Evaluation of the Design Requirements for Column Stiffeners and Doublers and the Variation in Properties of A992 Shapes

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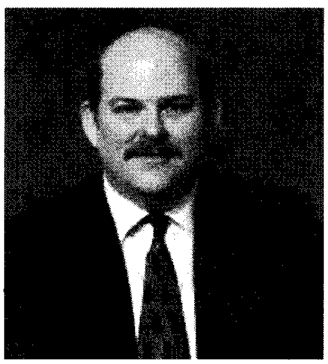

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Dr. Dexter received his B.S. in Civil Engineering, with Highest Honors, at the University of Texas at Austin in 1981. He then went to work in the Ocean Engineering Division at Southwest Research Institute (SwRI) in San Antonio. In 1983, Dr. Dexter joined the Materials and Mechanics Division and began to work in the area of fracture mechanics. In 1986, he completed his Master's thesis, "Underwater Wet Welds: Mechanical Properties and Design Guidelines" under the direction of Prof. Karl H. Frank.

In 1991, Dr. Dexter took a position as a Senior Research Engineer at Lehigh University. He completed his Ph.D. from the University of Texas in 1992 on "Investigation of Criteria for Ductile Fracture Under Fully-Plastic Conditions" under the guidance of Prof. Frank. Among the research Dr. Dexter performed at Lehigh was "Through-thickness Testing of Structural Sections" for the SAC Steel Project. He also completed a project entitled "Evaluation of Alternative Methods to Design for Fire Resistance" for the National Institute of Standards and Technology, which involved cooperation with the University of Liege (Belgium).

In 1997, he took his present position at the University of Minnesota. He is presently conducting research related to fatigue, bridge behavior, and repair welding. He is also conducting research related to the variation of material properties and behavior of beam-to-column connections in moment frames for AISC. He is a member of American Society of Civil Engineers, American Welding Society, the Research Council on Structural Connections, and the American Railways Engineers and Maintenance of Way Association. He is a registered Professional Engineer in several states.

Summary

This paper reports ongoing research on the behavior of welded tee-joints, the behavior of welded moment-resisting frame connections, and the properties of A992 shapes. In the connection research, the effects are examined of various column-stiffening details (such as continuity plates and web doubler plates) on the limit states of weld fracture, local web yielding, local flange bending, and panel zone shear. New alternatives for detailing column stiffeners in moment-resisting frame connections are presented. The connection research has three components: finite element analyses, nine pull-plate experiments, and five cyclically loaded cruciform girder-to-column connection experiments. Unstiffened column sections are tested as well as column sections with a variety of column stiffening details. The results show that AISC provisions for column stiffening are reasonable and slightly conservative for both non-seismic and seismic design. Based on the pull-plate and finite element results, 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. With respect to research on A992 shapes, mill test reports were collected for all of 1998 wide-flange shape production sold in the United States. Histograms of the reported yield strength, ultimate strength, yield-to-tensile ratio, and Charpy toughness are shown for A992 steel.

EVALUATION OF THE DESIGN REQUIREMENTS FOR COLUMN STIFFENERS AND DOUBLERS AND THE VARIATION IN MATERIAL PROPERTIES

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INTRODUCTION

The stiffening details (continuity plates and web doubler plates) in welded moment-resisting-frame connections have a significant effect on the stress and strain distribution in the connection and consequently on connection performance. Potential performance limit states include brittle fracture of the welds, low-cycle fatigue at the access hole detail, at the toes of the beam flange welds, or at the web welds, excessive panel zone shear deformation (PZ); and excessive local deformation of the column from local web yielding (LWY) and local flange bending (LFB).

Roeder (1997) 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. Ricles (2000) has shown better resistance to low-cycle fatigue failure at the weld access hole with stiffer panel zones. 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 welds when continuity plates are used (Roeder, 1997; El-Tawil et al., 1998). A decrease in stress concentration and triaxiality of the stresses would be expected to decrease the potential for brittle fracture. Also, the limit states of local web yielding (LWY), local flange bending (LFB), and panel zone (PZ) shear yielding are mitigated by the use of column stiffening.

Brittle fracture of the girder-flange to-column flange welds occurred in some steel moment-frame connections during the 1994 Northridge earthquake and in subsequent laboratory tests (Fisher et al., 1997; FEMA, 2000b). There was no Charpy requirement for the weld metal for these girder flange welds and the E70T-4 weld metal that was commonly used had very low Charpy toughness (Fisher et al., 1997). Subsequently, there has been a tendency to be overly conservative in the design and detailing of these connections. In some situations, continuity plates and web doubler plates have been specified when they are unnecessary and, when they are necessary, thicker continuity plates and web doubler plates have been specified than would be required. The specified type of welds have often been complete joint penetration (CJP) welds to join the continuity plates or web doubler plates to the column, even though the use of more economical fillet welds may have sufficed.

Among possible connection details recommended by the SAC Joint Venture research program (FEMA, 2000b) for moment-resisting-frame connections is a modified form of the welded connection used prior to 1994. The modified welded connection is referred to as the welded unreinforced flange - welded web connection (WUF-W). Unlike the pre-1994 connection, the WUF-W connection has a Charpy requirement for the weld metal (20 ft-lbs at 0 degrees F and 40 ft-lbs at 70 degrees F), a complete joint penetration (CJP) welded web, a special access hole detail, and specific details for continuity plates and web doubler plates. Design equations are recommended for the sizing of continuity plates and web doubler plates, but CJP welds are required between the continuity plates and the web doubler plate and the column flanges.

The WUF-W connection has performed well in many full-scale beam-column connection tests (Ricles et al., 2000). However, there are still many questions about the column stiffening details.

- Modest stiffening has been shown to be beneficial in some cases, but it is not clear if more conservative stiffening provides any additional benefit or in fact may be detrimental to performance. For example, as continuity plates are made thicker and attached with highly restrained CJP welds, they are causing cracking during fabrication (AISC k-line Advisory, 1997).
- It is possible that the continuity plates helped reduce the potential for brittle fracture when low-toughness flange welds were used in certain connections. However, given a sufficient level of toughness, these connections may perform well without continuity plates.
- It is not clear that if a specific type of column stiffening is beneficial in reducing the potential for one limit state, e.g. brittle fracture, that it would also be beneficial for other limit states, e.g. low-cycle fatigue. These

failure modes are caused by different micromechanical phenomena - brittle fracture is governed by a critical stress criterion and low-cycle fatigue or ductile tearing is governed by a critical strain criterion (Dexter, 1992; Dexter and Griesbach, 1993; Dexter and Roy, 1993; Roy and Dexter, 1993).

- Over-specified continuity plates and doubler plates are making the fabrication more complex and expensive (Carter, 1999). There may be more efficient column stiffening details that are equally effective.

This paper includes background of the LWY, LFB, and PZ yielding limit states. Also included is a brief description of the finite element analysis models (FEM) that were used to further understand the behavior of the tested specimens and justification of corroboration with the experimental tests. The results of the pull-plate specimens are compared in regards to defined failure modes and yield mechanisms relating to the limit states of LWY and LFB. Finally, the results of the first cruciform test are summarized. Testing of the remaining cruciform specimens is ongoing.

