Constrained Through-Thickness Strength of Column Flanges of Various Grades and Chemistries

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ABSTRACT

Tee-joint specimens were fabricated with high-strength "pull" plates welded transversely to opposite flanges of short lengths of heavy column shapes to determine strength, deformation, and fracture behavior of the flanges of wide-flange column shapes when loaded in the through-thickness direction. Forty-seven specimens were tested, including wide-flange shapes obtained from four steel mills from 1995 to 2000 conforming to specifications A572 Gr. 50, A992, and A913 Gr. 50 and 65. Sulfur levels ranged from 0.003 to 0.043 percent and Carbon from 0.05 to 0.20 percent, up to the limits of the current specifications. Several shapes with especially high sulfur were specially produced for this research. The through-thickness strength of the column flanges tested exceeded the 690 MPa (100 ksi) yield strength of the pull plates, well above any possible demand that could come from Gr. 50 beam flanges.

INTRODUCTION

In welded moment connections in steel frames, flanges of the beam are groove welded directly to the flanges of the column. Under lateral loading, large forces in the beam flanges from moments generated in the beam are applied normal to the column flanges. These normal forces are accompanied with shear forces and secondary bending of the beam flange. In the event of a major earthquake, plastic rotations are expected to develop within the beams at the beam-to-column connections.

The beam-flange-to-column-flange joint relies upon the strength of the beam flange, beam flange to column flange weldment, and the through-thickness strength of the column flange. Therefore, all potential failure modes of these elements of the load path must be investigated. Research on the potential through-thickness failure mode of the column flanges was performed for the SAC Joint Venture in 1997 (Dexter and Melendrez, 2000; Dexter and Melendrez, 1999). This research included a review of failures that occurred in the Northridge Earthquake and in subsequent testing, material characterization testing, and tee-joint tests. Subsequently, research has also been performed on A913 Gr. 65 column shapes and column shapes produced with intentionally high sulfur levels. This paper considers the results of all these investigations.

Many engineers remain concerned about lamellar tearing; a potential problem with through-thickness loading that is described in many textbooks. However, steel produced since 1980 is not expected to be susceptible to lamellar tearing because producers have been controlling the large non-metallic inclusions that used to cause lamellar tearing (Ganesh and Stout, 1976; Kaufmann, Pense, and Stout, 1981; Kaufmann and Stout, 1983).

Despite the inclusion control, the results of uniaxial through-thickness tensile tests performed in accordance with ASTM A770 still show a high degree of scatter including some low strength values that may even be below the minimum specified yield strength (MSYS) in the longitudinal direction. In addition, the reduction in area of these through-thickness tests is often less than the reduction in area in the longitudinal tests, indicating the ductility of the material is less in the through-thickness direction.

For example, Figure 1 shows the locations from which longitudinal and through-thickness uniaxial tensile specimens were taken from wide-flange column shapes (each length of wide-flange section from a unique production heat will be referred to as a "shape", multiple tee-joint specimens and material characterization specimens may have been extracted from each shape). Table 1 (Appendix) shows the results from a W14×176 column shape. Note that the reduction in area at the TA-2 location, in the web/flange core region, is particularly low. This particular shape gave the lowest such reduction in area among 11 shapes for which A770 tests were performed.

Barsom and Korvink (1998) have compiled similar uniaxial through-thickness test data. Barsom and Korvink point out that "the inelastic properties and behavior of steels

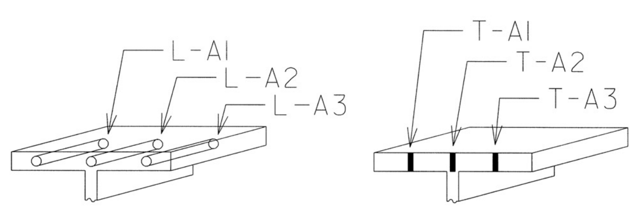

Fig. 1. Locations of uniaxial tensile test specimens.

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obtained from a simple uniaxial test….are basic measures of inherent material properties…. these fundamental properties differ for different states of stress and strain (i.e. constraint), temperature, and rate of loading." They demonstrate that the inelastic properties obtained from a uniaxial test do not represent the behavior of actual connections (Barsom and Korvink, 1998).

For example, the uniaxial through-thickness tests are not representative of the highly constrained conditions under which the column flange base metal is loaded in tee-joint connections. The experiments and analyses described in this paper indicate that under the constrained conditions typical of a tee-joint, the column shapes do not yield in the through-thickness direction. Because it is not possible to achieve yielding in the through-thickness direction, the measured through-thickness yield strength and reduction in area should not be a concern, i.e. the ductility in that direction is never needed.

THE EFFECT OF CONSTRAINT

In a uniaxial tension test, the stresses in the transverse directions are zero. We use the stress in the axial direction as the key result from a tensile test. However, the axial stress is not directly related to yielding. It is actually the shear stresses that cause yielding and flow, as evidenced by slant fractures, and cup-and-cone fractures. The key parameter for yielding is the shear yield stress. A cube of material acted on by various triaxial normal stresses will not yield unless the shear stress on any plane reaches the shear yield stress. This is why deviatoric stresses are used in constitutive models, because these stress differences are related to the shear stresses. The usual Cauchy stress (what we usually just call stress) can be thought of as the sum of deviatoric stress (which could cause yielding) and hydrostatic stress or pressure, which does not cause any distortion or yielding of the solid. The hydrostatic stress is simply the average of the three principle normal stresses.

To explain the effect of constraint, consider a "scoop" of column flange material that is postulated to pull away with the beam flange. This scoop of yielding column material is surrounded by column flange material that is not yielding, which prevents shrinkage in the column transverse and longitudinal directions to accommodate stretching in the through-thickness direction. The scoop of column material will begin to develop tension in both the transverse and longitudinal directions, in addition to tension in the through-thickness direction due to the applied load. Thus, constraint elevates the mean stress of the three principal directions, which in turn elevates the apparent yield strength in the through-thickness direction.

Yielding of steel can be idealized as being governed by the Von-Mises Yield Criterion:

$$\sigma_y = \sqrt{\frac{3}{2}\sigma'_{ij}\sigma'_{ij}} \tag{1}$$

where

σ_y = the uniaxial yield stress, and

σ'_{ij} = the deviatoric stress tensor.

The magnitude of deviatoric stress is dependent on the hydrostatic stress as given by:

$$\sigma'_{ij} = \sigma_{ij} - \sigma_{kk}/3 \tag{2}$$

where:

σ'_{ij} = the deviatoric stress tensor,

σ_{ij} = the applied stress tensor, and

$\sigma_{kk}/3$= the hydrostatic stress.

Figure 2 shows the Von-Mises Yield Criterion as a surface in three-dimensional space defined by the three principal stresses in orthogonal directions. The yield surface is represented by a cylinder about the axis defined by $\sigma_1 = \sigma_2 = \sigma_3$, i.e. where the deviatoric stresses are zero.

In the case of the tee-joint, the through-thickness direction for the material in the column flange would be the σ_1 direction, while the σ_2 and σ_3 direction would be the longitudinal and transverse directions in the plane of the column flange. At a given load level in the pull plate or beam flange, proportional to σ_1, if hydrostatic stress is increased by constraint (tension in the σ_2 and σ_3 direction), the magnitudes of the deviatoric stresses, which govern yielding, are reduced. Thus, because of constraint, larger σ_1 stress is required to yield than would be required if the loading were uniaxial (i.e. if σ_2 and σ_3 are zero). An apparent increase in the through-thickness strength is observed (relative to uniaxial behavior), and increases in applied load can be achieved.

