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End web stiffeners for connecting ductile replaceable links

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ABSTRACT

The ability to restore the integrity of structural lateral systems that utilize steel links to inelastically deform in earthquakes can be improved through the use of replaceable links. Replaceable links can be removed when damaged, especially when isolated from the gravity carrying system, and can provide engineers greater design freedom with respect to structural stiffness and strength. To ensure that ductile inelastic deformation is isolated to the effective length of the replaceable link, robust connection details capable of sustaining large cyclic deformations are needed. An integrated numerical and experimental research program was performed to evaluate a bolted end plate connection detail which utilized fillet welds, instead of full joint penetration welds, between the links and end plates in an effort to simplify fabrication. Continuum finite-element methods were used to evaluate various end stiffener options to shift flange plastic strains caused by link deformation away from the welds. The results from that study informed the selection of cases evaluated in large-scale experiments. Cyclic pseudostatic tests were conducted to experimentally demonstrate the performance of the selected end-stiffener configurations. Results from the experimental investigation showed that ductile inelastic deformation within the effective length of the links can be achieved using a bolted end plate connection with fillet welds between the shear link and end plate by utilizing end stiffeners placed parallel to the web. Design recommendations for incorporating the end stiffeners are outlined based on the data and experiences gained.

1. Introduction

Strategically placed steel link beams can be used as designated plastic hinge regions in lateral load resisting systems to provide energy dissipation and isolate damage imposed by seismic loading. Such links are most commonly utilized in eccentrically braced frames (EBF) [1], but have also been adopted for other structural systems including the linked column frame [11,18] as links between concrete shear walls [12] and between steel plate shear walls [5]. Depending on the relative relationship between the link plastic moment and shear capacities, the length of the links can be categorized as short, where shear governs the inelastic behavior, or long, where flexure governs the inelastic behavior, or intermediate for cases in between [AISC 2016a]. In traditional EBF, links are commonly designed by intentionally offsetting the work points in the brace-to-beam connection to create an eccentricity in the gravity carrying beams. While this approach has been shown to provide adequate seismic resistance, significant damage can develop within the link region of these gravity carrying beams even for earthquakes that are below the design level. Since these links are part of the continuous gravity system, repair or replacement can be difficult and costly. Furthermore, incorporating links sectional requirements for gravity beams can result in overdesigned beams, braces, columns, floor diaphragms, connections, and foundations as a result of capacity design requirements that stipulate that the gravity beam must be designed to resist the maximum force generated by the link [3].

Recently, strategies have been proposed to overcome the aforementioned deficiencies and improve the performance and resiliency of systems which rely on the ductile inelastic response of shear links. Using high yield strength steel in capacity protected elements (e.g. gravity beams) has been evaluated for both horizontal and vertical link configurations in EBF [15,16,28]. In addition, replaceable shear links have been evaluated for use as coupling beams in coupled shear wall systems [5,12] and extensively for use in EBFs [e.g. [8,19,26]; Bozkur and Topkaya 2017]. In all cases, the links are isolated from adjacent structural members which minimizes damage to capacity protected elements during seismic events. Further, in the case of EBF specifically, de-coupling the link from the sectional and/or material constraints of the gravity beam provides more specific design for lateral stiffness and strength of the system.

Bolted end-plate connections for replaceable shear links fabricated using complete joint penetration (CJP) welds [8,19,26], partial joint penetration (PJP) welds [19], and fillet welds [22]. In general, links

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with CJP or PJP welds at the link-to-endplate interface demonstrated ductile yielding within the effective length of the link beyond the code-specified rotation capacity, although ultimate failure in some cases resulted from fracture in the welded region. Ductility in links with fillet welds at the link-to-endplate interface was often limited by fracture of the fillet welds [13,22,27]. To reduce plastic strains in welded link connections, the concept of reduced link sections, similar to reduced beam sections in moment frames, had been numerically demonstrated [4,21]. However the high moment gradient in shear links has been shown to cause issues with practical applications of this concept [1].

This paper outlines a bolted end plate connection detail for replaceable links that utilizes fillet welds at the link-to-endplate interface and introduces end stiffeners to shift inelastic flange deformations away from the flange-to-endplate welds. The utilization of fillet welds has several advantages over CJP and PJP weld details including reduced inspection requirements [1] and lower fabrication costs. Further, the use of end stiffeners can provide engineers additional control over the behavior of the effective link length between the stiffeners without changing the overall geometry of the beam or the braces. In this study, a continuum finite element analyses were used to initially investigate the behavior of a number of different end stiffener configurations, whereby the results were used to select a candidate stiffener configuration for experimental evaluation. Pseudostatic experiments were then conducted to experimentally evaluate the performance of the selected stiffener configurations. Based on the results of the numerical and experimental study, recommendations are provided for the design of the link-to-end plate connection.

