Cyclic Behavior of Steel Moment-Resisting Connections Reinforced by Alternative Column Stiffener Details I. Connection Performance and Continuity Plate Detailing

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Following the Northridge earthquake of January 17, 1994, damage occurred to steel moment connections, most often consisting of brittle fractures of the bottom girder flange-to-column flange complete-joint-penetration (CJP) groove welds (Youssef, Bonowitz and Gross, 1995; Northridge Reconnaissance Team, 1996; FEMA, 2000a; FEMA, 2000c). The fractures were caused by the use of low toughness welds; connection design and detailing that led to larger moment-frame members, less system redundancy, and higher strain demands on the connections; the use of higher strength girders leading to unintentional under matching of the welds; and a number of other connection detailing and construction practices that were typical prior to the earthquake (Roeder and Foutch, 1996; FEMA, 2000a, 2000b, 2000c). Additionally, column reinforcement practices have been cited as a possible contributor to the fractures, largely as a result of observations that many of the connections fractured during the Northridge earthquake lacked continuity plates and that some had weak panel zones (Tremblay, Timler, Bruneau and Filiatrault, 1995). Finite element analyses (El-Tawil, Mikesell, Vidarsson and Kunnath, 1998; El-Tawil, Vidarsson, Mikesell and Kunnath, 1999; El-Tawil, 2000; Ricles, Mao, Lu and Fisher, 2003) also have shown an increase in stress and strain concentrations in the girder flange-to-column flange CJP welds associated with excessively weak panel zones or insufficient continuity plates, and it has been speculated that these stress and strain concentrations increase the potential for fracture. As a result of these observations, there has subsequently been a tendency to be more conservative than may be necessary in designing and detailing of the continuity plates and doubler plates in steel moment-resisting connections.

Research by the SAC Joint Venture has attempted to resolve many issues related to steel moment connections and has recently led to the recommendations for the design of new steel moment-frame buildings, including column reinforcement design and detailing (FEMA, 2000a). These design recommendations provide equations for determining whether continuity plates are required, and indicate that any required continuity plates must be at least of equal thickness to the girder flange for interior connections (thinner continuity plates are permitted for exterior connections), unless connection qualification testing demonstrates that the continuity plates are not required. Furthermore, the connection of the continuity plates to the column flanges must be made with CJP welds, and reinforcing fillet welds should be placed under the backing bars (if left in place). Also presented in FEMA (2000a) are new panel zone design provisions that seek to balance the onset of yielding between the panel zone and connected girders.

Design criteria for the limit states related to column reinforcement are presented in the AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 1993, 1999a), hereafter referred to as the AISC LRFD Specification. The limit states of primary importance for stiffening of connections include local flange bending (LFB), local web yielding (LWY), and panel zone yielding (PZ). Additional provisions for seismic design of doubler plates and continuity plates were included in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1992, 1997a, 1999b, 2000, 2002), hereafter referred to as the AISC Seismic Provisions. However, AISC (1997a, 2002) removed all continuity plate design procedures for Intermediate and Special Moment Frames, requiring instead that they be proportioned based on connection qualification tests.

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The potential for being more conservative than may be necessary in column reinforcement design has raised concerns about economy as well as the potential for cracking of the *k*-area in the column web near the web-flange junction during fabrication due to high residual stresses caused by highly restrained CJP welds on the continuity plates or doubler plates (AISC, 1997b; Tide, 2000).

There has been a great deal of past research on the issue of column reinforcement in steel moment connections, including recent work since the Northridge earthquake. However, most of the past experimental research in particular has focused on the effects of the presence or absence of continuity plates doubler plates, or both, and not on the associated design equations and detailing procedures. Therefore, this combined experimental and computational research study was conducted at the University of Minnesota to reassess the recent column reinforcement design and detailing provisions (AISC, 1992, 1993, 1997a, 1999a, 1999b, 2000, 2002) and recommendations (FEMA, 2000a), and to provide economical alternative stiffener details that minimize welding along the column *k*-line while retaining superior performance for nonseismic and seismic design.

A total of six full-scale, girder-to-column cruciform specimens were tested in this research program. A welded unreinforced flange-welded web (WUF-W) connection detail were used; this connection is discussed in detail in FEMA (2000a). It was originally planned to test five cruciform specimens in this experimental study. Due to premature fracturing in three of the four girder flange-to-column flange CJP welds in one of these five specimens (as discussed further below), this specimen was replicated with new base metal and weld consumables and the new specimen was tested.

The cyclic behavior of three alternative doubler plate details, specifically a back-beveled fillet-welded detail, a square-cut fillet-welded detail, and a box (offset) detail, was investigated, and the performance of these connections was evaluated. In addition, the performance of one continuity plate detail was investigated: a continuity plate with thickness approximately half of the girder flange thickness fillet-welded to both the column flanges and doubler plates. Based on the experimental results, the current LFB, LWY, and PZ design and detailing provisions and recommendations were reassessed, and the applicability of the newly suggested column reinforcement details was investigated. This research complements prior, related research done on this project (Lee, Cotton, Dexter, Hajjar, Ye and Ojard, 2002), which included nine pull-plate experiments (Prochnow, Dexter, Hajjar and Cotton, 2000; Prochnow, Ye, Dexter, Hajjar and Cotton, 2002; Dexter, Hajjar, Prochnow, Graeser, Galambos and Cotton, 2001; Hajjar, Dexter, Ojard, Ye and Cotton, 2003) that investigated the limit states of LFB and LWY, primarily for nonseismic design, and tested the alternative doubler plate and continuity plate stiffener details. Finite element analyses of all experimental specimens were also conducted as part of this research as well as parametric studies to assess the performance of various continuity plate and doubler plate details not tested (Ye, Hajjar, Dexter, Prochnow and Cotton, 2000).

This paper reports the results and performance of the six cruciform tests. In a companion paper (Lee, Cotton, Hajjar, Dexter and Ye, 2005), the cyclic panel zone behavior and design in moment-resisting connections, including these test specimens, is investigated.

COLUMN STIFFENER DESIGN EQUATIONS

Design equations for the column reinforcement included in the AISC LRFD Specification (AISC, 1999a) and AISC Seismic Provisions (AISC, 2002) are largely based on research conducted several decades ago. Research by Sherbourne and Jensen (1957) and Graham, Sherbourne, Khabbaz and Jensen (1960) established the provisions for Local Flange Bending (LFB) and Local Web Yielding (LWY), while research by Krawinkler, Bertero and Popov (1971), Bertero, Krawinkler and Popov (1973), and Krawinkler (1978) led to the seismic panel zone design criteria in AISC (2002). It should be noted, however, that these provisions were derived from research conducted on older A7 and A36 steels, and on member sizes smaller than typically used in current steel moment frame construction.

Recently, extensive research by the SAC Joint Venture has led to the recommendations (FEMA, 2000a) for the design of new steel moment connections. The recommended column reinforcement design equations within FEMA (2000a) are briefly described below in comparison with the AISC design equations (AISC, 1997a, 1999b, 2000, 2002).

AISC Design Provisions

The design of continuity plates in steel moment connections is primarily governed by LFB and LWY limit states. While the web crippling limit state is also applicable to steel moment connection design (AISC, 1999a), a study by Prochnow et al. (2000) showed that it never governed the need for continuity plates in typical moment connection configurations. For the LFB limit state for nonseismic design (AISC, 1999a) and older seismic design provisions (AISC, 1992), continuity plates must be provided if the required strength (or demand), R_u, exceeds the design strength (or capacity) of the column flange, given by

$$\phi R_n = \phi 6.25 t_{cf}^2 F_{yc} \tag{1}$$

[Equation K1-1 (AISC, 1999a)]

where

ϕ = 0.90

t_{cf} = column flange thickness

F_{yc} = specified minimum yield stress of the column

For the LWY limit state, similarly, continuity plates are required if R_u exceeds the column web design strength, given by

Interior condition

$$\phi R_n = \phi\left(5k + N\right) F_{yc} t_{cw} \tag{2}$$

[Equation K1-2 (AISC, 1999a)]

End condition

$$\phi R_n = \phi\left(2.5k + N\right) F_{yc} t_{cw} \tag{3}$$

[Equation K1-3 (AISC, 1999a)]

where

ϕ = 1.0
k = distance from outer face of column flange to web toe of column fillet
N = length of bearing surface = girder flange thickness in moment connections
t_{cw} = column web thickness

The required strengths, R_u, for the above two limit states (in other words, LFB and LWY) are based on the forces delivered to the column flanges in the connection. Several possibilities exist for the calculation of these required strengths, including (but not limited to):

$$R_u = F_{yg} A_{gf} \tag{4}$$

$$R_u = 1.8 F_{yg} A_{gf} \tag{5}$$

$$R_u = \frac{1.1 R_{yg} F_{yg} Z_g + V_g a}{0.95 d_g} \tag{6}$$

$$R_u = 1.1 R_{yg} F_{yg} A_{gf} \tag{7}$$

where

F_{yg} = specified minimum yield stress of the girder
A_{gf} = area of one girder flange
R_{yg} = ratio of expected yield strength of girder to specified minimum value
Z_g = girder plastic section modulus
V_g = shear force in girder at plastic hinge location
a = distance from column face to girder plastic hinge location
d_g = girder depth

The design of continuity plates should also conform to Section K1.9 of AISC (1999a).

Equation 4 is typically used for nonseismic design, representing the nominal yield strength of one girder flange. Equation 5 was included in the 1992 AISC Seismic Provisions (AISC, 1992). The 1.8 factor includes a "strain-hardening" factor of 1.3 to account for the probability of strength much greater than the minimum specified value and the increase in strength after significant yielding and another factor of 1.4 ($1.4 \times 1.3 \approx 1.8$) that is related to the assumption that the full plastic strength of the girder is carried by a force couple of the flanges only. This factor of 1.4 is the approximate upper bound ratio of the girder plastic section modulus, Z_g, to the flange section modulus, Z_{gf} (Bruneau, Uang and Whittaker, 1998). Although the stress state in the girder flange is not uniaxial, the girder flange required strength predicted by Equation 5 can be put in perspective by comparing to the maximum possible uniaxial tensile strength of ASTM A992 steel. Equation 5 predicts stresses in the flange of 90 ksi for F_{yg} of 50 ksi, well above the likely tensile strength of A992 steel. For example, a survey of more than 20,000 mill reports from (Dexter, 2000; Dexter et al., 2001; Bartlett, Dexter, Graeser, Jelinek, Schmidt and Galambos, 2003) showed that A992 steel has a mean tensile strength of 73 ksi. The 97.5 percentile tensile strength was 80 ksi, and the maximum value reported was 88 ksi.

Equation 6 is included in AISC Design Guide No. 13 (AISC, 1999c). This equation, and slightly modified forms of it, have been widely used for the design of column stiffeners in steel moment connections (FEMA, 2000a). Equation 7 was presented by Prochnow et al. (2000) to provide a more realistic representation of the girder flange force in steel moment connections, mostly for use in assessment of the pull-plate experiments (Hajjar et al., 2003).

The AISC Seismic Provisions for the panel zone design (AISC, 2002) include two design equations. The first specifies the shear strength of the panel zone and the second places a limitation on panel zone slenderness:

$$\phi_v R_v = \phi_v 0.6 F_{yc} d_c t_p \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_g d_c t_p}\right) \tag{8}$$

[Equation 9-1 (AISC, 2002)]

where

ϕ_v = 1.0
d_c = column depth
t_p = panel zone thickness
b_{cf} = column flange width

$$t \geq \left(d_z + w_z\right) / 90 \tag{9}$$

[Equation 9-2 (AISC, 2002)]

where

t = column web or doubler plate thickness; or total thickness if doublers are plug welded to the column web
d_z = panel zone depth
w_z = panel zone width

In Equation 8, the term in parentheses accounts for the post-yield strength of the panel zone, as proposed by Krawinkler (1978).

General procedures for the seismic panel zone shear required strength calculation are described in AISC (2002). This formula is a modified approach from that proposed in AISC Design Guide No. 13 (AISC, 1999c) in that it uses ϕ_v = 1.0 and removes the factor of 0.8 from the girder moments. The general formula can be given as

$$R_u = \sum_{girders} \frac{1.1 R_y F_{yg} Z_g + V_g a}{0.95 d_g} - V_c \tag{10}$$

where

V_c = story shear

FEMA 350 Design Recommendations

The design recommendations within FEMA (2000a) include two equations for determining the need for continuity plates in steel moment connections. Both are based on mitigating the LFB limit state. Continuity plates are required if

$$t_{cf} < 0.4\sqrt{1.8 b_{gf} t_{gf} \frac{F_{yg} R_{yg}}{F_{yc} R_{yc}}} \tag{11}$$

or

$$t_{cf} < \frac{b_{gf}}{6} \tag{12}$$

where

b_{gf} = girder flange width

t_{gf} = girder flange thickness

R_{yc} = ratio of expected yield strength of column to specified minimum value

The design of continuity plates should also conform to Section K1.9 of AISC (1999a).

