Experimental Verification of a New Single Plate Shear Connection Design Model

Kirsten A. Baldwin Metzger

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Thomas M. Murray, Chair
W. Samuel Easterling
Elisa D. Sotelino

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by

Kirsten A. Baldwin Metzger

Committee Chairman: Thomas M. Murray
Civil Engineering

(ABSTRACT)

Single plate shear connections are designed to have sufficient strength to resist the shear force and moment transferred from the beam. The connections must also provide sufficient ductility to allow the beam end to rotate freely. In the United States the current recommended design method is found in the AISC 13th Edition Steel Construction Manual (2005b). The limited experimental work which led to the current method necessitated additional single plate shear connection investigations.

This paper summarizes the results and analysis of eight full scale single plate shear connections tested at Virginia Polytechnic Institute and State University. The test setup consisted of a test beam attached to a test column with a single plate shear connection at one end and supported by a roller at the other end. The single plate was welded to the column flange and bolted to the beam web. Load was applied to the test beam at third points until failure of the connection or test beam.

The current design method used in the United States was examined with respect to the connection tests performed. In particular, the ultimate shear strength and the rotational capacity were investigated. Suggestions are made regarding changes to the method and further research.

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CHAPTER 1
INTRODUCTION

1.1 Overview

The single plate shear connection is currently one of the most common shear connections in the United States. These connections are also referred to as shear tabs, shear bars, web side plates, or fin plates. The connection consists of a single steel plate welded to the supporting member and bolted to the supported member. Its advantage lies in the simplicity of fabrication and erection. All welding and drilling can be done in the shop by a steel fabricator.

These connections are often idealized as pinned connections carrying only shear forces transferred by the supported beam. The pin allows the beam end to freely rotate. However, research has shown that shear tab connections are typically stiffer than other shear connections. This stiffness causes them to carry some moment which makes the study of their ductility and rotational capacity important.

Shear tab connections can be used in a variety of ways. They can connect a beam to a column web, a column flange, or a girder web. Shear tabs can also have extended plates, where the distance between the weld line and the bolt line is increased, to eliminate the need for costly coping of the beam flange in beam-to-column web or beam-to-girder web connections. Figure 1.1 illustrates several common uses of these connections.

1.2 Scope of Research

The goal of the research in this study was to examine the performance of connections designed in accordance with the procedure published in the AISC 13th Edition Steel Construction Manual (2005b). This procedure is a significant deviation from the method presented in the previous Edition (AISC, 2001) with regards to dimensional flexibility and eccentricity calculation. Single plate shear connections are divided into two categories. Those meeting several dimensional limitations are classified as conventional configuration. All others are classified as extended configuration.

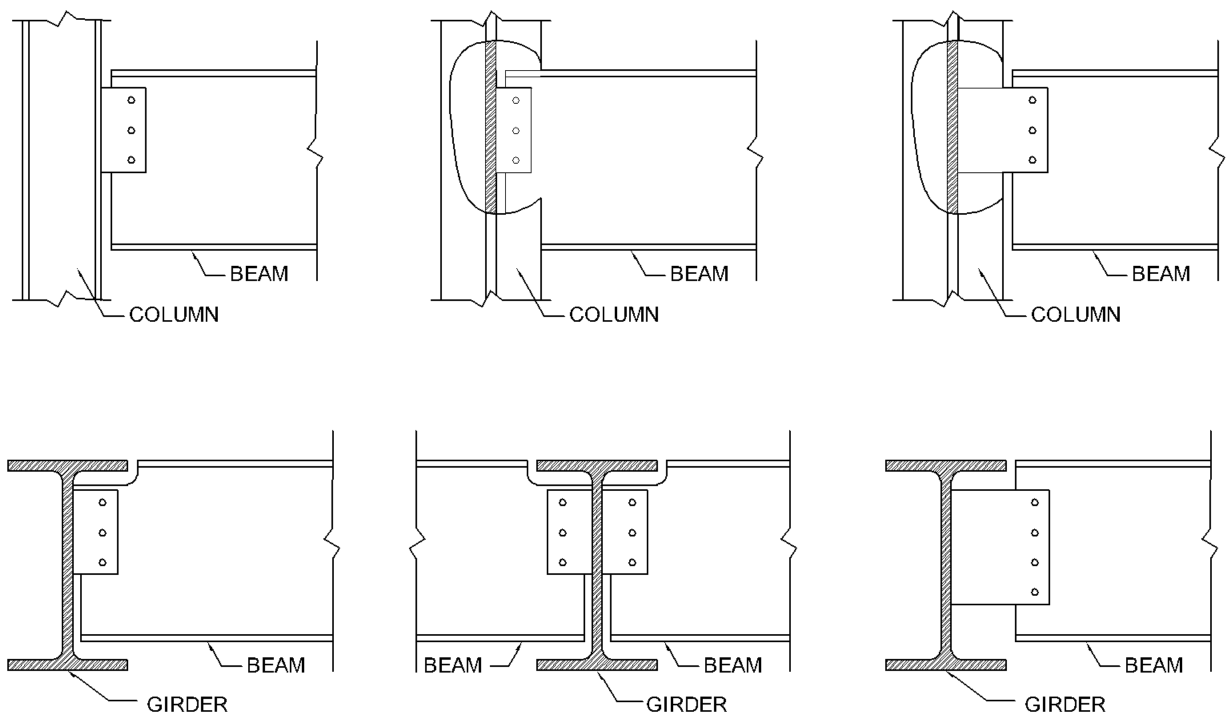

Figure 1.1 Typical Single Plate Shear Connections

The research program consisted of eight full scale experimental tests. Four shear tabs that met the conventional configuration requirements and four shear tabs that fell in the extended configuration category were tested. The test series included variations in the number of bolts, the number of vertical columns of bolts, and the connection length from the weld line to the center of the bolt group. Each test specimen was loaded to failure.

Experimental results were analyzed to determine the ultimate shear capacity, failure mode, and rotational ductility of each connection. These experimental values were compared with values predicted by the AISC Manual (2005b) design procedure. Recommendations were then made regarding the current design method.

1.3 Research Outline

The following is a brief outline of the topics covered in this study:

- In Chapter 2 a review of previous research work is presented. A chronological progression of work is discussed as it relates to the development of shear tab design models worldwide. The AISC Manual (2005b) updates are described, as well. Then a summary of the significant methods used overseas is presented along with a design example illustrating the differences between these methods. Lastly, previous experimental tests are compared to the predicted strengths determined by the new AISC design procedure (2005b).

- In Chapter 3 the experimental test program is described in detail. The design methodology is presented for the connection, test beam, and test column. The connection design drawings are included in Appendix D. Descriptions of the instrumentation and testing procedure used for each connection test are also included.

- In Chapter 4 the experimental results are presented. The results for each test are presented in the form of graphs illustrating the shear versus rotation behavior of the connection during testing. Failure modes, rotational ductility, and ultimate strength are also discussed.

- In Chapter 5 the experimental results are analyzed with respect to their ultimate strength, failure modes, and rotational ductility. These results are then compared to the behavior predicted by the new AISC design procedure (2005b).

- In Chapter 6 a summary of the significant findings from the test program and recommendations regarding the current design method are given. Suggestions for future research are also listed.

CHAPTER 2
LITERATURE REVIEW AND DISCUSSION

2.1 Overview

The single plate shear connection is designed to resist the end shear of a simply supported beam. The connection must also provide sufficient rotational ductility to allow the beam to reach its plastic moment capacity. The rotational ductility is considered carefully because this connection has a larger capacity to resist end moments than other shear connections due to greater stiffness. Research has shown that connection rotation sheds some of this end moment, measured in terms of connection eccentricity, to the mid-span of the beam. The careful balance required between strength and ductility has led to numerous analytical and experimental studies. A summary of single plate shear connection research is presented in this chapter.

2.2 Previous Research

Early work by Lipson (1968) examined three types of shear connections including single plate shear connections, which the researcher referred to as welded-bolted plates. The goals of the research were to determine the behavior of the connection under working loads, to examine the rotational capacity of the connection, to establish a consistent factor of safety, and to determine if the connection should be classified as flexible or rigid. Lipson identified three failure modes: tensile yielding of the plate, weld rupture, and vertical bolt tearout of the bottom bolt. Significant deformation of the bolt holes was also observed. The research found that the amount of end moment transferred to the supporting member was dependent upon five factors:

(1) Number, size, and configuration of bolt pattern
(2) Thickness of the plate and/or beam web
(3) Beam span/depth ratio
(4) Beam loading pattern
(5) Flexibility of supporting member

The connection was determined to be partially restrained with characteristics of both flexible and rigid connections.

Caccavale (1975) at the University of Arizona attempted to simulate the experimental results of Lipson using finite element analysis. Several single shear tests were performed on individual bolts to determine the load vs. deformation response. Then finite element models were created to simulate the Lipson's test setup. These models gave results consistent with the previous experimental work. Caccavale also noted that shear tab ductility is provided by distortion of the bolt holes, providing the bolts are of the necessary strength.

Richard et al. (1980) created a series of finite element models of single plate shear connections and a series of experimental bolt shear tests. From these models, a beam line was developed. The beam line is an equation defining the relationship between the end moment and the end rotation of a single span beam subjected to a uniformly distributed load. The beam line utilized the linear beam action and the nonlinear connection behavior to find the moment – rotation relationship for shear tab connections. Five full scale beam tests were performed to establish the validity of the beam line.

Richard et al. proposed a design procedure based on a connection to a rigid support with standard bolt holes. Ductility was controlled by limiting connection plate thickness to ensure plate yielding prior to any brittle limit states. The bolts were designed with an eccentricity, e, taken from

$$\frac{e}{h} = \left(\frac{e}{h}\right)_{ref} \times \left(\frac{n}{N}\right) \times \left(\frac{S_{ref}}{S}\right)^{0.4} \tag{2.1}$$

where

$(e/h)_{ref}$ = 0.06 L/d – 0.15, for L/d ≥ 6

= 0.035 L/d, for L/d < 6

n = number of bolts

N = 5 for 3/4 in., 7/8 in. bolts, and 7 for 1 in. bolts

S_{ref} = 100 for 3/4 in. bolts, 175 for 7/8 in. bolts, and 450 for 1 in. bolts

S = section modulus of beam

The welds in the procedure were also designed to resist moment, determined from

$$M = V \times (e + a) \tag{2.2}$$

where

V = beam shear

a = distance from the bolt line to the weld line

Young and Disque (1981) developed design aids to be used with Richard's procedure.

In a later study, Richard et al. (1982) examined the use of A307 bolts in short slotted holes. This connection was desirable because ductility could be predominantly provided by bolt movement in the slotted holes thus avoiding large bolt diameter-to-plate thickness ratios. However, in a discussion following Richard's work, Becker and Richard (1985) state that standard holes are more useful than short slotted holes for alignment purposes. Both the AISC 3rd Edition (2001) and the AISC 13th Edition (2005) design methods prohibit the use of A307 bolts; therefore, the design procedure developed in the study will not be discussed further.

Hormby et al. (1984) performed another set of experimental tests utilizing off-axis bolt groups, Grade 50 steel beams, and composite beams. An off-axis bolt group is a bolt group whose center of gravity does not line up with the neutral axis of the test beam. All single plate connections were made to a rigid supporting member. Four tests of off-axis bolt groups using A325 bolts were completed along with similar concentric bolt groups. The connections with off-axis bolt groups had eccentricities $\pm$9% of the concentric connection eccentricity. The tests also showed that the center of rotation of the connection was at the center of the bolt group.

In the tests using ASTM A572 Grade 50 steel beams, it was recommended that the eccentricity be modified as follows.

$$e_{50} = e_{36} \times \left(\frac{36}{50}\right) \tag{2.3}$$

where

e_{36} = eccentricity of the connection calculated in Equation 2.2

This modified eccentricity can be used with the design procedure outlined previously by Richard et al. (1980).

Analytical studies were also used to develop a modified procedure to be used with composite beam construction. This procedure was then tested with ten full scale specimens. It was concluded that composite construction results in small differences in connection moment, and the original beam line developed by Richard et al. (1980) may be used.

Flatt (1985) examined the behavior of single plate shear connections subjected to cyclic loading. The research was limited to 2 and 3 bolt connections. A beam with a connection to a rigid support on each end was loaded with a concentrated load at mid-span. The maximum beam end rotation achieved was 0.0021 radians. Even though tensile cracks occurred in the plate during early cycles of the testing, the single plate connection could withstand a significant number of load cycles after the cracking occured.

Patrick et al. (1986) conducted full scale experimental tests of single plate shear connections for the Australian Welding Research Association. All parameters were kept in accordance with the Australian Institute of Steel Construction specifications, and only the bolt group configuration was varied between tests. Four bolt groups were tested; 6 and 9 bolts in a single vertical column and 12 and 18 bolts in double vertical columns. The beams were connected to a rigid support and loaded with a single concentrated load. In addition, to simulate the shear-rotation relationship of a uniformly loaded beam, the far end of the beam was lowered as the loading proceeded. The connections were loaded to failure, and a rotation of approximately 0.01 radians was achieved. The single column bolt groups both failed due to bolt shear. The double column bolt groups failed due to plate shear along the inner column of bolts.

Stiemer et al. (1986) noted that all prior shear tab research tested connections attached to rigid supports. This group of researchers conducted four full scale tests of 2 and 3 bolt connections to one side of a girder web at the girder mid-span using A36 plate and beam material. Two of the connections were perpendicular to the girder, and two were skewed connections. The far ends of the beams were simply supported, and the beam was loaded with a single actuator. The girder ends were connected with moment end plates. The goal of the research was to study the loading effects on the supporting girders.

The research found that the girder provides a flexible support which behaves differently than the rigid supports in previous tests. A set of geometric ratios between the girder parameters and the plate parameters which would aid design of one sided connections to girders mid-span was developed. It was also found that the maximum shear force that could be resisted by the connections had to be less than 30% of the ultimate shear strength of the girder. However, this research was limited in scope as it only drew conclusions regarding one sided shear tab connections at mid-span of a girder.

Aggawal (1988) concluded from previous research that the single plate shear connection might be treated as a semi-rigid connection. A series of ten tests, designed in accordance with the Australian Institute of Steel Construction recommendations, was conducted to establish moment-rotation curves for connections with two bolts each in two vertical columns. The researcher concluded that eccentricity did exist in the connection because the connections failed at loads lower than their pure shear capacity. Beam end rotations for statically loaded tests varied from 0.025 to 0.065 radians at failure. Failure in these tests was indicated by large slips in the connection which prevented the taking of additional measurements. It was observed that the connection ductility depended on bolt slip within the bolt hole and bolt hole deformation.

In the late 1980s, Astaneh (1989c) conducted research at the University of California, Berkeley that led to the development of the AISC 2nd Edition Manual (AISC, 1993) Load and Resistance Factor Design method for single plate shear connections. Astaneh based the research on his earlier work (Astaneh, 1989b) examining the demand and supply of ductility in steel shear connections. Astaneh proposed a modified beam line, which differs from Richard's beam line (Richard et al., 1980) because it takes into account the inelastic properties of both the beam and the connection, thus allowing the beam line to be used with ultimate strength and factored load design methods.

The moment-rotation curves developed by Astaneh show three stages in the behavior of a simple shear connection. As load is initially applied to the beam, the connection demonstrates some stiffness and the ability to resist moment. This is the elastic region of the beam line. The inelastic or yield plateau portion of the beam line begins once the load is great enough to cause yielding in the connection. At this point, the beam is allowed to rotate and some of the connection moment is passed on to the supporting member. Finally, as the beam continues to rotate, the connection enters the strain hardening region in which the connection moment increases slightly.

Astaneh also developed a series of curves for beam end rotation versus beam end shear for a uniformly loaded beam. These curves were based on the beam cross section, the L/d ratio of the beam, and the grade of steel used. In general he found that there were three stages of behavior as shown in Figure 2.1. Segment AB represents the elastic connection behavior. At point B the beam yields initiating the inelastic portion of the curve. Astaneh's research has shown that for single plate shear connections this point typically corresponds to a rotation of 0.02 radians. At point C on the curve the beam develops its full plastic moment capacity, and the strain hardening region begins and continues until failure occurs. Point C typically occurs around 0.03 radians in single plate shear connections.

