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EXPERIMENTAL STUDY ON LINK-TO-COLUMN CONNECTIONS IN STEEL ECCENTRICALLY BRACED FRAMES

Taichiro OKAZAKI[1], Michael D. ENGELHARDT[2], Masayoshi NAKASHIMA[3], and Keiichiro SUITA[4]

SUMMARY

A total of twelve large-scale link-column specimens were tested to study the cyclic loading performance of link-to-column connections in eccentrically braced frames (EBFs). Four different connection types were tested: a connection representing the pre-Northridge practice; a connection incorporating welding improvements; free flange connections, and a no weld access hole connection. Each of the four connection types was tested with three different link lengths in order to consider a wide range of load and deformation environments. Test results from this experimental program indicate that the connections representing pre-Northridge design and construction practices do not allow the link to develop its intended inelastic deformation, even when improvements in welding are employed. Although the free flange connections and the no weld access hole connections showed improved performance, the majority of test specimens did not quite achieve the inelastic deformation required in US seismic building code provisions. This paper provides an overview of this experimental program, and discusses details of the specimens and some key test results.

INTRODUCTION

Eccentrically braced frames are seismic-resistant frame systems in which inelastic action is restricted primarily to ductile links. Some of the typical EBFs are arranged to have one end of the link connected to a column. In such EBFs, the integrity of the link-to-column connection is essential to the ductile performance of the link, and therefore, to the ductile performance and safety of the EBF.

Malley and Popov [1] observed that the large cyclic shear force developed in EBF links could cause repetitive bolt slippage in welded flange-bolted web connections. The bolt slippage ultimately induced sudden failure of the connection by fracture near the link flange groove weld. Based on these observations, exclusive use of welded flange-welded web connections has been mandated for EBF link-to-column connections. Engelhardt and Popov [2] tested long links attached to columns, and observed frequent failures at the link-to-column connections due to fracture of the link flange. Since these failures typically occurred before significant inelastic deformation was developed in the link, it was recommended

1 Research Assistant, Dept. Civil Engineering, The University of Texas at Austin, Austin, Texas, USA.
2 Professor, Dept. Civil Engineering, The University of Texas at Austin, Austin, Texas, USA.
3 Professor, Disaster Prevention Research Institute, Kyoto University, Gokasho, Uji, Kyoto, Japan.
4 Professor, Disaster Prevention Research Institute, Kyoto University, Gokasho, Uji, Kyoto, Japan.

that EBF arrangements with long links (link length, e, of $e > 1.6M_p/V_p$) attached to columns should be avoided.

Besides the exceptions discussed above, EBF link-to-column connections have been designed, detailed, and constructed very similar to beam-to-column connections in moment resisting frames. Therefore, many of the features responsible for the poor performance of moment connections during the 1994 Northridge Earthquake are also present in link-to-column connections in EBFs. However, while extensive research in recent years has resulted in substantial changes in the design and construction of moment connections (FEMA [3]; AISC [4]), less attention has been given to EBF link-to-column connections. On the other hand, the force and deformation environment at EBF link-to-column connections are significantly different, and in some cases more severe than at moment connections. Therefore, many of the new understandings and developments for moment connections may not be applicable to EBF link-to-column connections. Although the *2002 AISC Seismic Provisions for Structural Steel Buildings* (AISC [4]), hereinafter referred to as the *2002 AISC Seismic Provisions*, require qualifying cyclic test results for EBF link-to-column connections, very limited test data are currently available.

Recently, Tsai *et al.* [5] conducted an experimental study to investigate the cyclic performance of EBF shear link-to-box column connections. Besides conventional connections, connections with improved weld access hole configurations (Ricles *et al.* [6]) and with link flange backing bars fillet welded to the column flange were tested. All test specimens failed by fracture of the link flange near the groove weld, before developing the inelastic link rotation required in the *2002 AISC Seismic Provisions*, adding to the concerns for the safety of these connections.

A research program combining an experimental and analytical investigation has been conducted at the University of Texas at Austin to study the performance of link-to-column connections in seismic-resistant EBFs. This paper presents an overview of the experimental part of the research, which featured twelve large-scale cyclic loading tests. Link-column specimens were tested following the qualifying cyclic test procedure specified in Appendix S of the *2002 AISC Seismic Provisions*. To the knowledge of the authors, the tests conducted in this program are the first series of experiments specifically aimed at studying the seismic performance of large-scale EBF link-to-column connections constructed using realistic detailing and welding according to the US practice.

EXPERIMENTAL PROGRAM

Test Setup

The test setup devised for this investigation is shown in Fig. 1. The link-column specimen was connected to the test setup at the link end opposite the column and at the bottom of the column by high-tension bolts. Lateral bracing was provided near the four reaction points at the top and bottom of the column, and near the two ends of the horizontal beam. The loading ram supplied quasi-static vertical motion to the bottom of the column, which in turn, imposed large shear deformation on the link. As illustrated in Fig. 2, the force distributions realized in an EBF arrangement with one end of the link connected to a column (Fig. 2a) was reproduced in the specimen (Fig. 2b). The loading system subjected the link to constant shear, V, along its length, reverse curvature bending moment, M, typically with higher moment at the column end of the link than at the beam end, and negligible axial force, N. The test setup was designed to contain inelastic action primarily in the link of the specimen. Only very limited yielding was observed in the column panel zone during the tests.