Instead of incorporating the panel zone post-yield strength recognized in the AISC Seismic Provisions (AISC, 1992, 1997a, 1999b, 2000, 2002) and basing required strength on the ultimate strength of the connected girders, the recommendations of FEMA (2000a) attempt to balance the onset of yielding between the panel zone and connected girders. It has been shown by Roeder (2002) that this balanced mechanism leads to better overall connection performance. The panel zone shear required strength associated with this method is essentially the shear at the column centerline calculated from the flexural yielding of the girders, while the recognized shear strength of the panel zone is that at its first yield. For example, in the case of exterior connections, the required panel zone thickness, t_{req}, is determined as

$$t_{req} = \frac{C_y M_c \dfrac{h - d_g}{h}}{(0.9) 0.6 F_{yc} R_{yc} d_c \left(d_g - t_{gf}\right)} \tag{13}$$

$$C_y = \frac{1}{C_{pr} \dfrac{Z_{ge}}{S_g}} \tag{14}$$

where

M_c = moment at the centerline of the column

h = average story height of the stories above and below the panel zone

C_{pr} = peak connection strength coefficient defined in FEMA (2000a)

Z_{ge} = effective plastic section modulus of the girder at the zone of plastic hinging

S_g = elastic section modulus at the zone of plastic hinging

If the required thickness determined by Equation 13 is greater than the column web thickness, doubler plates should be placed in the panel zone or the column size should be increased to a section with adequate web thickness.

DESIGN OF CRUCIFORM TEST SPECIMENS

The five connection configurations tested in this research program were selected mainly based on a parametric study of panel zone stiffening requirements and on using sizes that would highlight specific aspects of the column limit states being investigated in this research (Lee et al., 2002). In particular, a relatively shallow girder depth (and thus a relatively large flange force) and relatively weak column panel zone were used so as to: (1) create specimens with different characteristics from the large body of tests that have deeper girders and stronger panel zones, for which a base of results has to some extent already been established (for example, Chi and Uang, 2002; Ricles, Fisher, Lu and Kaufmann, 2002a; Ricles, Mao, Lu and Fisher, 2002b; and many other studies); and (2) generate large strains in the connection region and thus create severe tests for the limit states being investigated in this research. The test matrix is outlined in Table 1. Figure 1 schematically illustrates the three doubler plate details used in this work, discussed below. Typical connection topologies for the cruciform specimens are shown in Figures 2 and 3. Due to unexpected premature brittle failure of girder flange-to-column flange welds in one of the five cruciform specimens (in other words, Specimen CR4, discussed further below), one additional specimen was replicated with new base metal and weld consumables and the new specimen was tested. This new specimen was identified as Specimen CR4R. In the following, designing and detailing of the five connection topologies are described.

Table 1. Test Matrix of Cruciform Specimens					
	CR1	CR2	CR3	CR4 and CR4R	CR5
Girder	W24×94	W24×94	W24×94	W24×94	W24×94
Column	W14×283	W14×193	W14×176	W14×176	W14×145
Doubler Plate (DP)	None	Detail II	Detail II	Detail III Box (Offset)	Detail I
DP Thickness	NA	0.625 in.	2 @ 0.5 in.	2 @ 0.75 in.	2 @ 0.625 in.
Continuity Plate (CP)	None	None	Fillet-welded	None	None
CP Thickness	NA	NA	0.5 in.	NA	NA

Column Stiffener Design and Detailing

As shown in Table 1, three doubler details were tested in this experimental study. Doubler plate Detail I and Detail II represent two different fillet-welded details, while doubler plate Detail III represents a groove-welded box (offset) detail (Figure 1). All doubler plates were vertically extended 6 in. above and below the girder flanges. The extension of the doubler plate corresponds to approximately ($2.5k + N$), implying this extension is partly effective in resisting Local Web Yielding (LWY). Although LWY was not the controlling limit state within the given girder-to-column combinations, this was done to be consistent with the doubler plate details used in the pull-plate experiments (Prochnow et al., 2000; Hajjar et al., 2003).

One continuity plate detail was also tested, in which the plate thickness was approximately equal to half the girder flange thickness, and the plate was fillet-welded to both the column flanges and doubler plates. The size of the fillet welds needed for both doubler and continuity plate details were calculated using procedures given in the AISC Design Guide No.13 (AISC, 1999c).

One completely unstiffened specimen with a W14×283 column (Specimen CR1) was also tested to verify the cyclic response of a specimen without continuity plates and doubler plates. In addition, Specimens CR2 and CR5 had no continuity plates although continuity plates were required as per AISC (1992) for Specimen CR2 and as per AISC (1992, 1999a, 1999c) for Specimen CR5. These specimens were used to examine the response of a column flange subjected to the LFB limit state under cyclic loading.

Doubler Plate Detail I and Detail II

Two fillet-welded doubler plate details are presented in Figures 1(a) and 1(b). Doubler plate Detail I presented in Figure 1(a) is essentially the detail shown in Figure C-I-9.3(b) of the AISC Seismic Provisions (AISC, 2002). This

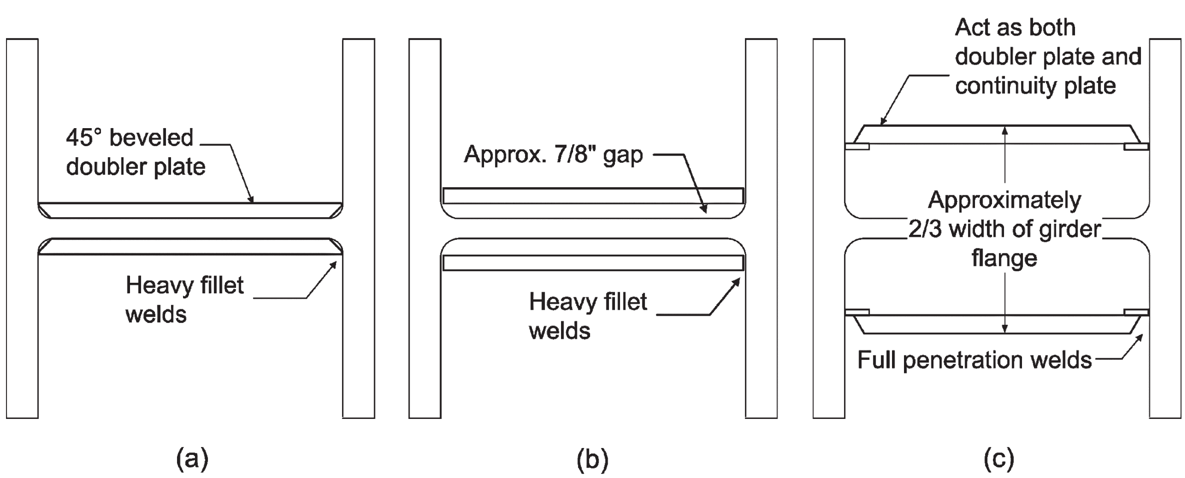


Fig. 1. Alternative doubler plate details: (a) back-beveled fillet-welded doubler (Detail I), (b) square-cut fillet-welded doubler (Detail II), (c) box (offset) doubler (Detail III).

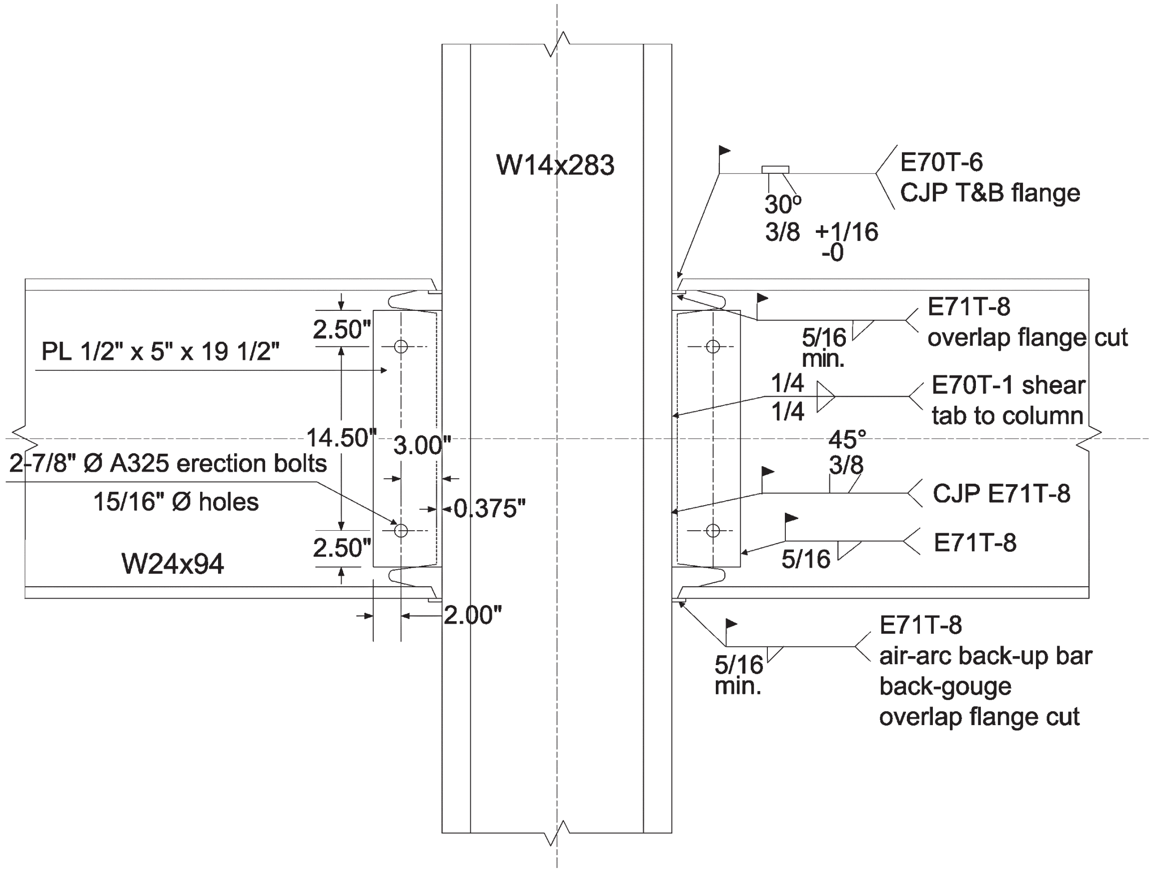


Fig. 2. Typical welding details used for cruciform specimens (Specimen CR1).

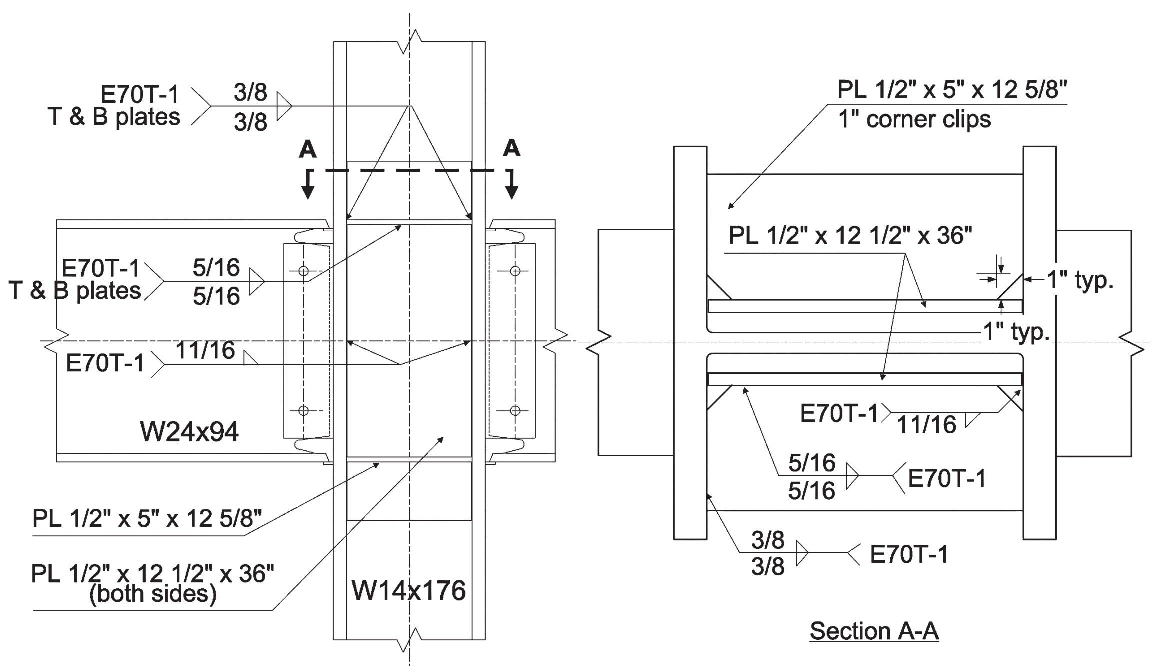


Fig. 3. Typical welding details used for cruciform specimens (Specimen CR3).

consists of doubler plates back-beveled at 45 degrees to minimize interference with the column radius region. It is intended that the plates be placed flush against the column web and fillet-welded to the column flanges. This stiffening detail avoids placing highly restrained CJP welds in the column *k*-area.