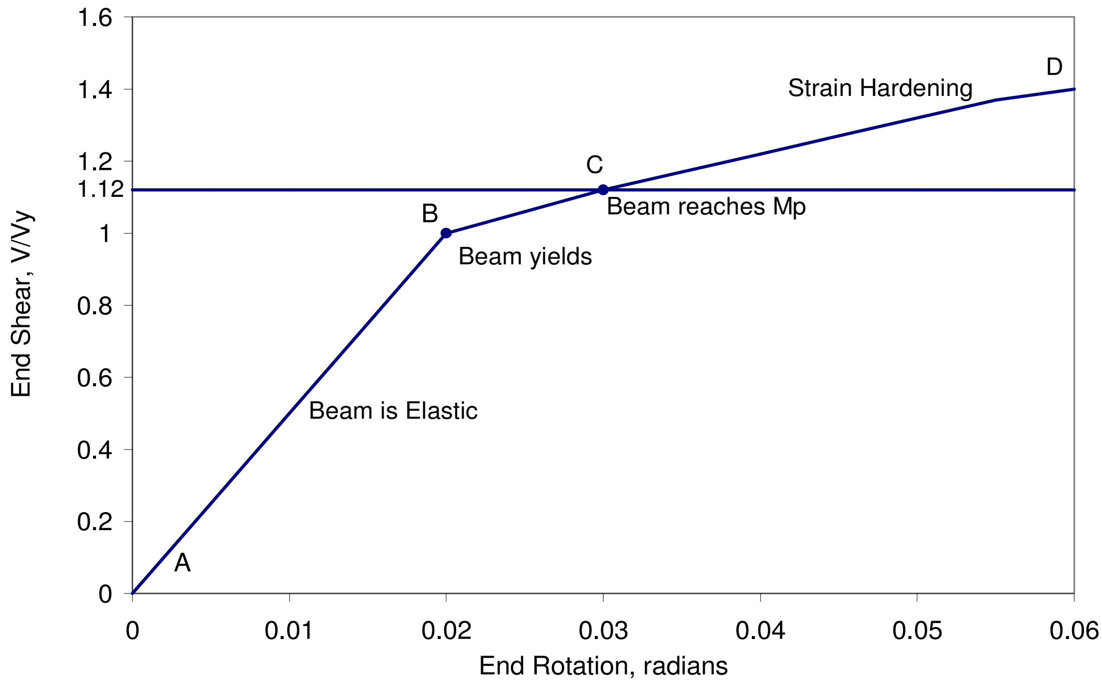

Figure 2.1 Astaneh's proposed shear-rotation for simple beams (Astaneh, 1989b)

This load-rotation path was the basis for the load path used by Astaneh for five full scale tests (Astaneh, 1989c). The specifications for these five tests are shown in Table 2.1, and the corresponding test results are shown in Table 2.2. All tests had standard holes and an a-distance of 2.75 in. The a-distance is defined as the distance from the weld line to the bolt line. Cantilever beams were loaded by two actuators; one near the connection to control the connection shear and one further away to control the connection rotation. The beams were loaded to rotations of 0.026 to 0.061 radians at the point of maximum shear.

Table 2.1

Astaneh (1989c) Test Specifications

Test	Bolts	Bolt Diameter (in.)	Bolt Type	Hole Type	Plate Dimensions[1] (in.)	Edge Distance (in.)	Plate Material[2]	Weld Size (in.)	Test Beam	Beam Material
1	3	3/4	A325-N	Standard	3/8x4-1/4x9	1-1/2	A36	1/4	W18x55	A36
2	5	3/4	A325-N	Standard	3/8x4-1/4x15	1-1/2	A36	1/4	W18x55	A36
3	7	3/4	A325-N	Standard	3/8x4-1/4x21	1-1/2	A36	1/4	W24x84	A36
4	3	3/4	A490-N	Standard	3/8x3-7/8x8-1/4	1-1/8	A36	7/32	W18x55	Gr. 50
5	5	3/4	A490-N	Standard	3/8x3-7/8x14-1/4	1-1/8	A36	7/32	W18x55	Gr. 50

[1] The a-distance was 2.75 in.

[2] The yield strength of the plate was experimentally found to be 35.5 ksi with an ultimate strength of 61 ksi

Table 2.2

Astaneh (1989c) Test Results

Test	Failure Mode	Shear Force (kips)	Beam End Rotation (radians)	Moment at Bolt Line (kip-in.)	Moment at Weld Line (kip-in.)
1	Bolts Fractured	94	0.056	20	279
2	Bolts Fractured	137	0.054	314	691
3	Bolts Fractured	160	0.026	306	745
4	Welds and Bolts Fractured	79	0.061	-47	170
5	Bolts Fractured	130	0.053	273	631

Based on these experiments five limit states were identified:

(1) Shear failure of bolts
(2) Yielding of gross area of plate
(3) Fracture of net area of plate
(4) Fracture of welds
(5) Bearing failure of beam web or plate.

Astaneh developed a strength based design procedure taking into account the required shear capacity as well as the necessary connection rotation to allow the beam to reach its full plastic moment capacity. The researcher achieved this rotation by ensuring that the ductile yielding of the plate would be reached prior to brittle failure, which is facilitated by limiting the plate thickness to less than $d_b/2 + 1/16$ in.

Using experimental data, Astaneh developed the following empirical equation for the eccentricity of a connection to a rigid support.

$$e_b = (n-1)\times(1.0) - a \qquad (2.4)$$

where,

a = distance between the bolt line and the weld line,

e_b = distance from the point of inflection to the bolt line.

For a flexible support the following equation was developed, though no experimental tests were completed in the study.

$$e_b = Max\left|\begin{matrix}(n-1)\times(1.0) - a \\ a\end{matrix}\right. \qquad (2.5)$$

The plate to column flange weld was designed for the following eccentricity;

$$e_w = Max\left|\begin{matrix}(n)\times(1.0) \\ a\end{matrix}\right. \qquad (2.6)$$

need not exceed 3/4 of the plate thickness.

Astaneh limited the recommended weld size to 3/4 of the plate thickness to ensure that the plate would yield prior to weld yielding. This limit was determined by ensuring that the Moment-Shear interaction curve for the plate was entirely inside the Moment-Shear interaction curve for the weld. The limit is based on a weld yield strength of one half times the tensile strength of the weld material and a plate yield strength of 36 ksi. The derivation for the plate thickness limit can be found in Astaneh's work (1989a).

A portion of Astaneh's design procedure came from a series of tests performed by Astaneh and Nadar (1989) on tee framing connections bolted to a beam web and welded to a column flange. This connection is classified as a simple shear connection based on rotation generated by yielding in the tee stem and flange. Several similarities were observed between the behavior of the tee stems in these connections and the plates in shear tab connections. Due to these similarities, Astaneh's formula for the fracture of the net area of the single plate utilizes the formula developed in this research for fracture of the net area of the tee stem. The requirement that the thickness of the plate not exceed $d_b/2$ + 1/16 in. was also developed for tee stems to allow desirable bearing deformations.

Sarkar (1992) conducted full scale beam tests of 2, 4, and 6 bolt single plate shear connections to evaluate the effectiveness of Astaneh's new procedure. All tests were performed with 3/4 in. diameter A325 bolts in either standard or short slotted holes with an a-distance of 3.5 in. The connections were beam-to-column web connections. Both the beams and columns were A36 steel. Unlike Astaneh, Sarkar loaded a simply supported beam with two concentrated loads. The starting location for these loads was placed to maximize beam end rotation. If the beam and connection did not fail at a rotation of 0.03 radians, the concentrated loads were moved closer to the connections to increase the end shear. The connections were then tested to failure. The test specifications are shown in Table 2.3, and the test results are shown in Table 2.4. The researcher then proposed a modified design method.

Sarkar identified failure modes for single plate shear connections. The list is similar to Astaneh's limit states listed above, except that lateral buckling of the shear tab is considered. Connection ductility is achieved by limiting the plate thickness-to-bolt diameter ratio to 0.42 and 0.52 for A325 and A490 bolts, respectively. This requirement is neglected for short slotted holes. The procedure adopts the bolt eccentricity calculation from Astaneh's procedure. The welds are limited to 3/4 times the plate thickness to allow the plate to yield before the weld. This value is typically greater than the value required to resist the eccentric load on the weld and thus governs the calculations.

Table 2.3

Sarkar (1992) Test Specifications

Test	Bolts	Bolt Diameter (in.)	Bolt Type	Plate Dimensions[1] (in.)	Hole Type	Weld Size (in.)	Beam	Beam & Plate Material[2]
1	2	3/4	A325-X	3/8x5x6	Standard	5/16	W12x35	A36
2	2	3/4	A325-N	3/8x5x6	Standard	5/16	W12x35	A36
3	4	3/4	A325-N	3/8x5x12	Standard	5/16	W18x76	A36
4	4	3/4	A325-N	3/8x5x12	Short Slotted	5/16	W18x76	A36
5	6	3/4	A325-N	3/8x5x18	Standard	5/16	W21x93	A36
6	6	3/4	A325-N	3/8x5x18	Short Slotted	5/16	W21x93	A36

[1] The a-distance was 3.5 in.

[2] Yeild Strength of Plate was experimentally found to be 47.7 ksi with an ultimate stregth of 65 ksi.

Table 2.4

Sarkar (1992) Test Results

Test No.	Failure Mode	Shear Force (kips)
1	Shear Distortion of Plate	64.3
2	Weld Tearing	51.8
3	Bolt Shear	66.5
4	Bolt Shear	129
5	Bolt Shear	102
6	Bolt Shear	168

Duggal (1996) expanded on Sarkar's study. In Sarkar's work, it was apparent that connection rotation was achieved more easily in short slotted holes then in standard holes. Duggal developed an equation to predict the amount of force required to slide a bolt in a slotted hole while sustaining shear load. Then, finite element models were used to compare the results of his equation with the results of Sarkar's study. The equation proved to be consistent with Sarkar's experimental results. When compared with Astaneh's design method (Astaneh, 1989c), Duggal's method is more accurate but involves greater detail in design because the plate and the weld are designed by calculating stress interactions. Astaneh's method does not specifically design the weld and the plate but simply ensures that the plate will yield prior to failure and that the weld will develop the plate strength. Duggal's method was never adopted by AISC.

In a later study, Astaneh et al. (2002) examined fifteen more single plate shear connections subjected to gravity loading. The researchers identified similar limit states to those found in previous work (Astaneh, 1989c). As in Caccavale's (1975) early studies it was found that, in addition to the yielding of the plate, ductility could also be gained by elongation of the bolt holes in the plate and beam web. Thus, the bearing failure of beam web or plate was separated into the ductile limit state, bearing on bolt holes in the plate and beam web, and the brittle limit state, fracture of edge distances of bolt holes.

Also included in this study were ten tests of beam-to-column flange connections with floor slabs under cyclic lateral loading. These connections showed the same hierarchy of limit states as the connections with gravity loading. The composite action was maintained until a drift of approximately 0.04 radians, and the connections were able to exhibit ductility even under additional drift.

Sherman and Ghorbanpoor (2003) completed a series of experimental tests to determine the strength of four unstiffened and thirteen stiffened extended shear tabs. The unstiffened tests consisted of three and five bolt connections to girder webs and three and five bolt connections to column flanges. All connections had short slotted holes with an extended a-distance between 6.3 and 10.0 in., A36 plate material, Gr. 50 beam and column material, and welds greater than 5/8 times the thickness of the plate. The test beams were simply supported on the far end and loaded with a single concentrated load.

Each of the unstiffened connections supported an end shear greater than the design load but significantly less than the design shear for a 3 in. standard a-distance. Researchers also noted that eccentricities found from strain gages were less than those determined from Astaneh's formulas found in the AISC LRFD Manual (AISC, 1993).

Therefore, a less conservative method of eccentricity calculation was proposed. Astaneh's weld limit of 3/4 times the plate thickness was maintained. Several failure modes were observed including: bolt bearing, bolt shear, and twisting. The twisting reported comes from inadequate bracing of the supported beam which would typically not be present with a traditional floor system. No beam end rotation values are presented.

Ashakul (2004) created a series of finite element models to evaluate the work done by Astaneh (1989c) and Sarkar (1992). The models were used to investigate several connection parameters including; the a-distance, plate material, plate thickness, and connection position with respect to the beam neutral axis. The researcher found that the bolt group capacity is not a function of the a-distance. However, the a-distances examined were within the relatively small range of 2.5 in. to 5 in. It was recommended that if plate thickness is limited to a certain value, the bolt group can be designed without consideration for the eccentricity. However, if the limiting plate thickness is not satisfied then moment is developed in the bolt group from horizontal bolt forces caused by bearing resistance.

Ashakul also proposed a different method to calculate shear yielding of the plate:

$$R_n = 0.6F_y \times [(n-1)p + L_e] \times t_p \quad (2.7)$$

where,

F_y = yield strength of the plate (ksi)

n = number of bolts in plate

p = vertical spacing between bolts (in.)

L_e = edge distance in vertical direction (in.)

t_p = thickness of plate (in.)

Ashakul developed finite element models of double column connections. The forces in the bolts in these connections redistributed when the plate thickness was less than 1/2 in. for Gr. 50 steel. If this was not the case, the second column of bolts failed first. No design method was proposed for double column connection due to the limited body of research regarding them.

Creech (2005) completed ten full scale tests of single plate shear connections. The researcher's goal was to compare the design procedure in the AISC LRFD 3rd Edition Manual (AISC, 2001) with the design methods used in Great Britain, Australia, and New Zealand as well as design methods of several independent researchers. An extensive collection of background information and methodology comparisons is found in his research.

The ten full scale tests sought to compare the measured eccentricity and ultimate strength of flexible versus rigid supports and standard versus short slotted holes. All tests used 3/4 in. A325 bolts, A36 plate material, and Gr. 50 beams, girders, and columns. The tests compared 3, 5, and 7 bolt connections. The test specifications are shown in Table 2.5. Each test consisted of a test beam with the connection to be tested at one end and a simple support at the other end. The test beam was loaded at two points, the locations of which varied, by actuators. A hydraulic ram was placed near the connection for additional shear loading if the connection did not fail under the actuator loading.

The test results are shown in Table 2.6. It was determined that the connections with short slotted holes allowed a greater connection rotation though there was no difference in ultimate strength between the connections with standard holes and those with short slotted holes. The connections to flexible supports did show a diminished strength capacity in comparison with connections to rigid supports. However, several of the connections to flexible supports had a simulated slab restraint which increased the capacity of the connection by approximately 5 percent.

Of the methods examined, the researcher found that the Astaneh eccentricity calculations used in the AISC LRFD 3rd Edition Manual (AISC, 2001) led to the closest conservative calculation of the ultimate strength of the connection. From the experimental calculations of connection eccentricity, it was found that eccentricity can be neglected for connections with more than three bolts if a 20 percent strength reduction factor, referred to herein as the bolt group action factor (BGAF), is included. The BGAF is included in the nominal bolt shear strength found in the AISC Specification (2005) and reduces the bolt shear strength by 20 percent to account for non-uniform load distributions in connections. As discussed in the Specification for Structural Joints (2004), the inclusion of this factor in the Specification is based on research done with shear splice plate connections. In these connections, it was found that connection bolts toward the end of the connection had higher levels of strain than those towards the center. Thus, the average bolt stress determined from the ultimate strength of the connection was less than the average bolt stress found from individual bolt shear tests by approximately 20 percent up to a connection length of 50 in. Creech's work indicated that the BGAF was not applicable in single plate shear connections. This is also supported by the Eurocode 3 (1992) which excludes the BGAF for framing connections and by AISC Design Guide 17 (2002) which states that "it is reasonable to think that the same phenomenon at least does not take place to the same degree," in framing type connections.

Table 2.5

Creech (2005) Test Specifications

Test	Bolts	Bolt Diameter (in.)	Bolt Type[1]	Hole Type	Support Condition	Plate Dimensions[2] (in.)	Edge Distance (in.)	Plate Material[3]	Weld Size (in.)	Test Beam	Beam Material
1	3	3/4	A325-N	SSL[4]	Rigid	3/8x4-1/4x9	1-1/2	A36	5/16	W16x50	A992
2	3	3/4	A325-N	Standard	Rigid	3/8x4-1/4x9	1-1/2	A36	5/16	W16x50	A992
3	3	3/4	A325-N	SSL	Flexible	3/8x4-1/4x9	1-1/2	A36	5/16	W16x50	A992
4	3	3/4	A325-N	Standard	Flexible	3/8x4-1/4x9	1-1/2	A36	5/16	W16x50	A992
5	3	3/4	A325-N	SSL	Flexible[5]	3/8x4-1/4x9	1-1/2	A36	5/16	W16x50	A992
6	2	3/4	A325-N	Standard	Flexible	3/8x4-1/2x6	1-1/2	A36	5/16	W16x50	A992
7	2	3/4	A325-N	SSL	Flexible	3/8x4-1/2x6	1-1/2	A36	5/16	W16x50	A992
8	2	3/4	A325-N	SSL	Flexible[5]	3/8x4-1/2x6	1-1/2	A36	5/16	W16x50	A992
9	7	3/4	A325-N	SSL	Rigid	3/8x4-1/2x21	1-1/2	A36	5/16	W27x84	A992
10	7	3/4	A325-N	SSL	Flexible[5]	3/8x4-1/2x21	1-1/2	A36	5/16	W27x84	A992

[1] The average measured bolt shear strength was 30.3 kips/bolt

[2] The a-distance was 3 in.

[3] The measured plate yield strength was 39.6 ksi and the ultimate strength was 62.1 ksi for Tests 1-8. The measured plate yield strength was 44.4 ksi and the ultimate strength was 66.3 ksi for Tests 9 and 10.

[4] Short Slotted Holes, SSL

[5] Connection included a simlated slab restraint.

Table 2.6

Creech (2005) Test Results

Test	Failure Mode	Max. Shear Force (kips)	Beam End Rotation (radians)	Bolt Eccentricty (in.)
1	-- [1]	78.8	0.036	1.6
2	Bolt Shear	90.7	0.027	2.1
3	-- [1]	71.8	0.039	-1.7
4	Bolt Shear	61.4	0.023	-2.0
5	-- [1]	75.6	0.031	0.1
6	Bolt Shear	44.2	0.012	-4.4
7	Bolt Shear	45.5	0.011	-2.4
8	Bolt Shear	47.9	0.013	-2.2
9	-- [1]	167	0.028	5.5
10	Bolt Shear	203	0.027	0.5

[1] Test beam failure occurred prior to connection failure.

