PROOF-OF-CONCEPT TESTING OF A LATERALLY STABLE ECCENTRICALLY BRACED FRAME FOR STEEL BRIDGE PIERS

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Abstract

This paper describes the design and testing of a proof-of-concept eccentrically braced frame specimen that utilizes a hybrid rectangular shear link that is not laterally braced. Equations used for design are given and references for their derivations are provided. The quasi-static cyclic proof-of-concept testing is described and results are reported. Stable and full hysteretic loops were obtained and no signs of flange, web, or lateral torsional buckling were observed. The link was subjected to 0.15 radians of rotation in the final cycle, which is almost twice the maximum rotation allowed in building codes for links with I-shaped cross-sections. Although the final failure mode was fracture of the bottom link flange, the large rotations achieved were well above what would be required in a seismic event, indicating that hybrid rectangular links without lateral bracing of the link can indeed be a viable alternative for applications in steel bridge piers in seismic regions.

Introduction

Eccentrically braced frames have been shown to exhibit excellent seismic performance. However, eccentrically braced frames have had limited use in the steel piers of bridges due to the difficulty to provide the lateral bracing required to prevent possibility of lateral torsional buckling of the link. An eccentrically braced frame system in which lateral bracing of the link can be avoided, would make it desirable in the context of bridge seismic design and retrofit especially since eccentrically braced frames have been shown to exhibit excellent seismic performance (Roeder and Popov (1977), Hjelmstad and Popov (1983), Kasai and Popov (1986a), Kasai and Popov (1986b), Engelhardt and Popov (1989), among others).

From this motivation the concept of an EBF utilizing a rectangular hybrid cross-section for the link is explored (hybrid in this case meaning the yield stresses of the webs and flanges may be different). Rectangular cross-sections inherently have more torsional stability than I-shaped cross-sections and may not require lateral bracing.

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This paper describes the design and testing of a proof-of-concept EBF having a link with a hybrid rectangular cross-section. First, equations used for design, including plastic shear force, plastic moment, stiffener spacing, and the limiting flange compactness ratio are given and references for their derivations are provided. Then, the specimen design and test setup are described. Finally, the experimental results are discussed and conclusions are drawn.

Design Equations

A link with a hybrid rectangular cross-section is shown in Figure 1. Assuming the moment on the section is less than the reduced plastic moment (described below), the plastic shear force, V_p, for the cross-section is (Berman and Bruneau, 2004):

$$V_p = \frac{2}{\sqrt{3}} F_{yw} t_w \left(d - 2t_f\right) \tag{1}$$

where F_{yw} is the yield stress of the webs, t_w is the web thickness, d is the depth of the section, and t_f is the flange thickness. The plastic moment, M_p, for the cross-section shown in figure is:

$$M_p = F_{yf} t_f \left(b - 2t_w\right)\left(d - t_f\right) + F_{yw} \frac{t_w d^2}{2} \tag{2}$$

where F_{yf} is the yield stress of the flanges, b is the section width, and other terms are as previously defined. This plastic moment may also be reduced the presence of the plastic shear force, as a simple way of accounting for shear-moment interaction in the presence of the full plastic shear. The reduced plastic moment, M_{pr}, can be written as:

$$M_{pr} = F_{yf} t_f \left(b - 2t_w\right)\left(d - t_f\right) + 2F_{yw} t_f t_w \left(d - t_f\right) \tag{3}$$

As with links having I-shaped cross-sections, hybrid links with rectangular cross-sections fall into one of three categories, shear links, intermediate links, and flexural links. Classification of links can be done using the normalized link length, ρ, defined as $e/(M_p/V_p)$, where e is the link length. Links having $\rho \leq 1.6$ are shear links and the inelastic behavior is dominated by shear yielding of the webs. Links having $1.6 < \rho \leq 2.6$ are intermediate links, for which inelastic behavior is a mix of shear and flexural behavior, and links having $2.6 < \rho$ are flexural links where inelastic behavior is dominated by flexural yielding. These ranges are the same as those for I-shaped links, since there is no expected difference in the factors used to determine them, namely, overstrength and strain hardening (Berman and Bruneau, 2004). For I-shaped links, the AISC Seismic Provisions (AISC, 2002) limit the inelastic link rotation for shear links and flexural links to 0.08 rads and 0.02 rads, respectively, with linear interpolation based on normalized link length to be used for intermediate links. At this point there is no data indicating these limits should be different for hybrid rectangular links.