Doubler plate Detail I has limitations, however, one of which led to the development of the second fillet-welded doubler plate detail (in other words, Detail II). As indicated in AISC (2001), the actual *k*-values for many W-shapes have become larger in recent years. The larger fillet radii of the column made it impossible to position the doubler plates without the gap between the doubler plates and the column webs (unless the doubler plates were made to be unreasonably thick). Forcing these doubler plates flush against the column web without the gap could result in other gaps between the doubler plates and column flanges of greater than 1⁄16 in., which would not be permitted for the fillet welds. Thus, proper fit-up of the 5⁄8-in.-thick back-beveled doubler plates could not be achieved in Specimen CR5 with Detail I due to the large column fillets. However, this specimen has member sizes similar to some of the pull-plate specimens (Prochnow et al., 2000; Hajjar et al., 2003). It was desired in this test program to replicate the doubler plate details tested on some of the pull-plate specimens to allow correlation between the tests. A gap between the column web and doubler plate of approximately 1⁄4 in. thus resulted in Specimen CR5. No welds were placed across the top and bottom of the doubler plate.

Doubler plate Detail II, shown in Figure 1(b), was developed as an alternative to the back-beveled doubler plate detail (Detail I). Instead of beveling the plate to fit against the column web, the doublers are cut square to the width between column flanges (approximately 12.5 in. for W14 columns) and placed in the column until they interfere with the column radius. As with Detail I, the doubler plates are then fillet-welded to the column flanges. By cutting the plates just narrower than the width between flanges, the gap between the doublers and column flanges remains below 1⁄16 in. A result of this detail, however, is a gap between the column web and doubler plate of approximately 7⁄8 in., and no welds were placed across the top and bottom of the doubler plate. This doubler plate detail is suggested as an economical alternative because it requires no beveling, and is fillet-welded as opposed to groove-welded. Doubler plate Detail II was used for Specimens CR2 and CR3.

Doubler Plate Detail III — Box (Offset) Detail

The third doubler plate detail tested in this experimental study is a box (offset) detail similar to that given in Figure C-I-9.3(c) of the 2002 AISC Seismic Provisions (AISC, 2002), with the exception that no continuity plates are used in the detail tested in this research. This detail is economically desirable in lieu of using four continuity plates and two doubler plates because the offset plates are not only carrying panel zone shear forces but also acting to stiffen the column flanges and webs. Excessive panel zone yielding as well as local flange bending and local web yielding can be effectively mitigated in this doubler plate detail. Like Detail I and Detail II, the box (offset) detail also avoids welding in the column *k*-area. As shown in Figure 1(c), CJP welds were used to join the plates to the column flanges. The box (offset) doubler plate detail was used for Specimens CR4 and CR4R.

The location of the doubler plates (in other words, the amount of offset from the column web) was decided based on the parametric finite element analyses conducted by Ye et al. (2000). This study showed that the doubler plates were most effective when placed between one-third and two-thirds of the half-girder flange width from the column web. In this location, the strain concentrations in the center of the girder flanges were significantly reduced without an excessive increase towards the girder flange tips. A location corresponding to an offset of two-thirds of the half-girder flange width was thus selected for the experiments. For a W24 × 94 girder, this equates to a gap of 2 in. between the column web and doubler plate. The finite element analyses (Ye et al., 2000) also showed that the shear strains carried by the offset doublers were similar to the strains in the doublers placed directly against the column web.

Continuity Plate Detail

Specimen CR3 included continuity plates with a thickness of approximately half the girder flange thickness, and continuity plates fillet-welded to both the column flanges and doubler plates. These approximately half-thickness, fillet-welded continuity plates were shown to perform adequately in the monotonically loaded pull-plate tests (Prochnow et al., 2000; Hajjar et al., 2003). This detail was also tested in this experimental program primarily to investigate its suitability for the seismic applications. The detail may be considered an economical alternative to the full-thickness, groove-welded continuity plates typically specified following the Northridge earthquake. A 5-in.-wide plate, which is slightly larger than half of the girder flange width, was selected in compliance with the requirements of $t/b = 1.79\sqrt{F_y/E}$ and $b \geq b_{gf}/3 - t_p/2$ contained in AISC (1999a) (see Figure 3). In addition, 1 in. clips were provided to avoid interference with the fillet welds connecting the doubler plates to the column flanges.

Panel Zone Design

As one focus of the cruciform tests is to investigate the design provisions for panel zones and column stiffeners and to test new stiffening alternatives, the panel zones were designed with the intent to exceed the shear deformation of $4\gamma_y$, where

Table 2. Comparison of Panel Zone Strengths				
Specimen	P_z/P_g (nominal)	P_z/P_g (measured)	$\phi_v R_v/R_u$ (AISC)	t_p/t_{req} (FEMA 350)
CR1	1.02	1.05	0.72	0.61
CR2	0.93	0.99	0.66	0.67
CR3	1.05	1.12	0.74	0.79
CR4	1.31	1.49	0.93	1.01
CR4R	1.31	1.47	0.93	1.01
CR5	1.04	1.21	0.74	0.82

γ_y is the panel zone shear yield strain. The design shear deformation level of $4\gamma_y$ was suggested by Krawinkler et al. (1971) and Krawinkler (1978) and is implied in the AISC nonseismic and seismic panel zone design equations (AISC, 1999a; AISC, 2002). Designing the specimens with relatively weak panel zones ensures that all column reinforcement details are rigorously tested through large localized cyclic strains, and provides a means for evaluating the strength of the panel zone at the level of design deformation.

It is also desirable, however, to ensure the panel zones are strong enough to allow for development of the plastic moment strength of the girders. This is necessary to develop large girder flange forces, thereby placing high force demands on the column flanges and continuity plates. To meet this balance of girder and panel zone strength, a method of estimating the relative strengths was used for the design of panel zones in this experimental study. The quantity of most interest for the purpose of the panel zone design was the ratio of nominal panel zone strength (P_z) to nominal girder strength (P_g). These strengths were calculated as the total girder tip loads required to reach the strength level under consideration, and are given by

$$P_z = \frac{0.6F_{yc}d_c t_p\left(1+\dfrac{3b_{cf}t_{cf}^2}{d_g d_c t_p}\right)}{\left(\dfrac{L_g}{d_g}-\dfrac{L_g+d_c/2}{L_c}\right)} \tag{15}$$

$$P_g = \frac{2Z_g F_{yg}}{L_g} \tag{16}$$

where

L_g = girder length between load pin and column face

L_c = column length between top and bottom load pins

These equations are similar to those used by El-Tawil et al. (1998) to make strength ratio comparisons in their parametric finite element studies. In Equation 15, the panel zone strength is based on the AISC Seismic Provisions (AISC, 1997a, 1999b, 2000, 2002) with a resistance factor of ϕ_v = 1.0. The girder strength presented in Equation 16 is calculated from the summation of the girder plastic moments at the column face.

A baseline value of P_z/P_g equal to 1.0 was targeted for the design of the panel zones. This implies that the panel zone strength (at an average shear distortion of $4\gamma_y$) is achieved at the same time the girders reach their plastic moment capacities. By selecting this ratio, the intent is to achieve the goals of exceeding both plastic moment M_p in the girders and shear distortion of $4\gamma_y$ in the panel zones. Table 2 expresses the panel zone strengths of the tested six specimens in terms of P_z/P_g ratio (using both the nominal and measured material strengths; the coupon test results of the material are presented in a later section), the $\phi_v R_v/R_u$ ratio [based on the AISC Seismic Provisions (2002) and nominal material properties], and the t_p/t_{req} ratio [where t_{req} was calculated from FEMA (2000a) using nominal material properties]. The panel zone required strength, R_u, was calculated from Equation 10.

As shown in Table 2, significant deviations from the target P_z/P_g were made for Specimens CR4 and CR4R. These specimens feature the box (offset) doubler plate detail, and it was expected that this type of detail would be less than fully effective, based on the results of Bertero et al. (1973). Thus, the doubler plates provided were approximately 30% thicker than those of Specimen CR3, which has the same W24×94 girder section and W14×176 column section. The capacity-to-demand ratios given in Table 2 for AISC (2002) and FEMA 350 (FEMA, 2000a) procedures reveal that the panel zones of most of the specimens are weak.

Description of Test Specimens

Table 3 presents the design strength-to-demand ratios using nominal material properties for the LFB, LWY, and panel zone limit states using various methods of required strength

Table 3. Nominal Capacity/Demand Ratios of PZ Yielding, LFB and LWY Limit States										
Specimen	$\frac{\sum M^*_{pc}}{\sum M^*_{pb}}$ (AISC)	PZ $\phi_v R_v/R_u$ (AISC)	LFB $\phi R_n/R_u$				LWY $\phi R_n/R_u$			
			Eq. 4[a]	Eq. 5[a]	Eq. 6[a]	Eq. 7[a]	Eq. 4[a]	Eq. 5[a]	Eq. 6[a]	Eq. 7[a]
CR1	1.50	0.72	3.04 (3.38)[b]	1.69 (1.88)	1.64 (1.82)	2.51 (2.78)	2.38	1.32	1.29	1.97
CR2	0.99	0.66	1.47 (1.63)	0.82 (0.91)	0.80 (0.89)	1.22 (1.36)	2.20	1.22	1.19	1.82
CR3	0.89	0.74	1.22 (1.36)	0.68 (0.76)	0.66 (0.73)	1.01 (1.12)	2.51	1.39	1.36	2.07
CR4 (&CR4R)	0.89	0.93	1.22 (1.36)	0.68 (0.76)	0.66 (0.73)	1.01 (1.12)	3.19	1.77	1.73	2.64
CR5	0.73	0.74	0.84 (0.93)	0.47 (0.52)	0.46 (0.51)	0.70 (0.78)	2.34	1.30	1.27	1.94

[a] Equation used to calculate demand, R_u.
[b] Values in parentheses reflect use of ϕ = 1.0.

calculations; in other words, using Equations 4 through 7. The design strengths shown for LFB and LWY are the design strengths of the column shape alone and do not include the column reinforcement, if any. Also shown in Table 3 are the column-girder moment ratios calculated from the 2002 AISC Seismic Provisions (AISC, 2002), assuming no axial compression in the column. In this table, the panel zone capacity-to-demand ratios presented in Table 2 are repeated for the comparison with other column reinforcement design criteria. As mentioned above, panel zone strengths are based on the AISC Seismic Provisions (AISC, 1997a, 1999b, 2000, 2002), with a resistance factor of ϕ_v = 1.0, while the LFB and LWY strengths are based on the AISC LRFD Specification (AISC, 1993, 1999a). Table 3 shows that Equations 5 and 6 provide similar required strength values. Key aspects of the specimens are discussed below, focusing on the details of the column stiffening and the limit states targeted in each test.

Specimen CR1 represents a relatively large, unreinforced interior connection with a relatively weak panel zone. It is intended primarily to study the panel zone strength provision for thick column flanges. The relatively thick column flange of 2.07 in. coupled with the unreinforced panel zone results in a post-elastic panel zone strength contribution of approximately 40% in Equation 8. This specimen meets the Strong Column-Weak Beam (SCWB) criteria of the 2002 AISC Seismic Provisions (AISC, 2002), also shown in Table 3 in the first column. No continuity plates are needed as per the 1992 AISC Seismic Provisions (AISC, 1992) (in other words, using Equation 5) or the AISC Design Guide No. 13 (AISC, 1999c) (in other words, using Equation 6). Specimen CR1 is also intended to show that continuity plates are not necessarily needed for all seismic moment connection details.

Specimen CR2 represents a moderately-sized, reinforced interior connection with a single-sided doubler plate. It is intended primarily as a verification of the LFB criteria of the 1992 AISC Seismic Provisions (AISC, 1992) and the AISC Design Guide No. 13 (AISC, 1999c). This specimen is also intended to confirm that continuity plates are not always needed for seismic moment connections. A relatively weak panel zone, similar to Specimen CR1, is provided. A square-cut fillet-welded doubler plate detail (Detail II) is utilized. The SCWB moment ratio is close to unity in Specimen CR2. The presence of the doubler plate and thinner column flanges reduces the value of the post-elastic panel zone strength contribution in Equation 8 to approximately 17%.