Test Specimens

A total of twelve link-column specimens were tested in this program. All specimens were constructed from W18x40 links and W12x120 columns, both of ASTM A992 steel. The sizes of the link and column sections as well as the column height of 2.44-m were selected to represent full or near full-scale

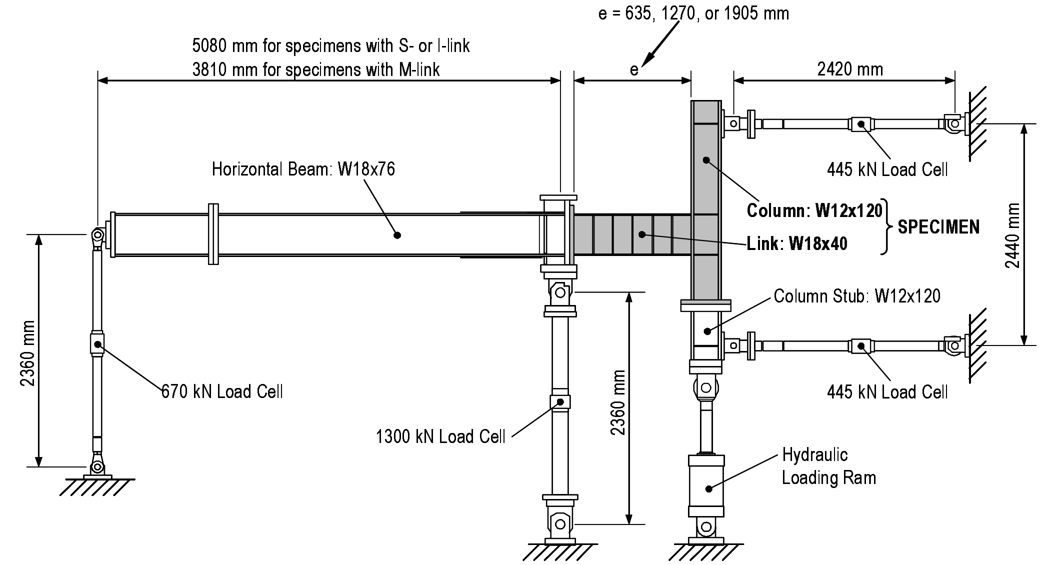


Fig. 1. Test Setup

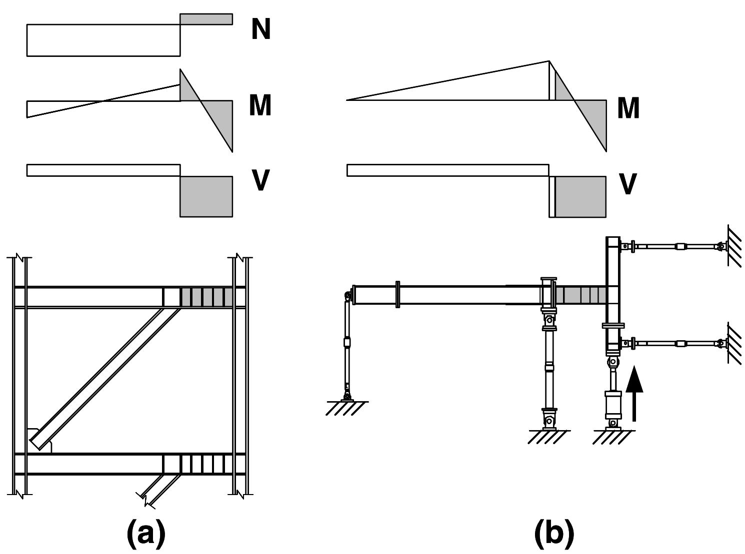


Fig. 2. Force Distribution in (a) Typical EBF link and beam, and in (b) Test setup.

Table 1. Test Section Properties

Section	F_y (MPa)		F_u (MPa)		Elongation (%)	
	Flange	Web	Flange	Web	Flange	Web
W18x40	352	393	499	527	34	31
W12x120	323	353	455	485	29	33

Note: The tabulated F_y is a static yield stress value, measured with the test machine cross-heads stationary. The tabulated F_u is a dynamic ultimate strength, measured with the test machine cross-heads in motion dimensions in actual EBFs. The column was provided with continuity plates, but with no doubler plate. Table 1 summarizes the measured mechanical properties of the test sections for coupon samples taken from the edges of the flanges, and from mid-depth of the web.

The primary test parameters for this investigation were the connection type and the link length. The specifications of each specimen, including the features of the link-to-column connection, link length, and link stiffener spacing, are summarized in Table 2. The links were detailed per the *2002 AISC Seismic Provisions*. The specimen names represent the two test parameters: the first two letters (PN/MW/FF/NA) represent the connection type, while the third letter (S/I/M) represents the link length.

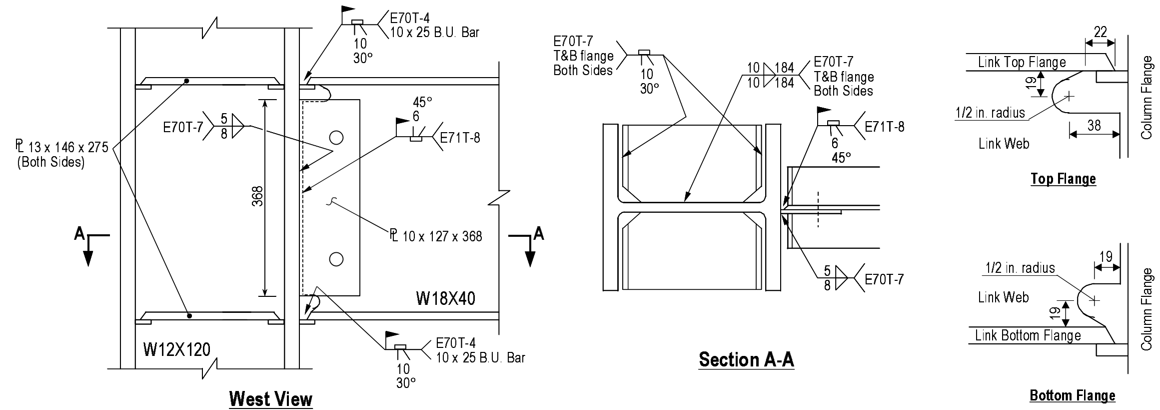
Table 2. Test Specimens

Spec.	Connection Feature	e (mm)	$e/(M_p/V_p)$	Intermediate Stiffeners
PNS	Weld metal with no notch toughness requirement; backing bars and weld tabs left in place.	635	1.11	4@127mm
PNI		1270	2.22	5@191mm
PNM		1905	3.34	229mm from each end
MWS	Weld metal with notch toughness requirement; modified welding details.	635	1.11	4@127mm
MWI		1270	2.22	5@191mm
MWM		1905	3.34	229mm from each end
FFS	Extended free flange length; shear tab welded to link web; modified welding details.	635	1.11	3@127mm
FFI		1270	2.22	4@184mm
FFM		1905	3.34	229mm from each end
NAS	No weld access hole; continuous placing of bottom flange groove weld; modified welding.	635	1.11	4@127mm
NAI		1270	2.22	5@191mm
NAM		1905	3.34	229mm from each end

Note: The stiffener spacing for the FF-specimens were measured starting at the end of the shear tab, as opposed the other three specimen types where the spacing was measured starting at the column face.