The Von-Mises Yield Criterion would predict that if the stresses in all three directions are equal, the material could theoretically tolerate infinite stress without yielding. Consider a cube of Gr. 50 material under extreme fluid pressure. Even if the fluid pressure exceeds 690 MPa (100 ksi) the cube of material will not yield, since it is acted on in all

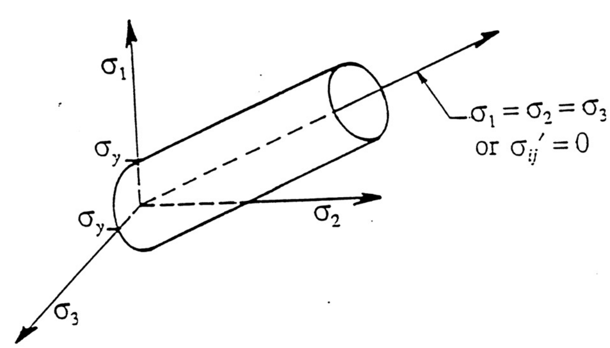


Fig. 2. Von-Mises Yield Criterion in principal stress space.

three directions by this pressure. This is the same principle at work in the column flange. However, unlike compressive hydrostatic stress (pressure), if a large enough tensile hydrostatic stress is applied, fracture will eventually occur.

Yielding will not occur unless there are significant differences between the stresses in the three principle directions. If there is tensile hydrostatic stress, the actual normal stress in the principal loading direction at yield will exceed the uniaxial yield strength and could even exceed the tensile strength. For example, you can show using the Von-Mises flow rule that if the stresses in the beam flange and the column are increased proportionally, and the stress in the transverse direction is constrained, i.e. equal to Poisson's ratio times the sum of the other stresses, then the stress in the through thickness direction at yielding will be 2.5 times the uniaxial yield stress, well above the tensile strength.

To represent isotropic strain hardening, the yield surface expands. Equation 1 gives the stress to cause continued plastic flow, where σ_y is replaced by an effective flow stress that increases from the uniaxial yield strength to the uniaxial tensile strength. Thus, the tensile strength is affected by the triaxial stress state in the same way that the yield strength is affected.

Therefore, the tensile strength is not a general "fracture criterion." The tensile strength is just the engineering stress in a uniaxial tensile test at which necking begins. The tensile strength is only applicable to uniaxial stress states similar to the stress state in the tensile test. In other configurations with different stress states, necking and failure will occur at peak stresses different from the uniaxial tensile strength.

Of course there is not a perfect hydrostatic state of stress in the column flange adjacent to the beam flange. However, the beneficial effect of constraint is clearly at work in these tee-joint tests and in the real welded moment connections. Another example of a beneficial effect of constraint is that it allows the weld metal strength to be slightly less than the base metal strength in butt joints, a practice called undermatching that is advantageous in specific situations (Dexter and Ferrel, 1995; Ferrel and Dexter, 1995). In a similar way, the constraint elevates the apparent yield strength of the butt weld preventing strain from localizing in the undermatched weld.

Ordinarily, constraint is viewed as undesirable because it tends to increase the potential for brittle fracture. The potential for fracture is unrelated to the tensile properties (yield strength or tensile strength). Fracture is dependent on the type of flaws and the fracture toughness of the steel. Unlike yielding which is driven by the deviatoric stresses, fracture is sensitive to the hydrostatic stress. Therefore constraint increases the potential for brittle fracture. However, if the weld, the column base material, and the heat-affected zone (HAZ) have sufficient fracture toughness, the constraint can be tolerated.

The requirement for the performance of the column flange material in a welded moment connection should be based on the maximum axial force and stress that could be delivered by an A572 Gr. 50 beam flange. In order to assure the beam can reach plastic moment on the gross section, the beam flanges at the connection to the column may have to strain harden considerably to compensate for the lower plastic modulus at the connection. Therefore, the maximum nominal axial stress in the beam flange, and hence the maximum demand in the column through-thickness direction, is the ultimate tensile strength of the beam (450 MPa or 65 ksi, if the discussion is limited to Gr. 50 beams).

EXPERIMENTS

The Von-Mises three-dimensional yield criterion predicts that if material is constrained from yielding in all directions, as in the case of the column flange material just under the surface attached to the beam flange weld, that it will not yield, even at very high stress levels. The Von-Mises Yield Criterion does not predict when fracture will occur. Brittle fracture will eventually be the failure mode, and high triaxial stresses increase the potential for brittle fracture. Therefore, it was important to experimentally verify, for the full range of column flange materials available, that this constrained material behavior occurs as predicted; and, that the column materials can take the high levels of triaxial stresses induced by beam flange forces without experiencing brittle fracture.

Material Characterization

Tee-joint experiments were conducted on column shapes from 14 different heats of steel obtained from 1995 to 2000 from four steel mills, as shown in Table 2. These shapes conformed to A572 Gr. 50, A992, and A913 Gr. 50 and 65. All of the shapes were continuously cast, with the possible exception of one recovered from the L.A. County Building (listed at the top of Table 1) that had suffered fractures in the Northridge Earthquake. Nothing else is known about this W14×455 shape; it was probably produced in the 1970s or 1980s. Three types of steel were included in the testing:

1. Scrap-based electric furnace steel;
2. Production from integrated mills; and
3. Quenched and self-tempered (QST) steel (ASTM A913).

In 1995, a W14×257 produced by Northwestern Steel and Wire was obtained that had exhibited a k-line fracture in a full-scale connection test. A572 Gr. 50 W14×176 and W14×257 column shapes were obtained in 1997 from TradeARBED, Corus Group (formerly British Steel), and Nucor-Yamato Steel. These shapes are from ASTM

A6/A6M Groups 3 and 4, respectively. The TradeARBED steel is all made with the QST process and therefore also conforms to the A913 Gr. 50 specification. Corus also provided a W14×455, also from Group 4. TradeARBED also provided A913 Gr. 65 W14×257 and W14×605 shapes in 1999. The W14×605 shape is from Group 5, and represents the thickest flanges (over 100 mm or 4 inches thick) typically used in the U.S.

In 2000, Nucor-Yamato Steel provided six specimens from three heats of specially prepared high-sulfur steel, including a Group 2 W10×68 shape. Therefore, the shapes tested represent a range in years (including one specimen probably produced in the 1980s), all the presently significant producers, all the U.S. specifications, and all relevant shape groups.

Both flanges of the entire stock of column shapes received in 1997 were ultrasonically tested to attempt to find any lamellar defects. None was found in any of the column shapes.

For most of the shapes, through-thickness tensile tests were conducted using the 13 mm (0.5 inch) diameter round uniaxial specimen of ASTM A770. Samples were taken from the locations in the flanges shown in Figure 1. All tests described in this paper were conducted at room temperature. Table 3 compares the yield strength and reduction in area in the flange/web core region for the two orientations for the typical Gr. 50 shapes, i.e. excluding the Gr. 65 shapes and the specially produced high-sulfur shapes, which are discussed separately. A small reduction in strength in the through-thickness direction is apparent for only a few of the shapes; i.e. only the W14×257 in the first and third row.

Since the TradeARBED shapes are produced by the quenched and self-tempered (QST) process (meeting both specifications A572 Gr. 50 and A913 Gr. 50); there is a gradient in the strength through the thickness of the web and flanges. The flanges are higher strength near the outer casing and softer in the middle of the thickness. The average strength, which is what is relevant to design and structural performance, must meet the specification. Usually, the full-thickness flat-strap specimens are tested to determine the average strength of the flange.

However, the results reported in Table 3 are from 13 mm (0.5 inch) diameter round tensile specimens that were machined from the softest region of these shapes near the mid-thickness of the flange. Consequently, the yield strength in the longitudinal direction is below the MSYS of the Gr. 50 shapes (345 MPa or 50 ksi). It is likely that if full-thickness specimens had been tested, however, that the strength would meet the MSYS. Note that the through-thickness strength for the QST shapes is essentially the same as the longitudinal strength at the center of the flange.

Table 3 also shows that the reduction in area is typically degraded in the through-thickness direction relative to the longitudinal direction. The W14×176 in the second row of Table 3 had the lowest reduction in area of all the A770 tests performed in this study. This is the shape for which data from all the locations were given in Table 1.