Specimen CR3 represents a moderately-sized, reinforced interior connection with both doubler plates and continuity plates. This is the second test of doubler plate Detail II. Specimen CR3 requires continuity plates as per the 1992 AISC Seismic Provisions (AISC, 1992) and the AISC Design Guide No. 13 (1999c), and is intended to show that a fillet-welded continuity plate detail using a continuity plate that is approximately half as thick as the girder flange can perform satisfactorily in cyclic loading applications. The nominal strength of this continuity plate, computed as

$$2\phi_t P_n = 2\phi_t A_g F_y = 2\left[(0.9)(50)(0.5)(5.0-1.0)\right] = 180 \text{ kips}$$

was approximately 30% less than the required strength:

$$\begin{aligned} &1.8F_{yg}A_{gf} - \phi(6.25)t_{cf}^2 F_{yc} \\ &= 1.8(50)(9.07)(0.875) - 0.9(6.25)(1.31)^2(50) \\ &= 232 \text{ kips} \end{aligned}$$

(The nominal strength is approximately equal to the required strength with a ϕ factor of 1.0 used for local flange bending). The value of 1.0 in the parenthetical difference term accounts for 1 in. clips in the continuity plate. The SCWB moment ratio is lower than unity in this specimen. The panel zone strength is similar to Specimens CR1 and CR2. Because of the thinner column flanges and heavier panel zone reinforcement as compared with the preceding two specimens, the predicted post-elastic strength of the panel zone is reduced to approximately 12%.

Specimen CR4 (and CR4R) represents a moderately sized, reinforced interior connection with relatively heavy panel zone reinforcement using the box (offset) detail and no continuity plates in a situation in which continuity plates would be required according to the 1992 AISC Seismic Provisions (AISC, 1992) and the AISC Design Guide No. 13 (AISC, 1999c). Doubler plate Detail III is used as the column reinforcement of this specimen in order to investigate the feasibility of this detail to resist both panel zone shear and column local flange bending. Unlike the other specimens, a relatively strong panel zone is provided, based on the recommendations of Bertero et al. (1973) on a similar stiffening detail tested on a smaller column size. The panel zone design thus meets the requirements of FEMA (2000a) (in other words, the panel zone thickness is larger than that calculated by Equation 13), although it is just below the design strength from AISC (2002). As with Specimen CR3, this specimen does not meet the SCWB criteria of AISC (2002) for the case of no axial compression. The thick doubler plates result in a post-elastic panel zone strength contribution just below 10%.

Specimen CR5 represents the smallest column section tested, with fillet-welded doubler plates and no continuity plates. Doubler plate Detail I, the back-beveled fillet-welded detail, is tested in Specimen CR5. This specimen requires continuity plates as per the nonseismic and seismic (AISC, 1999a; AISC, 1992, 1999c) design requirements [in other words, LFB, $\phi R_n/R_u$ = 0.84 as per AISC (1999a), $\phi R_n/R_u$ = 0.47 as per AISC (1992), and $\phi R_n/R_u$ = 0.46 as per AISC (1999c)], but no continuity plates were used. While tested cyclically, the nonseismic details of this specimen (in other words, lack of continuity plates) were intended to investigate the LFB design criteria of the AISC LRFD Specification (1999a) as well as to provide further evidence that continuity plates may not be required in all seismic moment connections. The panel zone strength is similar to Specimens CR1, CR2, and CR3. Because a smaller column was needed to breach the nonseismic LFB limits, the SCWB moment ratio is much lower than unity in Specimen CR5. A post-elastic panel zone strength contribution of approximately 8% is calculated for this specimen from Equation 8.

Weld Details

The weld details were as recommended in FEMA (2000a) for the WUF-W connection, as modified in connections tested by Ricles et al. (2002a, 2002b). Figures 2 and 3 illustrate the typical connection welding details used to fabricate the specimens in this experimental study (represented by the details for Specimens CR1 and CR3, respectively). All welding was done with the Flux-Cored Arc Welding (FCAW) process. The welding was done in two stages, with shop welds made at the fabricator and field welds made in the Structural Engineering Laboratory at the University of Minnesota by experienced erection welders.

E70T-1 (Lincoln Outershield 70) wire with 100% O_2 shielding gas was used for all shop welding, including the shear tab welding to the column flange (see Figure 2), the welding of the doubler plates (see Figure 3), and the welding of the continuity plates (see Figure 3). The notch toughness of the E70T-1 wire is required by AWS A5.20 (AWS, 1995) to be 20 ft-lb at 0 °F and, according to the Lincoln Electric product family literature, the typical values for Outershield 70 are 23 ft-lb at –20 °F and 28 ft-lb at 0 °F (Lincoln Electric Company, 2003). The shear tab was welded using ¼ in. fillet welds on each side of the plate, although this deviated from the recommended WUF-W connection welding details, which require partial-joint-penetration (PJP) groove welds at this location.

The field welds were made with the self-shielded FCAW process. The girder flange-to-column flange CJP groove welds were made in the flat position with E70T-6 (Lincoln Innershield NR-305) wire. Welds made with E70T-6 wire are required by AWS A5.20 (AWS, 1995) and AISC (2002) to have notch toughness of 20 ft-lb at –20 °F and 40 ft-lb at 70 °F. FEMA (2000a) has recommended minimum notch toughness requirements at two temperatures, 20 ft-lb at 0 °F and 40 ft-lb at 70 °F. According to the Lincoln Electric Company product family literature (Lincoln Electric Company, 2004), the typical values for NR-305 are 21 to 35 ft-lb at –20 °F and 21 to 54 ft-lb at 0 °F.

For the first two specimens that were fabricated (in other words, Specimens CR1 and CR4), 5/64-in.-diameter NR-305 wire was used for girder flange-to-column flange CJP groove welds. However, this wire and the weld procedures were subsequently found to produce weld metal with only 2 to 3 ft-lb at 0 °F that did not meet the FEMA (2000a) recommended minimum notch toughness requirements [a discussion of the possible causes for this low toughness is presented in Lee et al. (2002)]. For this reason, for the remaining specimens, it was decided to use a lot of NR-305 weld wire with 3/32-in.-diameter that was previously characterized by the Edison Welding Institute (EWI) and was known to have good notch toughness (Lee et al., 2002). This particular lot of 3/32-in.-diameter NR-305 wire was used for the CJP welds in Specimens CR2, CR3, CR4R, and CR5. In addition, as shown in Table 4, different welding equipment and procedures were also used for the remaining specimens. All CJP welds were ultrasonically tested by a certified inspector in conformance with Table 6.3 of AWS D1.1-2000 (AWS, 2000) for cyclically loaded joints.

Table 4. Summary of Parameters Used for CJP Welds						
	CR1	CR4	CR2	CR3	CR4R	CR5
Electrode Manufacturer	Lincoln	Lincoln	Lincoln	Lincoln	Lincoln	Lincoln
Trade Name	NR-305	NR-305	NR-305	NR-305	NR-305	NR-305
AWS Designation	E70T-6	E70T-6	E70T-6	E70T-6	E70T-6	E70T-6
Electrode Type	FCAW-s	FCAW-s	FCAW-s	FCAW-s	FCAW-s	FCAW-s
Electrode Diameter (in.)	$^5/_{64}$	$^5/_{64}$	$^3/_{32}$	$^3/_{32}$	$^3/_{32}$	$^3/_{32}$
Power Supply	Miller Maxtron 450	Miller Inverter	Lincoln DC-600	Lincoln DC-600	Lincoln DC-600	Lincoln DC-600
Wire Feeder	Miller S-64	Lincoln LN-25	Lincoln LN-10	Lincoln LN-10	Lincoln LN-10	Lincoln LN-10
Voltage (V)	28.5–29.5	29–30	26–28	26–28	26–28	26–28
WFS (ipm)	380	380	280	280	280	280
Current (A)	430–460	330–380	470–500	470–500	470–500	470–500
Preheat (°F)	150 min.	150 min.	50 min.	50 min.	50 min.	50 min.
Interpass (°F)	150 min.	150 min.	50 min.	50 min.	50 min.	50 min.
Electrode Extension (in.)	1	1	1	1	1	1
Travel Speed (ipm)	10–15	10–15	10–15	10–15	10–15	10–15

The out-of-position field welds, including the CJP welds connecting the girder web to the column flange and all reinforcing fillet welds were made with 0.068 in. diameter E71T-8 (Lincoln Innershield NR-203MP) wire for Specimens CR1 and CR4, and $^5/_{64}$-in.-diameter E71T-8 (Lincoln Innershield NR-232) wire for the other specimens (see Figure 2). Welds made with E71T-8 wire are required by AWS A5.20 (AWS, 1995) to have notch toughness of 20 ft-lb at –20 °F. According to the Lincoln Electric Company product family literature (Lincoln Electric Company, 2004), the typical values for NR-203MP are 50 to 200 ft-lb at –20 °F and the typical values for NR-232 are 20 to 69 ft-lb at –20 °F.

As shown in Figure 4, the shear tab was designed to extend approximately 0.25 in. into the top and bottom access holes and acted as the backing bar for the CJP welds of the girder web to the column flange. This extension acted as a short runoff tab, allowing the weld to extend the full depth of the girder web. Ricles et al. (2002a, 2002b) recommended that these runoff tabs of the vertical web weld be ground smooth, which is labor intensive. Since it was felt that this might not be necessary, these runoff tabs were not ground smooth in the specimens tested in the present study.

As shown in Figure 2, $^5/_{16}$ in. reinforcing fillet welds were placed under the top girder flange backing bar and below the back-gouged region under the bottom girder flange. Supplemental fillet welds were also provided between the shear tab and girder web, with $^5/_{16}$ in. fillet welds being placed along the full height of the shear tab using the E71T-8 wire.

Weld Access Hole Detail

The weld access holes for the test specimens were designed based on the research of Mao, Ricles, Lu and Fisher (2001) and Ricles et al. (2002a, 2002b) and are similar to those shown in AISC (2002). Figure 4 illustrates the dimensions of the access hole used in this experimental study. The key parameter in this type of access hole is the slope of the flat transition region between the girder flange and the drilled

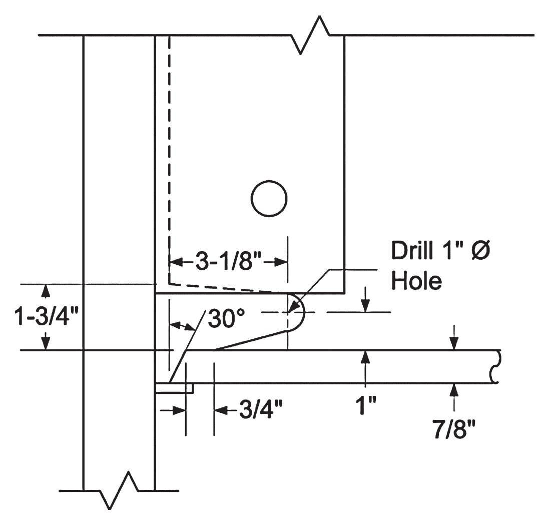


Fig. 4. Weld access hole detail.

Table 5. W-Shape Tensile Properties														
	W24×94 flange (All Girders Except CR3)	W24×94 web (All Girders Except CR3)	W24×94 flange (CR3 Girder)	W24×94 web (CR3 Girder)	W14×283 flange (CR1 Column)	W14×283 web (CR1 Column)	W14×193 flange (CR2 Column)	W14×193 web (CR2 Column)	W14×176 flange (CR3 & CR4 Column)	W14×176 web (CR3 & CR4 Column)	W14×176 flange (CR4R Column)	W14×176 web (CR4R Column)	W14×145 flange (CR5 Column)	W14×145 web (CR5 Column)
Coupon Test Results														
$F_{y,dyn}$ (ksi)	50.6	59.7	54.3	60.0	50.7	52.3	50.1	54.0	55.2	57.5	54.3	56.8	56.6	58.7
$F_{y,st}$ (ksi)	46.4	55.0	NA	NA	47.8	48.8	46.4	49.8	51.8	53.6	NA	NA	52.9	54.8
F_u (ksi)	69.2	74.1	72.3	7 6.0	73.1	72.6	72.2	72.4	76.6	76.1	73.8	74.3	77.2	77.2
E x 10^3 (ksi)	28.3	29.5	NA	NA	28.2	29.9	29.8	29.7	29.5	29.8	NA	NA	29.1	29.4
E_{sh} (ksi)	535	272	NA	NA	636	572	572	479	564	486	NA	NA	507	500
Y/T (%)	73.1	80.6	75.1	78.9	69.1	72.0	69.4	74.6	72.1	75.5	73.6	76.4	73.3	76.1
% Elong.	30.7	25.0	34.5	83.5	31.3	29.7	31.8	28.0	29.0	27.1	34.0	34.0	27.0	26.1
Mill Test Results														
F_y (ksi)	50.0		NA		54.0		54.5		57.0		55.0		57.5	
F_u (ksi)	68.5		NA		73.5		74.0		76.0		72.0		76.5	
% Elong.	27.5		NA		22.0		25.5		25.0		27.0		21.5	

Table 6. Plate Material Tensile Properties			
Plate Thickness	F_y (ksi)	F_u (ksi)	% Elongation
1/2 in. (CR3)	NA	NA	NA
5/8 in.[a] (CR2 & CR5)	62.0	81.0	34.0
5/8 in.[b] (CR2 & CR5)	57.8	81.0	80.5
3/4 in.[b] (CR4)	48.8	73.2	33.8
3/4 in.[b] (CR4R)	57.5	77.3	31.0

[a] Properties obtained from mill test report.
[b] Properties obtained from coupon test.

