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An innovative seismic performance enhancement technique for steel building moment resisting connections

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ABSTRACT

This study develops and experimentally validates an innovative technique for enhancing the seismic performance of steel beam to column moment connections. The technique involves reducing the strength of specified regions of the beam flanges by exposing them to high temperatures followed by slow cooling. Analogous to the reduced beam section (RBS) connection, yielding and plastic hinge formation is promoted in the heat-treated beam section (HBS). Moreover, because the elastic and inelastic modulus of the steel is unmodified by the heat-treatment and the beam cross section is not altered, an HBS connection does not sacrifice elastic stiffness or buckling resistance as does the RBS. Design of the HBS connection was performed through detailed finite element analysis and material testing. Two large scale connections modified with the HBS technique were tested in this study. The test program showed that the proposed heat-treatment technique was successful in the promotion of yielding and plastic hinge development in the heat-treated regions with specimens attaining interstory drifts as high as 6% without weld or near weld fracture. Strength degradation due to beam buckling within the HBS was the observed failure mechanism in both specimens. Detail analyses of strain and beam deformation data are presented to explain the HBS connection plastic hinge formation and gradual strength degradation. Broader applications of the technique to other structural components are identified.

1. Overview of steel moment connection advances since Northridge earthquake

Extensive damage to steel moment resisting frame connections during the 1994 Northridge earthquake instigated a comprehensive research effort led by the SAC joint venture intended to develop a broad and fundamental understanding of the seismic performance of moment frame connections [1]. A multitude of failure modes were observed in moment connections following the earthquake, the most common of which were fractures originating at or near the beam flange CJP welds which accounted for approximately 80% of the damage to welded steel moment frames [2]. In many cases these fractures propagated to other areas of the connection for example, column flanges, column web and beam web [3].

Initially, poor quality welds stemming from improper workmanship, use of welding electrodes which deposit weld metal with inherently low toughness, and a lack of careful inspection practice were thought to be the main causes for these brittle failures. However, studies that incorporated weld fracture mitigation measures alone showed that improvements in welding, inspection practice and weld metal are insufficient to guarantee adequate performance of these connections in high seismic areas [4–6]. One of these studies, performed by Stojadinovic et al. [4], evaluated the welded unreinforced flange-bolted web connection (WUF-B), which was one of the most commonly used moment resisting connections (prior to Northridge) due to their perceived ductility and economy [7,8]. As a part of the study, moment connections were fabricated to pre-Northridge standards and tested to in an effort to recreate the brittle failures discovered after Northridge and in doing so confirm the reasons for these failures. Subsequently, connections fabricated using “notch tough” weld metal, improved welding procedure, improved welding inspection and more careful connection detailing were evaluated. The study showed the pre-Northridge connections to be brittle as was also shown by Engelhardt and Husain [9] and Popov et al. [3]. The WUF-B connections exhibited little to no ductility with fractures emanating from the beam flange to column flange CJP weld root or toe and propagating along different paths. Among the failures were divot pull outs of the weld and column flange material, crack propagation through the column flange and web, crack propagation through the beam web, beam flange gross section fractures at the weld toe and weld root [4].

The so called ‘SAC post-Northridge’ connections tested by Stojadinovic et al. [4] displayed improved performance, however none of these connections were able to attain the required 0.03 radians plastic rotation in the standard proof test defined in the FEMA interim guidelines and the 1997 AISC Seismic provisions for steel buildings [4]. Careful examination of these connections through experimental and analytical studies,

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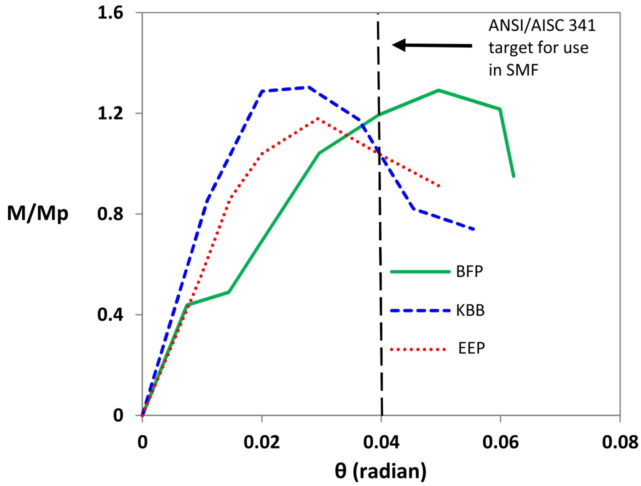

