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Cyclic loading behavior of EBF links constructed of ASTM A992 steel

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Abstract

Cyclic loading tests were conducted to study the behavior of link beams in steel eccentrically braced frames. A total of thirty-seven link specimens were constructed from five different wide-flange sections, all of ASTM A992 steel, with link length varying from short shear yielding links to long flexure yielding links. The occurrence of web fracture in shear yielding link specimens led to further study on the cause of these fractures. Since the link web fracture appeared to be a phenomenon unique to modern rolled shapes, the potential role of material properties on these fractures is discussed. Based on the test data, a change in the flange slenderness limit is proposed. The link overstrength factor of 1.5, as assumed in the current U.S. code provisions, appears to be reasonable. The cyclic loading history used for testing was found to significantly affect link performance. Test observations also suggest new techniques for link stiffener design and detailing for link-to-column connections.

Keywords: Cyclic tests; Steel structures; Seismic design; Flange slenderness ratio; Loading history; Fracture; k-area; Eccentrically braced frame

1. Introduction

The design intent for a seismic-resistant steel Eccentrically Braced Frame (EBF) is that inelastic action under strong earthquake motion is restricted primarily to the *links*. Therefore, the EBF design procedure prescribed in the *2005 AISC Seismic Provisions for Structural Steel Buildings* [1] relies on an understanding of link behavior under severe cyclic loading. The *AISC Seismic Provisions* contain U.S. building code rules for detailing steel structures, including EBFs, for seismic resistance. The current building code rules for EBFs in the *AISC Seismic Provisions*, including link design, link rotation limits, and link overstrength factors, were developed from rather extensive experimental studies conducted almost exclusively on wide-flange shapes of ASTM A36 steel [2]. However, structural steel shapes most commonly used in the U.S. today are produced according to the newer ASTM A992 standard, which provides for a higher yield and tensile strength than A36 steel.

The move to A992 steel raised concerns regarding the appropriateness of the flange width–thickness limits for EBF link sections. A limit of $0.30(E/F_y)^{1/2}$ was traditionally specified for the flange width–thickness ratio of EBF links. This flange slenderness ratio corresponds to 8.5 for A36 steel (with minimum specified yield strength of $F_y = 250$ MPa) and 7.2 for A992 steel (minimum specified $F_y = 345$ MPa). A number of rolled wide-flange shapes meet the flange slenderness limit of 8.5 but do not meet the limit of 7.2, and thus are disqualified from use as EBF links by the traditional flange slenderness limit. Meanwhile, the effect of flange slenderness ratio on link behavior has not been explicitly addressed in previous research. A secondary concern was the appropriateness of link overstrength factors used in the capacity design procedure for EBFs. Link overstrength is defined as the maximum shear force developed in the link divided by the plastic shear strength of the link. While the *2005 AISC Seismic Provisions* implicitly assume a link overstrength factor of 1.5, recent tests on large built-up shear links for use in bridge applications showed overstrength factors of nearly 2 [3,4]. This has led to concerns that current overstrength factors may be unconservative, particularly for shapes with heavy flanges, where shear resistance of the flanges contributes significantly to overstrength.

An experimental research program was conducted at the University of Texas at Austin. The initial objective for this program was to examine flange buckling and overstrength in

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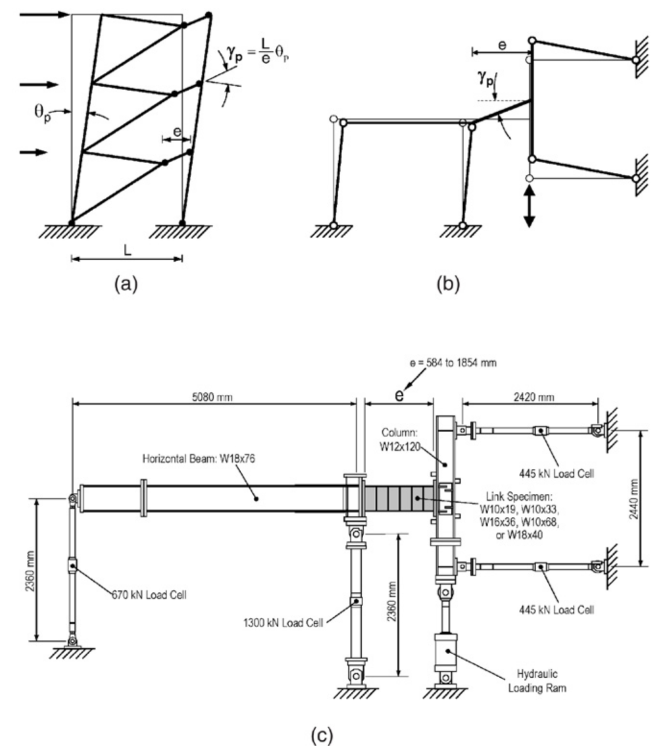

Fig. 1. Test setup: (a) energy dissipation mechanism of EBF; (b) schematic representation of test setup; and (c) details and dimensions.

links constructed of A992 steel. Link specimens with various sections and lengths were tested for this purpose. As discussed later, many of these initial specimens failed prematurely due to fracture of the link web. This type of failure mode was not typically reported in earlier tests [5–7], and thus motivated further testing to investigate the cause of the link web fracture.