Table 7. Tested Weld Material Properties (E70T-6 only)						
	E70T-6[a] 5/64 in. wire		E70T-6 3/32 in. wire			
	CR1[a]	CR4[a]	CR2	CR3	CR4R	CR5
CVN @ 0 °F (ft-lb)	2.6	2.0	34.3	44.3	33.0	33.0
CVN @ 70 °F (ft-lb)	19.3	2.3	54.3	73.3	58.7	53.7
F_y (ksi)	NA	NA	59.5	50.0	56.0	53.5
F_u (ksi)	NA	NA	79.5	72.5	78.2	75.5
% Elongation	NA	NA	25.0	23.0	27.5	26.0

[a] For Specimens CR1 and CR4, the CVN tests were performed on specimens machined after the experiment from the welds that did not fracture in the cruciform joints.

hole. A shallow slope reduces the plastic strain demand at the toe of the transition (Mao et al., 2001), delaying the onset of low cycle fatigue (LCF) cracks at this location. Using the dimensions of the access hole shown in Figure 4, a slope of 15 degrees was provided for the test specimens. As mentioned above, the runoff tabs were not ground smooth as they were in the specimens tested by Ricles et al. (2002a, 2002b).

MATERIAL PROPERTIES AND TEST SETUP

Material testing was performed on all wide-flange shapes and available stiffener plates used for the test specimens. All rolled sections were fabricated from ASTM A992 wide-flange sections, and ASTM A572 Grade 50 steel was selected for all stiffener materials. For each W-section, two coupons were taken from the flanges and two from the web. The edge of web coupons was taken no closer than 2 in. from the *k*-line, as recommended in the SAC (1997) testing protocol. For each thickness of the stiffener plate, when the materials were available, two or three coupons were prepared and tested. Tables 5 and 6 summarize the tensile test results (average values of the coupons are shown) and mill certificate values for the W-shapes and the stiffener plates. The ASTM specification for A992 steel specifies the yield strength between 50 and 65 ksi, a minimum tensile strength of 65 ksi, a maximum *Y*/*T* of 0.85, and a minimum elongation of 18%. Referring to Table 5, all shapes met these requirements of the ASTM specification when the dynamic (0.2% offset) yield strength values are used.

In order to verify the material properties of the girder flange-to-column flange CJP welds, one weld test plate was made as per Figure 2A of AWS A5.20-95 (AWS, 1995) for Specimens CR2, CR3, CR4R, and CR5 at the time of the connection welding. The test results, which followed ASTM E23 for the Charpy V-Notch (CVN) impact test and ASTM E8 for the tensile coupon test, are presented in Table 7. Table 7 also shows the CVN test results for Specimens CR1 and CR4. These CVN specimens were machined from one of the girder flange-to-column flange groove welds that showed no signs of fracture after the test. Specimens CR2, CR3, CR4R, and CR5 satisfied the minimum requirements discussed in FEMA (2000a), in other words, filler metals providing CVN toughness of 20 ft-lbs at 0 °F and 40 ft-lb at 70 °F, minimum tensile strength above 70 ksi, and elongation above 22%. As shown in Table 7, however, only the CJP welds in Specimen CR2 satisfied the supplemental requirements in FEMA (2000b) of the minimum filler metal yield strength of 58 ksi.

Figure 5 shows a schematic of the test setup. The load pins placed at the top and bottom of the column were designed to allow free rotation of the column ends during loading, simulating inflection points at the mid-height of the column in steel moment frames. The total length of the column was 171 in., measured to the pin centerlines. Each girder length, measured from the centerline of the column to the centerline of the actuator attachment, was 140 in. The effective length of the girders measured from the face of the column to the point of load application was approximately 132 in. For the W24×94 girders, this effective length requires approximately 96 kips of load to reach the nominal plastic moment strength of 12,700 kip-in.

Four bracing members, not shown in Figure 5, were attached to the diagonal load frame members to restrict the out-of-plane deformation of the girders due to lateral-torsional buckling. These braces were placed approximately 95 in. from the column face on both sides. This is in accordance with the AISC Seismic Provisions (2002) which, for the W24×94 section, limits the unbraced length to 0.086 $r_y E_s / F_y = 98.8$ in.

Quasi-static, anti-symmetric, cyclic loads were applied to the girder tips by using four MTS hydraulic actuators, in other words, two actuators for each side. Each actuator was capable of 77 kips at a stroke of +/– 6.0 in. The SAC (1997) loading history was applied to ensure results could be compared to numerous other SAC girder-to-column tests conducted, in which six cycles were applied at each interstory drift level of 0.375%, 0.5%, and 0.75%, and four cycles were applied at 1.0% interstory drift level, and two cycles were applied at each interstory drift level of 1.5%, 2.0%, 3.0%, and 4.0%. Completing two cycles at 4.0% interstory drift also satisfies the loading sequence requirement for qualifying beam-to-column connections according to AISC (2002).

TEST RESULTS AND DISCUSSION

All specimens, excluding Specimen CR4, completed the SAC (1997) loading history up to 4.0% interstory drift without noticeable strength degradation. The plots of moment versus total plastic rotation for one representative connection from each of the five specimens are presented in Figures 6 through 10. After completing the two cycles at 4.0% interstory drift required by the SAC (1997) protocol, additional 4.0% interstory drift cycles were applied until each specimen failed. Specimens CR1, CR2, CR3, and CR4R were subjected to 14, 16, 14, and 12 cycles, respectively, before significant strength degradation was noticed. It cannot be determined that there is any significance to the variation in number of cycles in the range from 12 to 16 cycles. Thus, it can be assumed that these four specimens performed equally well. Specimen CR5 also performed satisfactorily, completing six cycles at 4.0% interstory drift even though the column stiffeners of this specimen were significantly underdesigned as per the AISC Seismic Provisions (1997a, 2002) and the AISC LRFD Specification (1993, 1999a).

In these five successful tests, the primary failure mode was low cycle fatigue cracking and rupturing near the girder flange-to-column flange junction region. Visible cracking in the connections typically first occurred at the top or bottom edge of the shear tab in the 4.0% interstory drift cycles in some specimens (and in the second cycle at 2.0% interstory drift in Specimen CR5), but the connections suffered no strength degradation until the girder flanges were locally buckled or until the low cycle fatigue cracks in the girder flange were significant after several 4.0% drift cycles. Except for Specimen CR5, the initial girder flange cracks in these specimens originated in the center of girder flange width (in both the top and bottom flanges in the various specimens) at the toe of the girder-to-column fillet welds that reinforced the topside of the CJP welds. This initiation occurred visibly in the 3.0% interstory drift cycles for Specimens CR2 (second cycle), CR3 (second cycle), and CR5 (first cycle), in the 4.0% interstory drift in Specimens CR1 (eleventh cycle) and

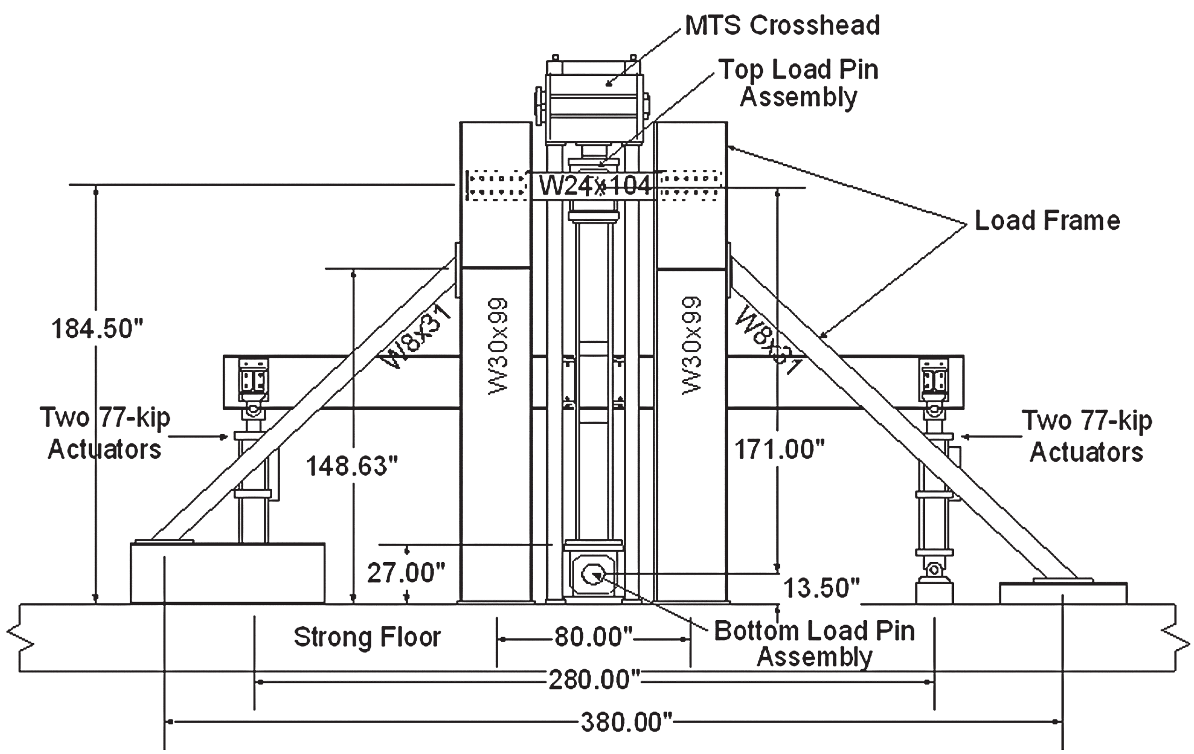


Fig. 5. Experimental test setup.

CR4R (second cycle). However, as discussed previously and seen in Figures 6 through 10, no strength degradation occurred until these cracks started propagating, which occurred after several cycles at 4.0% interstory drift. A typical girder flange failure, from Specimen CR2, is shown in Figure 11. On the other hand, as shown in Figure 12, the major crack in Specimen CR5 was observed in the middle of the CJP welds instead of at the toe of the CJP welds. Details of the progression of failure in each specimen are given in Lee et al. (2002).

Finite element analysis (FEA) was conducted on the five cruciform specimens for comparison to the experimental results. A detailed discussion of the models and results can be found in Ye et al. (2000). For computational efficiency, half of each specimen was modeled, using the mid-plane of the girder and column webs as a plane of symmetry. The nominal

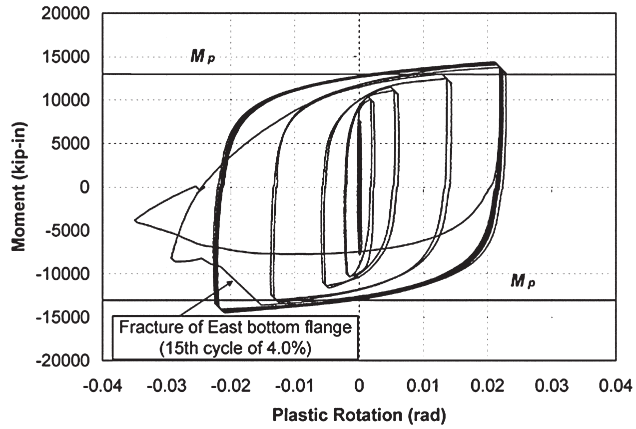


Fig. 8. Moment versus total plastic rotation for Specimen CR3 (East connection).

