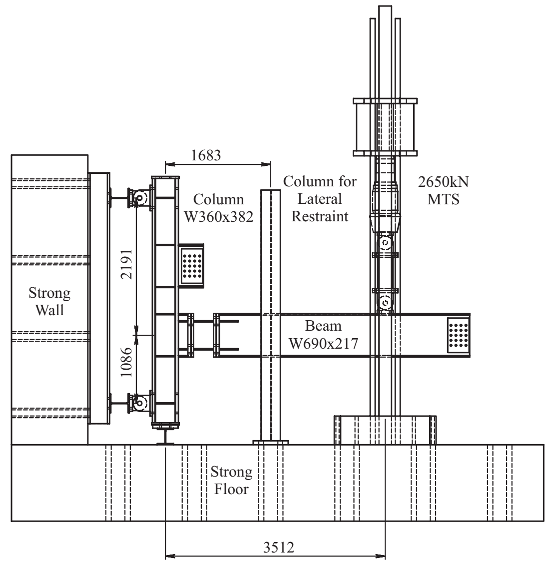


Figure 2.3 MRF test set-up

3 TEST RESULTS

3.1 EBF Test Results

A summary of test results is presented in Table 3.1. The hysteretic responses for specimens EBF-1B and 3B are shown in Figure 3.1 a) and b) respectively.

The behaviour of the end plate links closely resembled that of traditional chevron EBFs with shear link beam sections. Full hysteretic curves were observed. Both specimens had an almost identical loading history response. At the 0.09 radian cycle, cracks were observed at the corners of the link web, initiating from the weld access holes. Crack propagation in the link web extended from the access hole parallel to the flanges then vertical up at the connection interface, until fracture of the web panel at completion of the 0.11 radian cycle.

The double channel shear links experienced bolt slippage at the link connections. As a result, pinching was observed in the hysteretic curves. Connection rotation was responsible for 35% of link rotation in EBF-1A, and increased to 54% in EBF-1B. The bolt holes in the floor beam were already ovalized from the original link specimen. This indicated that thicker web reinforcement plates for the floor beam are required to prevent bolt hole elongation.

The hysteretic energy dissipated by the replaceable shear link as it undergoes inelastic behavior was compared to that dissipated by an elastic perfectly-plastic (EPP) link. The double channel links dissipated 70% of the EPP link. This is due to the observed pinching in the hysteresis due to bolt slippage in the connection. Nonetheless, as the bolts were pretensioned, energy was also dissipated as the webs of the link and the floor beam slipped with respect to each other. However, the W-section links with the end plate dissipated hysteretic energy slightly less than the EPP link. The end plate connection remained rigid throughout the test, and all the plastic link rotation was due to the inelastic link shear deformation.

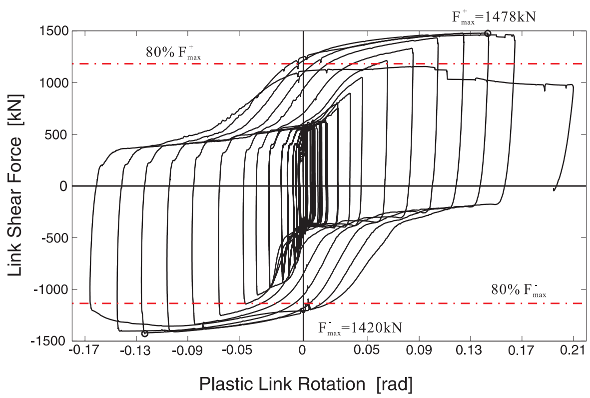
Table 3.1 Summary of EBF test results

Specimen No.	Total Link Rotation	
	γ	γp
EBF-1A	0.131	0.125
EBF-1B	0.214	0.210
EBF-2A	0.073	0.059
EBF-2B	0.11	0.103
EBF-3A	0.11	0.104
EBF-3B	0.11	0.105

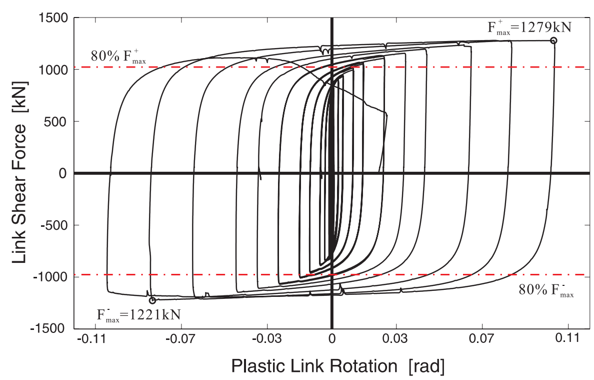
Table 3.2 Summary of MRF test results

Specimen No.	Total Storey Drift *	
	θ	θp
MRF-1A	0.07	0.059
MRF-1B	0.06	0.049
MRF-2A	0.04	0.031
MRF-2B	0.04	0.031