This paper also presents results from a companion study of the properties of A992 shapes, including tensile properties and toughness. The variability in these material properties is discussed and the impact of this variation on the resistance factors in the LRFD Specification is also described.

BACKGROUND

The design criteria for the PZ, LWY, and LFB limit states are provided in Section K1 of Chapter K of the American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings (AISC, 1999b). There are additional more stringent provisions in the requirements for Special Moment Frames (SMF) in the AISC Seismic Provisions for Structural Steel Buildings (1992). However, the 1997 AISC Seismic Provisions (AISC, 1997) 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. The PZ yielding limit state is applicable to the shear forces delivered into the column web by the girder flanges in a moment connection, and determines the need for doubler plates. This limit state is also included in the AISC Seismic Provisions (1997).

As part of the SAC Joint Venture research program, guidelines and an advisory were published (FEMA, 1997a; FEMA, 1997b) that pertained to these column reinforcements in seismic zones. These interim guidelines called for continuity plates at least as thick as the girder flange that must be joined to the column flange in a way that fully develops the strength of the continuity plate, i.e., this encouraged the use of complete joint penetration (CJP) welds. However, the final SAC documents (FEMA, 2000b) have reestablished design equations for the thickness of continuity plates and doubler plates in seismic zones.

The present provisions for LWY, LFB, and PZ are largely based on limit load or buckling analyses that were developed in conjunction with girder-to-column joint subassembly test data (Parkes, 1952; Wood, 1955; Sherbourne and Jensen, 1957; Graham et al., 1960; Krawinkler et al., 1971; Bertero et al., 1973). 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 (1978) and Popov et al. (1986).

Recent research has revealed that excessively thick and CJP-welded continuity plates are unnecessary. El-Tawil et al. (1998) 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. Yee et al. (1998) 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 Recommended Seismic Design Criteria for New Moment-Resisting Steel Frame Structures (FEMA, 2000b) uses the LFB equation and a seismic girder demand to calculate the need for continuity plates. The guidelines state that unless proven with tested connections, continuity plates with CJP welds to the column flanges (the web joint may be fillet welded) are required if the thickness of the column flange is less than either of the two following equations:

$$t_{cf} < 0.4\sqrt{1.8 b_{gf} t_{gf}\left(\frac{F_{yg}}{F_{yc}}\right)} \quad (1)$$

$$t_{cf} < \frac{b_{gf}}{6} \quad (2)$$

There is some consensus that continuity plates may be fillet-welded and may not always be required in non-seismic connections. However, there is strong consensus that continuity plates are required for connections in seismic zones, although there are differing opinions on the required width and thickness of the plate and on the type of weld that should be used to connect the stiffener to the column flange (Dexter et al., 1999).

FINITE-ELEMENT ANALYSES

As a part of this research, finite element analyses were conducted to study the connection behavior (Ye et al., 2000). The analyses include models of all the specimens tested in this research, as well as parametric studies. The model geometry, material properties, boundary conditions, and loading history reflect the experimental configurations.

The analysis program used was ABAQUS, version 5.8-1 (HKS, 1994). The analyses were conducted using a combination of 3D solid finite elements and beam elements to model the behavior of these connections, including the performance of the continuity plates, doubler plates, and weld details. For the full-scale beam-to-column joint assemblies, continuum 3D solid elements were used to model the connection region and a short length of the beam outside the connection region. The solid elements were 8-node brick elements (element C3D8I). At an appropriate length away from the joint region, where the longitudinal stress and strain gradients through the girder depth were linear and no significant local buckling was expected, a 3-node quadratic beam element (element B32) was used for the remaining girder and column lengths. This quadratic beam element used Timoshenko beam theory and included transverse shear deformation. Multi-point constraints (MPC) were used to enforce compatibility between the different types of elements. Mesh refinement was evaluated in all models to achieve converged results. Figure 1 shows the connection region of a typical mesh simulating one of the cruciform specimens.

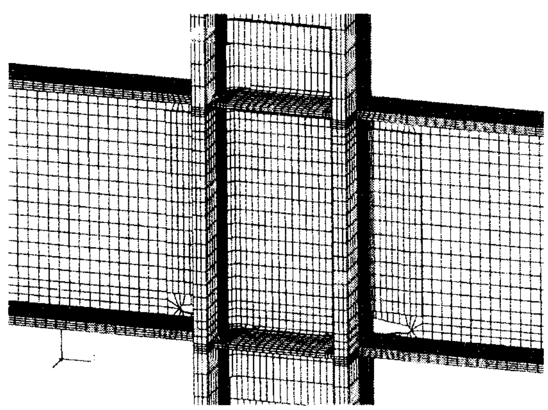


Figure 1 Typical finite element model for simulating cruciform connection experiments

For the analyses corroborating the experiments, the stress-strain properties for the analyses were obtained from the coupon tests of the specimens, as shown later. The analyses included geometric nonlinearity as well as material nonlinearity. Residual stresses, thermal effects, and fracture were not modeled.

Results included detailed modeling of all of the pull-plate specimens and preliminary modeling of all of the cruciform specimens. These analyses, along with observations from past experimental results, were used to identify specific criteria to be used to determine whether failure was occurring in the specimens due to local flange bending, local web yielding, or excessive panel zone yielding.

Figure 2 shows typical results from the pull-plate analyses. The finite element mesh, which is quarter symmetric, reveals the interaction between local flange bending and local web yielding in an unstiffened W14x132 column. Similar analyses were run on unstiffened W14x120, W14x145, and W14x159 column sections.

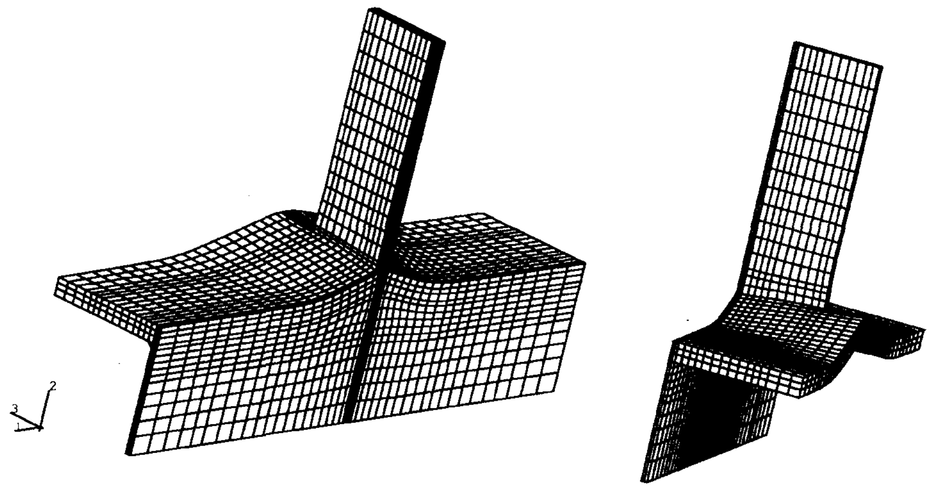


Figure 2 Deformed shape of pull-plate specimen showing local flange bending and local web yielding in a W14x132 (A992) column

PULL-PLATE TESTING

Figure 3 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.