The shapes from one producer (fifth and sixth row) did not show degraded reduction in area. This could possibly be related to the fact that these shapes also had the lowest sulfur content, typically less than 0.004 percent, less than 5 times smaller than the typical 0.02 percent values for the rest of the shapes (see Table 4).

The chemical compositions of all column shapes are listed in Table 4 (except the specially produced high-sulfur shapes which are discussed separately since they are atypical). The table shows that the carbon content ranges from 0.05 percent to 0.20 percent, up to the limits of A992 and previous specifications. The shape from the L.A. County Building had the greatest carbon content of 0.20 percent, followed by the Northwestern shape from 1995 with 0.15 percent. The shapes produced in 1997 or subsequently have had carbon contents less than 0.1 percent.

The sulfur content for most shapes is about 0.02 percent. The sulfur content in the column shapes from one manufacturer is significantly lower, ranging from 0.003 percent to 0.004 percent. This manufacturer reportedly performs desulfurization. Note that the Gr. 65 W14×605 also had very low sulfur levels and was also apparently desulfurized.

The Carbon Equivalent shown in the last row of Table 4 was calculated using the IIW formula that is also used in A913 and A992 specifications. This formula is:

$$CE = C + Mn/6 + (Cr + Mo + V)/5 + (Cu + Ni)/15 \tag{3}$$

Table 5 shows the tensile properties for the A913 Gr. 65 W14×257 and W14×605 shapes, obtained from ordinary flat-strap tensile specimens taken from the flange. Through-thickness tests according to ASTM A770 were not conducted for the A913 Gr. 65 shapes.

The A992 shapes procured in 2000 with intentionally high sulfur levels were from two different heats for the W14×257 and one heat for the W10×68. These shapes are from AISC Groups 4 and 2, respectively. The chemical compositions of these column shapes are listed in Table 6. The mill test reports compare reasonably well to the results we obtained from an independent test laboratory. Note the very high sulfur level for these shapes that is near the A992 specification limit of 0.045 percent. Such high sulfur levels can occur naturally in the smaller shapes, such as the W10×68. However, this level of sulfur is very unusual for larger column shapes such as the W14×257 (Dexter et al., 2000).

Table 7 shows the values of the longitudinal properties for these high-sulfur shapes that were obtained from a full-thickness flat strap specimen taken from the flange. Three through-thickness tests were also conducted for each of these shapes according to ASTM A770. The round through-thickness specimens were all taken from the T-A2 position shown in Figure 1, i.e. directly over the web.

For the lighter W10×68, the through-thickness yield and ultimate were not less than the longitudinal values. (The data for the yield on one specimen was lost and there was a great difference between the two measured through-thickness yield stress values). On the other hand, the through-thickness yield stress levels for the W14×257 shapes were well below the MSYS and the reduction in area for these shapes was relatively low. Comparing these data to the data for typical W14×257 shapes listed in Table 3, combined with the relatively good through-thickness properties from one producer with very low sulfur levels, would indicate that high-sulfur tends to decrease the through-thickness strength and ductility in uniaxial tests for these Group 4 jumbo shapes.

However, the Group 2 W10×68 shape is not affected as much as the shapes for which the data are shown in Table 3, and the worst reduction in area was from the W14×176 that had sulfur of 0.02 percent or less. Furthermore, the results of this uniaxial A770 test are not relevant to structural performance, as pointed out by Barsom and Korvink (1998). This is affirmed by good performance in the tee-tests (discussed later), which indicate that the sulfur does not have an adverse effect on the constrained through-thickness strength relevant to the actual conditions in the joint.

These material characterization data show that the tee-joint tests were performed on a variety of materials exhibiting the full range of carbon and sulfur allowable in the A992 specifications; and exhibiting a variety of uniaxial through-thickness strength and ductility values, including some that show significant degradation of strength and ductility in the through-thickness direction. As it turns out, there is no correlation between the results of any of these properties (the results of the A770 uniaxial through-thickness test, the chemistry, etc.) and the capacity of the column flanges in the through thickness direction under the constrained conditions of the tee-joint. In the tee-joint, the measured through-thickness strength was limited only by the strength of the pull plate, so all the results are approximately the same, as explained later.

Tee-joint Test Specimen

Tee-joint tests were designed to simulate the constraint and other conditions associated with the full-scale beam flange to column flange weld joint in typical moment connections. The tee-joint test specimen is shown in Figure 3. The specimen features a whole column shape with a replicate weld on both sides. Thus, two column flanges are subjected to the loads in each test. Tee-joint specimens like these have been used previously to characterize web yielding and flange bending failure modes (Graham, Sherbourne, Khabbaz, and Jensen, 1960).

Slightly different tee-joint specimens have been used recently to assess the strength and fracture behavior of welds (Kaufmann, Xue, Lu, and Fisher, 1996). In Kaufmann's tee-joint specimens, the column section was a tee section cut from a WF column shape and consequently had

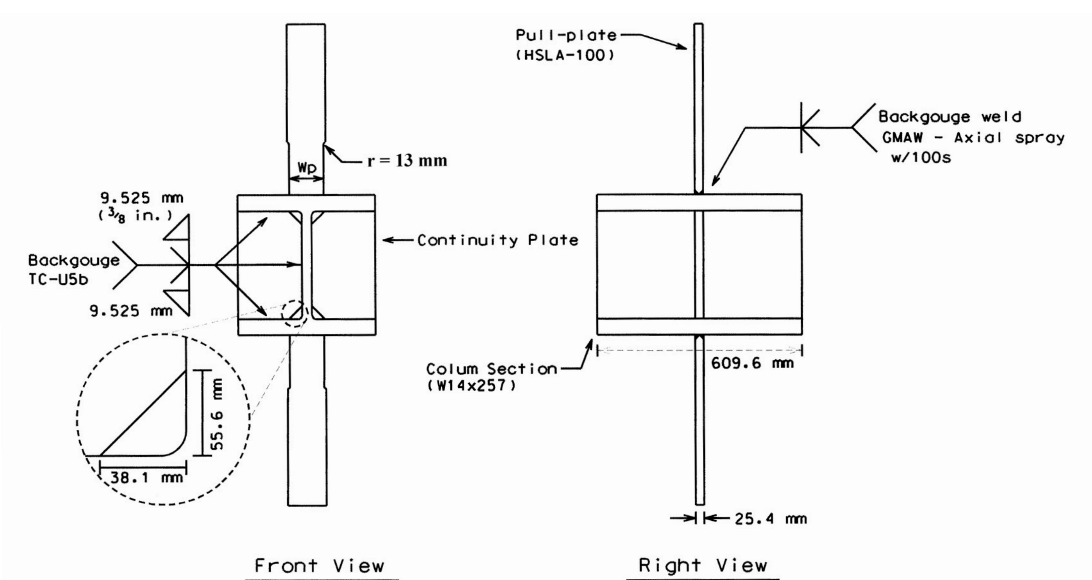


Fig. 3. Basic configuration of tee-joint test specimen (1 inch = 25 mm).

only one column flange and one tee joint. Kaufmann (and Graham et al., 1960) also used Gr. 50 pull plates.

In a two-sided interior moment connection in a lateral force-resisting frame, the tension flange is opposite a compression flange, so the direct transfer of tension through the column in the tee-joint specimen is not the same as in a moment connection. However, the direct transfer of tension through the column is believed to present a greater demand (in terms of the through-thickness strength of the column flange and the continuity plates) than the more realistic transfer of that tension into panel zone shear.

There are at least two other important differences between the tee-test and the actual moment connection. First, the actual beam flanges carry shear as well as axial force and bending. The shear force is not expected to have a significant effect on the potential for through-thickness fractures, however. Finally, the loading in these tests is monotonic, whereas the loading in the actual moment connection is cyclic. (Cyclic loads would have been very difficult to apply in these tests given the type of grips that were used.)

