ABSTRACT

CREECH, DUSTIN DWAYNE. Behavior of Single Plate Shear Connections with Rigid and Flexible Supports. (Under the direction of Emmett Sumner, Ph.D., P.E.)

Single plate framing connections are a type of shear connection which must sustain both shear and moment transferred from the supported beam. Commonly found in both rigid (beam-to-column flange) and flexible (beam-to-girder web) configurations, the single plate connection has proven to be both economical and easy to erect. A specified design method for single plate connections is presented in the AISC LRFD 3rd Edition Manual (AISC 2003). The design method calculates for associated limit states or failure modes and accounts for eccentric shear created by the moment sustained by the connection.

Overly conservative tabulated design strengths, which lead to heavier, more costly, connections, has emphasized the need for research to improve upon the current design procedure. The focus of this research is to address the conservative nature of the currently specified design method.

To create a basis for comparison to theoretically calculated design strengths, a total of ten full-scale tests were conducted. Tests incorporated both rigid and flexible support conditions, both standard and short-slotted bolt holes, and connections consisted of various numbers of bolts. Differing from previous research efforts, this series of research incorporated a simulated slab restraint with flexible supported tests. In order to evaluate experimental results and remove unknowns, supplemental tests were conducted to determine the material properties of components used in testing.
Major goals accomplished in this research included the investigation of connection limit states and component behavior. Focus was given to qualifying rotational behavior and quantifying the extent that eccentric shear is experienced by the connection as this is felt to be a potential cause of the overly conservative tabulated design values. In addition, comparisons of the AISC design method were made to design methods proposed by other researchers and design methods specified in other countries. All investigations focused upon improving the currently accepted AISC design procedure for single plate framing connections.
BEHAVIOR OF SINGLE PLATE SHEAR CONNECTIONS WITH RIGID AND FLEXIBLE SUPPORTS

by

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In dedication to
-Renaldo Lovisa, Jr.-

…My stepfather, Rennie, has been my role model giving me both inspiration to succeed and direction in life. Renaldo graduated from North Carolina State University in 1972 with a Bachelor of Science Degree in Civil Engineering. I am proud to follow in his footsteps…
My name is Dustin Creech, but my friends call me Dusty (and dusty I am after a long day of research in the lab). I was born and raised in Raleigh, North Carolina. My hobbies and interests include kite flying at the North Carolina coast, drawing, remote control car racing, or simply enjoying a drive through the countryside while listening to country or bluegrass music. Throughout my life, I have had an excellent upbringing where the importance of family and moral values were always stressed. From an early age, I was taught the importance of education, but I remember my first day of school…I cried. What I did not realize then was what potential and rewarding life experiences education would bring. As a senior in high school, I adopted this life motto:

‘Obstacles are what you see when you take your sight off your goals…
From high school and into college, I have applied myself to my studies graduating from North Carolina State University (NCSU) in December of 2002 with a Bachelor of Science degree in Civil Engineering. In the spring of 2003, I continued my education by pursuing my graduate degree as a Master of Science student in the Civil Engineering curriculum at NCSU. As an MSCE student, I have been fortunate to work jointly as a teaching and research assistant. Given the chance to instruct our department’s structures and measurements laboratory, I had the opportunity to explore my passion for teaching and helping others. I have always believed in helping others who are experiencing difficulties with which I have already experienced or have knowledge of. Because of that reason that is where I find my strength, my stride. To teach is one of the most rewarding experiences I have ever had. Since I began my graduate degree, I have been a teaching assistant for both graduate and undergraduate courses. All the while, I have worked as a research assistant at the Constructed Facilities Laboratory gaining such great hands on experience completing my thesis work.

Ultimately, I take pleasure from the simple things in life, but enjoy the challenge of applying myself and succeeding at the task at hand…striving to make those proud…
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There are many people who are owed countless thanks for their contribution to this research project. Foremost, senior laboratory technician, Mr. Jerry Atkinson, laboratory electronics specialist, Mr. Bill Dunleavy, and former laboratory technician Matt Vorys deserve particular mention as they assisted me considerably in the completion of the experimental portion of my research work at the Constructed Facilities Laboratory. Lucas Gelo, MCE is thanked for his work and assistance in conducting the bolt shear tests discussed in this thesis. Other students who directly contributed to this project include but are not limited to: Donlawit Aariyasajjakorn, John Acker, Thomas Bruebach, Charles Johnson, Jason Patrick, Jamal Tillman, Todd Whisenhunt, and Brandon Zent.

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CHAPTER 1  INTRODUCTION

1.1. OVERVIEW

Classified as a type of shear connection, single plate framing connections are idealized as a pinned connection subjected to shear forces only. Although this connection is thought to be relatively simplistic, the behavior of the single plate connection is complex. One inherent reason for the complexity is because the connections do sustain some moment as created by beam end rotation. Another reason for the complexity is the rotational characteristics of the single plate connection are affected by the configuration of the supporting member, the bolt hole type, and the number of bolts used in the connection.

Illustrated in Figure 1.1 is a detail of a single plate framing connection with rigid support condition where the supported beam frames into the flange of a supporting column. The connection is facilitated by a single plate, which is welded to the face of the column flange. The supported beam is connected to one side of the single plate with high strength bolts.

![Figure 1.1 Single Plate Connection Detail (Rigid Support)](image-url)
A considerable amount of research spread out over many years has been conducted to understand the behavior of single plate framing connections. Throughout the various research efforts, improvements in test setups and the application of load and rotation have been made to accurately reproduce field conditions. In the late 80’s and early 90’s, Abolhassan Astaneh-ASL headed a research effort which focused on testing single plate framing connections in a means that imposed shear and rotation upon the test connection. The research work was accepted by the American Institute of Steel Construction and specified as the standard design method for single plate connections in the AISC LRFD 3rd Edition Manual (AISC 2003). This research sought to investigate the behavior of single plate framing connections and verify the currently accepted design procedure listed in the AISC manual. Foremost, the current AISC design method was under investigation to determine the extent to which design strengths were conservatively predicted.

To verify the single plate framing connection design procedure included in the AISC LRFD 3rd Edition Manual (AISC 2003), a total of ten full-scale tests were conducted to create a baseline for comparison to the existing design procedure. In this research, a test setup was developed to combine a combination of shear and beam end rotation that more closely represented field conditions. The test setup, which incorporated varying test parameters typically found in the framing industry, was different from Astaneh’s. Yet one of the primary goals of the test methods was to impose a total beam end rotation of 0.03 radians at ultimate loading (which is the accepted rotational standard for beam rotation). Differing from previous research efforts, a simulated slab restraint was incorporated into certain tests to approximate the effects of a concrete slab on single plate framing connections. In addition, snug-tight bolts were used in both standard and short-
slotted holes. To remove additional unknown variances in calculations, supplemental tensile and bolt tests were conducted to identify material properties of the steel and connection bolts used in testing.

The focus of this research is to identify means for improving the currently accepted single plate framing design method specified in the AISC LRFD 3rd Edition Manual (AISC 2003). To distinguish this research from past research, design methods recommended by previous researchers as well as specified design methods from other countries including Australia, Great Britain, and New Zealand were gathered for analytical comparison purposes and to identify new considerations and perspectives. Theoretical design strengths based upon dimensions and material properties from conducted experimental tests were calculated for each limit state of the various design methods. Through analysis of experimental data, direct comparisons of theoretically calculated limit state strengths to observed experimental strengths were made. In addition, an investigation into the rotational behavior of the connection was conducted.

1.2. SINGLE PLATE SHEAR CONNECTIONS

A single plate shear connection, also called a single plate or shear bar connection, consists of a single plate welded to a supporting member (such as a girder or column) and bolted to the supported beam or girder using high strength bolts. The single plate shear connection is designed to have enough strength to withstand shear load transferred from the supported beam or girder to the supporting member. Simultaneously, the single plate must have enough rotational ductility to allow for beam end rotation. Although idealized as a pinned connection the single plate does resist moment incurred by beam end rotation.
Consequently, with the sustained moment, the connection bolts are subjected to a combination of moment and shear.

Single plate shear connections are identified by different configurations including rigid (beam-to-column flange) or flexible (beam-to-girder web) support conditions. Figure 1.2 shows a single plate connection with a rigid support condition. In this example, the single plate connection consists of a single plate welded to the outside face of the column flange and bolted to the supported beam. Figure 1.3 shows a single plate connection with a flexible support condition. In this example, the single plate connection consists of a single plate welded to the web of a girder and bolted to the supported beam. Both illustrations are of the connection region of actual tests from this series of research. Identified within each figure are the single plate connection, supported test beam, and supporting member.
In terms of configuration parameters, the remaining variables are the number of bolts of the connection and hole condition. Geometric dimensional variability of sizing the single plate is mainly specified and dictated by the number of bolts of the connection. In the United States, single plate framing connections consist of two to nine bolts. Because this is a bolted connection, either standard or short-slotted holes are used. From a behavioral standpoint, the support condition, hole type, and number of bolts within the connection affects both the stiffness and rotational behavior.

1.3. LIMIT STATES IN THE CONNECTION

As previously discussed, the single plate connection must be designed to sustain the beam end shear and the beam end rotation. This rotation is allowed through the ductility of the connection components. From a design standpoint, a hierarchy of failure modes must be established such that ductile failures occur first. Ductile or yielding failures are
placed at the top of the hierarchy, while brittle failures such as rupture are placed at the bottom. These failure modes also known as limit states are grouped by connection components. Limit states are grouped both by the connection component and by the load path of the connection.

The load path is defined as the path which load is transferred from the point of application to the foundation. Discussed in greater detail in later chapters, the load is transferred through the following components in the order given: supported beam, supported beam web, connection bolts, single plate, weldment, and finally to the supporting member. Each component is then designed to have adequate strength for multiple failure modes. For example, some failure modes of the single plate include shear yielding, bearing and tear out of the bolt holes, and block shear. Of the three limit states given for example purposes, the ductile limit states are listed first. Yielding failure modes, including the bearing of connection bolts within bolt holes, allow the beam end rotation while sustaining structural integrity of the framing system. Simply stated, the connection is only as strong as the weakest limit state.

The majority of the limit states are designed without moment (eccentric shear) considerations and are designed for directly applied shear forces. Because moment is sustained by the connection, shear is not directly applied through the bolt line, but eccentrically applied. For this reason, the connection component which requires additional consideration is often the connection bolts. Coupled with the eccentricity effects, the rotational behavior of the connections are affected by the configuration parameters such as support condition. For these reasons, the most critical component to design for is often the connection bolts.
1.4. SCOPE OF RESEARCH

The focus of this series of research was to investigate and improve the current single plate design method listed in the AISC LRFD 3rd Edition Manual (AISC 2003). To establish a reasonable sample size, a total of ten full-scale tests were conducted. Test parameters incorporated into this research included a varying support condition (either rigid beam-to-column flange or flexible beam-to-girder web), a varying number of bolts in the connection, and a varying hole type (either standard or short-slotted). In addition, for some tests with a flexible support condition, a simulated slab restraint was used to recreate the effects that a concrete slab would have upon a single plate framing connection. For sampling purposes, various combinations of the test parameters were combined such that the effects of individual test parameters could be identified.

The experimental results were analyzed to identify strengths of connection limit states. Using the material properties and geometric dimensions from testing, predicted strength calculations based upon theoretical specified limit states were made. To determine the efficiency of the currently accepted single plate framing design method, comparisons of the experimental results and theoretical strength calculations were made.

Design methods were gathered from researchers based in the United States who conducted extensive research in the field of single plate framing connections. In addition, design methodologies accepted as standard code in other countries were also compared. Comparisons were made to identify similarities and differences in calculation methods. In addition, the full-scale connections tested were analyzed by the various design methods. As such, theoretical values were obtained to compare to experimentally
measured results. From this set of comparisons, individual limit states and calculation methods from the various design methods could be quantified by accuracy of prediction.

In addition, the rotational behavior of the connection and the behavior of the connection components were investigated. Focus was also given to calculation of the connection eccentricity as multiple methods were used for verifying the connection eccentricity.

1.5. OUTLINE OF THESIS

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

- Chapter 2 contains an extensive literature review that describes research which pertains to single plate framing connections. A progression of knowledge and research will be presented from some of the earliest research to the most recent research. Focus is placed upon connections subjected to static loading and cyclic loading. In addition, references will be presented which pertains to the components of the connection such as the connection bolts, welds, and beam cope.

- Chapter 3 compares the various design methods to the currently accepted AISC LRFD 3rd Edition Manual (AISC 2003) single plate design method. Within this chapter the currently accepted design method of the AISC Manual is detailed. Also, identified in this section are the primary differences in the compared design methods.

- Chapter 4 consists of a detailed explanation of the experimental testing program. Details of the specimen design and test setup are presented. In addition, detailed descriptions of the various test configuration parameters are presented. Included in this chapter are descriptions of the supplemental material tests conducted.
Chapter 5 presents the primary results obtained from testing. Results are summarized in terms of test configuration parameters. Based upon the results of the experimental tests, the effects of test parameters used in testing are described. Within this chapter, limit states experienced by the connection are also quantified. Bolt eccentricity findings from the two calculation methods are compared. In addition, the supplemental material test results are presented.

Chapter 6 presents the analytical investigation conducted and consists of the following contents. Theoretical calculations utilizing the AISC manual single plate design method are presented with particular focus given to the connection bolts. Theoretically calculated strengths are presented alongside of experimentally obtained results. A separate section analyzes the rotational behavior of the connections tested. Finally, analyses and an interpretation of the behavior of the single plate are given.

Chapter 7 contains a series of comparisons which are based upon the analytical work from Chapter 6. Comparisons are made between theoretically predicted and experimental results to determine the conservative nature of the AISC design method. The next focus of this chapter compares various design methods from Chapter 3 to determine which method or which specific strength calculation most accurately predicts experimentally observed strengths. An investigation to the adequacy of the currently recommended bolt eccentricity equations is conducted.

Chapter 8 presents a summary of the findings from this research. From the experimental tests, analytical investigation, and analytical comparisons made, recommendations to the currently specified single plate framing connection design method are presented. In addition, the conclusions reached from this research are stated. Finally, the necessity for future research is discussed.

Appendix A includes all sample calculations associated with this body of work.

Appendices B through K, J1 and K1 present the test summary for each full-scale test conducted.
• Appendix L presents the primary findings of the supplemental tensile coupon tests.
CHAPTER 2  LITERATURE REVIEW

2.1. OVERVIEW

A significant amount of research has been conducted on single plate framing connections. Not confined to the United States alone, a good amount of research has been conducted in Australia, Canada, New Zealand, and Great Britain. The primary focus of this literature review is to present previous research conducted for single plate framing connections.

Within this literature review, commentary will be provided on the connection and test setup design which various researchers utilized. In addition, detailed information including the types of materials and loading procedures used by the various researchers will be presented. In some research programs, supplemental testing was conducted and will also be reported upon. Observations and results, including test behavior and controlling limit states and failure modes experienced will be discussed. Finally, if design recommendations for single plate framing connections were recommended, commentary regarding this subject matter will be included in this literature review.

Additional research pertaining to the design of single plate framing connections including but not limited to bolt and weld behavior due to eccentric loading and the effects of beam copes will also be presented.

2.2. NOMENCLATURE

Throughout this literature review, some common terminology will be used when citing various sources. The following is an overview of some of the commonly used terms related to single plate framing connections.
The single plate connection is classified as a simple support designed to sustain beam end reaction or shear allowing for free rotation. Because of this generalization, this type of connection is often referred to as a *shear tab*. Single plate connections also are referred to as *web side plate* connections in Australia. In addition, in Australia, single plates are referred to as *cleats*. However, in Great Britain, single plate connections are referred to as *fin plate* connections.

In comparison with other shear connections (single angle, double angle, and shear end plate), the single plate shear connection is much more rigid. As a result, a significant moment can develop within the connection components, specifically at the weldment and connection bolts. As seen in Figure 2.1, at a certain distance away from the bolt centerline or weld line the moment along the beam switches from positive to negative. In this diagram, this is the inflection point of the beam moment diagram and this indicates the location of the *effective pin*. Simulating a pinned connection, the effective pin marks the location where there is a shear force, but the moment is zero. However, the location of the effective pin is not limited to be toward the beam side of the connection as it can be located to the left of the bolt line in the opposite direction of the supported test beam.
The eccentricity of the single plate to supporting member weld or connection bolts is measured from the location of the effective pin. Unless otherwise specified, discussion regarding eccentricity refers to eccentricity of the bolt line, $e_b$. As mentioned earlier and shown in Figure 2.1, other locations of interest include the moments associated with the weld line, $M_w$, and the bolt line, $M_b$, which can influence the design of single plate framing connections.

Single plate shear connections are categorized by the rotational stiffness of the supporting member. Figure 2.2 shows eight support conditions commonly used with single plate shear connections.
The two support conditions most commonly referred to in this literature review are a rigid support condition and a flexible support condition. The connection illustrated as Figure 2.2 (a) is often referred to as a rigid support condition or beam-to-column flange single plate framing connection. This support is termed a rigid support condition because the column resists rotation as any load imposed on the connection causes the column to rotate about its strong axis. Accordingly, the single plate must sustain any beam end
rotations. As compared to Figure 2.2 (d) where the single plate is welded to the face of the column web, this connection has a lower resistance to rotation as the column bends about its weak axis. A *flexible support condition* or beam-to-girder web support condition is illustrated as Figure 2.2 (e). The flexible support condition consists of a single plate welded to the face of the girder web. The girder is supported by a column at each end. The end connections of the girder also serve as brace points against rotation of the girder. Along its unbraced length, the girder is free to rotate. Beam end rotation is sustained by both the deformation of the single plate and the rotation of the supporting girder. In comparison to a beam-to-column flange support, the beam-to-girder web connection is considerably more flexible when only one beam frames into the girder web. However, the other support conditions maintain varying degrees of rigidity and are rigid or flexible in their own right. Of the connections pictured in Figure 2.2, connections (a), (b), and (c) are rigid while connections (d) and (e) are flexible. However, depending on member properties and in effect rotational stiffnesses, connections (f), (g), and (h) are in between rigid and flexible. To clarify, the deeper beam of Figure 2.2 (f) is more flexible than the shallower beam. This is due to the fact that the larger load and beam will control the direction which the girder rotates. Because the girder will have already rotated counterclockwise, the clockwise rotation of the shallower depth beam must be sustained by deformation of the single plate. In the case of Figure 2.2 (g) where a beam frames to the face of a HSS tube, the rotation imposed by the beam can potentially cause the tube wall to deform in the direction of the beam to alleviate beam end rotation.
2.3. PREVIOUS SINGLE PLATE CONNECTION RESEARCH AND DOCUMENTATION

The following subsections include research and documentation pertaining to single plate framing connections and related topics which were published prior to the research which is presented in this thesis. The references included in each subsection are presented in chronological order and arranged by country in which the research was performed. This literature review will focus on publications from the United States, Great Britain, and Australia.

2.3.1. RESEARCH AND DOCUMENTATION BASED IN THE UNITED STATES AND CANADA

Some of the earliest research investigating single plate connections was conducted by White (1965). White conducted a series of tests investigating simple framing connections used in combination with square structural tubing. A major objective of the research was to resolve design questions concerning the best method of connecting tubes or wide flange beams to tubes. White utilized five types of simple framing connections in his testing. The following will discuss the framing connections utilized in White’s research which pertain to single plate connection research. Designated as a Type A connection (seen as Figure 2.2 (a)), a single plate was welded to the face of the structural tubing. Type B connections consisted of a portion of structural tee welded along the flanges. An example of a Type B connection is illustrated in Figure 2.3.
Type D connections (seen as Figure 2.2 (h)) consisted of a single plate welded to the structural tube at 45° to the principal axes. A total of eight Type A connections, four Type B, and two Type D connections were conducted. These simple framing connections were required to have sufficient strength to transfer shear from the connected beam, have sufficient flexibility to accommodate bending moment, be able to sustain beam end rotation, and not produce extensive deformation in the structural tubing. The test setup consisted of symmetrical double cantilevered specimens. Specimens were manufactured from A36 steel and the connections utilized A325 high strength connection bolts. Weldments were designed with conventional procedures, and for Type B connections, the eccentricity of loads was included in the design considerations. “Shear strengths were determined by inverting the specimen at the conclusion of the moment test and loading it such that each connection was subjected to essentially pure shear” (White 94). Rotations were recorded by pairs of horizontal and vertical dial gages measuring...
from the structural tube the rotations of the top and bottom flanges of the test beam. Test results were plotted as a relation of moment versus connection rotation. Beam-line solutions representing beam end rotation as a function of end moment were superimposed on the moment-rotation plots to determine whether the connection was rigid, semi-rigid, or simple. Beam line solutions were based upon a length-to-distance ($L/d$) ratio of 25 and an overload factor of 1.65 to justify a safe beam line.

Columns tested with Type A connections (a single plate welded to the face of the structural tube) ranged from 4”x4”x3/16” with 8in deep beams to 8”x8”x1/2” with 18in deep beams. For Type A connections, excessive deformation of the structural tube was the primary failure mode. It was observed that stresses were induced in the flexible tube wall as the plate rotated under the applied loading. In addition, high stresses developed in the structural tube wall at low load levels. Thicker walled tubes behaved stiffer. Considerable warping occurred in thinner walled structural tubes. Failure modes included the above mentioned excessive deformation of the structural tube or “local tube wall buckling”, web crippling of connected beams, excessive bearing around the bolt holes of the beam web, and one test experienced weld tearing. Supplemental tests were conducted to determine how much the sustained deformations of the tube walls reduced the axial load capacity of the structural tubing. The axial load capacity of the column was lessened by as much as 10 to 15 percent for a 3”x3”x3/16” structural tube to 30 to 40 percent for a 4”x4”x3/8” structural tube. Maximum reduction in strength was found to occur in tubes with a small width-to-thickness ratio of the tube wall with the lowest slenderness ratios. White recommended that Type A connections should only be used when the design load did not exceed 60 percent of the safe column capacity. Type B
connections (portion of structural tee welded along flanges to the structural tube) varied in results because the connection permitted reasonable rotational through distortion of the structural tee. The tension side of the structural tee could pull away from the face of the structural tube. Failure modes experienced included shear failure through the tee and weld failure. In hopes to reduce distortion of the structural tube, White investigated the behavior of a single plate connection if it were welded at 45° to the principal axes of the structural tube. Classified as a Type D connection, 6”x6”x3/16” and 6”x6”x1/2” tubes were tested. In comparison to Type A connections, the stiffness increased considerably. Distortions were more significant in thinner walled tubes as maximum loads were approached, however, thicker walled tubes showed no visible signs of distortion.

Lipson (1968) conducted a series of tests involving single plate beam framing connections at the Civil Engineering Department at the University of British Columbia. Three types of connections were investigated: a “bolted-bolted angle connection” where a single angle was bolted to both the supporting member and the web of the supported beam, a “welded-bolted angle connection” where the single angle was welded to the supporting member and bolted to the web of the supported beam, and a “welded-bolted plate connection” (or single plate connection) where the single plate was welded to the supporting member and bolted to the supported beam web. Of the three types of connections investigated by Lipson, the following summary will comment solely upon Lipson’s research of “welded-bolted plate connections” (or single plate connections). A major goal of the research was to devise a rational basis for design of this style of connection. The research focused on the behavior of the connections under working loads, the maximum rotation capacity, and determining a consistent factor of safety.
against ultimate load. In addition, the researchers wanted to determine whether this style of connection could be classified as flexible.

Test specimens used by Lipson (1968) were manufactured from A36 steel. Tensile coupon tests were conducted to determine the yield and ultimate stresses of the test specimens. Two to six ¾in diameter A325 high strength bolts were used to facilitate the connection, and welds were made with series E60 electrodes. Bolts were tightened to a predetermined torque of 356ft-lbs. The single plates used in testing were ¼in thick with 1¼in edge distances. Two types of tests were conducted with the first subjecting specimens to pure bending moment. In this symmetric test apparatus, a single plate was welded to each side of a 1in thick plate and a short piece of wide flange beam (21WF62) was bolted to each single plate. The cantilevered wide flange beams were supported at the free end by roller supports and load was applied at point loads near the connections. Beam rotations were measured via dial gages positioned on the beam web measuring to the single plate. The second type of test apparatus consisted of a combined shear and moment loading scheme. In this type of test, the single plate was welded to a 1in plate which was bolted to a heavy column (12WF106) at one end and supported by the test frame at the free end. In this setup, the test column was reused for the series of single plate tests. The supported end of the test specimen was such that rotation could be imparted to the beam. In this test apparatus, load was again applied as a point load near the test connection. The test specimen began in a neutral or level position. Lipson subjected the supported end to a predetermined rotation and applied load to the beam to achieve a desired slope of the beam. From this loading point, initial measurements were made. The beam remained at the predetermined rotation, however the load was increased
incrementally and rotations and load cell measurements were taken. Testing continued until a large vertical rotation had been reached. Three single plate tests were conducted using this loading procedure. Five single plate tests were tested with zero rotation and only additional shear. The remaining four tests were loaded in a manner of increasing both shear and rotation during the duration of the test.

Single plate connections used in pure moment testing indicated the following results. For the two bolt connection, long yielding at almost a constant moment was observed. With an increased number of bolts, the moment continues to increase but at a decreased rate, and the constant moment region decreases. Researchers determined that the constant moment region corresponded to the slipping of all bolts within the bolt holes. Shear-deflection curves showed some non-linearity at comparatively small displacements. Also, the center of rotation was found to be near the centroid of the bolt group. Failure of the single plate specimens occurred under load where either cracking occurred in the tension edge at the plate, cracking of the weld, or cracking in the plate below the bottom bolt. The bolts did not fail in shear, however significant bolt hole deformation was observed. From the results obtained, a design procedure was formulated based upon satisfying rotation capacity, load carrying capacity, and end fixity.

Research conducted at the University of Arizona at Tucson by Caccavale (1975) investigated the research conducted by Lipson (1968). Through a series of single bolt single shear tests in conjunction with finite element analysis, Caccavale studied the strength and ductility of single plate framing connections while attempting to validate Lipson’s results. In a single plate connection, the load on every bolt varies in relation to the distance from the center of gravity of the bolts. To investigate this behavior,
Caccavale sought to establish shear versus deformation response of single bolted lap joints. The single bolt single shear tests conducted utilized 3/4in diameter A325 strength bolts torqued to 350ft-lbs. In addition, the following four sets of test fixtures, which varied combinations of plate thicknesses, were used: 3/16in and 3/16in, 1/4in and 1/4in, 1/4in and 3/8in, and 5/16in and 5/16in. The test setup applied shear force to a single bolt while instrumentation monitored shear force and sustained deformations. From this series of experimental testing, the following results were found:

- Test bolts did not undergo any visible deformations
- Deformations were sustained by yielding of the plate material surrounding the bolt holes
- Thicker plates provided a small amount of additional initial stiffness
- When a pair of plates with varying sizes were used, bearing of the smaller plate controlled
- Slippage was found to affect the shear-deformation relationship test results
- The nature of the test fixture created a couple moment in the single shear tests
- Increased plate thickness increased the strength of the connection

Using the finite element analysis program INELAS, Caccavale analytically modeled the same experiments as conducted by Lipson. Lipson utilized two test configurations: a pure moment setup (monitoring moment versus bolt line rotation) and a combined shear and moment setup (monitoring shear at the bolt line versus vertical deflection of the connection). Models of each configuration were created in INELAS. The finite element models were such that the modeled fasteners allowed for bolt and plate deformation just as the single bolt single shear tests allowed. From the output of the finite element program, rotation characteristics of the connection were determined and the shear
deformation characteristics of the connection were modeled. The finite element analyses accurately predicted connection behavior indicating initial linear behavior followed by gradual yielding of the fasteners followed by plastic response of the connection. The outermost bolts were found to attain a plastic response first shifting the sustained load to the interior bolts until each bolt had reached the plastic limit. One limitation of INELAS is that slippage in the connection could not be modeled as the bolts are always in bearing. In addition, the boundary conditions used in modeling may have been overly stiff impacting results. If slippage could be modeled, finite element analysis results would have shown closer correlation to experimental results.

At the University of Arizona, Tucson, Richard et al. (1980) conducted a series of single shear tests, stub beam tests with single plate connections, and full-scale single plate shear connection tests to better understand the behavior of this type of simple connection. The researchers conducted a total of 126 fully-tightened A325 and A490 strength single bolt, single shear tests for a range of commonly used \( \frac{3}{8} \text{in}, \frac{7}{8} \text{in}, \text{and} \ 1\text{in} \) diameter bolts. Plate thicknesses ranged from \( \frac{1}{4} \text{in} \) to \( \frac{3}{8} \text{in} \) varying incrementally by 1/16in. Plate materials were either manufactured from A36 steel or A572 Grade 50 steel. The single shear, single bolt test setup is illustrated in Figure 2.4. From this figure, we see that the ends of the test fixture plates were attached to the load frame, while a single bolt was positioned through the overlapping plates. Tensile load (designated as \( T \) in Figure 2.4) was then applied by the load frame until failure occurred.
With experimentally obtained moment-deformation curves, inelastic finite element models were developed for multiple single plate connection configurations. Results from the finite element modeling showed that the ductility of the connection was due to bolt hole distortion and bolt hole deformations. Under low loading, the outer bolt forces were found to be horizontal and gave rise to the moment of the connection, while increased loading lead to the increased shear sustained by the bolts. Finite element modeling indicated that the outer bolts were loaded to near maximum capacity at loads well under the design service loading of the beam. Researchers concluded that adding more bolts caused the outer bolts to reach maximum capacity at even a lower beam load. From the finite element models, moment-rotation curves were generated. To validate the finite element modeling, a total of seven stub beam tests were conducted on two, three, five and seven bolt connections. Stub beams were sized such that the beam length was equivalent to the depth of the bolt pattern.

In addition to the stub beam tests, five full-scale single plate connection tests were conducted. Illustrated in Figure 2.5 is the full-scale test setup used for testing.
The single plates and beam used in the full-scale testing were manufactured from A36 steel. The test setup consisted of a single plate welded to the face of a column flange. A W-shape beam was supported at one end by the single plate connection and a roller support at the opposite end. In each test, the bolt pattern was symmetric about the major axis of the test beam. Load was applied as a concentrated point load at the midspan of the beam. “The eccentricity as a function of the applied load was measured by means of the strain gages located on the top and bottom flanges of the beam between the load and the connection, and also by computing the connection moment from the beam reaction” (Richard et al. 1980, 47). Test beams were loaded to the midspan yield moment, but were not loaded to failure. Post test inspection showed that there was no significant distortion or distress of the connection and in some cases there was minimal permanent centerline deflection. Full-scale test eccentricities at loads as high as 1.5 times allowable were verified with finite element analyses, beam line theory, and design curve prediction. One point of note is that connection eccentricities measured from the bolt line ranged from 4in for three ¾in bolts to 14in for five ¾in bolts to 42in for seven ⅞in bolts. The researchers recognized that the single plate connection derived its ductility from bolt
deformation in shear, plate and/or beam web hole distortion, and/or plane bending of the plate and/or beam web. Also, bolt slippage within the bolt holes created additional ductility in the connection. From this research, a design procedure was recommended. Following Richard et al.'s (1980) research, Young and Disque (1981) further discussed Richard’s design procedure with design examples and design tables. In addition, a further discussion by Griffith (1982) is presented in regards to Young and Disque (1981) article.

Richard et al. (1982) followed up the first series of tests with an additional investigation into single plate framing connections using A307 strength bolts in combination with short-slotted bolt holes. Using the same test setup as Richard et al. (1980), researchers conducted fifteen full-scale beam tests using A36 steel with 7/8in diameter A307 strength snug-tight bolts. As with previous research in each test, the centerline of the single plate coincided with the major axis of the test beam. In certain tests, researchers used the same bolts only finger tightened. Two different beam length-to-depth ($L/d$) ratios of 16 or 10 were used. Test setups were loaded up to 1.5 times the allowable load, not to ultimate strength. Eccentricity results of the larger beam length to depth ratios for this series of tests ranged from 8in to 12in for seven snug-tight bolts to 1.3in for the same number of bolts being finger tightened. Comparing similar number of bolts per the two different $L/d$ ratios, eccentricity results were smaller for the smaller $L/d$ ratio. For example, the five snug-tight bolt tests with the larger $L/d$ ratio had an eccentricity of 5in while the same test of lesser ratio exhibited an eccentricity of 1.7in. Results confirmed what investigators had suspected; short-slotted holes would lessen the moment at the connection. Eccentricities for this series of tests with short-slotted holes
were much less than eccentricities of the previous tests which used standard holes and higher strength bolts. From the results of these tests, researchers recommended a supplemental design formula/procedure to calculate the eccentricity when short-slotted holes are present. The recommended design formula could also be used for finger tight A325 and A490 strength bolts.

Becker (1985) followed Richard et al. (1982) research with a discussion regarding single plate connections with A307 strength bolts. Richard’s research used bolt diameter-to-plate thickness ratios which prevented bolt shear failure and bolt hole tear out. Richard followed up by recommending that designers avoid large diameter-to-plate ratios in order to maintain connection ductility. In addition, due to the extreme deformation/ductility of bolts in combination with extreme loads (i.e. 1.5 times the working load), it is recommended that short-slotted holes with A307 strength bolts not be used. As such, Becker recommended a supplemental design table providing considerations for uniformly loaded beams in correlation with beam lengths.

Continuing the research, Hormby et al. (1984) investigated single plate framing connections with Grade 50 steel and composite construction. This first portion of this study continued the previous research by investigating the use of slotted holes as well as off-axis bolt groups. Eight full-scale tests were conducted with four of the tests being connected with off-axis bolt groups. Figure 2.6 (a) illustrates a single plate shear connection where the bolt group is symmetric about the neutral axis of the test beam, while Figure 2.6 (b) illustrates an off-axis bolt group.
Researchers utilized the same test setup used in previous research. The test beam was a 32ft long W24x55 section connected with a \( \frac{3}{8} \)in A36 single plate. Three, four, five, or seven A325 strength grade \( \frac{7}{8} \)in bolts tightened by the turn-of-the-nut method were used and the specimen was loaded up to 1.5 times the working load. Upon post test examination, there was no record of distortion of the bolts or bolt holes. Due to the nature of the bolts in the short-slotted holes, the behavior differed slightly from the previous tests with standard holes. Eccentricities were found to be less in the presence of short-slotted holes. In one test where 6in bolt spacing was used, bolt slippage occurred. The eccentricity results of the four off-axis bolt group connections differed from the symmetrical connections by at most \( \pm 9 \) percent. Interestingly, researchers reported that the center of rotation for the off-axis connections was also at the center of the bolt group. Researchers concluded that tightened A325 or A490 strength bolts behave essentially the same in round or slotted holes, moment-rotation response of a single plate framing connection is unaffected by the location of the neutral axis of the beam it supports, and that the design curve created from the initial 1980 research was found to satisfactorily
predict the eccentricity of the single plate connections with off-axis tightened high strength bolt groups.

Beam line solutions for use with higher yield beam stresses ($F_y=50$ksi) were found to move the beam line out on both axes of the moment-rotation diagram. Moments were found to be approximately equal while the shear was found to be a ratio of 50/36 times larger than the lesser grade steel. This relationship was suggested to be used as a modification factor to the existing Richard design procedure. Additional provisions were also suggested to inhibit the occurrence of bolt shear or tension tearing of the single plate. This series of research also included experimentation of composite beams with shored and unshored composite construction. The second phase of research included an extensive analytical study of more than 5,000 analyzed designs, ten full-scale shored composite beam tests using three, four, or five ¾in diameter A325 strength bolts, and five full-scale shored composite beam tests using three, four, or five A307 strength bolts. Unshored construction was also studied. Conclusions reported that unshored construction offered two offsetting effects; part of the supported load increases the moment sustained by the connection, while the first yield load drops therefore decreasing the moment. Either phenomenon could dominate; thus explaining why the research proved to be inconclusive as to the exact amount of moment experienced by the connection. Over 2,000 beam line analyses indicated that the use of either shored or unshored construction has modest differences in connection moment that can be accounted for by the basic design curve/procedure. Repeatable results were found and again researchers concluded that the design procedure recommended from the 1980 series
of tests in addition to the recommended modifications from this research would result in adequate designs.

At the Department of Civil Engineering, University of British Columbia, Stiemer et al. (1986) conducted a series of full-scale tests which investigated the ultimate load capacity of single plate shear connections with flexible support condition (beam-to-girder web). Stiemer et al. focused on entire connections as opposed to individual connection components. This research differed from previous research in that it investigated single plate connections with flexible support conditions, it varied the skew angle of the supported beam at either 0°, 30°, or 45°, and that it varied the connection plate depth-to-girder depth ratio. The connection was facilitated by a single plate which was welded to the face of the girder web at the girder midspan. Including end supports, the girder measured approximately 8ft in length. The ends of the girder consisted of end plates which connected to supporting columns. A single beam deeper than 310mm (approximately 12in) was attached to one side of the single plate with 25mm (1in) diameter A490 strength bolts at 76mm (3in) or 102mm (4in) spacing. The end of the beam was supported by a load cell. Guides were placed at the load cell end of the beam to prevent out-of-plane movement. A single actuator was positioned near the connection which applied shear and torsional moment to the connection. Approximately 80 percent of the load was sustained by the connection. Positioned near the connection region, strain gages measured unidirectional strain in the longitudinal direction of the supporting girder. LVDTs were used to measure displacements of the connection.

To verify material properties of the test specimens, tension tests were conducted on coupons cut from the web of an additional length of beam. Four tests were used as the
basis for the results of this series of tests. Test observations, including noticeable yielding and deformation of test girders, were observed as the ultimate failure load was approached. Due to the torsional effects of the applied load, the top flange of the girder in the vicinity of the single plate connection twisted and the girder web buckled. However, directly opposite of the single plate, the girder web remained straight. Test results were plotted as load versus strain and load versus displacement curves. Connections with a 0° skew angle were found to withstand a higher torsional moment than skewed connections. With regards to the bending moment, any contribution from the connection plate was ignored. Connections with skew angles were found to have a higher resistance to induced bending moment. Ultimate failure was determined to be a combination of shear force, torsional moment, and bending moment.

Studies were conducted to investigate the effects of various geometric parameters and determine if those parameters affected the ultimate capacity of the connection. Three such parameters investigated include: plate depth-to-depth of the girder \((d_{plt}/D_{girder})\), depth from the girder top flange to the bottom of the connection plate \((d_{top}/D_{girder})\), and connection plate thickness-to-thickness of the girder web \((t_{plt}/t_{w})\). As \((d_{plt}/D_{girder})\) increases, the connection becomes more rigid. If \((d_{plt}/D_{girder})\) was less than 40 percent, then the torsional moment will result in the web of the supporting girder buckling just below the connection plate. If \((d_{plt}/D_{girder})\) was less than 60 percent, then the connection behaved flexibly as the ultimate capacity of the connection was approached. Also, if the single plate was welded further from the top flange of the girder, then the torsional rigidity of the connection was reduced. Stiemer et al. concluded that the supporting girder behaved flexibly when the ultimate capacity of the connection was reached. In
addition, the three ratios previously mentioned combine to form a design formula for single plate connections with a flexible support condition.

Abolhassan Astaneh-ASL and research assistants built upon previous research by continuing the investigation of single plate framing connection behavior during the late 80’s to early 90’s. With intentions to better investigate single plate framing connections, Astaneh sought to improve upon previous test methods used in past research. Previous researchers used only test setups which applied either moment-rotation with very small shear, direct shear with unrealistically small rotations, or a combination of shear-rotation which was less severe than potential field conditions. To perform more realistic tests, Astaneh developed a test setup which combined proportionate amounts of shear, moment, and rotation. Astaneh’s tests consisted of three different phases investigating the various configurations used with single plate connections. During the first phase of testing, Astaneh et al. (1988) investigated single plate connections with rigid support (beam-to-column) conditions and standard holes. The second phase of testing conducted by Astaneh and Porter (1990) investigated the affects of short-slotted holes used in combination with a rigid support condition. Concluding the three phases of testing, the third series of tests conducted by Astaneh and Shaw (1992) focused on the behavior of single plate connections when used with a flexible support (beam-to-girder web) condition.

Astaneh et al. (1988) conducted three full-scale single plate connection tests with a rigid beam-to-column support condition. Three, five, and seven bolt connections were tested with A325-N strength bolts and standard holes. Test beams and single plates were
fabricated using A36 steel. Illustrated in Figure 2.7 is the test setup Astaneh et al. used for single plate shear connection research.

![Figure 2.7 Astaneh et al. Single Plate Shear Connection Test Setup](image)

The test setup consisted of a beam (approximately 5ft in length), a short column (approximately 3ft in height), and two actuators. The column was connected to a reaction block. On the outer face of the column flange opposite and offset from the column web, a single plate was attached via a weld applied to each side of the plate to column interface. A cantilevered beam was then connected to one side of the single plate using high strength bolts. Bolts were fully tightened to approximately 70 percent of the proof load using the turn-of-the-nut method. The beam utilized stiffeners beneath the load line to prevent web buckling and used lateral braces at the end of the beam to prevent out of plane movement. Applying load to the top flange of the test beam, the ‘Shear Actuator’
was placed near the support to provide shear, while the ‘Rotation Actuator’ was placed near the end of the beam to provide moment and rotation. The main objectives of the research were to investigate the behavior of and develop design procedures for single plate framing connections subjected to realistic load and rotation deformation conditions while including shear effects. To realistically simulate the loading of a simple supported beam, researchers developed a computational methodology to establish a shear-rotation history to apply to test specimens. The program simulated loading of all beam cross sections from W16 to W33 with spans of 10ft, 30ft, and 50ft lengths up until the ultimate moment capacity \( \frac{M_p F_u}{F_y} \) at the midspan of the beam was reached. Program results indicated that the shear-rotation relationship was elastic until the onset of yielding. From yielding, large rotations developed due to relatively small load increases. The computational program assumed material to be elastic-perfectly-plastic, so strain hardening effects also needed to be taken into account. Based on the results of the many computational simulations, a standard shear-rotation curve corresponding to a beam span-to-depth \((L/d)\) of approximately 25 was selected and used for the experimental tests. The selected loading curve consisted of three critical points: the yield point of the beam (corresponding to 0.02 radians of rotation), the point at which the beam developed a plastic hinge at the midspan (corresponding to 0.03 radians of rotation), and the final point on the load rotation curve corresponding to the ultimate capacity of the beam (beam end rotation reaching 0.10 radians). Tensile coupon tests on the connection components were conducted for each plate to determine the yield point and ultimate strength of the materials. From which, \( M_y \) (beam yield moment) and \( M_p \) (beam plastic hinge moment)
were calculated. With these values, the shear-rotation curve was established and applied to the specimen until ultimate strength of the connection was reached.

The following limit states were investigated: eccentric shear failure of bolts, yielding of the gross plate area, fracture of the net area of the plate, fracture of the welds, and bearing failure of the beam web or plate. In this series of tests, the shear plate was designed for direct shear, and the bolts were designed for combined moment and shear effects (eccentric shear). All tests resulted in shear rupture of the bolts. The eccentricity of the bolts was determined from the test data to vary from approximately 0.7in (three bolt test) to 1.9in (five bolt test) to 2.8in (seven bolt test) from the centerline of the bolts toward the midspan of the beam. Rotations ranged from 0.03 radians (seven bolt test) to 0.054 radians (five bolt test) to 0.056 radians (three bolt test) at the point of maximum shear. Rotational ductility was found to decrease with an increase in the number of bolts. The single plate connections exhibited ductile behavior and allowed significant rotations within the connection through localized bearing deformation and considerable yielding of the gross plate section between the weld line and the bolt line. Researchers concluded that yielding of the plate caused a reduction of rotational stiffness and released end moment to the midspan of the beam. Researchers reported that the ductility of the plate is essential in accommodating beam end rotation. Accordingly, researchers recommended the single plate be made from lower yield stress steel. In addition, researchers identified that the vertical edge distance beneath the bottom bolt was most critical and recommended that edge distances of the plate follow AISC specifications of at least 1.5 times the diameter of the bolt to prevent bearing fracture of the plate. Based
upon this research, design procedures were recommended in correspondence with established failure modes.

It should be noted that the three tests conducted by Astaneh et al. (1988) were actually the first three tests in a series of five. The remaining two tests of the series were reported and discussed in Astaneh et al. (1989). These two tests consisted of three and five ¾in A490-N strength bolts used in standard holes with a rigid support condition. Paralleling past research by Astaneh et al. (1988), the same test setup and loading procedure was used. In these two tests, the single plates were manufactured from A36 steel and the test beams were manufactured from Grade 50 steel. Also varied in these tests were the edge distances and dimensions of the single plate. For the additional two tests, edge distances of 1-⅛in instead of 1-½in were used. In addition, the weld size was reduced from 1/4in to 7/32in for this series of tests. The failure modes for these two additional tests included bolt fracture of the five bolt test and weld and bolt fracture of the three bolt test. As compared to the previous five and three bolt tests using A325 strength bolts, the ultimate sustained applied shear force at the connection was less. The eccentricity of the bolts was determined from the test data to vary from approximately 2.0in (five bolt test) to 0.75in (three bolt test) from the centerline of the bolts toward the midspan of the beam. Rotations ranged from 0.053 radians (five bolt test) to 0.061 radians (three bolt test) at the point of maximum shear. In comparison to the first three tests, the eccentricity and rotation values were comparable in magnitude.

The American Institute of Steel Construction (AISC) adopted the design procedure recommended from Astaneh et al. (1988) research for use in the AISC-ASD manual (AISC 1989). Because Astaneh et al’s. (1988) research was limited to single plate
connections with standard holes and fully tightened bolts, the design method listed in the AISC-ASD manual (AISC 1989) did not address effects of short-slotted holes in combination with snug-tight bolts. Astaneh and Porter (1990) continued Astaneh et al.’s (1988) research with a series of four full-scale rigid beam-to-column tests with short-slotted holes. The test specimens utilized three, five, seven, or nine bolt single plate connections fabricated with short-slotted holes. High strength ¾in A490-N snug-tight bolts were used to connect the test beam to the single plate. The single plates were fabricated using A36 steel and the beams and columns were fabricated using A572 Grade 50 steel. One objective of this research was to closely adhere to the research methods used in Astaneh et al.’s (1988) research on beam-to-column connections with standard holes. By following previous research methods, any contradictory findings would be comparable allowing for corrections to be made to the adopted procedure with minimal impact.

Astaneh and Porter (1990) wanted to research the effects which short-slotted holes used in combination with snug-tight bolts had on the connection. Researchers also sought to determine if short-slotted holes reduced the shear capacity of the connection. In addition, the effects of the increased rotational flexibility and the eccentricity within the connection were researched. Previous research tested connections with up to seven bolts. This series of tests included a nine bolt test to determine the feasibility and behavior of a nine bolt single plate connection. Paralleling past research by Astaneh et al. (1988), the same test setup and loading procedure was used. Each test except the three bolt test failed due to shear rupture of the bolts. The three bolt test failed the top two bolts in shear rupture, but the bottom bolt tore through the connection plate material.
This failure mode was due to a small clear distance at the bottom of the plate. The plate was detailed with 1-\(\frac{1}{8}\)in clear distance as opposed to the recommended distance of 1-\(\frac{1}{2}\)in. During the nine bolt test, one crack in the single plate (approximately 5/16in) developed below the bottom bolt prior to failure. The crack originated from the bottom of the plate and extended vertically to the bottom bolt hole. The connection sustained additional load up to shear rupture of the bolts. Post failure inspection showed a significant crack near the weld line at the top of the plate. Concern was raised as to if the weld line crack occurred prior to or after bolt shear rupture. Determination of when the crack occurred was inconclusive, but investigators felt that it most likely occurred after failure despite the fact that there was a lack of plate yielding in the vicinity of the crack. Analyses of results show that beam-to-column connections with short-slotted holes and snug-tight bolts only differed with respect to location of the inflection point. The short-slotted holes caused the connection to behave more like a pinned connection by increasing flexibility and resisting less moment; therefore accounting for the lower eccentricity. Also, researchers observed that the specimen capacity was governed by shear yielding of the connection plate. Overall, the limit states were unaffected with the introduction of short-slotted holes and snug-tight bolts.

With a series of six full-scale tests, Astaneh and Shaw (1992) concluded Astaneh’s research regarding the behavior of single plate framing connections subjected to static loading. The tests were composed of one sided beam-to-girder (flexible support) connections. Research objectives included the need to verify the strength of flexible supported connections designed using the previously developed procedures for connections with a rigid support. Comparisons to rotational flexibility, moment capacity,
and component eccentricities were made. In addition, the girder web behavior as it was subjected to eccentric shear load was investigated. Test beams, girders, and single plates were fabricated from A36 steel. Both the test beam and the single plate utilized standard holes. The tests varied with either four or six ¾in A490-N strength bolts fully tightened using the turn-of-the-nut method. Tests utilized varying girder depth and girder web thicknesses to investigate beam-to-girder connections with varying stiffnesses. Again, researchers wanted to parallel the research methods used in Astaneh et al’s. (1988) research, such that results would be comparable and could be used to make additions to the AISC single plate design procedure. In doing so, the loading procedure and test setup were similar to Astaneh et al’s. (1988) test setup. However, this series of tests used a flexible beam-to-girder support condition. End plates were attached to the girder to simulate girder continuity and to restrain the girder flanges at the girder supports. A single plate was welded on both sides to the face of the girder web and a cantilevered beam was attached to one side of the single plate using high strength bolts. In this series of tests, the girder span was limited to approximately 30in while the girder sections were 18 to 24in in depth. In general, at very low loads the flexibility of the girder web helped to release stress and allowed the connection to behave similar to a pin. However, as loads increased, the stiffness of the connection increased as well as stresses, yet the rate of rotation of the connection lessened. It was noted that when a long free girder vertical span was present, the initial flexibility was due to elastic plate bending then the occurrence of plate yielding. On the other hand with the use of stiffer, smaller girder vertical free spans, no significant bending occurred before yielding in the connection plate. Yielding occurred in the web at a shear far below the ultimate connection load.
This also led to an increase in tension at the top of the weld causing fracturing or tearing of the weld. Although designed to fail the bolts, the eccentric force applied to the weld resulted in weld fracture of all tests. The onset of bolt bearing occurred as a vertical shear elongation of the holes. In a couple of isolated incidences, due to the limited clearance between the girder flanges, the rotation of the members resulted in the flanges of the two members touching. The following conclusions were presented: a stiffer connection resulted in greater bolt and weld eccentricities, and connection stiffness is directly related to the girder web clear distance, height, and free span, and indirectly related to web thickness and connection length.

The three phases of Astaneh et al.’s. single plate shear connection research were summarized in an article written by Astaneh, Liu, and McMullin (2002). The journal article is split into two sections. The first section describes the above mentioned research regarding gravity type loading and is followed by a section describing Astaneh and Liu’s research of single plate connections subjected to lateral load effects. (The research conducted by Astaneh and Liu (2000) is presented in Section 2.4.2.) Through many conducted tests, shear force was found to be dominant, but in combination with the development of bending moments, the capacity of the connection was significantly reduced. Typically, test specimens were relatively stiff at low loads and rotations and softened with the onset of local yielding. Maximum moment was found to occur at approximately 80 percent of the largest achieved rotation, not occurring at the maximum sustained shear force. Also, the point of inflection was found to move toward the connection with increased load and connection plate yielding and softening.
Astaneh’s design procedures are strength based ensuring ductility of the connection. Through limit states, potential failure modes of single plate connections are described. Six failure modes were established, and they are as follows:

- Yielding of gross area of plate
- Bearing yielding of plate and beam web bolt holes
- Fracture of edge distance of bolts
- Shear fracture of net area of plate
- Fracture of bolts
- Fracture of welds

Astaneh listed the proposed limit states as a progression of ductile to brittle failures. Yielding limit states are synonymous with ductile failures, while fracturing is associated with brittle failures. Encompassing each phase of research, Astaneh et al. sought to develop rational procedures for the safe and economical design of single plate shear connections.

Sarkar and Wallace (1992) conducted a series of tests to establish the best procedure for designing single plate shear connections while using the design methods suggested by Richard in 1980 and Astaneh in 1989 as a baseline. Sarkar and Wallace stated that the two baseline methods generated different results due to different assumptions and design criteria. For shallower connections with fewer bolts, the differences between the two methods were more evident. The differences between the two methods occurred in (1) identifying the component or failure method which allowed beam end rotation and (2) determining reaction eccentricity. Richard assumed that the bolt group withstood beam end rotation releasing part of the beam end moment. Richard’s method ensured bearing
deformation of the bolt holes prior to the bolt shear capacity being exceeded. Accordingly, in this design method, bolts were designed for only the beam end shear reaction, and the plate and weld were designed to behave elastically under shear and bending. In Astaneh’s design method, the single plate was assumed to be the ductile link and yield under combined bending and shear stresses. As such, the single plate was designed for shear only. The weld was designed for a combination of shear and moment due to the shear force multiplied by \( a \), the distance from the weld line to the bolt line. Connection bolts were assumed to behave elastically and were designed for direct shear and the moment created by the shear multiplied by the bolt eccentricity.

Six full-scale beam tests were conducted. The test setup used by Sarkar and Wallace (1992) is illustrated as Figure 2.8.