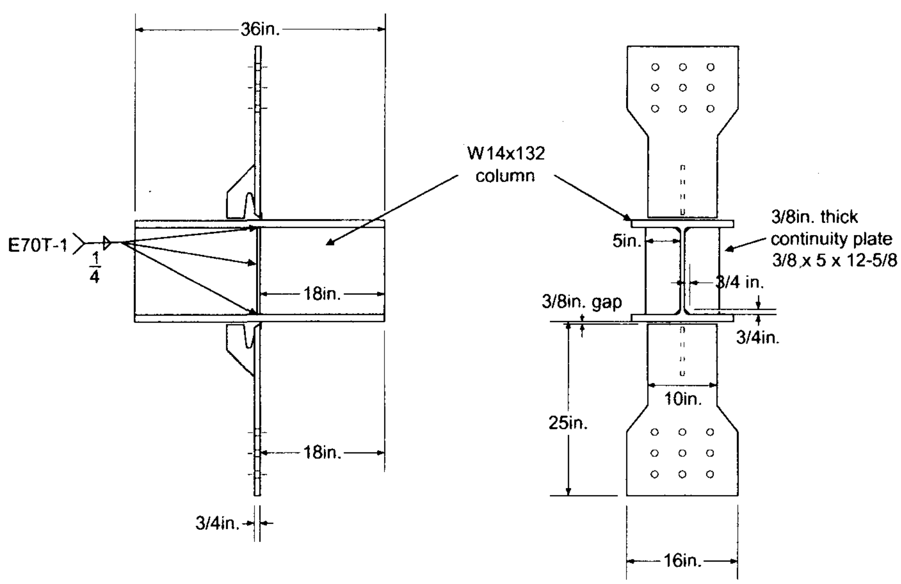


Figure 3 Typical pull-plate specimen with a half-thickness fillet welded continuity plate

The CJP welds joining the pull plates to the column sections were made using the self-shielded FCAW process and E70T-6 filler metal. The E70T-6 wire had a diameter of 0.068 in. The minimum toughness requirement for notch-tough weld metal is 40 ft-lbs at 70°F and 20 ft-lbs at 0°F (FEMA, 2000c). The weld metal that was used in this research program had a measured ultimate strength of 77 ksi and toughness measurements of 63.7 ft-lbs at 70°F and 19.0 ft-lbs at 0°F. Thus, the weld metal met the minimum ultimate strength requirement but did not quite meet the lower shelf minimum toughness requirement.

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 0.0625 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.

The design of the pull-plates for all specimens was 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 LFB, specimens focused on the interaction of LWY and LFB, and specimens aimed at investigating the effects of stiffening details on the connections. The nine pull-plate specimens were as follows:

1. Specimen 1-LFB: W14x 132 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. 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 4. These plates thus act both as continuity and doubler plates. This detail, first investigated by Bertero et al. (1973), is included in AISC (1997) and provides an economical alternative to connections that require two-sided doubler plates plus four continuity plates.

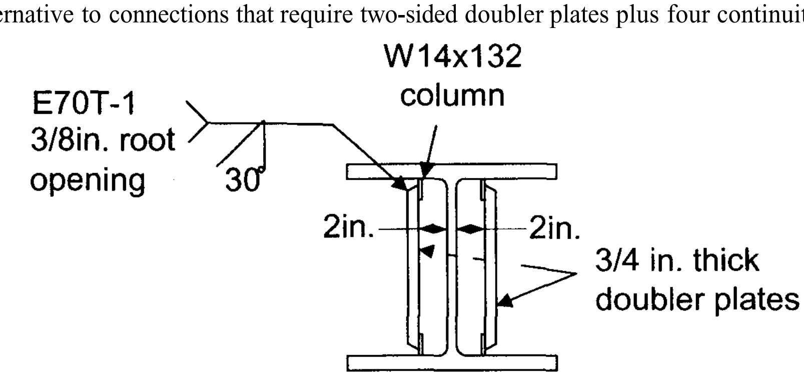


Figure 4 Box detail with doubler plates welded to column flange away from web with CJP welds

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 of vibration. 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.

These specimens were primarily intended to examine the LWY and LFB limit states for non-seismic construction. It is recognized that this specimen is not a good simulation of the actual conditions in the moment-resisting-frame connection since there is tension on both sides of the specimen and no shear. However, the condition in the pull-plate tests is expected to represent a worst case for the evaluation of LFB and LWY; therefore the specimen is appropriate for that purpose. There is also no shear or bending in the pull plate as there is in the girder flange in the connection; however this is not expected to influence LWY and LFB. Nevertheless, these effects could increase the potential for brittle fracture. Therefore, the fact that fracture did not occur in these experiments does not necessarily mean that it would not occur in the connection. That is why it is also essential to test these details in the cruciform tests, as discussed later.

PULL-PLATE TEST RESULTS AND ANALYSIS

Figure 5 shows FEM results for the three unstiffened column sections. Using similar failure criterion of Graham et al. (1960), 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.

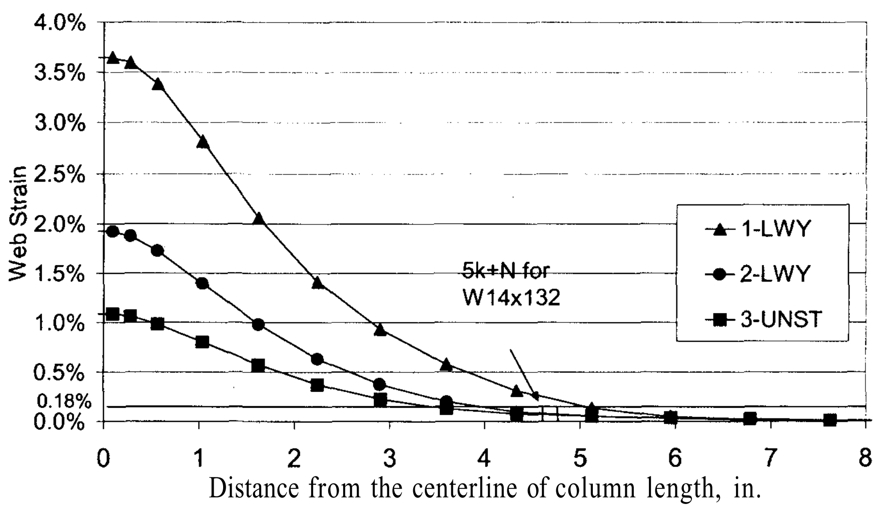


Figure 5 FEM strain distribution in the pull-plate specimens along the column k-line at 450 kips

Figure 6 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, 1998). The strain distribution also shows a much steeper gradient for the W14x132 (1-LWY) than the other two unstiffened sections. This gradient is 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.