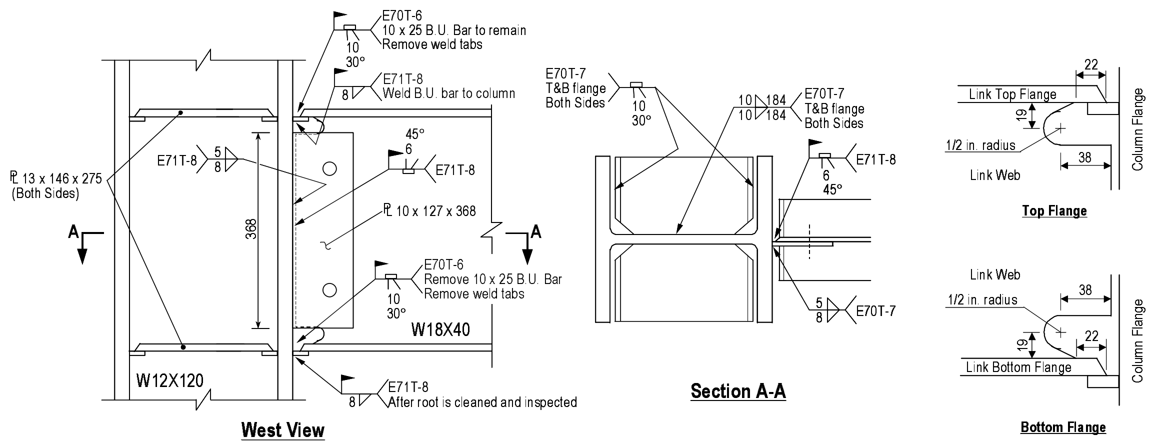
Three link lengths were chosen to represent the different link length categories defined in the *2002 AISC Seismic Provisions*. Shear dominates the inelastic behavior of links shorter than $1.6M_p/V_p$, whereas flexure is dominant for links longer than $2.6M_p/V_p$. Links of intermediate length are affected significantly by both shear and flexure. The provisions also require links to develop different levels of inelastic rotation depending on their length. Therefore, depending on the link length, very different force and deformation environments can be realized at the link-to-column connection. In this paper, links of $e = 1.1M_p/V_p$ are referred to as S-links (shear links); links of $e = 2.2M_p/V_p$ are referred to as I-links (intermediate links); and links of $e = 3.3M_p/V_p$ are referred to as M-links (moment links). The link length parameter, $e/(M_p/V_p)$, described herein was evaluated based on the measured section dimensions and the measured yield strength values.

Link-column specimens with the three link lengths described above were tested with four different connection types, designated as the PN- (Fig. 3), MW- (Fig. 4), FF- (Fig. 5), and NA-types (Fig. 6). The PN-connection featured detailing and construction typical of the pre-Northridge practice. The MW-connection adopted the modifications in welding which are currently widely accepted for moment connections. The FF- and NA-connections were designed based on the free flange moment connection developed by Choi *et al.* [7] and the no weld access hole (“non-scallop”) connections developed in Japan, respectively. Based on extensive literature review, these two connections were selected as the most promising for application to EBF link-to-column connections. Further details of the four connection types are discussed in the following.



(a) Connection Details **(b) Weld access hole**

Fig. 3. PN-Connection



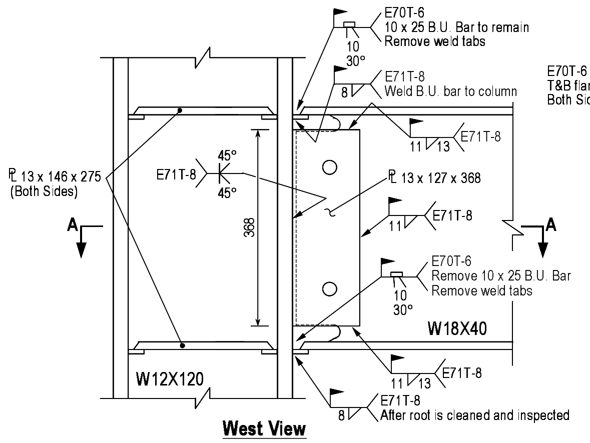
(a) Connection Details **(b) Weld access hole**

Fig. 4. MW-Connection

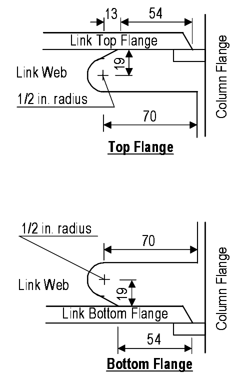
Details of Connection Types

The PN-connection was designed to represent the pre-Northridge practice in detailing and construction for link-to-column connections. As shown in Fig. 3, a self-shielded flux core arc welding (SS-FCAW) process with a 3.0 mm diameter E70T-4 electrode was used for the complete joint penetration (CJP) groove welds connecting the link flanges to the column flange. The low fracture toughness of the weld metal deposited by E70T-4 electrodes has been identified as a significant factor contributing to the premature failure of pre-Northridge moment connections (Engelhardt *et al.* [8]). The backing bars and weld tabs were left in place after completion of the link flange groove weld.

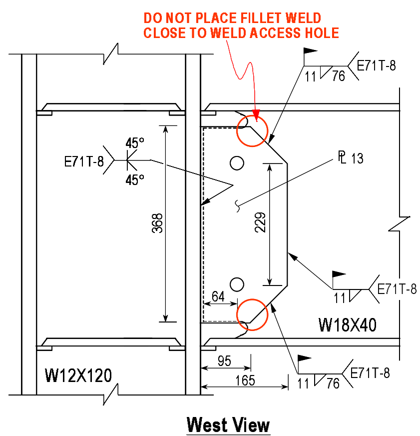
The main feature of the MW-connection, as shown in Fig. 4, was the modification in the link flange groove weld following FEMA [3]. Among the modifications were: the use of a 2.4 mm diameter E70T-6 electrode, which has a designated notch toughness requirement, for the link flange CJP groove welds; removal of the backing bar followed by the placement of a supplemental reinforcement fillet weld at the bottom flange groove weld; fillet welding the top flange backing bar to the column flange; removal of weld tabs at both top and bottom link flanges, and the use of the recommended [3] weld access hole configuration (Fig. 4b). These modifications in welding details were also adopted in the FF- and NA-connections.