2.3 AISC 13th Edition Steel Construction Manual Design Procedure

The AISC 13th Edition Manual (2005b) contains an updated design procedure for single plate shear connections. Connections which satisfy a given set of dimensional configurations can be designed using a simplified design method for the "conventional" configuration. All other connections can be designed using a method for the "extended" configuration. The full procedure as it appears in the Manual is reproduced in Appendix A. A summary of the major updates follows.

The eccentricity calculations for connections which meet the requirements of the conventional configuration have been altered to reflect the research by Creech (2005). For standard holes, no eccentricity is calculated for connections in the conventional configuration with less than 10 bolts. This change is based upon the recognition that the bolt shear strength values in the AISC Specification (2005a) have been reduced by 20 percent due to the bolt group action factor. As discussed previously, it has been shown that this reduction factor does not apply to single plate shear connections. Therefore, AISC ignores up to a twenty percent reduction in ultimate strength due to eccentricity. For 10, 11, and 12 bolt connections, the eccentricity provides a strength reduction of greater than 20 percent; therefore, eccentricity is taken into account. However, the calculated eccentricity coefficient, C, is then multiplied by 1.25. No eccentricity is used on short slotted holes, and no distinction is made between flexible and rigid supports.

Single plate connections designed using the extended configuration procedure are required to take into account an eccentricity equal to the a-distance of the connections. An exception is provided for the engineer to use alternate methods, such as those proposed by Sherman and Ghorbanpoor (2003), when justified by rational analysis.

The weld size required to develop the plate strength has been reduced from 3/4 times the plate thickness to 5/8 times the plate thickness. This change is based on work by Thornton (Muir, 2006b). Astaneh's original weld thickness limitation was determined by ensuring that the single plate would yield prior to the weld yielding. The weld yield strength was estimated as one half of the tensile strength for E70 electrodes. Thornton determined that the weld thickness should be found by ensuring that the single plate would yield prior to the weld fracture, as opposed to the weld yield. The theoretical relationship between shear and tension was used along with Astaneh's original derivation. It was found that a weld thickness equal to 5/8 times the plate thickness provides that the plate will yield prior to weld fracture in pure moment, pure shear, or a combination of shear and moment. This limit also ensures that the weld will not have excessive capacity beyond the capacity of the plate.

Both the conventional and extended configuration procedures require edge distances to be consistent with Chapter J of the AISC Specification (2005a). Several design checks were added to create dimensional flexibility in the extended configuration. These include: ensuring that the moment capacity of the plate is less than the moment capacity of the bolt group, checking plate flexure using Von-Mises shear reduction, and checking for plate buckling using the double coped beam procedure found in the AISC Manual (2005b), which is based on work by Muir and Thornton (2004).

2.4 Design Comparisons

Several of the previously discussed researchers proposed design methods for single plate shear connections that led to the development of manual and code procedures in a variety of countries. The methods in the AISC LRFD Manual of Steel Construction 3rd Edition (2001), the AISC Steel Construction Manual 13th Edition (2005b), the Australian OneSteel Market Mills Composite Structures Design Manual (2000), The British Constructional Steelwork Association's *Joints in Steel Construction* (2002), and New Zealand Heavy Engineering Research Association's *Structural Steelwork Connection Guide* (2005) are compared in the following sections. The AISC LRFD 3rd Edition Manual is based on Astaneh's work (1989c), which was discussed previously. The Canadian design procedure is also based on Astaneh's work, and because it is similar to the AISC (AISC, 2001) procedure, it will not be discussed here. Richard's work is not discussed since it is ASD based.

A discussion of the significant limit states follows. At the end of the discussion a design example is summarized in Table 2.7.

2.4.1 Connection Bolts

The design of the connection bolts in single plate shear connections has been at the center of many research studies. In the United States the bolt strength is based on eccentric loading of the bolt group. The AISC 3rd Edition expanded on Astaneh's work (1989c) and calculated an eccentricity depended on the number of bolts, the a-distance, the hole type, and the support type (rigid or flexible). As discussed in Section 2.3, the eccentricity calculations in the AISC 13th Edition have been altered to reflect newer research completed by Creech (2005). Calculation of eccentricity is no longer dependent on the support type. Bearing and tearout are also checked in both AISC methods as separate limit states.

Australia and New Zealand have similar procedures for determining connection bolt strength. In Australia, the bolt strength is calculated as the individual bolt strength multiplied by the section modulus of the bolt group. This section modulus is calculated from an interaction of the horizontal bolt force, the vertical bolt force, and the moment on the bolt. The eccentricity used to determine the bolt group section modulus is based upon the rotational stiffness of the connection and the flexural stiffness of the beam. In New Zealand, the process is similar except that the bolt group section modulus is based upon an eccentricity equal to the connection a-distance. In both countries the bolt strength is limited by the bearing and tearout capacity of the plate or the beam. These bearing and tearout capacity calculations are discussed in Section 2.4.2.

In Great Britain, the bolts are designed by ensuring that the capacity of an individual bolt is greater than the resultant force on the outermost bolt. The force on the outermost bolt is calculated from a combination of shear and moment. The eccentricity for the moment calculation is taken as the a-distance. The force on the outermost bolt is also limited by the bearing strength of the individual bolt.

2.4.2 Connection Bearing

In the United States, the bearing and tearout strength is calculated identically in the AISC 3rd Edition and the AISC 13th Edition. The strength at an individual bolt hole is calculated as the minimum of the bearing and tearout strength of the hole. The AISC Specification does not require checks in the horizontal and vertical directions. Instead the Specification states that tearout is calculated in the direction of the applied force. The individual bolt hole strengths for each bolt are then added to obtain the bearing and tearout capacity of the connection.

The Australian method determines the individual bolt hole strength as the minimum of the bearing strength, the vertical tearout strength, and the horizontal tearout strength. Both the beam and the plate are checked at these three limit states. These three bolt strengths are each multiplied by a specific bolt group modulus. The eccentricity used for the calculation of the modulus is based on the connection stiffness and the a-distance. The minimum of these three values is the bearing and tearout strength of the connection. The method is a “poison bolt” method in that the individual bolt with the lowest capacity controls the connection strength. The New Zealand method is conceptually similar to the Australian method except that the eccentricity is taken as the connection a-distance.

The bearing strength is considered in Great Britain in the same manner as in the United States except that tearout is not checked.

2.4.3 Connection Plate

Several limit states are typically considered with respect to the connection plate. The AISC 3rd Edition finds the plate strength based on shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate, and the bearing capacity of the plate. The plate dimensions are limited to prevent buckling of the plate. The AISC 13th Edition conventional configuration recommends the same checks as the AISC 3rd Edition. However, the AISC 13th Edition extended procedure requires additional checks due to the dimensional flexibility allowed. Plate buckling and plate flexure including Von Mises shear reduction must be checked.

The Australian single plate shear design method considers bearing and tearout as described above and shear yielding but not shear rupture. The procedure also includes a check of flexural yielding calculated with the plastic section modulus and an eccentricity based on the connection stiffness and the a-distance. In New Zealand, the plate limit states considered depend on the applied connection shear. Bearing and tearout are considered as described previously. At low connection shear, shear yielding and shear rupture are considered as separate limit states. At higher values of connection shear, a reduced plate capacity based on combined shear and flexural yielding at both the gross and net section is used. The flexural yielding calculation uses the plastic section modulus and an eccentricity equal to the a-distance. Flexural rupture is also considered in the New Zealand method.

The design method used in Great Britain for plate strength is also dependent on the level of applied connection shear. At low connection shear, a combination of shear and flexural yielding is considered elastically. At high connection shear, a combination of shear and flexural yielding is considered inelastically. The limit states of shear yielding and shear rupture are also checked individually. Block shear rupture of the plate is considered using the yielding strength of the plate. For long plates, where the a-distance is greater than the plate thickness divided by 0.15, plate buckling is also considered.

2.4.4 Additional Considerations

Welds in single plate shear connections are typically not explicitly designed. In the AISC 3rd Edition, it is recommended that a weld size equal to 3/4 times the plate thickness be used to ensure that the plate will yield prior to weld fracture. The AISC 13th Edition has reduced this weld size to 5/8 times the plate thickness for reasons discussed previously. The Australian design method still requires a weld size equal to 3/4 times the plate thickness similar to the AISC 3rd Edition. A more conservative weld size equal to 0.8 times the plate thickness is used in Great Britain. In New Zealand, the weld size is based on the ultimate tensile strength of the weld while allowing full tensile strength development of the plate.

The design procedures discussed above also contain recommendations for supported beam design and bracing. These recommendations will not be discussed here. For a thorough summary of these, see the work by Creech (2005).

2.4.5 Design Example

A design example illustrating the differences between these methods is summarized in Table 2.5. The design example is a three bolt connection with a single vertical column of bolts. The connection is assumed to have a rigid support member. Plate material has a nominal yield strength of 36 ksi, and the beam sections have a nominal yield strength of 50 ksi. The nominal shear stress of the bolts in a bearing type connection is 48 ksi. The calculations appear in Appendix B. All values in the table have been calculated excluding strength reduction (Φ) factors. Dimensional requirements of several methods have been violated to provide a specific connection for comparison.

All but one design method is controlled by bolt shear rupture strength. The design capacities range from 35 kips to 64 kips, and the eccentricities used in the bolt shear calculations range from 0 in. to 3 in. The lower bolt shear strengths occurred when the bolt eccentricity was taken as the a-distance. The Australian design method was controlled by connection bearing capacity. The significantly lower bearing strength determined by the Australian and New Zealand methods is caused by utilizing the lowest individual bolt capacity to calculate the connection capacity. In the United States and Great Britain, the individual bolt strengths are found and added to find the connection capacity. The higher bearing capacity found by the Great Britain method is because tearout is not considered in this method. Slight variations are also apparent in the shear rupture and block shear calculations due to different bolt hole dimensions used. The wide range of predicted values indicates that more research may be needed to develop a consistent design procedure.

Table 2.7
Design Comparison

Limit State Considered		AISC 13th Edition Conventional Configuration	AISC 3rd Edition	Australian	New Zealand	Great Britain
Dimensional Parameters	No. of Bolts	3	3	3	3	3
	Vertical Columns of Bolts	1	1	1	1	1
	a-distance	3 Iin.	3 in.	3 in. [1]	3 in. [4]	3 in.
	Plate Yield Strength	36 ksi	36 ksi	36 ksi	36 ksi	36 ksi
	Supported Beam Yield Strength	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
	Horizontal Edge Distance, L_{eh}	1.5 in.	1.5 in.	1.5 in. [2]	1.5 in.	1.5 in.
	Vertical Edge Distance, L_{ev}	1.5 in.	1.5 in.	1.5 in. [2]	1.5 in.	1.5 in.
	Vertical Bolt Spacing	3 in.	3 in.	3 in. [3]	3 in.	3 in.
	Plate Thickness	3/8 in.	3/8 in.	3/8 in.	3/8 in.	3/8 in. [5]
	Weld Size	1/4 in.	5/16 in.	5/16 in.	1/4 in.	5/16 in.
	Beam Section	W18x55	W18x55	W18x55	W18x55	W18x55
Bolts	Shear Strength (rigid support)	64 kips	57 kips	64 kips	35 kips	35 kips
	Eccentricity (rigid support)	neglected	1 in.	0 in.	3 in.	3 in.

Table 2.7

Design Comparison, Cont.

Plate	Shear Yielding	73 kips	73 kips	73 kips	73 kips	73 kips
	Shear Rupture	83 kips	83 kips	○	83 kips	86 kips
	Block Shear Rupture	90 kips	84 kips	○	○	94 kips
	Bearing Capacity	107 kips	107 kips	57 kips	57 kips	118 kips
	Flexural Yielding	○	○	91 kips	91 kips	● [6]
	Flexural Rupture	○	○	○	91 kips	○
	Combined Flexure and Shear	○	○	○	37 kips	57 kips
	Buckling Strength	○	○	○	○	101 kips
Weld	Shear Strength	◊	◊	◊	134 kips	◊
	Combined Flexure and Shear	◊	◊	◊	○	◊
Connection Strength		64 kips	57 kips	57 kips	35 kips	35 kips
Controlling Limit State		Bolt Shear	Bolt Shear	Bearing	Bolt Shear	Bolt Shear

Notes: The method was developed with an a-distance equal to 2.17 in. (55mm).

[2] The method requires vertical and horizontal edge distances equal to 1.38 in. (35mm).

[3] The method requires a vertical bolt spacing of 2.76 in. (70mm).

[4] The method was developed with an a-distance equal to 1.97 in. (50mm).

[5] Minimum plate thickness is recommended for Gr. 43 plate material. No recommendation is made for Gr. 36 plate material

[6] Flexural yielding is only checked at low levels of applied shear. Otherwise, the combined flexure and shear check is determined to be sutible.

Table Key:

●	Limt state is considered in design.
○	Limit state is not considered in design.
◊	Limit state is not directly calculated, but it is satisfied by a dimensional requirments.

2.5 Comparison of Previous Test Results with AISC 13th Edition Manual

To evaluate the validity of the AISC 13th Ed. design procedure, the results of previous tests by Astaneh (1989), Sarkar (1992), Sherman and Ghorbanpoor (2003), and Creech (2005) were compared to their strengths as predicted by the new design procedure. All tests consisted of a single vertical column of 3/4 in. bolts. The reported measured plate yield and ultimate stresses were used to determine limit state strengths. The measured bolt shear strength was used for the series of tests by Creech. The nominal bolt shear strengths as given in the AISC specification (2005a) were used for the tests by Sarkar, Sherman and Ghorbanpoor, and Astaneh since information regarding the measured bolt shear strength was not provided. No strength reduction (Φ) factors are included.

Table 2.8 summarizes the results of this analysis. The controlling design limit state for all experimental tests was bolt shear strength. None of Astaneh's experimental values are more than 2 percent below the predicted values. However, if measured bolt shear values were used this may not have been the case as research by Creech (2005) and the current research have found that measured bolt shear strength was fifteen percent or more above the nominal strength. Four of the experimental tests performed by Sarkar resulted in failure strengths twenty to fifty percent greater than those predicted. Conversely, two of the tests failed at strengths twenty percent less than the predicted strength. Shearman and Ghorbanpoor's experimental results were significantly higher than the predicted strengths. Each of his tests had an extended a-distance which may account for their conservative design. Again, these results are based on nominal bolt shear strengths. All of Creech's test results were within twenty-five percent of the predicted strength, and, with the exception of test 4, were either conservative or less than ten percent below the predicted value.

Overall, the experimental test strengths exceeded the AISC (2005b) predicted strengths by 26 percent; however, the standard deviation of the experimental strength-to-predicted strength ratios was 0.482. The ratios ranged from 0.79 to 2.60. Taken alone, the ratios from the experimental tests by Creech (2005) had a standard deviation of 0.125 and a mean value of 1.02. The use of measured bolt shear strengths for these test predictions almost certainly contributed to the increased accuracy.

Table 2.8

AISC (2005b) Predicted Strengths

Test		Parameters				Limit State Strengths					Experimental Connection Strength	Experimental/ Predicted Strength
		Bolts	a-distance (in.)	Weld Size (in.)	Bolt Eccentricity (kips)	PL Shear Yielding (kips)	PL Shear Rupture (kips)	PL Block Shear (kips)	PL Bearing (kips)	Bolt Shear (kips)	(kips)	
Astaneh	1	3	2.75	1/4	neglect	71.9	87.5	84.2	112	63.6	94	1.48
	2	5	2.75	1/4	neglect	120	146	132	195	106	137	1.29
	3	7	2.75	1/4	neglect	168	204	180	277	148	160	1.08
	4	3	2.75	7/32	neglect	65.9	77.2	72.6	102	79.5	79	0.99
	5	5	2.75	7/32	neglect	114	136	121	184	133	130	0.98
Sarkar	1	2	3.5	5/16	neglect	64.4	62.2	72.5	75.9	53.0	64.3	1.21
	2	2	3.5	5/16	neglect	64.4	62.2	72.5	75.9	42.4	51.8	1.22
	3	4	3.5	5/16	neglect	129	124	135	164	84.2	66.5	0.79
	4	4	3.5	5/16	neglect	129	124	135	164	84.2	129	1.53
	5	6	3.5	5/16	neglect	193	186	197	251	127.2	102	0.80
	6	6	3.5	5/16	neglect	193	186	197	251	127.2	168	1.32

Table 2.8

AISC (2005b) Predicted Strengths, Cont.

Test		Bolts	a-distance (in.)	Weld Size (in.)	Bolt Eccentricity (kips)	PL Shear Yielding (kips)	PL Shear Rupture (kips)	PL Block Shear (kips)	PL Bearing (kips)	Bolt Shear (kips)	Experimental Connection Strength (kips)	Experimental/Predicted Strength
		Parameters				Limit State Strengths						
Creech	1	3	3	5/16	neglect	80.8	89.1	92.1	114	72.7	78.8	1.08
Creech	2	3	3	5/16	neglect	80.8	89.1	92.1	114	72.7	90.7	1.25
Creech	3	3	3	5/16	neglect	80.8	89.1	92.1	114	72.7	71.8	0.99
Creech	4	3	3	5/16	neglect	80.8	89.1	92.1	114	72.7	61.4	0.84
Creech	5	3	3	5/16	neglect	80.8	89.1	92.1	114	72.7	75.6	1.04
Creech	6	2	3	5/16	neglect	53.9	59.4	65.1	72.5	48.5	44.2	0.91
Creech	7	2	3	5/16	neglect	53.9	59.4	65.1	72.5	48.5	45.5	0.94
Creech	8	2	3	5/16	neglect	53.9	59.4	65.1	72.5	48.5	47.9	0.99
Creech	9	7	3	5/16	neglect	210	222	221	301	170	167	0.98
Creech	10	7	3	5/16	neglect	210	222	221	301	170	203	1.20
Shearman and Ghorbanpoor	1	3	6.85	5/16	6.85	86.3	115	151	123	22.6	58.7	2.60
Shearman and Ghorbanpoor	2	5	6.3	5/16	6.30	144	192	209	212	66.3	89.3	1.35
Shearman and Ghorbanpoor	3	3	6.86	5/16	6.86	86.3	115	151	123	22.6	54.8	2.42
Shearman and Ghorbanpoor	4	5	10.04	5/16	10.0	182	244	265	271	44.0	98.7	2.24

Notes:

(1) The measured bolt shear strength of 30.3 kips/bolt (68.6ksi) is used for Creech; however, no measured bolt shear strength is given in Astaneh (1989c), Sarkar (1992), or Sherman and Ghorbanpoor (2003), thus, the nominal bolt shear strength was used. The bolt group action factor is included in all calculations.

