LOCAL FLANGE BENDING AND LOCAL WEB YIELDING LIMIT STATES IN STEEL MOMENT-RESISTING CONNECTIONS

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ABSTRACT

Nine pull-plate experiments were conducted to examine the effect of column stiffening on the limit states of local flange bending and local web yielding. The results show that AISC provisions for these limit states are reasonable and slightly conservative. Weld fractures did not occur despite the fact that some of the specimens were significantly under-stiffened. The use of half-thickness continuity plates fillet welded to the column web and flanges was shown to be sufficient in comparison to full-thickness continuity plates with CJP welds.

INTRODUCTION

Beam-to-column flange welds fractured in some girder-to-column connections in steel moment frames during the 1994 Northridge earthquake. These welds fractured primarily because of low fracture toughness of weld metal combined with a backing bar forming a notch at the weld root and weld root defects, Fisher, et al. (1). Subsequently, there has been a tendency to be overly conservative in the design and detailing of these connections. For example, there has been a tendency to over-specify column stiffeners even though there is no definitive evidence that inadequate column stiffeners contributed to the Northridge weld fractures. Continuity plates and web doubler plates have been specified when they are unnecessary and, when they are necessary, thicker stiffeners have often been specified than would be required and complete joint penetration (CJP) welds of the continuity plates have been specified when more economical fillet welds may have sufficed.

The tendency to be overly conservative with column stiffeners is understandable since they do have a significant effect on the stress and strain distribution in the connection and on connection performance. For example, Roeder (3) observed that girder-to-column joints with modest continuity plates and/or doubler plates performed better in cyclic loading tests than joints without such reinforcement. Also, it has been observed from finite element analyses of these joints that there is a decrease in stress concentration at the middle of the girder flange-to-column flange weld when continuity plates are used, e.g., Roeder (3), El-Tawil et al. (4).

The limit states of local web yielding (LWY), local flange bending (LFB), local web crippling (LWC), and panel zone (PZ) shear yielding are mitigated by the use of column stiffening. The design criteria for these limit states are provided in Section K1 of Chapter K of the AISC LRFD Specification for Structural Steel Buildings (5). There were additional, more stringent provisions in the requirements for Special Moment Frames

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(SMF) in the 1992 AISC Seismic Provisions for Structural Steel Buildings. However, the 1997 AISC Seismic Provisions (6) removed all design procedures related to continuity plates, requiring instead that they be proportioned to match those provided in the tests used to qualify the connection. As part of the SAC Joint Venture, interim guidelines and an advisory were published, FEMA (7), (8), that pertained to these column reinforcements in seismic zones. For example, the guidelines call for continuity plates at least as thick as the beam flange that must be joined to the column flange in a way that fully develops the strength of the continuity plate, i.e., this encourages the use of CJP welds. However, the SAC 100% draft document, FEMA (9), has reestablished design equations to determine whether continuity plates are required and, if so, what thickness they need to be. The present provisions for local web yielding, local flange bending, and panel zone shear are largely based on limit-load analyses that were developed in conjunction with girder-to-column joint subassembly test data, Graham et al. (10), Krawinkler et al. (11), Bertero et al. (12). The criteria for panel zone shear strength have been refined by numerous tests and analyses of girder-to-column subassemblies for seismic moment frames, e.g., Krawinkler (13) and Popov et al. (14).

Recent research has revealed that excessively thick continuity plates are unnecessary. El-Tawil et al. (4) performed parametric finite element analyses of girder-to-column joints. They found that continuity plates are increasingly effective as the thickness increases to about 60% of the girder flange. However, continuity plates more than 60% of the girder flange thickness brought diminishing returns.