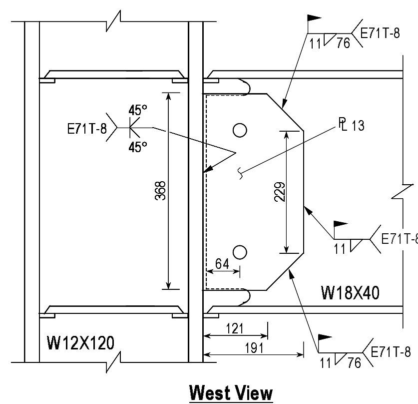
(a) FFS Connection Details



(d) Weld access hole

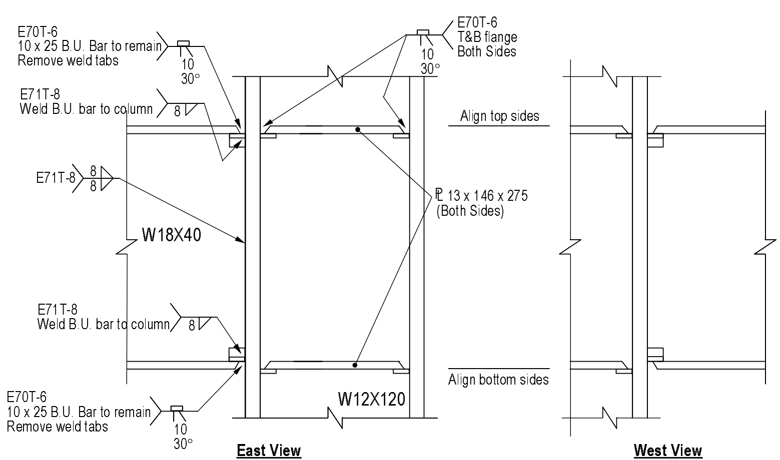


(b) FFI Shear Tab

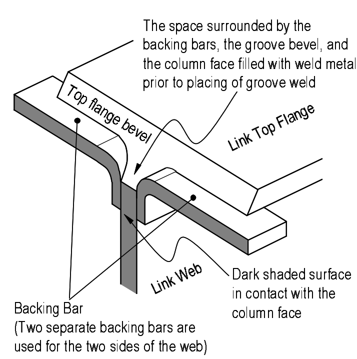


(c) FFM Shear Tab

Fig. 5. FF-Connections



(a) Connection Details



(b) Weld access hole

Fig. 6. NA-Connection

The FF-connections, as shown in Fig. 5, were designed based on the free flange moment connection developed by Choi *et al.* [7]. By extending the weld access hole and welding the shear tab to the link web, the FF-connection reduces the shear force transmitted by the link flanges, and consequently relaxes the local stress and strain demand near the CJP groove welds. While Specimen FFS (Fig. 4a) had a rectangular shear tab, Specimens FFI (Fig. 4b) and FFM (Fig. 4c) had tapered shear tabs. The shear tab was groove welded to the column flange along the entire depth. The link web was welded directly to the column flange. The free flange length (distance between the toe of the weld access hole and column face) was five times the flange thickness, as recommendation by Choi *et al.* Noting that the shear tab provides buckling restraint to the link web, the link stiffeners were placed outside the segment welded to the shear tab, taking the end of the shear tab as one end of the link.

The NA-connection, as shown in Fig. 6, was designed based on the no weld access hole detail developed in Japan. Many Japanese literatures (*e.g.* Suita *et al.* [9]) report excellent performance of moment connections with these details, and demonstrated that fracture initiating at the toe of the weld access hole could be precluded by eliminating the weld access holes. Since Engelhardt and Popov [2] observed fracture of the link flange typically initiating at the toe of the weld access hole, it was believed that elimination of the weld access holes is also beneficial to EBF link-to-column connections. The CJP groove welds at both the top and bottom link flanges were placed continuously without interruption by the web, thereby reducing the likelihood of weld defects in the bottom flange groove weld (Engelhardt *et al.* [8; 10]). Special backing bars were prepared to provide complete closure between the flange-web fillet of the link section and the column face, as shown in Fig. 6b. It is noted that the use of two separate backing bars does not conform to the full length backing required in *AWS D1.1* [11].

The link-to-column connections were welded and finished by a commercial structural steel welder. According to measurements from sample CJP groove welds, the E70T-4 metal had Charpy V-Notch (CVN) ratings of 15 J at –29°C, and 30 J at 21°C. The E70T-6 metal had CVN ratings of 40 J at –29°C, and 70 J at 21°C. While the E70T-6 met the minimum CVN toughness of 27 J at –29°C required in the *2002 AISC Seismic Provisions*, the E70T-4 did not. Welding inspection and ultrasonic testing was performed by a commercial welding inspection and testing firm. Ultrasonic testing of the link flange groove welds was performed per Table 6.2, Chapter 6 of *AWS D1.1* [11]. The link top flange of Specimen MWS was rejected due to discontinuities in the weld. After the discontinuities were removed and the weld was repaired, Specimen MWS passed the inspection and ultrasonic testing. No other weld was rejected.

TEST RESULTS

The cyclic loading sequence provided in Appendix S of the *2002 AISC Seismic Provisions* was used for all twelve tests. This loading sequence requires applying increasing levels of cyclic link rotation angle, γ, which was computed as the displacement of one end of the link relative to the other, divided by the link length. The provisions specify link-to-column connections in EBFs to be evaluated based on inelastic link rotation. The inelastic rotation, γ_p, was evaluated by removing the contributions of elastic response from the link rotation, γ. The S-, I-, and M-links, as defined in this paper, are required to develop an inelastic rotation of 0.08, 0.043, and 0.02 rad, respectively. In this paper, the inelastic rotation capacity, $\gamma_{p\text{-max}}$, is defined as the maximum inelastic link rotation amplitude sustained for at least one full cycle of loading prior to loss of strength of the specimen. Loss of strength is defined as either the link shear force or bending moment measured at the column face dropping to below 80-percent of their respective maximum magnitudes measured during the test.