However, if the tests had been loaded cyclically, it is expected that the effect of cyclic loading would not be enough to cause different failure modes; i.e. to cause through-thickness fractures in lieu of pull-plate fractures. If the effect of cyclic loading were significant with respect to through-thickness failure, through-thickness failures should occur occasionally in cyclic loading experiments and in frames loaded by earthquakes. In fact, there is a lack of such failures in cyclic loading that can be attributed to lamellar tearing or through-thickness strength or ductility of the column flange.

For example, no failures that occurred in the 1994 Northridge Earthquake can be attributed to the through-thickness properties of the column (Dexter and Melendrez, 2000; Paret, 2000; Paret and Freeman, 1997; FEMA, 1997a; Fisher, Dexter, and Kaufmann, 1997; Bonowitz, Durkin, Gates, Morden, and Youssef, 1995; Kaufmann and Fisher, 1995; Kaufmann, Fisher, Di Julio, and Gross, 1997; Tide 1997). Fractures did occur that appeared to scoop out a chunk of the column flange, however these failures originated in the welds and are therefore not attributable to the column flange properties (Fisher et al., 1997; Kaufmann and Fisher, 1995; Kaufmann et al., 1997; Tide, 1997). Similarly, no fractures that can be attributed to column through-thickness properties have occurred in cyclic testing (FEMA, 1997b; FEMA, 2000; Ricles, Mao, Kaufmann, Lu, and Fisher, 2000). The lack of through-thickness failures indicates that the resistance of the column flange to through-thickness failure under cyclic low-cycle fatigue loading is relatively better than other parts of the connection.

The tee-joint tests were performed at high and low strain rates and with two widths for the pull plate or simulated beam flange, as shown in the test matrix in Table 8. Bending loads were applied at slow strain rates in seven tests to simulate the effect of local bending that occurs in beam flanges in the vicinity of the weld access hole.

Most of the testing was performed with a 102 mm (4 inches) wide pull plate with a reduced section "gage length" of 152 mm (6 inches). These specimens were loaded by a 2700 kN (600 kip) capacity high-stroke rate universal-testing machine. Linear displacement variable transducers (LVDTs) were used to measure the elongation of the specimen. Most of these tests were conducted at a crosshead displacement rate of 5 mm (0.2 inches) per second. Most of the strain occurs within the two 152 mm (6 inch) long reduced-width sections of the pull plates but there is some stretching in the column shape, especially in the web, and there is also some compliance in the test machine. The measured strain rates were about 10^{-2} per second. To put the strain rate in perspective, for Gr. 50 steel, this would reach yield strain in 0.2 seconds, whereas for the HSLA-100 pull plates, it takes about 0.4 seconds to yield. The time to reach about 3.0 percent strain would be about 3 seconds. The slow strain rate tests were performed at a strain rate less than 10^{-4} per second.

The maximum force level that can be produced with a 102 mm (4-inch) wide pull plate is about 1800 kN (400 kips). This may be less than the force level produced by the flanges of some very large beam shapes. To investigate the effect of plate width and force level, some of the specimens had 305 mm (12-inch) wide pull plates, with a proportionally larger gage length of 457 mm (18 inches) and the same thickness and other dimensions. These specimens were tested in a 22000 kN (5,000 kip) testing machine at Lehigh University and were loaded at a quasi-static loading rate of 1.27 mm/min. (0.05 inch/min.) (strain rate less than 10^{-4}). Unlike the narrower pull plates, the 305 mm (12-inch) wide pull plate of 690 MPa (100 ksi) yield strength steel can deliver very high load (more than 5400 kN or 1,200 kips) through the column shape. Strain gages in the experiments with 305 mm (12-inch) wide pull plates show that this force level is sufficient to yield the continuity plates all the way across and to yield part of the web as well. This load is greater than the maximum beam flange force at 450 MPa (65 ksi) for all presently available WF beam shapes except 10 shapes with extremely large flange areas. To put this force in perspective, the maximum flange force for a W36×150 beam, which is considered a large beam, is 3250 kN (730 kips).

As shown in Table 8 and explained further below, many of these tests involved variations such as continuity plate details, whether just a flange was tested rather than the whole section, welds made with high heat input, and material variations. In addition, not shown in Table 8, there were variations in the weld reinforcement and weld details such as backing bars (Dexter and Melendrez, 1999).

Figure 4 shows a polished and etched macrosection of a typical pull plate to column flange weld. The pull-plate is the vertical element in Figure 4, and the flange is an A913 Gr. 65 W14×257. The double-groove welds were made with the GMAW process and an ER100S-G weld wire. No preheat was used, and two levels of heat input were used, approximately 1.5 kJ/mm (40 kJ/in.) and 3.5 kJ/mm (90 kJ/in.). The high heat input is known to be detrimental to the weld properties but was used anyway to try to maximize any potential effect of the weld on the column shape, creating a greater heat-affected zone (HAZ) width and possibly more brittle HAZ microstructures.

Weld reinforcement was specified to be less than 6 mm (0.25 inch) all around. The weld shown in Figure 4 has a typical amount of weld reinforcement. Reinforcement increases the area at the column face and provides a transition radius in the reentrant corner that reduces the stress concentration. It moves the location of the last cap pass, which is the location of weld toe defects and usually the location with the highest residual stress, away from the main stress path. In the case where the maximum reinforcement is used, the average stress values at the weld/column-flange interface (using the actual area including the reinforcement) would be about 60 percent of the average axial stress in the pull plates. In the tables showing tee-joint test results, both the nominal stress in the pull plate and the weld stress at a point 1.6 mm (0.06 inch) above the surface of the column shape are given. To assess the significance of the effect of the weld reinforcement, many of the tests were conducted with the reinforcement ground off completely.

The reinforcement in these specimens was typical of field welds in moment connections. Even though no reinforcement of these welds is required, there is some reinforcement since a perfect 90° corner in the tee joint is difficult to achieve and undesirable. Pre-Northridge welds were typically over-reinforced. Paret (2000) reports on measurements of weld height and estimated that the cross-sectional thickness at the face of the column was often 133 percent of the thickness of the beam flange. These single groove welds with a backing bar are reinforced only on the crown. Good practice now involves removing the backing bar, backgouging, and applying a reinforcing fillet at the root. These welds, reinforced on both crown and root, have even greater area at the column face.

Various continuity plate details were tested to investigate the effect of these details on the failure modes and strength in the through-thickness direction. In one test specimen (no. 33), no continuity plates were used. In all other speci-

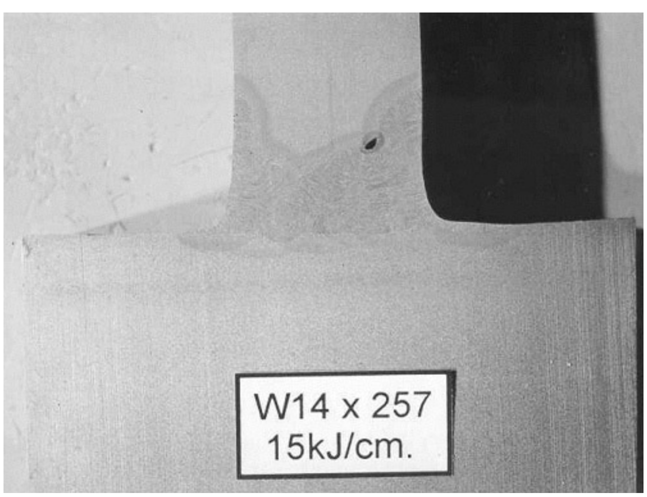
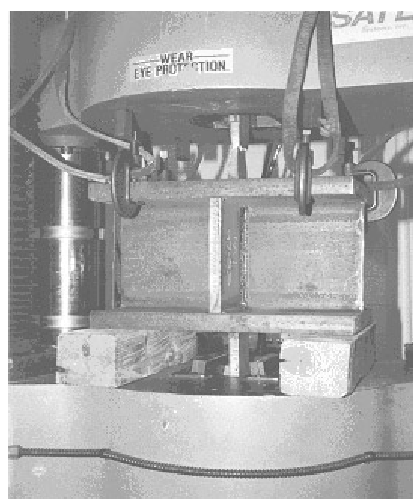


Fig. 4. Typical weld macrosection.