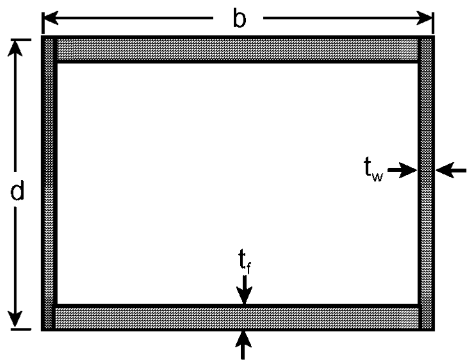

Figure 1. Hybrid Rectangular Link Cross-Section

Stiffener spacing limits for links with hybrid rectangular cross-sections have been proposed in Berman and Bruneau (2004). Their form is similar to that given in the AISC seismic provisions for I-shaped links which were derived by Kasai and Popov (1986a). It is proposed that stiffeners for shear links (maximum inelastic rotation of 0.08 rads) satisfy:

$$\frac{a}{t_w}+\frac{1}{8}\frac{d}{t_w}=20 \tag{4}$$

where a is the stiffener spacing and all other terms are as previously defined. For intermediate links with a maximum link rotation of 0.02 rads, the stiffeners should satisfy:

$$\frac{a}{t_w}+\frac{1}{8}\frac{d}{t_w}=37 \tag{5}$$

and linear interpolation can be used to find the maximum stiffener spacing for intermediate links with rotations between 0.08 and 0.02 rads. For flexural links, stiffeners are proposed at $1.5b$ from each end of the link as they are for I-shaped cross-sections.

Limiting web and flange compactness ratios are given for hollow structural sections (HSS) in the AISC seismic provisions as:

$$0.64\sqrt{E_s / F_y} \tag{6}$$

where E_s is Young's modulus for steel and F_y is the web or flange yield stress as appropriate. In the absence of finite element or experimental data for hybrid rectangular links, it is recommended that those limits be used. However, those limits are based on experimental results for HSS braces subject to repeated compression buckling. Therefore, they may not apply to the webs and flanges of hybrid rectangular links which are subject to large shear forces and bending moments. Berman and Bruneau (2004) derived the following limit for flange compactness based on inelastic plate buckling:

$$1.02\sqrt{E_s / F_y} \tag{7}$$

while for web compactness of shear links, it appears that it may be possible to satisfy either the Eq. (6) or Eq. (4) but not necessarily both. Again, more experimental evidence is necessary to assess the applicability of these limits before they can be codified.

The necessity for using hybrid rectangular cross-sections arises from the short link lengths required for HSS shapes that are listed in the AISC LRFD Manual of Steel Construction (AISC, 1998) to be shear links (i.e. have $\rho \leq 1.6$). The maximum length for a shear link, that also meets the compactness limits of Eq. (6), would be 460 mm and would be obtained with a HSS 250x250x16. Considering a 7.3 m wide by 3.7 m tall frame the drift at a plastic rotation of 0.08 rads is only 0.5% from (Bruneau et al., 1998):

$$\theta = \gamma \frac{e}{L} \tag{8}$$

where θ is the frame drift, γ is the link rotation, L is the frame width, and e is the link length. There may be instances where ductility demand exceeds this drift value. Additionally, a short link has a greater stiffness, which can translate into increased seismic demands on the surrounding framing and foundation. Furthermore, as a minor point, short links can cause congested details and fabrication difficulties. Therefore, hybrid rectangular cross-sections (Figure 1), which offer the ability to change the M_p over V_p ratio by changing the web and flange thicknesses independently, are desirable since they will allow longer link lengths while still having a shear link and the desired shear strength.