![FIGURE 2.8 SARKAR AND WALLACE TEST SETUP](image)

A total of three different beams were used including a 21ft W12x35, a 33ft W18x76, and a 25ft W21x93. Each beam was tested once and turned over and tested again with a set of new connection plates. The first testing of the beams utilized single plates with standard holes, and the second testing of the beams used single plates with short-slotted
holes. Connection plates were manufactured from A36 steel and were of \( \frac{3}{8} \)in thickness. Connections consisted of two, four, or six \( \frac{3}{4} \)in diameter A325 strength connection bolts. Only in the first test were the threads excluded from the shear plane. To determine material properties of the steel specimens, tensile tests were conducted, and single bolt lap tests investigated connection ductility. The symmetric test setup consisted of a beam framing into a column at each end. The connection at each end was facilitated by a single plate welded to the face of the column flange (or rather a test setup which utilized a rigid support condition). Welds consisted of E70xx strength electrodes. Static loading was applied symmetrically and simultaneously to the top flange of the beam by two hydraulic jacks. Lateral bracing was applied at various locations to the beam flanges to prevent out-of-plane buckling. Beam shear and rotation, bolt line and beam midspan deflections, and the location of the point of the inflection of the moment were measured.

The loading procedure was applied in the following manner. During the first stage, load was applied until failure was observed or end rotation of 0.03 radians of end rotation was obtained. At this point, the load was released, and if failure had not occurred, the loading points were moved closer to the beam ends and testing recommenced. In this manner, large rotations could be applied to tests at an early stage. When the hydraulic jacks were moved closer to the connections, larger reactions could then be imposed upon the test connections.

The primary connection behavior led to the identification of the following failure modes: yielding of the gross area of the plate and/or beam web, bearing yielding of the bolt holes, lateral buckling of the single plate, shear failure of the bolts, and weld fracture. In certain cases, failure at one end was observed and loading was stopped at that
end, but researchers continued to load the opposite end until failure of the remaining connection plate occurred. In other tests, an extreme bolt sheared, but testing continued to connection failure. In only one test did researchers continue loading until all bolts failed due to sudden shear rupture. The factor of safety for each test was calculated by comparing the maximum load to the allowable bolt shear capacities with no eccentricity. For this series of tests, factors of safety ranged from 2.1 to 3.5. Observations indicated that only a relatively small amount of bolt hole deformation occurred. From supplemental tension tests, the A36 steel plate material was found to have a higher yield strength of 47.4ksi and the bolt tensile strength was found to be 120ksi. Due to these facts, bolt hole deformation was limited considerably. Researchers found that the load path affected the results. Depending on the amount and rate at which the shear force and rotation were applied, the load path could potentially enhance the deformation behavior by reducing the moment capacity, “softening” the connection. In the same respect, the load path could hinder the connection by subjecting the bolts to a prematurely large moment corresponding to a “stiffer” connection. Connections with short-slotted holes were found to carry greater shear and sustain increased beam end rotation. In addition, the point of inflection was found to be less than a connection with standard holes. From the testing, Richard’s method was found to over-predict the test eccentricity and Astaneh’s test method was found to under-predict the test eccentricity.

Sarkar and Wallace (1992) also conducted a total of 16 single bolt lap tests to investigate the contribution of bolt plowing or bolt hole deformation to the ductility of the connection. Test setups varied plate thicknesses and edge distances of bolt holes. Researchers sought to obtain a plate thickness-to-bolt diameter ratio that would allow bolt
plowing prior to bolt shear rupture. Researchers found that varying the edge distance from 2in to 1.5in did not facilitate bolt plowing. Reducing the plate thickness from 5/16in to 1/4in was found to improve ductility, but in doing so, tension tearing of the net area for thinner plates was found to be an additional failure mechanism. Sarkar and Wallace concluded that reducing the plate thickness-to-bolt diameter ratio improved the ductility of the connection significantly. In place of A325 strength bolts, the same size A490 strength bolts, which are stronger in shear, were found to be more effective in plowing. Researchers used existing formulas from the AISC LRFD 1st Edition Manual (AISC 1986) to determine allowable plate thicknesses when given the ultimate strengths of the plate and bolts. Sarkar and Wallace utilized the existing design methods of Astaneh and Richard and suggested an improved design method from their findings from the conducted research.

Sherman (1996) conducted research in regards to designing with structural tubing as used in building construction. Sherman discussed the benefits of HSS sections (Hollow Structural Shapes). Some benefits mentioned include: the efficiency of these sections as columns, torsional stiffness of closed shapes and high weak axis moment which minimizes the need for lateral bracing (which lateral bracing is not needed with square HSS tubing), and cost efficiency in regards to minimal surface area when maintenance is required. Typically in the United States, HSS sections are manufactured from A500 Grade B steel ($F_y=46ksi$ and $F_u=58ksi$). However, due to inherent overstrengthening and manufacturing methods, sections behave stronger having properties more similar to Grade C steel ($F_y=50ksi$ and $F_u=62ksi$). Sherman conducted a total of 24 tests using simple connections commonly used with HSS tubes to monitor the behavior of HSS
sections such that strength was not compromised. Due to its ease of design, cost effectiveness of fabrication and erection, and that deformation of the walls of HSS sections is a concern, shear tab connections were used to a greater extent in this series of tests. The single plate connections tested were required to satisfy existing limit state considerations. However, when used with HSS sections, single plate connections created an additional failure mode due to high transverse strains (which exceeded yield even at service load). Because shear tabs were welded at the center of the HSS face, potential deformation and punching shear was imposed on the walls of HSS sections where punching shear was defined as the tearing of the thickness of the wall adjacent to the weld. Recognizing this as a potential failure mode, researchers concluded that verifying the punching shear criteria would limit the maximum pulling force transverse to the column wall and would prevent this problem. The test setup was constructed to determine if local distortion of the HSS tube wall was detrimental in terms of column capacity by comparing single plate and through-plate connections. Testing parameters included: varying $b/t$ ratios, testing unsymmetrical connections with only one single plate attached to one side of the HSS section, and testing symmetric connections with single plates attached to both sides of the HSS section. All tests utilized snug-tight bolts. From this series of tests, results indicated that single plate connections used with non-thin-walled HSS column sections developed the same column strength as through-plate connections. When used in combination with thin-walled HSS column sections, single plate connections were found to have a potential detrimental affect upon column strength. In addition, research indicated that column strength was unaffected by one-sided single plate connections. From research conducted previously by Sherman, the shear
eccentricity was found to be between the weld and the bolt line. In comparison to the eccentricities listed in AISC LRFD 2nd Edition Manual (AISC 1994), measured eccentricities were found to be less. Therefore, Sherman concluded that the AISC Tables for shear tabs were conservative.

Short-slotted holes enhance single plate framing connections by allowing for greater fabrication tolerances and relieve stiffness as bolts are allowed to move parallel to the hole. Recognizing these beneficial qualities, Duggal and Wallace (1996) researched the behavior of short-slotted holes when used with single plate framing connections. Objectively, they set out to quantify longitudinal stiffness by determining the amount of force required to slide a connection bolt the length of the slot. In addition, researchers sought to determine how this correlated to the maximum sustained moment of the connection. To accomplish this objective, several plates with short-slotted holes were tested under a combination of perpendicular and longitudinal force in order to develop an equation for predicting the longitudinal force (parallel to the length of the short-slotted hole) required to slide the bolt the length of the hole. Test configurations incorporated the use of ¼in, ⅜in, or ½in thick plates with ⅜in diameter bolts, or the use of ⅜in, ½in, or ¾in thick plates with 1in diameter bolts. Bolts were torqued to either 30lb-ft or 120lb-ft. Short-slotted holes measuring 13/16in x 1-7/8in were used with ¾in diameter bolts while 1-1/16in x 2-1/2in slots were used with 1in diameter bolts. Loading combinations of beam end shear with snug-tightened bolts, beam end shear with untightened bolts, or axially loaded specimens with snug-tightened bolts were used. The test fixture was configured as a single bolt, double shear test when untightened bolts were used and was configured as a single bolt, single shear test when tightened bolts were used. Load was
applied perpendicularly to the short-slotted hole and held constant. Longitudinal load was then applied to the test fixture until the bolt had traversed the length of the short-slot. From mathematical regression and curve fitting of results, longitudinal force equations were determined for both snug-tightened and untightened bolts. Researchers found that for untightened bolts, slot length did not matter. Rather the longitudinal force was dependent upon perpendicular bolt load, bolt diameter, plate thickness, and yield stress of the connection plate. For snug-tightened bolts, the longitudinal load was found to increase with longitudinal displacement. This is attributed to the frictional force between the test plates as created by the torqueing of the bolts, the frictional force due to lateral swelling of the slotted plate, and the force to overcome bolt plowing. The longitudinal force was found to be dependent upon bolt diameter, plate thickness, and a nominal stress factor (which incorporated the following: the coefficient of friction between the plates, the amount of bolt tensioning, a coefficient regarding lateral swelling, the perpendicularly applied load, slot stiffness, and longitudinal displacement along the slot length). Researchers also determined that there was virtually no difference in terms of longitudinal bolt force versus longitudinal displacement for bolts with threads included and bolts with threads excluded from the shear plane.

The second phase of Wallace’s research (Duggal and Wallace (1996)) consisted of extensive nonlinear (plasticity) static finite element modeling utilizing results obtained from Sarkar and Wallace’s (1992) experimental research on full-scale single plate framing connections. Researchers took great effort in accurately modeling component and material behavior of the connections. A combination of bilinear quadrilateral, constant strain triangular, two-dimensional constant stress elements, and truss elements
were used in modeling the beam and connection components. Through finite element analyses, the center of rotation of the connection was found to coincide with the bolt group centroid. Researchers commented that the model was conservative due to rotation assumptions (such as the extra rotation potentially allowed by bolt clearances in the bolt holes or the additional rotation created by the yielding of the beam at the midspan) and the horizontal stiffness assumptions from experimental single bolt double shear tests (as the plate thicknesses used in this testing were not an accurate representation of the beam web thickness). Overall, the finite element modeling was found to accurately predict rotational demands in the elastic range and after beam yielding. Eccentricity values were found to be conservative (as compared to Astaneh predicted eccentricity values) but were not conservative as compared to experimental results (as the bolts reached the ends of the slots prematurely). The following assumptions were made: the bolts shared beam end shear equally, rotation was about the bolt group centroid, and that horizontal displacement was found to increase linearly with distance from the center of rotation.

Duggal and Wallace (1996) concluded this series of research by proposing a design procedure. The design procedure was based upon the failure modes as recommended by Astaneh where ductile failures are desired first. In this recommended design procedure, the bolts are to be designed to resist the longitudinal sliding force and vertical bolt shear. The plate is designed for both shear and stress caused by moment due to the longitudinal sliding force and moment due to the design shear applied through the bolt line at the designated distance away from the weld. The weld is designed to resist both the longitudinal bolt sliding force and the design shear and corresponding moment. As compared to design procedures presented by Astaneh and Richard, Duggal and Wallace’s
recommended design procedure is more similar to Richard’s design methodology. Primary differences occur in that Astaneh does not design the plate for equilibrium moment rather such that the plate yield strength can sustain the design shear. Accordingly, plate thicknesses in this method are thicker. In Astaneh’s method, welds are designed to develop the plate yield strength, and as such, smaller welds are called for.

Extended shear tabs are appealing because this type of connection offers the ability to frame a beam to the web of a wide flange column or girder while eliminating the need for coping or flange reduction. Figure 2.9 is an illustration of two single plate framing connections with flexible support condition. Figure 2.9 (b) depicts an extended single plate which is welded to the face of the supporting girder web. Because the bolt line of the extended shear tab extends 3in beyond the flanges of the supporting girder, the supported beam does not need to be coped. Due to geometric constraints, the supported beam depicted in Figure 2.9 (a) must be coped.

![Figure 2.9](image)

**FIGURE 2.9 EXTENDED VS NONEXTENDED SHEAR TABS**

To research the behavior of and establish a uniform design procedure for extended shear tabs, Sherman and Ghorbanpoor (2002) presented a final report on the design of extended shear tabs to the American Institute of Steel Construction. This report was later
condensed into journal article form and can be found as Ghorbanpoor and Sherman (2003). The scope of this research included 31 full-scale tests conducted in three phases. In the first phase, 17 full-scale tests were conducted on three and five bolt shear tab connections to either column or girder webs. This phase focused on the stiffness of the supported beam, size of the supporting member, weld configuration, as well as varying connection configurations of standard holes with snug-tight bolts or short-slotted holes with fully tightened bolts. Weld configurations included unstiffened shear tabs having only vertical welds such as those illustrated in Figure 2.10.

![Figure 2.10](image)

**FIGURE 2.10 SHERMAN AND GHRBANPOOR UNSTIFFENED EXTENDED SHEAR TAB TESTS**

Other tests incorporated shear tabs with additional horizontal welds between the top of the shear tab and under side of the top flange of the girder (as pictured in Figure 2.11 (b)) or stiffened shear tab connections where two stiffener plates were welded to the inside faces of the column flanges positioned and welded at the top and bottom of the shear tab (as pictured in Figure 2.11 (a)).
FIGURE 2.11 SHERMAN AND Ghorbanpoor STIFFENED EXTENDED SHEAR TAB TESTS

In addition to the above mentioned test configurations, the following four special case test setups were tested: a test setup where only horizontal welds existed between the tab and the stiffening plates and the stiffening plates and the column flanges, two column test setups with stiffening plates also welded the column web, a girder test with only a single vertical weld on one side of the shear tab, and a column test with a 19in deep shear tab which simulated making a shear tab connection to continuity plates or bearing stiffeners. The second phase consisted of four tests investigating whether snug-tight bolts could be used with slotted holes. This phase sought to identify the necessary criteria to be used in sizing the stiffening plate. One other variation in testing was the use of a single stiffening plate between the column flanges positioned at the top of the shear tab. The final phase investigated the behavior of larger six and eight bolt connections. Girder and stiffened and unstiffened column tests were conducted. The third phase of testing included the investigation of additional stiffening effects of having the tab extended and welded to the top side of the bottom flange of the girder. From the three phases of testing, researchers sought to determine connection capacity, evaluate existing and
identify new limit states, determine bolt and weld line reaction eccentricities, and devise and recommend a uniform design procedure for extended shear tabs with stiffener plates. The behavior and capacity of the tests studied were evaluated as functions of end rotation or span-to-depth ($L/d$) ratio of the supported beam, width-to-thickness ($h/t_w$) ratio of the supporting member web, the size of the shear tab, number of bolts, hole type, and lateral bracing of the supported beam. Phase I utilized W12x87 ($L/d$ ratio of 23) test beams with three bolt shear tabs and W18x71 ($L/d$ ratio of 10) test beams with five bolt shear tabs. Phase II utilized W12x87 beams of the same span-to-depth ratios. For Phase III, due to the number of bolts of the connection and space limitations of the testing facility, test beams with low rotation or low span-to-depth ratios were chosen. For the six bolt tests, W24x146 ($L/d$ ratio of 14) test beams were chosen. For the eight bolt tests, W30x148 ($L/d$ ratio of 11) test beams were chosen. A total of eight combinations of width-to-thickness ($h/t_w$) ratios ranging from 22 to 54 of the supporting members were used. Columns of 8ft heights and girders of 10ft lengths were used. All supporting members were manufactured from A572 Grade 50 steel with the majority of the shear tabs manufactured from A36 steel. Two ¼in plates used in Phase II were manufactured from Grade 50 steel. All bolts used in testing consisted of ⅜in A325-X strength bolts while welds were created from E70 electrodes. Phase I and II tests utilized ¼in thick shear tabs with 3/16in welds while the majority of Phase III tests utilized 5/16in thick plates with one test utilizing a ⅜in thick plate. To determine properties of the material tested, tensile tests were conducted on material from the same plate stock used in tests as well as from the supporting member webs after testing.
The test setup consisted of a simply supported beam loaded via hydraulic jacks at a determined distance away from the connection. This distance of the effective concentrated point load simulated a simple supported beam with distributed loading. Because twisting was found to be a problem in unstiffened tests, lateral bracing elements were installed at two positions along the length of the beam. Load cells were used to measure the connection reaction while twist was measured by a pair of LVDTs positioned vertically along the edge of the top flange of the beam. Tiltmeters measuring rotation were positioned on the supported and supporting member. In each test, strain gages were placed on the shear tab, test beam, and the supporting member web and were used to monitor material behavior as well as determine the onset of yielding of the shear tab and supporting member. In addition, test specimens were whitewashed to monitor the onset of yielding. Three pairs of strain gages were installed at determined intervals in the middle on the outside faces of the top and bottom flanges of the test beam. Using the data collected from the pairs of strain gages, the location of the reaction shear could be determined. Loading was applied statically and paused at even intervals such that observations could be made. Researchers monitored in-test load-displacement, load-twist, and load-rotation plots for any sign of the onset of yielding. When data-plots indicated appreciable yielding (nonlinear behavior) or when the connection could not sustain further loading, testing was terminated. Data plots were used to determine the ultimate shear capacity, the point of connection yielding, and assist in the determination of failure modes.

As compared to eccentricity equations presented in Part 10 (pgs. 112-113) of the AISC LRFD 3rd Edition Manual (AISC 2003), the majority of eccentricity results
correlate better with the AISC rigid equation for unstiffened tests. However, for the majority of the unstiffened tests, the experimentally measured eccentricity is less than the AISC eccentricity value. For the stiffened tests, experimental eccentricities were found to correlate better with the flexible support eccentricity AISC equation. Similar to the stiffened tests, the unstiffened test eccentricities trend to be smaller than AISC eccentricity equations. Results indicate that eccentricity values were virtually identical for standard or short-slotted holes. Failure modes experienced were classified as either primary or secondary such that multiple failure modes could occur simultaneously. Primary failure modes experienced include: bolt bearing, shear yielding of the shear tab, twisting, web mechanism failure, plate buckling, and bolt fracture. Although listed as primary failure modes for some tests, these failure modes were observed as secondary failure modes in other tests. Additional failure modes experienced include: bolt shear, block shear rupture of the shear tab, weld failure by tearing, tearing near the top of the shear tab, and web shear. Twisting of shear tabs having five or more bolts was prevalent. Because the supported beam web attaches to one side of the shear tab, twisting results from the eccentricity of the reaction with respect to the centroid of the shear tab. As such, torsional shear stresses resulting from pure torsion of the rectangular shear tab cross section are created. In addition, web mechanism failure describes the failure of unstiffened tabs welded to column webs where substantial column web distortions were observed. The web mechanism failure was determined to apply only to connections framing to the column web.

Sherman and Ghorbanpoor (2002) recommended the use of shear tab stiffeners as pictured in Figure 2.11 to prevent deformation of extended shear tabs at the ultimate load.
level. In addition, stiffeners add strength enhancing qualities to extended shear tabs aiding them in attaining connection capacities similar to standard shear tab connections with 3in distance from the bolt line to weld line. With stiffeners, column web mechanism failure was greatly impeded. For the three and five bolt connections, researchers determined that calculated connection ultimate capacities using experimentally measured eccentricities correlate better with experimental results than capacities determined from using AISC eccentricity equations. AISC eccentricities predicted bolt shear failure or bearing as the primary failure mode while researchers found shear yielding, bolt shear, and web mechanism failure of the shear tab to be the primary failure mode. As mentioned, AISC eccentricities were found to be conservative (predicting larger eccentricities) for three and five bolt tests and unconservative for six and eight bolt tests. Researchers concluded that lateral bracing near the point of load and connection did not affect capacity, but aided in lateral stability of the beam. In addition to the normally considered failure modes, a new failure mode, web mechanism failure was found to be a critical check especially for columns with a high width-to-thickness ($h/t_w$) ratio. From the special case tests, the vertical weld between the shear tab and column web was found to be essential. Without this weld, less than half of the ultimate capacity was attained. Unstiffened shear tabs were deemed allowable for small beam reactions. Also, using one vertical weld of twice the typical weld thickness on one side of the shear tab did not affect connection capacity. Welding of the stiffening plates to the column web did not significantly affect connection capacity. Ultimate capacity of the connection was lessened in tests where the connection plate was welded down to the bottom flange of the girder. General test behavior indicated nonlinear behavior began in all tests well below
calculated shear loads for yielding through the depth of the plate. Researchers concluded that nonlinear behavior was a combination of shear yielding from direct shear and shear yielding from twisting of the extended shear tab. From this series of tests, Sherman and Ghorbanpoor recommended a design procedure for extended shear tabs based upon the range of variables used in the experimental program. This design procedure incorporates methods and calculation techniques which best correlate values with measured experimental values.

At the National Steel Conference held by the American Institute of Steel Construction in 2002, Forcier (2002) contributed a set of design procedures for shear tabs requiring special design considerations, or “custom shear tabs” not listed in the AISC Manual. Commonly found custom shear tabs include: shear tabs which accommodate infill beams framing into girders at severe skew angles and shear tabs which frames a beam to a deep supporting girder with very wide flanges. In the first case, if the skew angle of the shear tab is too great and the shear tab is thick, then the gap on the obtuse side prevents the use of a standard fillet weld. In the second case, to accommodate the beam shear, two vertical rows of bolts become necessary increasing the bolt group eccentricity. For the design of custom shear tabs, Forcier states that each component is checked to sustain the shear load and a flexural load of the beam end reaction multiplied by the connection eccentricity. The single plate connection is still evaluated with respect to:

- Bolt group strength (beam reaction multiplied by the distance of the bolt group center of gravity to the support face)
- Weld and support strength (potentially shear only unless flexural design need be considered)
- Plate strength including shear, bending, and bolt bearing (beam shear and moment equal to beam reaction multiplied by the distance of the nearest column of bolts from the support face to the support face)
- Supported beam end strength including shear, bending, and bolt bearing (beam shear and moment equal to beam reaction multiplied by the distance of the furthest column of bolts from the support face to the support face)
- Various strength verifications including gross area checks and potential block shear failures

Certain journal articles are published to test an engineer’s knowledge of different connection types used in steel design and construction. The Steel Quiz printed in the June 2003 edition of Modern Steel Construction focused on single plate shear connections. A few key points from the quiz dealt with proper design considerations. Those mentioned include the necessity for minimum plate thickness to allow ductile failures while being at least ¼in thick to prevent local buckling of the plate. Also, the article brings attention to the importance of eccentricity in design and that ductility is created in part from bearing deformations occurring in the single plate prior bolt shear rupture. In addition, a weld size should be chosen to develop the strength of the plate thus eliminating weld rupture as a failure mode. For example, when an ASTM A36 single plate is used in combination with an ASTM A992 column, the appropriate plate to column flange weld size is 75 percent the thickness of the plate.

Ferrell (2003) explained how to design with single plate connections using the design method recommended by Astaneh listed in the AISC LRFD 3rd Edition Manual (AISC 2003). The design commentary categorizes the use of single plates as either single
plate shear connections (beam-to-beam or beam-to-column flange) or extended single plate shear connections (beam-to-beam or beam-to-column web). Ferrell describes flexible support conditions as torsionally unrestrained such that beam end rotation is accommodated by the rotation of the support. For rigid support conditions, beam end rotation is accommodated solely by the deformation of the connection plate. Ferrell brings attention to new tables provided for ASD design of single plate connections which expand to include 9 to 12 rows of bolts in a single column. These considerations in addition to the current design procedure and criteria for single plate connections are presented in the AISC LRFD 3rd Edition Manual (AISC 2003). As stated in the article, the limit states considered in the AISC LRFD 3rd Edition Manual (AISC 2003) for single plate connections are:

- Bolt shear considering bolt eccentricity
- Material bearing strength of the bolt group for both the plate and the beam web
- Plate shear yielding
- Plate shear fracture
- Plate block shear
- Plate flexural yielding due to bending using the plastic section modulus of the plate
- Plate flexural fracture due to bending
- Weld strength for plate to supporting member
- Block shear for coped beams
- Flexural yielding of the coped section of the beam
- Rotational demand of connection for rigid connections only

For extended single plate shear connections, Ferrell points out additional considerations have to be made regarding shear and moment for supporting members.
For example, a single plate is attached to the web of a column (weak axis column considerations).

As published by the Canadian Institute of Steel Construction (CISC 2004), the Canadian *Handbook of Steel Construction* provides provisions for the design of single plate beam connections. Their design methodology is based upon the experimental program conducted by Astaneh et al. (1989). As such, ductility through bolt bearing is a consideration in the design. In addition, high strength material is not recommended for single plate use. In this design manual, bolt eccentricity considerations are presented for both rigid (beam-to-column flange) and flexible (beam-to-girder web) support conditions. Bolt group strengths are calculated for single shear conditions with threads included in the shear plane. This manual states that Astaneh et al.’s (1989) research was limited to standard size holes and therefore results are considered to be conservative for shortsotted holes. In addition, the design procedure listed is only applicable to punched or drilled holes with pretensioned or snug-tight bolts. Welds should be selected to develop the shear tab material as recommended by Astaneh. Design tables are presented with the Canadian *Handbook for Steel Construction* specifying factored resistance values for M20, M22 A325M, and ¾ and ⅞in diameter A325 strength bolts. The design table is presented for 300W steel (F_y = 43.5ksi) with E49xx fillet welds. The CISC (2004) tabulated design values are presented for two to seven bolts specifying required plate thicknesses and weld sizes.

At Virginia Polytechnic Institute and State University in Blacksburg, Virginia, Ashakul (2004) completed his dissertation work involving extensive finite element analyses of single plate shear connections with rigid support condition. Objectives of this
research included the goal of evaluating limitations and assumptions listed in the AISC LRFD 3\textsuperscript{rd} Edition Manual (AISC 2003), creating finite element models of test setups used in previous single plate shear connection research, comparing the models to experimentally obtained results from previous research projects, and as needed from these comparisons, recommending improvements to the existing design procedure listed in the AISC manual. As a basis for finite element modeling, Ashakul gathered together the findings and design methodologies presented by the following researchers: Richard et al. (1980), Astaneh et al. (1988, 1990, 1992), Sarkar and Wallace (1992), and Duggal and Wallace (1996). Comparisons were made between obtained experimental results and the design method presented in the AISC LRFD 3\textsuperscript{rd} Edition Manual (AISC 2003). The general trend of the comparison was that the AISC LRFD predicted connection strengths were considerably less than documented experimental test capacities. In addition, AISC LRFD predicted failure modes sometimes differed from observed experimental failure modes. Ashakul found that discrepancies existed between tabulated eccentricity values and experimental eccentricities, and in many cases, the calculated shear yielding capacity of the test plates were less than experimental test capacities. Ashakul pointed out that AISC LRFD equations assume a constant shear stress throughout the plate although fundamental structural analysis dictates that the shear stress in a rectangular section is parabolic in nature. Therefore, the AISC LRFD 3\textsuperscript{rd} Edition Manual (AISC 2003) was concluded to be inaccurate and ineffective in predicting the behavior of single plate shear connections.

Using the finite element program ABAQUS, Ashakul created a total of 45 finite element models; eight of which simulated previous research test setups in order to
compare to documented experimental results. Finite element modeling was limited to simulating single plate connections with rigid support conditions. Elements and modeling techniques were chosen to recreate as accurately as possible previous research test setups. Ashakul recreated the single plate, welds (which would be used to attach the single plate to face of column flange), test beam, and connection bolts. Care was taken to accurately choose boundary conditions of the welded edge of the connection plate as test columns were not modeled. Also, beams were modeled to only the midspan length as symmetry was taken into consideration. Constraints were imposed on the test beam such that the entire length was braced against lateral torsional buckling. In addition, modeled beams were strengthened with web stiffeners directly beneath concentrated loads. Connection bolts were modeled in a fashion which simulated the effect of a nut on the bolt. Simulations utilized a finely meshed model which was deemed the best as it most accurately simulated contacts. When modeling connection bolts, 1/16in gap assumptions between the bolt and the bolt hole (replicating bolt holes which are 1/16in larger than the bolt diameter) were imposed such that bearing stress of the bolt hole was correctly modeled. Gap elements were used between the top half of the holes of the beam web and the bottom half of the holes in the single plate. Tie constraints were used to represent initial contact between the bolts and bolt holes. In addition, finite element models included modeling considerations for tangential movement of contact surfaces. A standard modulus of elasticity and Poisson ratio was used in all tests, but yield stresses were varied between simulations. Welds and high strength bolts were modeled using an elastic perfectly-plastic strain relationship while beams were similarly modeled with an additional strain hardening criteria. A uniformly distributed load which simulated beam
self weight and a concentrated point load (applied at the same location used in experimental testing) were used.

Finite element models, which simulated tests by Astaneh et al. (1988) and Sarkar and Wallace (1992), were verified by comparing results to the corresponding experimentally obtained results. The following limit states were monitored: shear yielding of the plate, bolt rupture, and weld rupture. Beam end rotation and bending moment of the midspan section were also monitored. Plots depicting shear-rotation relationships, weld line moment vs. beam end rotation, shear vs. beam end rotation, and shear vs. distance to the point of inflection from the weld line were created to verify plate behavior, weld behavior, beam behavior, and eccentricity trends respectively. Tabulated shear stress results were monitored to judge when a connection had failed. The majority of models were found to predict failure loads within 10 percent of the reported ultimate test capacity. However, some models predicted strengths which were approximately 20 percent greater than reported test capacities. After accounting for a 20 percent reduction in bolt shear strength due to the inclusion of threads in the shear plane, finite element results better simulated experimental results. Of the 45 simulations ran; only eight simulated experimental tests. The remaining tests investigated the affects of varying properties and geometric details. From this research, the following major conclusions were reached:

- The bolt line to weld line distance, $a$, was found to not effect the bolt group capacity.
- Plates manufactured from Grade 50 steel as opposed to A36 steel were found to impede or reduce the capacity of the bolt group.
The bolt group was found to “move altogether about the center of rotation; the movement was not as assumed by the instantaneous center concept. The amount of displacement of a bolt is a function of the distance from the center of rotation (beam neutral axis), not the distance to the instantaneous center, which is an imaginary point” (Ashakul 122).

The bolt group capacity for off-axis bolt groups (as illustrated in Figure 2.6) was found to be reduced as rotation characteristics of the connection were affected.

From the above mentioned conclusions, Ashakul details design considerations of single plate shear connections in the following categories: behavior of the plate, behavior of the bolt group, forces on welds, ductility, and the concept of instantaneous center. Ashakul proposes a design model for single plate shear connections based upon the variables taken into consideration in the scope of this project. After analyzing the research data and interpreting the analyses, Ashakul recommended a modified equation to calculate the shear yielding strength of the single plate. This equation was based in part upon observations of the shear distribution through the single plate cross section when strain hardening of the steel was reached. In addition, bolt strength equations were also proposed with respect to the number of bolts, yield strength of the single plate, and single plate thickness.

2.3.2. RESEARCH AND DOCUMENTATION BASED IN GREAT BRITAIN

Great Britain has varying design manuals and texts regarding single plate framing connections. Limit States of Structural Steelwork, 3rd Edition by Nethercot (2001) is a textbook which presents overviews of subject matter pertaining to structural engineering based upon the BS 5950 standard. Included in this text is a chapter which briefly discusses various joints (including single plate framing connections) utilized in the
framing industry. Although this text is not a design manual for connections, the reader is referred to *Simple Construction Volume I: Design Methods, Second Edition* published by the BCSA (1993), which is the British design manual for building connections designed by the simple method. Included in this manual are the design procedures for beam-to-column and beam-to-beam single plate connections.

Not limited to use only in the United States, single plate connections are widely used in Great Britain and Australia as well. Researching for the Steel Construction Institute (SCI) and the British Constructional Steelwork Association (BCSA), Malik (1988) released a draft report detailing the design rules for structural connections. This report was closely based upon procedures used by the Australian Institute of Steel Construction. For use in Great Britain, there was a need to modify the Australian design procedures to conform to British design methods. Moore and Owens (1992) conducted a series of tests to verify the adequacy of a design procedure published by Malik (1988). Researchers utilized the following test setup pictured in Figure 2.12.

![Figure 2.12 Moore and Owens Test Setup](image)

**FIGURE 2.12 MOORE AND OWENS TEST SETUP**
To simulate the connection arrangement in a multistory building, an inverted H frame was used as the load frame. The test beam was connected at both ends. Beam length was 20 times the chosen depth. A total of eleven tests were conducted. Six tests used (W14x22) 356x127x33 Universal Beams (UBs) connected to (W8x31) 203x203x46 Universal Columns (UCs) while the other five tests used (W24x68) 610x229x101 UBs connected to (W12x65) 305x305x97 UCs. Connections were facilitated by single plates utilizing a rigid support condition. In the majority of tests, a single plate was welded to the face of the column offset from the center by one-half the beam web thickness. However, for a small number of tests, the single plate was welded to the column web to investigate the additional flexibility due to weak axis bending of the column. Columns were 1500mm (approximately 5ft) in height and were allowed to rotate via 50mm diameter captive roller bearings at either column end. However, columns were restrained against longitudinal movement. The test setup was such that both shear and moment was applied to the test beam. Two 1000kN hydraulic load cells positioned symmetrically along the test beam were used to apply load. The compression flange was restrained at regular intervals to prevent out-of-plane buckling. Punched holes were used in combination with preloaded Grade 8.8 M20 strength bolts. Instrumentation was applied to the test setup to measure the applied load, bending moments in the beam, moment-rotation response of the connection, and the overall deflected shape. Tests were classified as either ‘elastic or ‘to failure. Elastic tests, which were designed to model a single plate connection at the end of a uniformly loaded beam, were loaded in equal increments to a level approximately equal to the unfactored load and imposed load. Hydraulic jacks were positioned nearer the midspan for elastic tests. Otherwise, test
specimens were loaded to failure again in equal increments. For tests loaded to failure, hydraulic jacks were positioned nearer the connection to ensure failure.

Test results were calculated and verified by the following methods. Tensile tests were conducted on the test materials to verify material properties. Bending moments in the beam were calculated by two methods. The first method calculated the bending moment from straight line interpretation of strain gage measurements. The second method utilized load cell readings and known load cell positions in calculating a free bending moment. The elastic tests showed good agreement between the two methods, while the tests loaded to failure showed varied results between the two methods. For the tests loaded to failure, interpolation between the two methods was used to determine the bending moment. Researchers concentrated on the midspan displacement, moment of the connection, connection shear, moment distribution in the plate, and rotational capacity to quantify the test results. Typical behavior of elastically tested specimens is describes as follows: During the early stages of loading, some moment was released by bearing deformations of the single plate and beam web. Midspan displacement was found to be greater than calculated values using simple beam theory. Results indicated that serviceability as limited by $L/360$ may be the design criteria. The weld and the column of bolts experienced a combination of shear and moment. For test specimens with single plates welded to the column web, test rotations were found to be greater than similar connections with single plates welded to the face of the column flange. For tests loaded to failure, failure modes included the following:

- Lateral torsional buckling of the beam
- Local buckling of the beam web
• Excessive twisting of the web side plate
• Plastic hinge formation in the beam
• Deformation of the supporting frame
• Torsion of the web side plate

Short single plates were found to fail more frequently in lateral torsional buckling while long side plates were found to fail by local buckling of the beam web. Researchers also noted that there was no sign of weld yielding after any test. Although the connection bolts did not fail, a permanent deformation was sustained by the connection bolts. Also, the bolt holes in both the beam web and single plate exhibited permanent bearing deformations. For test specimens with internal connections, similar connection moments were found to develop, however, the internal connection exhibited small rotational ductility.

As mentioned, researchers wanted to adapt the Australian Institute of Steel Construction method. The modified design method differs with respect to the following:

• Criterion for failure mode checks
• The governing failure mode for bolts is such that plate bearing should control as opposed to bolt rupture. In doing so, plate bearing would allow for adequate connection rotation.
• Minimum moment criteria as well as coincident shear are required for all critical sections. In this respect, the connection would not have to be classified as either rigid or flexible.

Long single plate connections were found to have a tendency to twist and be subject to local torsional buckling. Accordingly, an additional check for local buckling strength was introduced. Researchers suggested that long single plates not be used with
unrestrained beams. Moore and Owens concluded that the design method was adequate and conservatively provides for safety margins of 1.57 to 3.57. As stated in the journal article, the design method becomes a series of multiple checks.

- Compliance with necessary detailing requirements
- Bolt group capacity under shear and moment (plate bearing governs the bolt strength)
- Net section capacity of supported beam
- Net section capacity of single plate
- Local buckling capacity of long single plates
- Capacity of attachment weld
- Local shear capacity of web of supporting element
- Structural integrity checks (for beam/column connections only)

In the British Constructional Steelwork Association, BCSA (2002) *Joints in Steel Construction* design manual, the British design method for single plate shear connections is presented. The design method is written to allow for ductile failure modes prior to brittle failure of the connection. Rotational capacity of single plate connections is obtained from bolt hole deformations of the single plate and supported beam, out of plane bending of the single plate, and shear deformation of the connection bolts. In addition, potential connection slippage accounts for some of the rotational ductility. The British design of single plate connections achieves ductile behavior through bolt bearing or shear and bending of the single plate. Examples of single plate connections included in the design method are: beam-to-girder web (flexible), beam-to-column flange (rigid) or beam-to-column web, and beam-to-hollow steel tube (rectangular or circular). In the case of beam-to-beam or beam-to-column connections, single plates are welded to the
supporting member offset to one side by one half of the thickness of the supported beam web. One primary difference in the British design method is that the line of action of the applied shear at the connection is assumed to be located where the single plate is welded to the support. As such, the weld is designed to sustain vertical shear only while the bolt group must resist the moment created by the applied shear imposed at a distance from the bolt line to the weld line, \( a \). It should be noted that the British design method for single plate connection allows for up to two columns of connection bolts. In terms of shear capacity, single plate connections with only a single column of bolts will develop a shear capacity of 25 to 50 percent of the beam shear capacity while a double column of bolts will allow for a shear capacity of 75 percent of the beam shear capacity.

2.3.3. RESEARCH AND DOCUMENTATION BASED IN AUSTRALIA AND NEW ZEALAND

Similar to the United States, Australia has varying design codes and books on topics including open section design and bolt design. These books are essential in the design of steel structures. One such book used by the researchers presented in this section is *The Bolting of Steel Structures*. This manual written by Firkins and Hogan (1984) includes but is not limited to the detailing of bolt characteristics, specification of bolt length, design methodology, and installation procedures. *The Behavior and Design of Steel Structures to AS 4100, 3rd Edition* by Trahair and Bradford (1998) is a textbook which presents overviews of subject matter pertaining to structural engineering based upon the AS 4100 standard. Presented in this textbook is a chapter which briefly discusses connections including single plate framing connections. Another manual titled *Design Capacity Tables for Structural Steel Volume 1: Open Sections* published by the Australian Institute of Steel Construction (1999) is utilized as a reference for material and
section properties, methods of structural analysis, and capacities for sections in bending, axial compression, axial tension, or combined actions.

During the late 70s and early 80s, the Australian design method for single plate shear connections was altered when researchers found that a plastic hinge could form in the web side plate at the stiff support line. This discovery lead to the potential introduction of a design criterion that would limit the connection based upon the maximum permissible moment. However, in doing so, the suited connection with stiff support condition and double row of bolts (with 140mm (5-1/2in) spacing between bolt lines) would create a situation where the strength of the bolts would exceed the strength of the plate creating a brittle failure. Earlier work by Pham and Mansell (1982) tested the standardized connection with 140mm spacing between bolts lines. From these results, revisions were made to the design model of web side plate connections. These revisions included the removal of a distinction between stiff and flexible supports and a reduction in spacing between bolt lines from 140mm to 70mm (2-3/4in). To verify the effectiveness of the revision, Pham (1985), at the CSIRO Division of Building Research, Melbourne, conducted a total of nine tests to investigate the behavior of the web side plate connection with stiff supports. Test beams ranged from 360UB45 (W14x30) to 460UB67 (W18x46) and grade 8.8/S (A325 strength) and 4.6/S (A307 strength) connection bolts were used. Test parameters included the use of the 70mm gauge spacing for double lines of bolts, testing of a single line of bolts as well as a double line of bolts, and the use of 6mm fillet welds (6E41) instead of the required 8mm fillet welds (8E41) of the standardized connection. Pham utilized the same test setup and measurement system from his 1982 research. This test setup consisted of cantilever test
beam attached to the face of the column flange via a single plate shear connection (rigid support condition). The free end of the cantilevered beam was supported via a load cell. Lateral bracing was provided to the test beam to prevent out-of-plane buckling. The testing procedure for this series of tests subjected the test setup to a rotation-to-shear ratio of 0.0004 rad/kN and the minimum distance between the weld line and load point was set to 1.5 times the beam depth. Test results were evaluated with respect to the shear force being compared separately to beam rotation, beam web rotation, web cleat rotation, and out-of-plane rotation. In addition, shear vs. moment, shear vs. vertical displacement at the bolt group centroid, and moment vs. beam rotation was modeled. Test behavior indicated a variation in failure modes for the multiple connection configurations. Weld behavior was important for the connections which incorporated the smaller weld size. Connections with a single line of bolts failed in the following manner: one test failed via ductile failure of the weld, while the other three tests failed via shear failure of the bolts. Pham pointed out that the weld size (although not conforming to standardized connection practice) in the latter three tests was not the critical component of the connections. Of the three tests that failed via bolt shear failure, only one of those tests utilized lower strength bolts. Connections with dual lines of bolts varied with either ductile failure of the weld, shear failure of the bolts, shear deformation of the plate, or tearing of the web side plate. In the instances when the higher grade bolts were used in conjunction with the larger weld, plate failures became the dominant failure mode. To compare to the allowable design loads, three methods were used: the ‘Modified’ Australian AISC model which is only valid for standardized web side plate connections, the ‘Traditional’ model which ignores the moment of the web side plate all together, and the ‘Simple’ model which
assumes the bolt group withstands only shear force and that web side plate and weld withstand shear and moment from bolt eccentricity. The three models can also be used to predict the ultimate load after determining the strength of the bolt and weld groups. Through calculations, the ‘Simple’ model was found to accurately predict the critical design component as found in all nine of the tests. An average factor of safety of 3.2 with a 44 percent variation is associated with the ‘Simple’ model. The modified Australian Institute of Steel Construction method was found to be overly conservative. Because of these reasons, researchers concluded that the ‘Simple’ model offers the best means for predicting web side plate behavior, and also, researchers recognized that bolt failure is a means of connection failure.

Patrick and Thomas (1986) conducted a series of tests for the Australian Weld Research Association, AWRA to verify the Australian Institute of Steel Construction’s Standardized Structural Connections Manual (Hogan 1981) design procedure of web side plates when used in combination with the largest member sizes in the manual. Connections were subjected to combined shear loading and beam end rotation to investigate rotational capacity, interaction of bending moment and shear, component behavior, and rotational stiffness. Vertical deformations were also investigated to ensure serviceability requirements. Serviceability at working loads and the moment-rotation relationships had been the previous focus of experimental investigations. This series of tests focused upon these relationships but loaded test specimens to failure. A total of four tests connections were tested. A total of two 760UB147 (W30x99) beams and a 310UC283 (W12x190) stub column with welded base plate were used. Web side plates were welded directly to the face of the column flange. A cantilevered test beam was
bolted to one side of the single plate and was loaded with a point load applied adjacent to the connection. The free end of the test beam was supported with a roller support and could be lowered to induce beam end rotation during testing. Lateral supports were used to prevent out-of-plane buckling and web stiffeners were used at the jack position to prevent web buckling. Material properties for the web side plates, test specimens, weld strengths were determined. The first two test connections were facilitated by a single column of either six or nine bolts, while the second two test connections were facilitated via double columns of either six or nine bolts. The interface of the single plate and beam web were painted with red oxide zinc chromate to reduce slipping. Connection bolts were less than snug-tightened. Connections were subjected to four equivalent static working load loading cycles which did not exceed the working load capacity. After each load increment, the beam end underwent displacements from zero to the maximum travel for the overall working cycle. Connections were then loaded to failure using displacements for the full working cycle. Test results indicated three important locations which defines the connection bending moment: the centroid of the bolt group, through the inner edges of the inner column of single plate holes (the plate critical section), and at the weld at the column face. The first two tests, which consisted of a single column of bolts, failed due to sudden shear failure of all the bolts. The onset of plate yielding was noted, and the bolt holes in the single plates were locally deformed. Tests utilizing a dual column of bolts failed via sudden shear fracture of the plate critical section. Some signs of yielding were initially present and the single plate was distorted in shear between the fracture and the weld. At approximately 85 percent of the ultimate shear loading, small cracks were observed at the top of each weld bead. Some necking due to tension
occurred in the region above the inner top bolt. Local deformation of the single plate bolt holes was noted during post test investigation. Researchers reached the following conclusions. The rotational stiffness was found to not be significantly affected by the magnitude of the loading. In addition, there was a clear interaction between the shear force and the bending moment. This was noted more so at higher shear loads. Bending moment was found to decrease as shear force was increased. Each connection component was found to be dependent on one another. For example, deformations resulted in increased ductility of the connection, affecting the forces which other components were subjected to. Connection strength was found to be a function of weld strength, the strength of the bolt group, yield strength of the single plate, and ultimate strength of the single plate. The Australian Institute of Steel Construction model was found to be adequate providing sufficient factors of safety for single plate connections when used with the largest member sizes of the manual.

Aggarwal (1988) conducted a series of tests with eight specimens which investigated the moment-rotation characteristics of single plate beam-to-column connections. The test setup consisted of a cantilevered test beam (200 UB 25.4kg/m, or approximately a W8x18) connected to a test column (200 UC 46.2kg/m, or approximately a W8x31) via a single plate with a dual row, dual line of bolts (four bolts total). The single plate was welded to the face of the column flange and offset to one of the column flange by one half the thickness of the web of the test beam. Connections were designed in accordance with the Australian standardized connections manual. The nominal yield stress of the steel was 250MPa (36ksi). Four 16mm, 20mm, or 24mm diameter AS-1252 strength bolts (which are approximately equivalent to A325 strength bolts) were used in
combination with 10mm, 12mm, or 16mm single plates respectively. The bolts used in
testing were fully tensioned. Positioned at the end of the test beam, a single hydraulic
jack was used to apply load to the test beam. 10mm stiffeners were positioned inside the
web cavity of the test beam above the hydraulic jack and near the test column opposite of
the single plate connection. Of the conducted tests, the first two utilized the above
mentioned setup, while the latter six tests utilized a seat angle to help alleviate the affects
of slippage. The seat angle was connected to the test column flange with two M20 bolts
and shop welded to the beam. Seven specimens were loaded gradually with increasing
static loads. The two specimens without angle seats were initially loaded to 30kN
(6.75kips) and the other specimens were loaded up to 40kN (9kips). The remaining
specimens were subjected to a pulsating loading of 1500 cycles where the maximum and
minimum of each cycle was increased to maintain a 30KN difference in the loading. Test
specimens were assumed to have failed when large displacements or slippage occurred
creating difficulties to record any further observations on account of large beam rotations.
Measurements were taken from a total of four LVDTs where one pair measured beam
rotation and the other pair measured column rotation. The test setup did not include out
of plane restraints, so researchers measured out of plane behavior with theodolites and
mirrors.

Theoretical calculations were made from a simple design model for web side plate
connections for rigid support conditions. Failure was assumed to be the lowest load
nominally necessary to cause failure in shear and moment, the bolt group, beam web and
column flange, or column flange and web side plate weld. For tests combining a web
side plate and angle seat connection, the maximum allowable load for each connection
was taken as the sum of the loads required to cause failure of the web side plate and the angle seat connection.

Specimens without single angle seats were observed to have large slips at loads much lower than the design loads. Large slips were found to occur at low loads for smaller diameter bolts, while for larger diameter bolts, slippage occurred at higher loads. With the presence of a seat angle, the connection was stiffer which allowed for higher sustained shear and moment. Seat angles were found to increase the end restraint of the connection up to 90 percent. Each test failed with yielding of the compression flange of the beam between the two lines of bolts. Researchers identified that beyond the angle seat moment resistance of the beam section was increased. Post test inspection showed that there was virtually no deformation in the column or web side plate. The only visible deformation was elongation of the two bolt holes nearer the tension flange of the beam. Both the major and minor axes of the column were unrestrained. As a result of which, significant out of plane rotations occurred at the joints.

Moment rotation characteristics plots indicated that thicker side plates displayed a lower level of end restraint. Researchers concluded that the restraint of a joint is a combination of the bolt diameter, web side plate thickness, and thickness of the beam web. A combination of a large bolt diameter with a thick side plate was found to increase joint stiffness considerably. Examination of test data for specimens subjected to pulsating loads indicated a reduction in ductility, strength, and end restraint causing a failure at much lower loads. In addition, small slip was observed during every load cycle. Static loading resulted in higher rotation capacity with little effect of strain hardening,
while pulsating loads resulted in lower rotational capacity on account of strain hardening of the specimen material.

From this series of tests, researchers were able to obtain moment-rotation curves for web side plate connections with and without seat angles. Curves indicated nonlinearity in moment-rotation behavior over the complete range of loading. Rotation was typically caused by slip and elongation of the bolt holes in the beam web and side plate. Researchers used end restraint as a means of quantifying the connection. “End restraint of a connection has been defined as the proportionate complement of $\beta$, where $\beta$ is the inverse of rotational stiffness, that is, if the joint rotates by $15.5 \times 10^{-3}$ radians under a moment of 48kN-m, then $\beta = \tan^{-1}(15.5/48.0) = 17.9$ degrees of the end restraint provided is $(90-17.9)/(90) = 80\%$” (Aggarwal 212). Without a seat angle, the average end restraint was approximately 80 percent while with a seat angle, end restraint was closer to 90 percent. Recommendations included the need for designs to account for the magnitude of the end restraint. The addition of moment led to failure at loads lower than those which were calculated.

HERA, the Heavy Engineering Research Association is New Zealand’s primary contributor to steel construction, welding, and metal fabrication and machining. This nonprofit research organization dedicates itself to serving the needs of metal-based industries in New Zealand. As such, HERA sponsors research projects and studies which have led to the single plate framing connection design procedure as presented in HERA (1999). The design objectives for single plate connections is as follows: (1) the design capacity must satisfy gravity ultimate limit state loads and (2) without collapsing, must have sufficient rotation ductility to sustain gravity load and seismic drift induced
rotations of 0.03 radians. Design considerations including end gap spacing and seismic shear and moment considerations are provided. Within the design procedure, limits are established for plate thicknesses such that strength and rotation demands are satisfied. Singles plates are designed for M20 8.8/S bolts with threads excluded from the shear plane where the thicknesses of the single plate and beam web are each not greater than 9mm (0.354in) in thickness. Provisions for combinations of other single plate and beam web thicknesses are made such that the threads of M20 8.8/S bolts are included in the shear plane.

Full-scale test setups combining single plate shear connections with a concrete slab are costly and accordingly limit the number of tests which can be performed. At Melbourne Laboratories, Vic., Australia, Adams et al. (1997) developed a special test setup which simulated the tension and compression forces experienced by a single plate connection as caused by a concrete slab. The test setup was more cost effective and allowed for a greater number of tests which generated a larger sample of results. The test setup consisted of a “cleat” or rather a 10mm (approximately ⅜in) thick single plate welded to a reusable support plate. The single plate was manufactured from Grade 250BHP stock with a nominal yield stress of 260MPa (approximately 38ksi). A web plate, which simulates the web of a double coped beam without the stiffening effect of the flanges was welded to a beam of 610UB maximum size or a BHP welded plate beam of 700WB or larger (nominal yield stress of 300Mpa (approximately 44ksi)). The web plate measured from the top of the beam to the top edge along the bolt holes was 210mm (8-1/4in) in height. The cleat and web plate were match-drilled or punched separately. The top edge of the cleat was positioned at a distance of 65mm (2-1/2in) below the top of
the steel beam. 20mm (approximately ¾in) diameter snug-tightened, high strength structural bolts of M20/8.8/S strength with threads included in the shear plane were used. Between the bottom flange and column face, there existed a 20mm gap allowing for 20mrad of rotation before interference. The following is an excerpt from the article detailing the test setup. Also, illustrated in Figure 2.13 is a diagram of the test setup used in testing.

“In the test rig the reinforcement is represented by a fixed pin which may be removed during testing to simulate the bare steel state…The test rig comprises the following major items: (1) a rigid loading beam supported from below at each of its ends by a load cell and a 1000kN capacity hydraulic jack operated in position control, (2) a test specimen housed in a slot in the load beam to restrain the bottom edge rotationally which is also machined flat for bearing, and bolted above to a rigid reaction frame, (3) a steel pin which engages one end of the loading beam when simulating the composite state, otherwise it is left out, (4) a frame at each end of the loading beam to provide lateral support, and (5) a roller box to prevent horizontal restraint forces from building up in the system” (Adams (1997), 579).
With the addition of the concrete slab, the location of the center of rotation, rotational stiffness, and moment capacity of the connection is affected. This research program examined the effects of cleat and beam web thicknesses, steel grade, bolt group geometry, and bolt tension. To measure this behavior, linear potentiometers were used to measure vertical movement relative to reaction frame. Also, the compressive force and line of action in the connection could be determined at any time because the test setup was statically determinate. Accordingly, the bending moment in the connection could also be calculated. Test results were as follows and were summarized by results from a 6x2 row connection. From looking at a typical moment-rotation plot, the single plate displayed four distinct phases of behavior: (1) a frictional phase occurring during the linear-elastic loading region where slippage did not occur, (2) the onset where slipping occurs, (3) bearing of bolts into the bolt holes, and (4) failure of a component of the test setup. Researchers found from the tests performed that a single column of bolts would shear either an extreme bolt (in bare steel tests) or would fail by buckling. Tests representing composite behavior and involving a double column of bolts would fail by buckling. For bare steel tests with a double-column of bolts, the tests failed by

FIGURE 2.13 ADAMS TEST SETUP
progressive tearing of the cleat along the net section of the inner bolt column. The crack would propagate from the edge of the cleat and extend through several bolt holes.