5a) specimen with groove-welded continuity plates and AISC recommended cutouts in the continuity plates (AISC, 1997)



5b) specimen with fillet welded continuity plates and inadequate cutouts in the continuity plates showing pull-plate failure in top pull plate

Fig. 5. Comparison of continuity plate details.

mens, there were continuity plates the same thickness as the pull plates (25 mm or 1 inch) as shown in Figure 3. In most cases, full penetration groove welds were used to join the continuity plates to the column shape, as shown in Figure 3 and in Figure 5a. In selected cases the continuity plates were attached by fillet welds as shown in Figure 5b.

The specimens with fillet welded continuity plates were also fabricated with inadequate cutouts, as shown in Figure 5b. Cutouts should be used in the corners of the flange/web junction to provide for termination of the web welds and the flange weld. In the basic specimen, the weld was terminated 55.6 mm (2.2 inches) from the inner flange face as shown in Figure 3 and in Figure 5a.

Tee-joint Test Results

Table 9 shows all the results from the 33 axially loaded test specimens with typical weld joints welded with normal heat input (less than 1.9 kJ/mm or 48 kJ/in.). All except four of these specimens broke in the pull plates at nominal pull-plate stress levels exceeding the MSYS of 690 MPa (100 ksi). These four were specimen 6, which had a poor continuity plate detail and fractured in the column web after yielding the pull plates, and specimens 10, 13, and 42 which fractured in the weld after yielding the pull plates. All of these tests also exceeded 690 MPa (100 ksi) nominal pull-plate stress.

A W14×176 column shape was tested with 102 mm (4-inch) wide pull-plates at a slow loading rate to investigate the effect of strain rate. Since this result at quasi-static strain rates is essentially the same as the result of the high-strain rate tests, it is concluded that strain rate did not have a significant effect on the failure mode or the through-thickness properties. Nevertheless, a majority of the tests were carried out at high strain rate since, if there were an effect, it would be expected to be deleterious on ductility.

To assess the significance of the effect of the weld reinforcement, five of these tests were conducted with the reinforcement ground flush. The through thickness stress on the weld area in these five specimens exceeded 688 MPa (100 ksi) in all cases. This level of through-thickness strength was much higher than expected, and is more than 50 percent greater than the performance requirement of 450 MPa (65 ksi) (which represents the maximum beam flange stress for a Gr. 50 beam). Because these specimens performed as well as the specimens with weld reinforcement, it can be concluded that weld reinforcement was not a significant factor in the outcome of these tee-joint tests.

All of the specimens tested in 1999 or earlier had sulfur levels less than 0.025 percent, as is typical for larger W shapes. However, the current A992 specification allows sulfur as high as 0.045 percent. Smaller shapes, such as the W10×68, often have relatively high sulfur levels toward the high end of the A992 limit. Additional tests were sponsored by Nucor-Yamato Steel to determine if sulfur levels near 0.045 percent could cause through-thickness failures. Specimens 43 through 48 are tests recently performed at the University of Minnesota on several shapes that were intentionally produced with very high sulfur levels. There were two tests each on two different heats of W14×257 (sulfur was 0.036 and 0.043 percent) and one heat of W10×68 (sulfur was 0.04 percent, see Table 6). As shown in Table 9, all these tests also resulted in pull-plate fracture and good ductility. High sulfur levels near the A992 specification limit apparently do not affect the through-thickness failure modes, strength, or ductility of column shapes. Therefore, it appears that the present limitation on sulfur in the A992 specification is adequate, since tighter restrictions would not be expected to improve the through-thickness failure modes, strength, or ductility of column shapes.

Specimen 2 was the Northwestern shape that had experienced a k-line fracture in previous connection testing and Specimen 42 was the shape from the L.A. County Building that had experienced column fractures away from the weld during the Northridge Earthquake. These specimens also had the highest carbon levels, 0.15 and 0.20 percent, respectively. These specimens performed as well as the others, therefore there is no indication that the carbon level or the history of previous fractures were related to poor through-thickness properties.

Two specimens were A913 Gr. 65 shapes that are labeled "flange only". In these tests, a single flange was cut from the WF shape, milled flat, and welded between the pull plates rather than the whole shape. The results from these tests were similar, so this variation in specimen configuration is also not thought to be significant.

The 305 mm (12-inch) Wide Pull-Plate Tests

As shown in Table 8, nine tee-joint tests were conducted on specimens with 305 mm (12-inch) wide pull-plates to investigate the effect of pull-plate width. Seven of these were ordinary joints welded with low heat input and the results for these are shown in Table 9. The other two were a specimen with no continuity plates at all, which is discussed later, and a specimen (no. 31) with a low-toughness E70T-4 weld root pass with the backing bar left in place.

As it became clear in this series of tests that there were going to be few if any through-thickness failures of the column flanges, various attempts were made to produce a through-thickness fracture, and specimen 31 was one of those. This was the only test that exhibited a pull-plate stress less than the 690 MPa (100 ksi) MSYS of the pull plate. Specimen 31 exhibited a pull plate stress of 654 MPa (95 ksi) and a weld stress of 632 MPa (92 ksi), 1.6 mm (0.06 inch) above the flange surface. The fracture originated in the weld root and propagated through the weld near the fusion line. This weld metal (E70T-4) is not recommended and this test result is not indicative of any properties of the column.

Load elongation curves for six of the specimens with continuity plates and ordinary joints are shown in Figure 6. (The test on the L.A. County Specimen (42) was done later and so is not shown in Figure 6, although the result was similar). Specimens 9, 11, 12, and 42 failed in the pull plates. Specimens 6, 10, and 13 had failures in the pull-plate-to-column flange welds. Examination of the fracture surfaces showed that these fractures initiated at slag inclusions (about 3 mm or 0.1 inch across) in the middle of the welds. The fractures propagated in the welds only. Although these specimens had weld failures, the peak pull-plate stress exceeded 760 MPa (110 ksi), and considerable elongation (more than 2 percent) took place before fracture, as shown in Figure 6.

Specimen 6 had continuity plates with fillet welds and with an inadequate cutout detail, as shown in Figure 5b. There were several distinct load drops in the stress vs. elongation curve for specimen 6 (see Figure 6) due to failure in the continuity plate-to-flange connection. After the continuity plate failures, the web began to tear and finally a brittle fracture occurred along the flange-web core region of the web (Figure 7). This specimen experienced a peak stress of 701 MPa (101 ksi) in the pull-plate and exhibited reasonable ductility (1.7 percent elongation) before finally fracturing. Although it can be seen that the performance was not as good as the specimens that were properly detailed, this specimen did meet the force and stress performance requirements.

Five other specimens with similar fillet welded continuity plates were tested with 102 mm (4-inch) wide pull plates. The results from these five specimens were essentially no different than the results of tests with groove welds and adequate cutouts. The force levels in these tests with the narrower pull plate were much less, however.

Specimen With No Continuity Plates

Specimen 33 also had a 305 mm (12-inch) wide pull plate but was fabricated without continuity plates. This specimen exhibited a divot-type failure of the column flange material, albeit at very high nominal stress levels (698 MPa or 101 ksi), as shown in Table 10. No unusual features such as lamellar defects were observed on the fracture surface of this specimen, as shown in Figure 8. The fracture initiated from small inclusions in the column flange not visible to the unaided eye.