Specimen Design and Test Setup

The proof-of-concept EBF with a hybrid rectangular shear link was designed to be as large as possible considering the constraints of the available equipment in the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University at Buffalo (UB). Quasi-static cyclic loading was chosen and the maximum force output of the actuator available (1115 kN) was divided by 2.5 to account for the possibility of obtaining material with a higher than specified yield stress as well as strain hardening, making the design base shear 445 kN. The general test setup is shown in Figure 2. Assuming the moments at the middle of the link and clevises are zero, the design link shear can be found to be 327 kN for the dimensions shown. To design the link, the equations described above (i.e, link shear force, Eq. (1), and the AISC web and flange compactness limits Eq. (6)) were used in conjunction with enforcing a minimum link length such that a drift of at least 1% corresponds to a maximum link rotation of 0.08 rads and assuring $\rho<1.6$ (i.e., maintaining a shear link). The following link dimensions were selected: $d = b = 150$ mm, $t_f = 16$ mm, $t_w = 8$ mm, and $e = 460$ mm. A yield stress of 345 MPa was assumed for the steel during the design process and ASTM A572 Gr. 50 steel was specified for fabrication of the link beam which gave an anticipated link shear of 381 kN.

The framing outside the link was designed using capacity principles. Factors to account for the difference between specified and expected (i.e., mean) yield stress of the link material as well as strain hardening were incorporated, resulting in the member sizes shown in Figure 3.

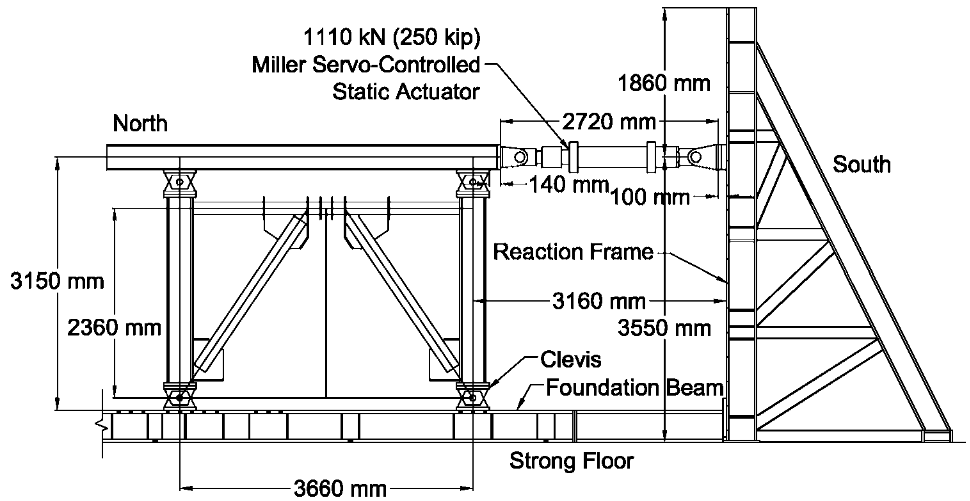

Figure 2. Test Setup

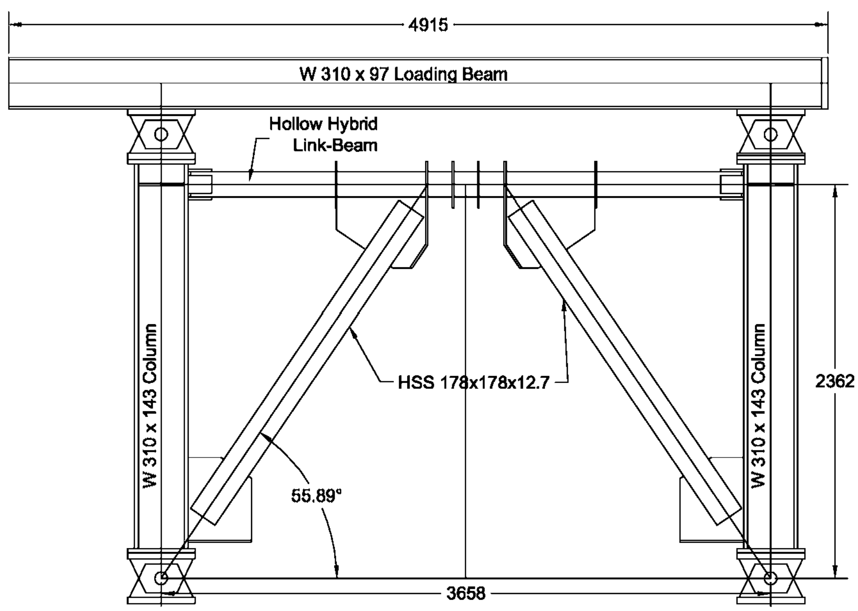

Figure 3. Test Specimen with Dimensions

Link stiffeners were placed at a spacing of 152 mm such that Eq. (5) was satisfied. A full penetration groove weld with the flange beveled at 45° was specified to assemble the link cross-section. The connection of the link to the eccentric braces was designed using standard procedures for the expected link capacity. Link and eccentric brace connection details are shown in Figures 4 and 5.