Furthermore, over-specification of column reinforcement may actually be detrimental to the performance of connections. As continuity plates are made thicker and attached with highly restrained CJP welds, they are sometimes causing cracking during fabrication. CJP welds have also been specified for the attachment of continuity plates to the web, where fillet welds have traditionally been adequate. Yee et al. (15) performed finite element analyses comparing fillet welded and CJP welded continuity plates. Based on principal stresses extracted at the weld terminations, it was concluded that fillet welded continuity plates may be less susceptible to cracking during fabrication than if CJP welds are used.

The research described in this paper is part of an ongoing project sponsored by AISC to reassess the design provisions for column stiffeners for non-seismic and seismic conditions, and to investigate new alternative column stiffener details. The project includes three components: monotonically-loaded pull-plate experiments to investigate the need for and behavior of transverse stiffeners, cyclically-loaded cruciform girder-to-column joint experiments to investigate panel zone behavior and local flange bending as well as innovative doubler plate and continuity plate details, and parametric finite element analyses to corroborate the experiments and assess the performance of various transverse stiffener and doubler plate details.

The test matrices for this project were designed by examining all practical combinations of girder and column sizes to identify which girder-to-column joints satisfy the limit states of LFB, LWY, web crippling, and panel zone yielding, as well as which combinations satisfy the strong-column/weak-beam (SCWB) provisions according to AISC (5,6). A parametric study was then conducted using three-dimensional nonlinear continuum finite element analysis (FEM) to model the behavior of these connections and the performance of various transverse stiffener and doubler plate details. These analyses permitted the behavior of these connections to be characterized in detail. Criteria were established to identify the limit states of LWY, LFB and panel zone yielding flange for stiffened and unstiffened specimens. The results of the parametric study showed that web crippling did not control the need for column stiffening in any of the practical combinations of girder and column sizes, and therefore was not further investigated in the research program. The results of the finite element analyses and the comprehensive investigation of the limit states for all beam and column combinations were then coupled with the results of past experiments to establish the test matrices for the present project. Nine laboratory experiments were conducted with pull-plates (simulating a girder flange) attached to column sections for the study of localized flange bending and web yielding, investigating both common and new alternatives for detailing. These monotonic tests focus on the non-seismic behavior, with some consideration given to seismic design as well. Additional experiments are then being conducted on five full-scale cyclic girder-to-column joint subassemblies. These tests will focus on seismic behavior, although they will provide useful information for non-seismic design as well.

This paper outlines the results of the nine pull-plate experiments and corresponding finite-element analyses. The complete literature review, including extensive background information on the various limit states investigated, description of the details of the nine pull-plate and five cruciform experiments and testing procedure, and further detail of the background of this research and justification for specific specimen selection were presented in Dexter et al. (16).

TESTING PROCEDURE

Figure 1 shows the basic schematic drawings of the pull-plate specimens. The pull-plate specimens consisted of three-foot-long sections of A992 columns between pull plates that represent the flanges of the girders in the actual connections.