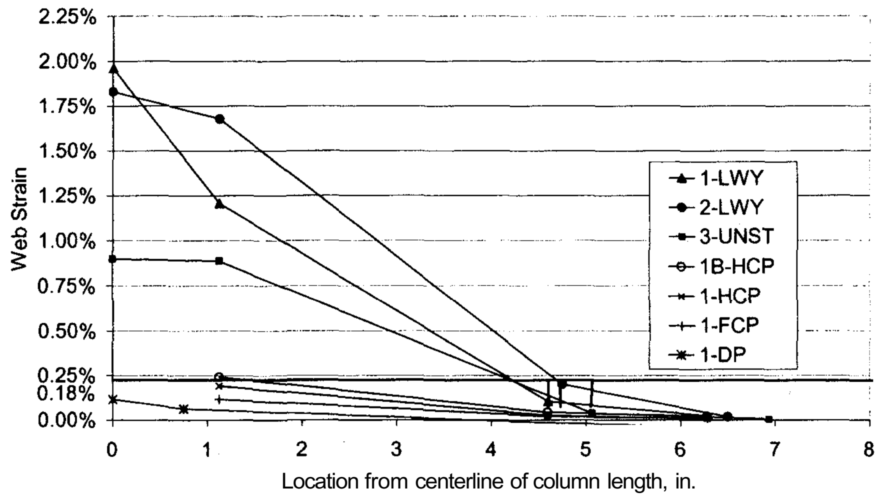


Figure 6 Strain distributions in the pull-plate tests along the column k-line at 450 kips

Figure 7 shows the separation of the flanges near the tips of the flanges along the column length for all nine specimens. 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.

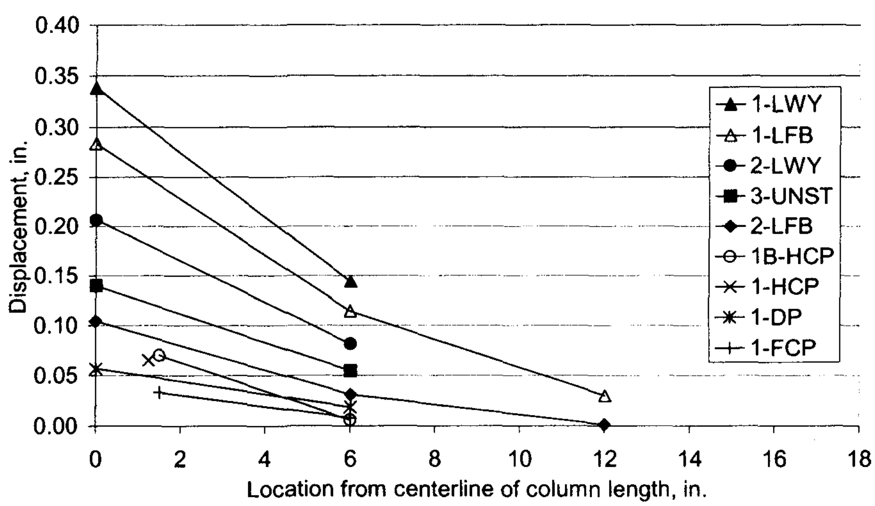


Figure 7 Column flange separation in the pull-plate tests at 450 kips

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 6 and 7 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. 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.

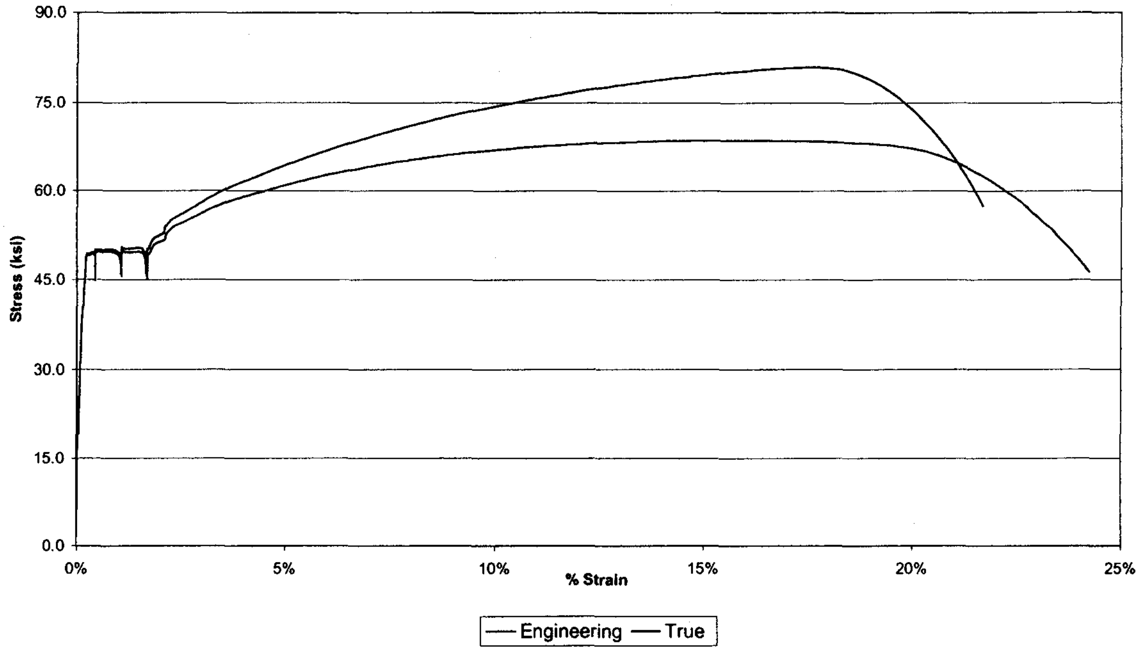
CRUCIFORM TESTING

Five full-scale cruciform experiments will be conducted as part of this project. Table 1 details the specimens to be tested. All material is A992. Figure 8 shows a typical stress-strain curve for the beam section used in all the tests and the W14x283 column. The girder was chosen to develop a relatively large flange force so as to test all stiffening details to their limit states.