* Total storey drift reached in last complete cycle before strength deteriorated below 80% of peak load

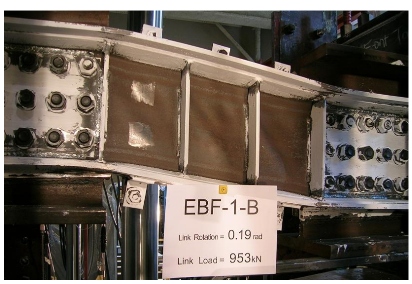


(a) EBF-1B



(a) EBF-3B

Figure 3.1 Hysteretic response of EBF specimens

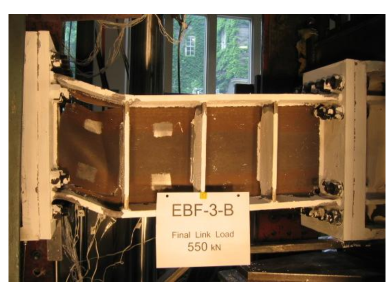


(a) EBF-1B, 0.19 radian rotation

(b) EBF-1B, end of test

Figure 3.2 Tested EBF-1B double channel shear links

(a) MRF-3B, 0.11 radian rotation

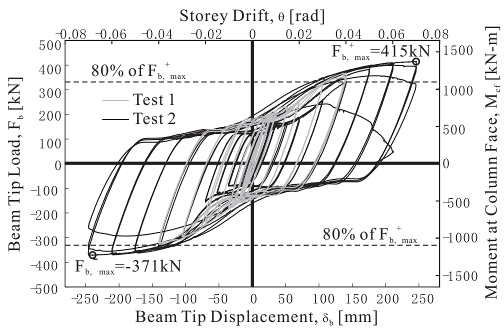


(b) EBF-3B, end of test

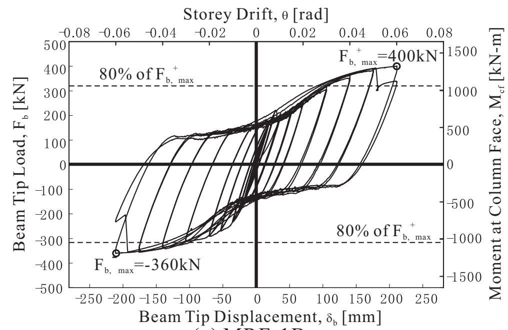
Figure 3.3 Tested EBF-3B welded end-plate shear links

3.2 MRF Test Results

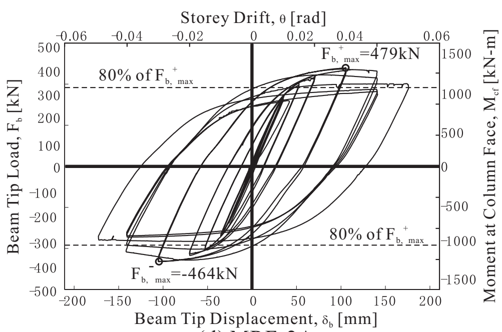
A summary of test results is presented in Table 3.2. The specimen hysteretic responses are shown in Figure 3.4.