(2) All bolt diameters are 3/4 in.

(3) All bolt groups consist of a single vertical column of bolts.

(4) The bolt group shear strength for Sherman and Ghorbanpoor tests includes an eccentricity, e, equal to the a-distance as dictated by the extended configuration procedure.

2.6 Need for Research

The current design method used in the United States is a culmination of years of research into the behavior of single plate shear connections. Several significant changes are included in the AISC 13th Edition Steel Construction Manual (2005b). These changes are based on experimental and analytical research work in the United States as well as comparison with design methods worldwide. As shown in Section 2.5, when used to predict the strengths of connections in several previous research studies, the new method performs reasonably well for connections with small a-distances. However, connections with extended a-distances were overly conservatively designed. Few connections with extended a-distances or with double vertical columns of bolts have been experimentally tested. Therefore, the need for a series of tests that includes extended configuration connections and the new weld size limitation was deemed necessary to examine the behavior of connections designed in accordance with the new method.

CHAPTER 3
EXPERIMENTAL TESTING

3.1 Overview

Eight full scale experimental tests were performed to evaluate the behavior of single plate shear connections. The connections were designed according to the procedure included in the AISC 13th Edition Manual and the requirements of the AISC 2005 Specification. All references to the AISC Specification included in this chapter refer to the AISC 2005 Specification and Commentary for Structural Steel Buildings (2005a). All references to the AISC Manual refer to the AISC 13th Edition Steel Construction Manual (2005b).

Four shear tab connections designed according to the conventional procedure and four shear tab connections designed according to the extended procedure were tested. The connection configurations were chosen based upon those typically found in the building industry. The goal during testing was to impose a combination of shear and rotation on the connection up to failure and to reach a beam end rotation of 0.03 radians. The specified rotation is in keeping with the typical beam end rotation for a simply supported uniformly loaded beam at failure as discussed in the AISC Commentary Section B3.6 (2005a).

Each test setup consisted of a test beam with a shear tab connection to a column flange (e.g. rigid support) at one end and a simple roller support set on a load cell at the other end. The beam was then loaded by two hydraulic rams placed to impose a specified rotation and shear on the connection. Braces were placed at points along the test beam length to prevent lateral torsional buckling. All single plates were welded to the test column flange and bolted to the test beam. Figure 3.1 illustrates a typical test setup.

Test identification codes were assigned to each connection. The identification code includes the number of bolts, the number of vertical columns of bolts, the a-distance, and the plate thickness. For example, connection test number one is designated as 3B1C-3-3/8, a three bolt connection with one vertical column of bolts, an a-distance of 3 in., and a single plate thickness of 3/8 in. Table 3.1 shows the test specimen data.

Figure 3.1 Typical Test Setup

Table 3.1

Connection Data

	Test Specimen	Bolt Columns	Bolt Rows	a-distance (in.)	Single Plate Dimensions	Beam Section	Beam Length
Conventional Configuration	3B1C - 3 - 3/8	1	3	3	3/8" x 4 1/2" x 8 1/2"	W18x55	21'-7"
	4B1C - 3 - 3/8	1	4	3	3/8" x 4 1/2" x 11 1/2"	W24x76	27'-7"
	5B1C - 3 - 3/8	1	5	3	3/8" x 4 1/2" x 14 1/2"	W24x76	23'-1"
	7B1C - 3 - 3/8	1	7	3	3/8" x 4 1/2" x 20 1/2"	W30x108	27'-7"
Extended Configuration	6B2C - 4.5 -1/2	2	3	4-1/2	1/2" x 7 1/2" x 8 1/2"	W18x55	18'-7"
	10B2C - 4.5 - 1/2	2	5	4-1/2	1/2" x 7 1/2" x 14 1/2"	W30x108	24'-7"
	7B1C - 9 - 3/8	1	7	9	3/8" x 10 1/2" x 20 1/2"	W24X62	22'-10.5"
	10B2C - 10.5 - 1/2	2	5	10-1/2	1/2" x 13 1/2" x 14 1/2"	W24X62	22'-10.5"

3.2 Connection Test Specifications

3.2.1 Connection Design: Conventional Configuration

Four full scale tests were designed to meet the requirements of the AISC conventional configuration design procedure. The test series included three, four, five, and seven bolt connections with standard holes. Each connection consisted of a single vertical column of bolts with an a-distance of 3 in. The horizontal edge distance on the plates was equal to two times the bolt diameter. The horizontal edge distance on the beams was 2 in., which is greater than two times the bolt diameter in all cases. The vertical edge distance on the plates met the minimum requirement of the AISC Specification Table J3.4 (2005a). All edge distances were measured from the center of the bolt hole. To ensure connection ductility, the single plates were designed with a thickness less than $d_b/2 + 1/16$ in., where d_b is the bolt diameter.

The connections were designed to have off-axis bolt groups. An off-axis bolt group is a bolt group whose center of gravity does not lie on the neutral axis of the test beam. The distance from the center of the upper-most bolt hole to the top of the beam was 3 in. for the 3, 4, and 5 bolt connections and 4 in. for the seven bolt connection. The connections were placed at this location in keeping with the common practices of the sponsoring steel fabricator.

The weld size required by the AISC design procedure is 5/8 times the thickness of the single plate. This is a reduction from 3/4 times the thickness of the plate which was recommended by the design procedure in the previous edition of the AISC LRFD Manual (2001). However, welds in the current research were 1/2 times the thickness of the plate. The goal in examining a reduced weld size was to decrease the weld volume by 36% and facilitate the use of single pass welds.

To determine the nominal strength of the connection, the following limit states were checked:

(1) Shear yielding of the plate (AISC Specification J4.2)
(2) Shear rupture of the plate (AISC Specification J4.2)
(3) Block shear rupture of the plate (AISC Specification J4.3)
(4) Bolt bearing and tear out on the plate (AISC Specification J3.10)
(5) Bolt shear rupture excluding eccentricity (AISC Specification J3.6)
(6) Bearing on the test beam (AISC Specification J3.10)

All design calculations used nominal values found in the ASIC Specification (2005a). Design drawings of the connections are included in Appendix D.

3.2.2 Connection Design: Extended Configuration

Four full scale tests were designed to meet the requirements of the AISC extended configuration design procedure. The first two tests consisted of two vertical columns of bolts and an a-distance of 4-1/2 in. The next test had a single vertical column of bolts with an a-distance of 9 in., and the final test had two vertical columns of bolts with an a-distance of 10-1/2 in. All bolts were placed in standard bolt holes. The horizontal edge distances were 1-1/2 in. and the vertical edge distances were 1-1/4 in. Both edge distances met the minimum requirements of the AISC Specification Table J3.4 (2005a). The edge distances were measured from the center of the bolt holes.

Similar to the conventional configuration connections, all bolt groups were off axis bolt groups in keeping with the common practices of the sponsoring steel fabricator. Also, a reduced weld size equal to 1/2 times the thickness of the plate was used in the extended configuration connections. To ensure connection ductility, the single plates were designed to have a moment capacity less than the moment capacity of the bolt group in shear.

To determine the design strength of the connection, the following limit states were checked:

(1) Shear yielding of the plate	(AISC Specification J4.2)
(2) Shear rupture of the plate	(AISC Specification J4.2)
(3) Block shear rupture of the plate	(AISC Specification J4.3)
(4) Bolt bearing and tear out on the plate	(AISC Specification J3.10)
(5) Bolt shear rupture including an eccentricity, e = a-distance	(AISC Specification J3.6, Manual 7)
(6) Plate flexure including the Von-Mises shear reduction	(AISC Manual 10-103)
(7) Plate buckling	(AISC Manual 9-9)
(8) Bearing on the test beam	(AISC Specification J3.10)

All design calculations used nominal values found in the ASIC Specification (2005a). Design drawings of the connections appear in Appendix D.

3.2.3 Test Beam Design

Each connection test utilized a test beam. The test beam was connected to the test column by a shear tab connection on one end and was supported by a simple roller support on the other end. Load was applied to the test beam at nominal third points by two hydraulic rams held in place by steel frames bolted to the reaction floor. Though calculations did not indicate the need for web stiffeners, stiffeners were added at the loading points and at the simply supported beam end to ensure stability during testing. An angle bolted to the beam web and extending the full distance between the beam flanges acted as a stiffener at the supported beam end. The test beam for test 3B1C-3-3/8 is shown in Figure 3.2. All other test beams were laid out similarly.

To prevent lateral torsional buckling of the test beam, lateral bracing mechanisms were used. The placement of these lateral bracing mechanisms is discussed in Section 4.2 and 4.3. Several different bracing mechanisms were also used for the simply supported beam end. These mechanisms are discussed in detail in Section 3.4.

The goal of the research was to impose a shear and rotation on the connection and to reach a beam end rotation of 0.03 radians at failure. Utilizing mechanics based calculations, it was determined that loading the beam at nominal third points could be used to simulate the effects of uniform loading. Calculations supporting this finding appear in Appendix E. The beam length was then chosen to place the design load on the connection at a specified rotation. The test beam sections and lengths are shown in Table 3.1.

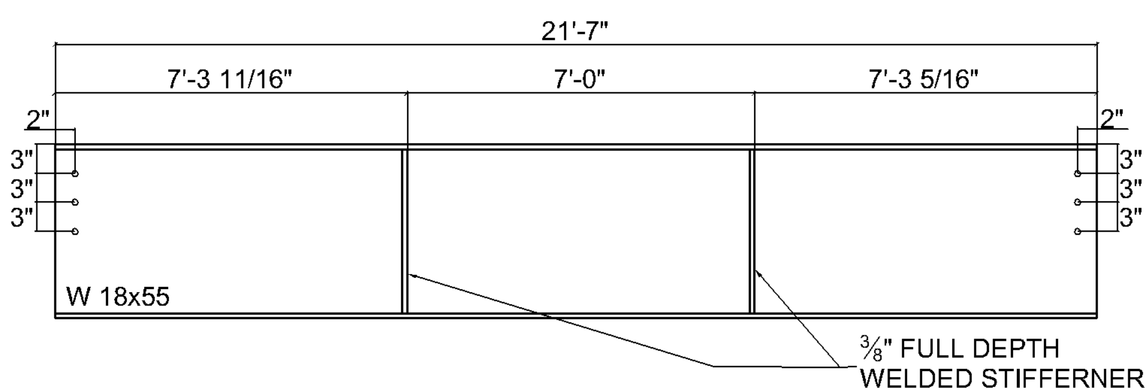

Figure 3.2 Test Beam for 3B1C-3-3/8

3.2.4 Test Columns

Two W21x62 test columns were used. Each test column had four shear tabs welded to the flanges; two on each side. The test columns were designed to be flipped vertically and rotated to create four tests from one column. The column details are shown in Figure 3.3. The single plates were welded off-center so that the centerline of the beam web and the column web are in line when the connection is in place. The test column size was chosen to facilitate use of available testing frames. A channel was bolted to the test column flange and to testing frame columns adjacent to the test column. The channel and testing frames provided bracing for the test column.

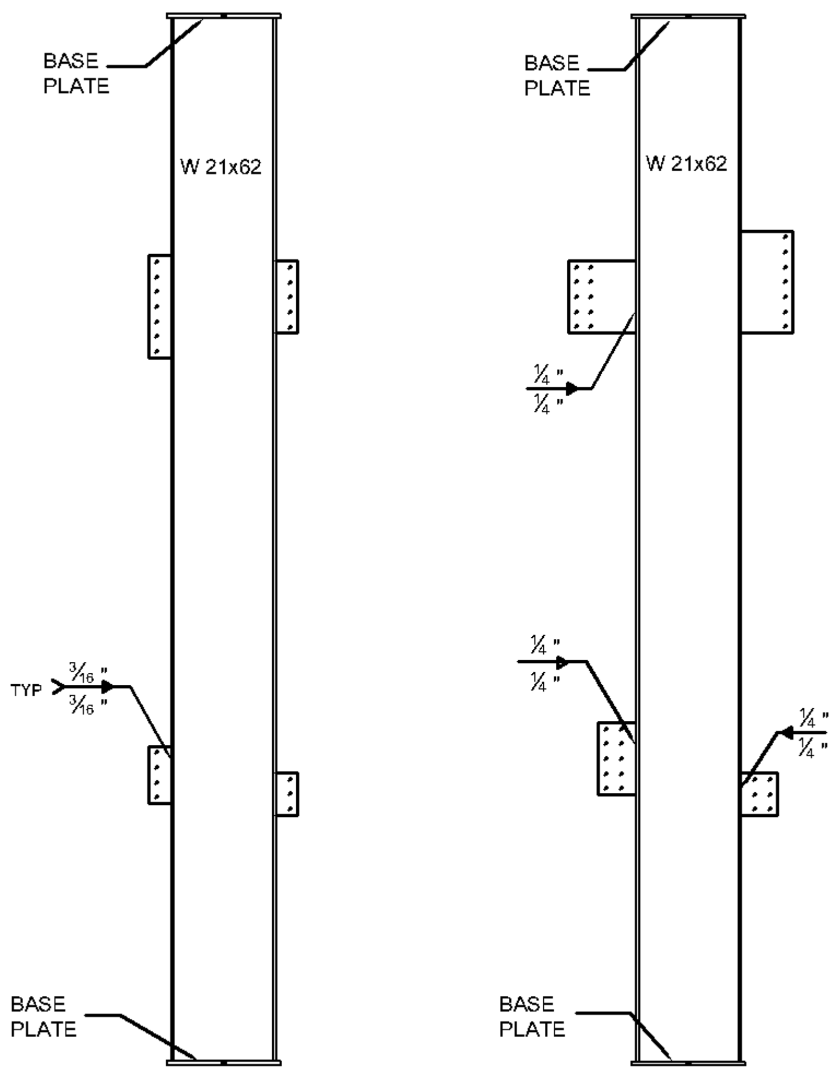

Figure 3.3 Test Columns

3.3 Test Fabrication and Materials

The test beams, columns, and connections were fabricated by the Mid-Atlantic Division of Cives Steel Company in Winchester, Virginia. The single plates were shop welded to the test column. The 3/16 in. welds were made using shielded metal arc welding, or stick welding, with 3/32 in. rods. A lower amperage was used on these welds due to their smaller than typical size. All welds larger than 3/16 in. were made using the submerged arc welding procedure (Muir, 2006a).

Steel used for the test beams and test columns was specified as ASTM A992 with nominal yield strength of 50 ksi. ASTM A572 Grade 50 steel was specified for the single plates. All bolts were ASTM A325-N with ASTM A563 nuts and ASTM F436 structural washers. One washer was placed under the nut and additional washers were used under the bolt head to ensure that bolt threads were in the connection shear plane as required for an N-type connection. The single plates were welded to the column flanges using E70 electrodes.

3.4 Test Setup

The test setup consisted of the test beam, supporting column, free end support beam, loading support frame, and lateral bracing frames. The test column was bolted to the reaction floor and connected to testing frame columns on either side by a bracing channel. The channel was bolted to the test column and each of the testing frame columns. The hydraulic rams were placed in two separate loading frames which were bolted to the reaction floor. An additional testing frame was placed between the loading frames to provide support for lateral bracing. The test beam was attached to the column flange with a single plate connection. The other end of the test beam was supported by a roller on a load cell supported by a beam bolted to the reaction floor. All test setup bolts, except those included in the shear tab connection, were tightened with an impact wrench. Figures 3.4 and 3.5 illustrate a typical test setup including bracing points.

In a typical test setup, lateral bracing mechanisms were attached to the beam at the loading points and at the beam mid-span. Additional bracing mechanisms were added at the connection for the tests with an a-distance greater than 4.5 in. due to twisting of the plate about the horizontal beam axis. Several different bracing mechanisms were used at the roller support end of the test beam. The bracing used initially was inadequate to prevent out-of-plane beam end rotation. An improved bracing design used in later connection tests is shown in Figure 3.6. This design provided enough resistance to prevent movement out of the untested plane of the beam web.