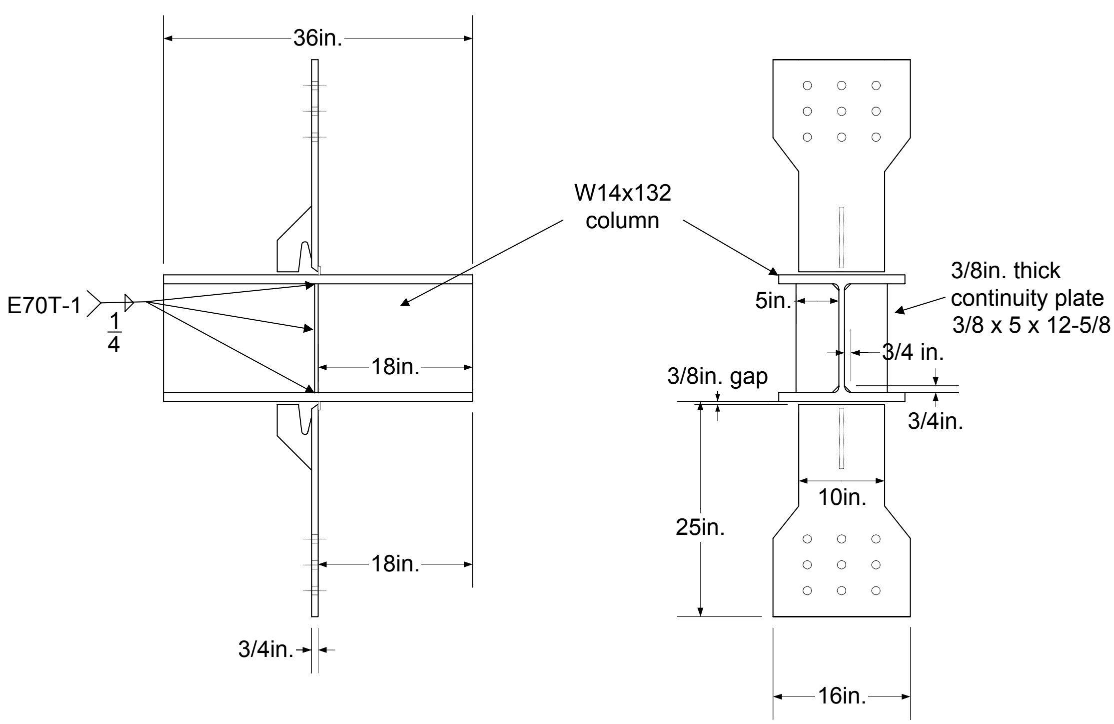

Figure 1 Typical pull-plate specimen with a half-thickness continuity plate, fillet welded to the column flange and web

The The CJP welds joining the pull plates to the column sections were made using the self-shielded FCAW process and E70T-6 filler metal with a minimum Charpy V-Notch (CVN) energy of 20 ft-lb at 0° F. The E70T-6 wire had a diameter of 0.068 in. Figure 2 shows the detail of the girder tension flange-to-column flange connection, including the weld type and access hole dimensions. Continuity plates and web doubler plates were fillet welded using the 100% carbon dioxide gas-shielded FCAW process and E70T-1 filler metal with a 0.063 in. diameter. In one case, CJP welds were used to join the continuity plate to the column flanges and in another case CJP welds were used to join the web doubler plate to the column flanges. These CJP welds were also made with the gas-shielded FCAW process and E70T-1 filler metal.