Tables 3 and 4 summarize the test results. Table 3 lists the $\gamma_{p\text{-max}}$ required in the *2002 AISC Seismic Provisions* alongside the $\gamma_{p\text{-max}}$ achieved by each specimen. Also listed is a brief description of the controlling failure mode for each specimen. Table 4 lists the maximum link shear force, V_{max}, and maximum column face moment, M_{max} (both positive and negative values are provided).

Table 3. Test Results

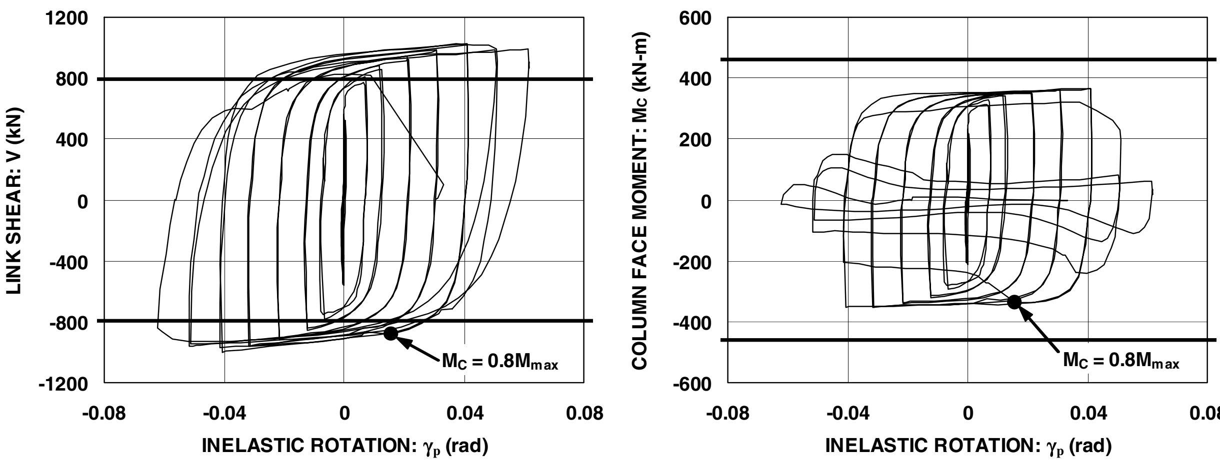
Spec.	$\gamma_{p\text{-}max}$ (rad) Required	$\gamma_{p\text{-}max}$ (rad) Measured	Observed Failure Mode
PNS	0.08	0.041	Fracture of top and bottom flanges near groove weld
PNI	0.043	0.018	Fracture of top flange near groove weld
PNM	0.02	0.008	Fracture of top and bottom flanges near groove weld
MWS	0.08	0.050	Fracture of top and bottom flanges near groove weld
MWI	0.043	0.018	Fracture of top flange near groove weld
MWM	0.02	0.007	Fracture of top and bottom flanges near groove weld
FFS	0.08	0.060	Fracture of web around shear tab
FFI	0.043	0.046	Fracture of top flange near groove weld, and fracture in shear tab/web groove weld
FFM	0.02	0.017	Fracture of top and bottom flanges near groove weld, and fracture in shear tab/link web groove weld
NAS	0.08	0.070	Fracture of web initiating at stiffeners welds
NAI	0.043	0.027	Fracture of top flange near groove weld
NAM	0.02	0.017	Fracture of top flange near groove weld

Table 4. Connection Forces

Spec.	V_{max}		M_{max}	
PNS	-999	+1025	-353	+365
PNI	-807	+748	-498	+484
PNM	-540	+621	-525	+505
MWS	-1051	+1.32	-348	+368
MWI	-795	+1043	-474	+447
MWM	-576	+768	-513	+524
FFS	-1093	+1088	-390	+396
FFI	-889	+899	-548	+580
FFM	-597	+651	-568	+567
NAS	-1100	+1127	-387	+399
NAI	-862	+826	-509	+530
NAM	-600	+597	-531	+557

Note: Based on the measured section dimensions and yield strength values, V_p = 793 kN and M_p = 453 kN-m.

Stable inelastic response was exhibited by the link of each specimen until failure of the specimen occurred. All specimens but NAS failed by fracture at the link-to-column connection. As shown in Table 3, only one specimen, FFI, sustained the link rotation required in the *2002 AISC Seismic Provisions*. Fracture at the link-to-column connection typically caused an abrupt and severe loss of strength, as represented by Specimen PNS. Fig. 7 illustrates the hysteretic response of this specimen and Fig. 8 shows the link-to-column connection after testing. The development of fracture in the link flanges caused severe degradation in the link moment at the face of the column, as shown in Fig. 7b. Specimen PNS failed prematurely, after developing only half of the required inelastic link rotation of 0.08 rad. Gradual strength degradation as observed in many of the moment connections prequalified by FEMA [3] was not typically exhibited by the link-to-column connections. It appeared that the excellent buckling control provided by link stiffeners left fracture at the link-to-column connection as the dominant failure mode of the specimen.



(a) Inelastic link rotation vs. Link shear **(b) Inelastic link rotation vs. Column face moment**

Fig. 7. Hysteretic response of Specimen PNS

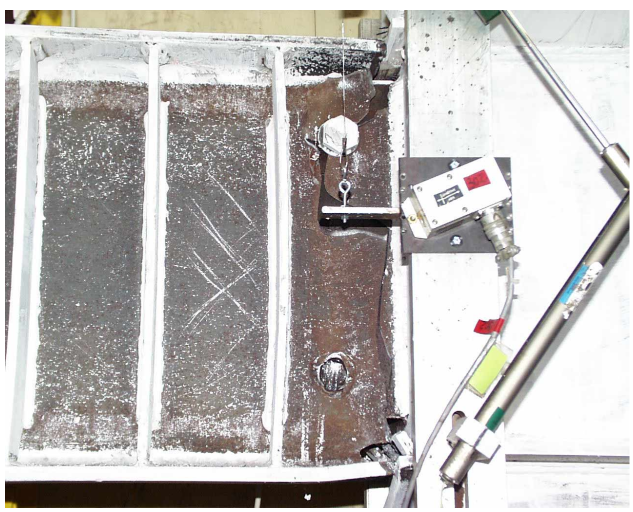


Fig. 8. Specimen PNS after testing

DISCUSSION OF TEST RESTLTS

Connection Type

Four different connection types were examined in this experimental program, including the PN-, MW-, FF-, and NA-connections described earlier. As indicated in Table 3, the connection details had a significant effect on the inelastic link rotation achieved by the specimens.