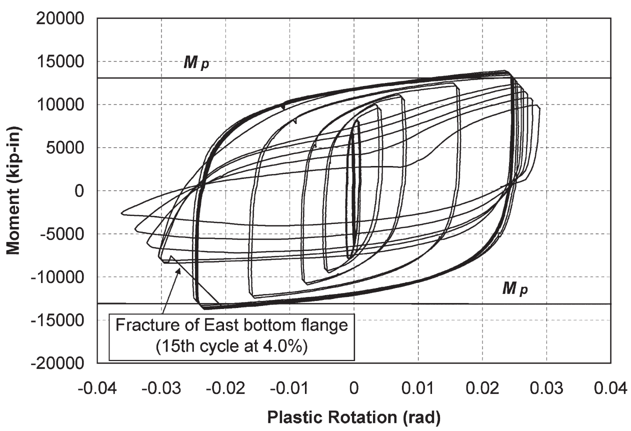


Fig. 6. Moment versus total plastic rotation for Specimen CR1 (East connection).

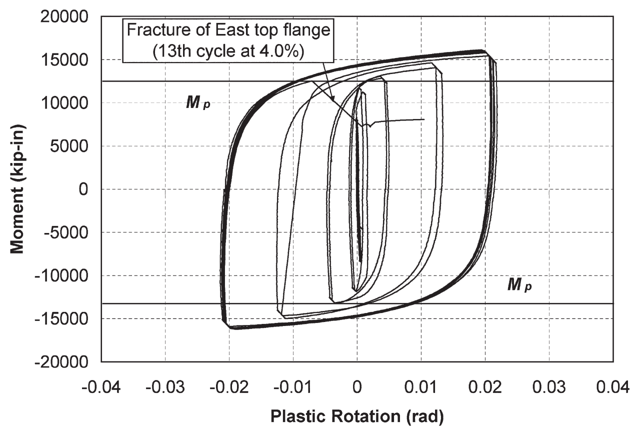


Fig. 9. Moment versus total plastic rotation for Specimen CR4R (East connection).

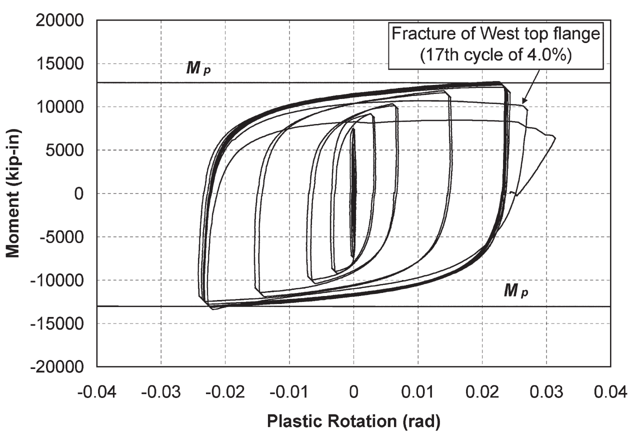


Fig. 7. Moment versus total plastic rotation for Specimen CR2 (West connection).

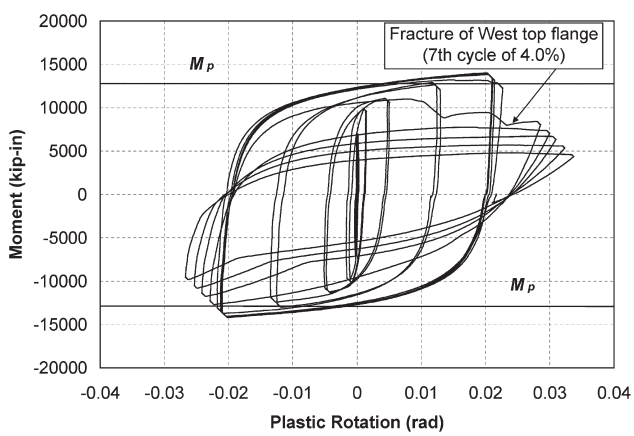


Fig. 10. Moment versus total plastic rotation for Specimen CR5 (West connection).

Fig. 11. Typical low cycle fatigue (LCF) rupture in cruciform specimens (Specimen CR2).

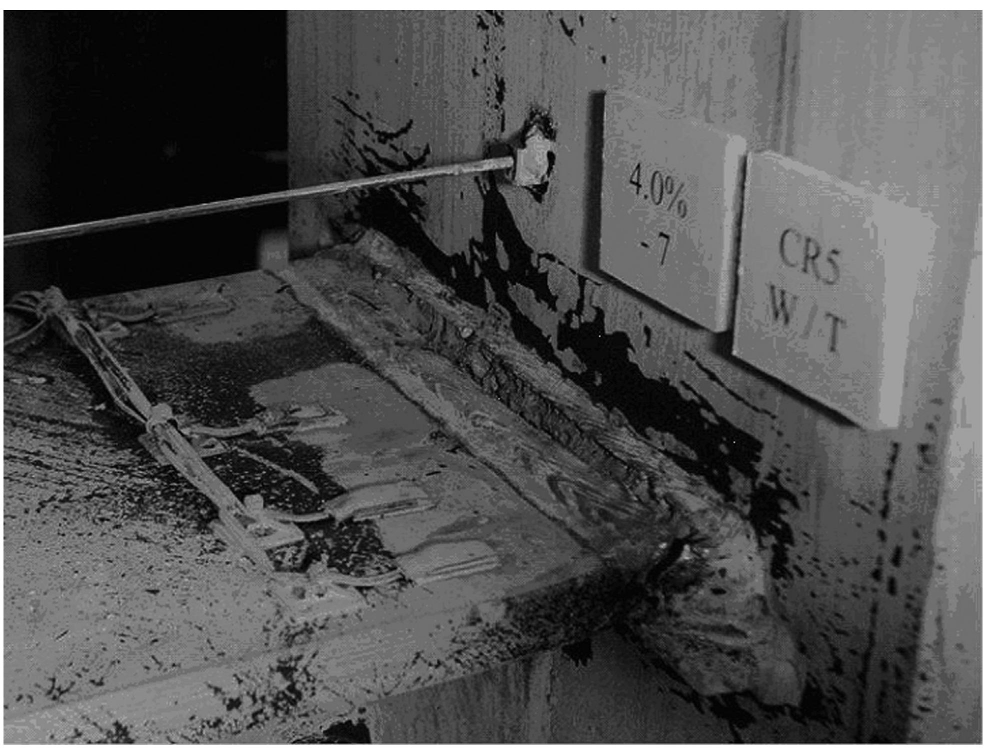


Fig. 12. Low cycle fatigue (LCF) rupture in Specimen CR5.

dimensions of all shapes were used to construct the models. Eight-node solid elements were used in the connection region, while two-node beam elements were used for the portions of girders and columns expected to remain elastic. Four layers of solid elements were used through the thickness of the girder flanges, girder webs, and column webs. Three layers were used through the thickness of the column flanges. Smaller element sizes were used in the connection regions in areas of expected high stress and strain gradients. The welds connecting the girder flanges and web to the column, as well as all stiffener welds, were explicitly modeled. Meshes were refined until convergence was seen in the results. A displacement controlled, antisymmetric load history was applied to the ends of the girders. For computational efficiency, monotonic displacements were applied to the girders at the same drift increments as used in the experiments and specified by the SAC (1997) protocol.

The yield and tensile strength properties used in the models from Ye et al. (2000) were taken from mill report data. The shape of the stress-strain curve was based on a study by Frank (1999), and included a yield plateau at the measured dynamic yield stress, F_y, a strain-hardening region spanning from 8.73 to 86.2 times the corresponding yield strain, followed by a plateau at the measured tensile strength, F_u. The static, nonlinear analyses were conducted accounting for both material and geometric nonlinearity. Figure 13 shows a typical comparison of results from the FEA as compared to the backbone of the experimental results for the East girder of Specimen CR3, plotting plastic rotation versus moment in the girder at the column face. These plots reveal the typically strong correlation between the experimental and computational results. Nonlinearity was observed earlier in the tests than predicted by the numerical analysis, possibly due to residual stress effects, which were not modeled in the FEA. The loads from the FEA are also somewhat under-predicted at larger drift levels, most likely due to the effects of cyclic strain hardening that were also not modeled in the monotonic analyses.

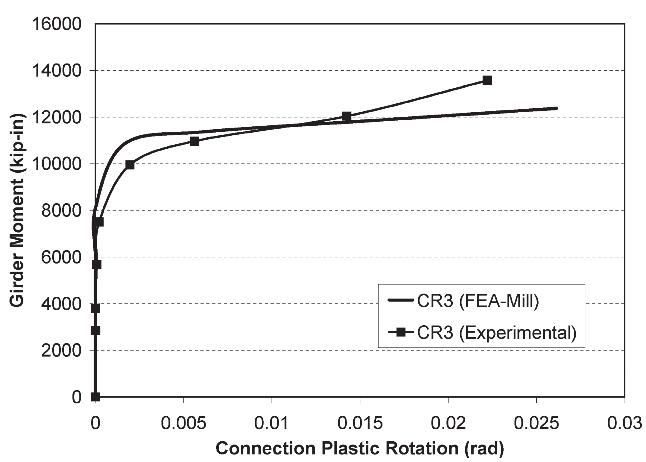


Fig. 13. Comparison of experimental and FEA girder moment versus plastic rotation for Specimen CR3.

Brittle Failure in Specimen CR4

Specimen CR4 exhibited premature brittle failure in three of four girder flange-to-column flange CJP welds in the early stage of the SAC (1997) loading history. The test of Specimen CR4 was stopped after one-half cycle at 2.0% interstory drift due to the fracturing of these three girder flange welds.

The welds were sectioned and polished and the sections and fracture surfaces were examined using a light microscope and scanning electron microscope (Lee et al., 2002). Charpy specimens and samples for chemical analysis were extracted from the welds. It was found that this specimen was unintentionally prepared with low toughness weld metal, having an average of 2.0 ft-lb at 0 °F and 2.3 ft-lb at 70 °F as shown in Table 7. In contrast, the FEMA (2000a) recommendations require 20 ft-lb at 0 °F and 40 ft-lb at 70 °F as a minimum notch toughness. The welds produced for the AWS Certificate of Conformance for this filler metal met the minimum toughness requirement of AWS A5.20 (AWS, 1995) of 20 ft-lb at –20 °F (Lincoln Electric Co., 1999).

Specimen CR4R was essentially a replicate test of Specimen CR4, except that the batch of weld metal used for Specimen CR4R met the minimum requirements of FEMA (2000a). In contrast to the performance of Specimen CR4, Specimen CR4R not only performed acceptably according to the SAC (1997) requirements, it performed as well as any of the other specimens successfully tested in this experimental study. This result is an example of the importance of weld metal notch toughness in achieving good seismic performance of groove-welded (fully restrained) connections. The fact that the box (offset) doubler plate detail performed well in Specimen CR4R indicates that the detail itself was probably not a factor in the fracture that occurred in Specimen CR4. These results also indicate that the relatively high stiffness of the panel zone in Specimen CR4 and CR4R was probably not a factor in the premature fractures in Specimen CR4.

Weld Metal Notch Toughness

Following the premature brittle failure in Specimen CR4, it was found that the previously tested Specimen CR1 also had relatively low weld-metal notch toughness, with an average of 2.6 ft-lb at 0 °F and 19.3 ft-lb at 70 °F as presented in Table 7. Specimen CR1, which was welded using the same wire and the similar welding unit as Specimen CR4 (see Table 4), performed very well, experiencing 14 cycles of 4.0%

interstory drift before the significant strength degradation. If the difference in the column stiffening between Specimens CR1 and CR4 is not a factor in the fracture of Specimen CR4, then the better performance of Specimen CR1 shows that the marginal difference in notch toughness between this specimen and Specimen CR4 may be sufficient to resist fracture. Thus, these two experiments have potentially closely bounded the approximate minimum notch toughness required for adequate performance of CJP welds.

In addition to this investigation of these brittle fractures (Lee et al., 2002), an outside group commissioned by AISC also investigated the fracture (Barsom and Pellegrino, 2001). At the end of these investigations, no definitive conclusions were obtained explaining the reason for the low toughness of the welds of both specimens.

Based on these results, further evaluation of the present weld toughness criteria is suggested. It is believed that the SAC requirements (FEMA, 2000a) for minimum toughness are adequate, provided they can be consistently met. The acceptable results of Specimen CR1, despite toughness below the specified minimums, are encouraging in this regard. FEMA (2000b) requires toughness testing on each production lot of the specified filler metal. However, upon approval of the Engineer, this requirement may be waived and the consumable manufacturer's certification testing may be used to verify the material's suitability (FEMA, 2000b). The certification testing need only be conducted once per year on a single production lot of the particular electrode. As described above, the 5/64-in.-diameter E70T-6 produced in 1999 had been certified by the manufacturer as meeting the minimum 20 ft-lb at –20 °F required by the AWS certification test (AWS, 1995).