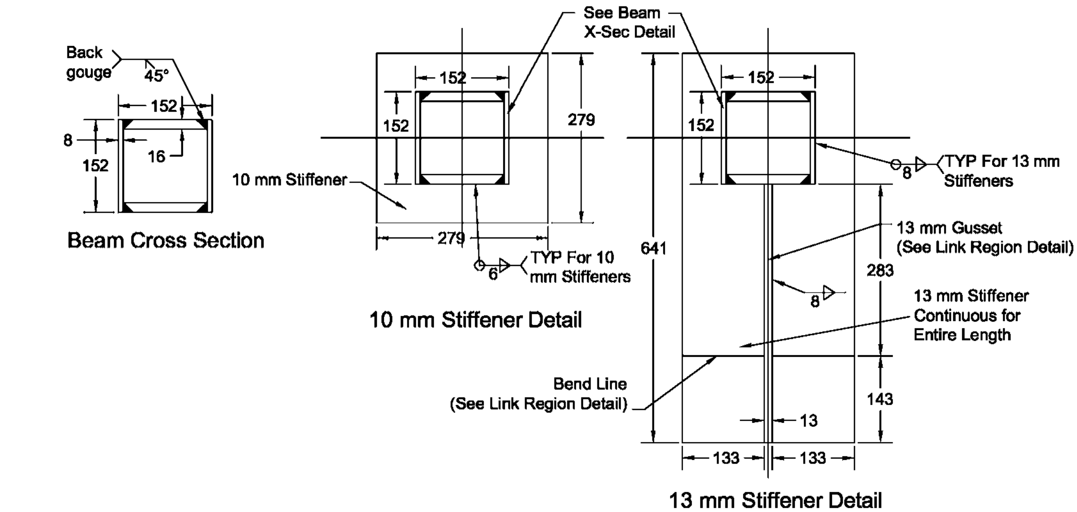

Figure 4. Link Cross-Section Details

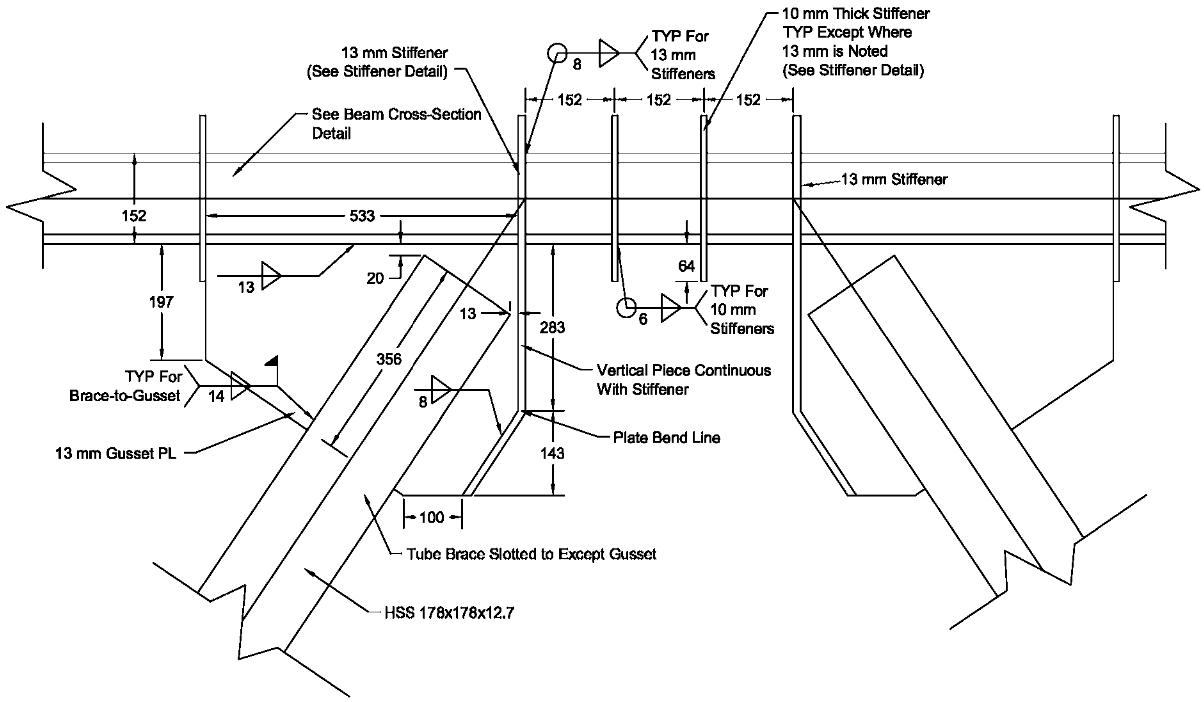

Figure 5. Link and Eccentric Brace Connection Details

Coupon tests of the delivered web and flange material for the link were performed and the results are shown in Figures 6a and 6b. As shown, the web material did not exhibit much of a yield plateau and the yield stress was found to be 448 MPa, which is significantly larger than the 345 MPa specified. The flange material had a defined yield plateau and a yield stress of 393 MPa. Using these yield stresses M_p and V_p were found to be 157.6 kN-m and 495 kN, respectively.

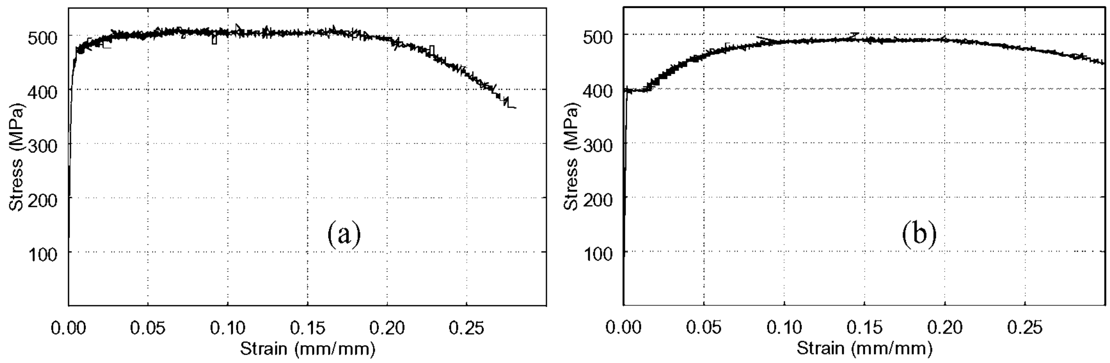

Figure 6. Coupon Test Results for (a) Web Material and (b) Flange Material