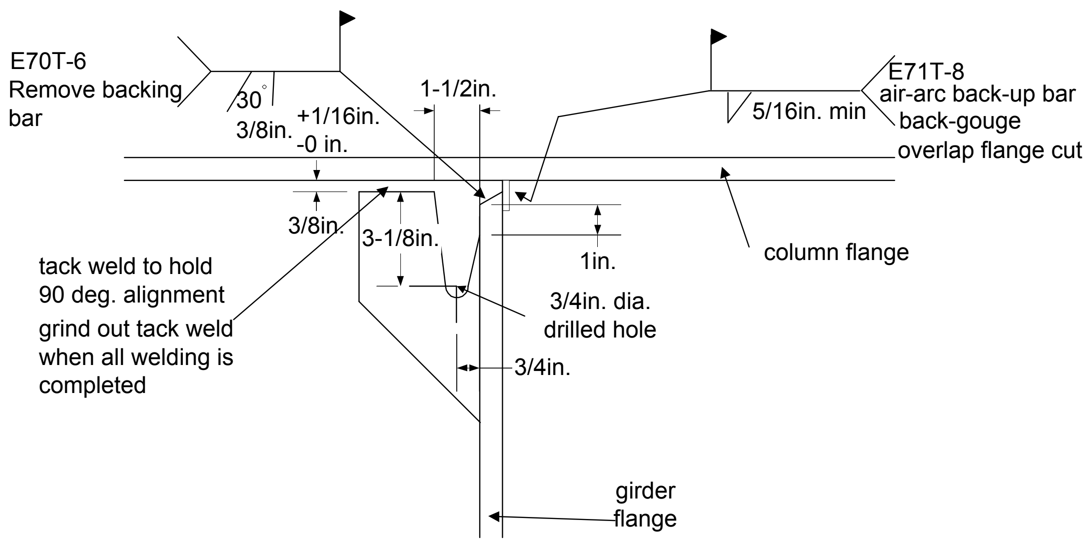

Figure 2 Girder to column weld detail

Table 1 is a comparison of the coupon test results and the mill reports. The reported coupon yield strength was defined by the 0.2% offset. All values given in Table 1 are averaged values and are in units of ksi.

The pull-plates for all specimens were based on the size of a girder flange from a W27x94 section. The variations of the specimens were the type and size of stiffeners and the column size. Three different column sections were tested, W14x132, W14x145, and W14x159. The stiffener details varied between half thickness and full thickness (relative to the pull-plate thickness) continuity plates and a doubler plate box detail. The nine specimens could be grouped into three categories - specimens used to evaluate local web yielding, specimens focused on local flange bending, and specimens aimed at investigated the effects of stiffening details on the connections.

Table 1 Material properties

	W14x132	W14x145	W14x159	Pull-plate	HCP*	FCP*	DP Box*	DP*
Coupon Yield	49.2–54.4	58.2–59.4	51.1– 2.2	48.2	50.0	46.0	46.5	56.2
Mill Yield	53.0	57.0	53.5	51.2	61.3	61.3	61.3	57.0
Coupon Tensile	69.4–70.3	74.1–75.1	71.5-71.8	72.5	72.2	72.5	72.5	73.8
Mill Tensile	70.5	73.5	72	72.1	80.4	80.4	80.4	71.0

HCP = half-thickness continuity plate, FCP = full-thickness continuity plates, DP Box = doubler plate box detail, DP = doubler plate

The nine pull-plate specimens were as follows:

1. Specimen 1-LFB: W14x132 without continuity plates, with doubler plates, examined LFB
2. Specimen 2-LFB: W14x145 without continuity plates, with doubler plates, examined LFB
3. Specimen 1-LWY: W14x132 without any stiffeners, examined LWY and LFB
4. Specimen 2-LWY: W14x145 without any stiffeners, examined LWY and LFB
5. Specimen 3-UNST: W14x159, without any stiffeners, examined LWY and LFB
6. Specimen 1-FCP: W14x132, with full-thickness continuity plates and CJP welds
7. Specimen 1-HCP: W14x132, with half-thickness continuity plates and fillet welds
8. Specimen 1B-HCP: repeat of 1-HCP to verify results
9. Specimen 1-DP: W14x132, with doubler plate box detail

Specimens 1 and 2 had beveled doubler plates fillet welded to the column flange to avoid welding in the column k-line. The doubler plates stiffened the web of the two specimens in order to isolate local flange bending as the governing limit state. Specimens 3 through 5 were unstiffened connections that looked at the interaction between local web yielding and local flange bending.

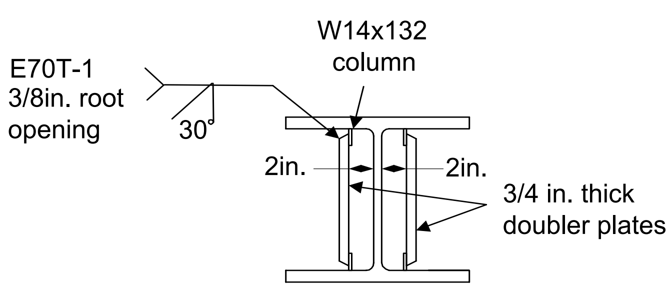

Figure 3 Box detail with doubler plates welded to column flange away from web with CJP welds

Specimens 6 through 8 tested connections either with full-thickness continuity plates and CJP welds, replicating details often seen in present practice, or half-thickness continuity plates with fillet welds. Specimen 9 included no continuity plate, but rather two doubler plates placed out towards the column flange tips, as shown in Figure 3. These plates thus act both as continuity and doubler plates. This detail, first investigated by Bertero et al. (12), is included in AISC (6) and provides an economical alternative to connections that require two-sided doubler plates plus four continuity plates.