The PN-connection was designed to represent pre-Northridge practices in detailing and construction of EBF link-to-column connections. The three PN-specimens achieved no more than half of their required inelastic link rotations, and failed due to fracture of the link flange near the groove weld. Fracture initiating at the root of the bottom flange groove weld, as observed in a large number of pre-Northridge moment connections (*e.g.* Engelhardt *et al.* [8; 10]), did not occur in any of the PN-specimens. The absence of this particular failure mode may be attributed to the relatively high CVN values of the E70T-4 weld metal in these specimens, and to the difference in the force and deformation environment between link-to-column connections and moment frame connections. The poor performance of the PN-specimens suggests that link-to-column connections in currently existing EBFs may not perform as intended. The safety of existing EBFs may merit further examination in light of these findings.

The MW-connection featured the use of a weld filler metal with a specified notch toughness requirement and modification in welding details. Comparison of the MW-specimens with the PN-specimens highlights the effects of welding. Specimen MWS achieved a 20-percent improvement in inelastic link rotation over Specimen PNS. However, Specimen MWI achieved no improvement in inelastic link rotation over Specimen PNI, nor did Specimen MWM over Specimen PNM. Similar to the PN-connections, the MW-specimens failed due to fracture of the link flanges near the groove welds. The marginal improvement of the MW-specimens over the PN-specimens suggest that although the modification in welding is beneficial, this alone is not nearly sufficient to improve the connection performance to the required level. Therefore, the conventional EBF link-to-column connection configuration may be inappropriate regardless of the quality of welding. Stojadinovic *et al.* [12] made a similar statement on the effect of the same weld modifications on welded flange-bolted web moment connections. The premature failure due to link flange fracture can occur not only in connections of a long link ($e > 1.6M_p/V_p$) to a column, as noted previously by Engelhardt and Popov [2], but equally as likely in connections with a short shear link, such as a link of $e = 1.1\ M_p/V_p$.

The FF-connections were configured to draw the shear force away from the link flanges, and thereby relax the stress and strain demand near the link flange groove welds. The FF-specimens were successful in preventing or delaying the occurrence of link flange fracture, and sustained significantly greater link rotations compared to the corresponding PN- and MW-specimens with identical link length. Nonetheless, Specimens FFS and FFM failed to meet their link rotation requirements. Specimen FFI failed immediately upon exceeding its link rotation requirement.

The restraint provided by the shear tab welded to the link web had a significant effect on link behavior in the FF-specimens. A direct effect was that yielding was precluded from the segment of the link web welded to the shear tab, as shown in Fig. 9 and Fig. 10, and thus, inelastic rotation was enhanced within an effectively shortened link. The shortening of the link had a more prominent effect on shorter links: the link length measured from the end of the shear tab to the beam end was 80-, 87-, and 90-percent of the full link length, respectively, for the S-, I-, and M-link. Correspondingly, the inelastic link rotation was magnified by roughly 25-, 15-, and 10-percent, for the respective links.

Specimen FFS failed at an inelastic link rotation 25-percent below the required level due to fracture of the link web along the toe of the fillet weld connecting the shear tab to the link web, as shown in Fig. 9. It is believed that the magnification in link rotation combined with the large restraint imposed on the link web material between the shear tab and the link flange generated severe cyclic inelastic strain demands near the right-angled corner of the shear tab, and induced fracture at this location. After noting the disadvantage of rectangular shear tabs, the subsequently tested specimens, FFI and FFM, were provided with tapered shear tabs. The failure mode of Specimen FFS was not reproduced in the other two FF-specimens.