In addition to the research discussed above, Adams et al. (1997) also conducted a series of double cantilevered tests to investigate moment-rotation behavior of single plate connections. Researchers conducted the parametric investigation by varying steel grade, bolt group geometry, and bolt tensioning with either snug-tightened or friction tightened bolts. With stronger steel, the rotation capacity of the connection was reduced because local bearing deformations were inhibited. Bolt geometry was found to reduce the rotational capacity of the connection when the number of bolt rows is increased or if the connection is located further down the beam web. Bolt tensioning was found to affect only the moment at which first slip occurred. However, friction tightened bolts were also found to provide additional rotational restraint in the connection. From the tests, researchers found that shear force did not have a major effect on the shape of the moment-rotation curve. Researchers observed that variations in moment-rotation behavior at lower moments can be attributed to variations in friction and bolt bearing of the connection. In addition, researchers attempted to simulate interference of the bottom of the beam and the column flanges by the placement of an additional load cell between those two locations. As a result, bending moment was found to increase rapidly to peak values, but the rotation capacity was less affected and the bolt group centroid remained at the center of rotation.

Directly related to the Adams et al. (1997) research, Adams and Patrick (1998) also conducted two series of tests to investigate cracking of the concrete slab in a composite connection (where web side plate connections are commonly used). The steel connection
was omitted from both series of tests. The first series of tests subjected a slab, which was connected to steel beams, to bending and tension. From this series of tests, the tensile force experienced in the slab was measured. The second series of tests investigated the behavior of the concrete slab as it was placed in direct tension as the spacing of the longitudinal reinforcing bars was varied. The test setup used consisted of a rigid loading frame with actuator which could apply tension to the concrete slab, a load cell, stiffened transfer plates where reinforcing bars were welded, and a lateral support frame with load cell and roller to keep the specimen straight. The stiffened plate was joined to the reaction frame with pivoting supports at either end. Controlled cracking was a primary focus of the testing, so critical areas were closely monitored. Load was applied at a slow rate less than 0.2mm/min. After a crack had formed across the width of the specimen, the load was removed and then reapplied. Results showed that the first crack forms over the lap joint of the steel sheeting in the specimen. Researchers noted that the yield capacity of the reinforcement significantly exceeded the tensile capacity of the uncracked section. In this manner, each specimen supported a higher load than the first crack which occurred prior to yielding of the bars. Researchers also outlined the effect that bar spacing has on cracking in the report.

The Australian design method for single plate framing connections has been much improved via research conducted over the past years. Published by OneSteel Market Mills (2000), the Design Booklet DB5.1 details the Australian design procedure for single plate framing connections. Supplement SDB5.1 to Design Booklet DB5.1 outlines the history of the Australian research on single plate connections. This manual is primarily based upon research conducted by Adams et al. (1997, 1998). This design
method improves upon previous design methods by classifying support conditions as either stiff or flexible. A stiff support is classified as having a rotational stiffness greater than or equal to the rotational stiffness of the connection with the connection assumed to be part of a statically indeterminate system. A flexible support is classified as a condition where the rotational stiffness of the support is less than the rotational stiffness of the connection. For a flexible support, the connection is assumed to be part of a statically determinate system and that the rotation occurs about the centroid of the support. Eccentricity concerns are found to depend upon such parameters as rotational stiffness of the connection, flexural stiffness of the beam, beam span and loading configuration. The eccentricity is a measure from the bolt or weld group to the point of contraflexure or point of zero moment (also described in Section 2.2 as the inflection point of the moment diagram). Within the design manual, the allowable design rotation capacity of the deepest connection (of nine bolts) is increased to 23mrad (approximately .007 radians). For two or three bolt connections, the design rotation capacity is increased up to 65mrad (approximately .019 radians). Weld detailing requirements and single plate thickness restrictions have been modified to optimize economy concerns.

2.4. RELATED RESEARCH AND DOCUMENTATION

This section contains research and documentation which is related to single plate framing connections. Subsections 2.4.1 and 2.4.2 include summaries of extensive research programs which range from single bolt bearing behavior of the bolt holes of plate stock to research regarding the cyclic behavior of single plate connections. In addition, some subsections pertain to sources which were referenced in the AISC LRFD 3rd Edition Manual (AISC 2003) in Part 7 Design Considerations for Bolts, and Part 8
Design Considerations for Welds. Other sources presented here are included because they pertain specifically to bolt and weld design. As such, these references directly relate to the design of single plate shear connections. Subsections 2.4.3 to 2.4.5 are a presentation of the history and progression of research and thought with respect to bolt, weld, and beam cope design. In addition, discussion will be presented regarding currently accepted design methods as used in the AISC LRFD 3rd Edition Manual (AISC 2003).

2.4.1. BOLT AND PLATE BEHAVIOR RESEARCH

Rex and Easterling (2003) investigated the behavior of bolt bearing on a single plate by modeling a partially restrained connection as a combination of individual components as cited in Eurocode 3 Annex J (1994), also referred to as the component method. Researchers isolated the bearing behavior through a series of single bolt lap plate connection tests. One goal of the research was to investigate the load-deformation behavior of a single bolt on a single plate due to bearing, flexural bending, and shear force. The experimental investigation included a series of approximately 50 tests conducted at Virginia Tech (VT), Blacksburg, VA, as well as a computational investigation involving both existing and newly created finite element models. In addition, researchers utilized existing data from a series of 52 tests conducted for a single plate bearing against single bolts at Oklahoma State University, Stillwater, Oklahoma (Lewis 1994) for finite element model verification. From these investigational methods, researchers sought to develop an analytical method to approximate initial stiffness, strength, and load-deformation behavior of a single bolt bearing on a single plate.
The experimental investigation conducted at Virginia Tech consisted of one test setup with configurations varying bolt diameter, clear edge distance, plate thickness, and plate width. Specimens also differed with respect to material properties as well as edge condition where the plate either had sheared or sawn edges. All test setups utilized standard holes and consisted of a single bolt lap plate connection loaded via means of tensile force. A pair of linear calipers was used to take deformation measurements. Failure modes experienced included bolt bearing, hole tear out, splitting, and curling. Although not typically defined as a failure mode, researchers defined curling of the region of the plate below the bolt as deformation far enough out of the original plane to cause conflicts with the setup or measurement equipment. Curling also resulted in a significant drop of load capacity.

Test results indicated that plate width did not have a significant affect on load-deformation behavior, but Rex and Easterling advised that too small a width would result in net section tension failure for the lap plate test setup. The test data was adjusted for use in verifying finite element modeling of the bolt and plate. Prediction models for initial stiffness values were verified with experimental test results and existing and newly developed finite element models. Researchers used ANSYS, a finite element modeling program and determined that a three-dimensional mesh in combination with (8 noded, 24 degree of freedom) brick elements best correlated with experimental initial stiffness results. The model did not include plate width and also neglected the plate edge condition. For bearing stiffness, modeling bolt bearing on a bolt hole was determined to be a very elaborate, involving material nonlinearity. To simplify the problem, researchers assumed the model to be two-dimensional and that the steel in contact with
the bolt was assumed to be at yield stress. The edge condition was found to affect the behavior of the plate. Because the steel between the bolt hole and the edge of the plate is under high tension, small flaws in the edge of sheared edged plates lead to splits which propagated upward toward the bolt hole. Researchers concluded that the strength of a plate with a sheared edge is less than that of a plate with a sawn edge. The plate strength was found to be dependent on steel strength and clear edge distance. Rex and Easterling pointed out that splitting and curling strength models do not currently exist. However, unlike curling, splitting is a failure mode that can be experienced with single plate connections. Accordingly, researchers recommended the need for development of a better strength understanding for splitting, and also, researchers recognized the need for isolated testing utilizing short-slotted holes.

2.4.2. CYCLIC SINGLE PLATE CONNECTION RESEARCH

To study the behavior of and verify ductile design procedures for single plate shear connections subjected to combined gravity and cyclic loading, Astaneh and Liu (2000) conducted a series of 16 full-scale beam-to-column cyclic tests. Researchers were interested in both determining what affect a floor slab had on the moment-rotation behavior of the connection and the lateral resistance of steel structures. The test setup was designed for a combination of gravity loads and lateral drift. Test members, slab parameters, and the test configuration were chosen to represent typical building construction. Members used in the test setup included W14×90 columns at 7.62m (25ft) spaces, and W18×35 beams framing into W24×55 girders. Specimens were manufactured from A572 Grade 50 steel (with specified yield stress of 345MPa, (50ksi)) while connection plates were manufactured from A36 steel (with specified yield stress of
248MPa, (36ksi)). To facilitate the connections, either four or six 22mm or 25mm (1in) diameter, A325-N strength tension-control bolts were used. Also, an eight bolt single plate connection with a W33×118 girder was tested. Floor slabs with varying slab reinforcement and shear stud spacing were used in the majority of the tests with some tests incorporating concrete in the column web. In addition, some tests incorporated a seat angle to support the bottom flange, creating a stiffened seat connection. The typical test setup is pictured as Figures 2.14 and 2.15.
Bare steel specimens were tested in addition to specimens with the floor slab. Ends of the columns as well as the ends of the beams utilized pins as boundary conditions. Pin-ended struts vertically supported beams and simultaneously served as load cells. In addition, out-of-plane restraint was also provided. The test setup was designed to simultaneously apply both gravity loads as well as cyclic drift displacements. At the top of the column, lateral cyclic drift displacement was applied. To approximate the initial gravity load of a building, two actuators on each beam were used to apply gravity loading to the test setup.

One key parameter defining specimen behavior was the drift angle. Researchers defined the drift angle as the measured displacement at the top of the column divided by the height of the column. In certain cases when the test setup was subjected to large rotations, the beam and column contacted one another. This contact resulted in increased connection stiffness sometimes causing fracturing of the shear tab (typically at the top and bottom near the weld line). The bare steel specimens were found to have rotational
stiffness and moment capacity with the four and six bolt specimens developing on average 16 and 22 percent of the plastic moment capacity of the beam. Ultimately, the four and six bolt specimens experienced drift angles of 0.14 and 0.09 radians respectively.

Test specimens with a floor slab were found to significantly affect the cyclic behavior of single plate shear connections. The presence of a floor slab resulted in roughly twice the maximum lateral load resistance of the test setup subassembly, but it did not adversely affect the rotation capacity of the connection. By 0.04 radians of drift, concrete damage, local buckling, and damage to the metal deck resulted in the loss of the composite action of the test setup. The four and six bolt tests experienced approximately 0.15 and 0.11 radians of drift. The use of normal or lightweight concrete did not have an affect on the behavior of the connection. Grid reinforcement to the concrete deck limited the visible damage to the concrete slab but did not increase the lateral resistance of the connection. Also, the maximum lateral load capacity was reduced by approximately 20 percent when the web cavity was void of concrete.

Results from this series of tests indicated the following trends. Cyclic tests were found to behave similarly to gravity load tests in that they adhered to the same failure modes. Ductile failures such as bolt slip, bolt hole elongation, and yielding of the shear tab occurred first up to large drift rotations. The four and six bolts tests sustained significant beam and plate yielding and bolt hole deformations. The occurrence of small fractures was noted at increased drift angles. Four bolt tests experienced small fractures at 0.12 and 0.14 radians while the six and eight bolt connections sustained small fractures at approximately 0.05 radians. The only incident of bolt fracture occurred during the
eight bolt test when all but three bolts had fractured. From this series of tests, researchers concluded that the drift angle was found to be inversely proportional to the depth of the tab. The bending moment capacity of the single plate connection with floor slab was found to be 30 to 60 percent of the plastic bending moment of the beam.

Crocker and Chambers (2004) investigated the behavior of single plate framing connections subjected to cyclic lateral displacements. When subjected to lateral displacements, single plate connections must sustain additional rotation requirements. With a series of three full-scale tests, researchers sought to determine the deformation demand placed on connection bolts, examine local connection failure mechanisms associated with cyclic loading, and recommend an analytical and experimentally verified procedure to augment empirically derived rotational capacity equations presented in the 2001 Federal Emergency Management Agency (FEMA) report 355D section 5.5.2. Three full-scale connections consisting of three, four, or six ¾ in diameter A325-N strength bolts were tested. Plates were ¾ in thick and were fabricated from A36 steel. The test setup consisted of a single plate fillet welded on both sides to the face of a W14x398 column flange. A cantilevered test beam approximately 12 ft in length was then connected to one side of the single plate using the above mentioned bolts. One test utilized tension control bolts. Standard bolts tightened via the turn-of-the-nut method were used in the other tests. At the unsupported end of the cantilevered beam, an actuator was connected to provide lateral loading to the test beam and connection. Test specimens were loaded with increased cyclic rotations up to a maximum rotation of 0.06 radians at a rate of 2 in/min. Testing was paused at maximum and minimum displacement positions such that measurements and observations could be made. Theoretically, the horizontal
rotational stiffness varies from the tension to compression portions of the plate and that the neutral axis migrates toward the compression side of the plate. Also, the rotational demand was determined to be greatest on the exterior bolts. Connection rotation causes the pretensioned bolts to slip and engage the bolt holes and potentially fracture the bolts. If the bolts do not fracture, the beam will rotate far enough and begin to bear on the column causing the neutral axis to move to point of contact.

Only the six bolt test failed due to shearing of the extreme top two and bottom bolts. Shearing occurred prior to the maximum rotation of 0.06 radians. Bolt fracture was audibly heard and confirmed upon post test examination as the bolts remained in position after fracture. Researchers stated that cyclic loading resulted in crack nucleation at multiple sites on the bolt. The method of bolting was determined not to have any effect on the performance of the connection. Calculated ultimate displacement demands at or below 0.34in of the failed bolts were consistent with the established maximum deformation value for fasteners. Hysteresis curves indicated that the maximum rotation magnitudes increased with each cycle, and also, pinching increased as the rotation magnitudes increased. Moment rotation curves showed the typical behavior of a single plate connection subjected to cyclic loading. Initially, the moment rotation curve was linear followed by plastic behavior resembling strain hardening. This behavior continued until the maximum stiffness was reached. If bolt shear failure occurred, the connection experienced a reduction in stiffness and showed secondary strength provided by the remaining bolts. Data analysis led to the development of an analytical equation to predict the local connection behavior prior to and immediately after the initial loss of the maximum moment capacity. Researchers found the mean location of the neutral axis to
be at 0.63 times the length of the connection plate with a coefficient of variation of 5.9 percent. Deformation demand was concluded to be a function of the location of the neutral axis of the connection and the thickness of the plate was thought to possibly influence the bolt load deformation behavior. Researchers concluded that additional design considerations are needed for steel frames with single plate shear connections which could be potentially subjected to large lateral displacements.

2.4.3. BOLTS AND WELDS

Bolt group design has been designed using various methods over the years. The elastic method was used to calculate the stress in each bolt. As discussed in Part 7 of the AISC LRFD 3rd Edition Manual (AISC 2003), a major assumption of this method is that the shear applied to the bolt group is distributed equally to each bolt. “The stress caused by moment is then distributed proportionally to each bolt by the ratio of the bolt distance from the center of gravity of the bolt group to the maximum distance to the same reference; the farthest bolt from the center will have the highest shear stress” (Ashakul 18). Although this method was convenient, it proved to be overly conservative as bolts were assumed to behave elastically. This method was succeeded by a method assuming the concept of an instantaneous center of rotation as generated from an effective eccentricity. As stated in Part 7 of the AISC LRFD 3rd Edition Manual (AISC 2003), this method produces rotation and translation of one connection with respect to the other. Based upon research by Crawford and Kulak (1968), a load-deformation relationship for calculating the shear strength of a single bolt at a specified deformation was determined. Together, the instantaneous center of rotation method and load-deformation relationship are used to date in the AISC LRFD 3rd Edition Manual (AISC 2003).
The AISC LRFD 3rd Edition Manual (AISC 2003) method for determining the strength of welds subjected to eccentric loads is based upon Lesik and Kennedy’s (1990) research. Combined with the instantaneous center of rotation method, Lesik and Kennedy’s research led to the development of a load-deformation relationship for a unit-length segment of the weld. Differing from the load-deformation relationship for bolts, the load-deformation relationship for welds is dependent upon that angle $\theta$ at which the resultant shear force is applied to the axis of the weld. In the AISC LRFD 3rd Edition Manual (AISC 2003), tabulated values using these combined methodologies are presented for load angles of $\theta = 0^\circ$, $30^\circ$, $45^\circ$, $60^\circ$, $75^\circ$, and $90^\circ$.

The tabular forms for design strengths listed in the AISC manual are a result of a series of calculations applied to a multitude of typical configurations. Higgins (1971) discusses how eccentrically loaded connections are treated in the AISC manual. Determining the appropriate yield stress levels is very important as is the case for mechanical fasteners which derive their strain from fastener deformation, connection deformation, and slip between connection parts. Depending on the size of the connection, strains can become very significant as eccentric load is applied. With larger connections, outer elements can reach the rupture before inner elements reach the assumed yield stress values. Higgins points out that with larger eccentricities, tabulated values become more conservative. Higgins also discusses fillet welds and related design concerns.

Butler et al. (1972) conducted a series of tests to determine the ultimate capacity of eccentrically loaded welded connections. Because of the eccentric loading, the weld must sustain both direct shear and moment. At the time, design procedures overestimated
the necessary factor of safety. This was due to the fact that previous researchers were concerned with the overall behavior of the connection and did not focus their research on the individual weld component. In this series of tests, loading was applied in the plane of the weld group. To determine the effect of angled loading, angles of 0º (longitudinal), 30º, 60º, and 90º (transverse) were applied to the weld group. Theoretically, the weld group will rotate about an instantaneous center and deformation would vary linear from the centroid of the weld group. Butler et al. (1972) presents detailed calculations regarding the instantaneous center of rotation. To verify the analytical results and attain load-deformation behavior of the fillet welds, a series of 23 weld coupon tests were conducted. To compare to the analytical study, parameters such as the angle of the applied load were varied. In addition, 13 full-size eccentrically loaded filled weld connections were tested. From analytical work and two series of experimental tests, a verified theoretical method for determining the ultimate weld capacity was recommended.

Kulak (1975) was one of the earliest researchers to investigate the behavior of eccentrically loaded bolted connections using friction type high strength bolts in which slip is undesired. The purpose of this research was to review a recommended design procedure for slip-resistant eccentrically loaded high strength bolts. The following assumptions are applied to slip resistant connections. Connection fasteners carry the same load, and the instantaneous center of rotation of the bolts must be determined as it does not necessarily coincide with the centroid of the bolt group. Typically, the instantaneous center of rotation is located on the side of the bolt group or connection opposite of the load. To verify the design procedure, a series of three full-scale tests were
conducted to monitor connection behavior and measure the slip coefficient. Steel specimens were manufactured from A36 steel. Faying surfaces were of clean mill scale. The test configuration included the use of \( \frac{3}{4} \)in A325 strength bolts. Bolt installation was via the turn-of-the-nut method with the clamping forces being measured to better predict the slip loads. The present design procedure listed in the AISC Manual of Steel Construction (AISC 1970) was originally developed for bearing-type eccentrically loaded connections. Similar to the current design procedure, Kulak’s procedure is also based upon the constant resistance of each bolt. The primary difference between the two methods is that the method listed in the AISC manual of steel construction assumes the center of the bolt rotation coincides with the centroid of the bolt group. Because of the similarities between the two methods, Kulak concluded that the design of slip resistant connections should be based upon the design procedure listed in the AISC Manual of Steel Construction (AISC 1970) which produced a safe design.

Perry (1981) conducted a total of 30 symmetrical butt splice tests to evaluate the effects of lateral confinement and bearing ratio on bearing type bolted connections. Focuses of this series of tests included the investigation of symmetrical lap splices, the study of connections with minimal lateral confinement, the effects of fully tensioned or snug-tightened bolts, the effects of slotted holes, and the need to reexamine the definition of what defines a failure in bolted connections. Test parameters included the variation of bearing ratio, bolt tension, bolt hole dimensioning, material behavior, and the effects of end distance. As illustrated in Figure 2.16, the test fixture consisted of two 2in thick A514 strength steel plates with two \( \frac{1}{4} \)in thick A36 strength plates (which were designed to fail) joined by 1in diameter A490 bolts.
Varying plate widths manufactured with either round or slotted holes (with slots oriented perpendicular to the applied load) were used. Two test specimens were created with short end distances. The goal of the testing was to investigate the interaction of net section and bearing failure. However, other failure modes including plate buckling and end tear out were encountered. From this series of tests, conclusions and recommendations regarding the avoidance of bearing tear out and connection behavior as related to the bearing ratio were presented. Fully torqued bolts were found to provide 10 percent greater capacity than snug-tightened bolts. Bearing stress failure was found to be highly dependent on lateral confinement. In addition, slotted holes were found to provide an ultimate load identical to specimens with round holes having the same bearing ratio. Slotted holes were found to be less stiff.

Wing and Harris (1983) conducted a series of tests to determine the best method for analyzing and designing eccentrically loaded bolted beam connections. Of the methods, a simple elliptical shear-moment interaction formula was proposed by Surtees & Pape (1972). This method was established purely by comparison with test results. The elastic theory has been the dominant theory in analyzing eccentrically loaded bolted
connections. Wing and Harris (1983) conducted a series of nine single bolt double shear tests to establish a baseline load-deformation response. In addition, testing was conducted on twelve full-sized connections with four different bolt patterns consisting of one or two lines of bolts and varying eccentricities. In both series of tests, A325 strength bolts tightened via the turn-of-the-nut method were used. The following observations from the single bolt tests were noted. Bolts were relatively undeformed while the bolt holes were slightly scored from bearing deformations. From the full-sized tests, bearing failure of the web was noted in all tests. Deformations were noted to be greatest in the tension region of the web. From this research, Wing and Harris (1983) determined that the traditional elastic approach gave the most consistent results and was the best suited for designing the bolt configurations used in testing. The simple elliptical shear-moment interaction formula was found to be as accurate as the elastic method and was as easy to apply. However, at the time of this research, the ultimate strength instantaneous center of rotation method was presented in the AISC 8th Edition Manual of Steel Construction (AISC 1980) instead of the previous elastic analysis method.

Iwankiw (1987) also conducted an investigation of which method is best suited for designing eccentric and inclined loads on bolt and weld groups. The ultimate strength method presented in the AISC 8th Edition Manual of Steel Construction (AISC 1980) was chosen because it better reflected the ductility and load redistribution nature of a bolt or weld group. From the AISC manual, tabulated eccentricity coefficients are listed to limit loads. Within the AISC 1st Edition Manual of Steel Construction (AISC 1986), design considerations for eccentric loads of 45° and 75° were included in addition to vertical 0° loading. However, interpolation between these angles was not possible. This is because
a linear relation between the angles did not exist. Iwankiw investigated the most appropriate method for evaluating strength coefficients for bolt or weld groups subjected to inclined loads not listed in the AISC manual. This investigation was conducted through the evaluation and comparison of old and new design solutions.

Nowak and Hartmann (1993) proposed designing eccentrically loaded connections with a noniterative and conservative geometric approach. This approach would improve upon the ultimate strength method used in the AISC LRFD 1st Edition Manual (AISC 1986) and the AISC ASD 9th Edition Manual (AISC 1989). The ultimate strength method can be limited as it cannot analyze connections with skewed loads (other than 0°, 45°, and 75° angles), general connection geometries, and mixed connector sizes. Stated limitations for the geometrical approach include the fact that as the eccentricity becomes smaller the geometric approach becomes less valid. Accordingly, there is a subsequent formula provided for calculations with smaller eccentricities. Nowak and Hartmann (1993) present the fundamentals of the geometric approach validated by normalized connection capacities and experimental test results. From the conducted investigation, the geometric approach was found to be the method which approximated the data with the least amount of error. As such, researchers concluded that this method allowed for the design of connections with arbitrary geometries, high and low eccentricity loads, and skewed loading.

Carter (1996) discussed the considerations which are included in specifying bolt length for high-strength bolts. Referring to the AISC LRFD 2nd Edition Manual (AISC 1994), Carter brings attention to tables used in determining proper bolt length for high strength bolts including tables which take into consideration washer thickness and grip.
Considerations are also included for the exclusion of threads from the faying surface of the connection. Within this source, Carter presents a design aid for selecting high strength bolt lengths for snug-tightened and fully tensioned bearing in addition to slip critical connections. The design aid is presented in terms of nominal bolt length for commonly used A325 and A490 strength bolt diameters. This design aid takes into consideration the use of ASTM F436 flat washers with an ASTM A563 nut. Carter also discusses bolt length, nut thickness, and washer thickness tolerances, thread run-out, stick-through requirements, the risk of jamming the nut, and washer requirements.

Ruby (2003) discussed the economic and technical differences between snug-tightened, pretensioned, and slip critical bolts. Ruby pointed out that the AISC LRFD 3rd Edition Manual (AISC 2003) uses the 2000 RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts. Discussion is presented on how to select the appropriate joint type. In terms of strength, snug-tightened and pretensioned joints are identical in strength, and connection strength is independent of slip movement. One main difference between the three types of bolted joints is the economy. Of the three joints, snug-tightened joints are the most economical as they do not require special surface preparation and require minimal labor.

*High Strength Bolts – A Primer for Structural Engineers* by Kulak (2003) is the most current high strength bolt design guide supplied by the American Institute of Steel Construction. This manual consists of subject matter including: the history of high strength bolts, descriptions of and how to appropriately choose installation techniques, means of designing bolts for tension, shear, or any combination thereof, fatigue of bolted joints, and other special topics related to high strength bolts. The design guide explains
the principle differences between Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD), but is written with respect to LRFD design methodology. From the information presented in this manual, engineers are given the means necessary to adequately design connections with this type of fastener.

2.4.4. BEAM COPES

Because a good portion of this research includes single plate shear connections with flexible support condition, this literature review will also include a section dedicated to beam copes. The research cited in this portion of the literature review focuses on references listed in Part 9 Design of Connection Elements of the AISC LRFD 3rd Edition Manual (AISC 2003).

Because a portion of the flange is removed when a beam is coped, the lateral buckling strength is significantly reduced. Cheng et al. (1986) conducted a theoretical parametric study on beam copes to determine what influence stress concentration, shear stress, cope length, and cope depth have on local web buckling capacity. Because of the discontinuity of the flange and web of the steel section at the cope, a stress concentration can occur at the coped corner causing the beam to fail in inelastic local web buckling. Thin webs fail via elastic local web buckling at the coped region. Utilizing the computer analysis program, BASP, researchers analyzed both compression flange coped beams (which was discussed in the cited journal article) and double flange coped beams. BASP performs buckling analyses of plates to provide solutions. The web of the test section was idealized as a two-dimensional finite element while the flanges were idealized as one-dimensional finite elements. Models incorporated the following limitation: cope lengths were less than twice the beam depth and cope depth less than one-half the depth.
of the beam. In addition, end restraint was not modeled. In conjunction with the computer analysis work, a total of ten tests were conducted. These tests were used to verify computer models and aid in the creation of a design procedure. Of the ten tests, five were designed to study shear and flexural inelastic web buckling with the inherent effects of end restraints while the remaining five were designed to fail by elastic web buckling. From the parametric computer investigation, the effect of stress concentration at the end of the cope was found to decrease as cope length increased. Researchers concluded that localized yielding as a result of stress concentration did not significantly affect the buckling capacity of coped beams.

Cheng et al. (1988) conducted a theoretical parametric study on beam copes to determine what influence cope length, cope depth, and span length have on lateral buckling capacity. As used in Cheng et al.’s. (1986) research, BASP allowed researchers to analyze a W16x26 section with a depth-to-thickness ratio of 60 as a baseline for the lateral buckling model. As with previous research, the web of this section was idealized as a two-dimensional finite element while the flanges were idealized as one-dimensional finite elements. It is noted that the web of the member is one of the most slender and that the results for other sections would be conservative. The model utilized a symmetric setup where identical cope details were applied at each end of the section. Restraints as provided by the connection were not considered in the analyses. From the parametric study, researchers found that for a short beam, the buckling capacity is controlled by the section of the beam which is coped. The longer beam tested behaved as if it were uncoped attaining 90 percent of the uncoped section buckling capacity. Deeper copes were found to have buckled shapes which show significant distortion of the cross-section
at the end of the cope. As the cope depth increases, the lateral deflection of the top flange was found to increase. Researchers concluded that the buckling strength of laterally unsupported steel beam copes is significantly affected by copes.

To verify the above mentioned parametric study, Cheng et al. (1988) conducted a series of six lateral buckling tests on a single W12x14 section with varying cope details. The wide flange section was manufactured from 50ksi steel. The test beam was connected to a column via a shear end plate connection with ¾in diameter A325 strength bolts. To simulate framing conditions, the bolts were tightened via the turn-of-the-nut method. From the tests, moment-rotation characteristics were obtained. Researchers were able to verify that the long and deep coped region controlled lateral buckling and that the lateral buckling strength is controlled by the interaction of the coped and uncoped section. From this series of tests, a proposed design method was recommended.

Additional research pertaining to beam copes can be found in the following references: Yura et al. (1982) conducted additional tests studying block shear in beam web connections, Ricles and Yura (1983) investigated the strength of double-row bolted-web connections where specimens used in testing included coped steel beams, and Gupta (1984) investigated the lateral torsional buckling behavior of coped beams.

2.4.5. CONNECTION CLASSIFICATION

The behavior of steel beam-to-column connections has been researched since 1917. Classification of joints as either pinned (no moment transfer) or rigid (complete rotational continuity) is very important to design considerations. However, as is the case for single plate shear connections, the connection is classified as a pinned connection. Yet, the
single plate framing connection is not completely a pinned connection as it sustains some moment from beam end rotation. Accordingly, an intermediate classification of “Semi-rigid Connection” may be applied to connections. Jones et al. (1983) discusses and reviews methods for incorporating semi-rigid end restraint into conventional analytical methods, experimental data of various structural connections, and means of modeling experimental data with analytical procedures. Also presented in this article is a literature review detailing the past research conducted on the topic of analyzing frames with semi-rigid connections. Rigidity is defined in part by flexural behavior, which is a combination of the moment sustained by the connection and the relative rotation of the two members joined by the connection. Jones et al. (1983) stated that rigidity is determined by many factors including but not limited to: depth and length of connected beams, connection parameters and configurations, and material properties. As such, many different methods and procedures are used in modeling end restraint.

Semi-rigid connection classification also describes the behavior of composite slabs with steel beams as joined via shear connections. The addition of the composite slab makes the connection behave in a manner that is neither pinned nor fully rigid. The introduction of the composite slab provides additional stiffness over a bare steel structure. Research was conducted by Wong et al. (1996) to devise a simple beam line model for the analysis of braced, non-sway composite frames with semi-rigid beam-to-column connections. Finite element analyses of subframe assemblies with varying connection rigidity were conducted. Using proposed equations and design charts for determining moments and deflection of composite beams, Wong et al. was able to verify the
developed finite element model. With a verified model of semi-rigid beam-to-column connections, Wong et al. recommended a design procedure.

Differing from bolts, steel pins do not have a head and are threadless, but behave similar to bolts in that steel pins are designed to sustain shear force transverse across its diameter. Bridge (1999) investigated the behavior of steel pins via an extensive testing program varying parameters including pin diameter, material properties of the pin, test fixture plate thickness and material properties, and clear edge distances. From this series of research, failure modes were to be identified and a unified design procedure was to be recommended. The test fixture tested a single pin in a double shear setup. Failure methods included large plate bearing deformations prior to shear rupture of the pin, shear rupture of the pin, and fracture of the plate at the cross section of the pin. From the results of these tests, a single design method composed from AS4100-1998 and Eurocode 3-1992 design methods was suggested for designing pins.
3.1. OVERVIEW

As seen from the research summaries presented in the literature review, Australia, Canada, Great Britain, New Zealand, and the United States have conducted extensive research into the behavior of single plate framing connections. As a result, different design methods have been adopted as national standards in each country. In an attempt to identify which method best predicts the behavior of single plate framing connections, the currently accepted design methods have been investigated. The investigation includes a comparison of all five major methods. In addition, the proposed methods from the United States including Richard, Astaneh, Sarkar and Wallace, Duggal and Wallace, and Ashakul are included in the comparison investigation.

The comparison of the various design methods are based upon the individual limit states or failure modes of the connections. Limit states are categorized by connection component (supported beam, connection bolts, single plate, and the weldment). The single plate framing design method listed in the AISC LRFD 3rd Edition Manual (AISC 2003) was chosen to be used as a basis for comparison. It should be noted that each method may utilize different strength reduction factors. However these variations are inherent to their own code. For comparison purposes, the strength reduction factors, as recommended by the AISC LRFD 3rd Edition Manual, are used in place of the differing design method strength reduction factors.

3.2. LIMIT STATE HIERARCHY

The design of single plate framing connections is based upon a hierarchy of limit states. Essentially, the connection must be adequate enough to sustain the shear
transferred from the supported beam. In addition, the connection must be ductile enough to sustain beam end rotation induced by applied loading. Accordingly, the connection must be designed to allow the onset of certain failure modes.

The hierarchy of limit states is defined as having the most ductile failures (such as bearing failures) occur first while brittle failures (such as sudden shear ruptures) occur after the ductile failures. By establishing such a hierarchy, the ductility of the connection is maintained and the maximum sustained shear can be obtained from the connection. An example of the hierarchy of limit states is as follows: the single plate must allow for bolt bearing of the bolt holes prior to sudden shear rupture of the connection bolts. In other words, the bearing and tear out strength must be less than the ultimate capacity of the connection bolts to facilitate the ductility of the connection. A second example is that the weld strength must be great enough to develop the tensile strength of the single plate.

The following is a typical hierarchy of limit states of single plate framing connections. Listed first are the most desired and ductile failures. Brittle failure limit states are listed toward the end of the list.

- Bearing yielding of plate and beam web bolt holes
- Single plate shear yielding
- Single plate flexural yielding due to bending
- Flexural yielding of test beam
- Single plate shear fracture
- Single plate block shear
- Block shear of coped beams
- Fracture of bolts
- Fracture of welds
3.3. **AISC LRFD 3RD EDITION MANUAL SINGLE PLATE DESIGN METHOD**

To predict the strength of the tested single plate framing connections used in testing, the limit states described in Section 3.2 were calculated for. The following subsections present each applicable limit state, listing all necessary equations. All strength calculations were based upon design procedures listed in the AISC LRFD 3rd Edition Manual (AISC 2003). Resistance factors were incorporated in limit state calculations. Inclusion of the resistance factors was deemed to be beneficial because resistance factors vary from 0.75 to 0.9 for different limit states. Comparisons, presented later in this thesis using design strengths, provide a direct comparison to experimental values while including the appropriate factors. All calculations from the following sections were conducted using U.S. customary units (kips, in., etc).

3.3.1. **BOLT GROUP SHEAR STRENGTH**

Of particular focus was the nominal shear strength of the bolt group. The prediction for the strength of the connection was based upon bolt shear strength with zero eccentricity. The design capacity of the bolt group was calculated using the following equation:

\[
\phi V_n = \phi n F_v A_b
\]

(EQ 3.1)

where:

- \( \phi \) = 0.75
- \( n \) = number of bolts
- \( A_b \) = cross-sectional area of a single bolt
- \( F_v \) = nominal single plane shear strength of a single bolt
- \( \phi V_n \) = design shear strength of the bolt group
From Table J3.2 of the AISC LRFD 3rd Edition Manual (AISC 2003), typical values for \( F_v \) are 48ksi for an A325-N strength bolt, or 60ksi for an A490-N strength bolt.

### 3.3.2. SINGLE PLATE STRENGTH

#### Shear Yielding

The design shear yielding strength of the plate was calculated as follows:

\[
\phi V_n = \phi 0.6 F_y A_g
\]

(EQ 3.2)

where: \( \phi = 0.90 \)

\( A_g \) = gross area of the single plate where:

\( A_g = L_p t_p \)  

(EQ 3.3)

where:

\( t_p \) = thickness of the single plate

\( L_p \) = length of the single plate

\( F_y \) = yield stress of the single plate

\( \phi V_n \) = design shear yield strength of the single plate

#### Shear Rupture

The design shear rupture strength of the plate was calculated using Equation 3.4.

\[
\phi V_n = \phi 0.6 F_u A_n
\]

(EQ 3.4)

where: \( \phi = 0.75 \)

\( A_n \) = net area of the single plate where:

\[
A_n = \left( L_p - n \left( d_h + \frac{1}{16} \right) \right) t_p
\]

(EQ 3.5)

where:

\( d_h \) = bolt hole diameter

\( n \) = number of bolts

\( t_p \) = thickness of the single plate
\[ L_p = \text{length of the single plate} \]
\[ F_u = \text{ultimate strength of the single plate} \]
\[ \phi V_n = \text{design shear rupture strength of the single plate} \]

**Single Plate Bending**

The single plate bending strength prediction is based upon \( a \), the distance from the weld line to the bolt line. This also assumes zero eccentricity of the bolts. The design bending strength of the single plate was calculated using Equation 3.6:

\[
\phi V_n = \phi \left( \frac{M_n}{a} \right) \tag{EQ 3.6}
\]

where:
\[
\phi = 0.90
\]
\[ a = \text{distance between the weld line and bolt line} \]
\[ \phi V_n = \text{design bending strength of the single plate} \]
\[ M_n = \text{maximum sustainable moment of the single plate} \]

moment, which is defined as follows:

\[ M_n = F_y S_x \tag{EQ 3.7} \]

where:
\[ F_y = \text{yield stress of the single plate} \]
\[ S_x = \text{elastic section modulus of the single plate} \]

which is defined as follows:

\[ S_x = \frac{t_p L_p^2}{6} \tag{EQ 3.8} \]

where:
\[ t_p = \text{thickness of the single plate} \]
\[ L_p = \text{length of the single plate} \]
**Bearing/Tear out**

Bearing and tear out of the bolt holes was calculated for by first determining each clear distance, $L_c$, associated with the top bolt hole or interior bolt hole. The clear distance is associated with the load path. Because shear is applied to the single plate in the longitudinal direction, the clear distance for the top bolt hole is defined as the distance between the edge of the bolt hole to the edge of the plate. The clear distance for an interior bolt is defined as the distance between the edges of two consecutive bolt holes. The clear distance is the spacing that is used to determine whether bearing or tear out will control. Equations 3.9 & 3.10 show how bearing and tear out of the bolt holes of the single plate were calculated.

For $L_c > 2d_b$, bearing controls:

$$\phi V_n = \phi 2.4d_b t_p F_u$$  \hspace{1cm} (EQ 3.9)

For $L_c < 2d_b$, tear out controls:

$$\phi V_n = \phi 1.2L_c t_p F_u$$  \hspace{1cm} (EQ 3.10)

where:  
\begin{align*}
\phi &= 0.75 \\
d_b &= \text{bolt diameter} \\
t_p &= \text{thickness of the single plate} \\
F_u &= \text{ultimate strength of the single plate} \\
\phi V_n &= \text{design bearing and tear out strength of the single plate}
\end{align*}

Bearing and tear out was then calculated for by adding the controlling bearing or tear out failure strength of each bolt hole.
Block Shear

Block shear strength of the single plate was calculated for by Equations 3.11 & 3.12. The following two equations calculate the sum of the shear strengths of the failure path(s) and the tensile strength of the perpendicular segment.

When \( F_u A_m \geq 0.6 F_u A_{nv} \):

\[
\phi V_n = \phi \left[ 0.6 F_y A_{gv} + F_u A_m \right] \leq \phi \left[ 0.6 F_u A_{nv} + F_u A_{ut} \right] \quad \text{(EQ 3.11)}
\]

When \( F_u A_m < 0.6 F_u A_{nv} \):

\[
\phi V_n = \phi \left[ 0.6 F_u A_{nv} + F_y A_{gt} \right] \leq \phi \left[ 0.6 F_u A_{nv} + F_u A_{ut} \right] \quad \text{(EQ 3.12)}
\]

where: \( \phi = 0.75 \)

\( A_{gv} \) = gross area subject to shear
\( A_{gt} \) = gross area subject to tension
\( A_{nv} \) = net area subject to shear
\( A_{nt} \) = net area subject to tension
\( F_u \) = ultimate strength of the single plate
\( F_y \) = yield stress of the single plate

3.3.3. BEAM AND SUPPORTING MEMBER STRENGTH

Shear Strength

The shear strength of the beam was calculated for using Equation 3.13 and is a function of the area of the supported beam web.

\[
\phi V_n = \phi \ 0.6 F_y A_w \quad \text{(EQ 3.13)}
\]
where: $\phi = 0.90$

$A_w = \text{gross area of the web, which is defined as:}$

$A_w = d_t_w$  \hspace{1cm} (EQ 3.14)

where: $d = \text{depth of the supported beam}$

$t_w = \text{thickness of the supported beam web}$

$F_y = \text{yield stress of the supported beam}$

$\phi V_n = \text{design shear yield strength of the supported beam web}$

**Bearing/Tear out**

Bearing and tear out of the bolt holes was calculated for by first determining each clear distance, $L_c$, associated with the top bolt hole or interior bolt hole. The clear distance is associated with the load path where shear is applied to the supported beam in the vertical direction. For the supported beam, bearing and tear out is calculated for a cope beam, which is the worst case scenario. The clear distance for the top bolt hole is defined as the distance between the top edge of the bolt hole to the horizontal edge of the supported beam cope. The clear distance between two consecutive interior bolts is defined as the distance between the edges of the bolt holes. The clear distance is the spacing that is used to determine whether bearing or tear out will control. Equations 3.15 & 3.16 show how bearing and tear out of the bolt holes of the supported beam web were calculated.

For $L_c > 2d_b$, bearing controls:

$$\phi V_n = \phi 2.4d_t_wF_u$$  \hspace{1cm} (EQ 3.15)

For $L_c < 2d_b$, tear out controls:
\[ \phi V_n = \phi 1.2 L_t t_w F_u \]  

(EQ 3.16)

where:  
\[ \phi = 0.75 \]

\[ d_b = \text{bolt diameter} \]

\[ t_w = \text{thickness of the supported beam web} \]

\[ F_u = \text{ultimate strength of the supported beam} \]

\[ \phi V_n = \text{design bearing and tear out strength per bolt hole of the supported beam web} \]

Bearing and tear out was then calculated for by adding the controlling bearing or tear out failure strength of each bolt hole.

**Block Shear (Coped Beam Only)**

Block shear strength of the beam was calculated from Equations 3.17 & 3.18. These two equations calculate the sum of the shear strengths of the failure path(s) and the tensile strength on the perpendicular segment.

When \( F_u A_{nt} \geq 0.6 F_u A_{nv} \):

\[ \phi V_n = \phi \left[ 0.6 F_u A_{gy} + F_u A_{nt} \right] \leq \phi \left[ 0.6 F_u A_{nv} + F_u A_{nt} \right] \]  

(EQ 3.17)

When \( F_u A_{nt} < 0.6 F_u A_{nv} \):

\[ \phi V_n = \phi \left[ 0.6 F_u A_{nv} + F_u A_{gt} \right] \leq \phi \left[ 0.6 F_u A_{nv} + F_u A_{nt} \right] \]  

(EQ 3.18)

where:  
\[ \phi = 0.75 \]

\[ A_{gy} = \text{gross area subject to shear} \]

\[ A_{gt} = \text{gross area subject to tension} \]

\[ A_{nv} = \text{net area subject to shear} \]
\[ A_{nt} = \text{net area subject to tension} \]
\[ F_u = \text{ultimate strength of the single plate} \]
\[ F_y = \text{yield stress of the single plate} \]
\[ \phi V_n = \text{design shear rupture strength of the supported beam web} \]

3.3.4. WELD STRENGTH

Weld Size

The thickness of the weld is specified to be three-fourths of the plate thickness. To verify that the thickness of the weld was satisfactory for eccentrically loaded weld groups, the following calculations were made.

- Using Table 8-5 (AISC 2003), the coefficient for eccentrically loaded weld groups, \( C \), was determined as follows:
  
  a. Table 8-5 variable \( a \), which is the out of plane eccentricity component, was calculated from the following equation:

  \[ a = \frac{a}{l} \]  \hspace{1cm} (EQ 3.19)

  where: \( a \) = weld line to bolt line distance

  \( l \) = length of the weld (also the length of the single plate, \( L_p \))

  b. The \( C \) coefficient was obtained from Table 8-5 by cross-referencing variable \( a \) from Equation 3.19 with \( k \) (which is equal to zero), which applies to the special case of a load not in the plane of the weld group

- Using Table 8-4 (AISC 2003), the electrode strength coefficient, \( C_I \) was determined.
With these variables found, the required thickness of the fillet welds was determined. This required value was compared with the specified weld thickness to ensure an adequate design.

\[ D \geq \frac{\phi R_n}{C C_1 I} \]  

(EQ 3.20)

where:  
\( D \) = number in sixteenths of an inch in weld leg length  
\( \phi R_n \) = design shear strength of the bolt group from Equation 3.1

Once the weld size was verified, the following steps were used in determining the weld strength. The strength of the weld was designed according to design considerations from Part 8 Design Considerations for Welds of the AISC LRFD 3rd Edition Manual (AISC 2003). The following calculations are comprised of a base metal strength calculation, the design strength of a welded joint, and a calculation which takes into consideration the combined effects of shear and moment at the weldment.

**Base Metal at Weld**

Localized failure of the base metal near the connection weld was also checked. For single plates manufactured from lesser strength steel, base metal failure of the single plate will occur as a tearing of the single plate near the weld. Shown as Equation 3.21, the shear rupture strength of the weld (represented as the shear strength per longitudinal inch of weld) was calculated for as follows:

\[ \phi R_n = \phi 0.6 F_{u} t_p \]  

(EQ 3.21)
where: $\phi = 0.75$

$t_p = \text{thickness of the single plate}$

$F_u = \text{ultimate tensile strength of the single plate}$

**Weld Design Strength**

Equation 3.22a shows the calculation used in determining the weld strength based upon the throat dimension of a single line of weld.

$$\phi R_n = \phi 0.6 F_{EXX} \left( \frac{\sqrt{2}}{2} \right) \frac{D}{16} l$$  \hspace{1cm} (EQ 3.22a)

where: $\phi = 0.75$

$l = \text{length of weld}$

$D = \text{number in sixteenths of an inch in weld leg length}$

$F_{EXX} = \text{minimum specified weld metal strength, where:}$

$= 70\text{ksi for E70xx electrodes}$

Similar to Equation 3.21, Equation 3.22a can be written to represent the weld strength per longitudinal inch of weld. In simplified form, Equation 3.22a can be written as follows:

$$\phi R_n = 1.392 D n_{\text{welds}}$$  \hspace{1cm} (EQ 3.22b)

where: $n_{\text{welds}} = \text{number of longitudinal fillet welds}$

Equation 3.23 was then used to calculate the combined effect of shear and moment on the weld by incorporating the previously found variables from the section pertaining to the weld size and values from Equations 3.20 and 3.22b.

$$\phi R_n = C C_i D I \left( \frac{\phi R_n \text{ (Equation 3.21)}}{\phi R_n \text{ (Equation 3.22b)}} \right)$$  \hspace{1cm} (EQ 3.23)
Comparing the results of Equations 3.21 and 3.22b, the lesser value will control. The shear strength of the weld was then found by multiplying the minimum value of the two above mentioned equations by the longitudinal length of weld, \( l \). Comparing this design strength with the strength calculated in Equation 3.23, the lesser of these two values will be the design strength for the weldment.

### 3.4. COMPARISON TO OTHER SINGLE PLATE DESIGN METHODS

#### 3.4.1. OVERVIEW

The purpose of this investigation is to identify the differences in the compared design methods in relation to the AISC LRFD 3rd Edition Manual (AISC 2003). Included in the investigation were design methods recommended by researchers based in the United States, such as Richard, Astaneh, Sarkar and Wallace, Duggal and Wallace, and Ashakul. Similarly, design methods accepted and presented as code in the United States, Australia, Canada, Great Britain, and New Zealand were incorporated in the analysis and comparison. It should be noted that the Canadian design method was determined to be similar to the method recommended in the AISC LRFD 3rd Edition Manual (AISC 2003). Both methods are based upon recommendations from Astaneh. Accordingly, the Canadian single plate framing design method is not presented due to the similarity to the AISC accepted method. In addition, Ashakul did not recommend a full design method, but improvements to the existing AISC LRFD 3rd Edition Manual (AISC 2003). The following subsection will summarize the existing methods by detailing the geometric checks and limit states that are incorporated in each design method.

The remaining subsections will investigate single plate framing connection limit states of the following connection components: the connection bolts, single plate,
supported test beam, and weldment. The limit states presented in Section 3.2 will be compared. In these subsections, which are categorized by connection component, only the differences in limit state calculation methods will be listed.

3.4.2. VARIATIONS IN DESIGN METHODS

The first comparisons of the various design methods can be found as Tables 3.1 and 3.2. These comparison tables present the variations of each design method in tabular format. Table 3.1 lists the design strength checks as calculated for in each design method. Investigation of the various design methods show that certain design methods do not have explicit calculations for specific limit states as they are accounted for via dimensional checks. For example, Astaneh recommends specific design checks for horizontal and vertical clear edge distances to account for bearing and tear out failure of the single plate. As such, Table 3.1 will indicate that the design strength was not explicitly calculated for. Table 3.2 will present the various dimensional checks as recommended in the design methodologies.
### TABLE 3.1 SUMMARY OF DESIGN METHODS (DESIGN STRENGTH CHECKS)

<table>
<thead>
<tr>
<th>Component Strength Checks</th>
<th>Design Method Recommended by:</th>
<th>Astaneh</th>
<th>AISC 2003</th>
<th>Richard</th>
<th>Sarkar &amp; Wallace</th>
<th>Duggal &amp; Wallace</th>
<th>Ashakul</th>
<th>Australia</th>
<th>New Zealand</th>
<th>Great Britain</th>
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<td>Parameters Considered</td>
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<td>St, SS</td>
<td>St, SS</td>
<td>SS</td>
<td>---</td>
<td>St</td>
<td>---</td>
<td>St</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Number of Columns of Bolts Allowed</td>
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<td>S</td>
<td>S</td>
<td>S</td>
<td>---</td>
<td>S, D</td>
<td>S</td>
<td>S, D</td>
<td></td>
</tr>
<tr>
<td></td>
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<td>Snug, Full</td>
<td>Snug, Full</td>
<td>Full</td>
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<td>Snug, Full</td>
<td>---</td>
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<td>Snug</td>
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<td>●</td>
<td>●</td>
<td>●</td>
<td>●</td>
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<td>●</td>
<td>●</td>
<td>●</td>
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<td>Bearing Capacity</td>
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<td></td>
<td>(Flexible)</td>
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<td>●</td>
<td>●</td>
<td>---</td>
<td>---</td>
<td>●</td>
<td>5</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
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<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>●</td>
<td>---</td>
<td>●</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Combined Flexure &amp; Shear</td>
<td>---</td>
<td>---</td>
<td>---</td>
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<td>●</td>
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Note: This table is continued on the following page.
### TABLE 3.1 (CONTINUED)

<table>
<thead>
<tr>
<th>Component Strength Checks</th>
<th>Design Method Recommended by:</th>
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<tbody>
<tr>
<td></td>
<td>Astaneh</td>
</tr>
<tr>
<td>Weld Strength (Rigid)</td>
<td>◊</td>
</tr>
<tr>
<td>(Flexible)</td>
<td>◊</td>
</tr>
<tr>
<td>Eccentricity, $e_w$ (in) (Rigid)</td>
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</tr>
<tr>
<td>(Flexible)</td>
<td>●</td>
</tr>
<tr>
<td>Combined Flexure &amp; Shear</td>
<td>◊</td>
</tr>
</tbody>
</table>

Notes: 1. This method consists of recommendations to the current AISC 2003 method
2. Calculations are made with respect to the following: (a) Bearing due to eccentric shear, (b) Tearout with and without eccentric shear considerations
3. Calculations are made with respect to Bearing only
4. Weld Strength is calculated with respect to ultimate tensile strength while allowing full tensile development of the single plate.
5. Calculated with plastic section modulus
6. Calculation is made considering the elastic section modulus at low shear forces while the plastic section modulus is used when high shear forces are present
7. For shear forces greater than 75 percent of the shear yielding strength, combined flexure and shear is considered

**Table Key:**
- Hole Condition:
  - St: Indicates a standard hole condition
  - SS: Indicates a short-slotted hole condition
- Number of Columns of Bolts Allowed:
  - S: Indicates that a single column of bolts is considered
  - D: Indicates that a double column of bolts is considered
- Pretensioning:
  - Snug: Indicates considerations for snug-tightened bolts
  - Full: Indicates considerations for fully-tightened bolts
  - Indicates an actual strength calculation
  - Although not explicitly calculated, this consideration is satisfied by the dimensional requirement
  - Indicates that design method does not include considerations for this support condition
  - Indicates that this was not a consideration in the design methodology
### TABLE 3.2 SUMMARY OF DESIGN METHODS (DIMENSIONAL CHECKS)

<table>
<thead>
<tr>
<th>Plate Dimensional Checks</th>
<th>Design Method Recommended by:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld thickness, $D$ (in)</td>
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</tr>
<tr>
<td>Astaneh</td>
<td>$D \geq (1.46(F_y/F_yw))^{-1}$</td>
</tr>
<tr>
<td>AISC 2003</td>
<td>$D \geq 0.75s_y$</td>
</tr>
<tr>
<td>Richard</td>
<td>$D = 0.75s_y$</td>
</tr>
<tr>
<td>Sarkar &amp; Wallace</td>
<td>$D = 0.75s_y$</td>
</tr>
<tr>
<td>Duggal &amp; Wallace</td>
<td>$D = 0.75s_y$</td>
</tr>
<tr>
<td>Ashakul</td>
<td>$D = 0.75s_y$</td>
</tr>
<tr>
<td>Australia</td>
<td>$D = 0.80s_y$</td>
</tr>
<tr>
<td>Canada</td>
<td>$D = 0.80s_y$</td>
</tr>
<tr>
<td>New Zealand</td>
<td>$D = 0.80s_y$</td>
</tr>
<tr>
<td>Great Britain</td>
<td>$D = 0.80s_y$</td>
</tr>
<tr>
<td>Weld line to bolt line distance, $a$ (in)</td>
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</tr>
<tr>
<td>Astaneh</td>
<td>$2.5\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>AISC 2003</td>
<td>$2.5\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>Richard</td>
<td>$2.5\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>Sarkar &amp; Wallace</td>
<td>$2.5\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>Duggal &amp; Wallace</td>
<td>$2.5\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>Ashakul</td>
<td>$2.5\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>Australia</td>
<td>$2.5\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>Canada</td>
<td>$2.17\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>New Zealand</td>
<td>$3\leq a &lt; 3.5$</td>
</tr>
<tr>
<td>Great Britain</td>
<td>$1.97\leq a &lt; 3.5$</td>
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<tr>
<td>Number of bolts, $n$</td>
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<tr>
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<td>$\leq n \leq 9$</td>
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<tr>
<td>AISC 2003</td>
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</tr>
<tr>
<td>Richard</td>
<td>$\leq n \leq 9$</td>
</tr>
<tr>
<td>Sarkar &amp; Wallace</td>
<td>$\leq n \leq 9$</td>
</tr>
<tr>
<td>Duggal &amp; Wallace</td>
<td>$\leq n \leq 9$</td>
</tr>
<tr>
<td>Ashakul</td>
<td>$\leq n \leq 9$</td>
</tr>
<tr>
<td>Australia</td>
<td>$\leq n \leq 10$</td>
</tr>
<tr>
<td>Canada</td>
<td>$\leq n \leq 10$</td>
</tr>
<tr>
<td>New Zealand</td>
<td>$\leq n \leq 10$</td>
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<tr>
<td>Great Britain</td>
<td>$\leq n \leq 10$</td>
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<tr>
<td>Vertical edge distance, $L_{ve}$ (in)</td>
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<td>$L_{ve} \geq 1.5d_b$</td>
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<td>$L_{ve} \geq 1.5d_b$</td>
</tr>
<tr>
<td>Richard</td>
<td>$L_{ve} = 0.5p$</td>
</tr>
<tr>
<td>Sarkar &amp; Wallace</td>
<td>$L_{ve} = 0.5p$</td>
</tr>
<tr>
<td>Duggal &amp; Wallace</td>
<td>$L_{ve} = 0.5p$</td>
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<td>Great Britain</td>
<td>$L_{ve} \geq 2d_b$</td>
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<td>Sarkar &amp; Wallace</td>
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<td>---</td>
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<td>Sarkar &amp; Wallace</td>
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<td>Duggal &amp; Wallace</td>
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<td>---</td>
</tr>
<tr>
<td>Richard</td>
<td>---</td>
</tr>
<tr>
<td>Sarkar &amp; Wallace</td>
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</tr>
<tr>
<td>Duggal &amp; Wallace</td>
<td>---</td>
</tr>
<tr>
<td>Ashakul</td>
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</tr>
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<td>Australia</td>
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<th>Design Method Recommended by:</th>
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<tr>
<td></td>
<td>Astaneh</td>
</tr>
<tr>
<td>Distance from end of supported beam to face of support (in)</td>
<td>---</td>
</tr>
<tr>
<td>Distance from top of supported beam to first bolt hole (in)</td>
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</tr>
<tr>
<td>Distance from top of supported beam to top horiz. single plate edge (in)</td>
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</tr>
<tr>
<td>Distance from horizontal edge of beam cope to the first bolt hole (in)</td>
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</tr>
<tr>
<td>Distance from end of supported beam to vertical bolt centerline (in)</td>
<td>---</td>
</tr>
</tbody>
</table>

Notes:
1. Weld is designed to fully develop the plate.
2. Consideration ensures no contact between end of supported beam and face of support
3. Consideration for a rigid support condition
4. Consideration for a flexible support condition
5. Thickness consideration for when the other consideration is not met. A separate consideration is presented to maintain ductility for bolt bearing.