This divot fracture looks similar to the divot fractures that occurred in the Northridge Earthquake (Kaufmann et al., 1997; Fisher, Dexter, and Kaufmann, 1997; Kaufmann and Fisher, 1995). However, those divot fractures in build-

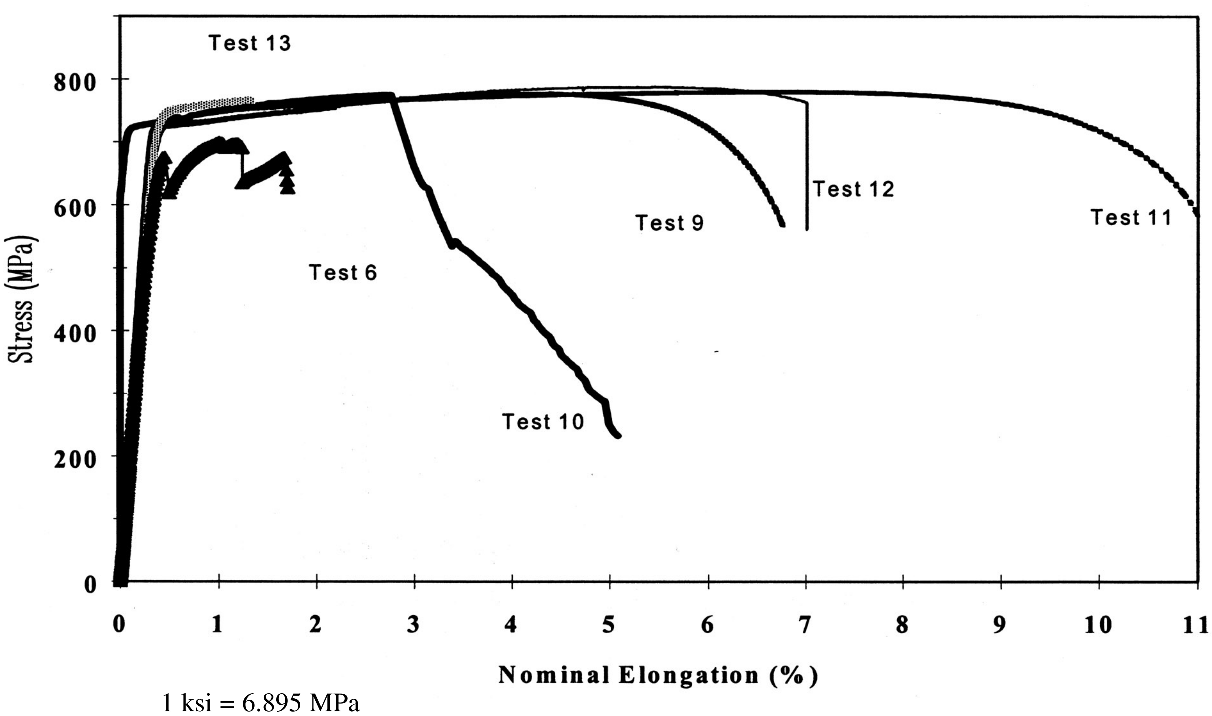


Fig. 6. Nominal pull-plate stress vs. elongation curves for six specimens with 305 mm (12 inch) pull plates.

ings during the earthquake originated in the weld at the tip of the backing bar notch, whereas the divot fracture shown in Figure 8 originated in the column flange material. It seems that the divot is a characteristic of propagation of the fractures in the column under through-thickness tension, i.e. it doesn't depend on the origin of the fractures.

The fact that this specimen triggered a through thickness failure and no other specimen did probably indicates that the lack of continuity plates aggravated the already high state of stress and strain in the column flange material below the interface of the beam flange to the column flange. It is not clear how significant this effect is however, since the strength levels attained in this test were not lower than normal. The authors do not believe this result should be taken as an indication that continuity plates are necessary. Further investigation of the need for continuity plates is presently underway at the University of Minnesota (Dexter, Hajjar, Prochnow, Graeser, Galambos, and Cotton, 2001; Prochnow, Ye, Dexter, Hajjar, and Cotton, 2000).

Other than the possible implications for continuity plates, there is not much significance to this through-thickness failure because it occurred at a nominal pull-plate stress level exceeding 690 MPa (100 ksi) and at a weld stress level of 676 MPa (98 ksi). These stress levels are well above the ultimate tensile strength of a Gr. 50 beam (450 MPa or 65 ksi), which we proposed as a performance requirement. This is well above the nominal stress level that could occur in a beam flange, even after strain hardening. The through-thickness properties should only have to be good enough to preclude through-thickness failure at the maximum possible applied stress levels.

The finding that a through-thickness failure occurred at this high stress level is not surprising. In fact, the finding that the flange material can resist failure with nominal through-thickness stress levels above the tensile strength (450 MPa or 65 ksi) is impressive. When the test specimen was designed with the 690 MPa (100 ksi) pull plates, it was envisioned that most if not all of these tests would result in a through-thickness failure before the pull plates were anywhere near yielding. As it turns out, the through thickness failure stress levels are apparently somewhat higher than 690 MPa (100 ksi).

In the case of this specimen 33 with no continuity plates, the additional stress from web yielding or flange bending was apparently enough to change the failure mode from fracture of the pull plates to a through-thickness fracture. Therefore, it is possible that the through-thickness fracture stress is just greater than 690 MPa (100 ksi).

If higher and higher through-thickness stress levels could be imparted to the other specimens in this test matrix which broke in the pull plates, and if their welds could hold up, a through-thickness failure of the column flange has to eventually occur. The specimen has to break somewhere. Consider what would have happened in the other specimens if they were designed with HY130 (890 MPa or 130 ksi MSYS) pull plates rather than the HSLA-100 (690 MPa or 100 ksi MSYS). If HY130 had been used, many other through-thickness failures may have occurred, albeit at higher stress values than attained in these tests. This failure in specimen 33 is an indication that the load in these experiments was close to the real through thickness strength in

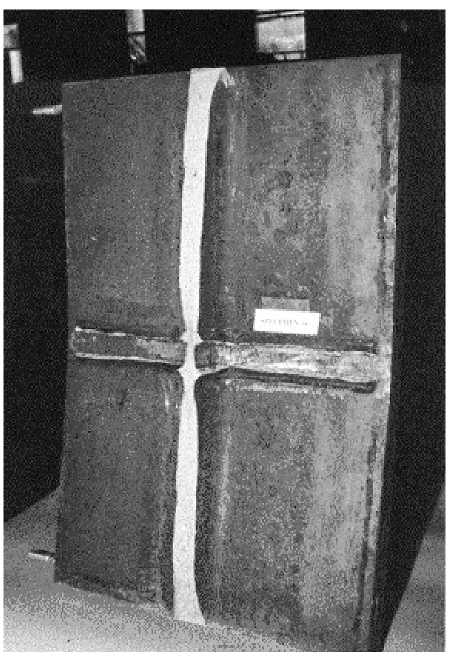


Fig. 7. Continuity plate and web failure for specimen 6. (Note, the fractured web is vertical in this photograph, and the remains of the fillet welds can be seen horizontally in the middle of the plate.)

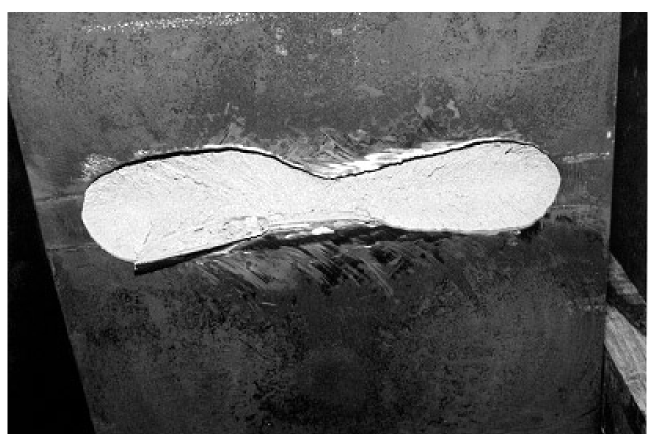


Fig. 8. View of divot-type fracture of column flange in specimen with no continuity plates. (Note, this view shows the brittle fracture on the surface of the column flange where the pull plate was. Chevron patterns are apparent on the fracture surface and point back toward the center of the flange, an indication of the origin of the fracture.)

these other specimens. This test and all the other tests establish a lower bound strength (690 MPa (100 ksi) pull plate stress) that is more than adequate.