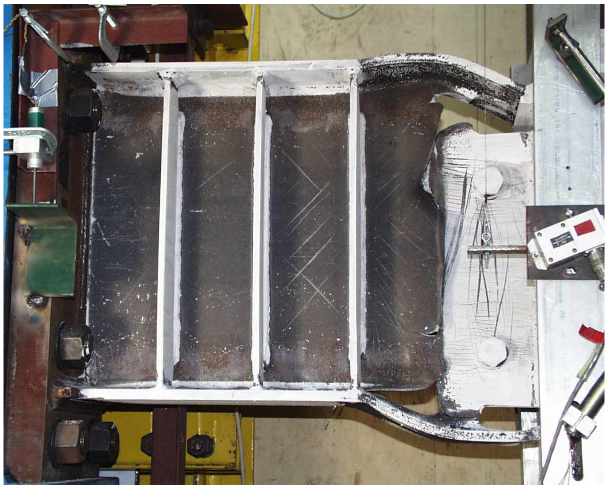


Fig. 9. Specimen FFS after testing

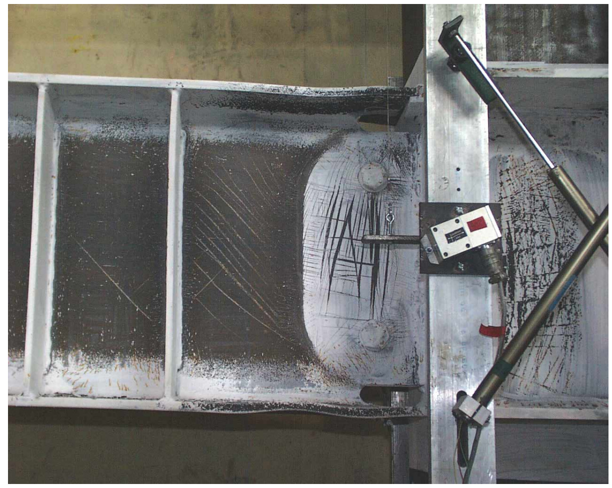


Fig. 10. Specimen FFI after testing

In Specimen FFI, fractures were observed at the top and bottom edges of the shear tab and link web, at the location where the shear tab and link web were welded to the face of the column. Fractures were also simultaneously observed in the link flanges at the CJP groove welds. In Specimen FFM, development of significant fractures in the shear tab and link web clearly preceded fracture initiation in the link flange. In both specimens, as a fracture progressed in the double bevel groove weld connecting the shear tab to the column flange, another fracture progressed at the interface of the link web base metal and groove weld metal. Since the shear tab weld was located at the root of the link web weld, the two fractures developed simultaneously, interacting with each other. It is quite likely that the progression of fractures in the shear tab and link web caused redistribution of bending and shear stress from the link web and shear tab to the link flange, thereby accelerating fracture of the link flange. Therefore, the FF-connections appeared to be very sensitive to fracture initiating in the link web to column flange connection, at the top and bottom edges of the link web and shear tab. Fig.10 shows Specimen FFI after the link top flange separated from the column flange, causing a large drop in link shear force and column face moment. Unlike free flange moment frame connections, yielding in the link did not spread from the flanges into the web. Rather, yielding in the link web spread from near the centroidal axis of the link to along the perimeter of the shear tab, resulting in the distribution of yielding shown in Fig. 10.

The NA-connection featured elimination of the weld access hole and a fabrication procedure that enabled continuous placement of the CJP groove weld at both the link top and bottom flanges. The NA-specimens sustained significantly greater link rotations compared to the corresponding PN- and MW-specimens of the same link length. Specimen NAS achieved an inelastic link rotation of 0.071 rad, short of the required 0.08 rad, before losing strength due to fracture of the link web initiating at the top and bottom terminations of stiffener welds. Crack openings were noted along the toe of the groove welds, and had the link forces not degraded due to link web fracture, these fractures could have progressed more substantially. Nonetheless, it appeared that the link-to-column connection maintained its strength when the test was terminated. The two other NA-specimens with longer links, Specimens NAI and NAM, failed prematurely due to fracture of the link flanges. Fracture of the link flange appeared to be the dominant failure mode of the NA-connection regardless of the link length.

Link Length

Links of length e = 1.1, 2.2, and $3.3M_p/V_p$ were designated as S-, I-, and M-links, respectively, in this research program. The three link lengths were selected to study the performance of link-to-column connections subjected to a wide range of different possible force and deformation environments. Table 3 indicates that the inelastic link rotation achieved by the specimens depended significantly on the link length. This statement applies directly to the link-to-column connections, since all specimens except Specimen NAS failed due to fracture at the connection. Table 4 also indicates that the forces at the link-to-column connection differed significantly depending on the link length. For example, for the MW-specimens, the maximum link shear force ranged from 1050 kN in Specimen MWS to 580 kN in Specimen MWM. The maximum column face moment ranged from 350 kN-m in Specimen MWS to 520 kN-m in Specimen MWM. Meanwhile, the link-to-column connection was required to accommodate different levels of link rotation in accordance with the link length. It is clear that the very significant difference in force and deformation environments in the different length links had a large influence on the performance of the link-to-column connection.

Despite their short length of $e = 1.1M_p/V_p$, flexural yielding was observed in the S-links. Although the column face moment was considerably smaller than the plastic flexural capacity of the link section of 453 kN-m, as listed in Table 4, yielding in the link flanges was noted in the region near the column face in Specimens PNS, MWS, and NAS. This was due to the unequal link end restraints that led to larger moment at the column end of the link than at the beam end, and more significantly due to moment-shear interaction at the link end. Fig. 11 shows the bending strain measured by strain gauges at the three indicated locations in Specimen MWS. The strain hysteresis is shown up to the loading cycle of $\gamma = \pm$ 0.04 rad. The figure indicates that while the link flanges yielded in flexure, the link web did not. The large shear force developed in the link apparently forced the link web to yield solely in shear, and thus precluded development of inelastic bending strain in the link web. Note that the plastic flexural capacity of the link section considering only the flanges was 304 kN-m. Fig. 11 indicates that yielding of the link

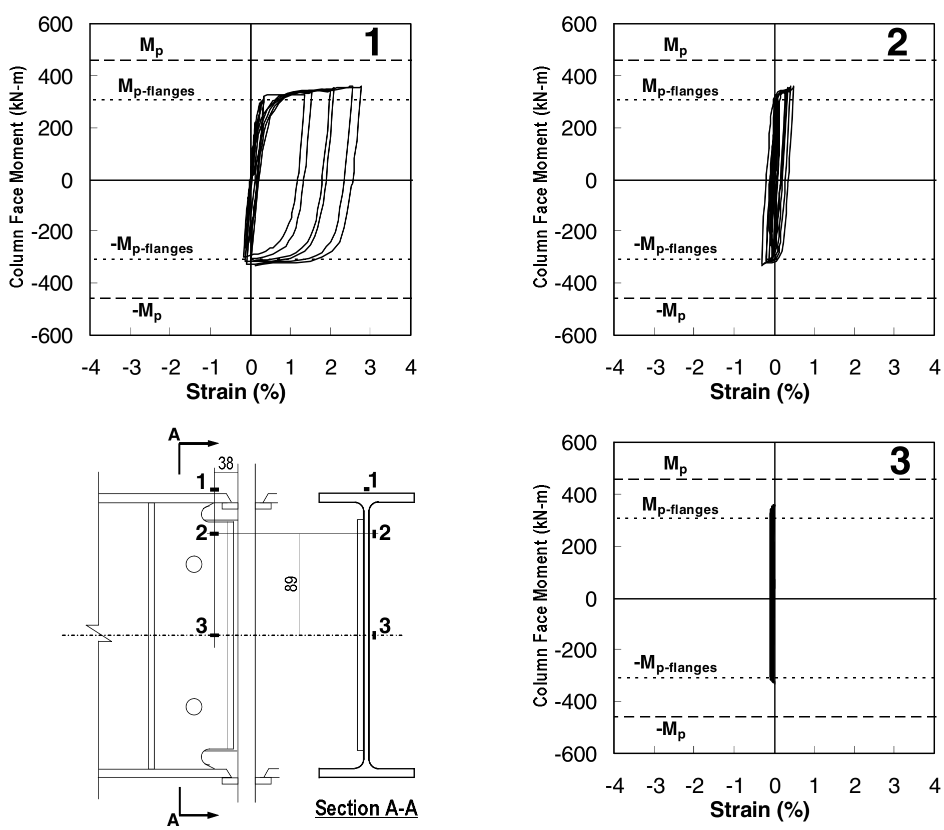


Fig. 11. Longitudinal strain measured at three locations in Specimen MWS

flanges took place once this reduced flexural plastic capacity was exceeded. The reduced flexural capacity was exceeded in all S-link specimens over a length of the link near the column end approximately corresponding to the region of the flanges where the whitewash flaked off. In Specimen FFS, the shear tab added to the flexural capacity of the link near the column face, and thereby precluded yielding in the link flanges near the groove weld.