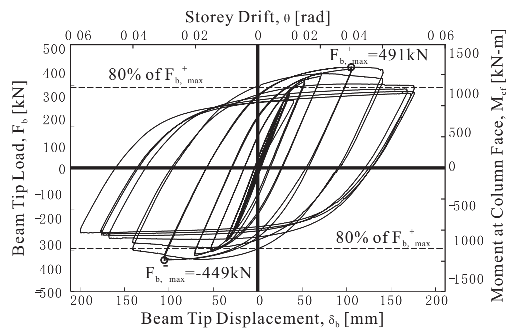
(b) MRF-1A



(c) MRF-1B

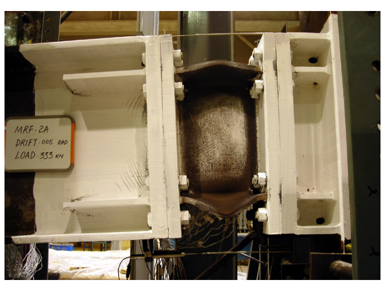


(d) MRF-2A

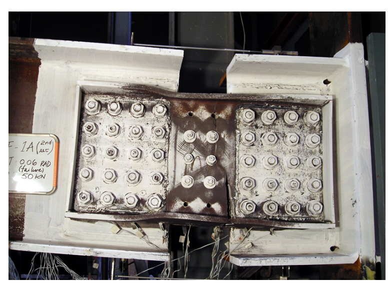


(e) MRF-2B

Figure 3.4 Hysteretic response of MRF specimens

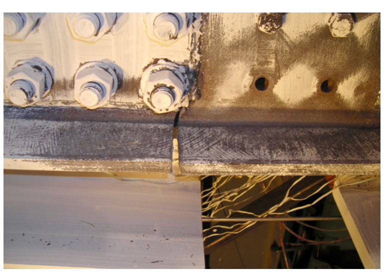


(a) MRF-2A, 0.05 radian storey drift

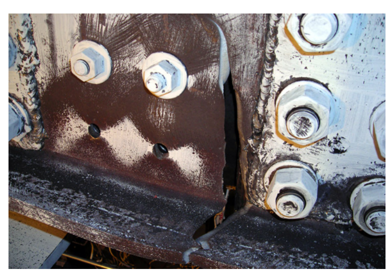


(b) MRF-1A, end of test

Figure 3.5 Deformed links



(a) MRF-1B, bottom flange fracture



(b) MRF-1A, tear in web and bottom flange

Figure 3.6 Damage details in channel links

The behaviour of the end plate link subassemblages closely resembled that of MRFs with reduced beam sections. Full hysteretic curves were observed. Due to local flange and web buckling, the specimen experienced significant strength degradation (below 80% of peak load achieved) after completing at least one cycle at 0.04

radian storey drift. At this drift level, link deformations contributed to approximately 90% of the total storey drift. Hairline cracks were observed near the weld access holes and the flange-to-end plate welds. No performance degradation was observed in the replacement link in comparison to the original link. Figure 3.5 a) shows specimen MRF-2A at 0.05 radian storey drift.

The double channel link subassemblages experienced bolt slippage at the link connections. As a result, pinching was observed in the hysteretic curves. Specimen 1B was tested first. After reaching 0.06 radian storey drift, it experienced brittle fractures across its flanges, originating from the corners of the doubler plate. When improved doubler plate welding details were used in specimen 1A, the link experienced strength degradation resulting from slowly propagating cracks on link flanges and webs. Figure 3.6 compares the damage in the two double channel links. At 0.04 radian storey drift and above, link deformations contributed to approximately 90% of the total storey drift. The rotations in the connections generated up to 55% of the link rotation. Specimen 1A was cyclically loaded according to the protocol up to 0.04 radian drift, returned to zero load with 0.0075 radian residual drift, and loaded according to the protocol again up to 0.07 radian storey drift. It met the acceptance criteria during both tests, indicating that if the double channel link was not replaced after a design level earthquake, it could potentially provide satisfactory performance during a subsequent seismic event. Figure 3.5 (b) shows specimen 1B at the end of the test.

4 CONCLUSIONS

Results from the full-scale experiments indicate that EBFs and MRFs with nonlinear replaceable links can provide strength and ductility equivalent to structures built following current practice. Links with end plate connections exhibited behavior very similar to that of the traditional construction, where the yielding fuse is an integral component of the floor beam. Connection design (including bolt bearing deformation and doubler plate welding details) was found to be critical for the double channel links. For both structural systems, the end plate links exhibited higher energy dissipation capacity than the double channel links. Because of their connection flexibility, on the other hand, the double channel links enjoyed significantly higher rotational capacity.

ACKNOWLEDGEMENTS

Financial support for this study was provided by the Steel Structures Education Foundation (SSEF), and the Canadian Institute of Steel Construction (CISC). Test specimens were donated by Walter Inc., Hamilton, Canada. This in-kind support is kindly appreciated and gratefully acknowledged.

REFERENCES

AISC (2005). Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc., Chicago, IL, USA.

Balut, N. and Gioncu, V. (2003). Suggestion for an improved 'dog-bone' solution. *Proc. of 2003 STESSA Conf.,* Naples, Italy, 129-134.

CISC (2005). Moment Connections for Seismic Applications, CISC, Willowdale, Ontario, Canada.

CSA. (2001). CAN/CSA-S16 Limit States Design of Steel Structures. Canadian Standards Association, Willowdale, Ontario, Canada.

Kulak, G.L., Fisher, J.W., Struik, J.H.A. (1987). Guide to Design Criteria for Bolted and Riveted Joints. AISC. Chicago, Illinois, USA.

Mansour, N., Christopoulos, C., Tremblay, R. (2006). Seismic design of EBF steel frames using replaceable nonlinear links. *Proc. of 2006 STESSA Conf.,* Yokohama, Japan, 745-750.

Moore, K.S., Malley, J.O., Engelhardt, M.D. (1999). Design of Reduced Beam Section Moment Frame Connections. Structural Steel Educational Council. Moraga, California, USA.

Stratan, A. and Dubina, D. (2004). Bolted links for eccentrically braced steel frames*, Proc. of the Fifth International Workshop on Connections in Steel Structures V*, 223-232. Amsterdam, the Netherlands.

Sumner, E.A. and Murray, T.M. (2003).Unified Design of Extended End-Plate Moment Connections Subject to Cyclic Loading. Faculty of Virginia Polytechnic Institute and State University. Blacksburg, Virginia, USA.