Doubler Plate Detail

This test program featured a range of doubler plate details. Three different doubler plate details, presented in Figure 1, were tested under excessive strain demand conditions developed by large girder flange forces coupled with large panel zone deformations. Specimen CR5, reinforced by doubler plate Detail I (back-beveled fillet-welded detail), satisfactorily completed the SAC (1997) loading history. Specimens CR2 and CR3 showed better cyclic connection performance, when compared with the test results of Specimen CR5, with the doubler plate Detail II (square-cut fillet-welded detail), although the column flanges and panel zones in general were stronger in these specimens, which more likely contributed to the difference in performance. In addition, the comparable performance of Specimen CR4R relative to Specimen CR3, which consisted of the same girder and column sizes, showed that the doubler plate Detail III (box detail) can also provide a similarly ductile connection performance and is equally effective in functioning as continuity plates. This finding was also indicated in the pull-plate tests conducted as part of this research (Prochnow et al., 2000; Hajjar et al., 2003), and the cruciform tests verify that cyclic loading test does not change those conclusions. Within the limited number of experiments, it has been found that the above three different variations had no significant impact on the cyclic connection performance. Thus, it cannot be concluded that any of those details are more advantageous. Instead, the most economical detail may be recommended.

Continuity Plate Detail

The continuity plates of Specimen CR3 were heavily instrumented and largely showed strains below yield for the full loading history. Figure 14 shows some of the largest strain magnitudes achieved, measuring strain in the continuity plate in a direction parallel to the column web at a location close to the fillet weld to the column flange, opposite the east top girder flange when that girder flange was in compression. At 4.0% interstory drift, some yielding was just beginning (strains just greater than 0.2%) on the continuity plate near the 1 in. clip (see Figure 3). Note that because of the clip, this cross section of the continuity plate has reduced area relative to the remainder of the continuity plate, which remained elastic.

While the multi-axial strain state in a continuity plate is complex, these results imply that the plastic moment may be achieved in the girder without significantly yielding the continuity plate. The fact that this specimen performed well provides two important conclusions. First, when continuity plates are used, it should not be necessary to use full thickness continuity plates that are groove-welded to the column flanges. Second, the use of smaller fillet-weld sizes is feasible for attaching thinner continuity plates to the column flanges. For Specimen CR3, only ⅜ in. fillet welds on each side were required to connect the ½ in. continuity plates to

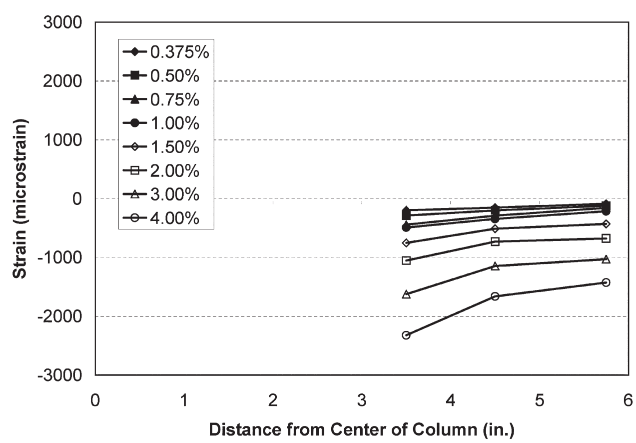


Fig. 14. Strain distribution in Specimen CR3 continuity plate close to flange fillet weld (East girder top flange in compression).

the column flanges, and 5⁄16 in. fillet welds were required to connect to the doubler plates (see Figure 3). Fillet welds, especially relatively small fillet welds such as these, pose a less significant risk of causing *k*-area cracking in the column due to the lower restraint and resultant tensile residual stress. It is thus proposed that it may be sufficient to design the continuity plate size and associated fillet welds using procedures given in the AISC Design Guide No.13 for both nonseismic and seismic design (AISC, 1999c).

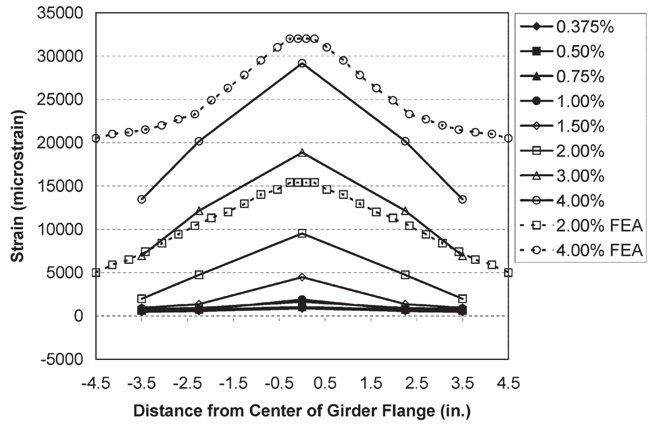
Previously reported pull-plate tests performed as part of this project (Prochnow et al., 2000; Prochnow et al., 2002; Hajjar et al., 2003) also support these conclusions. Two pull plate specimens with continuity plates that were only half as thick as the pull plates and were fillet-welded to both the column web and flanges performed satisfactorily, and performed as well as full-thickness groove welded continuity plates. The half-thickness continuity plates did not yield across the entire full-width region of the plates, and the plates effectively restrained the column section from excessive web yielding or flange bending. The fillet welds made with an E70T-1 electrode did not fracture.

The 1999 AISC LRFD Specification (AISC, 1999a) explicitly requires that the fillet welds develop the full strength of the continuity plate. This implies that the welds are required to essentially remain elastic when the plate is fully plastified. In this way, it is assured that the fillet welds are not the weak link in the column details. Although the fillet welds in these test specimens were designed for this criterion, the fact that the continuity plates are not fully plastified across their gross section indicates that the fillet welds need not necessarily develop the full plastic strength of the continuity plate. This issue often arises when continuity plates are sized greater than they need to be for LFB, to use a particular standard thickness for example or to accommodate a weak axis connection. In these cases the continuity plate will clearly not be yielded and the welds need not develop the full plate strength but rather need only to provide strength greater than the difference between the required strength and the LFB design strength of the flanges without continuity plates (accounting appropriately for stresses induced by a weak axis connection as well).

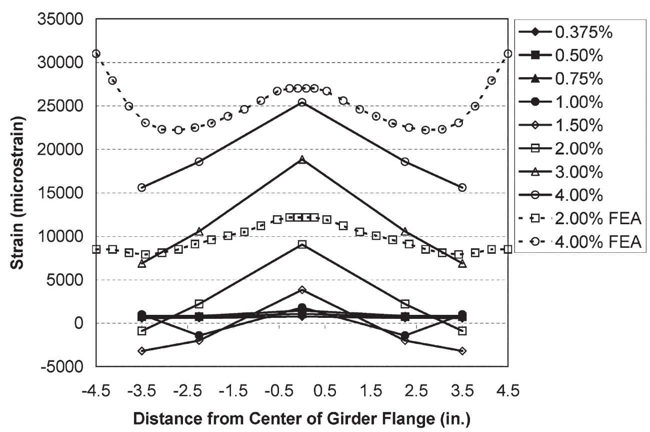
Strain Distribution in Girder Flange

To understand the complex stress and strain distributions and force flows near the girder-to-column junction, the distributions of strains in the longitudinal direction of girder flanges near the CJP welds were investigated (Lee et al., 2002). In all five successfully tested specimens, the maximum longitudinal tensile strains measured in the middle of the West girder top flanges were within the range of 19,000 to 26,500 $\mu\varepsilon$ at the first peak of 4.0% interstory drift. Similarly, the maximum longitudinal tensile strains in the middle of the East girder bottom flange were within the range of 10,000 to 33,000 $\mu\varepsilon$ at the first peak at 4.0% interstory drift. Figure 15 shows typical strain gradients in the East girder bottom flange in Specimens CR2 and CR3, without and with continuity plates, respectively. In Figure 15a, finite element results are shown for interstory drift levels of 2.0% and 4.0%. At 4.0% interstory drift, the computed strains agree fairly well with the measured strains. However, the computed strains were somewhat higher than measured strains at 2.0% interstory drift.

Although the peak strain levels are about the same, Specimens CR3 and CR4R (not shown in the figure) showed relatively lower strain gradients along the girder flange width at higher drifts as compared with the other three specimens. It is believed that the trend towards having lower strain



(a)



(b)

Fig. 15. Longitudinal strain on East girder bottom flange near column flange CJP weld (girder flange in tension): (a) Specimen CR2; (b) Specimen CR3.

gradients in Specimens CR3 and CR4R girder flanges is primarily due to their column reinforcement details. These results reinforce that both the half-thickness continuity plates (in the case of Specimen CR3) and the box (offset) doubler plate detail (in the case of Specimen CR4R) are effective as column stiffeners to mitigate the local flange bending in column flanges.

However, while the general trends in the results are as discussed above, Figure 15 shows that the actual difference in strain gradients between a specimen with and without a stiffened flange may often be subtle. For example, Specimen CR1 had low notch toughness [far less than the FEMA (2000a) requirements], yet the high strain gradient did not cause a brittle fracture. In addition, a low-cycle fatigue crack in the girder flange-to-column flange CJP weld of Specimens CR2, CR3 and CR5 developed in the 3.0% interstory drift rather than in the 4.0% interstory drift cycles as in Specimens CR1 and CR4R, although the specimens showed no detrimental behavior in connection performance until several cycles at 4.0% interstory drift were achieved, as discussed above. However, the strain gradient was also worse in the West girder top flange of Specimen CR1 than in Specimen CR2 (not shown in the figure), whereas Specimen CR1 did not require continuity plates and Specimen CR2 did, based upon the seismic girder required strength. Therefore, some of the variation in the strain gradient among the different specimens may be somewhat random, based upon local residual stresses, etc., and this research indicates that having a strain gradient in the girder flange does not necessarily precipitate premature fracture. These results provide evidence that if a column flange is sufficiently thick, such as for Specimen CR1, continuity plates should not be necessary.

Local Flange Bending and Local Web Yielding Design Criteria

While the pull-plate tests (Prochnow et al., 2000; Hajjar et al., 2003) investigated both the LFB and LWY limit states, the investigation of both nonseismic and seismic design criteria for LWY was limited with the cruciform tests. As shown in Table 3, the selected five cruciform specimens satisfied all the LWY design criteria considered in this research program, as would be customary for girder-to-column cruciform connections.

In the pull-plate tests (Prochnow et al., 2000; Hajjar et al., 2003), the LFB yield mechanism was defined by limiting the column flange separation, measured between the column flanges at the edges of the pull-plates (simulating the girder flanges). A flange separation limit of ¼ in. between the two flanges was established for the nonseismic design (Prochnow et al., 2000). In the cyclic cruciform experiments, this limiting deformation was taken as one-half the pull-plate separation value of ¼ in., as the deformation of only one flange measured. The measured maximum column flange displacements in Specimens CR1 and CR5 were thus compared with a limit of ⅛ in. Specimen CR1 developed relatively small column flange deformations up to 4.0% interstory drift cycles (just 26% of the ⅛ in. limit), as was expected from the large LFB design strength/demand ratios presented in Table 3. In Specimen CR5, an unexpectedly low maximum column flange deformation (49% of the ⅛ in. limit) was measured during 4.0% interstory drift cycles even though Specimen CR5 did not even meet the nonseismic LFB design criteria. These tests thus indicate that the LFB strength predicted by Equation 1 (AISC, 1999a) may be suitable for use for seismic design as well.

With respect to seismic demand, Specimen CR5 was the most substantially under-designed cruciform specimen with respect to LFB. As shown in Table 3, the design strength-to-demand ratio was only 0.84 for nonseismic required strength as per the AISC LRFD Specification (1999a), and it equaled to 0.47 and 0.46 for the seismic required strengths within the 1992 AISC Seismic Provisions (AISC, 1992) and the AISC Design Guide No. 13 (AISC, 1999c), respectively. However, Specimen CR5 performed satisfactorily up to 4.0% interstory drift cycles without any damage induced by the lack of LFB mitigation. Therefore, within this limited experimental study of the WUF-W connection, it may be possible to consider the use of the LFB strength provisions of AISC (1993, 1999a) (in other words, Equation 1) to be sufficient even for seismic loading, complementing similar conclusions reached by Prochnow et al. (2000) related to nonseismic design.

However, at a minimum, observation of the five successful cruciform tests and the pull-plate tests indicated that the seismic required strength given by Equation 6 (AISC, 1999c) is adequate and conservative for determining seismic LFB required strength when the design strength presented in Equation 1 is used. Equation 6 provides a more rational basis for calculation of the required force than does Equation 5 (both equations often yield similar values). Table 3 indicates that both Specimens CR2 and CR5 would require continuity plates if Equation 1 is used in conjunction with Equation 6 (using nominal properties). Placing a reduction factor of 0.85 on Equation 6, particularly for use with interior connections, would change these values to 0.94 and 0.54 respectively, putting Specimen CR2 just over the cusp of breaching the LFB limit state.