Testing of the pull-plate specimens followed the SAC protocol, SAC (17), where it was applicable. Since, the SAC protocol does not specify a strain rate for monotonic tensile tests, a high strain rate of 0.004 sec^{-1} was used, which approximates the strain rate from seismic loading at about a 2 second period. The high strain rate increases the yield strength of the materials and increases the probability for brittle fracture, thereby testing the specimens under the most severe conditions. There were three basic instrumentation plans, one for each of the three categories of specimens. All nine specimens had high-elongation strain gages on the pull-plates and LVDTs that measured the overall specimen elongation and the separation of the column flange tips. The data acquisition system collected 56 channels of data at 100 Hz.

RESULTS AND ANALYSIS

Before testing began, connection failure criteria were developed for the LWY and LFB limit states. The primary indicator of failure was whether the weld fractured prematurely. Brittle fracture was potentially still a possibility, because the fracture toughness of the E70T-6 weld metal is only marginally better than the E70T-4 weld metal that was used in the pre-Northridge connections, FEMA (18). If brittle fracture occurred in some cases but not in others, the influence of column stiffener details on the occurrence of brittle fracture could be investigated clearly. However, there may be other undesirable behavior besides premature fracture, such as excessive deformation. In these experiments, none of the welds fractured prior to the pull plate fracturing, so secondary failure criteria were established based on excessive deformation to identify problematic limit states.

The criteria were based on FEM analyses, AISC provisions for LWY and LFB, and previous research, e.g. Sherbourne and Jensen (19), Graham et al. (10). For each specimen, the column section was examined for failure at non-seismic and seismic girder demand load levels, R_u, calculated as: $R_u = F_{yg} A_{gf}$ (non-seismic) and $R_u = 1.1 R_y F_{yg} A_{gf}$ (seismic) where R_y = 1.1 for grade 50 or 65 rolled shapes (see AISC (5,6) for variable definitions). The calculation of the seismic girder demand takes into account strain hardening of the girder and includes an overstrength factor, R_y, of the shapes. Using the yield strength and non-seismic and seismic girder flange (pull-plate) dimensions, the girder flange demands were approximately 385 and 450 kips, respectively.

For each limit state, a two-part failure criterion was developed. The connection was classified as failing by LWY if at 450 kips the strain in the column k-line directly under the pull-plate was greater than 3.0%, or the strain in the column k-line was greater than the yield strain for the entire $5k+N$ area. The connection was defined as failing by LFB if at 450 kips the column flange tip separation was greater than ¼ in. The continuity plates were characterized as failed if the entire full-width region of the continuity plates was above the yield strain.

Justification for the LWY failure criterion can be seen in Figure 4, which plots the FEM results for the three unstiffened column sections. Using similar failure criterion of Graham et al. (10), which based LWY failure on yielding of the *5k+N* region of the column k-line, the W14x132 specimen would fail by LWY. Figure 4 shows that the strain in the W14x132 (1-LWY) k-line is greater than yield for the entire *5k+N* region, while the W14x145 (2-LWY) and W14x159 (3-UNST) are not. Therefore, if it is assumed that the W14x132 (1-LWY) fails and the W14x145 (2-LWY) does not, then another failure guideline could be a strain greater than 3% directly below the pull-plate.