The specimen was instrumented with strain gauges and temposonic magnetic strictive transducers (temposonics). Strain gauges were placed such that the forces and moments in the framing members could be obtained as well as on the web and flanges of the link so that specimen yield could be verified. Temposonics were placed so that both link rotation and frame drift could be obtained.

Loading followed the ATC-24 protocol (ATC, 1992) which is illustrated in Figure 7. Force control was used for cycles up to $1\delta_y$ while displacement control was used after that. The frame drift was used as the deformation control parameter.

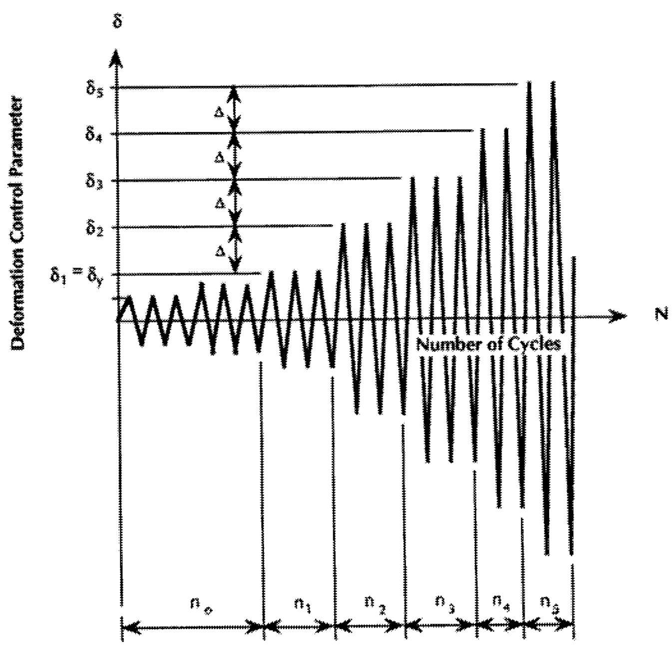

Figure 7. Loading Protocol

It should be noted that no lateral bracing was provided to the link, beam segments outside the link, eccentric braces, or columns. However, for safety, the loading beam shown in Figures 2 and 3 was laterally braced at points above the columns as shown in Figure 8. Figure 9 shows the completed test setup.