**Table Key:**
- \(c_p\): Table J3.8 of AISC (1994) or Table J3.6 of AISC (1989)
- \(d\): Depth of the supported beam
- \(d_b\): Diameter of the connection bolt
- \(k_{beam}\): Distance from outer face of flange to web toe of fillet of beam
- \(k_{girder}\): Distance from outer face of flange to web toe of fillet of girders
- \(n_p\): Number of bolt rows
- \(s, p\): Vertical bolt pitch
- \(F_y\): Specified yield stress of the single plate
- \(F_yw\): Specified yield stress of the weld electrode
- \(K\): The plate buckling coefficient tabulated in Part 9 of the AISC LRFD 3rd Edition Manual (AISC 2003) in the section allocated to the local buckling of beams coped at the top and bottom flanges
- \(T\): Distance between web toes of fillets at top and at bottom of web
The primary differences between the design method accepted by the AISC LRFD 3rd Edition Manual (AISC 2003) and various other methods are as follows:

- Bolt eccentricity considerations, as used in bolt group shear strength calculations, are omitted or calculated for differently in other design methods.
- Bearing and Tear out of the bolts acting upon the single plate material is calculated differently in other design methods.
- Flexural Rupture of the single plate is calculated by New Zealand.
- Combined Flexure and Shear Yielding of the single plate is calculated for by several other researchers and countries.
- The AISC 2003 method assumes elastic behavior of the single plate when calculating for flexural yielding while other design methods assume plastic behavior of the single plate in flexure.
- Various design methods incorporate supported beam span length-to-depth ($L/d$) ratios in design strength calculations.

As seen in Table 3.2, dimensional checks assure that several design strength calculations are satisfied. Such is the case for the design thickness of the weld leg length, bearing and tear out, and as seen for use by other countries, clearance checks to ensure that the supported test beam does not contact the supporting member. Some design methods require exact dimensions, while other design methods utilize less than or equal to, greater than or equal to, or must be equal to ranges such that design strengths are satisfied.
Due to the multitude of variables specified in the various design methods, the following list is composed of commonly used symbols and should be referenced as necessary.

\[ a = \] distance between the bolt line and the weld line
\[ d_b = \] bolt diameter
\[ d_h = \] bolt hole diameter
\[ e_b = \] bolt eccentricity
\[ n = \] number of bolts in the connection
\[ n_p = \] number of bolt rows
\[ p, s = \] vertical spacing between the bolt holes center lines
\[ t_p = \] thickness of the single plate
\[ t_w = \] thickness of the supported beam web
\[ A_b = \] cross-sectional area of a single bolt
\[ E = \] Young’s Modulus
\[ F_v = \] nominal single plane shear strength of a single bolt
\[ L = \] span of supported beam
\[ L_{eh} = \] horizontal clear edge distance
\[ L_{ev} = \] vertical clear edge distance
\[ L_p = \] length of the single plate
3.4.4. CONNECTION BOLTS

-AISC (2003), Astaneh et al. (1993)-

Specified in Section 6.3.1 Bolt Group Shear Strength, the calculation for the bolt group capacity without eccentric shear considerations is presented. However, the AISC manual designs the connection bolts such that both shear, $R_u$ and eccentric moment, $R_ue_b$, are sustained. As such, eccentric shear considerations (measured from the bolt line) in calculating the bolt group capacity are specified. The eccentric shear or bolt eccentricity equations presented in the manual are based upon Astaneh et al.s (1988, 1990, 1992) research. The following equations are the bolt eccentricity equations recommended in the AISC LRFD 3rd Edition Manual (AISC 2003).

- For a rigid support condition with standard holes:

$$e_b = \left| (n-1) - a \right|$$  \hspace{1cm} (EQ 3.24)

- For a rigid support condition with short-slotted holes:

$$e_b = \left\lfloor \frac{(2n)}{3} - a \right\rfloor$$  \hspace{1cm} (EQ 3.25)

- For a flexible support condition with standard holes:

$$e_b = \left\lfloor (n-1) - a \right\rfloor \geq a$$  \hspace{1cm} (EQ 3.26)

- For a flexible support condition with short-slotted holes:

$$e_b = \left\lfloor \frac{(2n)}{3} - a \right\rfloor \geq a$$  \hspace{1cm} (EQ 3.27)
Ashakul recommended specific equations for calculating the bolt group shear capacity for single plate connections with a rigid support condition. Ashakul does not specify bolt eccentricity equations, however, certain connections should be designed considering the effects of eccentricity. This eccentricity or moment is created by horizontal forces occurring from the bearing resistance of the single plate. In addition, Ashakul stated that the greater the vertical distance between the neutral axis of the supported beam and the bolt group centroid, the more the shear capacity of the bolts is reduced. For the following bolt group design capacity calculation, Ashakul specifies that the bolt group centroid must coincide with the neutral axis of the supported beam.

When: \( t_p \leq \frac{d_b}{2} \left( \frac{36}{F_y} \right) \); \hspace{1cm} (EQ 3.28)

- \( \phi V_n = \phi 0.95F_vA_b n \) \hspace{1cm} (EQ 3.29)

When: \( \frac{d_b}{2} \left( \frac{36}{F_y} \right) < t_p \leq (0.7d_b) \left( \frac{36}{F_y} \right) \); \hspace{1cm} (EQ 3.30)

- For \( n \leq 5 \),
  \( \phi V_n = \phi 0.84F_vA_b n \) \hspace{1cm} (EQ 3.31)

- For \( 5 \leq n \leq 7 \),
  \( \phi V_n = \phi \Sigma r_n \) \hspace{1cm} (EQ 3.32)

where:

(a) Bolts with a vertical distance to the neutral axis greater than 6in

\[ r_n = 0.64F_vA_b \] \hspace{1cm} (EQ 3.33)
(b) Bolts with a vertical distance to the neutral axis within 6in

\[ r_n = 0.70 F_y A_h \]  \hspace{1cm} (EQ 3.34)

where:  \( \phi = 0.75 \)

\[ r_n = \text{bolt shear strength of a single bolt} \]
\[ F_y = \text{yield strength of the single plate} \]
\[ \phi V_n = \text{design shear strength of the bolt group} \]

-Australia (OneSteel Market Mills (2000))-  

The Australian design method for single plate framing connections designs the connection bolts to be equivalent to the minimum strength of either the single shear capacity of the bolt group or the bearing or tear out strength of the single plate (which is specified in Section 3.4.5). It should be noted that the bolt group shear capacity is combined in the bearing considerations as a third calculation inclusive of \( \phi V_{df} \). Aside from this, the single shear capacity of the bolt group is calculated by determining the strength of a single bolt in single shear as described in Section 3.3.1 and then multiplying this value by \( Z_b \) of Equation 3.50, the section modulus for the extreme bolt in the bolt group.

-New Zealand (HERA (1999))-  

The New Zealand single plate design method is similar to the Australian design method in that the single bolt single shear capacity of a single bolt is multiplied by \( Z_b \), the bolt interaction factor. The resulting design strength is stated to be the bolt group capacity at the reaction eccentricity, \( e \), or rather the weld line to bolt line distance, \( a \). As
specified in the New Zealand design method, the shear capacity of the bolt group is defined as follows:

\[ \phi V_b = Z_b \phi V_f \]  
\[ \text{(EQ 3.35)} \]

where:  
\[ \phi V_b = \text{bolt group design shear capacity at eccentricity, } e \]  
\[ \phi V_f = \text{single bolt design shear capacity as defined in Section 3.3.1} \]  
\[ Z_b = \text{bolt interaction factor for single line of bolts,} \]
\[ n_p \neq 1: \]

\[ Z_b = \frac{n_p}{\sqrt{1 + \left[ \frac{6e}{s(n_p + 1)} \right]^2}} \]  
\[ \text{(EQ 3.36)} \]

In addition to the single plane shear bolt group shear capacity check, the New Zealand single plate design method specifies a bolt group moment capacity, \( \phi M_b \).

Again, the reaction eccentricity, \( e \), is equivalent to the weld line to bolt line distance, \( a \).

\[ \phi M_b = n_p Z_e \phi V_f e \]  
\[ \text{(EQ 3.37)} \]

where:  
\[ Z_e = \text{bolt group flexure factor, single line of bolts,} \]
\[ n_p \neq 1 \]

\[ Z_e = \frac{s(n_p + 1)}{6e} \]  
\[ \text{(EQ 3.38)} \]

\( \phi M_b = \text{bolt group moment capacity} \)
\( \phi V_f = \text{single bolt design shear capacity as defined in Section 3.3.1} \)
The shear capacity of the bolt group as specified in the Great Britain design method is limited by the bearing strength per bolt of the single plate or the supported beam. In addition, the capacity of the bolt group is limited by the force on the outermost bolt as it is subjected to combined shear and moment. The bolt group shear capacity is calculated as follows:

- Check bolt group capacity (taking into account the eccentricity, \(a\))

\[
F_r \leq P_{bs} \tag{EQ 3.39}
\]

where: 
- \(F_r\) = resultant force on outermost bolt due to direct shear and moment as defined below
- \(P_{bs}\) = the bearing capacity per bolt (defined as Equation 3.58)

The resultant force on the outermost bolt resulting from a combination of direct shear and moment, 

\[
F_r = \left( F_v^2 + F_m^2 \right)^{1/2} \tag{EQ 3.40}
\]

The force due to direct shear on the bolt, 

\[
F_v = \frac{Q}{n} \tag{EQ 3.41}
\]

The force on the outermost bolt is from moment, 

\[
F_m = \frac{Qa}{Z_b} \tag{EQ 3.42}
\]

\(Q\) = shear reaction at the weld line
\(Z_b\) = elastic section modulus of the bolt group
\(Z_b = \frac{n(n+1)p}{6} \tag{EQ 3.43}\)
3.4.5. SINGLE PLATE

*Shear Yielding*

_Ashakul (2004)_

As presented by Ashakul, the shear yielding capacity of the single plate is slightly modified from the AISC manual method in that only part of the plate cross section is used as the one of the vertical clear distances of the plate is unaccounted for. The shear yielding equation is as follows:

\[ \phi R_n = \phi 0.6 F_y \left[(n-1)s + L_{ce}\right] \]  

(EQ 3.44)

where:  
\[ \phi = 0.90 \]

\[ F_y = \text{yield strength of the single plate} \]

_New Zealand (HERA (1999))-

The New Zealand design method incorporates a design calculation for combined shear and flexure of the single plate. For different applied shear forces, the method specifies that shear yielding only should be calculated for at lower applied shear while combined shear and flexure should be considered for the single plate at higher applied shear. A detailed calculation is presented in the *Combined Shear and Flexure* section.

*Shear Rupture*

_Astaneh et al. (1993), Sarkar and Wallace (1992), Great Britain (BCSA 2002)_

The primary difference that occurs with the calculation of single plate shear rupture is that the diameter of the bolt (not the hole diameter) is increased by 1/16\textsuperscript{th} of an inch.
\[ \phi V_n = \phi (0.6 F_u) A_n \] (EQ 3.45)

where: \[ \phi = 0.75 \]

\[ A_n = \text{net area of the single plate, and is defined as follows:} \]

\[ A_n = \left( L_p - n \left( d_b + \frac{1}{16} \right) \right) f_p \] (EQ 3.46)

\[ F_u = \text{specified ultimate stress of the single plate} \]

\[ \phi V_n = \text{design rupture strength of the single plate} \]

-New Zealand (HERA (1999))-  

For the calculation of shear rupture of the single plate, the New Zealand design method uses different considerations at various applied shears. For lower shear forces, direct shear rupture of the plate is calculated for. However, for higher applied shears, a combination of shear rupture and flexure is considered.

-Shear rupture:

\[ \phi V_{n\text{si}} = \phi_s 0.6 f_{ui} A_{ni} \text{ if } V' \leq 0.75 \phi V_{n\text{fi}} \] (EQ 3.47)

-Shear rupture and combined flexure:

\[ \phi V_{n\text{si}} = \phi_s 0.6 f_{ui} A_{ni} \left[ 2.2 - \left( \frac{1.6 V'}{\phi V_{n\text{fi}}} \right) \right] \text{ if } V' > 0.75 \phi V_{n\text{fi}} \] (EQ 3.48)

where: \[ \phi_s = \text{steel capacity reduction factor, 0.75} \]

\[ f_{ui} = \text{single plate tensile strength} \]

\[ A_{ni} = \text{single plate net area (defined as Equation 3.46)} \]
\[ V^* = \text{design shear force} \]
\[ \phi V_{nsi} = \text{single plate net ultimate design shear capacity} \]
\[ \phi V_{nfi} = \text{single plate net ultimate design shear capacity}, \]

which is defined as Equation 3.94

**Bearing and Tear out**

- *Australia (OneSteel Market Mills (2000))*-

Differing from the AISC manual method for calculating bearing and tear out, the Australian design method calculates bearing separately from tear out. Tear out is defined in both the horizontal and vertical directions. Bearing and tear out is defined as the minimum of the three means of calculating. It should be noted that instead of multiplying the individual bearing and tear out strengths by the number of bolts the effective section modulus, \( Z_b \) or \( Z_{eh1} \), for the extreme bolt is incorporated into the design calculation.

- **Bearing and Tear out Design Strength**

\[
\begin{align*}
\left| Z_b \left( \phi V_{df} \right) \right| & = \text{Min} \left| n_p \left( \phi V_{ev} \right) \right| \\
& \quad \left| n_p Z_{eh1} \left( \phi V_{eh} \right) \right| \\
\end{align*}
\]

\[ (\text{EQ 3.49}) \]

where: \( \phi V_{df} = \text{bearing capacity design strength (defined as Equation 3.57)} \)

\( \phi V_{eh} = \text{horizontal tear out design strength (defined below)} \)

\( \phi V_{ev} = \text{vertical tear out design strength (defined below)} \)
\[ Z_b = \frac{n_p}{\sqrt{1 + \frac{6e}{s(n_p + 1)^2}}} \quad \text{(EQ 3.50)} \]

\[ Z_{ebh} = \frac{s(n_p + 1)}{6e} \quad n_p \neq 1 \quad \text{(EQ 3.51)} \]

\[ e \quad \text{= eccentricity of the connection defined as follows:} \]

- Eccentricity Considerations

-For the bolt group:

\[ e = e_{bolts} \quad \text{(EQ 3.52)} \]

where:  
\[ e \quad \text{= design eccentricity of design shear force} \]
\[ e_{bolts} \quad \text{= eccentricity of design shear force measured from the bolt group centroid for a connection attached to a rigid support,} \]
\[ \text{and is defined as follows for a prismatic beam:} \]

\[ e_{bolts} = \frac{1.24k_dL^2}{12EI + 6(1.24)k_dL} \quad \text{(EQ 3.53)} \]

where:  
\[ k_d \quad \text{= connection stiffness (as defined in Table 3.3, which is presented in metric units)} \]
TABLE 3.3 CONNECTION DESIGN PARAMETERS FOR THE STIFF SUPPORT CONDITION

<table>
<thead>
<tr>
<th>Bolt Group Size (no. of rows x no. of columns)</th>
<th>Design Rotation Capacity ($\theta_c$) (mrad)</th>
<th>Design Rotation Stiffness ($k_d$) (kNm/rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 x 1</td>
<td>65</td>
<td>55</td>
</tr>
<tr>
<td>3 x 1</td>
<td>65</td>
<td>225</td>
</tr>
<tr>
<td>4 x 1</td>
<td>60</td>
<td>470</td>
</tr>
<tr>
<td>5 x 1</td>
<td>45</td>
<td>940</td>
</tr>
<tr>
<td>6 x 1</td>
<td>36</td>
<td>1535</td>
</tr>
<tr>
<td>7 x 1</td>
<td>30</td>
<td>2445</td>
</tr>
<tr>
<td>8 x 1</td>
<td>26</td>
<td>3545</td>
</tr>
<tr>
<td>9 x 1</td>
<td>23</td>
<td>5110</td>
</tr>
</tbody>
</table>

- Calculate eccentricity of each component (for a rigid support condition)

- For the single plate:

$$ e = e_{bolts} + a $$  \hspace{1cm} \text{(EQ 3.54)}

where:  
- $e$ = design eccentricity of design shear force
- $e_{bolts}$ = eccentricity of design shear force measured from the bolt group centroid for a connection attached to a stiff support

- Calculate eccentricity of each component (for a flexible support condition)

- For the bolt group and single plate:

$$ e = a $$  \hspace{1cm} \text{(EQ 3.55)}

where:  
- $e$ = design eccentricity of design shear force

- For the coped beam web:

$$ e = a - a_{el} + L_c $$  \hspace{1cm} \text{(EQ 3.56)}

where:  
- $e$ = design eccentricity of design shear force
\[ a_{el} = \text{horizontal distance between the vertical end of the beam and bolt line} \]

\[ L_c = \text{horizontal length of the beam cope} \]

- **Bearing**

\[ \phi V_{df} = \min \left[ \phi (3.2) f_{ui} d_r f_{ui}, \phi (3.2) f_{uw} d_r f_{uw} \right] \quad \text{(EQ 3.57)} \]

\[ \phi = 0.9 \]

\[ f_{ui} = \text{design tensile strength of the single plate} \]

\[ f_{uw} = \text{design tensile strength of the supported beam web} \]

- **Tear out**

The tear out calculation as presented in the Australian design method is similar to the AISC manual method. As such, the tearing calculation is calculated as the minimum tear out strength of either the supported test beam (defined as Equation 3.16) or single plate (defined as Equation 3.10). It should be noted that the tear out strength is calculated for both the vertical and horizontal directions using, \( L_c \), the smallest clear distance from the edge of the plate to the nearest edge of the bolt hole in the respective direction.

- **Great Britain (BCSA 2002)**

Differing from the AISC manual method for calculating bearing and tear out, Great Britain only checks for the bearing capacity of the single plate and supported beam. The following calculation is used:

\[ \phi P_{bs} = \min \left[ \phi \left( d_b t_p P_{bs,p} \right) n, \phi \left( d_b t_w P_{bs,b} \right) n \right] \quad \text{(EQ 3.58)} \]
where: \( \phi = 0.75 \)

\[
p_{bs,b} = \text{bearing strength of the supported beam web (defined as } 2.4F_u \text{ from Equation 3.15)}
\]

\[
p_{bs,p} = \text{bearing strength of the single plate (defined as } 2.4F_u \text{ from Equation 3.9)}
\]

\[
\phi P_{bs} = \text{bearing capacity of the single plate or beam web}
\]

-New Zealand (HERA (1999))-  

Similar to the Australian design method calculation for bearing and tear out, the New Zealand design method calculation for bearing and tear out takes into account vertical and horizontal tearing as well as bearing of the bolts as induced by the applied shear. Again, a bolt group interaction factor is used in place of the standard number of bolts multiplier. The New Zealand bearing and tear out calculation is as follows:

- Web plate strength limits

\[
\text{Bearing Design Strength Capacity} = \min \left[ \frac{\phi V_{bi}}{\phi V_{tti}} \right] \quad \text{(EQ 3.59)}
\]

where: \( \phi V_{bi} = \text{single plate bolt hole 1st resultant bearing design shear capacity, and is defined as follows:} \)

\[
\phi V_{bi} = \phi_s Z_b 3.2 t_p d_b f_{ui} \quad \text{(EQ 3.60)}
\]

\( \phi V_{tti} = \text{single plate bolt hole 1st transverse tearing design shear capacity, and is defined as follows with the clear distance, } L_c, \text{ being the smallest horizontal distance} \)
between the edge of the plate and the nearest edge of the bolt hole:

\[ \phi V_{li} = n_p \left( \phi_s 1.2 L_c t_p f_{ui} \right) \]  \hspace{1cm} (EQ 3.61)

\( \phi V_{li} \) = single plate bolt hole 1st longitudinal tearing design shear capacity, and is defined as follows with the clear distance, \( L_c \), being the smallest vertical distance between the edge of the plate and the nearest edge of the bolt hole:

\[ \phi V_{li} = n_p Z_b \left( \phi_s 1.2 L_c t_p f_{ui} \right) \]  \hspace{1cm} (EQ 3.62)

\( Z_b \) = bolt interaction factor for single line of bolts, \( n_p \neq 1 \),

(defined as Equation 3.36)

\( Z_e \) = bolt group flexure factor, single line of bolts, \( n_p \neq 1 \),

(defined as Equation 3.38)

\( \phi_s \) = 0.75

\( f_{ui} \) = single plate tensile strength

\( L_{eh, a_{eiy}} \) = bolt edge distances defined later

\( e \) = bolt line to weld line distance, \( a \)

---

**Block Shear**

-Great Britain (BCSA 2002)-

The Great Britain single plate design method for calculating block shear of the single plate is similar to the AISC design manual method. The only difference is that the diameter of the bolt hole is not increased by 1/16th of an inch. Otherwise, the block shear equations are identical to Equations 3.11 and 3.12.
Flexural Yielding
-Richard et al. (1980, 1984)-

For the flexural yielding design strength calculation, the primary difference in calculation is the means by which connection eccentricity is calculated as well as the incorporation of the plastic section modulus. The following equations show the means for calculating both the connection eccentricity and the flexural yielding strength.

- Using the beam $L/d$ ratio and $(e/h)_{ref}$. Compute $h$:

$$h = (n-1)p$$

(EQ 3.63)

Use the following design formula to calculate $e/h$ for the connection.

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

(EQ 3.64)

where: $e$ = eccentricity measured from the bolt line

$(e/h)_{ref}$ = 0.06($L/d$) - 0.15 when $L/d \geq 6$ (EQ 3.65)

= 0.035($L/d$) when $L/d < 6$ (EQ 3.66)

$N$ = 5 for ¾in and ⁷⁄₈in bolts

= 7 for 1in bolts

$S_{ref}$ = 100 for ¾in bolts

= 175 for ⁷⁄₈in bolts

= 450 for 1in bolts

$S$ = section modulus of the beam

With the ratio $e/h$ and $h$ known, compute the connection eccentricity, $e$. 
• Compute the moment at the weldment:

\[ M = V(e + a) \]  \hspace{1cm} \text{(EQ 3.67)}

where:  \( V \) = beam shear force

\( e \) = eccentricity from above step

• Check the plate normal and shear stresses:

Plate bending stress:

\[ f_b = \frac{M}{\frac{1}{4}t_p L_p^2} \]  \hspace{1cm} \text{(EQ 3.68)}

Plate shear stress:

\[ f_v = \frac{V}{L_p t_p} \]  \hspace{1cm} \text{(EQ 3.69)}

It should be noted that the eccentricity calculated from Equation 3.64 will always be positive. This eccentricity is measured from the bolt line. Therefore, the weld eccentricity will always be greater than the distance between the weld line and bolt line.

From Richard et al.'s (1984) research involving single plate framing connections with Grade 50 steel beams, the following revisions to the Richard et al.'s (1980) design procedure were presented.

• First, the eccentricity must be modified to account for the use of Grade 50 steel beams as opposed to A36

\[ e_{50} = e_{36} \left( \frac{36}{50} \right) \]  \hspace{1cm} \text{(EQ 3.70)}

where:  \( e_{36} \) = eccentricity as calculated in Equation 3.64
- Use the following design formula to calculate $e/h$ for the connection

$$
\frac{e}{h} = \left( \frac{e}{h} \right)_\text{ref} \cdot \left( \frac{n}{N} \right) \left( \frac{S_{\text{ref}}}{S_g} \right)^{0.4} \left( \frac{S_{\text{g}}}{S_{\text{gnp}}} \right)^{0.5} \frac{36}{F_{yg}}
$$

(EQ 3.71)

where: $\left( \frac{e}{h} \right)_\text{ref} = 0.06(L/d) - 0.15$  

(EQ 3.72)

$N$ = 5 for $\frac{3}{4}$in and $\frac{7}{8}$in bolts  

= 7 for 1in bolts

$S_{\text{ref}}$ = 100 for $\frac{3}{4}$in bolts  

= 175 for $\frac{7}{8}$in bolts  

= 450 for 1in bolts

$S_g$ = governing section modulus

$S_{\text{gnp}}$ = governing section modulus with no cover plates

$F_{yg}$ = governing minimum steel yield stress except for sections

where concrete stress governs ($F_{yg} = 36$ksi)

-Sarkar and Wallace (1992)-

Again, the recommended flexural yielding design strength calculation incorporates the plastic section modulus. In this design method, the eccentricity is limited to the bolt line to weld line distance, $a$. The following shows the calculation for the flexural yielding strength.

- At the weld line, calculate the moment, $M$

$$
M = Va
$$

(EQ 3.73)

where: $V$ = design reaction

- For the single plate, check combined shear and bending

Plate bending stress: $f_b = \frac{M}{Z} \leq \phi f_y$

(EQ 3.74)
Plate shear stress: \[ f_v = \frac{V}{A_g} \leq \phi \left(0.60 f_y\right) \] (EQ 3.75)

where: \[ \phi = 0.90 \]
\[ f_y = \text{single plate yield stress} \]

Plate plastic section modulus, \[ Z = \frac{t_p L_p^2}{4} \] (EQ 3.76)

Plate cross-sectional area, \[ A_g = L_p t_p \]

-Duggal and Wallace (1996)-

This design method differs from other design methods in that it was developed specifically for short-slotted holes and has detailed force calculations required to induce bolt slip. Again, the plastic section modulus is used in calculations and the eccentricity is limit to the bolt line to weld line distance, \( a \). The following presents the calculations for both flexural yielding design strength and the longitudinal force in the bolts due to the beam end rotation. Considerations for a connection with rigid or flexible support condition in combination with short-slotted holes are presented.

As stated in Duggal and Wallace (1996), Duggal and Wallace’s proposed design procedure for single plate framing connections with flexible support condition and short-slotted holes is as follows:

- Check if the bolt can develop a moment greater than the equilibrium moment without bolt sliding:

\[ F_k = 0.80(\mu T + \mu_a P) \] (EQ 3.77)

where: \( \mu \) = coefficient of friction
Design plate for stresses due to longitudinal bolt sliding force, shear and affiliated moment

\[ f_v = \frac{R}{t_p L_p} \leq 0.9(0.60)F_y \]  
\[ f_h = \frac{M}{\left[ \frac{1}{6}(t_p)^2 L_p^2 \right]} \leq (0.9)F_y \]

where:  
- \( M = Ra \)
- \( R = \) design reaction

As stated in Duggal and Wallace (1996), Duggal and Wallace’s proposed design procedure for single plate framing connections with rigid support condition and short-slotted holes is as follows:

- Find the longitudinal force in the bolts due to a 0.03 radian rotation at the design shear force, \( R \):

If tensioned (torqued) bolts are used, the longitudinal bolt force is found using the following procedure steps:

1) Find the perpendicular shear force in each bolt by assuming that the bolts share the design shear (beam end shear) equally
2) Calculate maximum bolt displacement for a rotational demand of 0.03 radians, assuming that the connection rotates about the centroid of the bolt group, and that the longitudinal displacements increase linearly (along the connection depth) with distance from the center of rotation.

3) Use the perpendicular load from (1) and displacement from (2) in the following equation to find longitudinal bolt force:

\[ F_L = \mu T + \mu_s P + KD_L \]  \hspace{1cm} (EQ 3.80)

where: \( \mu \) = coefficient of friction
\( T \) = bolt tension
\( \mu_s \) = coefficient of lateral swelling friction
\( P \) = perpendicular load applied
\( K \) = slot stiffness
\( D_L \) = longitudinal displacement along the slot length

4) If the resultant bolt force is not less than design single shear strength of bolt, add one bolt and repeat steps (1) to (4).

If untensioned (untorqued) bolts are used, the plateau load, \( H \), is found using the following equation:

\[ H = 0.103P + 0.416 \left( \frac{P^2}{t_p d_b F_y} \right) \]  \hspace{1cm} (EQ 3.81)

where: \( P \) = perpendicular load at which the plateau load is achieved
\( F_y \) = single plate yield stress
• Design plate for stresses due to longitudinal bolt sliding force, shear and affiliated moment:

\[
f_v = \frac{R}{L_p t_p} \leq \phi(0.60)F_y \quad \text{(EQ 3.82)}
\]

\[
f_b = \frac{M}{\left[\frac{1}{6}(t_p)(L_p)^2\right]} + \frac{F_L \text{ or } H}{(t_p)(0.5a + L_{cv})} \leq \phi F_y \quad \text{(EQ 3.83)}
\]

and

\[
f_b = \frac{M}{\left[\frac{1}{6}(t_p)(L_p)^2\right]} + \frac{F_L \text{ or } H}{(t_p)(p)} \leq \phi F_y \quad \text{(EQ 3.84)}
\]

where: \( \phi = 0.9 \)

\( M = Ra \)

\( F_L \) = maximum longitudinal bolt force

\( H \) = plateau load (used for untensioned bolts)

\( F_y \) = specified minimum yield strength of single plate

Note:

1) The stress, \( f_b \), is the algebraic sum of the flexure stress due to equilibrium moment, \( M \), and longitudinal bolt force, \( F_L \) (a compatibility force). Since the design of the plate is in the elastic range, the elastic section modulus \( (1/6t_pL_p^2) \) is used to resist the equilibrium moment. The distribution of the stress due to longitudinal bolt force is a step function over the depth of the plate, which is calculated by dividing the longitudinal bolt force by the corresponding tributary plate length. The most severe stress zone lies near the top bolt (assuming the loads are applied downward) between the centerline of the bolt, which is below the top bolt, and the plate top edge.
2) The plate is also not designed for the resultant of the stress $f_h$ and $f_v$ because, in the actual stress distribution, these stresses do not reach their maximum simultaneously (i.e. at the same point).

-Australia (OneSteel Market Mills (2000))- 

The Australian design method for calculating the flexural yielding strength of the single plate utilizes the plastic section modulus and incorporates various eccentricity assumptions. The following equation is specified for calculating the flexural yielding strength of the single plate.

\[ V_a = \frac{\phi M_{si}}{e} \]  

(EQ 3.85)

where:  
\( \phi = 0.9 \)  
\( M_{si} = \) nominal moment capacity of the single plate which is defined as follows:  
\[ M_{si} = 0.25 f_{yi} t_p L_p^2 \]  

(EQ 3.86)

Note: Moment-shear interaction is ignored.

\( e = \) design eccentricity of design force specified in the bearing and tear out portion of Section 3.4.5

-New Zealand (HERA (1999))- 

The New Zealand design method utilizes the plastic section modulus and utilizes the bolt line to weld line distance, \( a \), as the connection eccentricity for calculating the flexural yielding strength of the single plate. The following equation is specified:

\[ \phi V_{gl} = \phi_s \frac{t_p L_p^2}{4e} f_{yi} \]  

(EQ 3.87)

where:  
\( \phi_s = 0.9 \)
\[ e \quad = \text{connection eccentricity assumed to be the bolt line to weld line distance, } a \]

\[ f_{si} \quad = \text{single plate yield strength} \]

\[ \phi V_{gli} \quad = \text{single plate gross flexure yield design shear capacity} \]

**Combined Flexure and Shear**

-Duggal and Wallace (1996)-

Duggal and Wallace specified that combined shear and bending for the single plate should be checked. The following check is specified for this design strength calculation for single plate framing connection with rigid support condition.

Axial Load capacity \( \geq \) Design Axial Load

\[
(n)(0.80)(\mu T + \mu_{ls} P) \geq \text{Design Axial Load} \quad \text{(EQ 3.88)}
\]

where: \( \mu \) = coefficient of friction

\( T \) = bolt tension

\( \mu_{ls} \) = coefficient of lateral swelling friction

\( P \) = perpendicular load applied

-New Zealand (HERA (1999))- 

The New Zealand design method incorporates a design calculation for combined shear and flexure of the single plate. For low applied shear forces, only shear yielding of the single plate should be checked. However, for higher applied shear forces, the method specifies that combined shear and flexure should be calculated for. The following equations detail this design calculation:
-Shear yielding:

\[ \phi V_{gsi} = \phi_s \cdot 0.6 t_p L_p f_{yi} \quad \text{if} \quad V^* \leq 0.75 \phi V_{ggi} \]  
\[ \text{(EQ 3.89)} \]

-Shear yielding and combined flexure:

\[ \phi V_{gsi} = \phi_s \cdot 0.6 t_p L_p f_{yi} \left[ 2.2 - \left( \frac{1.6V^*}{\phi V_{ggi}} \right) \right] \quad \text{if} \quad V^* > 0.75 \phi V_{ggi} \]  
\[ \text{(EQ 3.90)} \]

where:
- \( \phi_s = 0.90 \)
- \( f_{yi} \) = single plate yield strength
- \( V^* \) = design shear force
- \( \phi V_{ggi} \) = single plate gross flexure yield design shear capacity, (defined as Equation 3.87)
- \( \phi V_{gsi} \) = single plate gross yield design capacity

-Great Britain (BCSA 2002)-

The Great Britain single plate design method incorporates a combined shear and bending moment calculation into the design of the single plate. The formulation varies per amount of applied shear at the connection. At lower shears, elastic plate behavior assumptions are applicable, while at higher shears, plastic behavior is assumed.

-For shear and bending interaction:

\[ Qa \leq \phi M_c \]  
\[ \text{(EQ 3.91)} \]
-For low shear (i.e. $Q \leq 0.75P_v$):

$$\phi M_c = \phi p_y \frac{t_p L_p^2}{6}$$  \hspace{1cm} (EQ 3.92)

-For high shear (i.e. $Q > 0.75P_v$):

$$\phi M_c = \phi p_y \frac{t_p L_p^2}{4} \left(1 - \left(\frac{Q}{P_v}\right)^2\right)^{\frac{1}{2}}$$  \hspace{1cm} (EQ 3.93)

where:  
\begin{align*}
\phi &= 0.90 \\
p_y &= \text{strength of the single plate} \\
Q &= \text{shear reaction at the weld line} \\
P_v &= \text{shear capacity of the single plate, which is the lesser of the block shear, shear rupture and shear yielding capacity of the single plate}
\end{align*}

*Flexural Rupture*

-New Zealand (HERA (1999))-

The calculation for the flexural rupture strength is not performed by the AISC manual method for single plate framing connection design. The New Zealand design method was found to be the only design method to incorporate this design strength check.

$$\phi V_{nij} = \phi_s \left( \frac{f_w s_{net}}{e} \right)$$  \hspace{1cm} (EQ 3.94)

where:  
\begin{align*}
\phi_s &= 0.75
\end{align*}
\[ e = \text{connection eccentricity assumed to be the bolt line to weld line distance, } a \]

\[ S_{net} = \text{net section modulus of the single plate} \]

\[ \phi V_{nfl} = \text{single plate net ultimate design shear capacity} \]

**Additional Single Plate Design Checks**

- **Great Britain (BCSA 2002)**-

Great Britain also incorporates a design check to verify that “long” single plates will not buckle. This check is required when the supported beam is laterally restrained at its connection to the single plate. In addition, long single plates should not be used with unrestrained beams without experimental evidence to justify the design. In addition, the single plate is designed to be adequate for tensile or tie forces. This structural integrity check is required for tie forces greater than 75kN (16.86kips). The following design calculations were presented in the Great Britain design method:

- Lateral torsional buckling resistance of long single plates \( i.e. \ \text{where: } a > \frac{t_p}{0.15} \):

- Must satisfy the following requirements:

\[
\text{Req}(1): \quad Qa \leq \frac{M_b}{m_{LT}} \quad \text{(EQ 3.95)}
\]

\[
\text{Req}(2): \quad Qa \leq p_y Z_x \quad \text{(EQ 3.96)}
\]

where: \( m_{LT} = 0.6 \) (from BS5950-1, Table 18)

\( p_b = \text{bending strength of the single plate} \)
\( p_y \) = strength of the single plate

\( M_b \) = lateral torsional buckling resistance moment in the single plate and is defined as follows:

\[
M_b = p_y Z_x \tag{EQ 3.97}
\]

\( Z_x \) = elastic modulus of the single plate about the major axis

and is defined as follows:

\[
Z_x = \frac{t_p L_p^2}{6} \tag{EQ 3.98}
\]

- Structural integrity of single plate

![Diagram of structural integrity of single plate](image)

**FIGURE 3.1 GREAT BRITAIN NET SECTION OF SINGLE PLATE**

-For tension:

Tie force \( \leq \) Tension capacity of single plate

-For bearing:

Tie force \( \leq \) Bearing capacity of single plate
3.4.6. SUPPORTED TEST BEAM

Each design method incorporated various design checks for the supported beams. Presented with each design method are the design checks which were specified in the respective design methods.

-Australia (OneSteel Market Mills (2000))-

-For the coped beam web:

\[
    e = \text{Max} \left( \frac{e_{\text{bolts}}}{1 + \eta} + a_{e1} - L_c \right) \\
    \text{EQ 3.99}
\]

where:  
\( e \) = design eccentricity of design shear force \\
\( e_{\text{bolts}} \) = eccentricity of design shear force measured from the bolt group centroid for a connection attached to a stiff support \\
\( a_{e1} \) = horizontal distance between the end of the beam and column of bolts \\
\( L_c \) = horizontal length of the beam cope \\
\( \eta \) = the plasticity factor which is defined as follows:

\[
    \eta = \left( \frac{\theta_b}{\theta_{eb}} - 1 \right) \alpha^{a-1} \\
    \text{EQ 3.100}
\]

where:  
\( \frac{\theta_b}{\theta_{eb}} = 1.24 \) and \( n = 40 \) (for a steel beam) \\
\( \alpha = \frac{M}{M_b} \)

\[
    \bar{M} = \frac{M^*}{\phi}
\]
The New Zealand design method specifies all the required design checks for the supported beam. Within this section, the beam will be evaluated in terms of shear yielding, block shear, shear rupture, and bearing and tear out. Unless otherwise specified the design method is calculated using the same design checks and specifications as the AISC manual. However, for the bearing and tear out calculations, the test beam is calculated similar to the single plate bearing and tear out capacity, which was described in Equations 3.60 to 3.62. As necessary, please refer to the Bearing and Tear Out section of Section 3.4.5.

*No copes:

\[
\phi_{V_{wb}} = \text{Min} \left[ \phi_{V_{bw}}; \phi_{V_{tw}}; \phi_{gV_{t}} \right] \quad \text{(EQ 3.101)}
\]

where:

\( \phi_{V_{wb}} \) = web design shear capacity

\( \phi_{V_{bw}} \) = web bolt hole 1st resultant bearing design shear capacity

\( \phi_{V_{tw}} \) = web bolt hole 1st transverse tearing design shear capacity

\( \phi_{V_{btw}} \) = web bolt hole 1st longitudinal tearing design shear capacity

\( \phi_{gV_{tw}} \) = web gross shear yield design capacity

**Single web cope:

\[
\phi_{V_{wb}} = \text{Min} \left[ \phi_{V_{bw}}; \phi_{V_{tw}}; \phi_{V_{btw}}; \phi_{gV_{tw}}; \phi_{V_{btw}}; \phi_{V_{bw}} \right] \quad \text{(EQ 3.102)}
\]
where: \( \phi_{wb} \) = web design shear capacity
\( \phi_{bw} \) = web bolt hole 1st resultant bearing design shear capacity
\( \phi_{bw} \) = web bolt hole 1st transverse tearing design shear capacity
\( \phi_{bw} \) = web bolt hole 1st longitudinal tearing design shear capacity
\( \phi_{gfw} \) = web gross flexure yield design shear capacity
\( \phi_{gs} \) = web gross shear yield design capacity
\( \phi_{nsw} \) = web net ultimate design shear capacity
\( \phi_{bsw} \) = web block shear design capacity

- **Great Britain (BCSA 2002)**-

The following are the design calculations for the supported beam specified in the Great Britain design manual for single plate framing connections.

- Check strength of net section of supported beam

- For shear (Basic Requirement):

\[
Q \leq P_v
\]  

(EQ 3.103)

where: \( Q \) = shear reaction at the weld line
\( P_v \) = shear capacity of the beam which is defined as follows:

\[
P_v = \min \left( P_{vp}, P_{vsB} \right)
\]

plain shear capacity
block shear capacity

(EQ 3.104)
Shear and bending interaction of the beam web for long single plates

\[ i.e. \text{where: } a > \frac{t_p}{0.15} \] :

Note: This check is required to ensure that the crosshatched area ABCD shown in Figure 3.2 can resist a moment \( Qa \).

FIGURE 3.2 GREAT BRITAIN NET SECTION OF SINGLE PLATE

-Must satisfy the following requirements:

\[ Qa \leq M_{eBC} + P_{wAB}(n-1)p \]  

\( (EQ \ 3.105) \)

-For low shear (i.e. \( Q_{BC} \leq 0.75P_{wBC} \)):

\[ M_{eBC} = \frac{p_y t_w}{6}((n-1)p)^2 \]  

\( (EQ \ 3.106) \)
-For high shear (i.e. $Q_{BC} > 0.75P_{vBC}$):

\[
M_{eBC} = \frac{P_v t_w}{4} \left( (n-1)p \right)^2 \left( 1 - \left( \frac{Q_{BC}}{P_{vBC}} \right)^2 \right)^{\frac{1}{2}} \tag{EQ 3.107}
\]

where:  

\begin{align*}
M_{eBC} & = \text{moment capacity of the beam web BC} \\
P_v & = \text{shear capacity of the beam (defined as Equation 3.108)} \\
P_{vAB} & = \text{shear capacity of the beam web AB and is defined as follows:}
\end{align*}

\[
P_{vAB} = \min \left[ 0.9 \left( 0.6 p_y e_y t_w \right), \right. \\
0.75 \left( 0.6 p_u \left( e_y - \frac{d_y}{2} \right) t_w \right) \left. \right] \tag{EQ 3.108}
\]

\[
P_{vBC} = \text{shear capacity of the beam web BC and is defined as follows:}
\begin{align*}
P_{vBC} & = \min \left[ 0.9 \left( 0.6 p_y (n-1)p t_w \right), \\
0.75 \left( 0.6 p_u ((n-1)(p-d_y)) t_w \right) \right] \tag{EQ 3.109}
\end{align*}

\[
Q_{BC} = \text{shear force on the beam web BC and is defined as follows:}
\begin{align*}
Q_{BC} & = Q - (P_v - P_{vBC}) \text{ but } \geq 0 \tag{EQ 3.110}
\end{align*}

\begin{align*}
p_y & = \text{yield strength of the single plate} \\
p_u & = \text{ultimate strength of the single plate}
\end{align*}
Shear and bending interaction at the notch of supported beam

**FIGURE 3.3 GREAT BRITAIN CAPACITY OF COPED SUPPORTED BEAM**

-For single line of bolts and a beam with a single cope:

\[ Q(t_1 + L_c) \leq \phi M_{cN} \]  

(EQ 3.111)

where: For low shear (i.e. \( Q \leq 0.75P_{vN} \)),

\[ \phi M_{cN} = \phi p_y Z_N \]  

(EQ 3.112)

For high shear (i.e. \( Q > 0.75P_{vN} \)),

\[ \phi M_{cN} = \phi 1.5 p_y Z_N \left(1 - \left(\frac{Q}{P_{vN}}\right)^2\right)^{\frac{1}{2}} \]  

(EQ 3.113)

\[ \phi = 0.9 \]

\[ p_y = \text{strength of the supported beam} \]

\[ t_1 = \text{end projection} \]

\[ A_{vN} = (e_t + (n-1)p + e_b) \gamma_w \]  

(EQ 3.114)
\( L_c \) = length of the beam cope

\( M_{cN} \) = moment capacity of the beam at the notch in the presence of shear

\( P_{vN} \) = shear capacity at the notch which is defined as follows:

\[
P_{vN} = 0.6\rho Av_N
\]  \hspace{1cm} (EQ 3.115)

\( Z_n \) = elastic section modulus of the gross tee section at the notch

- Coped Beams – Local stability of notched beams restrained against lateral torsional buckling

\[
d_c \leq \frac{d}{2} \hspace{1cm} \text{(EQ 3.116)}
\]

\[
L_c \leq d \quad \text{for } d/t_w \leq 54.3 \text{ (S275 Grade 43 steel)} \hspace{1cm} \text{(EQ 3.117)}
\]
\[ L_c \leq \frac{160000d}{\left( \frac{d}{t_w} \right)^3} \quad \text{for } d/t_w > 54.3 \text{ (S275 Grade 43 steel)} \quad \text{(EQ 3.118)} \]

\[ L_c \leq d \quad \text{for } d/t_w \leq 48.0 \text{ (S355 Grade 50 steel)} \quad \text{(EQ 3.119)} \]

\[ L_c \leq \frac{110000d}{\left( \frac{d}{t_w} \right)^3} \quad \text{for } d/t_w > 48.0 \text{ (S355 Grade 50 steel)} \quad \text{(EQ 3.120)} \]

- Local shear and punching capacity of supporting beam web

\[ \frac{Q}{2} \leq \phi P_v \quad \text{(EQ 3.121)} \]

where: The local shear capacity of supporting beam web, \( P_v \) is defined as follows:

\[ \phi P_v = 0.9 \left( 0.6 p_y A_v \right) \quad \text{(EQ 3.122)} \]
-Punching shear capacity:

Note: Provided that either the conservative or rigorous requirement given below is satisfied, yielding of the single plate occurs before punching shear failure of the supporting member:

-Conservative:

\[ t_p \leq t_w \left( \frac{U_{SC}}{P_{sf}} \right) \]  

(EQ 3.123)

-Rigorous:

\[ t_p \leq t_w \left( \frac{U_{SC}}{f_b} \right) \]  

(EQ 3.124)

where:  
\( f_b \) = maximum bending stress in the single plate which is defined as follows:

\[ f_b = \frac{Qa}{Z_{\text{gross}}} \text{ but } f_b \leq 0.9 p_{sf} \]  

(EQ 3.125)

\( p_y \) = strength of the single plate

\( p_{sf} \) = design strength of the single plate

\( A_v \) = \( L_p t_w \)  

(EQ 3.126)

\( U_{SC} \) = Ultimate tensile strength of supporting member

\( Z_{\text{gross}} \) = elastic modulus of gross section of the single plate and is defined as follows:

\[ = \frac{t_p L_p^2}{6} \]  

(EQ 3.127)
**Additional Supported Beam Design Checks**

-Australia (OneSteel Market Mills (2000))-  

Within the Australian single plate design method, considerations are made such that contact does not occur between the bottom flange of the test beam and the support. In addition, a design calculation is suggested to ensure adequate ductility for a rigid support condition. The following design calculations are specified:

- To provide rotational clearance.

\[
\bar{\theta} \leq \frac{a - a_{el}}{a_c + 0.5L_p} \quad (\text{EQ 3.128})
\]

where:
- \(a_{el}\) = horizontal distance between the end of the beam and column of bolts
- \(a_c\) = distance between the base of the single plate and the bottom of the supported beam
- \(\bar{\theta}\) = end rotation of the supported test beam where for a prismatic beam, \(\bar{\theta} = \theta_b = 0.95 \left( \frac{f_{yf}}{E} \right) \left( \frac{L}{d} \right) \) \quad (EQ 3.129)

- \(\theta_b\) = total beam end rotation at plastic hinge condition
- \(f_{yf}\) = design yield stress of the steel beam flange

- To ensure adequate ductility for a single plate connection with rigid support condition.

\[
\theta_c \geq \bar{\theta} \quad \text{(EQ 3.130)}
\]

where:
- \(\bar{\theta}\) = end rotation under overload conditions, or the rotation of the beam end relative to the connection support
\[ \theta_c \quad = \quad \text{design rotation capacity of the connection (Table 3.3)} \]

**-New Zealand (HERA (1999))-**

The New Zealand single plate design method also incorporates considerations such that contact does not occur between the bottom flange of the test beam and the supporting member. The following design calculation is specified:

- Beam/column seismic end gap:

\[ \frac{a_c}{a - a_{el}} \leq 33 \quad \text{(EQ 3.131)} \]

- Beam/beam end gap:

\[ \frac{a_c}{a - a_{el}} \leq 50 \quad \text{(EQ 3.132)} \]

where: \[ a_c \quad = \quad \text{end gap} \]

\[ a_{el} \quad = \quad \text{distance between the bolt line to the end of the beam} \]

**-Great Britain (BCSA 2002)-**

Great Britain also incorporates a design check to verify that the supported test beam is designed to be adequate for tensile or tie forces. This structural integrity check is required for tie forces greater than 75kN (16.86kips). The following design calculations were presented in the Great Britain design method:
• Structural integrity – tension capacity of beam web

![Diagram of beam web and tie force](image)

**FIGURE 3.6 GREAT BRITAIN TENSION CAPACITY OF THE BEAM WEB**

-For tension:

\[
\text{Tie force} \leq \text{Tension capacity of beam web}
\]

-For bearing:

\[
\text{Tie force} \leq \text{Bearing capacity of the beam web}
\]

3.4.7. WELDMENT

-Ashakul (2004)-

Although Ashakul did not specifically propose any design procedure for calculating the weld strength, Ashakul recommended that connections satisfying the following plate thickness consideration be designed to sustain the shear force sustained by the bolt group at an eccentricity equal to the distance between the bolt line and the weld line

\[
t_p \leq \frac{d_k}{2} \left( \frac{36}{F_y} \right)
\]

(EQ 3.133)
Ashakul recommends that connections with a plate thickness which exceeds the value of Equation 3.137 be designed to sustain the vertical shear force at the weld line to bolt line distance, $V_a$, and the horizontal force, $M_b$, which is defined in Figure 3.7.