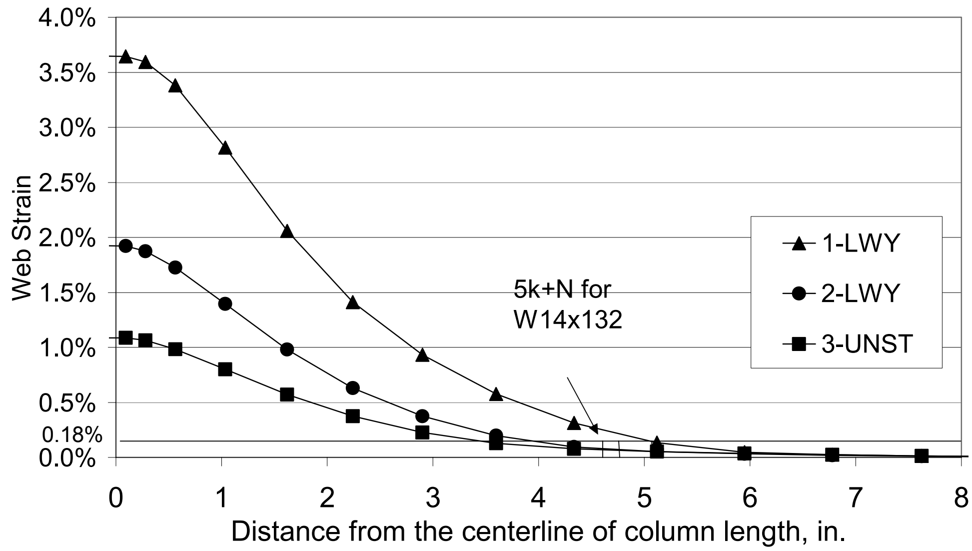

Figure 4 FEM strain distribution along the column k-line at 450 kips

Figure 5 shows the experimental strain distribution in the k-line of the column web for all seven specimens that were gaged to evaluate LWY. As shown in Figure 5, none of the specimens had strain levels exceeding 3% directly under the pull-plate and only the unstiffened W14x145 specimen (2-LWY) had strain values greater than yield for the entire 5k+N region. Initially these results seemed implausible, since a W14x145 nominally has a thicker web than a W14x132 section. However, measurements showed that the specific W14x145 section used in the test actually had a thinner web than the specific W14x132 section, which justifies the difference in the strain distribution between these specimens. There is no tolerance on web thickness in ASTM A6; the tolerance is only on the weight per foot, ASTM (20). The strain distribution also shows a much steeper gradient for the W14x132 (1-LWY) than the other two unstiffened sections. This gradient is also likely due to its thinner column flange. The thicker column flanges of the W14x145 and W14x159 act to distribute the load more evenly into the column web.