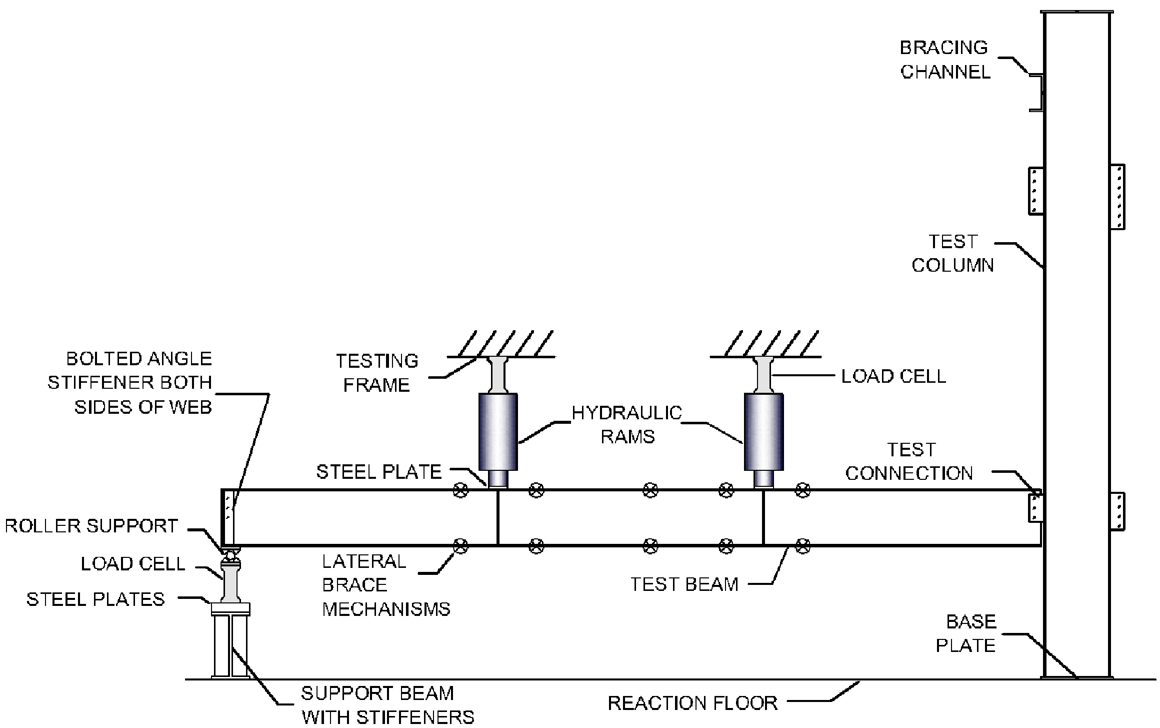

Figure 3.4 Schematic of Test Setup

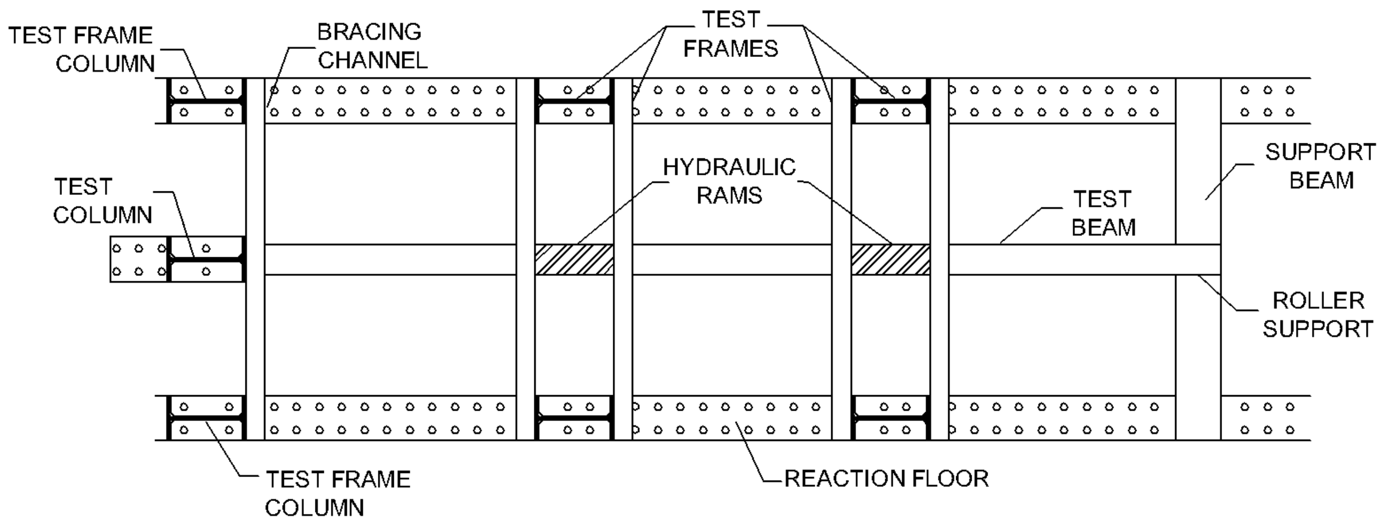

Figure 3.5 Plan View of Test Setup

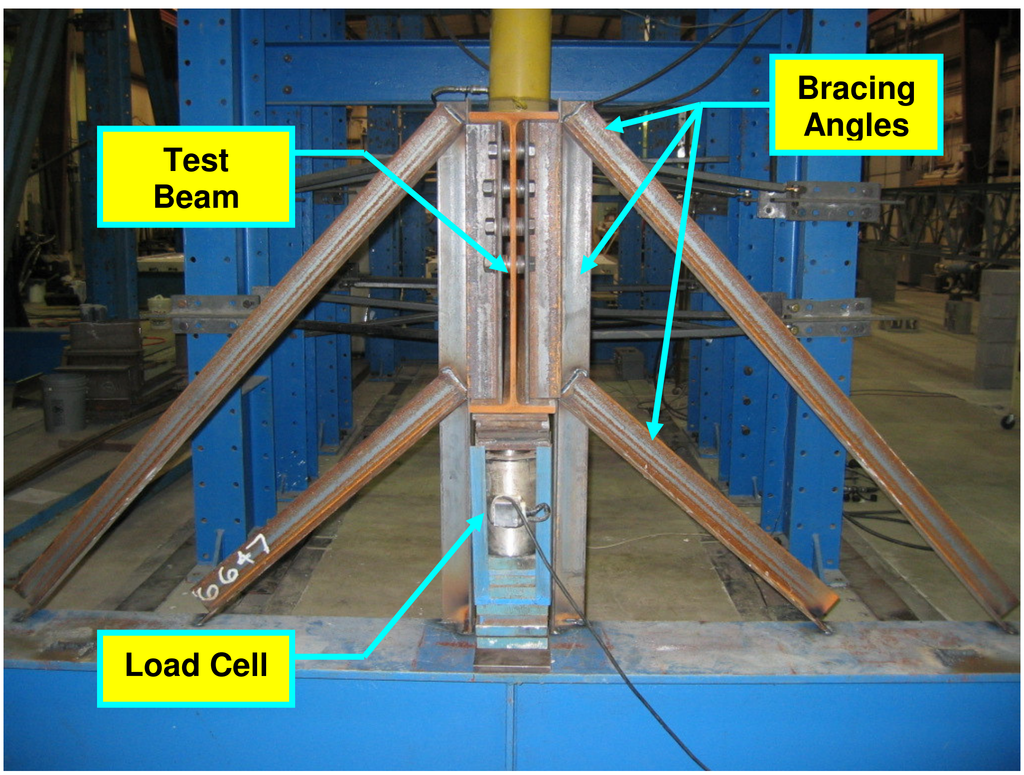

Figure 3.6 Beam End Bracing

3.5 Test Instrumentation

A variety of instrumentation was used to monitor the specimens throughout testing. Load cells were used at each hydraulic ram to measure the applied load. An additional load cell was placed under the simply supported end of the test beam to measure the force supported. Data from the three load cells was used to determine the applied connection force.

Vertical deflections were measured by string-type potentiometers attached to the bottom flange of the test beam at each end and at points of load application. The measurements recorded at the beam ends were used to account for support and connection settlement. The deflections measured at the loading points were adjusted to take into account these settlements. The adjusted deflections appear in all reported values.

Linear potentiometers were used to measure the beam end rotation relative to the test column. A steel angle was welded to a plate, and the plate was clamped to the test column so that the angle extended horizontally over the beam. The potentiometers were attached to the angle. One potentiometer was placed over the center of the bolt group and the second was placed 6 in. down the length of the beam from the first. The linear potentiometers at a typical connection are shown in Figure 3.7. The difference in the vertical deflection values was used to calculate the rotation of the beam end relative to the column.

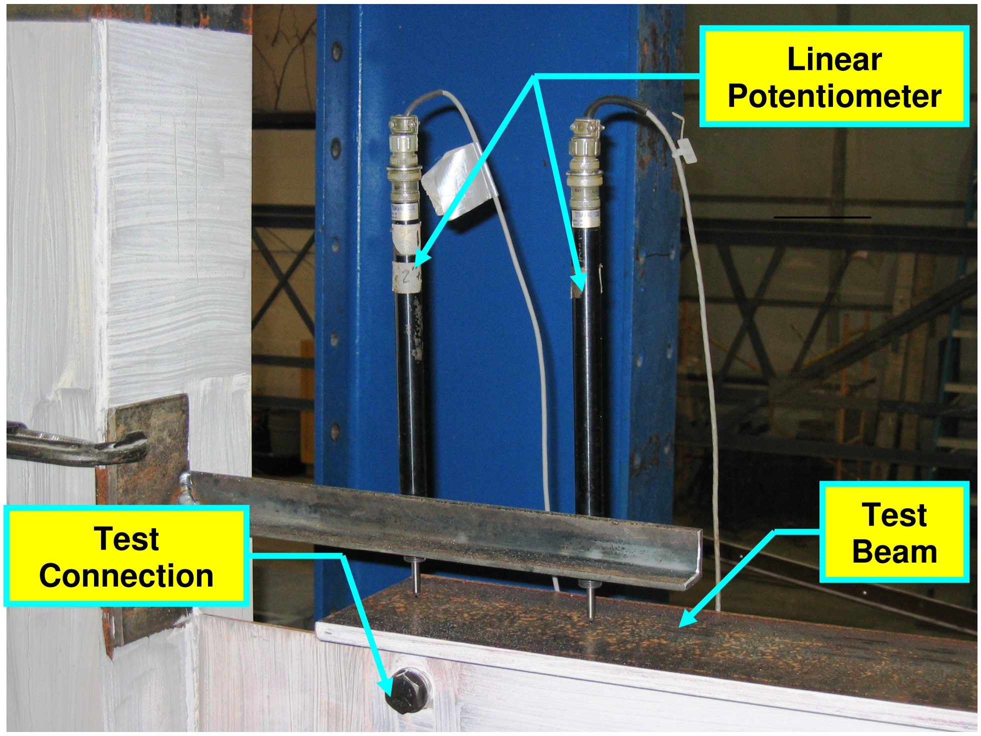

Figure 3.7 Linear Potentiometers at Beam Connection End

3.6 Connection Test Procedures

The test beam was installed in the test frame and connected to the test column with snug tight bolts and sufficient washers to ensure an N-type connection. The testing frames holding the hydraulic rams were positioned over the beam stiffeners. The lateral braces were bolted to the test beam and the testing frame. The beam was then cleaned, and a white wash was applied. All of the instrumentation was placed in position and connected to a PC-based data acquisition system. The instrumentation was then zeroed to read measurements relative to the initial position. The untested set up was photographed.

The beam was initially loaded twice to 10 percent of the design load to “settle” the setup. Next, the beam was loaded and unloaded with twenty percent and then fifty percent of the design load to check the instrumentation behavior. If parameter readings were as expected then testing to failure began. Otherwise, corrections to the instrumentation were made.

During testing, a PC-based data acquisition system was used to monitor the measurements taken. Plots of the connection load versus third point deflection and connection load versus beam end rotation were also monitored as the test proceeded. The test beam was loaded under load control in five kip increments at each load point simultaneously until the plots showed evidence of beam yielding. Often, yielding was also evident because of flaking of the white wash. Once yielding began, the load was applied under deflection control in 0.25 in. increments of the third point deflection measurement closest to the test connection. The beam was loaded in this manner until failure.

In several of the tests, issues occurred which prevented the beam from being loaded to failure. Lateral buckling occurred in some tests due to inadequate bracing strength because of a much higher than expected beam strength. If the test beam did not have significant plastic deformation, additional bracing was added, and the beam was retested. Several test beams had yield strengths considerably above the nominal yield strength. These test beams would not yield within the capacity of the equipment provided; therefore, the testing sections had to be adjusted as described in Chapter 4.

3.7 Supplemental Tests

3.7.1 Coupon Tests

Tensile tests were performed on specimens of all steel material to determine the yield strength, ultimate strength, and elongation. Coupons were machined out of extra plate material provided by the steel fabricator. Coupons were also cut from each of the beam flanges after testing was complete. The specimens were cut near the simple support end of the beam where no yielding of the steel had occurred. All coupon specimens were prepared in accordance with ASTM A370-05 "*Standard Test Methods and Definitions for Mechanical Testing of Steel Products*."

The specimen was placed vertically in a universal testing machine with a 300 kip capacity. A 2 in. extensometer was attached at the midsection of the specimen to measure elongation during the test. The instrumentation was zeroed, and the specimen was clamped in the testing machine. Load was applied at a constant rate until the specimen showed evidence of yielding. During this interval the load applied to the specimen and the strain in the specimen were recorded by a PC-based data acquisition system. Once the specimen reach yield the extensometer was removed, and load was continuously applied until fracture of the specimen occurred. During this interval the load applied was recorded.

3.7.2 Bolt Shear Tests

Bolt shear tests were performed on a sampling of the test connection bolts to determine the individual bolt shear stress. These tests were also used to determine whether bearing could occur on the single plates prior to bolt shear. Two sizes of bolts were used during testing; A325-N 3/4" x 2-1/4" and A325-N 3/4" x 2-1/2". Bolts from the 2-1/4 in. lot were used in connection Tests 1, 2, and 4. Bolts from the 2-1/2 in. lot were used in connection Tests 3, 5, 6, 7, and 8.

A test apparatus shown in Figure 3.8 was designed to test the bolt shear strength and the bearing strength on the single plate. The plates in the test setup were taken from the same plate material as the single plates in the shear tab connections. Two tests were made using 1/2 in. plate cut from the same material used in Tests 5, 6, and 8. A third test was made using 3/8 in. plate cut from the same material used in Test 7.

Two bolts were placed in the testing apparatus. The bolts were placed through 13/16 in. drilled holes and screwed into two nuts welded together between the plates. Washers were used to ensure that the shear plane occurred in the threads. The testing apparatus was placed in the universal testing machine and shear force was applied in 5 kip increments until bolt shear rupture. Three sets of tests were completed on a total of 6 bolts.

To determine the individual bolt shear strength of the 2-1/4 in. bolts, direct shear was applied to connection Tests 1 and 2. As discussed in Chapter 4, no failure occurred in these connection tests during third point loading. Due to this, a direct shear test of these two connections could be completed subsequently. The 2-1/4 in. bolts were not tested in the same manner at the 2-1/2 in. bolts because all bolts in the lot were used in connection tests. To perform the direct shear tests, a hydraulic ram was placed nominally 2 ft. from the connection, and load was applied in 5 kip increments until failure of the connection bolts occurred.

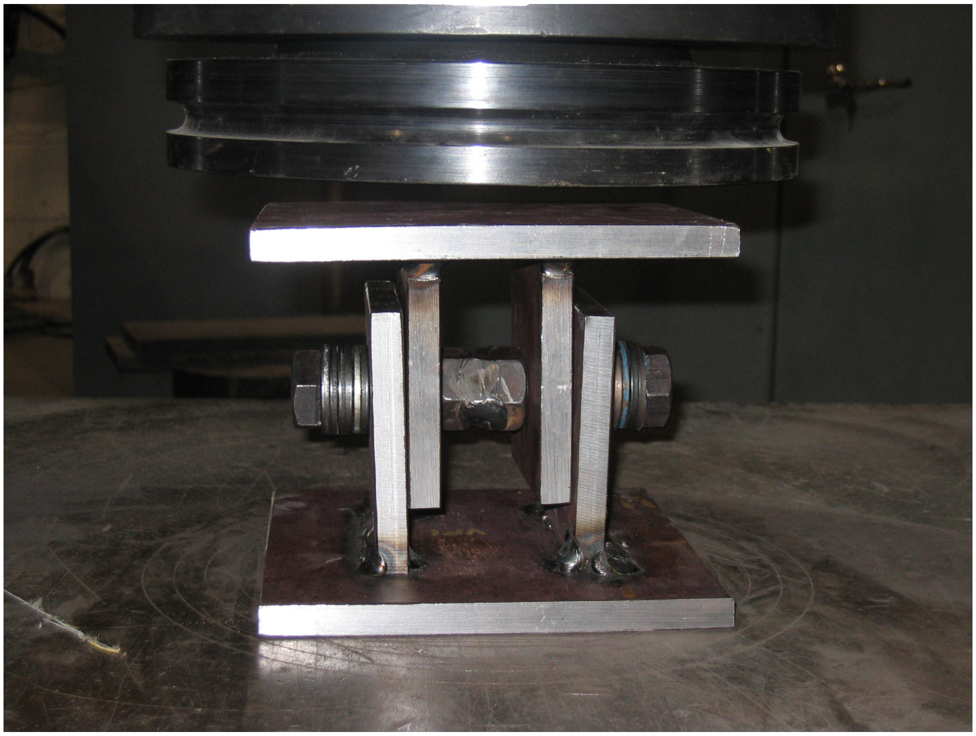

Figure 3.8 Bolt Shear Test

CHAPTER 4
EXPERIMENTAL RESULTS

4.1 Overview

The results of the connection and supplementary tests are discussed in the following sections. Detailed connection test summary reports are found in Appendix F. The connections were evaluated based on ultimate strength and rotational ductility. The summary reports include a description of test parameters, summary of peak values observed, various graphs showing specimen response, and photographs before and after testing.