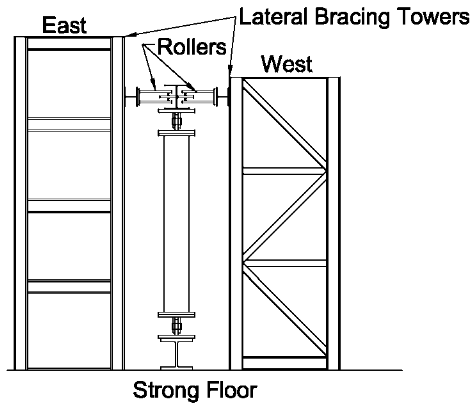

Figure 8. Lateral Bracing of Loading Beam

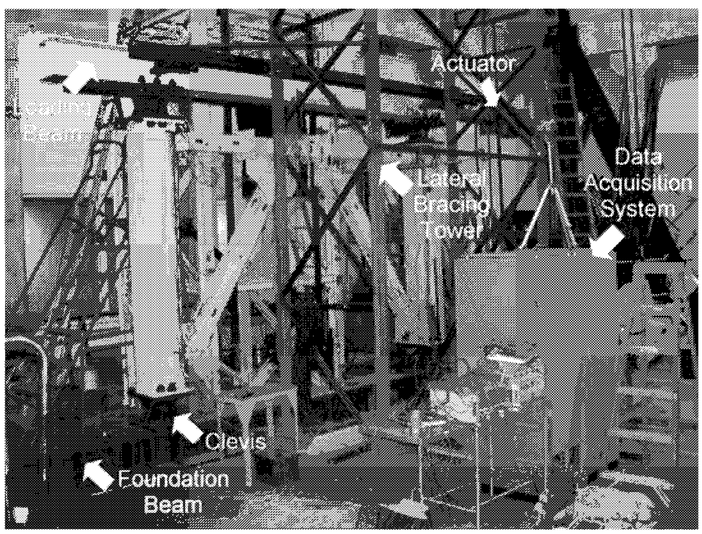

Figure 9. Completed Test Setup

Experimental Results

The experimentally obtained base shear versus frame drift hysteresis is shown in Figure 10 and the link shear force versus link rotation hysteresis is shown in Figure 11. From the elastic cycles of Figure 10, the initial stiffness of the specimen was found to be 80 kN/mm. The yield base shear and frame drift were 668 kN and 0.37% while the maximum base shear and drift were 1009 kN and 2.3%, respectively. Link shear force and rotation at yield were 490 kN and 0.014, while the maximum link shear and rotation achieved were 742 kN and 0.151 rads, respectively. Projecting the elastic and inelastic slopes of Figure 11 leads to an approximation of the plastic link shear force of 520 kN. The link shear force at

0.08 rads of rotation (the current limit for shear links) was 689 kN. Assuming equal link end moments, the end moments at specimen yield, development of V_p, and 0.08 rads of rotation were 112 kN-m, 119 kN-m, and 158 kN-m, respectively, and the maximum link end moment was 170 kN-m.