![Figure 3.7 ASHAKUL HORIZONTAL FORCES ON BOLTS (TOP HALF OF BOLT GROUP SHOWN)](image)

- Astaneh et al. (1993)-

Welds are designed for the combined effects of direct shear ($R_o$) and moment ($R_o e_w$) due to moment created from the eccentricity, $e_w$, which is measured from the weld line to the effective pin or inflection point of the moment diagram. This assures that the plate yields before the welds fail. As reported by Astaneh et al. (1993), the following equations specify the eccentricity of the weldment for various support conditions:

- For a rigid support condition:

$$
 e_w = (n-1)
$$

(EQ 3.134)

- For a flexible support condition:

$$
 e_w = \max \left\{ \frac{n}{0.0} \right\}
$$

(EQ 3.135)
From Astaneh et al.s (1990) research, additional provisions to the original design method were suggested. These provisions are as follows:

- Provision to ensure ductile behavior

The following provision ensures that the plate will yield prior to the weld yielding, which eliminates a brittle failure via weld fracture:

\[
\left( \frac{V}{V_y} \right)^2 + \left( \frac{M}{M_p} \right)^2 = 1.0
\]

(EQ 3.136)

Substituting terms by defining the capacity of the welds for the capacity of the section, Equation 3.140 becomes:

\[
\left( \frac{V}{V_{yw}} \right)^2 + \left( \frac{M}{M_{pw}} \right)^2 = 1.0
\]

(EQ 3.137)

where:

\[
V_{yw} = 2 \left( \frac{1}{\sqrt{3}} 0.5F_{Exx} \right) \left( 0.707t_wL_w \right)
\]

(EQ 3.138)

\[
M_{yw} = 2 \left( 0.5F_{Exx} \right) \left( 0.707t_wL_w^2 \right)
\]

(EQ 3.139)

-Richard et al. (1980)-

Richard specifies that the welds should be designed for the combined effects of direct shear \(R_o\) and moment \(R_o e_w\) due to moment created. Based upon the resultant of the normal and shear stresses from the design of the single plate, design the weldment:

\[
f_r = \left( f_h^2 + f_v^2 \right)^{0.5}
\]

(EQ 3.140)
-Duggal and Wallace (1996)-

Similar to Richard’s consideration for the weldment, Duggal and Wallace specify that the weldment must be designed for the combined effects of shear and moment. Also specified is the design consideration for initially determining the thickness of the weld.

\[
f = \left( f_v^2 + f_b^2 \right)^{0.5}
\]

(EQ 3.141)

\[
\frac{\text{Weld Size}}{16} = \frac{(f)(t_p)(16)}{[0.60][70][0.707][2]}
\]

(EQ 3.142)

-New Zealand (HERA (1999))-\n
In the New Zealand design method, the weld strength is calculated with respect to ultimate tensile strength while allowing full tensile development of the single plate. The following design considerations must be satisfied:

\[
N^*_{gti} \leq \phi N_{ww}
\]

(EQ 3.143)

where:  \( N^*_{gti} \) = cleat gross tension, and is defined as:

\[
N^*_{gti} = \phi_s L \phi_p f_{yi}
\]

(EQ 3.144)

\( \phi N_{ww} \) = weld fillet weld tension design capacity, as calculated using Equation 3.22a

\( \phi_s = 0.90 \)

\( f_{yi} \) = single plate yield strength
3.5. SUMMARY

As described within the literature review of Chapter 2, multiple countries including Australia, Canada, Great Britain, New Zealand, and the United States have specified design methods for single plate framing connections. In addition, previous researchers within the United States have also recommended design procedures in an effort to improve the nationally specified method. This chapter sought to compare each method and procedure in an attempt to identify primary differences.

At the beginning of this chapter, the AISC LRFD 3rd Edition Manual (AISC 2003) design method for single plate framing connections was presented as baseline for comparison. From the conducted comparison work, multiple differences in comparison to the AISC manual method for single plate framing connections were identified. From these differences, new considerations and design calculations and checks were identified. Summarized in Table 3.4 are the primary differences associated with the design of the connection bolts and single plate.
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<thead>
<tr>
<th>Component</th>
<th>Strength Checks</th>
<th>Design Method Recommended by:</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bolts</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength</td>
<td>Astaneh specifies equations for bolt eccentricity based upon hole type and support condition. The equations are a function of the number of bolts in the connection and the weld line to bolt line distance, ( e )</td>
<td>Astaneh specifies equations for bolt eccentricity based upon hole type and support condition. The equations are a function of the number of bolts in the connection and the weld line to bolt line distance, ( e ). Different bolt group shear strength equations incorporate various strength reduction factors.</td>
</tr>
<tr>
<td>Eccentricity, ( e_b )</td>
<td>( e_b )</td>
<td>( e_b )</td>
</tr>
<tr>
<td><strong>Single Plate</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Yielding</td>
<td>Utilizes a reduced cross sectional area</td>
<td>Utilizes a reduced cross sectional area</td>
</tr>
<tr>
<td>Shear Rupture</td>
<td>Differs in that the bolt hole diameter is not increased by ( 1/16 ) of an inch.</td>
<td>Differs in that the bolt hole diameter is not increased by ( 1/16 ) of an inch.</td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>( \cdot )</td>
<td>( \cdot )</td>
</tr>
</tbody>
</table>

Note: This table is continued on the following page.
### TABLE 3.4 (CONTINUED)

<table>
<thead>
<tr>
<th>Component Strength Checks</th>
<th>Design Method Recommended by:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Astaneh</td>
</tr>
<tr>
<td>Single Plate Corrosion</td>
<td>---</td>
</tr>
<tr>
<td>Block Shear</td>
<td>---</td>
</tr>
<tr>
<td>Flexural Yielding</td>
<td>---</td>
</tr>
<tr>
<td>Flexural Rupture</td>
<td>---</td>
</tr>
<tr>
<td>Weld</td>
<td>---</td>
</tr>
<tr>
<td>Weld Strength (Rigid)</td>
<td>0</td>
</tr>
<tr>
<td>Eccentricity, e&lt;sub&gt;n&lt;/sub&gt;</td>
<td>Astaneh specifies equations for weld eccentricity based upon number of bolts in the connection and support condition</td>
</tr>
<tr>
<td>Combined Flexure &amp; Shear</td>
<td>0</td>
</tr>
</tbody>
</table>

**Notes:**
1. This method consists of recommendations to the current AISC 2003 method
2. Weld Strength is calculated with respect to ultimate tensile strength while allowing full tensile development of the single plate.

**Table Key:**
- ● Indicates an actual strength calculation
- ○ Although not explicitly calculated, this consideration is satisfied by the dimensional requirement
- --- Indicates that this was not a consideration in the design methodology

**Dependent upon magnitude of applied shear:**
- At low shear, calculation is identical to AISC manual. At high shear, the plastic section modulus is utilized and the capacity is reduced by a strength ratio.
Based upon these findings, the necessity for additional research through experimental testing is called for. With experimentally measured test values from single plate connection tests, an analytical investigation using the various design methods could be conducted. The goal of the analytical investigation would be to determine which design check of the individual design methods most accurately predicted the observed test measurements.
CHAPTER 4 EXPERIMENTAL TESTING

4.1. OVERVIEW

To establish a baseline for comparing to previous design methods as presented in Chapter 3 and correlating to the currently accepted method presented in the AISC LRFD 3rd Edition Manual (AISC 2003), a total of ten full-scale single plate connection tests were conducted. Full-scale test specimens consisted of configurations that would commonly be found in the commercial building industry. A test setup was developed to achieve the following objectives: (1) to accurately model field conditions by combining both shear and rotation to the single plate connection and (2) to impose a total beam end rotation of 0.03 radians at ultimate loading. The test setup consisted of a simply supported configuration incorporating a single plate connection at one end and a roller support at the opposite end. Parameters incorporated both rigid (beam-to-column flange) and flexible (beam-to-girder web) support conditions.

In an attempt to simulate the effects of a concrete slab superstructure (such as typical concrete flooring in building construction), a simulated slab restraint was incorporated into each size connection tested that utilized a flexible support condition. The simulated slab restraint consisted of a tie plate (or rectangular piece of steel) which bridged the supporting girder and supported beam at the connection. The tie plate restricted the rotation between the top flanges of the supporting girder and supported test beam thus simulating a slab restraint. In addition to the simulated slab restraint which restrained the top girder flange at the connection against rotation, a pair of Channel shapes was used to restrain the top flange at two additional points along the supporting girder. Illustrations
and further descriptions of these testing configurations can be found in Sections 4.2.2 and 4.2.3 Specimen Fabrication and Test Preparation.

All tests utilized 3/4in ASTM A325 strength bolts to facilitate single plate connections. To simulate worst case field conditions, threads of the bolts were included in the shear plane. In addition, connection bolts were tightened to a snug-tight level. To determine material strength of components used in testing, tensile tests were conducted on material cut from extra portions of test beams, girders, and single plate stock. In addition, tensile tests and single and double shear tests were conducted on single bolts to measure the corresponding bolt stresses.

Single plate connections used in testing were designed to satisfy requirements listed in the AISC LRFD 3rd Edition Manual (AISC 2003). Figure 4.1 details a typical single plate configuration used with a rigid support condition, and Figure 4.2 details a typical single plate configuration used with flexible supported tests. The variable for distinction here is the number of bolts used in testing which effectively dictates the necessary plate length. Although Figures 4.1 and 4.2 depict a seven bolt test setup, plate dimensions such as the distance from the top of the girder to the top of the single plate, the bolt line to weld line distance \(a\), and the clear edge distances of the single plate \(L_{eb}, L_{ev}\) remained the same whether used in a two, three, or seven bolt test. Single plates were manufactured with either standard 13/16in diameter holes or horizontal short-slotted 13/16in x 1in holes. Both types of holes were oversized 1/16in to accommodate the 3/4in diameter connection bolts.
Test beams incorporated off-axis bolt groups. This terminology refers to connections where the neutral axis of the bolt group does not coincide with the major neutral axis of the supported beam. Alternatively stated, the neutral axis of the bolt group is closer to one flange of the supported test beam. Detailed drawings illustrating the connection end of beams utilized in either rigid or flexible supported tests can be seen in...
Figures 4.3 to 4.5. These figures call out off-axis bolt groups as used in testing and include various dimensions such as the beam cope length and depth as utilized in flexible supported tests. All test beams were fabricated with standard 13/16in diameter holes, which were oversized 1/16in to accommodate the ¾in diameter connection bolts. In addition, all beams utilized web stiffeners at the load points, midspan, and for the seven bolt tests at the end support to alleviate web local buckling. Web stiffeners consisted of the following configurations: for two and three bolt test beams, ⅜in x 3in full depth stiffeners with ¾in x ¾in clipped inside corners; for seven bolt test beams, ⅝in x 4in full depth stiffeners with 1in x 1in clipped inside corners.

FIGURE 4.3 TWO BOLT TEST BEAM
FIGURE 4.4 THREE BOLT TEST BEAMS

FIGURE 4.5 SEVEN BOLT TEST BEAMS
The test matrix presented in Table 4.1 details the various configurations used in testing and geometric properties such as the beam $L/d$ ratio and single plate dimensions. The test connection naming convention is a combination of the principal investigator (S for tests conducted in this series of research by Sumner), test number, support condition (F for flexible, R for rigid), hole type (SS for short-slotted, ST for standard), number of $\frac{3}{4}$in bolts in a single column, bolt classification, and SR if a simulated slab restraint was incorporated in the testing. A test designation of S8-FSS-2-A325-N-SR indicates the eighth test conducted by Sumner. The designated test incorporated a flexible support condition, a single plate with short-slotted holes, two $\frac{3}{4}$in diameter bolts ASTM designated A325 strength with threads included in the shear plane. In this particular test, a simulated slab restraint was used. The details of the experimental test program are presented in the subsequent sections. Details of the test specimens, test setups, and test procedure are included.
### TABLE 4.1 TEST MATRIX

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Support Condition</th>
<th>Hole Type</th>
<th>Simulated Slab Restraint</th>
<th>No. Bolts</th>
<th>Beam Size (Grade)</th>
<th>Beam (L/d) Ratio</th>
<th>Single Plate Size (Grade)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N 2</td>
<td>Rigid</td>
<td>Short-slotted</td>
<td>No</td>
<td>3</td>
<td>W16x50 (A992)</td>
<td>16.6</td>
<td>PL 3/8” x 4-1/2” x 0'-9&quot; (A36)</td>
</tr>
<tr>
<td>S2-RST-3-A325-N 2</td>
<td>Rigid</td>
<td>Standard</td>
<td>No</td>
<td>3</td>
<td>W16x50 (A992)</td>
<td>16.6</td>
<td>PL 3/8” x 4-1/2” x 0'-9&quot; (A36)</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N 3</td>
<td>Flexible</td>
<td>Short-slotted</td>
<td>No</td>
<td>3</td>
<td>W16x50 (A992)</td>
<td>16.7</td>
<td>PL 3/8” x 4-1/2” x 0'-9&quot; (A36)</td>
</tr>
<tr>
<td>S4-FST-3-A325-N 3</td>
<td>Flexible</td>
<td>Standard</td>
<td>No</td>
<td>3</td>
<td>W16x50 (A992)</td>
<td>16.7</td>
<td>PL 3/8” x 4-1/2” x 0'-9&quot; (A36)</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR 3</td>
<td>Flexible</td>
<td>Short-slotted</td>
<td>Yes</td>
<td>3</td>
<td>W16x50 (A992)</td>
<td>16.7</td>
<td>PL 3/8” x 4-1/2” x 0'-9&quot; (A36)</td>
</tr>
<tr>
<td>S6-FST-2-A325-N 3</td>
<td>Flexible</td>
<td>Standard</td>
<td>No</td>
<td>2</td>
<td>W16x50 (A992)</td>
<td>16.7</td>
<td>PL 3/8” x 4-1/2” x 0'-6&quot; (A36)</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N 3</td>
<td>Flexible</td>
<td>Short-slotted</td>
<td>No</td>
<td>2</td>
<td>W16x50 (A992)</td>
<td>16.6</td>
<td>PL 3/8” x 4-1/2” x 0'-6&quot; (A36)</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR 3</td>
<td>Flexible</td>
<td>Short-slotted</td>
<td>Yes</td>
<td>2</td>
<td>W16x50 (A992)</td>
<td>16.6</td>
<td>PL 3/8” x 4-1/2” x 0'-6&quot; (A36)</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N 4</td>
<td>Rigid</td>
<td>Short-slotted</td>
<td>No</td>
<td>7</td>
<td>W27x84 (A992)</td>
<td>10.9</td>
<td>PL 3/8” x 4-1/2” x 1’-9&quot; (A36)</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR 5</td>
<td>Flexible</td>
<td>Short-slotted</td>
<td>Yes</td>
<td>7</td>
<td>W27x84 (A992)</td>
<td>11.2</td>
<td>PL 3/8” x 4-1/2” x 1’-9&quot; (A36)</td>
</tr>
</tbody>
</table>

**Notes:**
1. 3/4in diameter A325-N strength bolts used in all connections.
2. W16x50 beam and W14x145 column used for 2 and 3-bolt rigid support tests.
3. W16x50 beam, W18x50 girder, and two W14x90 columns used for 2 and 3-bolt flexible support tests.
4. W27x84 beam and W14x145 column used for 7-bolt rigid support test.
5. W27x84 beam, W30x99 girder, and two W14x90 columns used for 7-bolt flexible support test.
6. Bracing Channels were used to brace the top flange of the supporting girder.
4.2. TEST SPECIMENS

4.2.1. SPECIMEN DESIGN

In an attempt to create a test setup which accurately modeled field conditions, the following components were desired to be incorporated into the test setup: A325-N strength bolts, single plates manufactured from A36 steel, and test beams manufactured from A992 Grade 50 steel. In addition, the test setup needed to incorporate both rigid and flexible support conditions with standard or short-slotted holes in the single plate and standard holes in the test beam. Calculations were made based upon established theory for beam end rotation and strength limit states to ensure the decided upon test setup would be adequate. The following sections describe the calculations which were made.

**Beam End Rotations**

One objective for the decided upon test setup was to combine both shear and rotation to the single plate connection while imposing a total beam end rotation of 0.03 radians at ultimate loading. This rotation is the assumed maximum beam end rotation for framing connections. Larger rotations are thought to impose excessive rotation upon connection bolts effectively limiting the strength of the connection. Utilizing the reference text *Steel Structures* by Salmon and Johnson (1997) in addition with AISC LRFD 3rd Edition Manual (AISC 2003), calculations were made for proposed test beam sizes to predict beam end rotations at various applied connection shear forces. Appendix A details the beam end rotation calculations.

**Strength Limit States**

To predict the ultimate strength of the connection and ensure an adequate test setup, individual strength limit states for the supported beam, connection bolts, single plate,
weld, and supporting girder (flexible supported tests only) were calculated for using established equations presented in the AISC LRFD 3rd Edition Manual (AISC 2003). The strength limit states within the LRFD path of the connection are as follows:

Supported Beam:
- Flexure (LRFD Section B5)
- (LRFD Appendix F1)

Supported Beam Web:
- Web Local Yielding (LRFD Section K1.3)
- Web Crippling (LRFD Section K1.4)
- Shear Strength (LRFD Section F2)
- Bearing/Tear out (LRFD Section J3 subsection 10)
- Block Shear (coped beam only) (LRFD Section J4.3)

Bolts:
- Bolt Shear Rupture (LRFD Section J3 subsection 6)

Single plate:
- Shear Yielding (LRFD Section J5.3)
- Shear Rupture (LRFD Section J4.1)
- Flexural Yielding (LRFD Part 9 Design of Connection Elements)
- Bearing/Tear out (LRFD Section J3 subsection 10)
- Block Shear (LRFD Section J4.3)

Weld:
- Weld Rupture Due to Eccentric Shear (LRFD Section J2)
- Base Metal Failure at Weld (LRFD Section J2)

Supporting Girder (Flexible Tests):
- Flexure (LRFD Section B5)
- (LRFD Appendix F1)

4.2.2. SPECIMEN FABRICATION

Materials and test specimens were fabricated by W&W Steel, Inc. and Carolina Steel, Inc. Specimens were fabricated in accordance with fabrication drawings created using the above mentioned design method. Originally fabrication had single plates welded to both faces of the girder web. However, the single plates were welded at the
incorrect height on the faces of the girder web. The single plates were removed from both sides of the girder. The North Carolina State University machine shop was commissioned to weld a replacement single plate cut from the original plate stock to the girder web. The design was modified slightly to use only one single plate per girder. Because of an insufficient number of coped beams, extra beams that would have been used in rigid tests were coped on site to specifications.

Simulated slab restraints used in connection tests with a flexible support condition consisted of a single tie plate or rectangular piece of steel. Figure 4.6 is a photograph of Test 5, which incorporated a flexible support condition with a simulated slab restraint. The center of the tie plate was positioned at the center of the gap between the supporting girder and supported test beam. As called out in Figure 4.6, multiple pass fillet welds equaling the height of the tie plate were applied at the sides of the tie plate which did not bridge the connection. The opposite sides of the simulated slab restraint, which bridged the connection, were not welded to allow for additional vertical displacement of the supported beam. As seen in Figure 4.6, the surface where the slab restraint was to be welded was not whitewashed allowing for uncontaminated welding surfaces. Slab restraints were welded to the connection on site prior to testing.
4.2.3. TEST PREPARATION: ADDITIONAL ROTATIONAL BRACING

As illustrated in Figure 4.7, a pair of Channel shapes, which were positioned equidistantly about the connection, was used in addition with the simulated slab restraint to restrain the top flange of the supporting girder against rotation. The bracing Channels were attached to the strong wall by threaded rods and pulled snugly against the supporting girder. To allow for vertical deflection of the supporting girder, Teflon strips, which simulated a frictionless surface, were placed between the bracing Channels and the supporting girder.
4.2.4. MATERIALS

The steel used for the test columns, beams, and supporting girders was manufactured from ASTM A992 steel with a nominal yield strength of 50ksi. The steel used for the single plates and tie plates was manufactured from ASTM A36 steel with a nominal yield
strength of 36ksi. ASTM A325-N strength bolts were used along with ASTM A563 nuts and ASTM washers. In Tests 1 through 8, a single washer was placed under each bolt head to ensure that the threads were included in the shear plane. In Tests 9 and 10, the washer was placed between the nut and single plate. Welding of the specimens was performed in accordance with all current American Welding Society specifications. E70 strength electrodes were used for welding.

4.3. TEST SETUP

4.3.1. THREE BOLT RIGID SUPPORT TESTS

The three bolt rigid support tests consisted of a W14x145 support column (A992) connected to a W16x50 test beam (A992). The beam-to-column connection was facilitated with a single plate connection. The single plate connection consisted of a 3/8” x 4-1/2” x 0’-9” plate (A36) welded to the outside face of the column flange and bolted to the supported beam with 3/4in A325-N strength bolts. The single plate was offset from flange center to one side by one-half the web thickness of the test beam. The test column was rigidly supported by the strong wall and floor. The free end of the test beam was supported by a roller. The beam was braced laterally at approximately 5’-0” intervals to prevent lateral torsional buckling. Load was applied by two 110kip MTS actuators located at the quarter and three-quarter points of the test beam. A hydraulic ram positioned near the connection was used to apply additional load as needed to fail the connection. A diagram of a typical rigid test setup is shown in Figure 4.8. In Figure 4.9, a photograph of an actual three bolt test with rigid support condition is shown.
FIGURE 4.8 RIGID SUPPORT TEST SETUP DIAGRAM

FIGURE 4.9 RIGID SUPPORT TEST SETUP
4.3.2. SEVEN BOLT RIGID SUPPORT TEST

The seven bolt rigid support test consisted of a W14x145 support column (A992) connected to a W27x84 test beam (A992). The beam-to-column connection was made with a single plate connection. The single plate connection consisted of a $\frac{3}{8}''$ x 4-½” x 1'-9” plate (A36) welded to the outside face of the column flange and bolted to the supported beam with ¾in A325-N strength bolts. The single plate was offset from flange center to one side by one-half the web thickness of the supported test beam. The test column was rigidly supported by the strong wall and floor. The free end of the test beam was supported by a roller. A single washer was positioned beneath the nut. The beam was braced laterally at approximately 7'-0” intervals to prevent lateral torsional buckling. Load was applied by two 440kip MTS actuators located at the one-third and two-thirds points of the beam. A hydraulic ram positioned near the connection was used to apply additional load as needed to fail the connection. A typical rigid test setup is shown in Figure 4.10.
4.3.3. TWO AND THREE BOLT FLEXIBLE SUPPORT TESTS

Tests with a flexible support condition consisted of two W14x145 test columns (A992) supporting a W18x50 (A992) test girder which was supporting a W16x50 (A992) coped test beam. The single plate connection consisted of a $\frac{3}{8}''$ x 4-1/2'' x 0'-9'' plate (A36) for a three bolt test or a $\frac{3}{8}''$ x 4-1/2'' x 0'-6'' plate (A36) for a two bolt test welded to the web of the girder and bolted to the supported beam with 3/4in A325-N strength bolts. The single plate was offset from the center of the girder to one side by one-half the web thickness of the test beam. The test columns were rigidly supported by the strong wall and floor. The free end of the test beam was supported by a roller. The beam was braced laterally at approximately 5'-0'' intervals to prevent lateral torsional buckling. Load was applied by two 110kip MTS actuators located at the quarter and three-quarter points of the beam. A hydraulic ram positioned near the connection was used to apply additional
load as needed to fail the connection. A typical flexible test setup is shown in Figure 4.11.

![FIGURE 4.11 TYPICAL FLEXIBLE SUPPORT TEST SETUP](image-url)

**FIGURE 4.11 TYPICAL FLEXIBLE SUPPORT TEST SETUP**

### 4.3.4. SEVEN BOLT FLEXIBLE SUPPORT TEST

The flexible support tests consisted of two W14x90 test columns (A992) supporting a W30x99 (A992) test girder which was supporting a W27x84 (A992) coped test beam. The single plate connection consisted of a \( \frac{3}{8}\)" x 4-1/2" x 1'-9" plate (A36) welded to the web of the girder and bolted to the supported beam with 3/4in A325-N strength bolts. The single plate was offset from the center of the girder to one side by one-half the web thickness of the test beam. The test columns were rigidly supported by the strong wall and floor. The free end of the test beam was supported by a roller. The supported test beam was braced laterally at approximately 7'-0" intervals to prevent lateral torsional buckling. Load was applied by two 440kip MTS actuators located at the one-third and
two-thirds points of the beam. A hydraulic ram positioned near the connection was used to apply additional load as needed to fail the connection. A diagram for a seven bolt flexible test setup is shown in Figure 4.12. In Figure 4.13, a photograph of Test 10, seven bolt test with flexible support and simulated slab restraint, is shown.

FIGURE 4.12 SEVEN BOLT FLEXIBLE SUPPORT TEST SETUP
4.4. INSTRUMENTATION

4.4.1. GENERAL

The instrumentation utilized in this series of tests included load cells, strain gages, string potentiometers, and linear potentiometers. All instrumentation was calibrated prior
to testing and was checked to insure correct measurement after placement on the test specimen. Instrumentation was placed similarly on each test.

*Loading Devices & Load Cells*

The two actuators used in testing have built in load cells and Linear Voltage Displacement Transducers to record the load and displacements imposed by the actuators. An example of the two 440kip actuators as used in Test 10 can be seen in Figure 4.13 as shown in the Section 4.3.4. Beneath the roller support at the supported end of the beam, a 150kip load cell (for the two or three bolt tests) or a 200kip load cell (for seven bolt tests) was positioned to measure the amount of load transferred to the end support. Figure 4.14 is a photograph of the end support of a typical test setup showing a 200kip load cell and roller support assembly.
In the event that the hydraulic ram was used, an additional 100kip load cell (for two or three bolt tests) or a 200kip load cell (for seven bolt tests) was placed between the hydraulic ram and the beam to measure additional load imposed on the beam. Figure 4.15 is a photograph taken of the hydraulic ram assembly and 200kip load cell as used in Test 10. This photograph was taken prior to testing as a pivoting spacer was positioned between the hydraulic ram and load cell to compensate for beam curvature.
FIGURE 4.15 HYDRAULIC RAM ASSEMBLY AND LOAD CELL
Figure 4.16 is a diagram of a typical test setup, which shows both the positions of and the loading devices and load cells used in testing. To determine the applied shear at the connection and to determine the eccentricity of the load with respect to the bolts, the following procedure is used. Shear was assumed to be positive downward. Equation 4.1 shows that the shear at the connection was calculated by adding the loads of the two actuators, the hydraulic ram load cell, and subtracting the end support load cell.

\[ V_{\text{connection}} = V_{\text{hydraulic ram}} + V_{\text{actuator 1}} + V_{\text{actuator 2}} - V_{\text{end support}} \]  

(EQ 4.1)

Equation 4.2 expresses how moment at the connection was calculated. The distance was measured from the bolt line to each load measuring device. Multiplying this distance by the corresponding load produces the moment created at the bolt line by the respective
load. Moments are then summed and are assumed to be positive in the clockwise direction. The resultant is the moment sustained at the bolt line. Eccentricity was determined in Equation 4.3 by dividing the moment at the bolt line by the shear force at the bolt line. Eccentricity was measured from the bolt line and was determined to be positive toward the support end of the beam and negative in the opposite direction.

\[
M_{\text{connection}} = (V_{\text{hydraulic}} d_1) + (V_{\text{actuator}_1} d_2) + (V_{\text{actuator}_2} d_3) - (V_{\text{end support}} d_4) \quad (\text{EQ 4.2})
\]

\[
e_{\text{connection}} = \frac{M_{\text{connection}}}{V_{\text{connection}}} \quad (\text{EQ 4.3})
\]

*Please note that the other means of calculating connection eccentricity is presented as Equation 4.4 in the ‘Beam Strain’ section.

**Beam Displacements**

The beam deflections were measured using string potentiometers attached near the center of the bottom flange of the supported beam. Figure 4.17 shows a test beam with string potentiometers attached.
For the two and three bolt tests, string potentiometers were placed at the quarter points, midpoint, and near the connection and end support of the beam. For the seven bolt tests, string potentiometers were placed at the third points, midpoint, near the connection, and end support of the beam. Figures 4.18 and 4.19 detail the typical placement of string potentiometers in the three and seven bolt tests with a rigid support.
For the flexible supported tests, due to the width of the girder bottom flange, the string potentiometer measuring connection deflection cannot be attached underneath the
connection bolts. The string potentiometer measuring connection deflection was positioned on the bottom flange of the supported test beam as near to the connection bolts as possible. For the flexible supported tests, an additional string potentiometer was fixed to the center of the bottom flange of the girder near the connection region. This potentiometer measured deflections at the midpoint of the girder. Figure 4.20 details the typical placement of string potentiometers in two and three bolt tests with a flexible support condition while Figure 4.21 details deflection measurements for a seven bolt test with a flexible support condition.

FIGURE 4.20 TWO OR THREE BOLT FLEXIBLE SUPPORT BEAM DEFLECTIONS
Beam Strain

Uniaxial strain gages were used to measure beam strains and verify connection eccentricity. Strain gages were paired along the neutral axis of the beam on the outside face of the top and bottom beam flanges near the connection. Figure 4.22 shows an actual test connection prior to being whitewashed where strain gages have been applied.
Figures 4.23 and 4.24 illustrate a typical strain gage layout for a three bolt and seven bolt single plate connection with rigid support condition. Figure 4.25 shows (1) a typical strain gage layout for a test beam used in a two or three bolt connection with flexible support condition and (2) the typical single plate strain gage layout for a three bolt connection. Figures 4.26 and 4.27 depict a typical strain gage layout for both a two or three bolt and a seven bolt single plate connection with flexible support condition with simulated slab restraint. Shown in the single plate strain subsection is the typical single plate strain gage layout for two bolt tests.
FIGURE 4.23 THREE BOLT RIGID SUPPORT STRAIN GAGE LAYOUT

FIGURE 4.24 SEVEN BOLT RIGID SUPPORT STRAIN GAGE LAYOUT
FIGURE 4.25 TWO OR THREE BOLT FLEXIBLE SUPPORT STRAIN GAGE LAYOUT

FIGURE 4.26 TWO OR THREE BOLT FLEXIBLE SUPPORT W/ SIMULATED SLAB RESTRAINT STRAIN GAGE LAYOUT
For tests with simulated slab restraints, the first strain gage (TF1) on the top flange of the test beam could not be installed due to the position of the simulated slab restraint. However, the remaining gages were installed in the same locations as found in a typical flexible supported test having either two, three, or seven bolts. The first strain gage of the bottom flange was applied unpaired to record strain.

For the two or three bolt tests, after the placement of the initial strain gage a small distance away from the edge of the beam, a spacing of 6in was used thereafter. For seven bolt tests, a spacing of 9in was used after the initial gage. Equation 4.4 expresses the method by which eccentricity of the connection was calculated from measured strain values. Strain gage readings were positive when recording tension within the beam and negative when recording compression within the beam. Eccentricity was determined by analyzing one pair of strain gages at a time.
\[
e_{\text{connection}} = L_{\text{gage}} + \frac{E S_x \left( \frac{\varepsilon_{TF} - \varepsilon_{BF}}{2} \right)}{V_{\text{connection}}} \quad \text{(EQ 4.4)}
\]

where:
- \( L_{\text{gage}} \) = distance from the bolt line to the pair of strain gages
- \( E \) = Young’s Modulus of the beam (29,000ksi)
- \( S_x \) = elastic section modulus of the beam
- \( \varepsilon_{TF} \) = strain measured by the strain gage at the top flange of the beam
- \( \varepsilon_{BF} \) = strain measured by the strain gage at the bottom flange of the beam

After determining the eccentricity for each pair of strain gages, the resulting values are averaged to determine the eccentricity of the bolts. The first pair of strain gages sometimes yielded questionable eccentricities because the point of zero moment is near that location and the stress distribution in the beam is non-uniform. Therefore, the remaining three pairs of gages were averaged to verify eccentricity. Due to this fact, the bolt eccentricity of a flexible supported test with simulated slab restraint could still be found via calculation with data from the three pairs of strain gages located away from the connection.

**Single Plate Strain**

Each single plate was fitted with multiple uniaxial strain gages oriented in the direction of the length of the supported test beam to record the imposed strain. Strain gages were attached to one side of the single plate between the bolt holes and the weldment. Strain gages were placed symmetrically about the center of the single plate. In doing so, a stress distribution as calculated from strain gage readings could be produced for the length of the plate. Figure 4.28 shows the typical single plate strain
gage layout for a two bolt test with a flexible support condition. While previously shown, Figures 4.23 to 4.27 depict the single plate strain gage layouts for three and seven bolt connections. Identical single plate strain gage layouts were used for both the rigid and flexible supported conditions tests where single plates were identical in size.

![Strain Gage Layout](image)

**FIGURE 4.28 SINGLE PLATE STRAIN GAGE LAYOUT FOR A TWO BOLT TEST WITH FLEXIBLE SUPPORT**

4.4.2. RIGID SUPPORT TESTS

Instrumentation was placed to measure horizontal and vertical displacements of the bolts, single plate, and beam. For connection measurements, potentiometers were positioned in relation to the bolts and bolt line. Potentiometers were oriented either parallel or perpendicular to the ground and were fixed to the beam with magnetic bases. The measurements were made relative to the supporting component and the displacements were calculated to account for the approximate condition. An example of a fully instrumented test with rigid support condition is shown as Figure 4.29.
Horizontal displacement measurements were made at the top and bottom of the beam, single plate, and bolts to capture relative rotations. The distance between the pair of potentiometers was measured. By subtracting the bottom potentiometer measurement from the top and dividing the resultant by the distance between the two potentiometers, the rotation of the component is determined. Vertical displacement measurements were made on the bolts and single plate and vertical deflection measurements were made of the test beam as described in Section 4.4.1. Figure 4.30 details the placement of the Linear Potentiometers.
displacement potentiometers for a three bolt test with rigid support. Similarly, Figure 4.31 details the placement of the displacement potentiometers for a seven bolt test with rigid support.

FIGURE 4.30 THREE BOLT RIGID SUPPORT CONNECTION INSTRUMENTATION

FIGURE 4.31 SEVEN BOLT RIGID SUPPORT CONNECTION INSTRUMENTATION
4.4.3. FLEXIBLE SUPPORT TESTS

Instrumentation was placed to measure horizontal and vertical displacements of the bolts, single plate, and beam. Horizontal displacements of the girder and girder web were also measured. For connection measurements, potentiometers were positioned in relation to the bolts and bolt line. Potentiometers were oriented either parallel or perpendicular to the ground and were fixed to the beam with magnetic bases. The measurements were made relative to the supporting component and the displacements were calculated to account for the approximate condition.

Horizontal displacement measurements were made at the top and bottom of the beam, single plate, and bolts to capture relative rotations. Small pieces of angle were clamped to the edges of the top and bottom flanges to provide a surface for potentiometers to measure girder and beam rotations. A rigid base was placed near the girder opposite of the single plate and then displacement potentiometers were positioned to measure absolute girder rotation. For the girder web rotation, a long slender piece of metal was fixed to the flanges of the girder and displacement potentiometers were attached to this piece of metal. The potentiometers measured girder web rotations on the rear side of the girder directly opposite the single plate. The distance between the pair of potentiometers was measured. By subtracting the bottom potentiometer measurement from the top potentiometer measurement and dividing the resultant by the distance between the two potentiometers, the rotation of the component is determined. Vertical displacement measurements were made on the bolts and single plate and vertical deflection measurements were made of the test beam as described in Section 4.4.1.
Figures 4.32 and 4.33 are of Test 6, which is a two bolt connection with flexible support condition. Instrumentation on both sides of the support girder is shown.

FIGURE 4.32 INSTRUMENTED SINGLE PLATE CONNECTION WITH FLEXIBLE SUPPORT (CONNECTION SIDE SHOWN)
Figure 4.34 details the placement of the displacement potentiometers for a three bolt single plate connection with flexible support condition. Figure 4.35 details the placement of the displacement potentiometers for a two bolt single plate connection with flexible
support condition. In addition, Figure 4.36 details the placement of the displacement potentiometers for the seven bolt single plate connection with flexible support condition.

FIGURE 4.34 FLEXIBLE SUPPORT THREE BOLT CONNECTION INSTRUMENTATION

FIGURE 4.35 FLEXIBLE SUPPORT TWO BOLT CONNECTION INSTRUMENTATION
4.5. TEST PROCEDURE

The test specimen was installed into the test frame. The specimen was cleaned and a white wash was applied to the beam. For two and three bolt connection tests, the end support consisting of a load cell and steel roller support was centered and placed 3in from the end of the beam. For the seven bolt tests, the end support consisting of a load cell and steel roller support was centered and placed 1ft from the end of the beam.

A Skidmore-Wilheim bolt tension calibrator was used to determine the amount of tension incurred on a bolt from tightening with a spud wrench. A photograph of the Skidmore-Wilheim bolt tension calibrator is shown as Figure 4.37.
Each bolt was then pretensioned to a snug-tight level of approximately 10kips. Lateral brace mechanisms were tightened to brace the beam and prevent out of plane buckling. As needed for certain tests with a flexible support condition, bracing Channels were installed to brace the top flange of the supporting girder. Displacement potentiometers were calibrated prior to placement. Instrumentation was positioned and connected to the data acquisition system.

Each actuator was positioned such that it was plumb and the actuator plate was centered to the top flange of the beam. The data acquisition system and actuator control system were zeroed and set to begin recording data. The data acquisition system was set up such that data points were recorded and real-time plots were displayed for in-test monitoring. A preload of not more than 10 percent of the expected failure load was
manually applied at each actuator. The actuators were then placed under displacement control, loading the specimen at approximately 0.1 in/min. Due to varying stiffness of the test setup the displacement rate of the actuator nearer the connection was adjusted as necessary such that both actuators applied load equally. Loading continued and when test loads neared service load conditions, testing was paused and observations were made. For the two and three bolt tests, loading resumed and observations were recorded at approximately 5 kip intervals until failure of the connection. For the seven bolt tests after service conditions were reached, loading resumed and observations were recorded at approximately 10 kip intervals until the beam could not sustain additional load or connection failure occurred. In some tests, the connection would not sustain additional load from the actuators. Additional load was applied adjacent to the connection region via a hydraulic ram until connection failure occurred. In doing so, the hydraulic ram increased the shear force applied to the connection while not applying additional or excessive beam end rotation.

The above mentioned test procedure was applied to all tests. An objective of testing was to determine the ultimate capacity of the connection. In the seven bolt connection test with rigid support condition, loading was applied via the actuators until the test beam began to develop a plastic hinge. At this point, additional load was applied by both the actuators and hydraulic ram until the severity of the plastic hinge (and failure of lateral bracing mechanisms) dictated an end to testing. For this test, the connection did not fail as failure of the test beam controlled. For the seven bolt test with flexible support condition, prior to the development of the plastic hinge in the beam, additional load was applied to the beam by both actuators and the hydraulic ram while maintaining a
reasonably linear applied shear at the connection-deflection curve. This means of loading continued until connection failure occurred.

4.6. **SUPPLEMENTAL TESTS**

4.6.1. **BOLT TESTS**

*Bolt Tensile Test*

To predict the bolt tensile strength of the bolts used in the full-scale tests, a bolt tensile test was conducted on an extra bolt from the lot of bolts used in the testing program. The bolt was tightened on the Skidmore-Wilheim bolt tension calibrator until failure occurred.

*Bolt Shear Tests*

To predict the bolt shear strength of the bolts used in the full-scale tests, bolt shear tests were conducted on extra bolts from the lot of bolts used in the testing program. Bolt shear tests were conducted by fellow graduate student Lucas Gelo and documented in Gelo (2003), a research report submitted in partial fulfillment of MCE requirements. Two types of tests were conducted: a single bolt single shear test and a single bolt double shear test. Bolt shear tests were performed in accordance with ASTM F606-02: “*Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, and Rivets.*” ASTM F606-02 recommends the use of Military Standard, MIL STD 1312 Test 20 “*Fastener Test Methods Method 20 Single Shear*” and MIL STD 1312 Test 13 “*Fastener Test Methods Method 13 Double Shear.*”
A total of three single shear and three double shear tests were performed. Test specimens consisted of ¾in diameter ASTM A325-N strength bolts manufactured by Lohr Structural Fasteners. Considerations to plate strength and hardness were made in order to prevent yielding of the fixture plates, significant bearing deformation of the holes, or damage to the bolts. Comparative to the measured bolt properties of minimum 120ksi tensile stress and Rockwell hardness of 24-35 C, Grade 4142 tool steel was chosen as the test fixture material. This steel has a nominal yield strength of 130ksi and a Rockwell hardness of 30 C. Test fixture plates were chosen to accommodate the relatively small bolt thread length of 1.5in. For single shear tests, two ⅜in plates were used. While for double shear tests, two ¼in and one ½in plate were used. An example of a single shear test setup is shown in Figure 4.38 and a double shear test setup is shown below in Figure 4.39.

FIGURE 4.38 SINGLE SHEAR TEST SETUP AND INSTRUMENTATION

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Bolt shear tests were conducted using an MTS 220kip load frame under displacement control. A uniform displacement rate of 0.2in/min was chosen as the standard loading rate for both the single and double shear tests. Bolt shear tests utilized the following test procedure:

- Test fixtures were positioned vertically in the MTS test frame.
- 5kips of clamping force was applied to the gripping section of the test fixtures.
- Holes in the test fixtures were aligned, and a single test bolt was inserted and tightened to a sung-tight pretension of approximately 10kips.
- As pictured in Figure 4.38 and Figure 4.39, linear potentiometers held by magnetic bases were positioned to measure the displacements of the plates and test bolt.
• All measurement readings were zeroed and load was applied via downward movement of the lower head at the chosen displacement rate of 0.2 in/min until failure (which was found to be sudden shear rupture of the bolt).

4.6.2. TENSILE TESTS

Tensile coupon tests were conducted on the material cut from the beam flanges and single plate stock used in the testing program. A photograph depicting a typical tensile test setup is shown in Figure 4.40.

A total of two specimens per beam (for all tests), four specimens cut from two and three bolt plate stock, and two specimens cut from seven bolt plate stock were tested. The standard tensile specimens were prepared, measured, and tested in accordance with ASTM A370 “Standard Test Methods and Definitions for Mechanical Testing of Steel
“Products”. Tensile coupon tests were conducted using an MTS 220kip load frame under displacement control and utilized the following test procedure:

- A tensile coupon was positioned vertically in the MTS load frame.

- 5kips of clamping force was applied to the gripping section of the tensile coupon.

- A 2in extensometer was positioned upon the tensile coupon such that the knife edges of the extensometer were positioned in the 2in gage marks of tensile coupon.

- All measurement readings were zeroed and load was applied via downward movement of the lower head at a decided upon displacement rate.
CHAPTER 5  EXPERIMENTAL RESULTS

5.1. OVERVIEW

The following sections will summarize the primary findings and trends of the data recorded from the ten full-scale experimental tests and report the results of the supplemental bolt and tensile tests conducted. For the full-scale tests, trends regarding ultimate strength, connection eccentricity, and connection rotation behavior will be reported. In addition, the observed behavior due to the following test parameters will be discussed: number of bolts in the connection, support conditions: either rigid (beam-to-column flange) or flexible (beam-to-girder web), and the effect of the simulated slab restraint of tests with flexible support condition.

A summary of the ultimate shear at the connection is presented in Table 5.1. Measured maximum beam and girder rotations and calculated eccentricity of the bolts of each test are presented in the table. Test summary reports for each full-scale test are included in Appendices B through K. Included in each report is a test summary sheet, individual test setup diagrams, photographs of the setups and connections before and after failure, and individual result plots. The test summary sheet includes test specimen information, measured material strengths, experimental results, and test observations. Plots of the applied shear at the connection versus the corresponding rotations of the beam, single plate, bolts, and girder (for tests with a flexible support), the applied shear at the connection versus deflection of the beam, single plate, bolts, and girder (for tests with a flexible support), and the applied shear at the connection versus the bolt eccentricity are included in each test summary. The bolt eccentricity shown in the applied shear at the
connection and the resulting eccentricity plot was calculated using the load-distance method presented as Equation 4.3.
### TABLE 5.1 SUMMARY OF TEST RESULTS

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Description</th>
<th>Max. Applied Shear at Conn.(^1) (kips)</th>
<th>Beam End Rotation at Max. Shear (rad)</th>
<th>Bolt Eccentricity (in.)</th>
<th>Beam End Rotation (rad)</th>
<th>Girder Rotation (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>78.8 (^{2,3})</td>
<td>0.036</td>
<td>1.6</td>
<td>0.008</td>
<td>-----</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>90.7</td>
<td>0.027</td>
<td>2.1</td>
<td>0.008</td>
<td>-----</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>71.8 (^4)</td>
<td>0.039</td>
<td>(-) 1.7</td>
<td>0.011</td>
<td>0.037 (0.66 in.)</td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>61.4</td>
<td>0.023</td>
<td>(-) 2.0</td>
<td>0.010</td>
<td>0.031 (0.56 in.)</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>75.6 (^4)</td>
<td>0.031</td>
<td>0.1</td>
<td>0.010</td>
<td>0.017 (0.31 in.)</td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>44.2</td>
<td>0.012</td>
<td>(-) 4.4</td>
<td>0.005</td>
<td>0.013 (0.23 in.)</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>45.5</td>
<td>0.011</td>
<td>(-) 2.4</td>
<td>0.003</td>
<td>0.026 (0.47 in.)</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>47.9</td>
<td>0.013</td>
<td>(-) 2.2</td>
<td>0.004</td>
<td>0.005 (0.09 in.)</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>166.5 (^{2,3,4})</td>
<td>0.028 (^7)</td>
<td>5.5</td>
<td>0.006 (^7)</td>
<td>-----</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>202.5</td>
<td>0.027 (^7)</td>
<td>0.5</td>
<td>0.012 (^7)</td>
<td>0.010 (0.29 in.)</td>
</tr>
</tbody>
</table>

**Notes:**
1. Tests stopped after shear rupture of connection bolts
2. Test stopped prior to shear rupture of connection bolts
3. Test stopped due to lateral torsional buckling failure of test beam
4. Failure of lateral brace mechanism(s) occurred at end of test
5. Shear at the connection = 15 kips (2-bolt), 30 kips (3-bolt), 100 kips (7-bolt)
6. Value in parentheses ( ) is the girder flange horizontal displacement resulting from the observed rotation
7. Rotation value is based upon a prediction of beam rotation
5.2. THREE BOLT RIGID SUPPORT TESTS

Figures 5.1 and 5.2 show typical plots of applied shear versus beam & component rotation at the connection. Component rotations increase in the following order, the single plate exhibits the least rotation, the bolts, and then the beam exhibits the most rotation. Two rigid support, three bolt tests were conducted. Test 1 was stopped prior to shear rupture of connection bolts because the supported beam had failed due to lateral torsional buckling and was unable to transfer any additional load to the connection. Both tests exhibited similar load-rotation behavior. Comparisons between Test 1 and Test 2 indicated the following trends at the design load: the supported beam rotated less in the test with short-slotted holes, and the bolts and single plate rotated more with the presence of short-slotted holes.

![Test 2A (Rigid, Standard, Three-Bolt) Applied Shear vs. Rotation](image)

FIGURE 5.1 TYP. THREE BOLT RIGID CONNECTION APPLIED SHEAR VS. BEAM ROTATION
Figure 5.3 shows the applied shear at the connection versus bolt eccentricity. Both three bolt rigid supported tests yielded positive bolt eccentricities, which indicated that the point of zero moment lies within the test beam. Test 2, which utilized standard holes, had an eccentricity larger than Test 1 throughout the test. As shown in Figure 5.3, this connection converges to one distinct bolt eccentricity.
Figure 5.4 shows the applied shear at the connection versus the component vertical deflections. This plot illustrates typical deflection behavior of the connection. Deflection plots are based upon absolute measurements and do not show relative deflections of each component. The single plate deflects the least while the bolts deflect slightly more. The beam deflection is a result of the combination of the single plate and bolt deflections and deflects the most.

Called out in Figure 5.4 are the relative component movements for the supported test beam and connection bolts. Previously, the component rotations were measured in a global relationship where all measurements were in relation to the ground. In a relative relationship, the connection bolts are found to deflect more than the supported test beam.
One potential explanation for this behavior is that the single plate allows for bolt bearing to a greater extent than the supported test beam, which is manufactured from a higher grade steel. Therefore, relative to the single plate the connection bolts deflect the most.

![Graph showing Applied Shear vs. Deflection for Test 2A (Rigid, Standard, Three-Bolt)](image)

**FIGURE 5.4 TYP. THREE BOLT RIGID CONNECTION APPLIED SHEAR VS. DEFLECTIONS**

### 5.3. SEVEN BOLT RIGID SUPPORT TEST

Because only one seven bolt test with a rigid support condition was conducted, please refer to Appendix J: Test 9 for test summary and results plots. Test 9 was stopped prior to the shear rupture of connection bolts because failure of the lateral bracing mechanisms occurred alongside buckling of the supported beam. It should be noted that in Test 9, movement of test column away from the strong wall was observed upon post
test inspection. This movement adversely impacted experimental results. However, adjustments were made to compensate for test column movement. Please refer to Appendix J1 for a detailed explanation of the modification to the raw experimental data. Figure 5.5 illustrates the modified applied shear versus beam end rotation data. It should be noted that the test beam rotates similarly to other tests with a rigid support condition.

![Graph of Test 9A (Rigid, Slotted, Seven-Bolt) Applied Shear vs. Rotation](image)

**FIGURE 5.5 MODIFIED SEVEN BOLT FLEXIBLE CONNECTION APPLIED SHEAR VERSUS COMPONENT ROTATION**

In Test 9, bolt and single plate rotations were measured relative to the supported beam, and are plotted based upon absolute measurements and do not show relative rotations of each component. From Figure 5.6, the bolts appear to follow the rotation of the test beam very closely. The rotational behavior of the single plate is similar to that of the bolts up to approximately 70kips at which point a shift occurs (possibly magnified by
the approximation of the test column rotation seen in Figure J1 of Appendix J1). At the end of testing, the component rotations rank in magnitude in the following order, the single plate exhibits the least rotation, the bolts, and then the beam exhibits the most rotation.

![Test 9A (Rigid, Slotted, Seven-Bolt) Applied Shear vs. Component Rotation](image)

**FIGURE 5.6 SEVEN BOLT RIGID CONNECTION COMPONENT ROTATION BEHAVIOR**

Similar to the three bolt rigid support tests, the applied shear at the connection versus bolt eccentricity for the seven bolt rigid supported test was also positive. This indicates that the point of zero moment lies within the beam. As with the three bolt tests, bolt eccentricity was determined by approximating the trend of the data set. For this test, the eccentricity of the connection was not as clearly defined on the applied shear versus
eccentricity plot. From the data plot, a bolt eccentricity of 5.5in, significantly larger than an average 1.8in for Tests 1 and 2, was recorded.

In comparison to the three bolt rigid supported tests, the applied shear at the connection versus the component vertical deflection behavior is similar. The single plate deflects the least while the bolts deflect slightly more. The beam deflection is a result of the combination of the single plate and bolt deflections and deflects the most.

5.4. TWO AND THREE BOLT FLEXIBLE SUPPORT TESTS

Figure 5.7 shows a typical plot of applied shear versus component rotation at the connection for a two or three bolt test with flexible support condition. Component rotations increase in the following order: the beam, the bolts, the girder, the girder web, and the single plate. Each test exhibited similar behavior. Comparing the two bolt tests (Test 6 (with standard holes) and Test 7 (with short-slotted holes)) at the design load, the following trends are observed: (1) the beam rotation of the test with standard holes is greater throughout the duration of testing, and (2) the bolt, girder, and girder web rotation is greater for the test with short-slotted holes throughout the duration of testing.
Tests 5 and 8 with flexible support condition and short-slotted holes incorporated a simulated slab restraint. To determine the affect of the simulated slab restraint, a comparison between Test 3 (no simulated slab restraint) and Test 5 (with simulated slab restraint) revealed the following trends in Test 5: (1) the rotation of the girder was significantly reduced, (2) the girder web and single plate rotated less, and (3) the beam and bolts rotated relatively the same if just slightly less for the test with simulated slab restraint.

For tests with simulated slab restraints, examination of the isolated component rotations shows that the girder web was subjected to increased rotations and that the single plate and bolts exhibited slightly increased rotations to compensate for the
restrained movement of the girder. Comparing the horizontal displacement measurements of the linear potentiometers of the single plate and bolts for tests with simulated slab restraints, measurements of the bottom linear potentiometer greatly exceeded that of the top potentiometer. This indicates that the point of rotation of the connection shifts toward the simulated slab restraint, potentially affecting bolt and single plate behavior. A further discussion regarding the center of rotation of the connections is presented in Section 6.5 Rotational Behavior.

Figure 5.8 shows the applied shear at the connection versus bolt eccentricity. Test 5, which incorporated a simulated slab restraint, yielded a slightly positive eccentricity while the other flexible supported tests yielded a negative bolt eccentricity. Negative bolt eccentricity correlates with the direction opposite the beam span. Compared to the three bolt rigid tests, the two and three bolt flexible tests had larger eccentricities. In addition to the support condition, the hole type possibly affects the resulting eccentricity. In general, tests with standard holes had eccentricities that were larger in magnitude. Simulated slab restraints, which were used in conjunction with short-slotted holes, shifted the overall bolt eccentricity toward the direction of the supported beam. Bolt eccentricity was determined by approximating the trend of the data set. As presented in Figure 5.8, this connection had one distinct bolt eccentricity.
Figure 5.9 shows the applied shear at the connection versus the component deflections. This plot illustrates behavior typical of a two or three bolt connection with a flexible support condition. Deflection plots are based upon absolute measurements and do not show relative deflections of each component. The girder deflects the least while the single plate and bolts deflect slightly more. The beam deflection is a result of the combination of the single plate and bolt deflections and deflects the most.
5.5. SEVEN BOLT FLEXIBLE SUPPORT TEST

Because only one seven bolt test with a flexible support condition was conducted, please refer to Appendix K: Test 10 for the test summary and resulting data plots. This test was loaded to its ultimate load which resulted in bolt shear rupture. Similar to Test 9, the test data for Test 10 required alterations due to a significant shift that occurred in the connection at approximately 95kips. This shift in the connection adversely impacted experimental results and modifications were made to compensate accordingly. Please refer to Appendix K1 for a detailed explanation of the modification to the raw experimental data. Figure 5.10 shows the final applied shear versus component rotation plot at the connection for Test 10 (seven bolt connection with flexible support condition and simulated slab restraint). As predicted in Figure 5.10, the trend of Test 10 shows that
at ultimate capacity the component rotations increase in the following order: the beam, the bolts, the girder, the girder web, and the single plate. The main difference is that the girder rotation begins as the least and then surpasses the bolts and beam rotation prior to the connection failure.