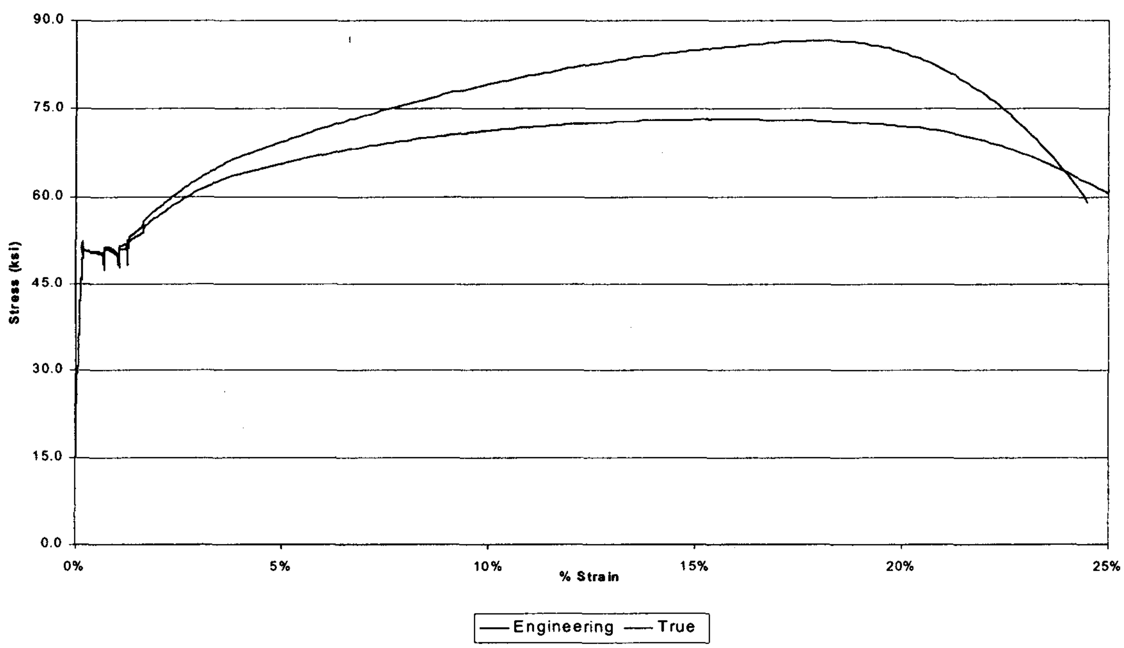
Table 1 Cruciform test matrix

	SPECIMEN				
PARAMETER	1	2	3	4	5
Girder	W24x94	W24x94	W24x94	W24x94	W24x94
Column	W14x283	W14x193	W14x176	W14x176	W14x145
Doubler Plate	None	Detail 2	Detail 2	Detail 3	Detail 1
DP Thickness	NA	0.625 in.	2 @ 0.5 in.	2 @ 0.75 in.	2 @ 0.625 in.
Continuity Pl.	None	None	Fillet welded	None (box)	None
CP Thickness	NA	NA	0.5 in.	NA	NA

Figure 9 shows the experimental test setup. Reverse cyclic loading is applied to the girder flange tips, with the top and bottom of the columns pinned against translation. The SAC loading history protocol is being followed for all specimens (SAC, 1997). Due to actuator stroke limitations, if a specimen has not failed following the 4.0% drift level (two cycles), additional 4.0% cycles are conducted to failure or significant strength deterioration.



(a) W24x94



(b) W14x283

Figure 8 Typical stress-strain curves for girder and column sections tested

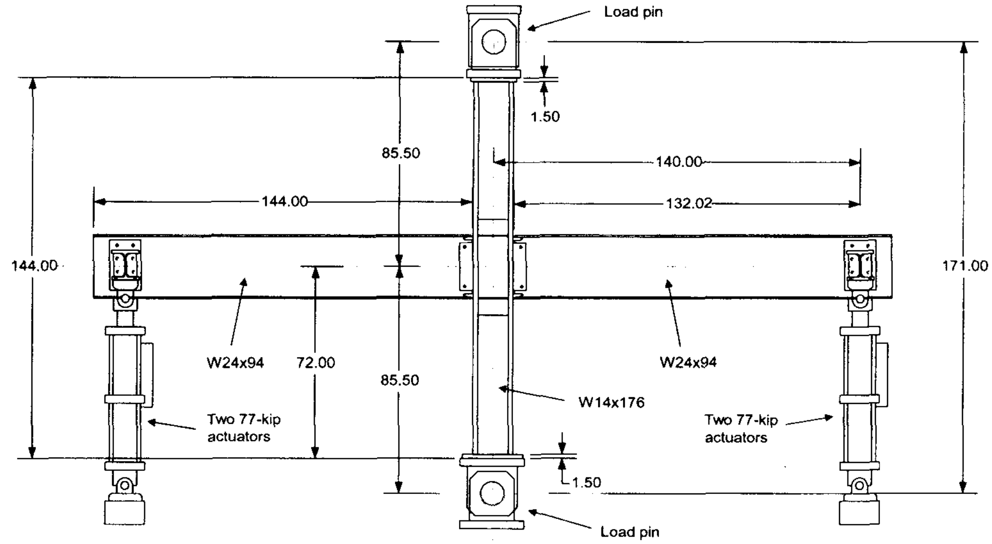


Figure 9 Experimental test setup for cruciform specimens

Specimens 1 through 4 satisfy the SCWB criterion (AISC, 1997) if there is a low column axial compression. All specimens with the exception of the box doubler detail were designed to have approximately the same ratio of calculated panel zone strength to girder plastic moment strength [by comparing the actuator force to initiate each of these limit states according to AISC (1997) and FEMA (1997a)]. However, it is anticipated that the behavior of the panel zones will differ between the specimens, permitting investigation of the relative ductilities between the stiffening details. The final strengths of most panel zones in these tests are somewhat weaker than the required design strengths according to these seismic provisions, so as to test thoroughly the continuity plate and doubler plate details in these specimens. The exception is Specimen 4, which meets the panel zone strength requirements of AISC (1997) and FEMA (2000b).

The connection detail being tested for all specimens is similar to the welded flange-welded web detail (WUF-W) tested for SAC at Lehigh University (Ricles et al., 2000; FEMA, 2000b). The CJP welds joining the girder flanges to the columns are 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 used has a diameter of 5/64 in.

The CJP welds joining the girder webs to the columns are made using the self-shielded FCAW process and E71T-8 filler metal, which also meets minimum CVN requirements. This electrode is used for all reinforcing fillet welds as well. Figure 10 shows the weld details of a typical girder-to-column connection. 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 1/16 in. diameter. For the box doubler detail of Specimen 4, 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.

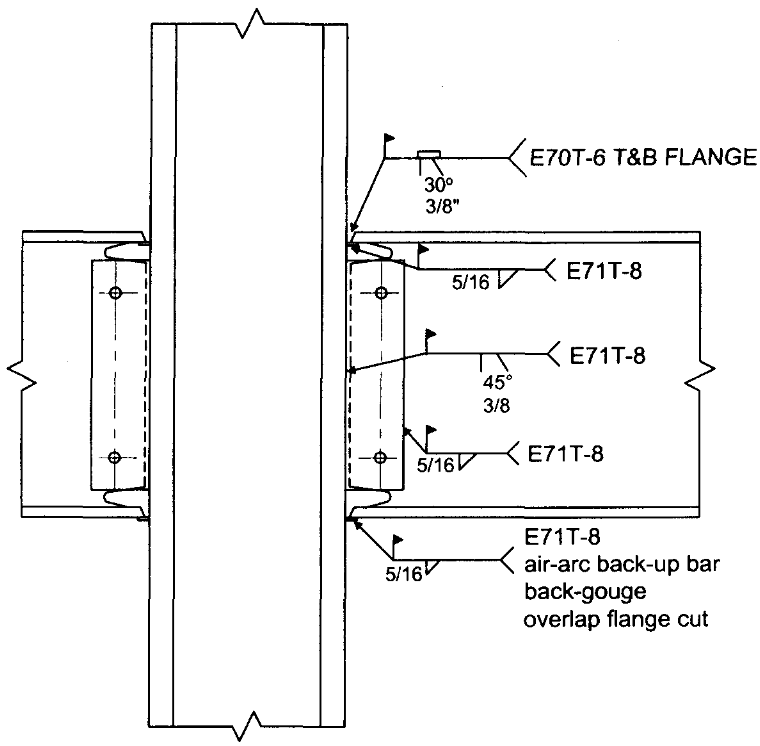


Figure 10 Typical moment connection weld details

The five cruciform specimens investigate three different doubler plate details, shown in Figure 11. Detail 1 is based on Fig. C-9.3(b) of the 1997 AISC Seismic Provisions, while Detail 3 is based on Fig. C-9.3(c); both details were tested in the pull-plate investigation. The intent of this stiffening detail is to eliminate the need for continuity plates by stiffening the column flanges. By offsetting the plates from the column web, the doublers are intended to act both at transverse stiffeners and web doubler plates. Doubler plate Detail 2 is a result of problems that arose during fabrication. The fillet radii of all column sections were measured and found to be significantly larger than the values implied by the *k* dimensions in the LRFD Manual (AISC, 1999b), causing interference between the fillet and doubler plate bevels. A recent AISC Advisory addressed this issue (AISC Dimension Advisory, 2001). Detail 2 avoids the fit-up problems associated with the larger radii by using a square-edged plate that rests on the top edge of the fillet. A gap between the column web and doubler of approximately 7/8 in. typically results. Fillet welding similar to Detail 1 is used to connect the doubler plate to the column flanges. The continuity plates of Specimen 3 were designed similar to the half-thickness, fillet-welded continuity plates tested in the pull-plate experiments.