2. Link design and connection details

To study the connection detail for both shear and flexurally dominated options for the links, short ($e' < 1.6M_p/V_p$) and long ($e' > 2.6M_p/V_p$) links were designed, where M_p is the plastic moment strength, V_p is the plastic shear strength, and e' is the effective link length. Note that for unstiffened links, the effective link length e' is equal to the total link length e. An overview of link geometry for resulting W12x96 and W12x22 for the short and long links respectively is illustrated in Fig. 1, and the cross-sectional properties and calculated capacities are summarized in Table 1. The W12x96 and W12x22 links are herein referred to as *short shear* and *long flexural* links respectively. Link capacities in Table 1 were calculated using yield strengths from tension tests conducted on the rolled sections of experimental specimens; the steel stress strain curves are illustrated in Fig. 2. The link length of

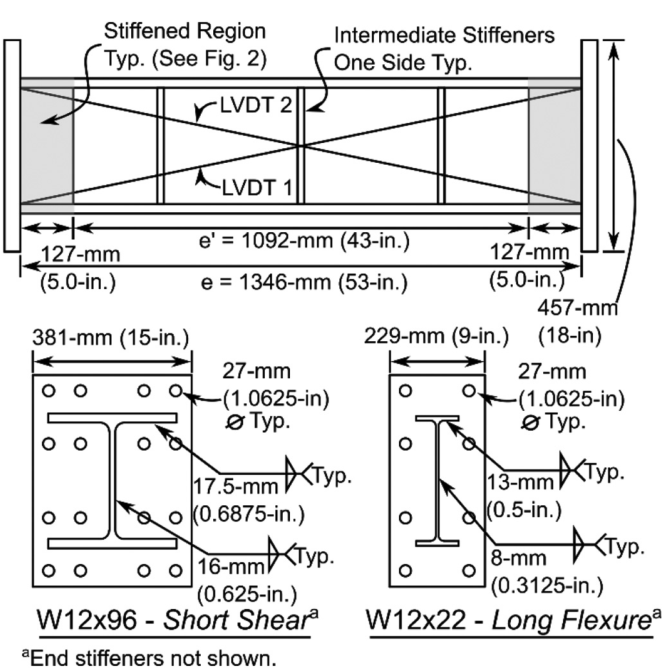

Fig. 1. Link geometry and bolted endplate connection design.

Table 1
Link properties calculated using measured material properties.

	Links	
	W12x22 (*long flexure*)	W12x96 (*short shear*)
Intermediate Stiff.		
Thickness, mm (in.)	9.5 (0.38)	16.0 (0.625)
Spacing, mm (in.)	150.0 (6.0)	338.0 (13.3)
Web		
d, mm (in.)	312.0 (12.3)	323.0 (12.7)
t_w, mm (in)	6.6 (0.26)	14.0 (0.55)
f_y, MPa (ksi)	434.0 (63.0)	393.0 (57.0)
Flanges		
b_f, mm (in.)	102.0 (4.0)	310.0 (12.2)
t_f, mm (in.)	10.7 (0.43)	23.0 (0.9)
f_y, MPa (ksi)	352.0 (51.0)	386.0 (56.0)
Nominal Strength		
M_p, kN-mm (kip-in.)	1.69×10^5(1494.3)	9.30×10^5(8232.0)
V_p, kN (kip)	407.9 (91.7)	912.0 (201.4)
Classification Check		
$1.6M_p/V_p$, mm (in.)	662.9 (26.1)	1630.7 (64.2)
$2.6M_p/V_p$, mm (in.)	1076.9 (42.4)	2651.8 (104.4)
End Stiff.		
l_d mm (in.)	127.0 (5.0)	127.0 (5.0)
t_d mm (in.)	9.5 (0.375)	15.9 (0.625)
End Plate Weld Size		
Flange, mm (in.)	13 (0.5)	17.5 (0.685)
Web, mm (in.)	8 (0.3125)	16 (0.625)

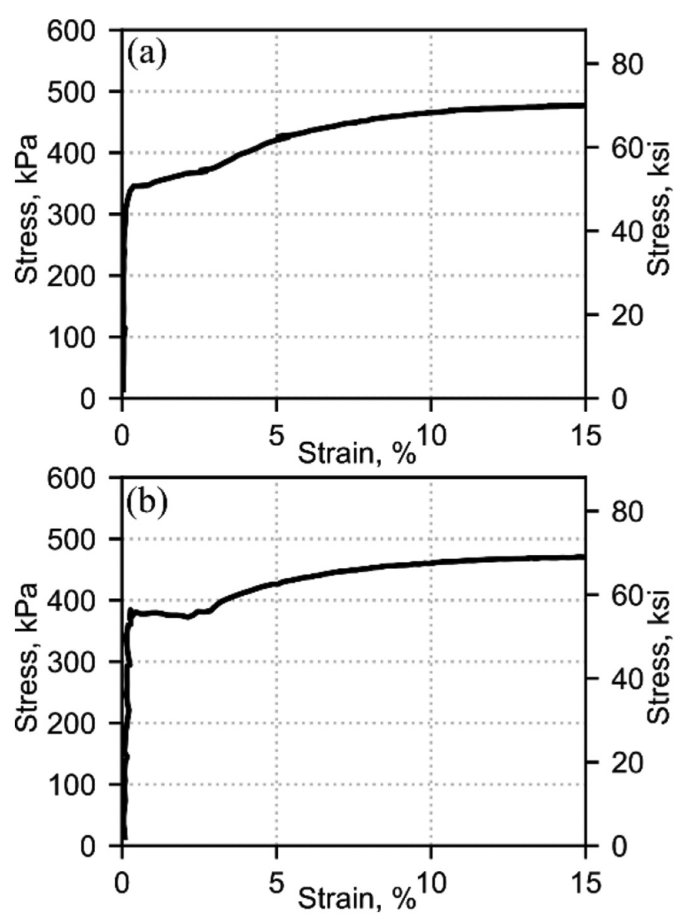

Fig. 2. Steel stress strain curves for (a) W12x22 and (b) W12x96 Links.