![Test 10A (Flexible, Slotted, Slab Restraint, Seven-Bolt) Applied Shear vs. Rotation](image)

**FIGURE 5.10 MODIFIED SEVEN BOLT FLEXIBLE CONNECTION APPLIED SHEAR VERSUS COMPONENT ROTATION**

When looking at the applied shear at the connection versus bolt eccentricity for Test 10, the connection eccentricity is observed as positive. This indicates that the point of zero moment lies within the beam. Prior to the shift in the connection at 95kips, the eccentricity was well defined at 1in and after the shift the eccentricity tended to 0.5in.
Comparatively, Test 5, which had a simulated slab restraint, yielded a slightly positive eccentricity as well.

Figure 5.11 shows the applied shear at the connection versus the component deflections for Test 10. Similar to the applied shear vs. rotation plots, Figure 5.11 is a depiction of the modified deflection data as potentiometers measuring vertical deflection of test components were shifted as well.

![Figure 5.11 Modified Seven Bolt Flexible Connection Applied Shear vs. Deflection](image)

**FIGURE 5.11 MODIFIED SEVEN BOLT FLEXIBLE CONNECTION APPLIED SHEAR VS. DEFLECTION**

Deflection plots are based upon absolute measurements and do not show relative deflections of each component. Similar to other connections with a flexible support condition, this plot illustrates typical deflection behavior. The girder deflects the least
while the single plate and bolts deflect slightly more. The beam deflection is a result of the combination of the single plate and bolt deflections and deflects the most.

5.6. EXPERIMENTAL BOLT ECCENTRICITY RESULTS

As presented in Chapter 4, two methods (listed as Equations 4.3 and 4.4) for determining the eccentricity of the connection were utilized. As presented in Table 5.1, the bolt eccentricities reported for each test were based upon Equation 4.3 at the tabulated design load of the connection as listed in the AISC LRFD 3rd Edition Manual (AISC 2003). To verify the bolt eccentricity of the connection, Equation 4.4 was used. Figure 5.12 depicts an applied shear versus bolt eccentricity plot using data from Test 1. In this figure, the bolt eccentricity data from the statically calculated and averaged beam strain gage data are shown. As presented, both calculation methods indicate the bolt eccentricity for Test 1 is equivalent to 1.6in.
Table 5.2 consists of a summary of bolt eccentricity values as found by both methods of calculating bolt eccentricity. From Table 5.2, the following trends were noted. For three bolt single plate connections tested, the eccentricity values as calculated by Equations 4.3 and 4.4 were near identical. For two bolt connections, eccentricity values calculated using Equation 4.3 were greater than or equal to eccentricity values calculated from Equation 4.4. A similar trend was noted for the seven bolt connection with rigid support condition. However for the three and seven bolt tests with flexible support condition and simulated slab restraint, the bolt eccentricity calculated using the strain gages on the supported test beam was greater in magnitude. However, the reason for reporting the eccentricity values as calculated using Equation 4.3 was because those values tended to be larger in magnitude and would thus be a more conservative value.
### TABLE 5.2 EXPERIMENTAL BOLT ECCENTRICITY

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Description</th>
<th>Bolt Eccentricity (Eqn. 4.3) (^4) (in.)</th>
<th>Bolt Eccentricity (Eqn 4.4) (^5) (in.)</th>
<th>Ratio (Eqn 4.3/Eqn 4.4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>1.6</td>
<td>1.6 (^3)</td>
<td>1.00</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>2.1</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>(-) 1.7</td>
<td>(-) 1.5 (^2)</td>
<td>1.17</td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>(-) 2.0</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.1</td>
<td>(-) 1.4 (^2)</td>
<td>0.07</td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>(-) 4.4</td>
<td>(-) 1.3 (^2)</td>
<td>3.33</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>(-) 2.4</td>
<td>(-) 0.5 (^2)</td>
<td>4.80</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>(-) 2.2</td>
<td>(-) 0.9 (^2)</td>
<td>2.44</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>5.5</td>
<td>1.5 (^3)</td>
<td>3.67</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.5</td>
<td>(-) 1.1 (^2)</td>
<td>0.45</td>
</tr>
</tbody>
</table>

**Notes:**
1. Shear at the connection = 20 kips (2-bolt), 30 kips (3-bolt), 100 kips (7-bolt)
2. Average eccentricity of the three pairs of strain gages furthest from the connection
3. Average eccentricity of all strain gages located on the supported test beam
4. Eccentricity statically calculated using Equation 4.3
5. Eccentricity calculated from strain gages on supported test beam using Equation 4.4

### 5.7. LIMIT STATES OF THE CONNECTIONS TESTED

The primary failure mode of the connections tested was observed to be shear rupture of the connection bolts. However, the onset of secondary failure modes occurring prior to ultimate loading were observed. One of the experimental objectives was to monitor the behavior of the connection components in a way such that the formation of a failure mechanism (limit state) was captured. From the applied shear versus rotation and deflection plots, nonlinear behavior potentially indicated the onset of other failure modes, such as the case for shear yielding, plate bending (or plate flexural yielding), and bearing and tear out of the single plate. Illustrated as Figures 5.13 and 5.14, the applied shear...
versus component rotation and deflection plots are shown for Test 2. The analysis procedure for determining the corresponding limit state failure load is discussed herein.

**FIGURE 5.13 SINGLE PLATE LIMIT STATE PREDICTION (FLEXURAL YIELDING AND BEARING AND TEAR OUT)**
FIGURE 5.14 SINGLE PLATE LIMIT STATE PREDICTION (SHEAR YIELDING)

As illustrated in the figures, the behavior of the connection components began linearly. Due to the ductile behavior of the connection and the magnitude of the applied loads, rotation and deflection behavior becomes non-linear as the ultimate load is approached. As seen by the dashed lines fitted to the data, a quantifiable prediction of the strength at which other limit states began to ensue is shown as the intersection of the two dashed lines. In Figure 5.13, the limit states of plate flexural yielding and bearing and tear out were monitored as they are failure modes associated with the rotation of the supported test beam. Since shear yielding is characterized by non-linear vertical deflection of the connection plate, the onset of this failure mode can be determined from the experimental data. The observed limit states as determined from the full-scale test results are summarized in Table 5.3.
TABLE 5.3 EXPERIMENTAL LIMIT STATE STRENGTHS

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Observed Limit State Strengths (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test No.</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Shear Yielding</td>
<td>77.50</td>
</tr>
<tr>
<td>Flexural Yielding (a = 3)in</td>
<td>74.50</td>
</tr>
<tr>
<td>Bearing/Tear out</td>
<td>62.00</td>
</tr>
<tr>
<td>Bolt Shear Strength</td>
<td>-----</td>
</tr>
</tbody>
</table>

Indicates the failure mode and ultimate strength experienced by the connection.

5.8. SUPPLEMENTAL TESTS

Appendix L contains detailed results for the supplemental tests conducted. For each tensile coupon test, a test summary sheet containing the following information is provided: measured dimensions of each coupon, loading rates utilized in testing, stress-strain plots, and results such as the yield stress, ultimate strength, and modulus of elasticity of the coupon.

5.8.1. BOLT TESTS

Bolt Tensile Test

A torqued bolt tensile test was conducted to determine the ultimate tensile strength of a single bolt from the lot of bolts used in the testing program. As indicated by the Skidmore-Wilheim bolt tension calibrator, the ultimate tensile strength of the ¾in diameter A325 strength bolt was 47kips.

Bolt Shear Tests

From the supplemental bolt shear tests conducted and documented by Gelo (2003), the following experimental results were reported. The A325-N strength bolts tested yielded consistent and repeatable results. In single shear, the average shear strength was
found to be 30.3 kips per bolt, and in double shear, the average shear strength was found to be 29.3 kips per bolt. Table 5.4 shows the summary of the results obtained.

### TABLE 5.4 SUMMARY OF BOLT SHEAR TEST RESULTS

<table>
<thead>
<tr>
<th>Description</th>
<th>Test No.</th>
<th>Maximum Applied Load (kips)</th>
<th>Relative Plate Displacement at Failure (in.)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Shear</td>
<td>1</td>
<td>29.9</td>
<td>0.1434</td>
<td>Bolt Shear Rupture</td>
</tr>
<tr>
<td>Single Shear</td>
<td>2</td>
<td>31.4</td>
<td>0.1668</td>
<td>Bolt Shear Rupture</td>
</tr>
<tr>
<td>Single Shear</td>
<td>3</td>
<td>29.6</td>
<td>0.1415</td>
<td>Bolt Shear Rupture</td>
</tr>
<tr>
<td>Double Shear</td>
<td>4</td>
<td>58.8</td>
<td>N/A</td>
<td>Bolt Shear Rupture</td>
</tr>
<tr>
<td>Double Shear</td>
<td>5</td>
<td>60.0</td>
<td>0.1473</td>
<td>Bolt Shear Rupture</td>
</tr>
<tr>
<td>Double Shear</td>
<td>6</td>
<td>57.0</td>
<td>0.1268</td>
<td>Bolt Shear Rupture</td>
</tr>
</tbody>
</table>

Table 5.5 shows a comparison of experimental to predicted strengths. The predicted strength of a single bolt in single shear with threads included in the shear plane was calculated to be 26.5 kips per bolt. This value is obtained in the following manner:

\[
R_n = \frac{F_v A_b}{0.8} \tag{EQ 5.1}
\]

where:
- \( A_b \) = nominal unthreaded cross-sectional area of a single bolt:
  \( A_b = 0.4418 \text{in}^2 \)
- \( F_v \) = nominal shear strength, 48 ksi (Table J3.2 of the AISC LRFD 3rd Edition Manual (AISC 2003))
- \( R_n \) = predicted shear strength of a single bolt

The 0.8 factor in Equation 5.1 is a strength reduction factor to account for non-uniform load distribution in a connection that consists of more than two bolts. Dividing by 0.8 will yield the predicted shear strength for a single bolt. In comparison to the
formulated shear strength value of 26.5kips per bolt, results indicated that the test bolts were over-strength.

**TABLE 5.5 BOLT SHEAR TEST STRENGTH COMPARISON**

<table>
<thead>
<tr>
<th>Description</th>
<th>Test No.</th>
<th>Predicted Load (kips)</th>
<th>Maximum Applied Load (kips)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Shear</td>
<td>1</td>
<td>26.5</td>
<td>29.9</td>
<td>1.13</td>
</tr>
<tr>
<td>Single Shear</td>
<td>2</td>
<td></td>
<td>31.4</td>
<td>1.18</td>
</tr>
<tr>
<td>Single Shear</td>
<td>3</td>
<td></td>
<td>29.6</td>
<td>1.12</td>
</tr>
<tr>
<td>Double Shear</td>
<td>4</td>
<td></td>
<td>58.8</td>
<td>1.11</td>
</tr>
<tr>
<td>Double Shear</td>
<td>5</td>
<td></td>
<td>60.0</td>
<td>1.13</td>
</tr>
<tr>
<td>Double Shear</td>
<td>6</td>
<td></td>
<td>57.0</td>
<td>1.07</td>
</tr>
</tbody>
</table>

The general behavior of the single shear bolt test and double shear bolt test can be seen in Figures 5.15 and 5.16. In both figures, applied load is plotted against relative plate displacement (the separation between the outside edges of the two test fixture plates).
Predicted Single Plane Shear Strength:
3/4in dia. A325-N Bolt (26.5kips)

Predicted Double Plane Shear Strength:
3/4in dia. A325-N Bolt (53.0kips)

FIGURE 5.15 TYP. SINGLE SHEAR APPLIED LOAD VS. RELATIVE PLATE DISPLACEMENT

FIGURE 5.16 TYP. DOUBLE SHEAR APPLIED LOAD VS. RELATIVE PLATE DISPLACEMENT
The following observations were made from the supplemental bolt shear tests. Each test failed due to shear rupture of the test bolt. An example of a failed bolt from a single shear test can be seen in Figure 5.17. A bolt failed from a double shear test can be seen in Figure 5.18. The bolt pictured in Figure 5.18 was the first bolt tested in double shear. As can be seen, this test bolt exhibits two distinct shear failure planes. After the first double shear test, the test fixture became slightly deformed and may have possibly incurred a prying force on the test bolts. This deformation of the fixture plates can be seen in Figure 5.19. In the remaining double shear tests, failure occurred at the shear plane closest to the head of the bolt with significant shearing deformation at the other shear plane.

FIGURE 5.17 SINGLE SHEAR TEST BOLT AT END OF TEST (FIT BACK TOGETHER)
FIGURE 5.18 DOUBLE SHEAR TEST BOLT AT END OF TEST

FIGURE 5.19 DOUBLE SHEAR TEST LAP PLATE PRYING (AT END OF TEST)
5.8.2. TENSILE TESTS

From the supplemental tensile tests conducted in accordance with ASTM A370 “Standard Test Methods and Definitions for Mechanical Testing of Steel Products”, the following section summarizes the obtained experimental results. Detailed results for each tensile coupon test are included in Appendix L. Each test summary sheet includes a test summary section, coupon measurements, loading rates, determined properties, an overall stress-strain plot, and a plot for determining the yield strength.

Table 5.6 shows the summary of the supplemental tensile coupon tests conducted and correlates the results with the test specimen component from which the coupon was cut. Included within this table are the yield stress and ultimate tensile strength of each tensile coupon tested. The yield stress of the material was determined using a 0.2 percent offset of the recorded stress-strain relationship. The ultimate tensile strength and the total elongation were also determined.
### TABLE 5.6 SUMMARY OF TENSILE COUPON RESULTS

<table>
<thead>
<tr>
<th>Description</th>
<th>Test I.D.</th>
<th>Thickness (in.)</th>
<th>Yield Strength (ksi)</th>
<th>Tensile Strength (ksi)</th>
<th>Elongation 8in Gage (%)</th>
<th>Average Yield Str. (ksi)</th>
<th>Average Tensile Str. (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W16x50 Flange</td>
<td>S1-RSS-3-A325-N</td>
<td>0.634</td>
<td>53.7</td>
<td>69.6</td>
<td>29</td>
<td>53.6</td>
<td>69.3</td>
</tr>
<tr>
<td>W16x50 Flange</td>
<td>S2-RST-3-A325-N</td>
<td>0.610</td>
<td>52.7</td>
<td>68.9</td>
<td>28</td>
<td>52.5</td>
<td>69.2</td>
</tr>
<tr>
<td>W16x50 Flange</td>
<td>S7-FSS-2-A325-N</td>
<td>0.609</td>
<td>54.6</td>
<td>70.1</td>
<td>30</td>
<td>54.2</td>
<td>69.6</td>
</tr>
<tr>
<td>W16x50 Flange</td>
<td>S8-FSS-2-A325-N-SR</td>
<td>0.557</td>
<td>39.3</td>
<td>61.9</td>
<td>27</td>
<td>39.6</td>
<td>62.1</td>
</tr>
<tr>
<td>W16x50 Flange</td>
<td>S4-FST-3-A325-N</td>
<td>0.581</td>
<td>53.9</td>
<td>70.9</td>
<td>28</td>
<td>53.5</td>
<td>70.9</td>
</tr>
<tr>
<td>W16x50 Flange</td>
<td>S3-FSS-3-A325-N</td>
<td>0.380</td>
<td>40.2</td>
<td>62.0</td>
<td>21</td>
<td>39.6</td>
<td>61.9</td>
</tr>
<tr>
<td>W16x50 Flange</td>
<td>S5-FSS-3-A325-N-SR</td>
<td>0.380</td>
<td>39.5</td>
<td>61.5</td>
<td>31</td>
<td>39.6</td>
<td>62.1</td>
</tr>
<tr>
<td>W16x50 Flange</td>
<td>S6-FST-2-A325-N</td>
<td>0.380</td>
<td>39.3</td>
<td>62.8</td>
<td>29</td>
<td>39.6</td>
<td>62.1</td>
</tr>
<tr>
<td>W18x50 Flange</td>
<td>-Tests- (S3-S8)</td>
<td>0.592</td>
<td>55.0</td>
<td>70.7</td>
<td>28</td>
<td>54.1</td>
<td>70.4</td>
</tr>
<tr>
<td>W27x84 Flange</td>
<td>S9-RSS-7-A325-N</td>
<td>0.620</td>
<td>53.1</td>
<td>70.0</td>
<td>28</td>
<td>59.6</td>
<td>75.0</td>
</tr>
<tr>
<td>W30x99 Flange</td>
<td>S9-RSS-7-A325-N</td>
<td>0.683</td>
<td>60.2</td>
<td>75.0</td>
<td>25</td>
<td>59.6</td>
<td>75.0</td>
</tr>
<tr>
<td>3/8&quot; Plate</td>
<td>S9-RSS-7-A325-N</td>
<td>0.367</td>
<td>44.6</td>
<td>66.2</td>
<td>28</td>
<td>44.4</td>
<td>66.25</td>
</tr>
</tbody>
</table>

The tensile tests conducted on material from test beams (for all tests), test girders (for all tests), and the single plates (used in Tests 9 and 10) behaved in a typical manner of hot rolled steel as pictured in Figure 5.20. Plots from tests conducted on single plate tensile coupons used in Tests 1-8 reveal a shorter yield plateau as pictured in Figure 5.21. However, the overall shape of the stress-strain curve for the plate steel reflected a typical stress-strain plot for steel.
5.9. DISCUSSION OF TEST PARAMETER EFFECTS

The ten full-scale tests conducted incorporated various test parameters. As discussed in previous sections, tests behaved similar to each other with respect to support condition and number of bolts in the connection. The following subsections will comment on the various trends and observations in terms of the number of bolts in the connection, effect of hole type (short-slotted or standard holes), effect of support condition (rigid (beam-to-
column) or flexible (beam-to-girder web)), and the effect of a simulated slab restraint. Table 5.1, which contains a summary of the prominent test results, should be referred to as necessary.

5.9.1. NUMBER OF BOLTS IN CONNECTION

In comparing trends of two bolt, three bolt, and seven bolt connections, a primary difference is in the strength of the connection. As measured from the supplemental single bolt single shear tests, the average single shear strength of a test bolt was found to be 30.3kips. The connection bolt shear rupture strength (no eccentric shear considerations) of the two, three, and seven bolt connections is 60.6kips, 90.9kips, and 212.1kips. Table 5.7 summarizes the strengths of the connections tested alongside the experimentally measured bolt strength. Of the ten tests conducted, Tests 2 through 8 and 10 failed in shear rupture of the connection bolts. From Table 5.7, as intuitively expected, the strength of a three bolt connection is greater than the strength of a two bolt connection. Similarly, the strength of the seven bolt connection is greater than the strength of a three bolt connection. However, the observed ultimate capacities of the tests conducted were not equal to the predicted bolt group shear strength as predicted from the supplemental bolt shear tests. A further discussion of this behavior is presented in Section 5.9.3.
<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Test Description</th>
<th>Obs. $e_b$ (in.)</th>
<th>Max Load (kips)</th>
<th>Predicted Vu (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>1.6</td>
<td>-----</td>
<td>90.90</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>2.1</td>
<td>90.7</td>
<td>90.90</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>(-) 1.7</td>
<td>71.8</td>
<td>90.90</td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>(-) 2.0</td>
<td>61.4</td>
<td>90.90</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.1</td>
<td>75.6</td>
<td>90.90</td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>(-) 4.4</td>
<td>44.2</td>
<td>60.60</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>(-) 2.4</td>
<td>45.5</td>
<td>60.60</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>(-) 2.2</td>
<td>47.9</td>
<td>60.60</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>5.5</td>
<td>-----</td>
<td>212.1</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.5</td>
<td>202.5</td>
<td>212.1</td>
</tr>
</tbody>
</table>

5.9.2. BOLT HOLE TYPE

The effects of short-slotted and standard bolt holes were investigated as a part of this research. This comparison was made primarily for Tests 1-4, 6, and 7. Tests with similar number of bolts and support conditions were compared to investigate the effect of the hole type. Analysis of the experimental connection rotational stiffness at various applied loads indicated that for both the rigid and flexible supported tests, the test beam rotated less in the presence of short-slotted holes. In addition, short-slotted holes were found to allow for greater absolute girder, girder web, single plate, and bolt rotation. However, in terms of ultimate capacity, the behavior was approximately the same for both hole types. This can be seen between Tests 6 and 7, which failed at 44.2kips and 45.5kips respectively, which is the only example of this behavior.
5.9.3. SUPPORT CONDITION

The first notable effect of support condition was the ultimate capacity sustained by the connection. Referring to Table 5.7, a comparison of the experimental values leads to the following observation: tests with a flexible support condition failed at less than the average single shear strength of 30.3kips per bolt, while tests with a rigid support condition failed very near to the experimentally measured average single shear strength. For flexible supported tests, this is a trend because many tests support this observation. However, Test 2 is the only rigid supported test to fail via shear rupture of the connection bolts.

An interesting observation was made from the results of Test 2. This test, which was a test with rigid support condition, failed near the average single shear bolt shear strength even with near 2in of bolt eccentricity. As discussed, other tests with a flexible support condition which had near this magnitude of eccentricity did not fail near the experimentally measured average single bolt single shear strength. It should be noted that tests with a flexible support condition were subjected to combined rotations of both the supporting girder and supported test beam. This increased rotation is one of the parameters that reduced the connection strength.

It should be noted that the general trend for rigidly supported connections is that the eccentricity begins excessively positive (within the span of the supported test beam) and quickly stabilizes to a specific eccentricity as seen in Figure 5.3. In contrast, for single plate connections with a flexible support condition, the eccentricity begins excessively negative and then stabilizes as seen in Figure 5.8.
5.9.4. EFFECT OF THE SIMULATED SLAB RESTRAINT

In Tests 5, 8, and 10, a simulated slab restraint was incorporated with a flexible support and short-slotted hole condition. To facilitate a direct comparison, Tests 5 and 8 utilize the same connection configuration as Tests 3 and 7. Comparing results and behavior, the ultimate capacity of the connection was increased by approximately 5 percent with the addition of the simulated slab restraint. It should be noted that the eccentricity for the flexible support tests with simulated slab restraint was smaller in magnitude.

5.10. SUMMARY

From the ten full-scale tests conducted, a baseline of results for typical single plate framing connections subjected to realistic applied loads beam end rotations was created. From the plotted test results, approximations to the onset of limit states of the single plate were established. As experienced by the majority of tests, failure occurred via shear rupture of the connection bolts. Based upon measured ultimate sustained shear loads, connection capacities were determined. From both the predicted limit states and the ultimate capacities of the connections, the means for comparisons to theoretical predicted connection capacities were established.

The ten full-scale tests incorporated the following test parameters: either two, three, or seven bolt connections with either short-slotted or standard holes, either rigid (beam-to-column) or flexible (beam-to-girder web) support conditions, and certain tests incorporated a simulated slab restraint. As commented upon in the previous sections, each parameter potentially had an effect upon the behavior of the connection.
In addition to the full-scale tests, supplemental bolt and material tensile tests were conducted to measure the material properties of the components used in testing. With these measured properties, component materials could be identified as having nominal strength properties as listed in the AISC LRFD 3rd Edition Manual (AISC 2003) or as being over-strength. As such, the measured strengths can then be used to explain the connection behavior of the full-scale tests in terms of stiffness and strength.
CHAPTER 6  ANALYTICAL INVESTIGATION

6.1.  OVERVIEW

As previously discussed, the design strength of single plate framing connections is based upon fifteen limit states. The limit states are grouped according to connection component which include the connection bolts as well as the single plate itself. In an attempt to better understand the behavior and predict the strengths of critical test components such as the single plate and connection bolts, analyses utilizing experimentally measured data were conducted. Utilizing the data obtained from the full-scale tests and the measured material properties of the test components from the supplemental tests, the theoretical design strength for each limit state was calculated. Within this chapter, tabulated analytical results will be presented alongside experimentally obtained data for critical limit states indicated by testing.

The first portion of this chapter will focus upon the bolt group shear strength. Calculations will incorporate various bolt eccentricity assumptions. In addition to providing comparisons for tests which are a part of this research, experimental data from full-scale tests as conducted by Astaneh et al. (1988, 1989, 1990, 1992, and 1993) are analyzed.

Analytical investigations regarding the connection components such as the supported beam, single plate, and weldment are discussed as appropriate. Comparisons of the analytical and experimental results are made. In addition, the bending behavior (elastic or plastic) of the single plate is investigated. Within this investigation, both plastic and elastic behavioral assumptions are applied to the analysis.
The rotational behavior of single plate framing connections is also investigated. From testing, the rotational behavior of the connections was found to be primarily dependent upon the support condition (rigid or flexible) and the use of a simulated slab restraint with flexible support condition. Other design parameters, such as the bolt hole type, potentially affect the rotational behavior as well. A discussion of the primary observations is included herein.

6.2. CONNECTION STRENGTH PREDICTIONS: CONNECTION BOLTS

6.2.1. GENERAL

The following subsections will present analytical results for bolt group shear capacities associated with tests conducted in this series of research as well as research conducted by Astaneh et al. (1988, 1989, 1990, 1992, and 1993). The analytical investigation is conducted to evaluate the necessity of the following considerations in accurately predicting observed connection capacities: (1) incorporating a group action factor, which reduces the bolt group shear strength by 20 percent to account for non-uniform load distributions in connections consisting of two or more bolts and (2) applying eccentric shear considerations to the bolt group. The following bolt group shear capacity analytical investigations are conducted here in:

- Nominal LRFD bolt group shear strength calculations with various eccentricity assumptions for both tests conducted in this series of research and research conducted by Astaneh et al. (1988, 1989, 1990, 1992, and 1993). The reason for using nominal bolt shear strength values is to facilitate comparable analyses between research conducted by Astaneh et al., which did not measure the strength of test bolts, and tests conducted in this series of research, where the strength of test bolts was measured.
• For this series of tests, bolt group shear strength predictions based upon measured material properties from supplemental tests with various eccentricity assumptions and without a group action factor.

For both means of calculation, eccentricity assumptions include: zero eccentricity, observed test eccentricity, (bolt line to weld line) \( a \)-distance limited eccentricity, and eccentricity as based upon the Astaneh eccentricity prediction equations, which are presented as Equations 3.24 to 3.27. It should be noted that the Astaneh bolt eccentricity values utilized were not limited to be greater than or equal to \( a \) (bolt line to weld line distance) for tests with a flexible support condition.

6.2.2. LRFD BOLT GROUP CAPACITY ANALYSIS OF CONDUCTED TESTS

This comparison involved the computation of nominal LRFD bolt group shear strengths as calculated with and without the effects of bolt eccentricity. Eccentricity assumptions included: (1) zero eccentricity \( (e_b = 0\text{in}) \), (2) observed test eccentricity, (3) limiting the test eccentricity to the bolt line to weld line distance, \( a \), which was 3in for this series of tests, and (4) predicted Astaneh bolt eccentricity. The LRFD bolt group shear strength values without eccentric shear considerations were calculated as described in Section 3.3.1, with a nominal single bolt single shear strength, \( F_v \) of 48ksi and without a strength reduction factor, \( (\phi = 0.75) \). Bolt group shear capacity considering eccentric shear was calculated using Table 7.17, Coefficients C for Eccentrically Loaded Bolt Groups, of the AISC LRFD 3rd Edition Manual (AISC 2003). Entering this table with the various eccentricity assumptions, the effective number of bolts of each connection was determined. This value was then multiplied by the experimentally found nominal shear strength of a single A325-N strength bolt. Table 6.1 contains the analytical results for
each calculation method. In addition, the Astaneh predicted bolt eccentricity values are presented.
### TABLE 6.1 LRFD & OBSERVED ECCENTRICITY BOLT STRENGTH RESULTS

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Test Description</th>
<th>Obs. $e_b$ (in.)</th>
<th>Max. Load (kips)</th>
<th>Predicted Vu (kips)</th>
<th>Predicted Vu (kips)</th>
<th>Predicted Vu (kips)</th>
<th>Predicted Vu (kips)</th>
<th>Predicted Vu (kips)</th>
<th>Astaneh Pred. $e_b$ (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>1.6</td>
<td>78.8</td>
<td>63.6</td>
<td>51.4</td>
<td>37.1</td>
<td>57.5</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>2.1</td>
<td>90.7</td>
<td>63.6</td>
<td>46.3</td>
<td>37.1</td>
<td>57.5</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>(-) 1.7</td>
<td>71.8</td>
<td>63.6</td>
<td>50.3</td>
<td>37.1</td>
<td>57.5</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>(-) 2.0</td>
<td>61.4</td>
<td>63.6</td>
<td>47.3</td>
<td>37.1</td>
<td>57.5</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.1</td>
<td>75.6</td>
<td>63.6</td>
<td>63.6</td>
<td>37.1</td>
<td>57.5</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>(-) 4.4</td>
<td>44.2</td>
<td>42.4</td>
<td>13.5</td>
<td>18.7</td>
<td>25.0</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>(-) 2.4</td>
<td>45.5</td>
<td>42.4</td>
<td>22.5</td>
<td>18.7</td>
<td>27.1</td>
<td>1.67</td>
<td></td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>(-) 2.2</td>
<td>47.9</td>
<td>42.4</td>
<td>23.8</td>
<td>18.7</td>
<td>27.1</td>
<td>1.67</td>
<td></td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>5.5</td>
<td>166.5</td>
<td>197.9</td>
<td>102.0</td>
<td>128.5</td>
<td>140.4</td>
<td>1.67</td>
<td></td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.5</td>
<td>202.5</td>
<td>197.9</td>
<td>148.4</td>
<td>128.5</td>
<td>140.4</td>
<td>1.67</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1. Test stopped prior to shear rupture of connection bolts
6.2.3. EXPERIMENTALLY OBTAINED BOLT GROUP STRENGTHS

A second comparison utilizes results from the single bolt shear tests conducted by Gelo (2003). Strength calculations were made considering (1) zero eccentricity (the obtained direct shear value), (2) observed test eccentricities, and (3) limiting the test eccentricity to the bolt line to weld line distance, $a$, which was 3in for this series of tests, and (4) Astaneh predicted bolt eccentricities. These strength calculations are tabulated and presented as Table 6.2.

For direct shear calculations, the average single bolt single plane shear strength from Gelo (2003) was multiplied by the number of bolts in the connection to attain the theoretical bolt group shear strength. For strength predictions including eccentric shear considerations, the observed bolt eccentricity or the $a$-distance from each test was entered into AISC Table 7.17 (Coefficients $C$ for Eccentrically Loaded Bolt Groups). Bolt group shear strengths were then found by multiplying the effective bolt group coefficient $C$ listed in AISC Table 7.17 by the average single bolt single plane shear strength from Gelo (2003). It should also be noted that the Table 6.2 did not include a group action factor which reduces the strength by 20 percent to account for non-uniform load distributions in connections consisting of two or more bolts.
### TABLE 6.2 BOLT SHEAR TEST CAPACITY W/ ECCENTRICITY ASSUMPTIONS

(Bolt Shear Test) Bolt Group Strength (w/o Group Action)

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Test Description</th>
<th>Obs. (e_b) (in.)</th>
<th>Max. Load (kips)</th>
<th>Predicted (Vu) (kips)</th>
<th>Predicted (Vu) (kips)</th>
<th>Predicted (Vu) (kips)</th>
<th>Predicted (Vu) (kips)</th>
<th>Astaneh Pred. (e_b) (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>1.6</td>
<td>78.8</td>
<td>90.9</td>
<td>73.4</td>
<td>53.0</td>
<td>82.1</td>
<td>1.0</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>2.1</td>
<td>90.7</td>
<td>90.9</td>
<td>66.1</td>
<td>53.0</td>
<td>82.1</td>
<td>1.0</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>(-) 1.7</td>
<td>71.8</td>
<td>90.9</td>
<td>71.9</td>
<td>53.0</td>
<td>82.1</td>
<td>1.0</td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>(-) 2.0</td>
<td>61.4</td>
<td>90.9</td>
<td>67.6</td>
<td>53.0</td>
<td>82.1</td>
<td>1.0</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted,</td>
<td>0.1</td>
<td>75.6</td>
<td>90.9</td>
<td>90.9</td>
<td>53.0</td>
<td>82.1</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Simulated slab restraint</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>(-) 4.4</td>
<td>44.2</td>
<td>60.6</td>
<td>19.3</td>
<td>26.7</td>
<td>35.8</td>
<td>2.0</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>(-) 2.4</td>
<td>45.5</td>
<td>60.6</td>
<td>32.1</td>
<td>26.7</td>
<td>38.8</td>
<td>1.67</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted,</td>
<td>(-) 2.2</td>
<td>47.9</td>
<td>60.6</td>
<td>33.9</td>
<td>26.7</td>
<td>38.8</td>
<td>1.67</td>
</tr>
<tr>
<td></td>
<td>Simulated slab restraint</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>5.5</td>
<td>166.5 (^1)</td>
<td>212.1</td>
<td>145.7</td>
<td>183.6</td>
<td>200.5</td>
<td>1.67</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted,</td>
<td>0.5</td>
<td>202.5</td>
<td>212.1</td>
<td>212.1</td>
<td>183.6</td>
<td>200.5</td>
<td>1.67</td>
</tr>
<tr>
<td></td>
<td>Simulated slab restraint</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:** 1. Test stopped prior to shear rupture of connection bolts
6.2.4. LRFD BOLT GROUP CAPACITY ANALYSIS OF ASTANEH TESTS

To increase the sample size, fifteen tests conducted by Astaneh, which are summarized in Astaneh et al. (2002), were analyzed and included in the data comparison. Of the fifteen tests, ten tests failed via bolt shear rupture. A summary of these tests is presented in Table 6.3. In this table, analytical calculations for the LRFD bolt group shear strengths were made. Eccentricity assumptions included: (1) zero eccentricity \( e_b = 0 \text{in} \), (2) observed test eccentricity, (3) limiting the test eccentricity to the bolt line to weld line distance, \( a \), and (4) Astaneh predicted bolt eccentricity. For tests conducted by Astaneh, the \( a \)-distance varied as follows: for Tests A1 to A5 and A14, the \( a \)-distance was 2.75in, while for Tests A6 to A9, the \( a \)-distance was 3.125in. For this analysis, nominal single bolt single shear strengths, \( F_v \) of 48ksi and 60ksi were used for A325-N and A490-N strength bolts respectively. Bolt group shear strengths considering eccentric shear were calculated using Table 7.17, Coefficients C for Eccentrically Loaded Bolt Groups, of the AISC LRFD 3rd Edition Manual (AISC 2003). Entering this table with the various eccentricity assumptions, the effective number of bolts of each connection was determined. This value was then multiplied by the nominal single bolt single shear strength.
### TABLE 6.3 BOLT STRENGTH ADDITIONAL TEST RESULTS

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Test Description</th>
<th>Obs. $e_b$ (in.)</th>
<th>Max. Load (kips)</th>
<th>Predicted Vu $(e_b=0\text{ in.})$</th>
<th>Predicted Vu $(e_b=a)$</th>
<th>Predicted Vu $(e_b=\text{Astaneh})$</th>
<th>Astaneh Pred. $e_b$ (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1-RST-7-A325-N</td>
<td>7 Bolt, Rigid, Standard</td>
<td>3.08</td>
<td>160</td>
<td>148.4</td>
<td>127.7</td>
<td>130.7</td>
<td>125.9</td>
</tr>
<tr>
<td>A2-RST-5-A325-N</td>
<td>5 Bolt, Rigid, Standard</td>
<td>2.16</td>
<td>137</td>
<td>106.0</td>
<td>91.4</td>
<td>85.3</td>
<td>100.9</td>
</tr>
<tr>
<td>A3-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>0.95</td>
<td>94</td>
<td>63.6</td>
<td>58.0</td>
<td>39.7</td>
<td>60.0</td>
</tr>
<tr>
<td>A4-RST-5-A490-N</td>
<td>5 Bolt, Rigid, Standard</td>
<td>2.00</td>
<td>130</td>
<td>132.5</td>
<td>116.4</td>
<td>106.6</td>
<td>126.1</td>
</tr>
<tr>
<td>A5-RST-3-A490-N</td>
<td>3 Bolt, Rigid Standard</td>
<td>0.75</td>
<td>79</td>
<td>79.5</td>
<td>75.0</td>
<td>49.6</td>
<td>75.0</td>
</tr>
<tr>
<td>A6-RSS-9-A490-N</td>
<td>9 Bolt, Rigid, Short-slotted</td>
<td>2.55</td>
<td>250.8</td>
<td>238.6</td>
<td>220.7</td>
<td>215.1</td>
<td>217.7</td>
</tr>
<tr>
<td>A7-RSS-7-A490-N</td>
<td>7 Bolt, Rigid Short-slotted</td>
<td>0.83</td>
<td>189</td>
<td>185.6</td>
<td>184.8</td>
<td>159.0</td>
<td>176.9</td>
</tr>
<tr>
<td>A8-RSS-5-A490-N</td>
<td>5 Bolt, Rigid Short-slotted</td>
<td>0.48</td>
<td>151.9</td>
<td>132.5</td>
<td>132.5</td>
<td>101.7</td>
<td>132.5</td>
</tr>
<tr>
<td>A9-RSS-3-A490-N</td>
<td>3 Bolt, Rigid Short-slotted</td>
<td>(-) 0.36</td>
<td>92.1</td>
<td>79.5</td>
<td>79.5</td>
<td>45.2</td>
<td>70.2</td>
</tr>
<tr>
<td>A14-FST-4-A490-N</td>
<td>4 Bolt, Flexible, Standard</td>
<td>(-) 2.35</td>
<td>133</td>
<td>106.0</td>
<td>83.3</td>
<td>77.9</td>
<td>106.0</td>
</tr>
</tbody>
</table>

Nominal LRFD Bolt Group Strength (w/out Strength Red. Factor, $\phi$)
6.3. CONNECTION STRENGTH PREDICTIONS: VARIOUS COMPONENTS

6.3.1. SUPPORTED TEST BEAM

Of the ten full-scale tests conducted, two tests were stopped prior to failure of the connection. Test 1 (three bolt connection with rigid support condition) and Test 9 (seven bolt connection with rigid support condition) experienced lateral torsional buckling failure of the supported test beam. Test 9 developed a plastic hinge at the center span alongside failure of lateral bracing mechanisms. Tables 6.4 and 6.5 report the predicted bending strength of the supported test beams for Tests 1 and 9. From a quick design calculation according to the approximate unbraced length, a theoretical applied load for first yielding and for fully developing the beam is presented. For both test beams it should be noted that theoretical calculations predicted the failure of the supported beam within a reasonable allowance.

<table>
<thead>
<tr>
<th>TABLE 6.4 SUPPORTED TEST BEAM BENDING STRENGTH (TEST 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SUPPORTED BEAM BENDING STRENGTH</strong></td>
</tr>
<tr>
<td>(Test1A: Three-Bolt, Rigid Support, Slotted Holes)</td>
</tr>
<tr>
<td>Calculations based on Fy (ksi) = 53.6, with phi factors</td>
</tr>
<tr>
<td>Moment Capacity (kip-ft)</td>
</tr>
<tr>
<td>First Yield, $\phi M_r$</td>
</tr>
<tr>
<td>Full Plastic, $\phi M_p$</td>
</tr>
</tbody>
</table>
### TABLE 6.5 SUPPORTED TEST BEAM BENDING STRENGTH (TEST 9)

<table>
<thead>
<tr>
<th>SUPPORTED BEAM BENDING STRENGTH</th>
<th>(Test9A: Seven-Bolt, Rigid Support, Slotted Holes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculations based on Fy (ksi) = 54.1, with phi factors</td>
<td></td>
</tr>
<tr>
<td><strong>Moment Capacity</strong></td>
<td><strong>Unbraced Length</strong></td>
</tr>
<tr>
<td>(kip-ft)</td>
<td>(ft)</td>
</tr>
<tr>
<td>First Yield, $\phi M_r$</td>
<td>705</td>
</tr>
<tr>
<td>Full Plastic, $\phi M_p$</td>
<td>990</td>
</tr>
</tbody>
</table>

Other applicable limit states of the supported beam including web crippling, web shear strength, and bearing and tear out of the test beam bolt holes were not experienced in these two tests or any other conducted test. The primary reason for this is that considerations were incorporated into the design of the setup to prevent these failure modes. An example is web stiffeners being added to the test beam to prevent web crippling. In addition, the supported test beams were manufactured from higher grade steel than the single plate, so other failure modes such as bearing and tear out would occur in the single plate prior to the supported test beam.

### 6.3.2. SINGLE PLATE

Connections tested in this series of research did not experience single plate limit states as primary failure modes. Certain limit states such as shear yielding, flexural yielding, and bearing and tear out may be considered secondary failure modes. These limit states have in part been quantified in Section 5.7 and will be used in comparisons of Chapter 7. A detailed analysis of the single plate bending strength follows in Section 6.4.

### 6.3.3. WELD STRENGTH

From the ten full-scale tests conducted, weld failure was not experienced as a primary failure mode. However, in Tests 7 and 8, partial weld tearing (attributed to the
combination of shear yielding and plate bending) was observed at the top portion of the single plates. The weld strength was never experimentally measured. Therefore, nominal weld stress values must be used for calculation purposes. Tables 6.6 and 6.7 provide the theoretical applied shears to cause weld failures as based upon base metal and eccentric weld failure.

**TABLE 6.6 WELD STRENGTH COMPARISON (TEST 7)**

<table>
<thead>
<tr>
<th>WELD STRENGTH (Test7A: Two-Bolt, Flexible Support, Slotted Holes)</th>
<th>Theoretical Applied Load (kips)</th>
<th>Max. Applied Shear at Conn. (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculations based on single plate Fu (ksi) = 62.1, with phi factors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Metal Failure (Combined effect of shear and moment on Weldment)</td>
<td>38.8</td>
<td>45.5</td>
</tr>
<tr>
<td>Weld Strength ($e_w = a = 3\text{in}$)</td>
<td>51.6</td>
<td></td>
</tr>
<tr>
<td>Weld Strength ($e_w = a + e_b = 0.6\text{in}$)</td>
<td>67.0</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 6.7 WELD STRENGTH COMPARISON (TEST 8)**

<table>
<thead>
<tr>
<th>WELD STRENGTH (Test8A: Two-Bolt, Flexible Support, Slotted Holes, Slab Restraint)</th>
<th>Theoretical Applied Load (kips)</th>
<th>Max. Applied Shear at Conn. (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculations based on single plate Fu (ksi) = 62.1, with phi factors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Metal Failure (Combined effect of shear and moment on Weldment)</td>
<td>38.8</td>
<td>47.9</td>
</tr>
<tr>
<td>Weld Strength ($e_w = a = 3\text{in}$)</td>
<td>51.6</td>
<td></td>
</tr>
<tr>
<td>Weld Strength ($e_w = a + e_b = 0.8\text{in}$)</td>
<td>66.3</td>
<td></td>
</tr>
</tbody>
</table>

Weld eccentricity considerations include both the weld line to bolt line distance, $a$, and the combination of the $a$-distance and the experimental test eccentricity. In addition, the ultimate sustained shears at the connections are reported within the two tables. As
seen in the two Tables, the base metal calculation conservatively estimates the maximum applied shear at the connections.

6.4. SINGLE PLATE BENDING STRENGTH

6.4.1. OVERVIEW

The single plate, which acts as a short cantilevered beam, is subjected to bending as due to the rotation of the supported beam and the eccentric shear. The supported beam imposes the rotation through the connection bolts upon the single plate. Figures 6.1 and 6.2 illustrate the combination of forces which induce plate bending otherwise referred to as flexural yielding. Figure 6.1 depicts the typical flexural yielding behavior for a test with a rigid support condition while Figure 6.2 depicts the typical flexural yielding behavior for a test with a flexible support condition.

![Figure 6.1 Forces Inducing Plate Bending (Typ. Rigid Support)](image-url)

FIGURE 6.1 FORCES INDUCING PLATE BENDING (TYP. RIGID SUPPORT)
The main difference between Figures 6.1 and 6.2 is that the moment due to the beam end rotation, $M_1$, is dependent upon the support condition. For a flexible support condition, $M_1$ is a result of beam end rotation and/or girder rotation. A counterclockwise moment is induced upon the single plate when a flexible support condition is present. However, a clockwise moment is induced upon the single plate when a rigid support condition is present. The moment created by the eccentrically applied shear, $M_2$, is assumed to always create a clockwise or positive moment. Combining the moment induced by the beam end rotation and the eccentric shear, a resultant moment, $M_T$, is found. When a flexible support condition is present, the magnitudes of $M_1$ and $M_2$, could cause the resulting moment, $M_T$, to be either counterclockwise (negative) or clockwise (positive). The phenomenon can cause the theoretically calculated plate bending strength to be overly conservative.

An investigation of the bending strength behavior of the single plate was conducted utilizing experimentally measured strains and solid mechanics. The goal of the investigation was to determine if the single plate exhibited plastic or elastic flexural behavior. The plate bending strength was calculated using the following expressions:
\[ \phi V_n = \phi \left( \frac{M_n}{a + e_b} \right) \]  

(EQ 6.1)

where:

\[ \phi = 0.90 \]

\[ a = \text{distance between the weld line and bolt line} \]

\[ e_b = \text{bolt eccentricity} \]

\[ \phi V_n = \text{design bending strength of the single plate} \]

\[ M_n = \text{maximum sustainable moment of the single plate} \]

moment, which is defined as follows:

\[ M_n = F_y S_x, \text{ -or- } F_y Z_x \]  

(EQ 6.2)

where:

\[ F_y = \text{yield stress of the single plate} \]

-For elastic design considerations:

\[ S_x = \text{elastic section modulus of the single plate, which is defined as follows:} \]

\[ S_x = \frac{t_p L_p^2}{6} \]  

(EQ 6.3)

-For plastic design considerations,

\[ Z_x = \text{plastic section modulus of the single plate, which is defined as follows:} \]

\[ Z_x = \frac{t_p L_p^2}{4} \]  

(EQ 6.4)

where:

\[ L_p = \text{length of the single plate} \]

\[ t_p = \text{thickness of the single plate} \]

As previously described, each single plate was instrumented with uniaxial strain gages to observe the stress distribution through the length of the plate. The strain data
was utilized to generate stress distribution plots at various levels of applied load. Detailed stress distribution results for each test can be found in the corresponding Appendices B through K.

6.4.2. SINGLE PLATE STRESS DISTRIBUTIONS

The following analysis utilizes the readings from the single plate strain gages to determine if the single plates behaved plastically or elastically. Of the analyzed tests, Test 1 was chosen due to its near theoretical behavior. Figure 6.3 illustrates the stress distribution throughout the single plate for Test 1. Each data point on the graph corresponds to a particular strain gage location on the single plate. It should be noted that the stress distributions as seen in Figure 6.3 are limited to the yield stress, $F_y$, of the single plate material.

![Figure 6.3 Single Plate Bending Stress Distribution (Test 1)](image)

**FIGURE 6.3 SINGLE PLATE BENDING STRESS DISTRIBUTION (TEST 1)**
From the test results, the following trends and observations were noted:

- Each test was found to behave differently as there was no “typical” behavior.

- The single plate stress distributions were found to be nonlinear throughout the depth of the single plate at each level of applied load.

- The stress distributions varied such that tension and compression regions of the plate were unevenly distributed.

- Strain gage readings indicated that some single plates did not yield to the same extent as Test 1.

- Stress distributions indicated that when the applied shear force to the connection was at lower applied shears, single plates behaved elastically. Only at higher applied shears did the extreme strain gages indicate that the steel had yielded. At the highest applied shear level, significant yielding through the cross section was observed indicating plastic behavior.

For Test 1, shown in Figure 6.3, intuitive behavior was observed. Of all the tests, the three bolt tests with a rigid support condition behaved near theoretically, allowing for direct correlation between the test results and theoretical calculations. However, this behavior was not typical for each the flexible supported connection. Foremost, it should be noted that the applied load for a flexible supported connection was considerably less than a rigid supported test. Comparing to a similar sized connection with a flexible support condition, only the upper portion of the single plate indicated yielding. Figure 6.4 depicts this behavior.
FIGURE 6.4 SINGLE PLATE BENDING STRESS DISTRIBUTION (TEST 3)

At ultimate loads for the flexible tests consisting of only two bolts, results indicated that only the bottom extreme edge of the single plate had only just begun to yield. Imposing the simulated slab restraint again altered the single plate stress distribution. Figure 6.5 depicts Test 8 (two bolt connection with flexible support condition and simulated slab restraint). Notably in this graph at ultimate load, extensive tensile forces exist below the plate neutral axis.
6.4.3. PLASTIC VS ELASTIC BEHAVIOR

As previously discussed, single plates were found to behave either elastically or plastically at various applied loads. Incorporating both plastic and elastic considerations into theoretical calculations, this analytical investigation sought to determine the best means of predicting the flexural yielding strength of single plates used in single plate framing connections. Due to the theoretical behavior that Test 1 (three bolt connection with rigid support condition and short-slotted holes) exhibited, this test was chosen as a basis for comparison between experimental and theoretical results. To create a greater sample size and include a connection with a different test configuration, a secondary comparison is presented for Test 8 (two bolt connection with flexible support condition, short-slotted holes, and simulated slab restraint). Combined within Tables 6.8 and 6.9 are results found from single plate stress distribution plots (Section 6.4.2), results from
approximated flexural yielding strengths (Section 5.7), and theoretical calculations. As indicated by the single plate strain gage data, the results of each test are typical of other tests with similar test configurations (such as number of bolts and support condition).

### TABLE 6.8 SINGLE PLATE FLEXURAL YIELDING SUMMARY (TEST 1)

<table>
<thead>
<tr>
<th>Appl. Shear vs. Rotation Plot</th>
<th>Single Plate Strain Gage Readings</th>
<th>Calculated Code Value ( (e_{pl} = a) )</th>
<th>Test Eccentricity ( (e_b = (+)1.6\text{in}) )</th>
<th>Astaneh Prediction ( (e_b = (+)1\text{in}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>74.5 (^6)</td>
<td>35.0 (^4)</td>
<td>66.8 (^2)</td>
<td>43.6 (^2)</td>
<td>50.1 (^2)</td>
</tr>
<tr>
<td></td>
<td>62.2 (^5)</td>
<td>100.2 (^3)</td>
<td>65.4 (^3)</td>
<td>75.2 (^3)</td>
</tr>
</tbody>
</table>

**Notes:**
1. Calculations based on \( F_y \) (ksi) = 39.6, without strength reduction factor
2. Based upon Elastic Section Modulus, \( S_x \)
3. Based upon Plastic Section Modulus, \( Z_x \)
4. (From Figure 6.6) Approximately when the extents of the single plate began to yield
5. Approximate applied shear when all single plate strain gages indicated yielding
6. (From Table 5.3) Approximate applied shear where flexural yielding ensued

### TABLE 6.9 SINGLE PLATE FLEXURAL YIELDING SUMMARY (TEST 8)

<table>
<thead>
<tr>
<th>Appl. Shear vs. Rotation Plot</th>
<th>Single Plate Strain Gage Readings</th>
<th>Calculated Code Value ( (e_{pl} = a) )</th>
<th>Eccentricity from Test ( (e_b = (-)2.2\text{in}) )</th>
<th>Astaneh Prediction ( (e_b = (-)1.67\text{in}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>44.7 (^5)</td>
<td>46.0 (^4)</td>
<td>29.7 (^2)</td>
<td>111.4 (^2)</td>
<td>60.29 (^2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>44.6 (^2)</td>
<td>167.1 (^3)</td>
<td>100.5 (^3)</td>
</tr>
</tbody>
</table>

**Notes:**
1. Calculations based on \( F_y \) (ksi) = 39.6, without strength reduction factor
2. Based upon Elastic Section Modulus, \( S_x \)
3. Based upon Plastic Section Modulus, \( Z_x \)
4. Approximate applied shear when bottommost single plate strain gage indicated yielding
5. (From Table 5.3) Approximate applied shear where flexural yielding ensued
The applied load necessary to induce single plate flexural yielding as found from the single plate strain gage readings was determined in the following manner. Stress distribution plots generated from single plate strain gage readings were analyzed to determine the load at which the onset of flexural yielding occurred. Figure 6.6 illustrates the single plate bending stress distribution for Test 1. Superimposed on this graph are lines indicating the yield strength of the plate steel. The intersection of the yield stress lines and the outer edge of the single plate indicates when the single plate first begins to yield. As seen in Figure 6.6, the approximation for where the outer edges of the single plate began to yield can be seen as an extension of the 35kip data set. Therefore the experimentally measured elastic flexural yielding strength is approximated as 35kips.

![Figure 6.6: Single Plate Yielding Behavior (Test 1)](image)

**FIGURE 6.6 SINGLE PLATE YIELDING BEHAVIOR (TEST 1)**
Alternatively, at higher sustained applied shears, the single plate behaves plastically. In an attempt to determine when the majority of the single plate had yielded, the strains for the inner two single plate strain gages were analyzed to determine the specific applied shear at which yielding occurred. Averaging the applied shear load values determined for the two interior strain gages of the single plate, an applied shear of 62.2kips was found to cause yielding at all four strain gages along the length of the plate. At this loading, the single plate is experimentally thought to behave plastically.