The first specimen was designed as an unreinforced column with thick flanges. This column was selected to have a substantial contribution from the flanges to the panel zone behavior, permitting evaluation of the post-yield strength contribution of thick column flanges. The column meets the seismic LFB requirements contained in AISC (1992) and FEMA (2000b). It is intended to show that the use of completely unstiffened columns in seismic zones is justified if the column is of sufficient size.

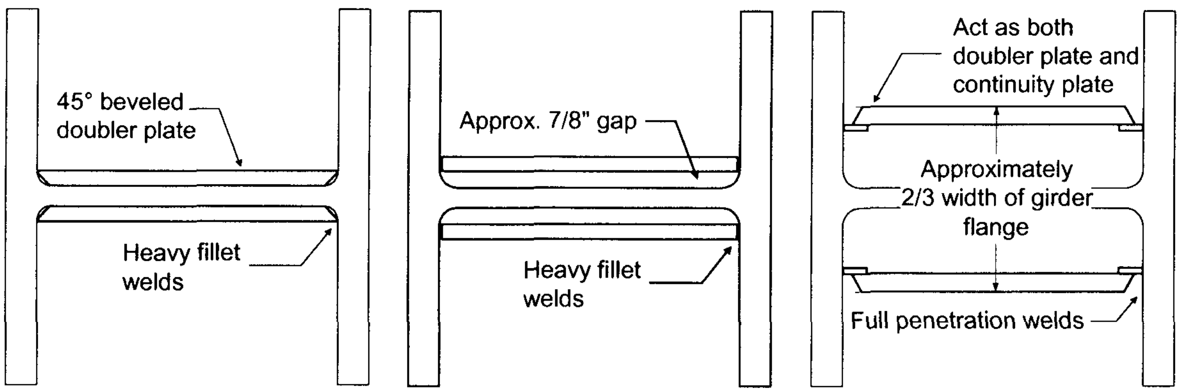


Figure 11 Doubler plate details

The second specimen has a smaller column that includes a doubler plate but no continuity plates. The doubler plate is welded as per Figure 1 lb. This specimen will be used to investigate the cyclic response of a column section just on the cusp of the seismic limit state for LFB (with no ϕ factor), and will be used to investigate the doubler plate effectiveness when attached to the column with fillet welds.

The third specimen tests a weaker column that includes fillet-welded continuity plates that are half the thickness of the girder flange, and doubler plates welded as per the detail in Figure 1 lb. The cyclic response of both the continuity plate and doubler plate details, and the effectiveness of the panel zone stiffening, will be investigated in this test.

The fourth specimen features the box detail of Figure 11c, and no continuity plates. For this specimen, the thickness of the two doubler plates has been increased from that provided in the third specimen in anticipation of the lower effectiveness of this detail, as discussed in Bertero et al. (1973). It is intended to verify the pull-plate results on this detail for seismic applications.

The fifth specimen just fails the non-seismic LFB limit state of AISC (1999b) (with no ϕ factor). This limit state will be investigated in this experiment. For better comparison to the pull-plate results of Specimen 2-LFB, the doublers were welded as per Figure 11a.

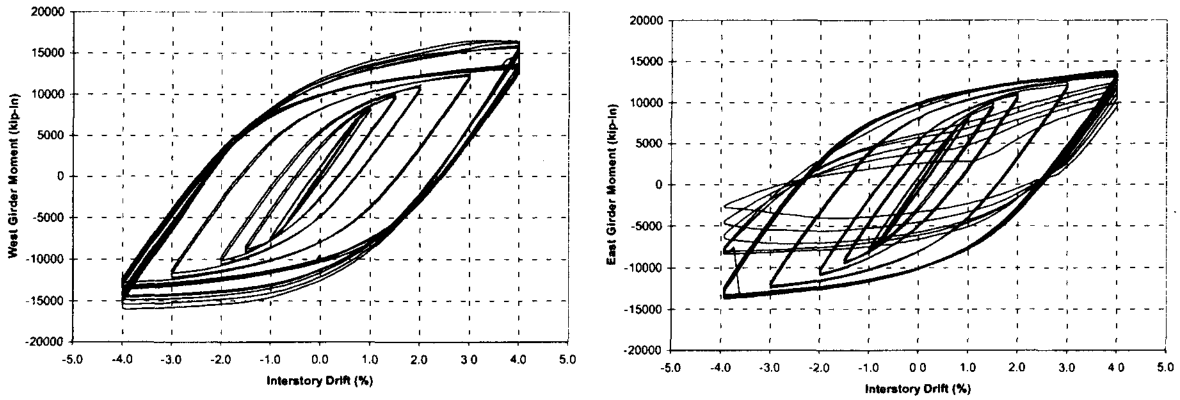
All specimens were heavily instrumented with strain gages to characterize the stress and strain distributions at critical locations, allowing comparison of the effects of the various stiffening details. LVDTs were used to measure the relative contributions of the girders and panel zone to total connection rotation, and to measure column flange separation for the LFB limit state.

CRUCIFORM RESULTS AND ANALYSIS

The testing is ongoing and only the results from Specimen 1 are available at this time. As stated previously, the connection detail being tested for all specimens is the WUF-W connection similar to the detail tested for SAC at Lehigh (Ricles et al., 2000). E70T-6 wire was used for all beam flange welds, while E71T-8 wire was used for the web welds and all reinforcing fillet welds. A different E70T-6 wire was used for the beam flange welds than was used in the pull plate experiments. The wire diameter was 5/64 in. as opposed to the 0.068 in. diameter that was used previously. One difference between these connections and those tested by Ricles et al. (2000) is the weld metal for the critical beam flange welds. E70TG-K2 weld wire was used for the Lehigh specimens and most other SAC tests. As far as has been ascertained to date, E70T-6 weld metal was used only in reduced beam section (RBS) connections (FEMA, 2000d).