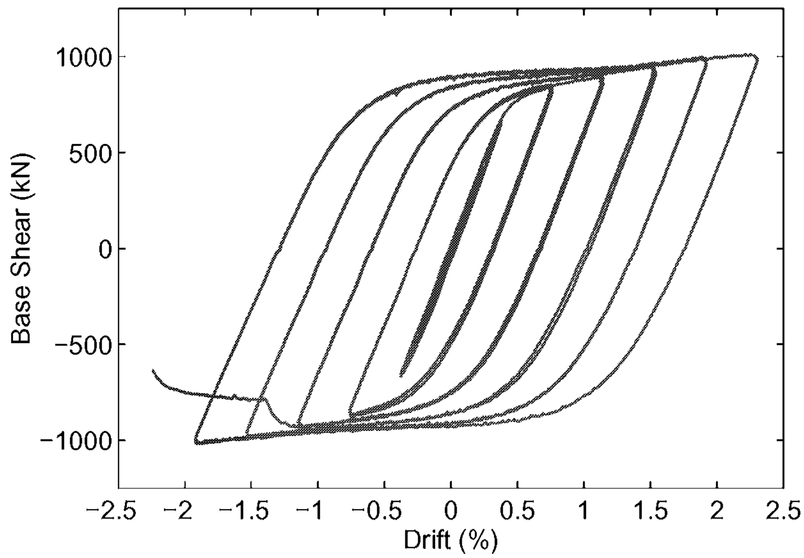

Figure 10. Base Shear vs. Frame Drift

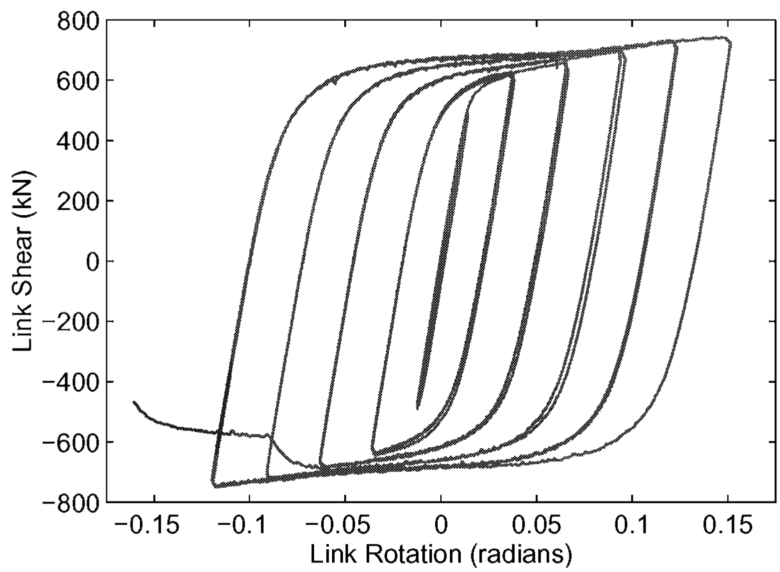

Figure 11. Link Shear vs. Link Rotation

The link achieved a rotation level 0.151 rads, which is almost twice the current limit for shear links in EBF for buildings (0.08 rads) and the target rotation for the specimen. Furthermore, the EBF reached a frame displacement ductility 6.0 and link rotation ductility of over 10 (the difference in these is due to the flexibility of the surrounding framing). Figure 12 shows the deformed link at 0.123 rads (1.92% drift) during Cycle 19.

Link failure occurred when the bottom flange at the north end of the link fractured as shown in Figure 13. The fracture occurred in the heat affected zone (HAZ) of the flange adjacent to the fillet weld used to connect the stiffener for the gusset of the brace-to-link connection to the link. Inspection of the failure surface was performed using a magnifying glass and light-microscope with 30x magnification (personal communication, Mark Lukowski, metallurgist, and Dr. Robert C. Wetherhold, mechanical engineer, Department of Mechanical and Aerospace Engineering, University at Buffalo, september 2003). The fracture was assessed as having initiated by cracking in the HAZ of the previously mentioned fillet weld and the propagation of those cracks under load reversals. There was no evidence of crack initiation in the full penetration groove weld used to assemble the webs and flanges of the link.