In Tables 6.8 and 6.9, a supplemental experimental predicted value is presented for predicting the single plate flexural yielding strength. The applied shear as found from the Applied Shear vs. Rotation plots reported in Section 5.7 is presented. As indicated from Table 5.3 (which is based upon applied shear at the connection versus rotation plot data), there was no indicated applied shear at which flexural yielding was thought to occur. However, as reported in Table 5.3 for Test 7 (two bolt connection with flexible support condition and short-slotted holes), an approximated flexural yielding strength of 44.7kips was reported and is listed in Table 6.9.

In addition, predicted applied shears were calculated based upon the theoretical flexural strength of the single plate using Equations 6.1 to 6.4. In each calculation the bolt line to weld line distance, $a$, was incorporated. In addition to analyzing for the $a$-distance only, the $a$-distance was combined with various bolt eccentricity assumptions such as the experimentally measured bolt eccentricity and Astaneh predicted bolt eccentricity. It should be noted that the Astaneh predicted bolt eccentricity values were calculated as follows: the rigid supported test eccentricity was calculated typically while the absolute value operator and $a$-distance limitation was not applied to the flexible
supported equation. The purpose of which was to determine which eccentricity assumption most accurately predicted the flexural yielding strength of the single plate.

6.5. ROTATIONAL BEHAVIOR

6.5.1. GENERAL

Rotational behavior of single plate framing connections is not widely understood. The various parameters used in testing potentially affect the location of the center of rotation. Due to the rotational characteristics of the connection, capacities and behavior can be adversely impacted. In an attempt to establish the rotational behavior of the connection, two methods for identifying the rotational characteristics were analyzed. One method relies on linear potentiometers which measured the beam end rotation of the experimental tests. The second method identifies the rotational characteristics of the connection from the single plate stress distribution plots created from the single plate strain gages. Discussed within this section are the two analytical methods used in determining rotational behavior and the effect which the test parameters had upon the rotational behavior.

6.5.2. SINGLE PLATE STRESS DISTRIBUTION ROTATIONAL CHARACTERISTICS

Using single plate stress distribution plots, an investigation of the rotational behavior of the connection was conducted. From these plots, a determination as to the location of the rotational neutral axis of the connection in relation to the plate neutral axis could be determined. Figures 6.7 and 6.8 illustrate the single plate stress distributions as measured by the single plate strain gages of Tests 1 and 9 respectively.
FIGURE 6.7 BENDING STRESS ROTATIONAL BEHAVIOR (TEST 1)

FIGURE 6.8 BENDING STRESS ROTATIONAL BEHAVIOR (TEST 9)
From the single plate stress distribution graphs, the rotational neutral axis is determined by noting the vertical location where the datasets cross the zero stress vertical axis. In Test 1 (illustrated as Figure 6.7), the rotation reasonably occurs about the neutral axis of the single plate. However, for Test 9 (illustrated as Figure 6.8), the rotational neutral axis appears to be located above the neutral axis of the single plate (as the three bottommost single plate strain gages indicated compression throughout the test). From Figure 6.9, the rotational neutral axis is predicted to be from 5in to 7in above the single plate neutral axis.

Of the analyzed tests, stress distribution graphs indicated distinct results for rigidly supported tests. Rotational behavior of tests with a flexible support condition was not as clearly defined as the supporting girder alters the rotational characteristics of the connection. For tests with a flexible support condition as the applied load is increased, the bottom portion of the single plate begins to develop near zero or even tensile stresses. In the following subsection, Table 6.10 summarizes the rotational neutral axis locations as found from single plate stress distribution analyses.

6.5.3. LINEAR POTENTIOMETER ROTATIONAL CHARACTERISTICS

To verify the location of the center of rotation, linear potentiometers measuring the horizontal displacement at the top and bottom flanges of the supported test beam was analyzed at various applied shears at the connection. These linear potentiometers are depicted as ‘Δ Top Beam’ and ‘Δ Bot. Beam’ in Figures 4.30 and 4.31 for tests with a rigid support condition and Figures 4.34 to 4.36 for tests with a flexible support condition. A sample size of four applied shears at the connections was analyzed for each full-scale test. With the knowledge of both the distance from the beam neutral axis to
each potentiometer and the horizontal displacement measurements recorded from testing, linear interpolation of the data was used to determine the location of the center of rotation. The center of rotation was assumed to be located along the bolt line. In addition to this data, the center of rotation as approximated from the single plate stress distribution analyses from Section 6.5.2 is presented. Table 6.10 consists of the data from both methods of analysis and reports the location of the center of rotation in relation to the bolt group neutral axis.

**TABLE 6.10 CENTER OF ROTATION OF CONNECTION**

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Description</th>
<th>Center of Rotation Location in Relation to Bolt Group N.A. ¹ Linear Potentiometers (in.)</th>
<th>Center of Rotation Location in Relation to Bolt Group N.A. ² Strain Gages (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>(-) 0.2</td>
<td>0.0</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>1.0</td>
<td>(-) 1.3</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>(-) 2.6</td>
<td>(-) 1.5</td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>(+) 2.2</td>
<td>-----</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>4.9</td>
<td>3.5</td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>(-) 1.7</td>
<td>-----</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>(-) 0.8</td>
<td>-----</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>(-) 1.1</td>
<td>-----</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>(-) 0.7</td>
<td>5 to 7</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>15.1</td>
<td>-----</td>
</tr>
</tbody>
</table>

**Notes:**
1. Center of rotation location is the average location of rotation of the relative beam end rotation from linear potentiometer readings at the top and bottom flanges of the supported test beam taken at four random applied shears at the connection
2. Center of rotation location is an approximation of the trend of single plate strain gage behavior plots found in Appendices B-K

From Table 6.10, the most reliable data is that which was measured from linear potentiometers. Further analysis of the data indicated the following rotational trends for
various parameters used in testing. Figure 6.9 depicts three bolt single plate connections as used in testing with various support conditions. Illustrated in this figure for each support condition is the location of the center of rotation in relation to the bolt group neutral axis.

Analyses indicated that for connections with any number of bolts and a rigid support condition, the average location of the center of rotation of the supported test beam approximately coincides with the bolt group neutral axis (Figure 6.9 (a)). For tests with a flexible support condition without a simulated slab restraint, the location of the center of rotation was found to be on average 1.7in below the bolt group neutral axis (Figure 6.9 (b)). For tests with a flexible support condition and a simulated slab restraint, results varied per size of the connection. As seen in Table 6.10, the location of the center of
rotation was above the bolt group neutral axis for the three and seven bolt tests while for the two bolt test the center of rotation was found to be below the bolt group neutral axis. Figure 6.9 (c) illustrates that the center of rotation is shifted toward to the simulated slab restraint for a three bolt connection.

However, potential sources of error for rotational neutral axis calculations based upon linear potentiometer readings for tests with a flexible support condition and without a simulated slab restraint may be as follows: (1) the girder allows for rigid body translation of the supported test beam toward the supporting wall and (2) the girder rotates as the load is transmitted to the single plate. For tests with a rigid support condition, the column acts rigidly and does not allow translation of the test beam. Therefore, the rotational results for tests with a rigid support condition are the most accurate.

6.6. SUMMARY

From the analytical investigation, several studies were conducted. The primary focus of the analytical investigations was to analyze the connection components based upon the corresponding limit states. Particular focus was given to the connection bolts, which was the critical component failing via shear rupture. In addition to the connection bolts, other test components such as the supported beam and weldment were analyzed. Analytical investigation incorporated various design considerations, such as eccentricity assumptions for connection bolts.

From experimental testing, the behavior of the single plate was investigated to determine whether elastic or plastic assumptions best predicted experimental results as
recorded by the single plate strain gages. From the analytical investigation, the behavior of the single plate was found to act elastically at lesser loads and approach plastic behavior at loads near ultimate capacity. Because rigid supported tests sustain greater applied shears, single plates were found to undergo extensive plastic behavior.

The rotational behavior of single plate connections was found to vary per connection tested. Tests with a rigid support condition were found to rotate approximately about the neutral axis of the single plate. Flexible supported tests were found to exhibit complex rotational behavior as the supporting girder applies additional rotation to the connection. Results indicated that the rotational neutral axis for tests with a flexible support condition was located below the single plate/bolt group neutral axis. Finally, tests with a flexible support condition which incorporated a simulated slab restraint were found to have the center of rotation shifted toward the simulated slab restraint.
CHAPTER 7 COMPARISONS

7.1. OVERVIEW

The primary failure mode of the ten full-scale tests conducted as a part of this research was shear rupture of the connection bolts. Within this chapter, comparisons between the ultimate strength of the ten full-scale tests and theoretically calculated bolt group shear strength capacities are made. Comparisons are based upon analyses which incorporated various bolt eccentricity considerations and utilized an experimentally measured single bolt single shear capacity as determined from supplemental bolt shear tests. In addition, comparisons of the analytical investigation of the flexural yielding behavior of the single plate will be presented. From this comparison, the focus was to identify the best means for predicting the behavior and flexural yielding capacity of the single plate.

A major focus of this research was the investigation of additional design methods related the design of single plate framing connections. As a basis for comparison, Chapter 3 Design Methods detailed the currently accepted single plate design method specified in the AISC LRFD 3rd Edition Manual (AISC 2003). Differences between the various design methods as specified in other countries including Australia, Canada, Great Britain, and New Zealand and as recommended by researchers in the United States were identified. Using experimentally measured material properties of the full-scale tests conducted, the design strength for each limit state specified in the various design methods was calculated. In addition, comparisons to observed experimental limit states will be made to determine which design method most accurately predicts the behavior of the tested connections.
A supplemental investigation is provided for the bolt eccentricity equations currently accepted in the AISC LRFD 3rd Edition Manual (AISC 2003). Experimentally measured bolt eccentricities, as reported from the ten full-scale tests conducted in this series of research in addition to the full-scale tests conducted by Astaneh et al., were incorporated into a comparison to determine the adequacy of the presently accepted eccentricity equations.

7.2. COMPARISON OF BOLT SHEAR RUPTURE STRENGTHS

7.2.1. OVERVIEW

Comparisons of the experimental and analytical connection bolt shear rupture strengths are presented herein. The analytical investigations contained in the subsections of Section 6.2 will serve as the basis for comparison. Comparisons will be presented as a ratio of the analytical predicted value to the experimentally observed value. Table 7.1 is a presentation of all the comparisons conducted in Section 6.2. The table contains comparisons which are based upon both the tests conducted in this series of research and the tests conducted by Astaneh et al. The table is reorganized such that connection tests with similar number of bolts and similar support condition are grouped together. An analytically predicted value is considered conservative when it is less than unity ($\leq 1.0$) and unconservative when greater than unity ($\geq 1.0$). The comparison table also includes statistical data, such as the maximum- minimum range, mean, and standard deviation. Of the comparisons presented, the most pertinent results and trends will be discussed.
TABLE 7.1 COMPARISON OF BOLT SHEAR RUPTURE STRENGTHS

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Max. Load (kips)</th>
<th>$e_b$ (in.)</th>
<th>$e_b$ = 0 in without</th>
<th>$e_b$ = $a$ without</th>
<th>$e_b$ = 0 in without</th>
<th>$e_b$ = $a$ without</th>
<th>(Astaneh $e_b$) without</th>
<th>(Test $e_b$)</th>
<th>(Test $e_b$)</th>
<th>(Astaneh $e_b$)</th>
<th>(Test $e_b$)</th>
<th>(Astaneh $e_b$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S6-FST-2-A325-N</td>
<td>44.2</td>
<td>-4.40</td>
<td>0.96</td>
<td>0.31</td>
<td>0.42</td>
<td>0.57</td>
<td>1.37</td>
<td>0.44</td>
<td>0.60</td>
<td>0.81</td>
<td>0.45</td>
<td>1.10</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>45.5</td>
<td>-2.40</td>
<td>0.93</td>
<td>0.49</td>
<td>0.41</td>
<td>0.60</td>
<td>1.33</td>
<td>0.71</td>
<td>0.59</td>
<td>0.85</td>
<td>0.70</td>
<td>1.07</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>47.9</td>
<td>-2.20</td>
<td>0.88</td>
<td>0.50</td>
<td>0.39</td>
<td>0.57</td>
<td>1.26</td>
<td>0.71</td>
<td>0.56</td>
<td>0.81</td>
<td>0.76</td>
<td>1.01</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>71.8</td>
<td>-1.70</td>
<td>0.89</td>
<td>0.70</td>
<td>0.52</td>
<td>0.80</td>
<td>1.27</td>
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<td>1.14</td>
<td>0.99</td>
<td>1.01</td>
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<tr>
<td>S4-FST-3-A325-N</td>
<td>61.4</td>
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<td>0.94</td>
<td>1.48</td>
<td>1.10</td>
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<td>1.34</td>
<td>0.50</td>
<td>1.18</td>
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<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>75.6</td>
<td>0.10</td>
<td>0.84</td>
<td>0.84</td>
<td>0.49</td>
<td>0.76</td>
<td>1.20</td>
<td>1.20</td>
<td>0.70</td>
<td>1.09</td>
<td>0.60</td>
<td>1.01</td>
</tr>
<tr>
<td>A10-FST-4-A490-N</td>
<td>193.0</td>
<td>3.25</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>1.00</td>
<td>1.10</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>A12-FST-4-A490-N</td>
<td>122.0</td>
<td>-1.05</td>
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<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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<tr>
<td>A14-FST-4-A490-N</td>
<td>133.0</td>
<td>-2.35</td>
<td>0.50</td>
<td>0.63</td>
<td>0.59</td>
<td>0.80</td>
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<td>-----</td>
<td>0.11</td>
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</tr>
<tr>
<td>A15-FST-4-A490-N</td>
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<td>1.45</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
</tr>
<tr>
<td>A11-FST-6-A490-N</td>
<td>148.0</td>
<td>2.05</td>
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<td>A13-FST-6-A490-N</td>
<td>180.0</td>
<td>0.15</td>
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<td>-----</td>
<td>-----</td>
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<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>202.5</td>
<td>0.50</td>
<td>0.98</td>
<td>0.73</td>
<td>0.63</td>
<td>0.69</td>
<td>1.05</td>
<td>1.05</td>
<td>0.91</td>
<td>0.99</td>
<td>3.34</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Note: This table is continued on the following page.
### TABLE 7.1 (CONTINUED)

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Max. Load (kips)</th>
<th>(e_b) (in.)</th>
<th>((e_b = 0)) without (\phi)</th>
<th>((e_b = a)) without (\phi)</th>
<th>(Astanich (e_b)) without (\phi)</th>
<th>((e_b = 0))</th>
<th>((e_b = a))</th>
<th>(Astanich (e_b)/Test (e_b))</th>
<th>((e_b = 0))</th>
<th>((e_b = a))</th>
<th>(Astanich (e_b))</th>
</tr>
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<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>78.8</td>
<td>1.60</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>0.63</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>90.7</td>
<td>2.10</td>
<td>0.70</td>
<td>0.51</td>
<td>0.41</td>
<td>0.63</td>
<td>1.00</td>
<td>0.73</td>
<td>0.58</td>
<td>0.91</td>
<td>0.48</td>
</tr>
<tr>
<td>A3-RST-3-A325-N</td>
<td>94.0</td>
<td>0.95</td>
<td>0.68</td>
<td>0.62</td>
<td>0.42</td>
<td>0.64</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>A5-RST-3-A490-N</td>
<td>79.0</td>
<td>0.75</td>
<td>1.01</td>
<td>0.95</td>
<td>0.63</td>
<td>0.95</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>A9-RSS-3-A490-N</td>
<td>92.1</td>
<td>-0.36</td>
<td>0.86</td>
<td>0.86</td>
<td>0.49</td>
<td>0.76</td>
<td>-----</td>
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<tr>
<td>A2-RST-5-A325-N</td>
<td>137.0</td>
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<td>-----</td>
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</tr>
<tr>
<td>A4-RST-5-A490-N</td>
<td>130.0</td>
<td>2.00</td>
<td>1.02</td>
<td>0.90</td>
<td>0.82</td>
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<tr>
<td>A6-RSS-5-A490-N</td>
<td>151.9</td>
<td>0.48</td>
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<td>0.87</td>
<td>0.67</td>
<td>0.87</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>166.5</td>
<td>5.50</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>A1-RST-7-A325-N</td>
<td>160.0</td>
<td>3.08</td>
<td>0.93</td>
<td>0.80</td>
<td>0.82</td>
<td>0.79</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>A7-RSS-7-A490-N</td>
<td>189.0</td>
<td>0.83</td>
<td>0.98</td>
<td>0.98</td>
<td>0.84</td>
<td>0.94</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>A6-RSS-9-A490-N</td>
<td>250.8</td>
<td>2.55</td>
<td>0.95</td>
<td>0.88</td>
<td>0.86</td>
<td>0.87</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
</tbody>
</table>

### Notes:
1. Test stopped prior to shear rupture of connection bolts
2. Ratios \( \leq 1.0 \) are conservative
3. Ratios > 1.0 are unconservative

### Average Ratios:
- Max. Load: 1.04, 0.98, 0.86, 0.97, 1.48, 1.20, 0.91, 1.34, 1.86, 1.18, 0.96, 0.73, 1.07
- Min: 0.68, 0.31, 0.39, 0.57, 1.00, 0.44, 0.56, 0.81, 0.11, 0.80, 0.35, 0.45, 0.65
- Average: 0.89, 0.72, 0.59, 0.77, 1.25, 0.87, 0.69, 0.99, 0.70, 1.00, 0.69, 0.55, 0.79
- St. Deviation: 0.10, 0.19, 0.16, 0.13, 0.16, 0.26, 0.13, 0.19, 0.40, 0.13, 0.21, 0.11, 0.15

Indicates the best conservative ratio of the analytical calculation method
The focus of this portion of research was to identify which means of calculating the bolt group shear strength of single plate framing connections is most appropriate. Multiple methods were used to determine which means of calculation best correlated with experimentally obtained bolt group shear strengths of the tested connections. From Section 6.2, nominal LRFD bolt group strength calculations and calculations utilizing measured bolt group shear strengths were used in the analytical investigation. The following considerations were analyzed: (1) incorporating a group action factor, which reduces the bolt group shear strength by 20 percent to account for non-uniform load distributions in connections consisting of two or more bolts and (2) applying various eccentric shear considerations to the bolt group.

The first consideration was to determine the necessity of incorporating a group action factor. As discussed, the group action factor reduces the bolt group shear strength by 20 percent to account for non-uniform load distributions in connections consisting of two or more bolts. Although standard practice is to incorporate this reduction factor, the question raised in this investigation is the necessity of the group action factor for connections with two or three connection bolts. Load distributions in connections consisting of two and three bolts are potentially more evenly distributed than a connection consisting of seven bolts.

The second consideration is of particular focus. Test results indicated that connection bolts experienced eccentric shear. As such, the question was raised to determine to what extent does eccentric shear considerations need to be applied to bolt group strength calculations. The particular concern is that a large eccentricity applied to
a small number of connection bolts is more significant than a small eccentricity or eccentricity of similar size applied to a connection with many connection bolts.

Within Table 7.1, comparisons are based upon: (1) Nominal LRFD bolt group shear strengths (calculated without a strength reduction factor, ($\phi = 0.75$)) and (2) bolt group capacities based upon single bolt single shear tests conducted by Gelo (2003) calculated with and without the use of a group action factor. Both baselines for comparisons incorporated the following eccentric shear assumptions: direct shear (zero eccentricity), observed test eccentricity, eccentricity limited to the bolt line to weld line distance, $a$, and Astaneh predicted bolt eccentricity. Nominal LRFD bolt group strength analyses utilized a nominal single bolt single shear strength of 48ksi and 60ksi for the A325-N and A490-N bolts respectively.

7.2.2. OBSERVATIONS

Within Table 7.1, the various means of calculation (e.g. Nominal LRFD Bolt Strength, Bolt Shear Test w/o Group Action, and Bolt Shear Test w/ Group Action) were evaluated to determine which considerations (including eccentricity and the inclusion of the group action factor) yielded ratios of unity or conservative ratios of less than unity. The best conservative ratios were identified for each analyzed test with gray shading. The following will summarize the major trends illustrated in Table 7.1.

Nominal LRFD Bolt Strength

Indicated in Table 7.1, the LRFD calculated bolt strength (without eccentric shear considerations did a reasonable job for predicting the bolt group shear capacities, however, some tests reported unconservative ratios. As such, eccentric shear
considerations should be taken into consideration. From the various eccentricity considerations evaluated, the following trends were noted: the next best conservative ratio was found to be the Astaneh predicted bolt eccentricity for both the flexible and rigid supported tests. For the ratios which consider the Astaneh predicted bolt eccentricity, ratios ranged from 0.57 to 0.97 with an average of 0.77. For flexible supported connections with only two bolts, results indicated the magnitude of the eccentricity appears to have a significant effect on the predicted strength. For the flexible supported tests with more than two bolts, ratios based upon the Astaneh predicted or observed test eccentricity were nearer to unity.

From Table 7.1, the majority of the non-eccentric nominal LRFD bolt shear strength ratios had a conservative average, predicting strengths less than observed maximum applied test connection shears. Applying the various eccentricity assumptions increased the conservative nature of the prediction. It should be noted that the ratios for the rigid supported tests were nearer to unity. For rigid supported tests with more than three connection bolts, ratios, which did not consider eccentric shear, were found to accurately predict the observed maximum sustained shear of the connections tested. However, some of the zero eccentricity ratios were unconservative. Therefore, some extent of eccentric shear should be taken into consideration. Observed test eccentricity results appeared to yield the next most accurate results.

Because many design methods from the investigational study of Chapter 3 indicated that the test eccentricity was limited to the bolt line to weld line distance, \( a \), this eccentricity consideration was evaluated. For the nominal LRFD bolt strength
calculations, this eccentricity assumption yielded the most conservative results and consequently the least accurate results.

**Bolt Shear Test (with and without Group Action Factor)**

For the bolt shear test ratios, only the tests conducted in this series of research were evaluated because the shear strength of the bolts used in testing were experimentally measured. Bolt group shear strengths calculated without eccentric shear considerations were found to have an average ratio of 1.25 indicating a nonconservative prediction by approximately 25 percent. As discussed, the analytical data used for comparison purposes did not include a 20 percent reduction in strength to account for non-uniform load distributions in connections consisting of two or more bolts. When the group action factor was applied, ratios decreased to an average value of unity, which is desired, yet, ratios still ranged from 0.80 to 1.18 indicating that certain ratios remained unconservative. Therefore, the inclusion of the group action factor appears to be beneficial and the unconservative values indicate that eccentric shear considerations could compensate for any unconservative values. However, as seen in Table 7.1 for Test 2 (three bolt connection with a rigid support condition), a predicted-to-observed strength ratio of unity was reported. It should be noted that this ratio did not take into account eccentric shear considerations or the bolt group action factor. Even though this is only one test to support this finding, this indicates that eccentric shear considerations and the group action factor is unnecessary for single plate framing connections with a rigid support condition consisting of three bolts.

Contradictory to the results of Test 2, conventional practice incorporates the group action factor in calculations for connections consisting of two or more bolts. Similar to
the nominal LRFD bolt strength results, the bolt shear test with group action factor results indicated that eccentricity assumptions should be applied to strength calculations. Comparing the observed test, $a$-distance limited, and Astaneh predicted bolt eccentricity for the tests conducted in this series of research indicated the following results:

- For connections with three or more bolts, the most accurate yet conservative ratios were found to be based upon the observed test eccentricity. Thus, this indicates that the inclusion of the group action factor along with eccentric shear considerations is desired.

- Contradictory to the preceding finding, bolt shear test with the group action factor ratios for the two bolt flexible supported tests were overly conservative. As such, the incorporation of the group action factor with two bolt connections appears to be unnecessary. Without the group action factor, the two bolt flexible supported tests with the Astaneh predicted eccentricity considerations yielded conservative results nearest to unity.

Although the Astaneh predicted eccentricity considerations were found to be the most accurate for use with the two bolt flexible supported connections, this was not the case for tests with more connection bolts. For tests conducted in this series of research, one potential explanation for this trend is indicated by the average ratio of the Astaneh predicted eccentricity to the observed test eccentricity. After omitting the outlying ratios of Tests 5 and 10, the predicted Astaneh bolt eccentricities are noticeably less than the observed test eccentricities. The average ratio being 0.55 indicates that the average Astaneh prediction is only 55 percent of the observed value.

Comparisons indicated that when eccentricity is a consideration of bolt group shear strength, a conservative eccentricity estimate could favorably or adversely affect bolt
group shear strength predictions. The following examples indicating this trend are taken from the bolt shear test without group action factor portion of comparisons. Looking at Test 3, the reported ratio with Astaneh bolt eccentricity is 1.14, while the ratio based upon observed test bolt eccentricity is unity. In contrast, for a two bolt test such as Test 7 with Astaneh predicted bolt eccentricity, a ratio of 0.85 was reported, while a ratio of 0.70 was reported for the same test with observed test bolt eccentricity. Notably, the Astaneh eccentricity prediction calculates eccentricity less than observed test eccentricities for two bolt connection tests with a flexible support condition. For the second example, the conservative Astaneh predicted eccentricity estimate predicts the ultimate strength sustained by the connection with better accuracy. As such, the effective number of bolts as calculated with the lesser predicted eccentricity correlates better with the observed ultimate shear sustained at the connection. The opposite is true for three bolt connections with a flexible support condition as an underestimated bolt eccentricity adversely affects the reported strength.

7.2.3. SUMMARY

From the comparison ratios of Table 7.1 and investigation of trends as described in previous subsections, the following conclusions and recommendations are made for calculating the bolt group shear strength of single plate connections.

- When material properties are unknown, the strength reduction phi factor is essential in bolt group shear strength calculations.

- Results indicated that the group action factor appears to be necessary for single plate framing connections of either a rigid or flexible support condition consisting of three or more connection bolts.
The following conclusions and recommendations were found for eccentric shear considerations:


- For tests with a rigid support condition, eccentric shear considerations appear to be unnecessary for connections consisting of more than three connection bolts. Any recommendation for connections with more than three bolts utilizing a flexible support condition cannot be made as not enough experimental data exists.

- Ratios indicated that the nominal LRFD calculated bolt group shear strength with Astaneh predicted bolt eccentricities (based upon Equations 3.24 to 3.27 applying the Astaneh predicted bolt eccentricity values to Table 7.17 in the AISC LRFD 3rd Edition Manual (AISC 2003)) more accurately predicted the ultimate strength of two bolt flexible supported connections and reasonably predicted the ultimate strength of other connections tested.

7.3. COMPARISON OF SINGLE PLATE FLEXURAL YIELDING

7.3.1. GENERAL

Based upon observed test behavior and experimentally reported results, the question was raised as to how to best calculate the strength at which flexural yielding of the single plate ensues. As presented in Chapter 3, other design methods calculate for flexural yielding of the single plate in a variety of ways. The primary difference in methods occurs in using either the elastic or plastic section modulus. Other differences occur in using only the weld line to bolt line distance, \( a \), which assumes that the applied shear acts
directly through the bolt line. This assumption excludes any additional eccentric shear considerations based upon bolt eccentricity. Finally, the other important distinction occurs in the use of various equations at different applied shears as was the case in the Great Britain design method. The following subsections will discuss which method is the most accurate means for calculating the flexural yielding strength of single plates in single plate framing connections.

7.3.2. FLEXURAL YIELDING COMPARISON

As presented in Section 6.4.3 Plastic vs. Elastic Behavior, various means for determining the behavior of the single plate were analyzed. Both Test 1 and Test 8 were evaluated. By comparing the strength predictions from Chapter 6 to the experimentally observed behavior from strain gage readings, the comparisons are presented as ratios of predicted to observed. The predicted to observed ratios are considered conservative when less than unity and unconservative when greater than unity. Table 7.2 shows the single plate flexural yielding comparisons for Tests 1 and 8.
### TABLE 7.2 SINGLE PLATE FLEXURAL YIELDING COMPARISON

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Exp. Measured</th>
<th>Ratio: LRFD Predicted $V_u$ 1-to-Exp. Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Applied Load (kips)</td>
<td>Calculated Code Value ($e_m = a$)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>35.0 4</td>
<td>1.91 2</td>
</tr>
<tr>
<td></td>
<td>62.2 5</td>
<td>1.61 3</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>46.0 6</td>
<td>0.64 2</td>
</tr>
<tr>
<td></td>
<td>0.97 3</td>
<td>3.63 3</td>
</tr>
</tbody>
</table>

Indicates the best approximation(s) of the experimentally measured data

**Notes:**

1. Calculations based on $F_y$ (ksi) = 39.6, without strength reduction factor
2. Based upon Elastic Section Modulus, $S_e$
3. Based upon Plastic Section Modulus, $Z_e$
4. (From Figure 6.6) Approximately when the extents of the single plate begin to yield
5. Approximate applied shear when all single plate strain gages indicated yielding
6. Approximate applied shear when bottommost single plate strain gage indicated yielding

For Test 1, a ratio of 1.24 was found to accurately predict the 35kip experimental approximation of when the extreme edges of the single plate began to yield (indicating elastic behavior). This ratio is associated with the observed test eccentricity and elastic section modulus, $S_e$. The plastic behavior, experimentally found to be 62.2kips, of the single plate used in Test 1 was most accurately predicted by the bending strength calculated using the observed test eccentricity and the plastic section modulus, $Z_e$.

For Test 8, slightly different behavior was noted. Only at ultimate loading of the connection did the bottommost strain gage from Test 8 report strains to suggest yielding of the plate. Therefore, the majority of the single plate behaved elastically. The most accurate yet conservative ratio was found to be 0.97 for predicting the experimentally measured flexural yielding strength. This ratio corresponds to calculations using the
plastic section modulus without eccentric shear considerations (only taking into account the weld line to bolt line distance, \( a \)). Although the plate is observed to behave elastically, the strength is best approximated by plastic behavior assumptions (with the plastic section modulus, \( Z_c \)).

7.3.3. EXPERIMENTALLY OBSERVED CONSIDERATION

The single plate strain gages indicated that the single plates exhibited near a full range of behavior from elastic behavior, to first flexural yield, to near full plastic behavior. However, no single plates used in testing failed via fully plastic behavior. As such, the ultimate applied shear at the connection for each test (which corresponds to shear rupture of the connection bolts) does not correspond to the ultimate flexural yielding capacity of the single plate. Therefore, the single plate behavior may best be approximated by utilizing the plastic section modulus, \( Z_c \), in place of the elastic section modulus, \( S_e \), presented in Equation 3.7. Table 7.3 presents a comparison of the predicted flexural yielding strengths of the single plates (as calculated with either the elastic or plastic section modulus) in comparison to the ultimate flexural yielding capacity of the single plate.
## TABLE 7.3 PROPOSED FLEXURAL YIELDING CONSIDERATION

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Max. Applied Shear at Conn. (kips)</th>
<th>Calculated Code Value $^1$ (kips) $(S_x, e_{pl} = a)$</th>
<th>Ratio (Pred./Obs.)</th>
<th>Calculated Code Value $^2$ (kips) $(Z_x, e_{pl} = a)$</th>
<th>Ratio (Pred./Obs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>78.8</td>
<td>66.8</td>
<td>-----</td>
<td>100.2</td>
<td>-----</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>90.7</td>
<td>66.8</td>
<td>0.74</td>
<td>100.2</td>
<td>1.11</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>71.8</td>
<td>66.8</td>
<td>0.93</td>
<td>100.2</td>
<td>1.40</td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>61.4</td>
<td>66.8</td>
<td>1.09</td>
<td>100.2</td>
<td>1.63</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>75.6</td>
<td>66.8</td>
<td>0.88</td>
<td>100.2</td>
<td>1.33</td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>44.2</td>
<td>29.7</td>
<td>0.67</td>
<td>45.4</td>
<td>1.01</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>45.5</td>
<td>29.7</td>
<td>0.65</td>
<td>45.4</td>
<td>0.98</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>47.9</td>
<td>29.7</td>
<td>0.62</td>
<td>45.4</td>
<td>0.93</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>166.5 $^1$</td>
<td>363.8</td>
<td>-----</td>
<td>545.7</td>
<td>-----</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>202.5</td>
<td>363.8</td>
<td>1.80</td>
<td>545.7</td>
<td>2.70</td>
</tr>
</tbody>
</table>

**Notes:** All calculations performed without strength reduction factors  
1. Test stopped prior to shear rupture of connection bolts  
2. Calculations based on $F_y$ (ksi) = 39.6 for Tests 1 through 8  
   $F_y$ (ksi) = 44.4 for Tests 9 and 10

As indicated in Table 7.3 for the three and seven bolt tests, the most desired (conservative) ratios correspond with the elastic behavioral assumption. However, as discussed, shear rupture of the connection bolts and not flexural yielding of the single plate was experienced as the primary failure mode of the tested connections. As such, the flexural yielding capacity of the single plate is better approximated with plastic behavioral assumptions as additional capacity can be sustained in flexural yielding prior to failure.

### 7.3.4. SUMMARY

The flexural yielding strength of the connection is calculated as the moment sustained by the single plate divided by the plate eccentricity, $(M_n/e_{pl})$. The plate eccentricity, $e_{pl}$, can be limited to the weld line to bolt distance, $a$, or can be calculated as the $a$-distance combined with the measured or predicted bolt eccentricity of the connection $(a + e_b)$. As indicated in Table 7.2, the plate eccentricity can significantly
affect the flexural yielding strength calculation. Using Test 8 as an example, the plate eccentricity, when taking into account the observed test bolt eccentricity, is calculated to be 0.8in. A plate eccentricity less than one increases the flexural yielding strength. As such, excessively unconservative ratios were produced. Accordingly, the incorporation of bolt eccentricity assumptions into the flexural yielding equation is advised against. Therefore, the use of the bolt line to weld line distance, \( a \), is recommended. From the analyses conducted, the flexural yielding capacity of the single plate should be calculated with plastic behavior assumptions utilizing the plastic section modulus, \( Z_x \).

7.4. COMPARISON OF DESIGN METHODS

7.4.1. GENERAL

A comparison of the design methods outline in Chapter 3 is presented herein. The purpose of the comparisons is to determine which method most accurately predicts the observed experimental behavior.

7.4.2. DESIGN METHOD COMPARISONS OF EXPERIMENTAL FULL-SCALE TESTS

Utilizing the nine design methods, comparisons were made for each of the ten full-scale tests as tested as a part of this research. Tables 7.4, 7.5, and 7.6 present the results of the calculations and the tables are organized with respect to number of bolts in the connection (two, three, or seven bolts).

It should be noted that the AISC LRFD recommended strength reduction factors for each limit state were omitted from comparison calculations because the yield strengths of the steel used in testing were measured. With measured yield strengths, a considerable amount of variability is removed justifying the omission of the strength reduction factor.
As such, measured material properties from supplemental tests of the steel and connection bolts (discussed in Chapters 4 and 5) used in testing were incorporated into the design method calculations. The single bolt single shear design strength was calculated using Equation 7.1 with the group action factor and incorporates the experimental results from supplemental single bolt single shear strength with threads included in the shear plane as tested by Gelo (2003).

\[
\phi V_n = r_v (0.8)
\]

(EQ 7.1)

where: \( r_v \) = 30.3 kips per bolt (measured by Gelo (2003))

\[
\phi V_n = 24.2 \text{kips, single bolt single shear strength}
\]

It should also be noted that for calculation purposes, the strength of the weld was calculated as the minimum of Equation 3.20 (eccentrically loaded weld groups), Equation 3.21 (base metal failure), and Equation 3.22a (directly loaded weld strength). Because the material properties of the weld were not experimentally measured, the strength reduction factor was included in weld related calculations except the base metal failure calculation, which utilizes measured single plate properties and includes the strength reduction factor. The following tables present the various limit states of various design methods as calculated for two, three, and seven bolt connections as tested in this series of research.
### TABLE 7.4 LIMIT STATE STRENGTH PREDICTIONS (TWO BOLT CONNECTION)

<table>
<thead>
<tr>
<th>Connection Component</th>
<th>Design Method Recommended by:</th>
<th>Bolt 1</th>
<th>Single Plate 1</th>
<th>Weld 1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Astaneh 2003</td>
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**Notes:**
1. Limit states for connection component calculated without strength reduction factors
2. Limit states associated with weld are calculated with strength reduction factors
   while limit states associated with base metal failure is calculated without strength reduction factor
## TABLE 7.5 LIMIT STATE STRENGTH PREDICTIONS (THREE BOLT CONNECTION)

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<th>Richard (Flexible)</th>
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<th>Duggal &amp; Wallace (Rigid)</th>
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<th>Great Britain</th>
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**Notes:**
1. Limit states for connection component calculated without strength reduction factors
2. Limit states associated with weld are calculated with strength reduction factors
   while limit states associated with base metal failure is calculated without strength reduction factor
**TABLE 7.6 LIMIT STATE STRENGTH PREDICTIONS (SEVEN BOLT CONNECTION)**

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**Notes:**
1. Limit states for connection component calculated without strength reduction factors.
2. Limit states associated with weld are calculated with strength reduction factors while limit states associated with base metal failure is calculated without strength reduction factor.
As discussed in Section 5.7, limit states, such as shear yielding, plate bending (flexural yielding), and bearing and tear out, were approximated and presented for each test in Table 5.3. Table 7.7 consists of the values from Table 5.3, which have been grouped and averaged according to the number of bolts in the connection. Similarly, from Table 5.1, the bolt eccentricity values as reported have been averaged per size of connection into the following table.

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</table>

Notes: 1. Averaged from bolt eccentricity values reported in Tests 6 and 7
2. Averaged from bolt eccentricity values reported in Tests 1 and 2
3. Averaged from bolt eccentricity values reported in Tests 3 and 4

With these approximated limit state values as found from the ten full-scale tests, direct comparisons to limit state design strengths are made. The comparisons are presented as ratios of predicted to observed. Tables 7.8, 7.9, and 7.10 present the comparison ratios for two, three, and seven bolt connections respectively. Within Tables
7.8 to 7.10, the best conservative ratios were identified for each analyzed test with gray shading.
### TABLE 7.8 COMPARISON STRENGTH PREDICTIONS (TWO BOLT CONNECTION)

<table>
<thead>
<tr>
<th>Connection Component</th>
<th>Limit State/Corresponding Eccentricity</th>
<th>Exp. Value</th>
<th>Astaneh</th>
<th>AISC 2003</th>
<th>Richard (Rigid)</th>
<th>Richard (Flexible)</th>
<th>Sarkar &amp; Wallace</th>
<th>Duggal &amp; Wallace (Rigid)</th>
<th>Duggal &amp; Wallace (Flexible)</th>
<th>Ashakul</th>
<th>Australia</th>
<th>New Zealand</th>
<th>Great Britain</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bolt</strong></td>
<td>Shear Strength (Rigid)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
<td>-----</td>
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<tr>
<td></td>
<td>(Flexible)</td>
<td>45.9</td>
<td>0.47</td>
<td>1.06</td>
<td>-----</td>
<td>-----</td>
<td>1.06</td>
<td>1.06</td>
<td>0.47</td>
<td>0.47</td>
<td>0.82</td>
<td>0.82</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>Eccentricity, $e_b$ (in) (Rigid)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>-3.4</td>
<td>0.88</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
<td>0.06</td>
<td>0.88</td>
<td>0.88</td>
<td>0.88</td>
</tr>
<tr>
<td><strong>Single Plate</strong></td>
<td>Shear Yielding</td>
<td>44.3</td>
<td>1.21</td>
<td>1.21</td>
<td>1.21</td>
<td>-----</td>
<td>1.21</td>
<td>-----</td>
<td>1.21</td>
<td>1.21</td>
<td>0.72</td>
<td>1.21</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td>Bearing &amp; Tearout (Rigid)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Bearing &amp; Tearout (Flexible)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding (Rigid)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>44.7</td>
<td>-----</td>
<td>0.66</td>
<td>-----</td>
<td>0.79</td>
<td>-----</td>
<td>0.66</td>
<td>-----</td>
<td>1.00</td>
<td>1.00</td>
<td>0.66</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>Eccentricity, $e_b^2$ (in) (Rigid)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>3.0</td>
<td>-----</td>
<td>1.00</td>
<td>-----</td>
<td>1.26</td>
<td>-----</td>
<td>1.00</td>
<td>-----</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

---

**Notes:**
1. Limit states for connection component calculated without strength reduction factors
2. Experimental value assumed to be the weld line to bolt line distance, $a$
3. Ratios ≤ 1.0 are conservative
4. Ratios > 1.0 are unconservative

---

Indicates that direct shear was specified in the design method.
### TABLE 7.9 COMPARISON STRENGTH PREDICTIONS (THREE BOLT CONNECTION)

<table>
<thead>
<tr>
<th>Connection Component</th>
<th>Limit State/Corresponding Eccentricity</th>
<th>Exp. Value</th>
<th>Astaneh</th>
<th>AISC 2003</th>
<th>Richard (Rigid)</th>
<th>Richard (Flexible)</th>
<th>Sarkar &amp; Wallace</th>
<th>Duggal &amp; Wallace (Rigid)</th>
<th>Duggal &amp; Wallace (Flexible)</th>
<th>Ashakul</th>
<th>Australia</th>
<th>New Zealand</th>
<th>Great Britain</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bolt 1</strong></td>
<td>Shear Strength (Rigid)</td>
<td>90.7</td>
<td>0.72</td>
<td>0.80</td>
<td>0.80</td>
<td>-----</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.76</td>
<td>0.75</td>
<td>0.44</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>73.7</td>
<td>0.58</td>
<td>0.99</td>
<td>-----</td>
<td>0.99</td>
<td>-----</td>
<td>0.99</td>
<td>0.99</td>
<td>-----</td>
<td>0.55</td>
<td>0.55</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>Eccentricity, ε_s (in) (Rigid)</td>
<td>1.8</td>
<td>0.56</td>
<td>-----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>0.41</td>
<td>1.67</td>
<td>1.67</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>-1.9</td>
<td>1.62</td>
<td>-----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>1.62</td>
<td>1.62</td>
<td>1.62</td>
<td></td>
</tr>
<tr>
<td><strong>Single Plate 4</strong></td>
<td>Shear Yielding</td>
<td>73.2</td>
<td>1.10</td>
<td>1.10</td>
<td>1.10</td>
<td>1.10</td>
<td>1.10</td>
<td>1.10</td>
<td>0.91</td>
<td>1.10</td>
<td>0.66</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bearing &amp; Tearout (Rigid)</td>
<td>66.0</td>
<td>1.73</td>
<td>1.73</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>1.39</td>
<td>1.16</td>
<td>1.91</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bearing &amp; Tearout (Flexible)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding (Rigid)</td>
<td>75.4</td>
<td>0.89</td>
<td>0.75</td>
<td>1.33</td>
<td>0.69</td>
<td>-----</td>
<td>-----</td>
<td>1.06</td>
<td>1.33</td>
<td>0.58</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>74.7</td>
<td>0.89</td>
<td>0.73</td>
<td>0.89</td>
<td>0.89</td>
<td>0.89</td>
<td>0.89</td>
<td>1.34</td>
<td>1.34</td>
<td>0.58</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Eccentricity, ε (in) (Rigid)</td>
<td>3.0</td>
<td>-----</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.25</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>3.0</td>
<td>-----</td>
<td>1.00</td>
<td>1.83</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

---

**Notes:**

1. Limit states for connection component calculated without strength reduction factors
2. Experimental value assumed to be the weld line to bolt line distance, a
3. Ratios ≤ 1.0 are conservative
4. Ratios > 1.0 are unconservative

---

**Table Notes:**

- Indicates that direct shear was specified in the design method.
<table>
<thead>
<tr>
<th>Connection Component</th>
<th>Limit State/Corresponding Eccentricity</th>
<th>Exp. Value</th>
<th>Astaneh</th>
<th>AISC 2003</th>
<th>Richard (Rigid)</th>
<th>Richard (Flexible)</th>
<th>Sarkar &amp; Wallace</th>
<th>Duggal &amp; Wallace (Rigid)</th>
<th>Duggal &amp; Wallace (Flexible)</th>
<th>Ashakul</th>
<th>Australia</th>
<th>New Zealand</th>
<th>Great Britain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt ¹</td>
<td>Shear Strength (Rigid)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>(Flexible)</td>
<td>202.5</td>
<td>0.73</td>
<td>0.84</td>
<td>-----</td>
<td>0.84</td>
<td>0.84</td>
<td>0.84</td>
<td>-----</td>
<td>-----</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.84</td>
</tr>
<tr>
<td>Eccentricity, e₃ (in) (Rigid)</td>
<td>5.5</td>
<td>0.30</td>
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<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>0.36</td>
<td>0.55</td>
<td>0.55</td>
<td></td>
</tr>
<tr>
<td>(Flexible)</td>
<td>0.5</td>
<td>6.00</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>6.00</td>
<td>6.00</td>
<td>6.00</td>
<td></td>
</tr>
<tr>
<td>Single Plate ²</td>
<td>Shear Yielding</td>
<td>182.7</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
<td>1.07</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
<td></td>
</tr>
<tr>
<td>Bearing &amp; Tearout (Rigid)</td>
<td>165.9</td>
<td>1.81</td>
<td>1.81</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
<td>1.38</td>
<td>1.72</td>
<td>1.89</td>
<td></td>
</tr>
<tr>
<td>Bearing &amp; Tearout (Flexible)</td>
<td>201.8</td>
<td>1.49</td>
<td>1.49</td>
<td>-----</td>
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<td>-----</td>
<td>-----</td>
<td>1.13</td>
<td>1.41</td>
<td>1.55</td>
<td></td>
</tr>
<tr>
<td>Flexural Yielding (Rigid)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
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<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td></td>
</tr>
<tr>
<td>(Flexible)</td>
<td>200.0</td>
<td>-----</td>
<td>2.04</td>
<td>-----</td>
<td>0.96</td>
<td>-----</td>
<td>2.04</td>
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<td>-----</td>
<td>3.06</td>
<td>3.06</td>
<td>2.04</td>
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</tr>
<tr>
<td>Eccentricity ², e (in) (Rigid)</td>
<td>3.0</td>
<td>-----</td>
<td>1.00</td>
<td>3.12</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.67</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>(Flexible)</td>
<td>3.0</td>
<td>-----</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

---

**Notes:**
1. Limit states for connection component calculated without strength reduction factors
2. Experimental value assumed to be the weld line to bolt line distance, a
3. Ratios ≤ 1.0 are conservative
4. Ratios > 1.0 are unconservative

---

*Indicates that direct shear was specified in the design method*
Tables 7.8 to 7.10 were evaluated with respect to comparison ratio. As discussed earlier, a ratio of unity or slightly less than unity was preferred. For the compared limit states, the ratio, which closest met the above criterion, was selected for each limit state. Each ratio was then cross-referenced to determine which researcher or design method was associated with the given ratio. The following observations were made:

- For the three and seven bolt connections, the AISC LRFD 3rd Edition Manual (AISC 2003) method was found to most accurately predict the bolt group shear strength of the connection.
  - It should be recalled that the bolt group shear strength was calculated using measured properties from supplemental shear tests without a strength reduction factor.
  - The bolt group shear strength value was calculated without the use of eccentric shear considerations

- The shear yielding strength of the single plate was best predicted by the New Zealand design method for a two bolt connection and Ashakul’s design method recommendations for three and seven bolt connections. However, Ashakul’s design method was not calculated for a two bolt connection because the design method was reported for rigid supported tests only. Had Ashakul’s shear yielding design strength been calculated, Ashakul’s design method for shear yielding would have been found to better predict the shear yielding behavior for two, three, and seven bolt connections.

- For the seven bolt connection, the Australian design method for calculating the bearing and tear out strength was deemed to be the most accurate of the compared methods. For the three bolt connection, the Australian method was second to the New Zealand design method for calculating the bearing and tear out strength.

- Flexural yielding results varied as no specific method most accurately predicted the flexural yielding capacity for all connection sizes. However, the following trends were indicated:
– For two bolt connections with a flexible support condition, the New Zealand and Australian design methods, which assumed a plastic section modulus and a plate eccentricity of 3in, most accurately predicted flexural yielding of the single plate

– For three bolt connections, the AISC LRFD 3rd Edition Manual (AISC 2003) method, assumed an elastic section modulus and a plate eccentricity of 3in, was found to most accurately predict the flexural yielding strength of the single plate

– For seven bolt connections with a flexible support condition, the design method as recommended by Richard, which assumed a plastic section modulus and a plate eccentricity unique to that method, most accurately predicted flexural yielding of the single plate

In Tables 7.8 to 7.10, comparisons regarding shear strength of the bolt group and shearing yielding, bearing and tear out, and flexural yielding of the single plate were presented. Because the other limit states or failure modes were not experienced, direct comparisons could not be made. However, because the ultimate applied shear of the connection was recorded, the other limit states of the connection could be evaluated with respect to the observed maximum applied shear sustained by the connection. Tables 7.11 to 7.13 present comparisons of limit states which were not experienced in testing. These limit states include shear rupture, block shear, and flexural rupture of the single plate and failure of the weldment. If the various design methods indicated that the connection should have failed at less load than was observed as the ultimate applied shear at the connection, then the design method could be said to be conservative (less than unity) for that particular limit state. However, in these comparisons, predictions greater than unity are acceptable as the limit state did not occur.
### TABLE 7.11 UNEXPERIENCED LIMIT STATE COMPARISONS (TWO BOLT CONNECTION)

<table>
<thead>
<tr>
<th>Connection Component</th>
<th>Limit State</th>
<th>Corresponding Eccentricity</th>
<th>Obs. Max Load or Ecc.</th>
<th>Astaneh 2003</th>
<th>Richard (Rigid)</th>
<th>Richard (Flexible)</th>
<th>Sarkar &amp; Wallace</th>
<th>Duggal &amp; Wallace (Rigid)</th>
<th>Duggal &amp; Wallace (Flexible)</th>
<th>Ashakul</th>
<th>Australia</th>
<th>New Zealand</th>
<th>Great Britain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Rupture</td>
<td>45.9</td>
<td>1.33 1.29</td>
<td>0.98</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>1.06</td>
<td>1.00</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>45.9</td>
<td>1.33 1.29</td>
<td>0.98</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>1.06</td>
<td>1.00</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>Flexural Rupture</td>
<td>45.9</td>
<td>1.33 1.29</td>
<td>0.98</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>1.06</td>
<td>1.00</td>
<td>1.06</td>
</tr>
<tr>
<td>Weld</td>
<td>Weld Strength (Flexible)</td>
<td>45.9</td>
<td>1.17 1.13</td>
<td>0.94</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>1.06</td>
<td>1.00</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>Eccentricity, $e_w$ (in) (Flex)</td>
<td>-0.40</td>
<td>5.00 7.50</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
</tr>
</tbody>
</table>

Notes: 1. As compared to averaged weld eccentricity values reported in Tests 6 and 7 ($e_w = -0.4\text{in}$)  
2. As compared to averaged weld eccentricity value reported in Tests 6 ($e_w = -1.4\text{in}$)  
3. As compared to averaged weld eccentricity values reported in Tests 7 and 8 ($e_w = 0.7\text{in}$)

---

### TABLE 7.12 UNEXPERIENCED LIMIT STATE COMPARISONS (THREE BOLT CONNECTION)

<table>
<thead>
<tr>
<th>Connection Component</th>
<th>Limit State</th>
<th>Corresponding Eccentricity</th>
<th>Obs. Max Load or Ecc.</th>
<th>Astaneh 2003</th>
<th>Richard (Rigid)</th>
<th>Richard (Flexible)</th>
<th>Sarkar &amp; Wallace</th>
<th>Duggal &amp; Wallace (Rigid)</th>
<th>Duggal &amp; Wallace (Flexible)</th>
<th>Ashakul</th>
<th>Australia</th>
<th>New Zealand</th>
<th>Great Britain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Rupture</td>
<td>90.7</td>
<td>1.01 0.98</td>
<td>0.98</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>1.09</td>
<td>1.09</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>90.7</td>
<td>1.01 0.98</td>
<td>0.98</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>1.09</td>
<td>1.09</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>Flexural Rupture</td>
<td>90.7</td>
<td>1.01 0.98</td>
<td>0.98</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>1.09</td>
<td>1.09</td>
<td>1.09</td>
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<tr>
<td>Weld</td>
<td>Weld Strength (Rigid)</td>
<td>90.7</td>
<td>0.88 0.88</td>
<td>0.75</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.86</td>
<td>0.86</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>73.70</td>
<td>1.09 1.09</td>
<td>0.91</td>
<td>0.75</td>
<td>0.63</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.86</td>
<td>0.86</td>
<td>0.86</td>
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<tr>
<td></td>
<td>Eccentricity, $e_w$ (Rigid)</td>
<td>4.80</td>
<td>0.42 0.63</td>
<td>1.11</td>
<td>0.63</td>
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<td>0.63</td>
<td>0.63</td>
<td>0.86</td>
<td>0.86</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>(Flexible)</td>
<td>1.15</td>
<td>2.61 2.61</td>
<td>4.77</td>
<td>4.77</td>
<td>4.77</td>
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<td>4.77</td>
<td>4.77</td>
<td>4.77</td>
</tr>
</tbody>
</table>

Note: Ratios are presented as calculated-to-observed. (Pred./Obs.) Indicates a conservative prediction.
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Rupture</td>
<td>202.5</td>
<td>1.13</td>
<td>1.10</td>
<td>-----</td>
<td>-----</td>
<td>1.13</td>
<td>1.10</td>
<td>-----</td>
<td>-----</td>
<td>1.13</td>
<td>1.13</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>202.5</td>
<td>-----</td>
<td>1.14</td>
<td>-----</td>
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<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>1.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flexural Rupture</td>
<td>202.5</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>2.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weld</td>
<td>Weld Strength (Flexible)</td>
<td>202.5</td>
<td>1.04</td>
<td>1.04</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Eccentricity, e_w (in) (Flex.)</td>
<td>3.50</td>
<td>2.00</td>
<td>0.86</td>
<td>-----</td>
<td>2.73</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>2.73</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Ratios are presented as calculated-to-observed. (Pred./Obs.)