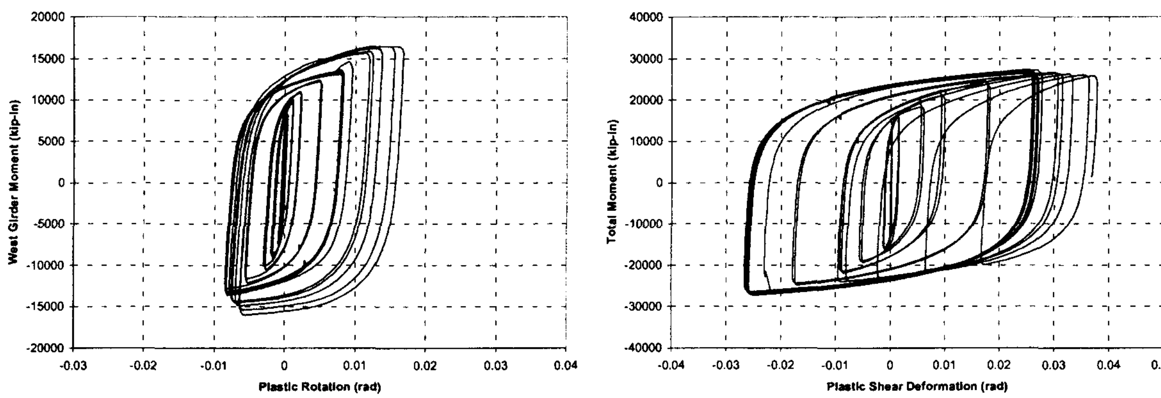
Specimen 1 was an unstiffened W14x283 column with W24x94 girders. High girder flange forces and significant panel zone yielding, with some kinking of the column flanges, were developed in this specimen (as was intended for all the cruciform tests) so as to test the column detailing extensively.

Figure 12 shows the moment vs. interstory drift behavior of the West and East girders of Specimen 1. Strength deterioration following a low-cycle fatigue fracture of the East beam is evident in Figure 12b, during which time the West beam picked up additional load (Figure 12a). Figure 13 illustrates typical plastic rotation components from this test. From this figure, it is clear that panel zone deformation dominated the behavior of Specimen 1 (as expected), accounting for approximately 75% of the total plastic deformation of the connections.



(a) West girder (b) East girder

Figure 12 Specimen 1 moment vs. interstory drift



(a) West girder plastic rotation (b) Panel zone plastic deformation

Figure 13 Specimen 1 plastic rotation components

This specimen performed very well, achieving three to four cycles at 4% interstory drift before any cracks were seen to initiate either through observation or in the instrument data. Low-cycle fatigue cracks initiated at the top and bottom of the welds of the girder web to the column flange, using the shear tab as a backup bar. These cracks initiated at the top and bottom of the shear tab either at the toe of the shear tab fillet welds or possibly at the toe of the vertical CJP welds. Low-cycle fatigue cracks later initiated at the toe of both the bottom and top girder flange CJP welds, in the center of the girder flange width. The cracks progressed slowly, with one bottom flange weld finally severing the bottom flange from the column.

Twenty cycles at 4.0% interstory drift were executed prior to the end of the test, providing valuable constant-amplitude low-cycle fatigue data. A maximum column flange separation of 0.07 in. was measured, suggesting that completely unstiffened columns in seismic zones may be used if the column is of sufficient size. This is, in general, in keeping with final SAC recommendations (FEMA, 2000b) and with other recent research nationwide.

PROPERTIES OF A992 SHAPES

Our LRFD resistance factors and other important design equations were derived from the database of material properties from the 1960's. However, the A992 shapes of today are much different than the predominantly A36 shapes of the 1960's. Near-net-shape continuous casting has reduced the amount of rolling deformation in the production of these shapes. The modern steel is made primarily from scrap rather than iron ore. Rotary straightening is used over a broader range of section sizes today. All of these factors contribute to subtle differences in the properties. The microstructure is finer and the carbon content is lower, and this generally leads to more consistent properties and improved fracture toughness. On the other hand, the yield-to-tensile strength ratio (Y/T) has increased, raising some concerns about ductility.

The other obvious difference is the new specification, A992."Steel for Structural Shapes for Use in Building Framing". Relative to the A36 and A572 specifications, this specification tightened previous chemistry limits, limits residual elements, and requires reporting of Tin. For the first time, this specification places specific limits on carbon equivalent (CE), on the yield point (65 ksi), and the maximum Y/T ratio (0.85).

The Structural Shape Producers Council (SSPC) sponsored compilation of a data base of all the mill test reports that were produced in 1998 by its members for sale in the United States. This information will be used in an ongoing AlSC-sponsored research project to determine whether resistance factors in the AISC LRFD specification, based on data collected in the 1960's for A36 steel, are appropriate.

To incorporate the SSPC data base into the reliability calibration process, it is necessary to determine the relationships between information reported on the mill test certificate and various properties of the steel, including:

- the relationship between the mill test yield strength, which is determined using a relatively rapid rate of loading, and the lower static yield strength, that defines the strength of a steel structural member loaded at a slow rate of loading (see Figure 8).
- the relationship between the steel strength at the location on the shape where the coupon is removed for testing, and the strength at other locations on the cross section..

These objectives will be achieved through a joint testing program at the University of Western Ontario, under the direction of Prof. Michael Bartlett, and the University of Minnesota. The testing program involves five tensile coupons from each of 30 shapes provided by American mills that are members of AISC. The shapes selected were wide flange sections and were chosen to represent a wide range of thicknesses and weights. The full stress-strain curve in terms of engineering stress and strain and true stress and strain will be obtained as shown in Figure 8. Many different parameters will be obtained from the stress strain curves for comparison, including different definitions of yield strength, power law fit the stress strain curve, etc.

This research parallels a project currently under way at University of Western Ontario, sponsored by the Steel Structures Education Foundation of the Canadian Institute of Steel Construction and the National Research Council of Canada, to review the resistance factor for steel in Canadian Standard CSA S16.1 "Limit States Design of Steel Structures".

The statistical distributions of the SSPC data were characterized by plotting a histogram of the occurrence in a range of intervals (bins). Figure 14 shows the histogram for the yield strength for A992 steel, for example. The height of the bar is the frequency of occurrence as shown by the left axis. The actual value of the frequency is also given to three significant figures at the top of each bar. Presumably, various probability density functions may be fit using these values. The line in each plot represents the cumulative probability of occurrence, as shown on the right axis. The cumulative probability is the summation of all occurrences less than that value.

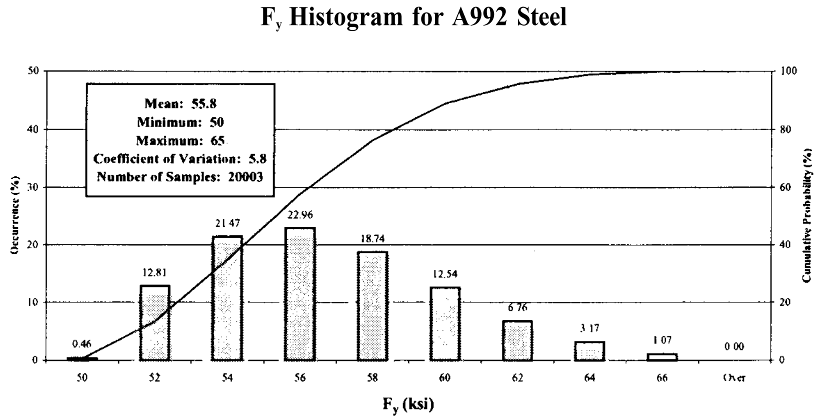


Figure 14 Histogram of the yield strength of A992 wide-flange shapes

Standard deviation and mean were calculated for the data set shown on each plot. The mean is shown in the box on each plot, along with the coefficient of variation, i.e. the standard deviation divided by the mean, expressed in percent. Also given are the minimum and maximum values and the number of samples. It can be seen that the yield strength of the 20,003 samples of A992 steel ranged from 50 to 65, with a mean of 55.8 ksi.