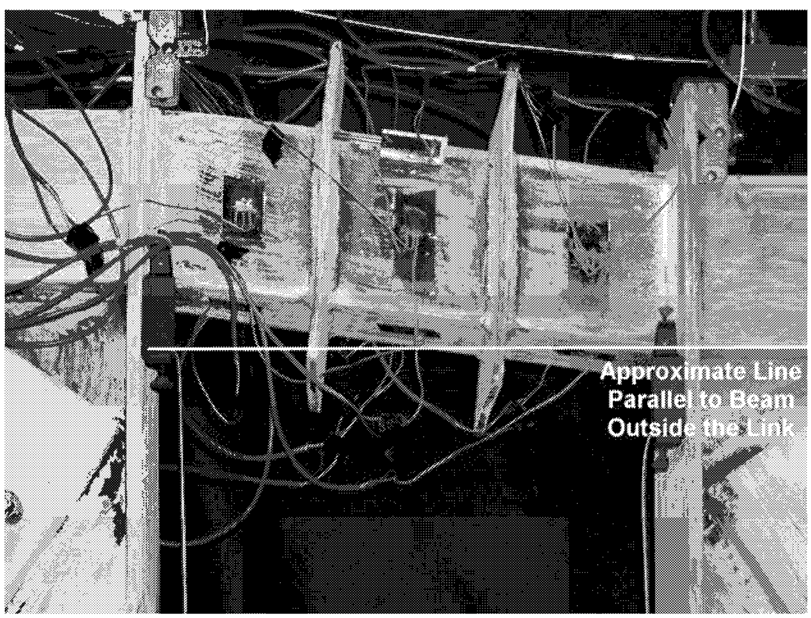

Figure 12. Link Deformed at 0.123 rads of Rotation During Cycle 19

The framing outside the link remained elastic for the duration of testing. Out-of-plane moments in the beam segments outside the link and eccentric braces remained less than 2.5% of those members' yield moments. This small moment can be resisted by the connections to the columns and need not be considered in design. Small out-of-plane moments and the fact that no evidence of lateral torsional buckling was observed, indicates that the goal of developing a link that does not require lateral bracing was achieved.

Furthermore, a link rotation of almost double the maximum allowed in design codes for I-shaped links was achieved prior to any strength degradation, indicating that hybrid rectangular shear links are viable alternatives for new and retrofit construction of steel bridge piers. It should also be noted that no evidence of web or flange buckling was observed, indicating that in this case the stiffeners and compactness limits were effective in preventing those buckling modes.

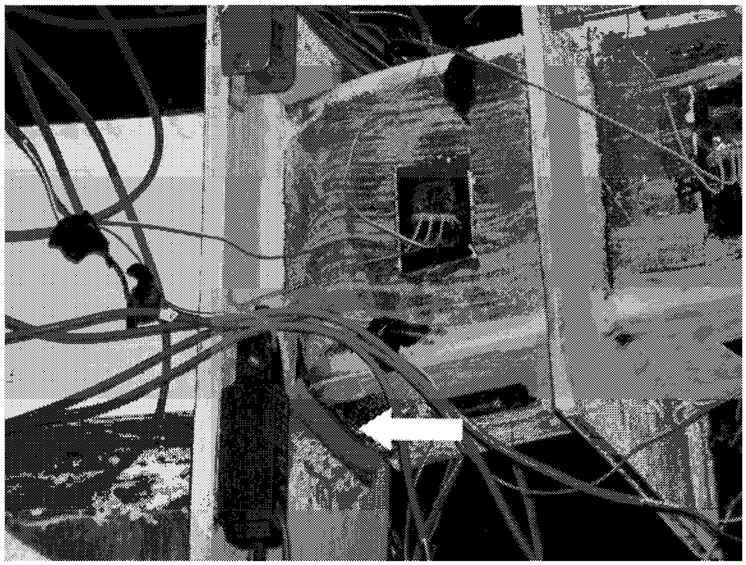
Figure 13. Fractured Bottom Flange at North End of Link

Conclusions

A laterally stable link for eccentrically braced frames for use in bridge piers where lateral bracing to prevent lateral torsional buckling is difficult to provide, has been developed. Selected design equations to achieve laterally stable hybrid rectangular links have been given and references for their derivations have been provided. The proof-of-concept testing of a single eccentrically braced frame with a hybrid rectangular shear link was successful in that it reached a link rotation of 0.151 rads (almost twice the maximum allowed in building codes for I-shaped links) without strength degradation and showed no signs of lateral torsional buckling. Link failure occurred after reaching a frame displacement ductility of 6 (which corresponded to a link rotation ductility of more than 10) when the bottom flange fractured at the north end of the link. The fracture was found to have started in the heat affected zone of the flange near the fillet weld used to connect the gusset stiffener for the eccentric brace connection to the link and was not affected by the full penetration groove weld used to connect the webs and flanges of the link.

Acknowledgements

This research was conducted by the State University of New York at Buffalo and was supported by the Federal Highway Administration under contract number DTFH61-98-C-00094 to the Multidisciplinary Center for Earthquake Engineering Research. However, any opinions, findings, conclusions, and recommendations presented in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

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