The connection test results are separated into two groups. The first group contains the conventional configuration connections and consists of tests 1 through 4. The second group contains the extended configuration connections and consists of tests 5 through 8. Table 4.1 provides a summary of the test results. Supplementary test results are included in Section 4.4.

Table 4.1

Experimental Results

	Test	Test ID	Columns of Bolts	Bolts	Maximum Connection Shear (kips)	Maximum Connection Rotation (radians)	Connection Failure Mode
Conventional Configuration	1	3B1C - 3 - 3/8	1	3	81.0	0.032	-- [1]
	2	4B1C - 3 - 3/8	1	4	110	0.027	-- [1]
	3	5B1C - 3 - 3/8	1	5	146	0.030	Bolt Shear
	4	7B1C - 3 - 3/8	1	7	173	0.018	Bolt Shear
Extended Configuration	5a	6B2C - 4.5 -1/2	2	6	89.7	0.030	-- [2]
	5b	4B2C - 4.5 - 1/2	2	4	88.0	0.037	Weld Rupture
	6	10B2C - 4.5 - 1/2	2	10	200	0.026	Weld Rupture
	7	7B1C - 9 - 3/8	1	7	97.0	0.034	-- [1]
	8	10B2C - 10.5 - 1/2	2	10	97.0	0.035	-- [1]

[1] No connection failure occurred prior to beam failure.

[2] No connection failure occurred, for details see Section 4.3

4.2 Conventional Configuration Connections

Each of the conventional connections demonstrated similar behavior in the early stages of loading. The shear-rotation relationship was linearly elastic until the beam began to yield. Yielding in the beam was demonstrated by nonlinear behavior of the load versus vertical deflection plots and the shear versus rotation plots as well as by visible flaking of the whitewash from the beam flanges and web. As the beam continued to yield, rotations at the connection continued to increase. The shear versus rotation plots for the four conventional configuration connections are shown in Figure 4.1.

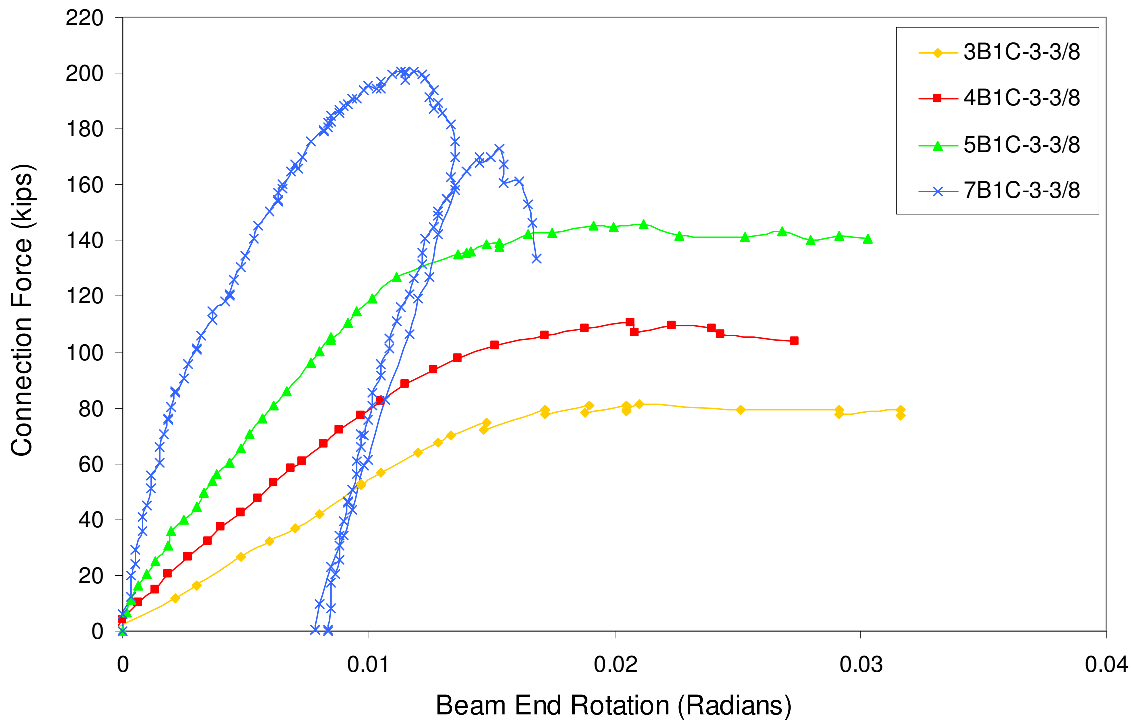

Figure 4.1 Shear versus Rotation for Conventional Connection Tests

Tests 1 and 2. In Tests 1 (3B1C-3-3/8) and 2 (4B1C-3-3/8) failure occurred in the test beam instead of in the connection. The test beams were designed to reach their plastic moment capacity at connection failure; therefore, the lateral braces were placed to ensure that the distance between them, L_b, was less than L_p, where L_p is the maximum distance between lateral braces required to prevent lateral torsional buckling up to the plastic moment capacity of the beam as defined by Equation F2-5 of the AISC Specification

(2005a). However, the test connections proved to have sufficient strength to allow the beams to yield significantly, and lateral torsional buckling occurred in the beams. When no more load could be resisted by the beam, testing was halted. The maximum forces resisted by the connections in Tests 1 and 2 were 81 kips and 110 kips with beam end rotations of 0.032 and 0.027 radians, respectively. In the remaining six tests lateral bracing was placed to ensure that L_b was less than L_{pb}, where L_{pb} is the maximum distance between lateral braces required to prevent lateral torsional buckling up to the plastic strength of the beam as defined in Equation A-1-7 of the AISC Specification (2005a).

After lateral torsional buckling occurred in Tests 1 and 2, the load was removed, and the connection was retested using a single hydraulic ram nominally placed 2 ft from the test column flange. This testing location was chosen to approximate a shear-only condition at the connection. Both connections failed in bolt shear where all bolts in the connections ruptured. The failure loads for Tests 1 and 2 were 81 kips and 119 kips, respectively. These direct shear results were used to approximate the shear strength of the bolts as discussed in Section 3.7.2.

Test 3. The additional bracing used in Test 3 (5B1C-3-3/8) prevented significant lateral buckling of the beam and allowed failure to occur in the connection. The failure mode was bolt shear during third point loading. The top bolt in the connection ruptured at a connection force of 146 kips.

Test 4. In Test 4 (7B1C-3-3/8), the test beam was loaded to a connection force of 200 kips and a beam end rotation of 0.013 radians. Beyond this point the capacity of the testing equipment would be exceeded. The low rotation was caused by the high yield strength of the test beam. The test setup was designed for a beam with an estimated yield strength of 55 ksi; whereas the actual yield was 62.6 ksi. The load was removed from the beam, and the beam section at midspan was reduced by cutting 1-1/2 in. from each side of the beam bottom flange for a 2 ft length at midspan. The reduced beam section decreased the plastic strength of the beam and allowed increased rotation within the capacity of the testing equipment. Failure occurred in the connection in the same manner as in Test 3; the top bolt of the connection ruptured at a connection force of 173 kips which was less than the initial loading of 200 kips.

Summary. One of the goals during testing was to impose a beam end rotation of 0.03 radians at connection failure. Tests 1 and 3 achieved the desired rotation. Test 2 achieved a rotation of 0.027 radians, and Test 4 achieved a rotation of only 0.018 radians at failure. The excessive beam strength in Test 4 contributed to this low rotation as discussed previously. Significant vertical deflection occurred in all four test beams. Tests 1 and 2 had significant lateral deformations. Local buckling occurred in the top flange at midspan in Test 2. No significant deformation occurred in the bolts or at the bolt holes in any of the connections. No yielding was observed in the single plate or the plate-to-column flange weld.

4.3 Extended Configuration Connections

Due to the fact that the connection parameters vary significantly in the extended configurations, each connection test is discussed individually. Tests 5a, 5b, and 6 consisted of two vertical columns of bolts with conventional a-distances. Test 7 consisted of one vertical column of bolts and an extended a-distance, and Test 8 had two vertical columns of bolts and an extended a-distance. The initial behavior of each connection was similar to the behavior of the conventional configuration connections. A linear elastic shear versus rotation curve was followed by an inelastic period of rotation prior to failure.

Tests 5a and 5b. The shear versus rotation behavior for Test 5 (6B2C-4.5-1/2) is shown in Figure 4.2. A connection force of 89.7 kips and a beam end rotation of 0.030 radians were achieved. However, the test was stopped prior to the beam reaching its full plastic moment. Once the design load and desired rotation were reached, the load was removed. The bottom two bolts were then removed from the connection, and load was reapplied to the beam, as Test 5b, until failure occurred due to weld rupture. During the retest, a connection force of 88.0 kips and a beam end rotation of 0.036 radians were achieved. Significant vertical deflection occurred in the beam. No deformation was observed in the bolts or at the bolt holes. No yielding was observed in the single plate.

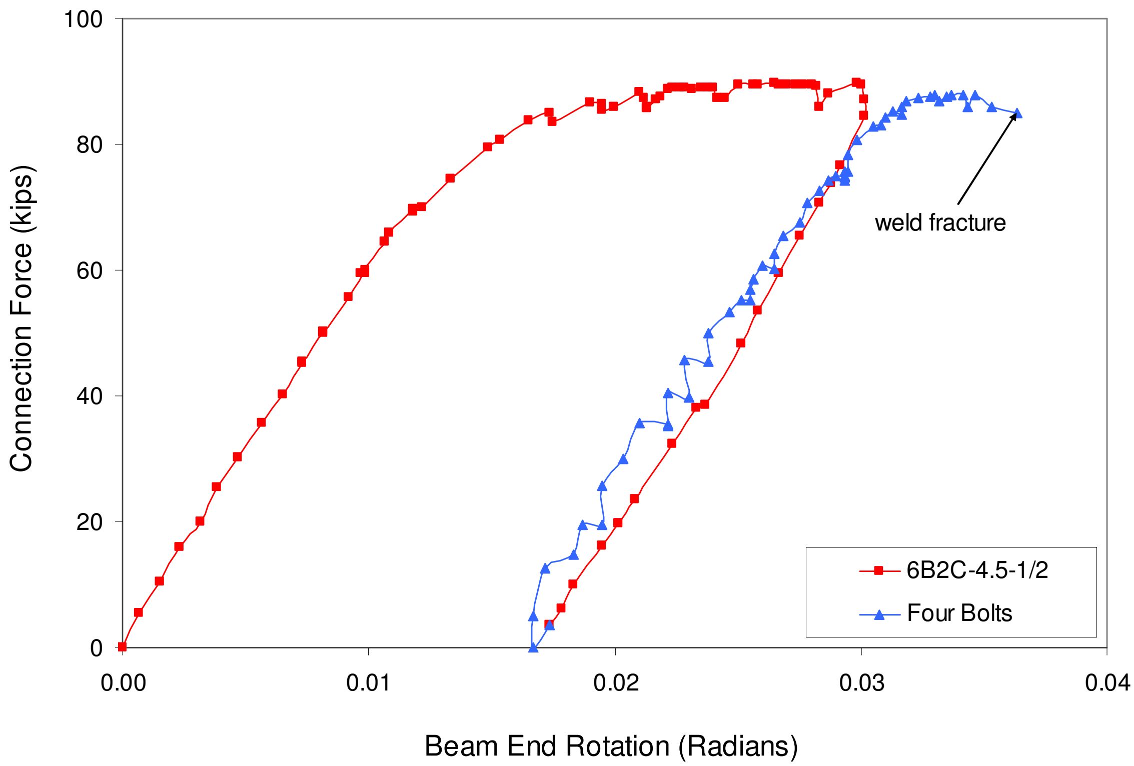

Figure 4.2 Shear versus Rotation for 6B2C-4.5-1/2

Test 6. In Test 6 (10B2C-4.5-1/2), load was applied to a connection force of 200 kips and a beam end rotation of 0.010 radians. Beyond this point the capacity of the testing equipment would be exceeded. Similar to Test 4 (7B1C-3-3/8), the low rotation was caused by the high yield strength of the test beam (61.5 ksi). The load was removed from the test beam, and the beam section at midspan was reduced by cutting 2-1/4 in. from each side of the beam bottom flange for a 2 ft length at midspan. The beam was reloaded to a connection force of 197 kips when testing was halted due to lateral buckling of the test beam. The beam was unloaded, and additional bracing was placed at the roller support end of the test beam. Load was again applied to the beam; however, determining the exact force at the connection was not possible because of vertical restraint introduced by the additional bracing. Failure occurred when the plate-to-column flange weld ruptured at a connection force of approximately 200 kips and a beam end rotation of 0.025 radians. The shear versus rotation behavior during the three loading cycles is shown in Figure 4.3.

Significant vertical and lateral deformations were visible in the test beam. No deformation was observed in the connection bolts, at the bolt holes, or in the single plate.

To check the original weld integrity, the single plate was re-welded to the column flange with the same size weld. Then load was applied at a nominal distance of 2 ft from the face of the column flange. The weld reached a force of 275 kips without failure. Due to the low weld strength determined from connections 5 and 6, the original single plates for connections 7 and 8 were cut off, ground, and re-welded to the column with welds sized 5/8 times the plate thickness. This size is consistent with the recommended weld size from the AISC Manual (2005b).

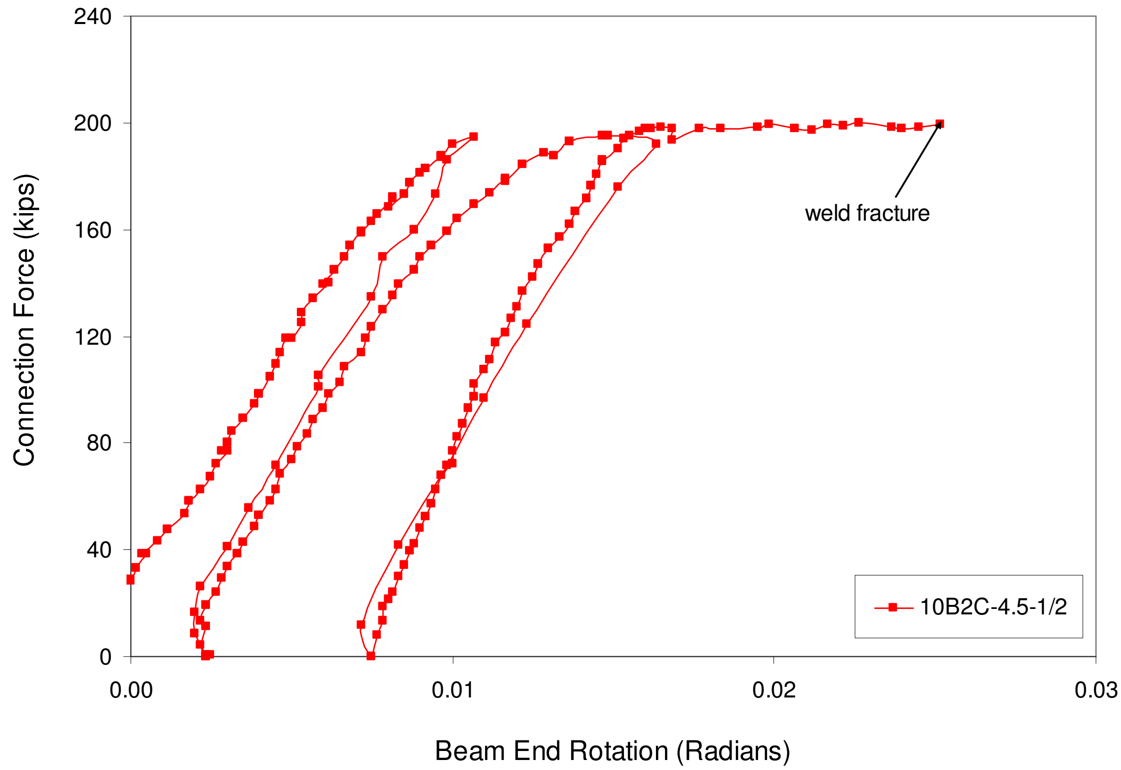

Figure 4.3 Shear versus Rotation for 10B2C-4.5-1/2

Test 7. Test 7 (7B1C-9-3/8) was a single column connection with an extended a-distance. Preliminary loading showed instability at the test connection due to twisting of the single plate and excessive moment at the column face demonstrated by sufficient column movement to bend the channel bracing the column against overturning. At this point, the connection had reached a force of 98 kips and a beam end rotation of 0.023 radians.

Additional lateral braces were added to the beam end at the connection, and a wide flange was placed in a similar manner to the bracing channel but at column mid-height. The connection was reloaded to a force of 97 kips and a beam end rotation of 0.034 radians. Failure occurred when the test beam laterally buckled at midspan and no additional force could be applied to the connection.

The shear versus rotation behavior during the loading cycles is shown in Figure 4.4. Vertical and lateral deformation was evident in the test beam. Yielding was observed in the beam around the top two bolt holes and the bottom bolt hole. Yielding was also evident in the single plate at the bottom near the beam side. Figures 4.5 and 4.6 show photographs of the tested connection.