Figure 15 shows the histogram for the ultimate strength data and Figure 16 shows the histogram for the Y/T ratio. The properties are generally as expected based on the specifications. The A992 has slightly smaller mean and ranges than the other Grade 50 steels, as expected because of the specification. A36 steel has more extreme values than A992 on the minimum and the maximum for yield strength and Y/T, as expected. However, the mean yield strength of the A36 steel is less than the A992 steel as expected, and the tensile strengths were much lower than the grade 50 steels.

Figure 17 shows the histogram of the CVN data for A992 steel at 40 degrees F. The data show that the steel shapes have very good toughness with a mean value over 120 ft-lbs.

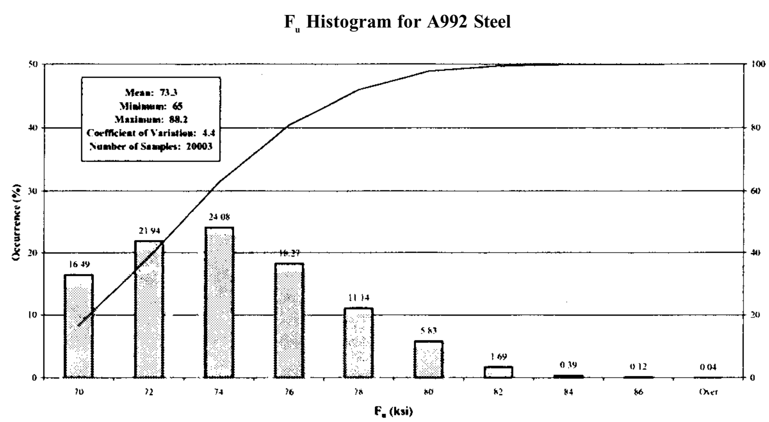


Figure 15 Histogram of the ultimate tensile strength of A992 wide-flange shapes

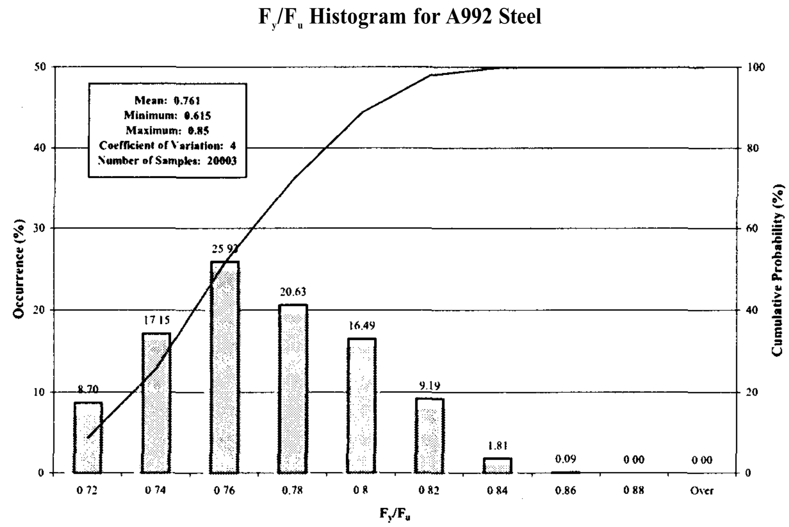


Figure 16 Histogram of the yield-to-tensile strength ratio for A992 wide-flange shapes

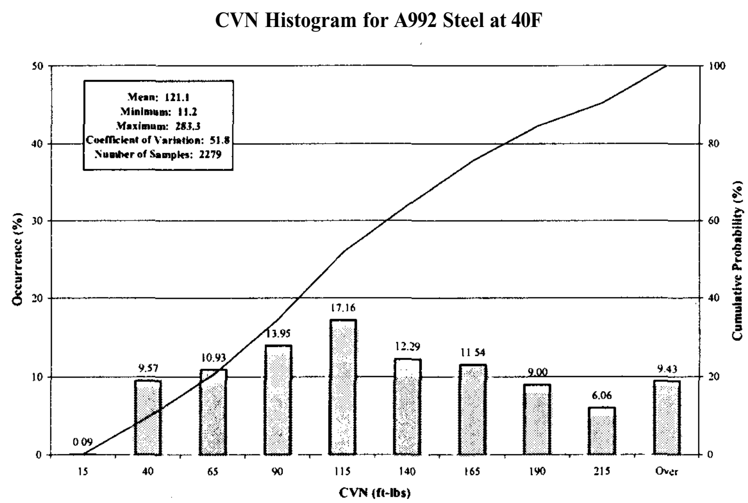


Figure 17 Histogram of the Charpy toughness of A992 wide-flange shapes

CONCLUSIONS

A combined experimental/computational research program is being conducted to reassess the design criteria and investigate new alternatives for transverse stiffeners and doubler plates. Experimental components of this research include nine pull-plate tests to investigate the limit states of local web yielding and local flange bending, as well as investigating new alternatives for the detailing of continuity plates, particularly for non-seismic design; and cyclic cruciform joint tests to investigate panel zone shear behavior, test new doubler plate detailing alternatives, and verify continuity plate details, particularly for seismic detailing.

The preliminary conclusions from these tests include:

1. The AISC provisions for LWY and LFB are reasonable and slightly conservative in calculating the need for column stiffening.
2. None of the E70T-6 CJP welds fractured in the pull-plate tests or in the unstiffened cruciform test, despite plastic deformation, even when the flange tip separation was over ¼ in, indicating that continuity plates 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.

3. Based on the results of the pull-plate tests and finite element analyses, 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.
4. The new stiffener details, i.e. the box detail and beveled doubler plates fillet welded to the column flanges, performed satisfactorily in the pull-plate tests and provided sufficient stiffness to avoid LWY and LFB.

A database of all the material test reports for 1998 was assembled. The data were sorted by grades and shape group and histograms were plotted of the data. Summary statistics including the mean, median, extreme values, standard deviation, and coefficient of variation were evaluated. The conclusions from this study are:

1. The tensile properties of A992 steel are not substantially different than the tensile properties of the grade 50 steels in previous surveys. The mean yield strength of 55.8 ksi, is 5.8 ksi greater than the minimum specified yield strength. The mean tensile strength is 73.3 ksi, which is 8.3 ksi more than the minimum specified tensile strength. The mean ratio of yield-to-tensile strength is 0.76.
2. The CVN properties of A992 steel were very good and are not substantially different than the CVN that were reported by AISC in 1995 for Grade 50 steels. Only 0.25% of the values at 32 F are less than 15 ft-lbs, and only 25% of the values at 32 F are less than 40 ft-lbs.

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