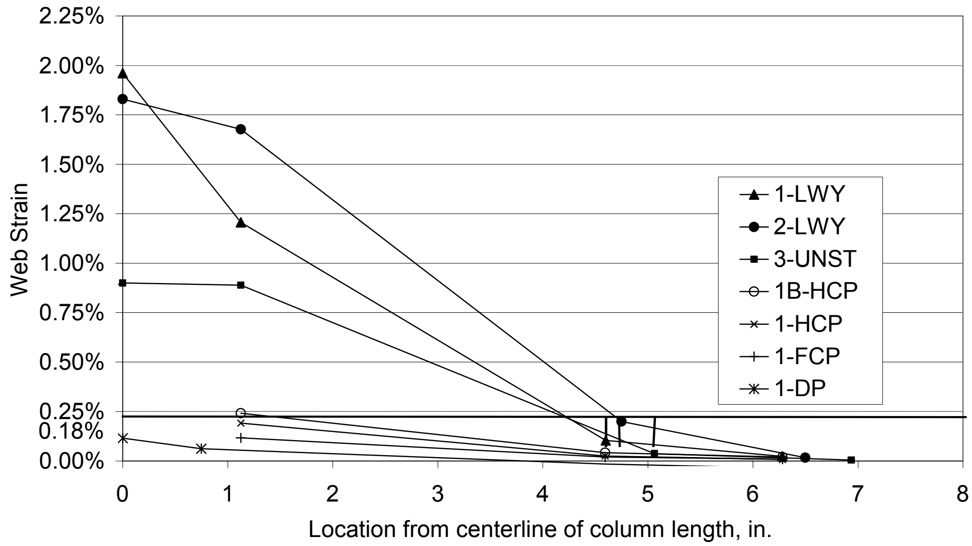

Figure 5 Strain distribution in the column k-line at 450 kips

The LFB failure criterion was based on the permissible variations in cross section sizes, ASTM (20). The provisions allow the flanges of a W section to be ¼ in. out of square. Presumably, this amount of flange out-of-flatness is tolerable and still has sufficient resistance to local flange buckling. Therefore, it was assumed that it would also be acceptable to have this much out-of-flatness caused by deforming of the girder flanges. The probability of an initially out-of-square flange combining with additional deformation due to the girder was deemed to be insignificant.

Figure 6 shows the separation of the flanges near the tips of the flanges along the column length for all nine specimens. The W14x132 unstiffened and the W14x132 with doubler plates on the web (1-LFB) both failed this LFB criterion. By comparing the specimens without continuity plates but with web-doubler plates (1-LFB and 2-LFB) to those with no stiffeners at all (1-LWY and 2-LWY), it can be seen that a significant portion of the flange separation is due to web deformation. In the case of the W14x145 (2-LWY and 2-LFB), which has a stiffer flange and, as it turns out, a thinner web, half of the flange separation is due to web deformation.

The results of the stiffened specimens (1-HCP, 1B-HCP, 1-FCP, and 1-DP) showed that, at least for monotonically loaded connections, a half-thickness continuity plate was adequate to avoid web yielding and flange bending. Figures 5 and 6 show a significant difference between the unstiffened and stiffened specimens and that the half-thickness continuity plates (1-HCP and 1B-HCP) are well below the LWY and LFB failure criterion.