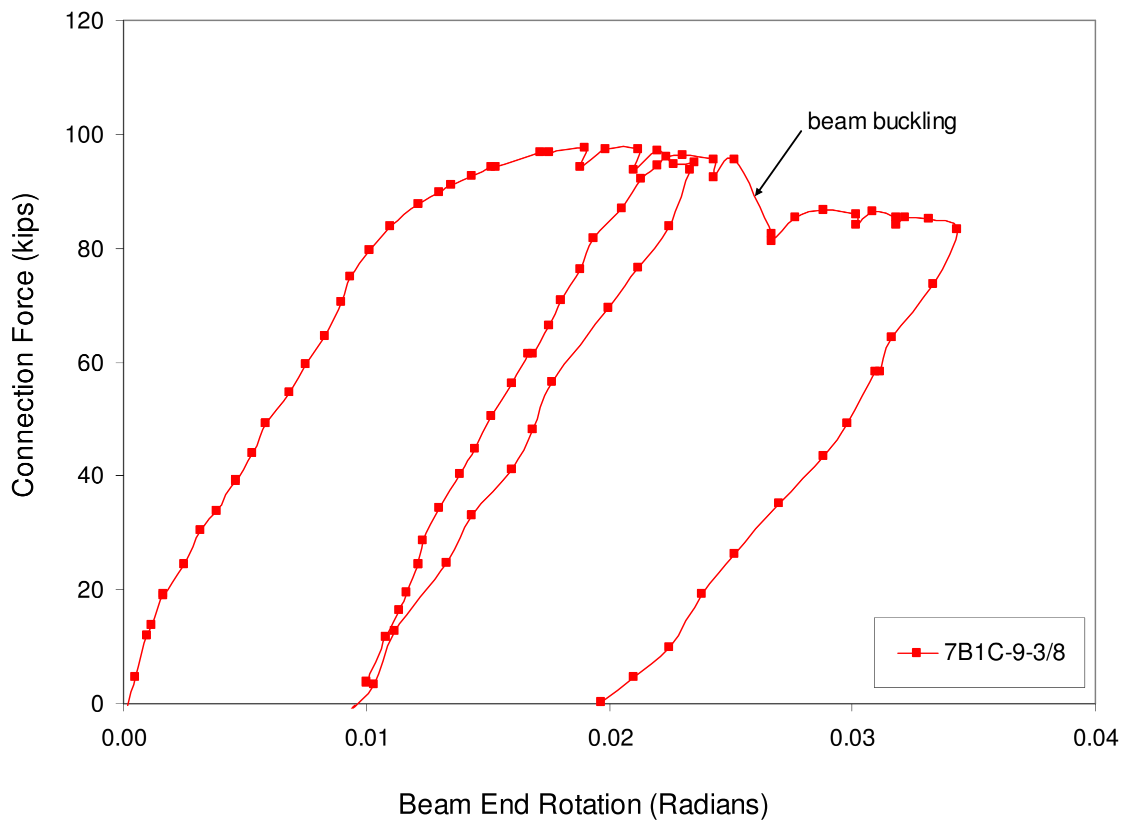

Figure 4.4 Shear versus Rotation for 7B1C-9-3/8

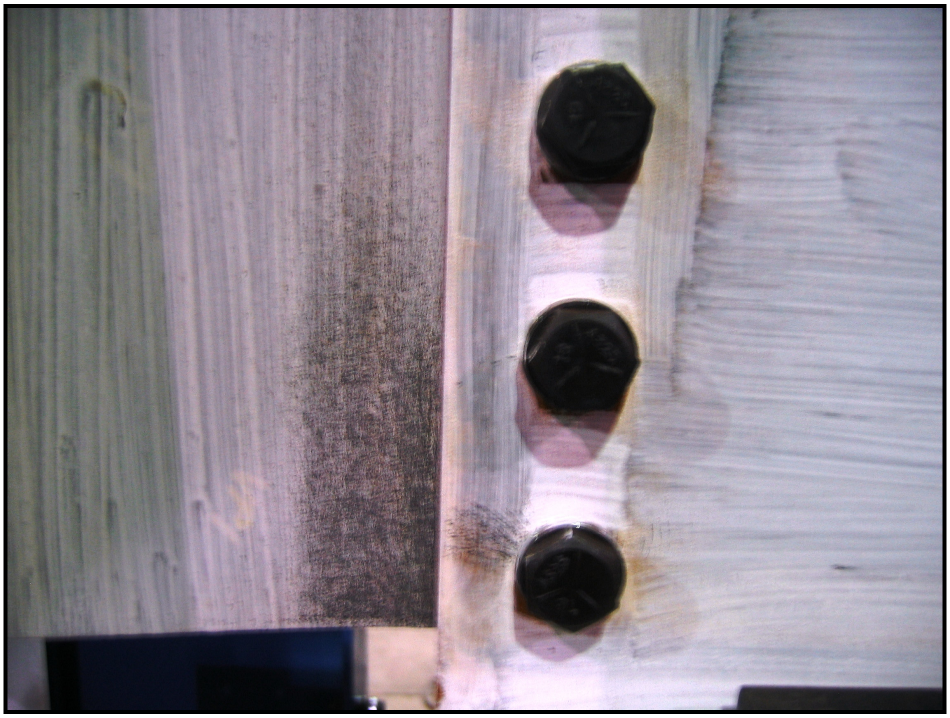

Figure 4.5 Test 7 (7B1C-9-3/8) Plate Yielding

Figure 4.6 Test 7 (7B1C-9-3/8) Yielding in Beam at Bolt Holes

Test 8. Test 8 (10B2C-10.5-1/2) utilized the same additional bracing as Test 7. The connection was loaded in four cycles to demonstrate the inelastic behavior of the connection and test beam. Failure occurred in the test beam by local buckling of the web at midspan. At this point, no additional force could be applied to the connection. A maximum connection force of 97 kips and a maximum beam end rotation of 0.035 radians were achieved. Significant vertical and lateral deformations were evident in the test beam. Yielding at the bottom of the plate was also observed. Figure 4.7 is a photograph of the tested connection showing plate yielding. The shear versus rotation behavior during the loading cycles is shown in Figure 4.8.

Figure 4.7 Test 8 (10B2C-10.5-1/2) Plate Yielding

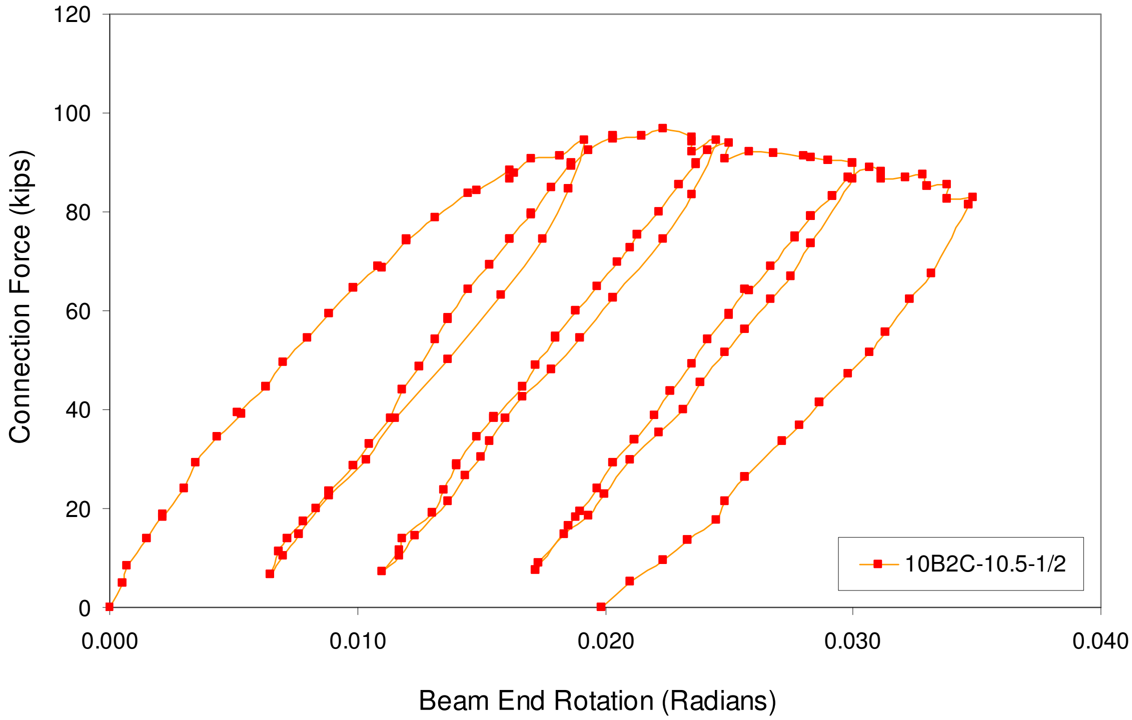

Figure 4.8 Shear versus Rotation for 10B2C-10.5-1/2

4.4 Supplementary Test Results

4.4.1 Coupon Tests

Coupon tests were conducted on test beam and single plate material as described in Section 3.7.1. Table 4.2 summarizes the results of these tests. The table includes the yield strength of the material determined from a 0.2 percent offset of the stress versus strain relationship, the ultimate strength, and the percent elongation based on an 8 in. gauge length. All material strengths exceeded the nominal material strengths found in the AISC Specification (2005a). The single plate strengths were significantly higher than the 50 ksi nominal strength.

Table 4.2

Tensile Coupon Test Results

Material Tested	Corresponding Tests	Tension Test Specimen Dimensions (in.)	Specimen Cross Sectional Area ($in.^2$)	Yield Load (kips)	Ultimate Load (kips)	Yield Stress, F_y (ksi)	Ultimate Stress, F_u (ksi)	Elongation 8 in. %
3/8 in. Plate	7B1C-9-3/8	0.384 x 1.495	0.574	39.8	55.3	69.3	96.3	20
3/8 in. Plate	3B1C-3-3/8, 4B1C-3-3/8, 5B1C-3-3/8, 7B1C-3-3/8	0.379 x 1.492	0.565	37.3	55.1	68.1	97.5	28
1/2 in. Plate	6B2C-4.5-1/2, 10B2C-4.5-1/2	0.502 x1.496	0.751	51.2	73.4	68.2	97.7	22
1/2 in. Plate	10B2C-10.5-1/2	0.503 x 1.495	0.752	51.5	72.9	68.5	97.0	27
W18x55	3BIC-3-3/8	0.620 x 1.495	0.927	51.0	66.7	55.0	72.0	36
W24x76	4B1C-3-3/8	0.659 x 1.498	0.987	55.8	73.9	56.5	74.9	29
W24x76	5B1C-3-3/8	0.660 x 1.498	0.989	56.9	74.4	57.6	75.3	28
W30x108	7B1C-3-3/8	0.707 x 1.498	1.059	66.3	84.8	62.6	80.1	25
W18x55	6B2C-4.5-1/2	0.630 x 1.499	0.944	55.6	73.3	58.9	77.6	27
W30x108	10B2C-4.5-1/2	0.707 x 1.504	1.063	65.4	84.3	61.5	79.3	31
W24x62	7B1C-10-3/8	0.576 x 1.500	0.864	50.1	66.6	58.0	77.1	27
W24x62	10B2C-11.5-1/2	0.570 1.503	0.857	50.0	66.5	58.4	77.6	27

4.4.2 Bolt Shear Tests

Shear tests were performed on the test connection bolts to determine their shear rupture strength as described in Section 3.7.2 with the results shown in Table 4.3. The average bolt shear stress was calculated by dividing the maximum shear force by the bolt area based on a nominal bolt diameter, 3/4 in., and the number of bolts in the test. The average bolt stresses shown were used as the measured shear strength of the connection bolts. A comparison is shown between the experimental and the nominal stresses. The experimental stresses found exceeded the nominal stresses by 2 percent for the 2-1/4 in. bolts and by 17 percent for the 2-1/2 in. bolts.

Table 4.3
Bolt Shear Test Results

<table>
<tr><th>Type of Bolt</th><th>Diameter x Shank Length
(in.)</th><th>Corresponding Tests</th><th>Plate Thickness
(in.)</th><th>Bolts</th><th>Maximum Shear Force
(kips)</th><th>Bolt Shear Stress
(ksi)</th><th>Average Shear Stress
(ksi)</th><th>Nominal Bolt Shear Stress
(ksi)</th><th>Experimental Stress/Nominal Stress</th></tr>
<tr><td>A325-N</td><td>3/4 x 2-1/4</td><td rowspan="2">3B1C-3-3/8,
4B1C-3-3/8,
7B1C-3-3/8</td><td>3/8 in.</td><td>3</td><td>81.0</td><td>61.1</td><td rowspan="2">61.4</td><td>60</td><td>1.02</td></tr>
<tr><td>A325-N</td><td>3/4 x 2-1/4</td><td>3/8 in.</td><td>4</td><td>109</td><td>61.7</td><td>60</td><td>1.02</td></tr>
<tr><td>A325-N</td><td>3/4 x 2-1/2</td><td rowspan="2">5B1C-3-3/8,
6B1C-4.5-1/2,
10B2C-4.5-1/2,
10B2C-10.5-1/2</td><td>1/2 in.</td><td>2</td><td>61.2</td><td>69.3</td><td rowspan="3">70.2</td><td>60</td><td>1.17</td></tr>
<tr><td>A325-N</td><td>3/4 x 2-1/2</td><td>1/2 in.</td><td>2</td><td>61.9</td><td>70.1</td><td>60</td><td>1.17</td></tr>
<tr><td>A325-N</td><td>3/4 x 2-1/2</td><td>7B1C-9-3/8</td><td>3/8 in.</td><td>2</td><td>63.1</td><td>71.4</td><td>60</td><td>1.17</td></tr>
</table>

The individual bolt shear tests were also used to examine the ability of the single plates used in the connection tests to allow bolt hole deformations prior to bolt failure. The test apparatus was made from the same plate material as the single plates in the shear tab connections. As detailed in Table 4.3, Tests 1 and 2 were made using 1/2 in. plate cut from the same material used in Tests 5, 6, and 8. Test 3 was made using 3/8 in. plate cut from the same material used in Test 7. The apparatus was loaded until bolt shear failure. The plates were then examined to determine if bolt hole deformation had occurred. As shown in Figure 4.9 and 4.10 no significant bolt hole deformation occurred when the bolts were tested to failure.

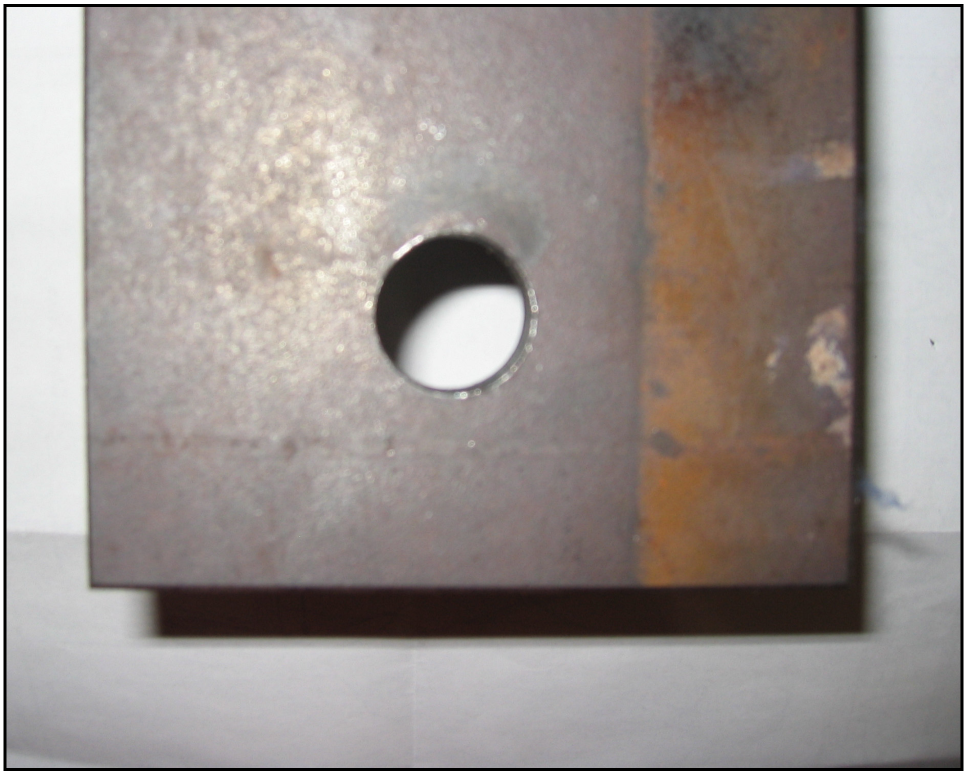

Figure 4.9 Bolt Hole Deformation, 1/2 in. Plate

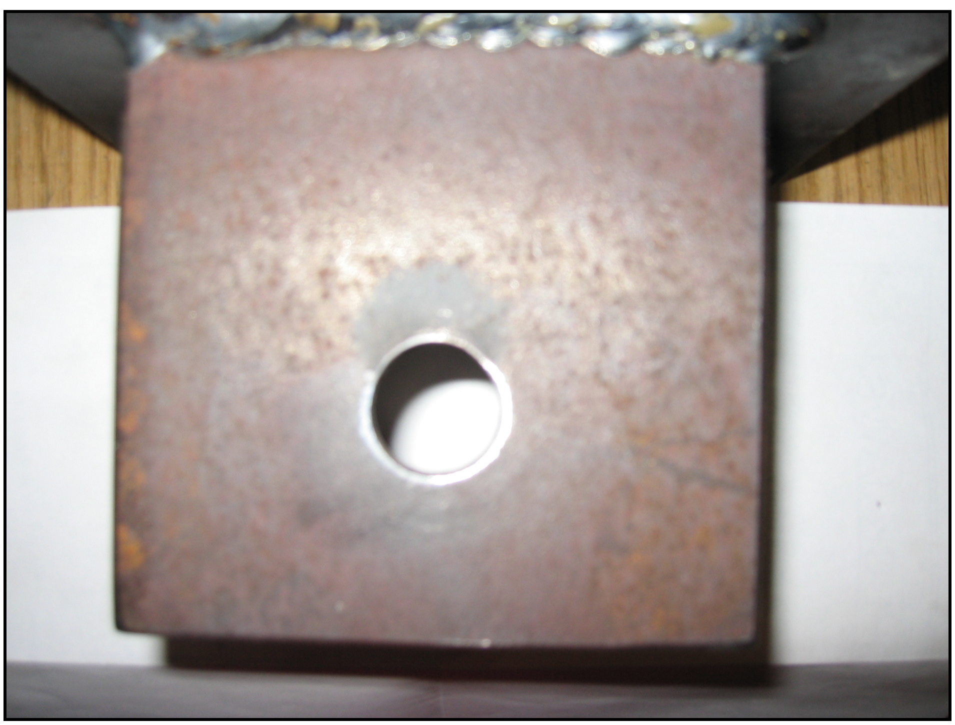

Figure 4.10 Bolt Hole Deformation, 3/8 in. Plate

CHAPTER 5
ANALYSIS OF EXPERIMENTAL RESULTS

5.1 Overview

The experimental connection results discussed in Chapter 4 are analyzed in the following sections. The experimental results are compared to predicted strengths determined according to the procedure in the AISC 13th Edition Manual (2005b) using nominal material properties and measured material properties. Connection failure modes are analyzed, and the rotational behavior of the various connection components is discussed.