Specimens Welded With High Heat Input

Table 10 also shows the results of five tests where the welds were made with high heat input (3.5 kJ/mm or 90 kJ/inch). All but one of these (specimen 32) were also ground to eliminate most of the weld reinforcement. (Specimen 32 has a low weld stress as a consequence of this reinforcement.) Heat input this high is outside the recommended range of heat input for this filler metal, so it was expected that the weld toughness may be degraded slightly.

The intent of the high heat input was to maximize the potential detrimental effect of the weld on the column shape. It was expected that the high heat input would create a greater heat- affected zone (HAZ) width and possibly more brittle HAZ microstructures. Most of these high-heat input specimens failed because of a fracture in the weld. Examination of the fracture surface showed these fractures were clearly associated with slag inclusions about 10 mm (0.4 inches) across in the weld metal and not associated with the column shape or even the HAZ of the weld to the column shape. The pull plate stress exceeded 737 MPa (107 ksi) for all of these high heat input welds and there was at least 1.5 percent elongation, therefore the performance of these welds was adequate. It was concluded that the column shape did not show any adverse effects from the high-heat-input welding.

Specimens With Details Designed to Induce Bending and Prying

Local bending in the beam flange has been identified as a potential contributing factor to connection fractures. In order to investigate the effect of bending stress on the potential for through-thickness fractures, seven tests were conducted specifically to induce bending and plastic rotations on the pull-plates and the weld. These specimens have pull-plates 204 mm (8 inches) wide with another plate lapped onto the pull plate giving 25 mm (1 inch) of eccentricity. One of these even had the backing bar left in place on the tension side of the weld.

In these plates, the bending moment exceeded the gross-section plastic moment and the load path essentially straightened out. The pull plates underwent large plastic rotations but the specimens did not fail. These tests show that local bending stress in the beam flange is no more likely to result in a divot fracture or other through-thickness fracture than tensile stress.

CONCLUSIONS

Test specimens were designed with high-strength pull plates and high quality welds that could deliver through-thickness stresses into the column in excess of 690 MPa (100 ksi). Forty-seven tests were performed including 33 specimens with ordinary welds, five specimens welded at high heat input, one specimen without continuity plates, and seven specimens with eccentricity intended to induce bending and prying. One specimen was welded with low-toughness weld metal in the root pass and the backing bar was left in place. All of these tee-tests showed that the through-thickness tensile strength of constrained column flange material in a beam-to-column joint exceeds the ultimate tensile strength of Gr. 50 or Gr. 65 beam flanges.

Because the result was almost uniform in all these tests, the results do not appear to be dependent on any of the variables, including: weld reinforcement, width of the pull plates, bending, heat-input, material grade and chemistry, or the strain rate.

Sulfur levels as high as 0.043 percent, close to the limit of 0.045 percent, had no effect on the outcome of the tests. Therefore, specifications that restrict sulfur more than the A992 limit are not required.

A specimen was tested without continuity plates, which resulted in a divot-type through-thickness failure of the column flange material, albeit at very high nominal stress levels. A through-thickness fracture at this high force and stress level should not be a concern, since these high force and stress levels would never exist in a Gr. 50 or Gr. 65 beam flange. Although this result shows that the lack of continuity plates probably had some detrimental effect, the result should not be taken as evidence that continuity plates are needed, since the force levels were much higher than would be delivered from typical beam shapes. No unusual features such as lamellar defects were observed on the fracture surface of this specimen.

The experiments show that the column material adjacent to the beam flange weld behaves in accordance with the Von-Mises yield criterion, i.e. that the column flange material will not yield significantly in the through-thickness direction. There is no significant demand for ductility in the through-thickness direction because yielding is limited.

The through-thickness failure of column shapes is very unlikely in moment connections or other types of T-joints. It is recommended that the through-thickness strength does not need to be explicitly checked in the design of welded beam-to-column connections.

The through-thickness strength is not a rationale for having to use a higher yield strength material for the columns than for the beams. Rather, the total moment and shear capacity of the panel zone should be compared to the capacity of the beams, and this could lead to the requirement for a high-strength Gr. 65 column.

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In 1999, tests sponsored by TradeARBED on their A913 Gr. 65 shapes were performed at Lehigh. The tests on the high-sulfur shapes were sponsored by Nucor-Yamato Steel and were performed in 2000 at the University of Minnesota by Sara Prochnow under the direction of Paul Bergson.

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Table 1. Example of Longitudinal and Through-Thickness Tensile Properties of an ASTM A572 Grade 50 W14×176 Column Shape				
Label	**Type of Specimen**	**Yield Strength* (MPa)**	**Tensile Strength* (MPa)**	**Area Reduction (percent)**
L-A1	Longitudinal	374	496	77
L-A2	Longitudinal	359	477	77
L-A3	Longitudinal	349	494	73
T-A1	Through-Thickness	391	511	51
T-A2	Through-Thickness	353	483	13
T-A3	Through-Thickness	370	500	44

*1 ksi = 6.895 MPa

Table 2. Description of Different Heats of Steel from which Test Specimens Were Made				
Mill	**Year**	**Specification**	**Shape**	**Comments**
Unknown	Unknown	A572 Gr 50	W14x455	L.A. County Building
Northwestern	1995	A572 Gr 50	W14x257	had k-line fracture
Nucor	1997	A572 Gr 50	W14x176	
Nucor	1997	A572 Gr 50	W14x257	
TradeARBED	1997	A913/A572 Gr 50	W14x176	
TradeARBED	1997	A913/A572 Gr 50	W14x257	
Corus	1997	A572 Gr 50	W14x176	
Corus	1997	A572 Gr 50	W14x257	
Corus	1997	A572 Gr 50	W14x455	
TradeARBED	1999	A913 Gr 65	W14x257	
TradeARBED	1999	A913 Gr 65	W14x605	
Nucor	2000	A992	W14x257	high sulfur
Nucor	2000	A992	W14x257	high sulfur
Nucor	2000	A992	W10x68	high sulfur

Table 3. Summary of Yield Strength in the Flange/Web Core Region for Grade 50 Shapes		
Shape Size (ASTM A572 unless noted)	**Longitudinal, MPa* (Reduction in area)**	**Through-thickness, MPa* (Reduction in area)**
W14x257	377 (63%)	338 (33%)
W14x176	359 (77%)	353 (13%)
W14x257	350 (75%)	338 (28%)
W14x176	370 (75%)	NA
W14x257	366 (75%)	386 (75%)
W14x455	354 (71%)	359 (63%)
W14x176 (A913 Gr 50)	308 (74%)	338 (26%)
W14x257 (A913 Gr 50)	305 (72%)	303 (47%)

*1 ksi = 6.895 MPa

Table 4. Chemical Compositions of Typical Column Shapes, Percent											
Grade	**A572 Gr50**							**A913 Gr. 50**		**A913 Gr. 65**	
W 14 X	**455***	**257**	**176**	**257**	**176**	**257**	**455**	**176**	**257**	**257**	**605**
C	0.20	0.15	0.08	0.05	0.08	0.08	0.09	0.08	0.08	0.08	0.09
Mn	1.15	1.03	1.38	1.33	1.45	1.45	1.42	1.09	1.09	1.37	1.37
Si	0.24	0.32	0.24	0.22	0.31	0.31	0.30	0.19	0.19	0.25	0.17
P	0.043	0.012	0.01	0.02	0.017	0.017	0.018	0.025	0.025	0.022	0.016
S	0.004	0.022	0.02	0.02	0.003	0.003	0.004	0.021	0.021	0.019	0.008
Cu	0.15	0.16	0.32	0.35	0.33	0.33	0.29	0.28	0.28	0.20	0.22
Ni	0.05	0.08	0.11	0.12	0.23	0.23	0.22	0.16	0.16	0.11	0.14
Cr	0.029	0.1	0.08	0.06	0.025	0.025	0.029	0.19	0.19	0.15	0.18
Mo	0.024	0.019	0.03	0.03	0.003	0.003	0.003	0.043	0.043	0.028	0.027
V	0.054	0.28	0.04	0.05	0.12	0.12	0.12	0.001	0.001	0.043	0.002
Nb			0.001	0.005	0.004	0.004	0.004	0.006	0.006	0.023	0.034
Al	0.018									0.002	0.025
Ti										0.003	0.001
Sn			0.01	0.01							
B≤			5E-04	5E-04							
N										0.012	0.008
CE	0.43	0.39	0.37	0.33	0.39	0.39	0.39	0.34	0.34	0.37	0.38