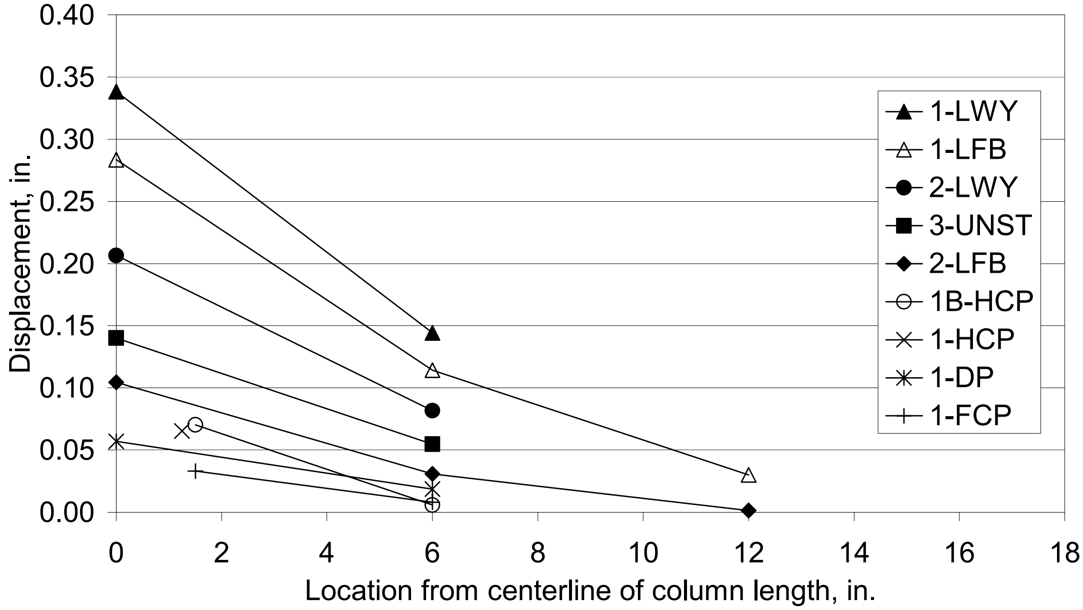

Figure 6 Column flange separation at 450 kips

The failure criterion for the continuity plates was complete yielding across the full-width section of the plates at 450 kips. The full width portion of the continuity plates, as shown in Figure 7, was defined as the area just outside of the ¾ in. clips. Figure 8 shows a comparison of the results of the strain distribution in the continuity plates of the 1-HCP and 1-FCP specimens. Neither of the specimens fully yielded across the width of the continuity plates, and therefore both were still capable of resisting load and had not failed. The half-thickness continuity plates fillet welds also did not fracture. The CJP welds of the full-thickness continuity plates did not cause any problems during fabrication or testing.

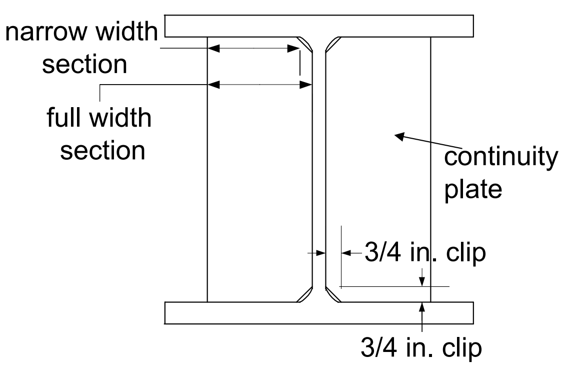

Figure 7 Narrow and full-width sections of continuity plates

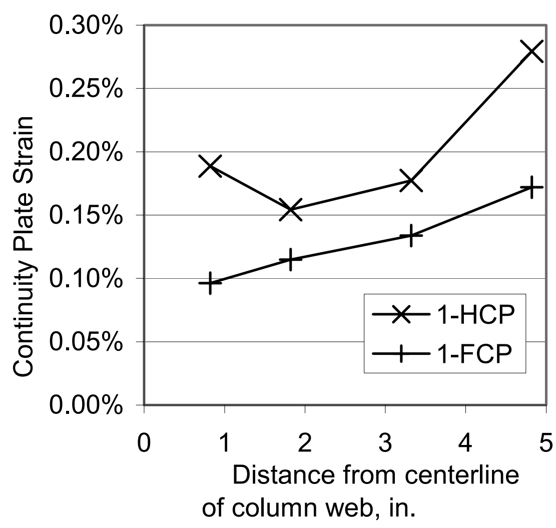

Figure 8 Continuity plate distribution along full-width of plate at 450 kips

CONCLUSIONS

This paper has summarized the results of nine pull-plate tests and corresponding finite-element analyses studying column-stiffening details. The preliminary conclusions from these tests are focused on monotonic loading applications and will be synthesized with future cyclic loading experiments.

- The AISC provisions for LWY and LFB are reasonable and slightly conservative in calculating the need for column stiffening.
- None of the E70T-6 CJP welds fractured despite plastic deformation, even when the flange tip separation was over ¼ in, indicating that column stiffener details may have little influence on the potential for brittle weld fracture provided the weld is specified with minimum CVN requirements and backing bars are removed.
- The use of half-thickness continuity plates fillet welded to both the column web and flanges is sufficient in comparison to the traditional full-thickness continuity plates with CJP welds.
- The new stiffener details, i.e. the box detail and beveled doubler plates fillet welded to the column flanges, performed satisfactorily and provided sufficient stiffness to avoid LWY and LFB.

The research project continues as five cruciform tests are underway. Results of the cyclically tested cruciform specimens will be combined with the pull-plate test results to evaluate the local web yielding and local flange bending criteria and new stiffener details in seismic applications.

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