5.2 Predicted Strength

Table 5.1 and 5.2 show comparisons of experimental values to nominal and predicted values for the limit states in the AISC procedure (AISC, 2005b). A check of bearing on the beam web is also shown. A check of the bolt group strength calculated as recommended by AISC, but excluding the bolt group action factor (0.8) is included for comparison. The nominal values are based on the strengths given in the AISC Specification (2005a) for A992 beam material, A572 Gr.50 plate material, and A325-N type bolts. The predicted values are based on measured material properties listed in Tables 4.2 and 4.3. Strength reduction (Φ) factors were excluded from all calculations.

Table 5.1

Summary of Connection Limit State Values, Conventional Conections

		3B1C-3-3/8		4B1C-3-3/8		5B1C-3-3/8		7B1C-3-3/8	
		Nominal	Measured [1]	Nominal	Measured [1]	Nominal	Measured [1]	Nominal	Measured [1]
Yield Strength, Fy (ksi)	Plate Steel	50.0	68.1	50.0	68.1	50.0	68.1	50.0	68.1
	Beam Steel	50.0	55.0	50.0	56.5	50.0	57.6	50.0	62.6
Ultimate Strength, Fu (ksi)	Plate Steel	65.0	97.5	65.0	97.5	65.0	97.5	65.0	97.5
	Beam Steel	65.0	72.0	65.0	74.9	65.0	75.3	65.0	80.1
Connection Limit States (kips)	Bolt Shear Rupture (including bolt group action factor)	64.0	65.1	84.8	86.8	106	124	148	152
	Bolt Eccentricity	Neglected	Neglected	Neglected	Neglected	Neglected	Neglected	Neglected	Neglected
	Bolt Shear Rupture (excluding bolt group action factor)[2]	80.0	81.4	106	109	133	155	186	190
	Plate Shear Yielding	96	130	129	176	163	222	231	314
	Plate Shear Rupture	86	129	117	176	148	222	210	315
	Plate Block Shear	100	154	131	196	162	242	224	334
	Plate: Bearing	113	169	156	234	200	300	288	432
	Beam: Bearing	137	152	206	237	257	298	446	550
Experimental Values	Failure Mode	--- [3]		--- [3]		Bolt Shear		Bolt Shear	
	Maximum Rotation (radians)	0.032		0.027		0.030		0.018	
	Maximum Connection Shear (kips)	81.0		110		146		200	
	Experimental/Design Strength	1.27		1.30		1.38		1.35	
	Experimental/Predicted Strength	1.24		1.27		1.18		1.32	

[1]Material strength values are taken from tensile tests and bolt shear tests performed at Virginia Tech.

[2]Not a permitted limit state according to procedure in the AISC Manual (2005b)

[3]Beam failure occurred prior to connection failure.

Table 5.2
Summary of Connection Limit State Values, Extended Configuration

		6B2C-4.5-1/2		10B2C-4.5-1/2		7B1C-10-3/8		10B2C-11.5-1/2	
		Nominal	Measured [1]	Nominal	Measured [1]	Nominal	Measured [1]	Nominal	Measured [1]
Yield Strength, Fy (ksi)	Plate Steel	50.0	68.2	50.0	68.2	50.0	69.3	50.0	68.5
	Beam Steel	50.0	58.9	50.0	61.5	50.0	58.0	50.0	58.4
Ultimate Strength, Fu (ksi)	Plate Steel	65.0	97.7	65.0	97.7	65.0	96.3	65.0	97.0
	Beam Steel	65.0	77.6	65.0	79.3	65.0	77.1	65.0	77.6
Connection Limit States (kips)	Bolt Shear Rupture (including bolt group action factor)	60.0	70.0	136	159	73.0	84.6	72.0	84.1
	Bolt Eccentricity	4.5	4.5	4.5	4.5	9.0	9.0	10.5	10.5
	Bolt Shear Rupture (excluding bolt group action factor)[2]	75.0	87.5	170	199	91.3	106	90.0	105
	Plate Shear Yielding	128	174	218	297	231	319	218	298
	Plate Shear Rupture	115	172	197	297	210	311	197	295
	Plate Block Shear	202	304	285	427	224	332	285	426
	Plate: Bearing	300	451	534	802	289	426	534	797
	Beam: Bearing	274	327	638	778	352	417	503	601
Experimental Values	Failure Mode	Weld Rupture		Weld Rupture		--- [3]		--- [3]	
	Maximum Rotation (radians)	0.030		0.025		0.034		0.035	
	Maximum Connection Shear (kips)	89.0		200		97.0		97.0	
	Experimental/Design Strength	1.48		1.47		1.33		1.35	
	Experimental/Predicted Strength	1.27		1.26		1.15		1.15	

[1]Strength values are taken from machined coupon tests performed at Virginia Tech.
[2]Not a permitted limit state according to procedure in the AISC Manual (2005b)
[3]Beam failure occurred prior to connection failure.

The nominal and predicted controlling limit state was bolt shear for all connections. The AISC (2005b) recommended bolt shear strength for conventional configurations with less than 10 bolts includes the bolt group action factor and neglects connection eccentricity. The AISC (2005b) recommended bolt shear strength for extended configurations includes both eccentricity and the bolt group action factor. All connections reached experimental shear values which exceeded the design strengths by 36 percent on average, with a standard deviation of 0.08. All connections also reached experimental shear values which exceeded the predicted strengths by 23 percent on average, with a standard deviation of 0.06. The experimental strengths found for the extended configuration connections indicate that the BGAF most likely does not apply. The predicted strength values could be increased by 1.25, as is already done for the conventional configuration connections. Since not all connections failed, the maximum shear values reported provide a lower bound to the connection strength.

5.3 Failure Modes

5.3.1 Bolt Shear Rupture

The predicted failure mode for all connections was bolt shear rupture. Only connections 3 (5B1C-3-3/8) and 4 (7B1C-3-3/8) failed in this mode. The top bolt in these two connections ruptured at the maximum connection shear indicated in Table 5.1. Top bolt shear rupture, as opposed to all the connection bolts rupturing simultaneously, indicates eccentricity in the connection. Table 5.3 evaluates four combinations of assumptions to predict the bolt shear strength of a single plate connection.

Table 5.3

Bolt Shear Strength Predictions

Test No.	**Test ID**	**Predicted Values**				**Experimental Value**
		1	2	3	4	
		Including eccentricity, e=a, Including BGAF	Including eccentricity, e=a, Excluding BGAF	Excluding eccentricity, Including BGAF	Excluding eccentricity, Excluding BGAF	
3	5B1C-3-3/8	121 kips	151 kips	124 kips	155 kips	146 kips
4	7B1C-3-3/8	132 kips	164 kips	152 kips	189 kips	200 kips

[1] BGAF, Bolt Group Action Factor

The AISC 3rd Edition Manual (2001) design procedure for single plate connections includes both connection eccentricity and the bolt group action factor as seen in Column (1). This method gives the most conservative strength prediction. The AISC 13th Edition Manual procedure (2005b) includes the bolt group action factor but neglects the connection eccentricity, Column (3). This method is still conservative but less so than the 3rd Edition Manual procedure. Excluding both the eccentricity and the bolt group action factor, Column (4), appears to be slightly unconservative for Test 3; however, if a strength reduction factor is included this prediction will also give a conservative value. Including the eccentricity equal to the a-distance and excluding the bolt group action factor, Column (2), provides the closest prediction. This method is also consistent with the theoretical behavior of the connection since single plate shear connections have been shown to have some amount of eccentricity through experimental studies, but the bolt group action factor does not apply to this type of connection. All bolt group shear strength predictions would be conservative predictors with the inclusion of the strength reduction factor for bolt shear of 0.75.

5.3.2 Weld Rupture

In connections 5 (6B2C-4.5-1/2) and 6 (10B2C-4.5-1/2) failure occurred in the welds. Both welds ruptured near the leg of the weld attached to the single plate. The weld failure for Test 5 is shown in Figure 5.1. Failure at this point in the weld, as opposed to through the throat of the weld, indicates eccentricity in the connection. Both connections failed at connection shears lower than the nominal direct shear strength of the welds. This could also support the idea that the connections resisted eccentric shear. However, the steel fabricator, Cives Steel Company, used lower amperage than usual because these welds were smaller than those typically made in their shop practice. This may have adversely affected the welds and led to lower strengths than expected (Muir, 2006a). Table 5.4 summarizes the weld shear strengths with and without eccentricity for connections 5 and 6.

Both connections that failed by weld rupture were designed with a weld size equal to 1/2 times the single plate thickness as opposed to the AISC Manual procedure (2005b) which requires a weld size of 5/8 times the single plate thickness. Connections 7 and 8 were designed with the AISC Manual procedure (2005b) required weld strength, and neither weld ruptured.

Figure 5.1 Weld Rupture at Column Flange (6B2C-4.5-1/2)

Table 5.4

Weld Rupture Strength Predictions

Test No.	Test ID	Weld Size (in.)	Weld Length (in.)	Nominal Values[1]		Experimental Value
				Direct Shear strength	Eccentric shear strength, e = a	
5	6B2C-4.5-1/2	1/4	8.5	126 kips	82 kips	89 kips
6	10B2C-4.5-1/2	1/4	14.5	215 kips	180 kips	200 kips

[1]Weld rupture strength taken as 70 ksi

5.4 Rotational Behavior

As indicated in Table 5.1, five of the test connections reached the target beam end rotation of 0.030 radians, two reached a beam end rotation greater than 0.025 radians, and one reached a beam end rotation of only 0.018 radians. Through previous experimental work (Richard et. al., 1980; Astaneh, 1989c), it was determined that the rotational ductility of single plate shear connections is allowed by yielding of the single plate and deformation at the bolt holes. However, in the tested connections yielding in the single plate was observed only in connections 7 and 8, and no deformation of the bolt holes was observed.

To ensure ductility via plate yielding and deformation of the bolt holes, Richard et al. (1980) proposed an experimentally based maximum plate thickness, $t_p \leq d_b/2+1/16$ in. This value was shown to allow considerable but tolerable deformation at the bolt holes prior to bolt shear rupture. The conventional configuration connections in the new AISC procedure (2005b) utilize Richard's maximum plate thickness. However, this recommendation was based on experimental work using 36 ksi plates as opposed to the 50 ksi plates used in the current research. No yielding in the plate was evident in Tests 1 through 4 of the current research. No bolt hole deformation was observed in the plates in any of the tests.

In the new AISC procedure (2005b), the maximum plate thickness is determined differently for extended configuration connections. Instead of the $t_p \leq d_b/2 + 1/16$ in. recommendation, the maximum plate thickness is determined by ensuring that the moment capacity of the single plate is less than the pure moment capacity of the bolt group. Yielding was observed in the plates in connections 7 and 8 as shown in Figures 4.5 through 4.7. However, no yielding occurred in connections 5 and 6. The two connections in which yielding was evident have extended a-distances which should lead to increased eccentricity and moment. The connections also reached a higher beam end rotation than the other connections.

The lack of deformation at the bolt holes in the connection tests is supported by the supplemental bolt shear tests, whose results are described in Section 4.4.2. No evidence of bolt hole deformation was evident prior to bolt rupture under direct shear loading for the 3/8 in. or the 1/2 in. plate material. This was as expected since the connection bearing capacity is greater then the connection bolt capacity as shown in Tables 5.1 and 5.2.

CHAPTER 6
CONCLUSIONS

6.1 Summary

The purpose of this study was to examine the behavior of single plate shear connections designed according to the AISC 13th Ed. Manual procedure (2005b). Four conventional configuration and four extended configuration connections were tested. The bolt group configuration and the connection a-distances were varied between the tests. The results of the tests were compared to predicted values for the connection strength. Predicted limit state strengths were determined from the AISC 13th Ed. procedure (2005b) using measured material strengths for the beam, plate, and connection bolts.

The rotational ductility of the single plate shear connection was also examined for a target rotation of 0.030 radians at the plastic moment capacity of the beam. The connection characteristics by which this rotational ductility was achieved were investigated. Supplemental tests were completed to determine the material strengths of the connection elements. The plate and beam strengths were found through coupon tests. The bolt shear strength was determined from direct shear tests of the connection bolts.

6.2 Conclusions

6.2.1 Connection Strength

The experimental results indicate that the AISC 13th Ed. procedure (2005b) conservatively predicts the ultimate strength of both the conventional configuration and the extended configuration single plate shear connections. However, it is more acturate than the previous AISC method (AISC, 2001). Connection failure was not achieved in all tests; therefore, the maximum shear values presented in this study reflect a lower bound strength prediction.

6.2.2 Connection Bolt Strength

The bolt group shear strengths were conservatively predicted by the AISC 13th Edition method. However, this method was more accurate than the previous AISC method (AISC, 2001). The most accurate method for bolt shear strength prediction determined in this study was found by excluding the bolt group action factor (0.8) and including an eccentricity equal to the connection a-distance. This method is not currently used in the AISC procedure (2005b) because it would necessitate an apparent violation of the nominal bolt shear strength found in the AISC Specification (2005a). Unless future specifications included information regarding the bolt group action factor, establishing less conservative calculations for common use would be difficult.

6.2.3 Weld Strength

A weld size of 1/2 times the plate thickness was used in Tests 1 through 6 in an attempt to reduce the current AISC recommendation (AISC, 2005b) of 5/8 times the plate thickness. The recommended weld size was determined by ensuring that the plate would yield prior to weld rupture. Tests 1 through 4 showed no evidence of plate yield, and no weld rupture occurred. However, weld rupture occurred in Tests 5 and 6, but no plate yield was observed. Due to atypical welding procedures used to fabricate the welds, it was not possible to confirm if the weld rupture was due to fabrication issues or the weld size itself. Until further testing is completed on connections with a reduced weld size, it is recommended that the current AISC requirement (AISC, 2005b) be used for both conventional and extended configuration connections.

6.2.4 Rotational Ductility

Rotational ductility in single plate shear connections is typically obtained by elongation of the bolt holes in the plate or beam and/or yielding of the plate. However, the dimensional requirements to ensure that the ductility is provided are experimentally based on testing using A36 plates. No elongation of the bolt holes was found in this series of tests, where measured plate yield stresses were over 60 ksi. Plate yielding was observed in only two of the eight tests. It is recommended that a series of tests be performed which vary the bolt diameter-to-plate thickness ratio to determine a maximum allowable plate thickness for nominal 50 ksi plate material.

6.2.5 Connection Bracing

The connections tested with extended a-distances required additional bracing at the beam end near the connection due to twisting of the single plate. If a connection in a building is inadequately braced, similar instability could occur. It is recommended that bracing be required at the connection if the a-distance is large. In many structures, this bracing requirement may be satisfied by the presence of a composite slab. Future tests should determine the limiting a-distance for the necessity of this requirement.

6.3 Suggestions for Future Research

Additional research is required to develop recommendations to the current AISC design method (2005b). If a reduced weld size is desired, a series of tests varying the weld size-to-plate thickness ratio should be completed. All welds should be made using the same procedure to ensure that any variation in behavior was due to the ratio and not the weld quality.

The rotational ductility of nominal 50 ksi plates should be examined to establish a plate thickness-to-bolt diameter ratio which would provide sufficient connection ductility for the required beam end rotation. Consideration should also be given to the fact that actual plate yield stresses are significantly above nominal values. Past research has found that the current requirement is sufficient for 36 ksi plates. The required rotation could also be achieved by studying the beam web thickness-to-bolt diameter ratio.

This series of tests concentrated on single plate shear connections to rigid supports. Additional work should be done to confirm the validity of the AISC design procedure (2005b) for connections to flexible supports.

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