Failure of the initial specimens to meet rotation requirements led to questions concerning the loading protocol for testing EBF links specified in the *2002 AISC Seismic Provisions* [8]. A total of thirty-seven link specimens were tested during the course of this investigation. In the rest of this paper, the test specimens and test procedure is discussed, followed by an overview of test results. Key observations pertaining to loading history, link web fracture, and link end welds are discussed. The test data is used to evaluate the flange slenderness limit, inelastic rotation limit, and overstrength factor for EBF links prescribed in the *2005 AISC Seismic Provisions* [1]. Suggestions on new techniques for link stiffener design and new detailing for link-to-column connections are made. While some results from the research program have been discussed in a previous publication by the authors [9], these results are also mentioned in this paper for completeness.

2. Experimental program

2.1. Test setup

A test setup was devised to reproduce the force and deformation environment imposed on a link in an EBF with one end of the link attached to a column, as shown in Fig. 1(a). Fig. 1(b) illustrates that the kinematics of the test setup follows the energy dissipation mechanism of the EBF. Full details and dimensions of the test setup are shown in Fig. 1(c). The link length is indicated in the figures by the letter *e*. The link specimens were welded to heavy end plates at each end, as shown in Fig. 2. The end plates were, in turn, bolted into the setup, between the vertical column and horizontal beam.

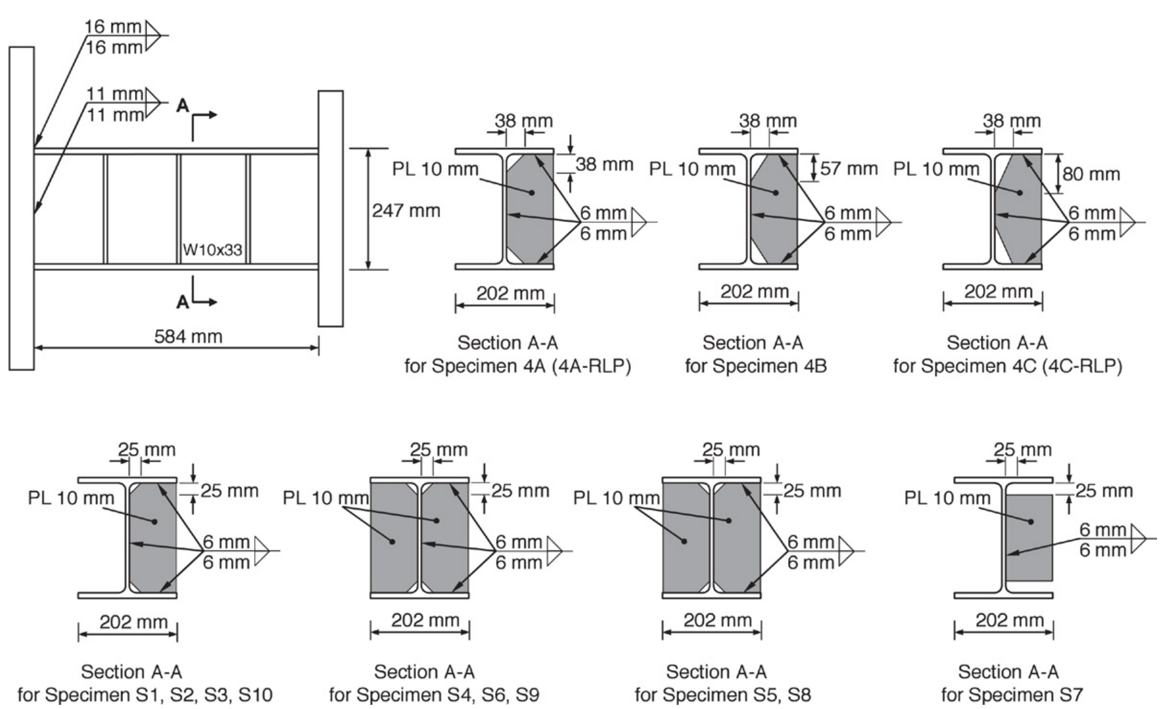

Fig. 2. Details of selected link specimens.