*Column shape from LA County Building

Table 5. Tensile Properties of A913 Grade 65 Shapes			
Material	**Type of Specimen**	**Yield Strength (MPa)***	**Tensile Strength (MPa)***
W14x257Gr. 65	Longitudinal	544	667
W14x605Gr. 65	Longitudinal	466	591

*1 ksi = 6.895 MPa

Table 6: Chemical Compositions of A992 Column Shapes With Intentionally High Sulfur						
Shape	**W14x257 (a)**		**W14x257 (b)**		**W10x68**	
Laboratory*	MTR	Ind	MTR	Ind	MTR	Ind
C	0.05	0.07	0.07	0.09	0.08	0.07
Mn	1.23	1.23	1.38	1.38	1.13	1.17
Si	0.22	0.19	0.24	0.23	0.23	0.25
P	0.020	0.020	0.014	0.014	0.014	0.016
S	0.043	0.054	0.036	0.036	0.040	0.032
Cu	0.37	0.33	0.28	0.25	0.35	0.28
Ni	0.14	0.14	0.10	0.11	0.08	0.10
Cr	0.15	0.15	0.09	0.10	0.07	0.08
Mo	0.03	0.03	0.03	0.02	0.02	0.02
V	0.05		0.05		0.00	
Nb	0.000		0.000		0.018	
Sn	0.01		0.01		0.01	
B	<0.005		<0.005		<0.005	
CE	*0.33*	*0.34*	*0.36*	*0.37*	*0.32*	*0.31*

* *MTR* is the mill test report; *Ind* is an independent test

Table 7. Tensile Test Results for A992 Column Shapes with Intentionally High Sulfur				
Material	**Type of Specimen**	**Yield Strength (MPa)***	**Tensile Strength (MPa)***	**Reduction in Area (percent)**
W14x257 (a)	**Longitudinal**	**349**	**487**	**61**
	Through-thickness	309	480	26.7
	Through-thickness	307	480	22.1
	Through-thickness	300	475	26.2
W14x257 (b)	**Longitudinal**	**386**	**520**	**55**
	Through-thickness	325	499	21.0
	Through-thickness	324	473	15.2
	Through-thickness	323	492	26.1
W10x68	**Longitudinal**	**355**	**527**	**68**
	Through-thickness	NA	558	34.3
	Through-thickness	375	529	35.9
	Through-thickness	536	563	37.8

*1 ksi = 6.895 MPa

Table 8. Matrix for Pull-Plate Tests (specimen number and code shown for each test)				
Stain rate, width: / **Shape**	**High-rate 102 mm**	**Slow 102 mm**	**Slow 305 mm**	**Bending 204 mm**
W14x605	35HS 37HS,F 39HS,HH 41HS,HH,F			
W14x455	14 15		11 42	19 23 24
W14x257	2CP 3CP 7CP 8CP 17 18 28 29 32HH 34HS 36HS,F 38HS,HH 40HS,HH,F 45CP,S 46CP,S 47CP,S 48CP,S		6 9 13 31* 33CP	22 26
W14x176	4CP 16 20 21 30	5	10 12	25 27
W10x68	43CP,S 44CP,S			

1 inch = 25.4 mm
CP = special continuity plate detail
F = flange only specimen
HH = high heat-input welds
HS = high-strength Grade 65
S = high-sulfur shapes
*E70T-4 root pass with backing bar left in place

Table 9. Summary of Results of Typical Tension Tee-Tests						
Test #	**Column Size**	**Peak load (kN)**	**Pull-plate Stress (MPa)**	**Weld stress** (MPa)**	**Failure Mode**	**Type of "T-joint" test**
14	W14x455	2077	805	452	Pull plate	102 mm pull-plate High strain rate
15	W14x455	2062	799	450	Pull plate	
16	W14x176	2033	788	462	Pull plate	
17	W14x257	2059	798	572	Pull plate	
18	W14x257	2059	798	572	Pull plate	
20	W14x176	2008	778	427	Pull plate	
21	W14x176	2063	800	413	Pull plate	
28	W14x257	2056	797	381	Pull plate	
29	W14x257	2070	802	422	Pull plate	
30	W14x176	2061	799	412	Pull plate	
34*	W14x257	2098	813	699	Pull plate	A913 Grade 65 High strain rate
35*	W14xs605	2070	796	690	Pull plate	
36*	W14xs257	2066	801	689	Pull plate	A913 Grade 65 High rate, flange only
37*	W14x605	2065	800	688	Pull plate	
2+	W14x257	1971	764	580	Pull plate	102 mm pull-plate High strain rate Fillet welded Continuity plates
3+	W14x257	2062	799	503	Pull plate	
4+	W14x176	2075	804	506	Pull plate	
7+	W14x257	2064	800	529	Pull plate	
8+	W14x257	2064	800	558	Pull plate	
43	W10x68	2100	814	442	Pull plate	
44	W10x68	2113	819	485	Pull plate	
45	W14x257(a)	2126	824	478	Pull plate	
46	W14x257(b)	2122	822	467	Pull plate	
47	W14x257(a)	2126	837	448	Pull plate	
48	W14x257(b)	2108	817	495	Pull plate	
5+	W14x176	2039	790	510	Pull plate	102 mm pull-plate Quasi-static strain rate
6+	W14x257	5427	701	493	Weld	305 mm pull-plate Quasi-static strain rate
9	W14x257	6014	777	542	Pull plate	
10	W14x176	5961	770	485	Weld	
11	W14x455	6032	779	574	Pull plate	
12	W14x176	6098	788	462	Pull plate	
13	W14x257	5916	764	450	Weld	
42*%	W14x455	5871	762	733	Pull plate	

1 kip = 4.448 kN, 1 ksi = 6.895 MPa, 1 inch = 25.4 mm
* Weld had no reinforcement
**This weld area is measured on a plane 1.6 mm above the column surface, and weld stress is the peak load divided by this weld area.
+ This specimen had fillet welded continuity plates with inadequate cutout detail
% L.A. County Building
(a) the higher sulfur (0.043 percent) heat, see Table 6
(b) the lower sulfur (0.036 percent) heat, see Table 6

Table 10. Results from Specimens Designed to Increase the Potential for Brittle Fracture Including High-Heat Input Welds and a Specimen with No Continuity Plates						
Test #	**Column Size**	**Peak load (MN)**	**Pull-plate stress (MPa)**	**Weld stress** (MPa)**	**Failure Mode**	**Type of "T-joint" test**
38	W14x257	2.017	782	672	Weld	Grade 65 102 mm wide pull-plate w/high heat input weld
39	W14x605	2.077	805	692	Pull plate	
40	W14x257	1.903	737	634	Weld	Grade 65 flange only 102 mm wide pull-plate w/high heat input weld
41	W14x605	1.989	771	663	Weld	
32*	W14x257	1.966	762	393*	Weld	Grade 50 102 mm wide pull-plate w/high heat input weld
33	W14x257	5.405	698	676	Column flange through thickness	305 mm wide pull-plate w/out continuity plates

1 kip = 4.448 kN, 1 ksi = 6.895 MPa, 1 inch = 25.4 mm
* Weld had significant reinforcement, all other had no reinforcement
**This weld area is measured on a plane 1.6 mm above the column surface, and weld stress is the peak load divided by this weld area.