Use of Equation 6 with a reduction factor of 0.85 for interior connections, coupled with Equation 1 to compute strength, also compares favorably when compared to other test results. For example, Specimens C1 and C3 from Ricles et al. (2002a, 2002b), which each had two W36×150 (50 ksi) girders framing into a W14×398 (50 ksi) column (for Specimen C1) and a W27×258 (50 ksi) column (for Specimen C3) using the WUF-W connection with no continuity plates, performed adequately through the SAC (1997) loading history. Their ratio of LFB design strength to required strength using Equation 6 with a reduction factor of 0.85 was 2.32 and 0.76, respectively. Thus, Specimen C1 clearly should not need continuity plates. Specimens CR2 from this work and Specimen C3 (Ricles et al., 2002a, 2002b) could also both go without continuity plates, thus showing that the proposed 0.85 factor on Equation 6 would often be conservative.

Panel Zone Behavior

All of the WUF-W specimens, shown in Table 2, had inadequate panel zone strengths as per AISC (1999b, 2002). In spite of the weaker panel zones, however, these specimens (excluding Specimen CR4) showed stable, ductile panel zone response. The analyses of the panel zone elastic and inelastic behavior indicated significant energy dissipation in this region for Specimens CR1, CR2 and CR3. Relatively mild energy dissipation was observed in Specimens CR4R and CR5 even though the measured maximum amount of the connection total plastic rotation was similar in all cases. The smaller panel zone energy dissipation in these two specimens were mostly caused by the design of a stronger panel zone in the case of Specimen CR4R, and by the larger column flange yielding around each girder flange in the case of Specimen CR5.

As discussed further in Lee et al. (2005) and Ricles et al. (2002a, 2002b), columns with weaker panel zones will cause higher stress concentrations at the ends of the weld of the girder web to the column flange, or in the adjacent shear tab weld. However, with the exception of Specimen CR5, which was well under-designed for both panel zone shear and local flange bending, cracking at the top and bottom of the web welds occurred only during the 4.0% interstory drift cycles after having completed the SAC loading history. Additionally, fracturing in the shear tab edges seemed to be more directly affected by girder flange local buckling, low cycle fatigue crack opening in the girder flanges, or both, under large connection deformations. Local buckling in the bottom girder flange, for example, can increase the inelastic demand in the top girder flange, which may contribute to the initial crack at the shear tab top edge. A detailed discussion of the panel zone behavior in this research, and evaluation of the seismic panel zone design criteria, are presented in the companion paper (Lee et al., 2005).

CONCLUSIONS

A total of six full-scale interior steel moment connections, classified as Welded Unreinforced Flange-Welded Web (WUF-W) connections, were tested to investigate the current criteria of column reinforcement design and detailing, and to provide economical alternative column stiffener details that avoid welding in the *k*-area of the columns. These experimental results were corroborated by finite element analyses (Ye et al., 2000) and by related research conducted by the authors on pull plate specimens subjected to monotonic loading (Prochnow et al., 2000). Based on the study of cyclic connection performance, several conclusions may be made:

1. Specimens CR1, CR2, CR3, CR4R and CR5 completed the SAC (1997) loading history up to 4.0% interstory drift cycles without any significant strength degradation in the connections, and ductile failure modes were observed in all specimens [completing two cycles at 4.0% interstory drift without significant strength degradation also satisfies the loading sequence requirement for qualifying beam-to-column connections according to AISC (2002)]. The primary failure mode of these five specimens was low cycle fatigue crack growth and eventual rupture of one or more girder flange-to-column flange complete-joint-penetration (CJP) groove welds. The cracks initiated typically at the toe of the fillet welds that reinforced the topside of these CJP welds. A crack became visible in the 3.0% interstory drift cycles for Specimens CR2 (second cycle), CR3 (second cycle), and CR5 (first cycle), in the 4.0% interstory drift in Specimens CR1 (eleventh cycle) and CR4R (second cycle). However, no strength degradation occurred until these cracks started propagating, which occurred after several cycles at 4.0% interstory drift. The weld access hole detail chosen for this experimental study showed good performance under repeated large cyclic connection deformations. No low cycle fatigue cracking occurred at the toe of the weld access hole prior to significant cracking occurring elsewhere in the connection, particularly at the toe of the CJP welds of the girder flange to the column flange.

2. These experimental results showed that, when properly detailed and welded with notch-tough filler metal, the WUF-W steel moment connections can perform adequately under large quasi-static cyclic loads even though relatively weak panel zones and low local flange bending strengths were chosen as per the current design provisions and recommendations (AISC, 1992, 1997a, 1999a, 1999b, 2000, 2002; FEMA, 2000a).

3. Specimen CR4 was unintentionally prepared with weld metal that had Charpy V-Notch (CVN) values that were much lower than the minimum requirements of FEMA (2000a). This was the only test that did not complete the SAC (1997) loading history with two cycles at 4.0% interstory drift level. The premature brittle failure of this specimen reconfirmed that achieving the required minimum CVN toughness in the girder flange-to-column flange CJP welds is critical for good performance in steel moment connections.

4. Application of the alternative column reinforcement details (including a back-beveled fillet-welded doubler plate detail; a square-cut fillet-welded doubler plate detail; a groove-welded box (offset) doubler plate detail; and fillet-welded ½-in.-thick continuity plates) in the WUF-W steel moment connections was successfully verified. No cracks or distortions were observed in the welds connecting these stiffeners to column flanges before the rupturing of the girder flange-to-column flange CJP welds. Additionally, no cracking occurred in the *k*-area of the columns in these column-stiffened specimens.

5. For the range of column sections and doubler plate detailing investigated in this work, strain gradients and strain magnitudes well above the yield strain in the girder flanges did not prevent the specimens from achieving the connection prequalification requirement of completing two cycles at 4.0% interstory drift without significant strength degradation. This was even the case for Specimen CR1, which had notch toughness in the CJP weld metal connecting the girder flange to the column flange that was significantly below the requirements in the FEMA (2000a, 2000b) guidelines. In addition, for the cruciform specimens, the measured maximum column flange deformation due to the concentrated girder flange force in the unstiffened specimens ranged from 26% of the assumed yield mechanism limit of ⅛ in. flange deformation in the case of Specimen CR1, to 49% of the ⅛ in. limit in the case of Specimen CR5. Specimen CR5 was the most substantially under-designed specimen for local flange bending—the design strength-to-required strength ratio was only 0.84 for nonseismic required strength as per AISC (1999a), and it was equal to 0.47 for seismic required strength as per AISC (1992). Specimens CR1, CR2, CR4R and CR5, none of which had continuity plates (although Specimen CR4R included the offset doubler plate detail), showed ductile connection behavior even though only Specimen CR1 met the seismic requirements of AISC (1992) and FEMA (2000a) with respect to continuity plates for the limit state of local flange bending. These results indicate that the column reinforcement detailing may not have a significant effect on the potential for brittle fracture at the girder flange-to-column flange weld and that continuity plates may thus not be necessary in interior columns in steel moment connections that satisfy the limit state equations based upon the column flange or web thicknesses. Note that this conclusion is contrary to previous finite-element analyses reported in the literature using theoretical fracture criteria that have predicted a significant effect of using or omitting continuity plates. Design provisions similar to those in AISC (1992, 1999a) or FEMA (2000a) permitting the design, or lack of inclusion, of continuity plates are thus recommended for consideration for reintroduction into the AISC Seismic Provisions. Specifically, it is recommended that the provisions for LFB strength of AISC (1992, 1999a) (Equation 1) be considered for seismic design, and that the calculation for LFB required strength according to AISC (1999c) (Equation 6) is conservative and may be considered for use; reducing this LFB required strength by a factor of 0.85 for interior connections is also recommended for consideration.

6. If continuity plates are required, fillet-welded continuity plates that were approximately half of the girder flange thickness performed well. The results showed that only minor local yielding occurred in these continuity plates in a portion of the cross section next to the clip at peak drift level and that these strains were not sufficient to cause cracking or distortion in the continuity plate or to change the strain gradients in the girder flange substantially. These results were consistent with previously reported pull-plate test results (Hajjar et al., 2003). Since continuity plates do not significantly yield, it may not be necessary to size the welds large enough to develop the continuity plate. Rather the weld and the plate may only need to be designed for the difference between the required strength and the design strength of the column shape without the continuity plate (however, more research is required to verify this conclusion, particularly for cyclic applications).

7. The box (offset) doubler plate detail was found to function effectively as continuity plates while simultaneously serving as column web doubler plates. Ye et al. (2000) showed in a finite element parametric study that the doubler plates were likely to be most effective when placed between one-third and two-thirds of the half-girder flange width from the column web. There was insufficient testing to determine definitively an appropriate means of calculating the nominal strength of the doubler plate to quantify its effectiveness as a continuity plate to withstand local flange bending. However, a reasonable calculation for this nominal strength to resist LFB and LWY would be to use a formula based on the limit state of LWY for an end condition, Equation 3. For this calculation, the *k* dimension would be taken as the column flange thickness (since the doubler plate is away from the column fillet), the *N* dimension would be taken as the girder flange thickness, and the thickness would be that of the two doubler plates in the box (offset) detail if they are assumed to resist as a pair the required strength imparted by the girder flange. Using a thickness equal to the larger required for resisting (1) LFB and LWY; and (2) PZ yielding seems reasonable, but further research is necessary to confirm this.

8. In all the five successful tests, the seismic performance of the relatively weak panel zones was stable and ductile, and the panel zones exhibited good energy dissipation.

9. Within the limited number of WUF-W experimental studies, a strong correlation between the panel zone strength and fracturing of the girder web weld to the column flange at its top and bottom edges was not observed [these cracks became visible during the 4.0% interstory drift cycles, except for Specimen CR5 which exhibited cracking at the ends of the shear tab during the 2.0% interstory drift cycles, but with little strength degradation resulting (Lee et al., 2005)]. Instead, fracturing in the shear tab edges seems to be more directly affected by girder flange local buckling, low cycle fatigue crack opening in the girder flanges, or both, under large connection deformations.

Extrapolation of these conclusions beyond WUF-W connections may be reasonable by recognizing that, with the use of notch tough weld metal and a properly detailed weld access hole, neither varying strain gradients in the connected girder flanges nor weak panel zones caused premature fracture in the connections that led to strength degradation. With respect to the response of other connection types, Pantelides, Okahashi and Reaveley (2004) report similar conclusions for RBS connections having no continuity plates, for example. Thus, the conclusions related to the effects of local flange bending of the column on connection performance, and related to the column reinforcement detailing procedures investigated in this work, are likely to be reasonably independent of the connection type for several different types of moment resisting connections in which the girder flanges are welded to the column flange, the girder web is attached is some manner to the column flange, and the resulting flow of forces from the girder to the column is similar to that of the WUF-W connection.

As a result of this study, the following additional research is recommended. First, column reinforcement detailing for deep columns should be assessed. Ricles et al. (2002a, 2002b, 2003) have conducted some preliminary research on using deep columns with WUF-W connections and no continuity plates, showing good results, but indicating that further research is required. Alternately, Chi and Uang (2002) had poorer results using deep column sections due to excessive twisting of the column. Second, continuity plates with undersized fillet welds should be tested to confirm that the weld need not develop the full continuity plate strength. Third, further work is needed to develop a procedure for computing the nominal strength of the doubler plate in the box (offset) detail relative to it serving as a continuity plate to withstand local flange bending and local web yielding in combination with panel zone yielding. Fourth, because of the possibility of obtaining brittle weld metal despite the fact that weld certifications show adequate toughness, additional studies should be considered to characterize the typical variability in the CVN and other properties of the CJP weld. Consideration should be given to use of filler metals and welding procedures with a distribution of CVN such that there is a sufficiently small probability of not meeting the minimum required values. Fifth, the local flange bending and local web yielding criteria proposed in this work, if they are to be adopted for general design purposes, should be evaluated for a wider range of concentrated loading cases (for example, due to support reactions on beams). Finally, similar research for other connection configurations that have a markedly different flow of forces between the girder and column as compared to the WUF-W connection should be conducted to help to bolster the conclusions reported in this work for column reinforcement.

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While this paper was under review, co-author Robert Dexter passed away unexpectedly in November 2004. Dr. Dexter was known nationally and internationally for his work on fracture and fatigue of steel structures. His sudden death cut short a rising career in applied steel research, one that influenced an unusually wide number of areas, including fracture problems in buildings, fatigue in bridges, cracking in ship panels, and collapse of overhead highway signs. He contributed extensively to the writing of the AISC Specification, the AASHTO steel bridge design specification, the RCSC bolt specification, and the AWS welding specification. His contributions to the research reported in this paper were integral throughout the project. He will be greatly missed by his colleagues around the world.

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