Specimen	Failure Mechanism
BFP (Specimen 7 Schneider and Teeraparbwong [16])	Fracture of beam flange in region of severe local buckling
KBB (Specimen 8 Adan and Gibb [14])	Fracture of beam flange in region of severe local buckling
EEP (8 bolt stiffened extended end plate. Specimen 8ES-1.25-1.75-30 - Murray et al [12])	Test stopped due to lateral instability of beam

Fig. 1. Moment-rotation experimental backbone curves (in positive bending) of stiffened connections.

suggested that beam flanges were "overstressed" due to stress redistribution and stress risers imposed by the configuration of the connection which ultimately limited connection plastic rotation capacity [10]. However, a study conducted by Xue et al. [11] showed evidence, albeit with a limited number of tests, that weld fracture mitigation strategies combined with a fully welded beam web could produce plastic rotations larger than 0.03 rads. Nonetheless, Stojadinovic et al. [4] concluded that weld fracture mitigation strategies alone are insufficient and that in future, moment connections intended for use in special moment frames (SMF) should incorporate both "weld mitigation measures as well as overstress mitigation measures". As such, many subsequent research studies focused on lowering these stress and strain demands through various means. The fundamental change in the design philosophy of these moment connections to that of the pre-Northridge connection is the development of a plastic hinge away from the beam end intended to act as a reliable and ductile seismic fuse. In most of these connections the relocation of the plastic hinge is accomplished through strengthening and stiffening the connection by adding endplates with stiffeners [12], cover plates [13], castings [14], haunches [15] etc. to encourage plastic hinging in the beam just beyond the strengthened region. The intent being that the strengthened connection would primarily remain elastic while plastic hinging of the beam takes place, thereby reducing the plastic strain demands and stress concentrations in the critical connection region.

Studies conducted by Sumner and Murray [12] demonstrated the use of extended end plate (EEP) connections with and without stiffeners to enhance the ductility of moment connections. Schneider and Teeraparbwong [16] and Sato et al. [17] demonstrated that carefully designed bolted flange plate (BFP) connections were successful in causing hinging of the beam section beyond the flange plate and that this led to a ductile and reliable failure mechanism. Adan and Gibb [14] evaluated proprietary cast high strength steel "Kaiser" bolted brackets (KBB) which were either bolted or welded to the beam flange and then bolted to the column flange. In this study it was demonstrated that similar to stiffened extended end plate and bolted flange plate connections the KBB strengthens the beam column connection region and forces plastic hinging, buckling and eventually fracture of the beam flange just beyond the end of the bracket.

Fig. 1 shows experimental backbone curves for selected EEP, BFP and KBB connections from these studies. Moments have been normalized based on plastic moment capacities reported in the respective studies. Both moments and rotations were computed with respect to the column centerline. Note in Fig. 1 that all these connections were successful in achieving their plastic moment capacity and exceeded the 2010 ANSI/AISC 341 Seismic Provisions [18] performance criteria for use in SMF's. As a result, they are now included in the 2010 ANSI/AISC 358 Prequalified Connections for Special and Intermediate Moment

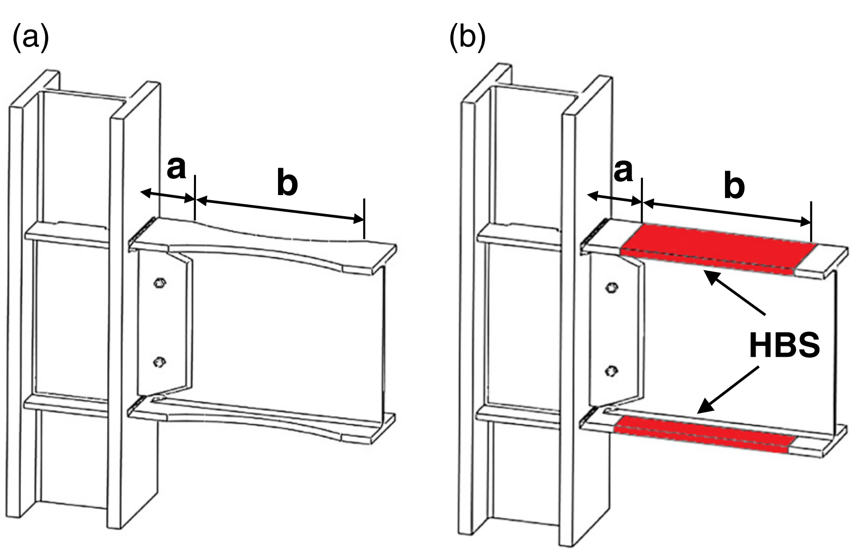

Fig. 2. Sketch of (a) reduced beam section (RBS) and (b) heat-treated beam section (HBS).