Specimens PNS and MWS failed prematurely due to fracture of the link flange near the groove weld. The significant bending stress near the groove welds, as discussed above, likely contributed to this fracture. The premature failure of Specimen FFS was caused by the unique configuration of the FF-connection. Although Specimen NAS failed by fracture of the link web away from the link ends, crack openings were noted along the toe of the link flange groove welds during the test. Therefore, with the exception of the FF-connection, failure of connections of an S-link to a column was controlled by fracture of the link flange. The inelastic link rotation capacity of Specimens PNS, MWS, FFS, and NAS were 51-, 63-, 75-, and 88-percent, respectively, of the required 0.08 rad. The link rotation of Specimen NAS was limited by the link itself, and not by the link-to-column connection.

Specimens PNM, MWM, FFM, and NAM failed due to fracture of the link flange near the groove weld. All four specimens failed to meet the link rotation requirement. Specimens PNM and MWM achieved 40-percent of the required inelastic link rotation of 0.02 rad, while Specimens FFM and NAM achieved 80-percent of 0.02 rad. Engelhardt and Popov [2] observed that moment links typically exhibit severe local flange buckling and lateral-torsional buckling prior to fracture at the link ends. However, such behavior was not observed in the four M-link specimens. Therefore, it appears that the four M-link specimens failed well before the link developed its inherent inelastic deformation capacity. As noted by Engelhardt and Popov, connections of a moment link to a column are subjected to very severe inelastic bending strains, and are quite susceptible to fracture at the link flange groove welds.

Specimens PNI, MWI, FFI, and NAI failed due to fracture of the link flange near the groove weld, similar to the M-link specimens. Specimens PNI and MWI both achieved 42-percent of the required inelastic link rotation of 0.043 rad, while Specimens FFI and NAI achieved 108- and 63-percent, respectively, of 0.043 rad. Whereas Specimen FFI displayed a significant improvement in link rotation over Specimens PNI and MWI, Specimen NAI achieved a much smaller improvement. However, Specimen FFI exceeded the link rotation requirement with only a small margin, since it failed immediately after completing the first loading cycle with inelastic link rotation beyond 0.043 rad.

The results from the eight specimens with I-links or M-links agree with previous observations by Engelhardt and Popov [2] that connections of a long link ($e > 1.6M_p/V_p$) to a column are dominated by premature fracture of the link flange. Fracture of the link flange appeared to be less dominant for S-links. Nonetheless, the current tests suggest that premature failure is a significant concern not only for connections of a long link to a column, but also for connections with a short shear link, such as a link of $e = 1.1M_p/V_p$ used in this test program.

While the FF- and NA-connections generally performed better than the PN- and MW-connections, the comparison between the FF- and NA-connections provides somewhat mixed results. In terms of inelastic link rotation, Specimen FFS performed 14-percent better than Specimen NAS; Specimen FFI performed 60-percent better than Specimen NAI; Specimens FFM and NAM performed similarly. Therefore, the optimum configuration for link-to-column connections may differ depending on the link length.

CONCLUDING REMARKS

This paper provided an overview of an experimental program conducted at the University of Texas at Austin to investigate the cyclic loading performance of EBF link-to-column connections. Link-column specimens with four different connection types and three different link lengths, ranging from a short shear yielding link to a long flexure yielding link were tested. The welded flange-welded web connections were constructed using realistic detailing and welding according to the US practices.

The specimens representing pre-Northridge practices in detailing and construction for link-to-column connections developed only half of the inelastic link rotation required in the *2002 AISC Seismic Provisions*. The specimens featuring the use of weld filler metal with a specified notch toughness requirement and modifications in welding details provided marginal improvement over the pre-Northridge type specimens. The free flange and the no weld access hole type connections were effective in preventing or delaying the occurrence of link flange fracture compared to the two more conventional connections, and thereby sustained greater link rotation. However, only one specimen sustained a link rotation exceeding the required level, and this specimen failed immediately after completing the first loading cycle at the required amplitude.

The performance of the link-to-column connection depended strongly on the link length, with the inelastic link rotation capacity decreasing significantly with increase in the link length. The effect of the link length was reflected in the substantial difference in link shear force and column face moment. The maximum link shear force ranged between 0.7 and $1.4V_p$ depending on the link length, while the maximum column face moment ranged between 0.8 and $1.2M_p$.

The majority of link-column specimens failed by fracture of the link flanges near the groove weld. This fracture typically developed rapidly, causing abrupt and severe degradation in the strength of the specimen. It appeared that the excellent buckling control provided by the link stiffeners left fracture at the link-to-column connection as the dominating failure mode of the specimens. The test results suggest that premature failure of the link flange is a concern not only for connections of a long link ($e > 1.6M_p/V_p$) to a column, as noted previously, but also for connections with short shear links.

The test results reported herein suggest that link-to-column connections in EBFs are prone to premature and abrupt failure due to fracture of the link flanges. Further research is needed to identify connection details that can reliably develop the required inelastic deformation capacity of links. Until further research is available, it is suggested that EBF arrangements with links attached to columns should be avoided.

Further testing is continuing by the authors to develop safe and economical link-to-column connections for EBFs. These subsequent tests are being conducted using a revised loading protocol for testing link-to-column connections, recently developed and proposed by Richards and Uang [13]. Alongside the experimental investigation presented in this paper, an analytical study has also been in progress. Finite element models of link-column specimens were developed and analyzed to further understand the observations made from the tests. The finite element models are also being used to aid in the development of improved connection details. The results of this on-going research on link-to-column connections will be reported in subsequent publications.

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