Indicates a conservative prediction
In Tables 7.11 to 7.13 strength reduction factors were only applied to weld strength calculations as the material properties of the weld were not experimentally measured. Isolating the AISC LRFD 3\textsuperscript{rd} Edition Manual (AISC 2003), only the shear rupture strength of the single plate used in the three bolt connection test was found to be inadequate as the ratio indicated that shear rupture failure of the single plate should have occurred. Weld design strengths were indicated to be adequate in the two and seven bolt connections. However, for the three bolt connection, base metal failure was indicated to have occurred.

In the New Zealand design method, flexural rupture of the single plate yielded conservative estimates of design strength for two and three bolt connection sizes. The two and three bolt connection ratios regarding flexural rupture ranged from 0.79 to 0.86 respectively. As such, the design calculation for flexural rupture was found to be inadequate as the ratio indicated that flexural rupture failure of the single plate should have occurred.

7.5. COMPARISON OF ECCENTRICITY EQUATIONS

7.5.1. AISC LRFD 3\textsuperscript{rd} EDITION MANUAL ECCENTRICITY EQUATIONS

A comparison was made to determine the adequacy of the bolt eccentricity equations specified in the AISC LRFD 3\textsuperscript{rd} Edition Manual (AISC 2003) which are based upon recommendations from Astanek. The comparison presents both the specified eccentricity equations and the experimentally obtained bolt eccentricities from full-scale tests conducted in this series of research and tests as conducted by Astanek et al. The following figures group the results with respect to the following test parameters: (1) rigid support with standard holes as shown in Figure 7.1, (2) rigid support with short-slotted
holes as shown in Figure 7.2, (3) flexible support with standard holes as shown in Figure 7.3, and (4) flexible support with short-slotted holes as shown in Figure 7.4. Imposed on each graph is the Astaneh recommended equation, which was defined within Equations 3.24 to 3.27. Bolt eccentricity equations for single plate connections with a flexible support condition are plotted without the following considerations: (1) the \( a \)-distance limitations and (2) the absolute value operator. As such, Equations 3.26 and 3.27 become Equations 7.2 and 7.3. Equations 3.24 and 3.25, which define the bolt eccentricity for single plate connections with a rigid support condition with standard holes (RST) and rigid support with short-slotted holes (RSS), remained unchanged.

- For a flexible support condition with standard holes (FST):

\[
e_b = (n-1) - a \quad \text{(EQ 7.2)}
\]

- For a flexible support condition with short-slotted holes (FSS):

\[
e_b = \left( \frac{2n}{3} - a \right) \quad \text{(EQ 7.3)}
\]

where:
- \( a \) = bolt line to weld line distance
- \( e_b \) = bolt eccentricity
- \( n \) = number of bolts

Within Figures 7.1 to 7.4, the slightly modified flexible support equations (without the \( a \)-distance limitation and absolute value operators) are plotted as solid lines. The original bolt eccentricity equations are plotted as dashed lines.
FIGURE 7.1 RIGID SUPPORT WITH STANDARD HOLES ECCENTRICITY PLOT

FIGURE 7.2 RIGID SUPPORT W/ SHORT-SLOTTED HOLES ECCENTRICITY PLOT
FIGURE 7.3 FLEXIBLE SUPPORT WITH STANDARD HOLES ECCENTRICITY PLOT

FIGURE 7.4 FLEX. SUPPORT W/ SHORT-SLOTTED HOLES ECCENTRICITY PLOT
It should be noted that the bolt eccentricity equations are dependent upon the bolt line to weld line distance, $a$. As such, the $a$-distance varied with respect to tests conducted by Astaneh and Sumner. For example, tests conducted in this series of research utilized an $a$-distance of 3in, while Astaneh utilized a varying $a$-distance of 2.75in or 3.125in. Accordingly, in Figures 7.1 to 7.4, there were two pair of lines representing calculated theoretical eccentricities, as $a$-distances varied. However, although $a$-distances varied, eccentricity comparison plots indicated that the theoretically predicted eccentricity equation lines are very near each other.

As seen in Figures 7.3 and 7.4, the modified eccentricity Equations 7.2 and 7.3 (without the $a$-distance limitation and absolute value operators) appear to better represent the experimentally obtained bolt eccentricity results. When the typical eccentricity equation is used, the $a$-distance limitation and absolute value operators force the predicted eccentricity away from the experimentally obtained eccentricities.

### 7.5.2. IMPROVEMENTS TO THE ACCEPTED ECCENTRICITY EQUATIONS

Using the current design method for single plate shear connections, the magnitude of Astaneh’s bolt eccentricity predictions were found to be less than the observed test eccentricities for the majority of tests. Figures 7.5 to 7.8 are based upon Figures 7.1 to 7.4, yet these figures differ in the following respects: (1) the observed eccentricity data points have been plotted such that only the absolute value of each eccentricity measured for each connection are shown (the purpose of which was to better group the test data), (2) imposed on these graphs are dataset trend lines, and (3) also plotted are the improved recommendations for calculating connection eccentricity. Improved eccentricity equation
recommendations were generated from iterative processes to best approximate the experimentally obtained eccentricity data.

As recommended by Astaneh, bolt eccentricity equations are based upon both the number of bolts in the connection and weld line to bolt line distance, \( a \). Trend lines are indicated on each graph as the series labeled ‘Linear (trend)’, while the recommended eccentricity equations are denoted with a classification such as ‘RST (a=2.75in)’. Appearing in Figures 7.5 to 7.8, the recommended eccentricity equations are plotted with respect to the two \( a \)-distances utilized by Astaneh (\( a = 2.75 \text{in} \)) and Sumner (\( a = 3 \text{in} \)).

![Rigid Support with Standard Holes Eccentricity Comparison](image)

**FIGURE 7.5 RIGID SUPPORT WITH STANDARD HOLES ECCENTRICITY PLOT**
FIGURE 7.6 RIGID SUPPORT W/ SHORT-SLOTTED HOLES ECCENTRICITY PLOT

FIGURE 7.7 FLEXIBLE SUPPORT WITH STANDARD HOLES ECCENTRICITY PLOT
Plotted in Figures 7.5, 7.7, and 7.8, trend lines and corresponding equations can be seen. It should be noted that these equations were calculated for selected portions of the datasets. For example, in Figure 7.8, only Tests 3, 5, 8, and 10 of this series of research were chosen as the basis for the trend. In each figure with a trend line, the general behavior for tests with a flexible support condition is that the slope is negative as plotted. With the negative slope, the trend line would produce a negative eccentricity when connections with a greater number of bolts are present. Accordingly, trend lines were not chosen as the final recommended form of the equation.

As currently accepted, the eccentricity equations are categorized based upon support condition (rigid beam-to-column flange, or flexible beam-to-girder web) and hole condition (standard or short-slotted). This means of classifying a connection works well
for calculating the eccentricity of a connection. Of Equations 3.24 to 3.27, three of the
four were revised. Only the equation for single plate connections with rigid support
condition and short-slotted holes remained unchanged. The recommended equations are
as follows:

- For a rigid support condition with standard holes:

\[ e_b = \left( \frac{2n}{3} - \frac{a}{2} \right) \geq \frac{a}{2} \]  \hspace{1cm} \text{(EQ 7.4)}

- For a rigid support condition with short-slotted holes:

\[ e_b = \left| \left( \frac{2n}{3} \right) - a \right| \]

(defined as Equation 3.25)

- For a flexible support condition with standard holes:

\[ e_b = \left( -\frac{3}{4}n + \frac{7}{4}a \right) \geq \frac{a}{3} \]  \hspace{1cm} \text{(EQ 7.5)}

- For a flexible support condition with short-slotted holes:

\[ e_b = \left( -\frac{3}{4}n + \frac{5}{4}a \right) \geq \frac{a}{3} \]  \hspace{1cm} \text{(EQ 7.6)}

where: \( a \) = bolt line to weld line distance
\( e_b \) = bolt eccentricity
\( n \) = number of bolts
From these bolt eccentricity equations, bolt group shear design strengths were computed for each connection tested. Table 7.14 summarizes the calculated design strengths and presents comparisons of predicted-to-observed ultimate strength values. Bolt group shear strengths are presented with and without the applied strength reduction factor ($\phi = 0.75$). From Section 7.2, recommendations were made in regards to the application of the group action factor and necessity to consider eccentric shear effects. These recommendations were incorporated into Table 7.14 in a direct comparison for bolt group shear strengths as calculated with bolt eccentricity recommendations from the currently accepted Astaneh equations and the newly recommended eccentricity equations presented above. Design strengths calculated with the newly recommended eccentricity equations are denoted as ‘Sumner $e_b$’ within Table 7.14.
### TABLE 7.14 LRFD BOLT GROUP SHEAR STRENGTH W/ PROPOSED ECCENTRICITY CONSIDERATIONS

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Test Description</th>
<th>Obs. $e_b$ (in.)</th>
<th>Max. Load (kips)</th>
<th>Predicted Sumner $e_b$ (in.)</th>
<th>Predicted Astaneh $e_b$ (in.)</th>
<th>Predicted $Vu$ (kips)</th>
<th>Ratio (Pred./Max.)</th>
<th>LRFD Bolt Strength (Sumner $e_b$) w/o $\phi$</th>
<th>LRFD Bolt Strength (Astaneh $e_b$) w/o $\phi$</th>
<th>Ratio (Pred./Max.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>1.6</td>
<td>78.8$^1$</td>
<td>1.00</td>
<td>1.00</td>
<td>65.7</td>
<td>-----</td>
<td>65.7</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>2.1</td>
<td>90.7</td>
<td>1.50</td>
<td>1.00</td>
<td>59.9</td>
<td>0.66</td>
<td>65.7</td>
<td>0.72</td>
<td></td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>(-) 1.7</td>
<td>71.8</td>
<td>1.50</td>
<td>1.00</td>
<td>59.9</td>
<td>0.83</td>
<td>65.7</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>(-) 2.0</td>
<td>61.4</td>
<td>3.00</td>
<td>1.00</td>
<td>42.4</td>
<td>0.69</td>
<td>65.7</td>
<td>1.07</td>
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<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted,</td>
<td>0.1</td>
<td>75.6</td>
<td>1.50</td>
<td>1.00</td>
<td>59.9</td>
<td>0.79</td>
<td>65.7</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>(-) 4.4</td>
<td>44.2</td>
<td>3.75</td>
<td>2.00</td>
<td>22.4</td>
<td>0.51</td>
<td>35.8</td>
<td>0.81</td>
<td></td>
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<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>(-) 2.4</td>
<td>45.5</td>
<td>2.25</td>
<td>1.67</td>
<td>33.5</td>
<td>0.74</td>
<td>38.8</td>
<td>0.85</td>
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<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted,</td>
<td>(-) 2.2</td>
<td>47.9</td>
<td>2.25</td>
<td>1.67</td>
<td>33.5</td>
<td>0.70</td>
<td>38.8</td>
<td>0.81</td>
<td></td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>5.5</td>
<td>166.5$^1$</td>
<td>5.00</td>
<td>1.67</td>
<td>169.7</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
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</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted,</td>
<td>0.5</td>
<td>202.5</td>
<td>1.00</td>
<td>1.67</td>
<td>169.7</td>
<td>0.84</td>
<td>169.7</td>
<td>0.84</td>
<td></td>
</tr>
</tbody>
</table>

---

**Notes:**
1. Calculated without the 20 percent group reduction factor
2. Calculated without eccentric shear effects
3. Test stopped prior to shear rupture of connection bolts

---

**Indicates the better prediction ratio**
As presented in this section, the goal was to establish eccentricity equations which better predicted the eccentricity of single plate framing connections. Although eccentricity equations were formulated which more accurately predicted the eccentricity of the connection, the resulting bolt group shear strength calculations indicated that shear strength calculations with Astaneh predicted bolt eccentricities better estimated the ultimate strength of the connections tested. The reason for this is because the effective number of bolts as determined from Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) is heavily affected by the eccentricity placed on the bolt group. As such, the effective number of bolts of the connection can significantly limit bolt group strength predictions. With larger eccentricities as predicted by the newly recommended bolt eccentricity equations, the effective number of bolts is lessened more so than when the Astaneh predicted bolt eccentricities are incorporated.

The issue at hand is that the eccentricity of the connection can be better approximated while adversely impacting the predicted strength of the connection. One possible explanation is that the instantaneous center of rotation method utilized in Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) may not directly apply to single plate framing connections. In other words, the various framing parameters, such as support condition (rigid or flexible), off-axis bolt groups, incorporation of a simulated slab restraint, and hole condition (standard or short-slotted) potentially affect the rotational characteristics of the connection such that Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) becomes inapplicable. Table 7.15 presents a study to determine the eccentricity necessary to predict the effective number of bolts to accurately calculate the observed maximum capacity of the full-scale single plate connections tested.
The column ‘Calculated Effective No. of Bolts’ was calculated in the following manner using experimentally measured bolt shear strengths from supplemental bolt shear tests.

\[
\text{Effective No. of Bolts} = \frac{\text{Obs. Max. Load}}{\text{Calculated } V_n}
\]  
(EQ 7.7)

### TABLE 7.15 REQUIRED ECCENTRICITY TO PREDICT EFFECTIVE NO. OF BOLTS

<table>
<thead>
<tr>
<th>Test I.D.</th>
<th>Test Description</th>
<th>Obs. $e_b$ (in.)</th>
<th>Max. Load (kips)</th>
<th>Calculated Effective No. of Bolts $^1$</th>
<th>Eccentricity from Table 7.17 Required to Predict Eff. No. of Bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RSS-3-A325-N</td>
<td>3 Bolt, Rigid, Short-slotted</td>
<td>1.6</td>
<td>78.8</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S2-RST-3-A325-N</td>
<td>3 Bolt, Rigid, Standard</td>
<td>2.1</td>
<td>90.7</td>
<td>3.74 $^3$</td>
<td>Max of 3.19 w/ zero ecc. $^1$</td>
</tr>
<tr>
<td>S3-FSS-3-A325-N</td>
<td>3 Bolt, Flexible, Short-slotted</td>
<td>(-) 1.7</td>
<td>71.8</td>
<td>2.96</td>
<td>0.48</td>
</tr>
<tr>
<td>S4-FST-3-A325-N</td>
<td>3 Bolt, Flexible, Standard</td>
<td>(-) 2.0</td>
<td>61.4</td>
<td>2.53</td>
<td>1.38</td>
</tr>
<tr>
<td>S5-FSS-3-A325-N-SR</td>
<td>3 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.1</td>
<td>75.6</td>
<td>3.12 $^3$</td>
<td>0.15</td>
</tr>
<tr>
<td>S6-FST-2-A325-N</td>
<td>2 Bolt, Flexible, Standard</td>
<td>(-) 4.4</td>
<td>44.2</td>
<td>1.46 $^2$</td>
<td>1.07</td>
</tr>
<tr>
<td>S7-FSS-2-A325-N</td>
<td>2 Bolt, Flexible, Short-slotted</td>
<td>(-) 2.4</td>
<td>45.5</td>
<td>1.50 $^2$</td>
<td>0.93</td>
</tr>
<tr>
<td>S8-FSS-2-A325-N-SR</td>
<td>2 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>(-) 2.2</td>
<td>47.9</td>
<td>1.58 $^2$</td>
<td>0.67</td>
</tr>
<tr>
<td>S9-RSS-7-A325-N</td>
<td>7 Bolt, Rigid, Short-slotted</td>
<td>5.5</td>
<td>166.5</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>S10-FSS-7-A325-N-SR</td>
<td>7 Bolt, Flexible, Short-slotted, Simulated slab restraint</td>
<td>0.5</td>
<td>202.5</td>
<td>8.35</td>
<td>2.49</td>
</tr>
</tbody>
</table>

**Notes:**
1. Calculated number of bolts necessary to predict ultimate strength without a strength reduction factor
2. Calculated without the 20 percent group reduction factor
3. Effective number of bolts cannot exceed the number of bolts of the connection
4. Test stopped prior to shear rupture of connection bolts

From this table, it can be seen that there is no direct correlation between the eccentricity values necessary to predict the effective number of bolts and the observed test eccentricity values. It should be noted that predicted eccentricity values from Table 7.15 are nearer to the eccentricity values predicted by the Astaneh bolt eccentricity equations (seen in Table 7.14). As such, a general eccentricity of approximately 1.0in applied to Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) seems to
predict the ultimate strength of the connections when combined with the recommendations from Section 7.2.

7.6. SUMMARY

At the heart of this research, the focus was to improve the AISC LRFD 3rd Edition Manual (AISC 2003) design method for single plate framing connections. To do so, several investigational studies, which were based upon the limit states of the single plate connection, were conducted. From this chapter, emphasis was placed upon the connection bolts, the currently accepted bolt eccentricity equations, and the flexural yielding behavior of the single plate. Although a significant portion of the experimental data came from the experimental tests conducted in this series of research, tests conducted by Astaneh et al. were incorporated into the comparison study to increase the sample size of the experimental data. In addition, the design methods presented in Chapter 3 were analyzed to determine which method most accurately predicted the individual limit states of the ten full-scale single plate connections tested in this series of research.

In the preceding sections of this chapter, individual summaries of results and trends were presented. However, the following summarizes the pertinent findings and recommendations from the investigational study of this body of research. Only the recommendations to improve the currently accepted AISC manual method will be presented.
**Connection Bolts**

The AISC LRFD 3rd Edition Manual (AISC 2003) single plate framing connection design method was determined to be the most accurate method for predicting the bolt group shear capacity. However, the recommendations and considerations from Section 7.2.3 should be applied to this design calculation limit state.

Undisputedly as indicated by full-scale test results, the connection bolts in single plate framing connections experience eccentric shear. For the bolt eccentricity considerations, at present time the Astaneh recommended equations are found to be satisfactory in that they predict bolt eccentricities conservatively. This conservative nature appears to coordinate well with effective number of bolt calculations from Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003). However, as discussed, Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) may not be completely applicable to the eccentrically loaded bolt group of single plate framing connections.

**Single Plate: Bearing and Tear Out**

The Australian design method for the bearing and tear out strength of the single plate was found to be the most accurate means for predicting the design strength associated with this limit state. The Australian design method calculation for the bearing and tear out strength of the single plate is defined as Equation 3.49. For rigid supported tests, due to the complex nature by which the eccentricity is calculated, the eccentricity as recommended by Astaneh and specified in the AISC manual should be substituted for the single plate eccentricity, $e$, within the Australian bearing and tear out equations.
Single Plate: Flexural Yielding

From the study of experimental data of the flexural yielding behavior of the single plate as presented in Section 7.3 and the design method comparison of Section 7.4, a modified design consideration for flexural yielding of the single plate is recommended. Plastic behavior of the single plate is recognized as the plastic section modulus, $Z_x$, should be used in conjunction with the weld line to bolt line distance, $a$, for calculating the flexural yielding strength of the single plate.
CHAPTER 8 SUMMARY, RECOMMENDATIONS, AND CONCLUSIONS

8.1. SUMMARY AND DISCUSSION OF RESULTS

8.1.1. GENERAL

This research of single plate framing connections with rigid and flexible supports set out to accomplish the following goals: to identify and better understand the behavior of single plate framing connections and improve the AISC LRFD 3rd Edition Manual (AISC 2003) design method for single plate framing connections. To accomplish these goals and establish a reasonable sample size, ten full-scale single plate connection tests using various testing parameters were conducted using a test setup, which accurately modeled field conditions by combining both shear and rotation while imposing a total beam end rotation of 0.03 radians at ultimate loading to the single plate connection. In addition, supplemental tensile and bolt tests were conducted to measure material properties of the steel and connection bolts used in testing and eliminate existing unknown variability.

Results obtained were used in analytical and comparative studies which focused on identifying the most accurate way to predict the ultimate capacity of single plate framing connections. As is common practice in designing single plate framing connections, limit states of the test components define the strength of the single plate framing connection. The investigational studies were conducted to evaluate the various limit states individually. Two particular connection components were investigated: the single plate (particularly flexural yielding assumptions and considerations) and the connection bolts as these are two of the most critical components in the connection. Particular focus was placed upon the connection bolts as shear rupture of the connection bolts was designed to
be and in most experimental tests found to be the primary failure mode or limit state. Questions were raised as to the applicability of the following considerations:

- The necessity for the bolt group action factor, which is a 20 percent reduction in strength for non-uniform load distributions in connections consisting of two or more bolts.

- The necessity and extent to which eccentric shear considerations should be placed upon the bolt group.

In an attempt to improve the currently accepted design procedure accepted by the American Institute of Steel Construction, multiple design methods from researchers within the United States and design methods specified by other countries were investigated. Differences in design methodologies were identified in terms of calculation methods and limit states calculated. From the comparisons, design criteria, which most accurately predicted experimentally measured strengths, were identified.

From the various experimentation and analytical and comparative investigations conducted, the following subsections present the major findings and recommendations from this series of research.

8.1.2. SINGLE PLATE FRAMING CONNECTION BEHAVIOR

_Ultimate Capacity (Test Parameters)_

Identified in Chapter 5 were the primary trends and observations of the experimental behavior of the full-scale single plate framing connection tests conducted. Trends were associated with the various test parameters. Eight of ten tests failed via shear rupture of the connection bolts. The strength of the connections increased with the number of connection bolts. However, the observed ultimate capacities of the tests conducted were
not equal to the predicted bolt group shear strength (based upon supplemental bolt shear tests conducted by Gelo (2003)). This is in part due to eccentric shear and the rotational behavior of the connections, which are affected by the support condition and other test parameters. Comparing tests with a similar number of connection bolts indicates that connections with a flexible support condition failed at lesser sustained shear than connections with a rigid support condition.

Tests with a similar number of bolts and support condition were compared to investigate the effect of the hole type. Analysis of the experimental connection rotation stiffness at various applied loads indicated that for both the rigid and flexible supported tests, the test beam rotated less in the presence of short-slotted holes. In addition, short-slotted holes were found to allow for greater absolute girder, girder web, single plate, and bolt rotation. However, in terms of ultimate capacity, the behavior was approximately the same for both hole types.

One distinguishable test parameter included is that this series of research was the use of a simulated slab restraint with flexible supported tests with short-slotted holes. Comparing results of tests with a simulated slab restraint to tests which utilized the same connection configuration, the ultimate capacity of the connection was increased by approximately 5 percent with the addition of the simulated slab restraint.

**Bolt Eccentricity**

For rigid supported connections, the bolt eccentricity begins excessively positive (within the span of the supported beam) and quickly stabilizes to a specific positive eccentricity. For single plate connections with a flexible support condition, the bolt
eccentricity begins excessively negative and then stabilizes to a certain magnitude opposite of the supported beam. For flexible supported tests with a simulated slab restraint, the bolt eccentricity was found to be smaller in magnitude than flexible supported tests without a simulated slab restraint.

**Rotational behavior**

The point of rotation or rotational neutral axis of single plate connections can significantly affect connection capacity and behavior. For rigid supported tests, the rotational neutral axis was found to closely coincide with the centroid of the bolt group. However, for tests with a flexible support condition with or without a simulated slab restraint, test results indicated that the point of rotation does not coincide with the centroid of the bolt group. Intuitively, tests with a flexible support condition were subjected to combined rotations of both the supporting girder and supported test beam. For flexible supported tests, the rotational neutral axis was found to be below the centroid of the bolt group. For flexible supported tests with a simulated slab restraint, the rotational axis was shifted considerably above the centroid of the bolt group toward the simulated slab restraint. Due to this finding, Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) which is used in calculating the effective number of bolts for an eccentrically loaded connection may become inapplicable for certain connections. The AISC table is based upon the instantaneous center of rotation of the bolt group. The table appears to be based upon the bolts being in a rigid support condition such that the additional rotation imparted by the girder of a flexible supported condition would not be included.
8.1.3. BOLT GROUP SHEAR STRENGTH

The AISC LRFD 3rd Edition Manual (AISC 2003) design method was found to be the most accurate design method in predicting the bolt group shear capacity for single plate. The bolt group shear capacity was evaluated taking into account the bolt group action factor and eccentric shear considerations. The following considerations were found to be most accurate for predicting the bolt group shear capacity:

- The bolt group action factor is appropriate for single plate framing connections of either a rigid or flexible support condition consisting of three or more connection bolts.
- The following conclusions and recommendations were found for eccentric shear considerations:
  - For tests with a rigid support condition, eccentric shear considerations are unnecessary for connections consisting of more than three connection bolts.
  - The nominal LRFD calculated bolt group shear strength calculated with Astaneh AISC predicted bolt eccentricities more accurately predicted the ultimate strength of two bolt flexible supported connections and conservatively predicted the ultimate strength of other connections tested.

8.1.4. ASTANEH BOLT ECCENTRICITY FORMULAS

The AISC LRFD 3rd Edition Manual (AISC 2003) is based upon bolt eccentricity considerations as recommended by Astaneh. From Section 7.5, the bolt eccentricity
formulas were plotted alongside experimentally obtained bolt eccentricity results. The general trend is that the bolt eccentricity equations specified in the AISC manual under predicted measured test bolt eccentricities. Following the recommendations presented in Section 8.1.3, the bolt group shear capacity of the connection (using considerations from Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003)) is more accurately predicted using the AISC manual bolt eccentricity equations as opposed to the measured experimental bolt eccentricity. The investigational study showed that the predicted theoretical eccentricities are smaller in magnitude and as such generate a greater number of effective bolts which better approximate experimentally measured bolt group shear capacities. Equations were proposed to better predict the eccentricity of the connection, but this does not improve the bolt group shear capacity prediction.

8.1.5. SINGLE PLATE FLEXURAL YIELDING STRENGTH

Indicated by measured strains, single plate flexural yielding behavior ranged from elastic behavior at lower applied shears to plastic behavior at higher applied shears. From the compared design methods and the analytical and comparative study from Chapters 6 and 7, it was concluded that the AISC flexural yielding were equations needed to be revised. From the analytical and comparative investigations conducted, the flexural yielding strength of single plate is more accurately predicted with plastic behavioral assumptions. As such, the plastic section modulus, \( Z_x \), and single plate eccentricity assumed to be from the bolt line to weld line distance, \( a \), should be used in calculating the flexural yielding strength of the single plate.
8.2. PROPOSED DESIGN PROCEDURE

The following design procedure utilizes the AISC LRFD 3rd Edition Manual (AISC 2003) as a basis for the design of single plate framing connections. Proposed recommendations recognize the necessity for a hierarchy of limit states as stated in Chapter 3. As such, ductile failure modes, such as bearing of connection bolts within in bolt holes, is desired prior to brittle failure modes, such as bolt shear rupture. As observed from the experimental full-scale tests, bearing of the connection bolts in the single plate, shear yielding, and flexural yielding of the single plate will occur prior to failure of the connection. This ductility of the single plate allows for the beam end rotation imposed by the supported test beam. As such, the ductility of the single plate connection must be maintained.

From the experimental findings and analytical and comparative investigational studies, the following design procedure/recommendations are proposed. The following design procedure states each required design calculation and details any new provisions that were proposed. The following subsections present the strength limit states within the LRFD path of the connection as discussed in Section 4.2.1. In addition, the provisions for the weld thickness, single plate thickness, and geometric dimensional considerations specified within the AISC LRFD 3rd Edition Manual (AISC 2003) remain unchanged and are recommended for use with the following proposed design procedure.

Within the following subsections, design recommendations pertaining to the design of the supported beam, supported beam web, and supporting girder remain unchanged as this research program focused primarily on the connection bolts, single plate, and weldment.
8.2.1. SUPPORTED BEAM

For the supported beam, the flexural strength, as calculated using LRFD Section B5 and LRFD Appendix F1, should be calculated.

8.2.2. SUPPORTED BEAM WEB

*Shear Strength*

♦ Use Equation 3.13 (LRFD Section F2)

*Bearing and Tear Out*

♦ Use Equations 3.15 and 3.16 (LRFD Section J3.10 Equation J3-2a)

*Block Shear (Coped Beam Only)*

♦ If the supported beam is coped, use Equations 3.17 and 3.18 (LRFD Section J4.3 Equations J4-3a and J4-3b)

8.2.3. CONNECTION BOLTS

The connection bolts are found to experience both a combination of shear and moment as transferred from the beam end rotation of the supported test beam.

♦ Use Equation 3.1 (LRFD Section J3.6) to calculate the single bolt single shear strength

♦ Calculate the bolt eccentricity using Equations 3.24 to 3.27 (presented on pages 10-112 and 10-113 of the AISC LRFD 3rd Edition Manual (AISC 2003). The following recommendations should be met:

  – Eccentric shear considerations, using the Astaneh bolt eccentricity equations, are necessary for two and three bolt connections with either a rigid or flexible support condition.
For tests with a rigid support condition, eccentric shear considerations should not be applied for connections consisting of more than three connection bolts.

Use Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) and calculate the effective number of bolts in the connection, $C$. Multiply the effective number of bolts by the single bolt single shear strength from Equation 3.1.

8.2.4. SINGLE PLATE

*Shear Yielding*

The shear yielding capacity has been revised to be calculated using Equation 3.44 presented in Section 3.4.5 ‘Shear Yielding’

*Shear Rupture*

Use Equation 3.42 (LRFD Section J4.1 Equation J4-1)

*Flexural Yielding*

The flexural yielding capacity should be calculated as follows:

$$\phi V_n = \phi \left( \frac{M_n}{a} \right)$$

where: $\phi = 0.90$

$$M_n = F_y Z_x$$

where: $Z_x = \text{plastic section modulus of the single plate, which is defined as follows:}$

$$Z_x = \frac{t_p L_p^2}{4}$$

Note: The terminology presented above is defined in Section 3.3.2 ‘Single Plate Bending’
**Bearing and Tear Out**

- The bearing and tear out design strength of the single plate has been revised to resemble the Australian design method for calculating the bearing and tear out strength of the single plate connection. As presented in Equation 3.49, the bearing and tear out strength should be calculated with the following modification. For rigid supported connections, the eccentricity specified in the bolt interaction factors should be calculated as the Astaneh predicted bolt eccentricity for rigid supported tests, specified as Equations 3.24 or 3.25. The Australian eccentricity consideration for flexible supported tests remains unchanged.

**Block Shear**

- Use Equations 3.11 and 3.12 (LRFD Section J4.3 Equations J4-3a and J4-3b)

8.2.5. WELDMENT

The weldment is subjected to a combination of shear and moment as imparted by the beam end rotation to the single plate. To account for this combination of forces, both weld rupture due to eccentric shear and base metal failure at the weldment as defined in LRFD Section J2 should be calculated for. These design considerations are also presented in Section 3.3.4 of this thesis.

8.2.6. SUPPORTING MEMBER

For the supporting girder as used in flexible supported tests, flexure considerations of the supporting girder should be calculated for. The flexure calculations are specified in LRFD Section B5 and LRFD Appendix F1.
8.3. CONCLUSIONS

8.3.1. BOLT ECCENTRICITY

- Depending on the magnitude of the eccentricity and number of bolts, the eccentricity can be a major influence on strength of the bolts or be just a minor consideration in the connection design. The combined moment and shear of the bolts reduce the strength of the connection. As concluded from this series of research, two and three bolt connections should take into account eccentric shear considerations. The magnitude of eccentricity for connections with more than three connection bolts was found to be insignificant as calculating the direct shear strength of the connection was found to be adequate.

8.3.2. SNUG-TIGHT BOLTS

- Snug-tight bolts were found to prevent slippage in the connection up to an applied shear great enough to overcome the friction incurred by the connection bolts. After slippage occurred, the connections were found to enter into bearing.

8.3.3. ROTATIONAL BEHAVIOR OF THE CONNECTION

- The rotational behavior of the single plate connection was found to be complex in that the rotational behavior is affected by various design aspects including but not limited to support condition and incorporation of the simulated slab restraint.

- As determined from the experimental investigations conducted, the rotational neutral axis of the connection does not coincide with the centroid of the bolt group. The rotational neutral axis shifts up or down as a result of the connection components/parameters such as support condition.
• One important observation made from testing is that for flexible supported tests, the additional rotation imparted by the supporting girder significantly reduces the capacity of the connection as the connection bolts must sustain additional torque.

8.3.4. DUCTILITY OF THE CONNECTION

• The beam end rotation imparted by the end of the supported beam must be sustained by the single plate. The ductility of the connection is defined as the bearing and yielding that occurs within the connection, specifically the single plate.

• To facilitate ductility in the connection, the single plate should be manufactured from a lower grade of steel and must satisfy the thickness requirements specified in the AISC LRFD 3rd Edition Manual (AISC 2003).

• The hierarchy of limit states, which details that ductile limit states should occur prior to brittle failure modes, should be maintained.

8.3.5. SINGLE PLATE FLEXURAL YIELDING BEHAVIOR

• Single plates were found to behave differently at different levels of applied shear. At lesser levels of applied shear, single plates were found to behave elastically. For higher levels of applied shear, plastic behavior was observed and experimentally measured.

8.3.6. HOLE TYPE

• No significant change in ultimate capacity was noted from the use of standard holes as opposed to short-slotted holes.
• Rotational behavior of the connections varied slightly in terms of varying component rotations.

8.3.7. SIMULATED SLAB RESTRAINT

• Incorporation of the simulated slab restraint with flexible supported connections affected the rotational behavior of the connection. The simulated slab restraints acts as a fixed point where the connections rotate about.

• Connection bolts furthest from the simulated slab restraint must sustain a greater amount of rotational torque. In addition, the single plate must allow for this behavior.

• In terms of connection eccentricity, the simulated slab restraint was found to lessen the magnitude of the eccentricity of the connection.

8.4. NEED FOR FUTURE RESEARCH

From this series of research, the following needs for future research were indicated.

• Rotational behavior of the connections tested was found to be excessively complex in nature. From this research, trends of the general location of the rotational neutral axis were documented, yet the ability to accurately predict the location of the rotational neutral axis has not been developed. Future research should be conducted to identify and create the means of predicting the exact location of the rotational neutral axis. Parameters, such as off-axis bolt groups, should be incorporated to determine the corresponding effects.
• Future research should also be conducted to evaluate the applicability of Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003), which is used in calculating the effective number of bolts for eccentrically loaded connections, for single plate framing connections which incorporate various test parameters such as a flexible support condition.

• Additional flexible supported connection tests with more than three connection bolts should be conducted to attain additional data for failure modes. From additional testing, a determination could be made as to the applicability of eccentricity considerations for flexible supported tests.

• The majority of the current research (similar to this research) is limited to test configurations that incorporate one single plate framing connection at one of the supported beam. Within this series of tests, the opposite end of the test beam was supported by a roller, which is not typically found in the framing industry. Perhaps additional observations and trends could be observed by conducting flexible supported tests with a single plate framing connection at either end of the supported beam. Rigid support conditions should not be used as catenary forces would develop in the supported beam and consequently single plates.


ASTM A370-03, Standard Test Methods and Definitions for Mechanical Testing of Steel Products, American Society for Testing and Materials, West Conshohocken, PA.


Kulak, G.L. (2003), *High Strength Bolts A Primer for Structural Engineers*, American Institute of Steel Construction, Chicago, IL.

Lewis, B.E. (1994), “Edge Distance, Spacing, and Bearing in Bolted Connections,” *Draft Report*, Civil Engineering Department, Oklahoma State University, Stillwater, OK.


APPENDIX A

SAMPLE CALCULATIONS:

- BEAM END ROTATION

- DESIGN METHOD EXAMPLE 1

- DESIGN METHOD EXAMPLE 2
**Beam End Rotation Calculation:**

The following is a derivation of the theoretical calculation used to predict the beam end rotation at various loadings of test beams used in the ten full-scale tests. From the fundamental theories including the slope-deflection method as presented in Salmon and Johnson (1997).

SIMPLY SUPPORTED BEAM WITH SYMMETRICALLY APPLIED CONCENTRATED LOADS

- Positive deflections of simply supported beam

\[
M_a = M_{Fa} + \frac{4EI}{L} \theta_a + \frac{2EI}{L} \theta_b \quad \text{(EQ A1)}
\]

\[
M_b = M_{Fb} + \frac{2EI}{L} \theta_a + \frac{4EI}{L} \theta_b \quad \text{(EQ A2)}
\]
where:  \( E \) = Young’s Modulus of Elasticity of the supported beam

\( I \) = Moment of Inertia of the supported beam about the strong axis

- Combining and simplifying the above equations

\[
\frac{6EI}{L} \theta_a = 2(M_a - M_{Fa}) - (M_b - M_{Fb}) \quad \text{(EQ A3)}
\]

\[
\frac{6EI}{L} \theta_b = -(M_a - M_{Fa}) + 2(M_b - M_{Fb}) \quad \text{(EQ A4)}
\]

- Subtracting Equation A3 from A4

\[
\frac{6EI}{L} (\theta_a - \theta_b) = 3(M_a - M_{b}) - 3(M_{Fa} - M_{Fb}) \quad \text{(EQ A5)}
\]

- Due to Symmetry

\[
M_b = -M_a \quad \text{(EQ A6)}
\]

\[
M_{Fb} = -M_{Fa} \quad \text{(EQ A7)}
\]

\[
\theta_b = \theta_a \quad \text{(EQ A8)}
\]

Therefore:

\[
\frac{2EI}{L} \theta_a = M_a - M_{Fa} \quad \text{(EQ A9)}
\]

or because this test beam is assumed to be a pinned-pinned connection, Equation A9 can be rewritten as:

\[
\theta_a = \frac{M_{Fa}L}{2EI} \quad \text{(EQ A10)}
\]
For a two point loading configuration (obtained from AISC LRFD 3rd Edition Manual (AISC 2003) Table 5-17 Shear, Moments, and Deflections Case 17: Beam Fixed at Both Ends – Concentrated Load at Any Point)

\begin{equation}
M_{fa} = \frac{Pa^2b}{L^2} + \frac{Pab^2}{L^2}
\end{equation}

(EQ A11)

where: \( b = L - a \)

which can be rewritten as:

\begin{equation}
M_{fa} = \frac{P}{L^2}(a^2b + ab^2)
\end{equation}

(EQ A12)

Combining Equations A10 and A12, Equation A13, which is used to calculate beam end rotation for a simply supported beam with two symmetrically applied concentrated load, is obtained

\begin{equation}
\theta_a = \frac{P(a^2b + b^2a)}{2EIL}
\end{equation}

(EQ A13)
Design Method Example Calculation 1:

The following is a design example. A single plate framing connection should be designed to sustain 40kips \((R_u = 40\text{kips})\). The connection is to be a rigid supported connection where a supported beam will frame into a supporting column flange. The test beam is assumed to be a W16x50. Connection components should be designed to sustain the applied shear. Recommended dimensional standards should be incorporated into the design. The hierarchy of limit states should be maintained. The following criteria must be met:

Given:

**Connection Bolts (A325-N):**

\[ F_v = 48\text{ksi} \]
\[ d_b = 0.75\text{in} \]

**Single Plate (A36):**

\[ F_y = 36\text{ksi} \qquad F_u = 58\text{ksi} \]
\[ a = 3\text{in} \qquad L_e v ; L_e h = 1.5\text{in} \qquad s = 3.0\text{in} \]

Single plate width, \( b = a + L_e h = 4.5\text{in} \)

Hole Type: Short-slotted

\[ d_h = d_b + (1/16) = 0.8125\text{in} \]

**W16x50 Supported Beam (A992):**

\[ F_y = 50\text{ksi} \qquad F_u = 65\text{ksi} \]
Hole Type: Standard

d_b = d_b + (1/16) = 0.8125in

Distance between vertical edge of beam and vertical centerline of bolts = 1.5in

**Weldment:**

Two fillet welds manufactured from E70XX electrodes (F_{XX} = 70ksi)

---

**Design Calculation:**

- Determine required number of bolts, plate length, and plate thickness

- Calculate the required number of connection bolts. Because eccentricity is a consideration, the effective number of bolts, C, should be calculated:

\[ \phi V_n = \phi (F_v A_b)C \]

Solving for C:

\[ C = \frac{\phi V_n}{\phi (F_v A_b)} = \frac{40}{0.75(48*0.4418)} = 2.51 \]

where:  \( \phi = 0.75 \)

\[ A_b = \frac{\pi d_b^2}{4} = \frac{\pi (0.75)^2}{4} = 0.4418\text{in}^2 \]
Calculate the bolt eccentricity for a single plate connection with rigid support condition and short-slotted holes is calculated as follows:

\[
e_b = \left[ \left( \frac{2n}{3} \right) - a \right] = \left[ \left( \frac{2(3)}{3} \right) - 3 \right] = 1.0 \text{in}
\]

Because the effective number of bolts is 2.51, **try three bolts.**

Using the calculated bolt eccentricity and three bolts, enter into Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) and determine if three bolts are adequate.

- For three bolts in one vertical row with 2in of eccentricity, \(C = 2.23\)
- For three bolts in one vertical row with 3in of eccentricity, \(C = 1.75\)
- Using linear interpolation, \(C = 2.71 \geq 2.51\) (OK)

Length of the single plate

\[
L_p = (n-1)s + 2L_{ev} = (3-1)*3 + (2*1.5)
\]

\[
L_p = 9.0 \text{in}
\]

\[
L_p > \frac{T}{2} = \frac{13.625}{2} = 6.8 \text{in} \quad \text{(OK)}
\]
- **Thickness of single plate**

As recommended in the AISC manual, the single plate must satisfy the following requirements:

\[
t_{p_{\text{min}}} \left( \frac{L_p}{K} \right) \geq \frac{1}{4} \text{ in} \quad \text{and} \quad t_{p_{\text{max}}} = \frac{d_b}{2} + \frac{1}{16} \geq t_{p_{\text{min}}}
\]

\[
t_{p_{\text{min}}} \left( \frac{9}{0.8} \right) \geq \frac{1}{4} \text{ in} \quad \text{and} \quad t_{p_{\text{max}}} = \frac{0.75}{2} + \frac{1}{16} \geq t_{p_{\text{min}}}
\]

where: \( K = 0.8 \) (from Part 9 of the AISC LRFD 3rd Edition Manual (AISC 2003))

Calculating the plate aspect ratio as \( L_p/2a \), \( t_{p_{\text{min}}} \) must satisfy the following requirement:

\[0.25\text{in} \leq t_p \leq 0.4375\text{in}\]

Assuming shear yielding of the plate will control, choose \( t_p \):

**Shear Yielding**

\[
\phi R_n = \phi 0.6 F_y \left[ (n-1)s + L_{cr} \right] f_p
\]

\[
\phi R_n = 0.9 \times 0.6 \times 36 \left[ (3-1) \times 3 + 1.5 \right] f_p
\]

\[t_p \geq \frac{40}{145.8} = 0.274\text{in}\]

**Choose** \( t_p = 0.375\text{in} \geq 0.274\text{in} \) (OK)
- Verify applicable limit states for chosen parameters:

- Supported Beam Web

**Shear Strength**

\[ \phi V_n = \phi 0.6 F_s A_w = 0.9 \times 0.6 \times 50 \times 6.19 = 167.1 \text{kips} \geq 40 \text{kips} \text{(OK)} \]

where: \( \phi = 0.90 \)

\[ A_w = d t_w = 16.3 \times 0.38 = 6.19 \text{in}^2 \]

**Bearing and Tear Out**

Because the test beam is uncoped, the clear distance, \( L_c \), between two interior bolt holes will control. As such, \( L_c = s - d_h = 3 - 0.8125 = 2.19 \text{in} > 2d_b \).

For \( L_c > 2d_b \), bearing controls:

\[ \phi V_n = \phi 2.4d_h t_w F_u = 0.75 \times 2.4 \times 0.75 \times 0.38 \times 65 = 33.3 \text{kips} \]

where: \( \phi = 0.75 \)

The bearing and tear out capacity of the supported beam is calculated as follows:

\[ n(\phi V_n) = 3 \times 33.3 = 99.9 \text{kips} \geq 40 \text{kips} \text{(OK)} \]
• Connection Bolts

**Bolt Group Shear Capacity**

\[ \phi V_n = \phi (F_v A_h)C = 0.75 \times (48) \times 0.4418 \times 2.71 = 43.1 \text{kips} \geq 40 \text{kips (OK)} \]

where:  \( \phi = 0.75 \)

\( C = 2.71 \)

• Single Plate

**Shear Yielding**

\[ \phi R_n = \phi 0.6 F_y \left[ (n - 1)s + L_{cy} \right] \]

\[ \phi R_n = 0.9 \times 0.6 \times 36 \times (3 - 1) \times 3 + 1.5 \times 0.375 = 54.7 \text{kips} \geq 40 \text{kips (OK)} \]

where:  \( \phi = 0.90 \)

**Shear Rupture**

\[ \phi V_n = \phi 0.6 F_u A_n = 0.75 \times 0.6 \times 58 \times 2.39 = 62.4 \text{kips} \geq 40 \text{kips (OK)} \]

where:  \( \phi = 0.75 \)

\[
A_n = \left( L_p - n \left( d_h + \frac{1}{16} \right) \right) t_p
\]

\[
A_n = \left( 9 - \left( 3 \times \left( 0.8125 + \frac{1}{16} \right) \right) \right) 0.375 = 2.39 \text{in}^2
\]
**Block Shear**

- First check if the following inequality is true:

$$F_u A_{nt} \geq 0.6 F_u A_{nv}$$

where: $58 \times 0.3984 = 23.1 \text{kips} < 0.6 \times 58 \times 1.99 = 69.3 \text{kips}$

- Using Equation 3.12, the block shear strength is calculated as follows:

$$\phi V_n = \phi [0.6 F_u A_{nv} + F_y A_{gt}] \leq \phi [0.6 F_u A_{nv} + F_u A_{nt}]$$

$$\phi V_n = 0.75 [(0.6 \times 58 \times 1.99) + (36 \times 0.563)] \leq 0.75 [(0.6 \times 58 \times 1.99) + (58 \times 0.3984)]$$

$$\phi V_n = 67.1 \text{kips} \leq 69.27 \text{kips} \text{ AND } \phi V_n = 67.1 \text{kips} \geq 40 \text{kips} (\text{OK})$$

where: $\phi = 0.75$

$$A_gv = (L_p - L_{cv}) t_p = (9 - 1.5) \times 0.375 = 2.813 \text{in}^2$$

$$A_{gt} = (L_{ch}) t_p = (1.5) \times 0.375 = 0.563 \text{in}^2$$

$$A_{nv} = \left( (L_p - L_{cv}) - \left( (n - 0.5) \left( d_h + \frac{1}{16} \right) \right) \right) t_p$$

$$A_{nv} = \left( (9 - 1.5) - \left( (3 - 0.5) \left( \frac{13}{16} + \frac{1}{16} \right) \right) \right) \times 0.375 = 1.99 \text{in}^2$$

$$A_{nt} = \left( L_{ch} - 0.5 \left( d_h + \frac{1}{16} \right) \right) t_p$$

$$A_{nt} = \left( 1.5 - 0.5 \left( \frac{13}{16} + \frac{1}{16} \right) \right) \times 0.375 = 0.3984 \text{in}^2$$
Eccentricity Considerations (rigid support condition)

-For the single plate:

\[ e = e_b = 1.0\text{in} \]

Bearing

\[
\phi V_{df} = \text{Min} \begin{cases} 
\phi(3.2) f_{d_h} f_{wi} = 0.9 \times 3.2 \times 0.375 \times 0.75 \times 58 = 47.0\text{kips} \\
\phi(3.2) f_{d_v} f_{ww} = 0.9 \times 3.2 \times 0.38 \times 0.75 \times 65 = 53.4\text{kips}
\end{cases}
\]
\[ \phi V_{df} = 47.0 \text{kips} \]

where: \[ \phi = 0.9 \]

\[ f_{ui} = \text{design tensile strength of the single plate} \]

\[ f_{uw} = \text{design tensile strength of the supported beam web} \]

- Tear out

- Horizontal Tear out

\[ \phi V_{eh} = \min \left\{ \phi(1.2)r_p L_c f_{ui} = 0.75 \times 1.2 \times 0.375 \times 1.094 \times 58 = 21.4 \text{kips} \right\} \]

where: \[ \phi = 0.75 \]

\[ L_{c,\text{single plate}} = \left( L_{eh} \right) - \frac{d_h}{2} = 1.5 - \frac{0.8125}{2} = 1.094 \text{in} \]

\[ L_{c,\text{beam}} = \left( L_{eh} \right) - \frac{d_h}{2} = 1.5 - \frac{0.8125}{2} = 1.094 \text{in} \]

- Vertical Tear out

It should be noted that because the supported test beam is not coped, then vertical tear out is not a consideration.

\[ \phi V_{ev} = \min \left\{ \phi(1.2)r_p L_c f_{ui} = 0.75 \times 1.2 \times 0.375 \times 1.094 \times 58 = 21.4 \text{kips} \right\} \]

where: \[ \phi = 0.75 \]

\[ L_{c,\text{single plate}} = \left( L_{ev} \right) - \frac{d_h}{2} = 1.5 - \frac{0.8125}{2} = 1.094 \text{in} \]
Flexural Yielding

\[ \phi V_n = \phi \left( \frac{M_n}{a} \right) = 0.9 \left( \frac{273.38}{3} \right) = 82.0 \text{kips} \geq 40 \text{kips (OK)} \]

\[ M_n = F_y Z_x = 36 \times 7.594 = 273.38 \text{ kips - in} \]

where: \( \phi = 0.90 \)

\[ Z_x = \frac{t_p L_p^2}{4} = \frac{375 \times 9^2}{4} = 7.594 \text{ in}^3 \]

- Weldment
  
  As recommended in the AISC manual, the weldment must satisfy the following requirement:

  \[ D \geq 0.75 t_p \]

  \[ D \geq 0.75 \times 0.375 \]

  \[ D \geq 0.28 \text{ in} \]

  **Choose \( D = 0.3125 \text{ in} \)** to develop the plate.

Size Verification

Verify that the chosen weld size is adequate to develop the plate.

\[ D \geq \frac{\phi R_s}{C \ C_1 l} \]

\[ D = 5.0 \geq \frac{40}{2.224 \times 1.0 \times 9} = 2.0 \text{ (OK)} \]
where:  
\[ C = 2.224 \text{ (found from AISC Tables 8-5)} \]
\[ C_1 = 1.0 \text{ (found from AISC Tables 8-4)} \]
\[ D = \text{number in sixteenths of an inch in weld leg length} \]
\[ \phi R_n = \text{applied shear} \]

**Base Metal at Weld**

The shear rupture strength of the weld (represented as the shear strength per longitudinal inch of weld) is calculated for as follows:

\[ \phi R_n = \phi \cdot 0.6 F_u t_p = 0.75 \cdot 0.6 \cdot 58 \cdot 0.375 = 9.79 \text{kips/in} \]

where:  
\[ \phi = 0.75 \]
\[ F_u = \text{ultimate tensile strength of the single plate} \]

**Weld Design Strength**

Written in terms to represent the weld strength per longitudinal inch of weld, the following calculation was made:

\[ \phi R_n = 1.392 D n_{ welds} = 1.392 \cdot 5 \cdot 2 = 13.92 \text{kips/in} \]

where:  
\[ n_{ welds} = \text{number of longitudinal fillet welds} \]

From the previous calculations, the combined effect of shear and moment on the weld is defined as follows:

\[ \phi R_n = C C_1 D l \left( \frac{9.79}{13.92} \right) = 2.224 \cdot 1.0 \cdot 5 \cdot \phi \left( \frac{9.79}{13.92} \right) = 70.4 \text{kips} \]
The shear strength of the weld was then found by multiplying the lesser of 9.79kips/in and 13.92kips/in by the longitudinal length of weld, $l$, which yields a shear strength of 88.1kips. Comparing 88.1kips to 70.4kips, the lesser of these two values, 70.4kips will be the design strength for the weldment.

$$\phi R_n = 70.4\text{kips} \geq 40\text{kips} \text{ (OK)}$$

♦ Summary:

The following detail summarizes the plate dimensions selected. In addition, a tabulated form the limit states evaluated will be presented.

**Limit States:**

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Design Strength (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts</td>
<td>Bolt Shear ($e_b = 1$ in)</td>
<td>43.1</td>
</tr>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>54.7</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>62.4</td>
</tr>
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<td></td>
<td>Flexural Yielding</td>
<td>82.0</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tear out</td>
<td>64.2</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>67.1</td>
</tr>
<tr>
<td>Weldment</td>
<td>Weld Strength</td>
<td>125.3</td>
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<td></td>
<td>Base Metal</td>
<td>88.1</td>
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<tr>
<td></td>
<td>Combined</td>
<td>70.4</td>
</tr>
<tr>
<td>Beam Web</td>
<td>Web Shear</td>
<td>167.1</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tear out</td>
<td>99.9</td>
</tr>
</tbody>
</table>
Connection Detail:

CONNECTION DETAIL DESIGN EXAMPLE 1
Design Method Example Calculation 2:

The following is a design example. A single plate framing connection should be designed to sustain 25kips ($R_u = 25\text{kips}$). The connection is to be a flexible supported connection where a supported beam will frame into the supporting girder web. The test beam is assumed to be a W16x50. Connection components should be designed to sustain the applied shear. Recommended dimensional standards should be incorporated into the design. The hierarchy of limit states should be maintained. The following criteria must be met:

Given:

**Connection Bolts (A325-N):**

\[ F_v = 48\text{ksi} \]
\[ d_b = 0.75\text{in} \]

**Single Plate (A36):**

\[ F_y = 36\text{ksi} \quad F_u = 58\text{ksi} \]
\[ a = 3\text{in} \quad L_{ev} ; L_{eh} = 1.5\text{in} \quad s = 3.0\text{in} \]

Single plate width, \[ b = a + L_{eh} = 4.5\text{in} \]

Hole Type: Short-slotted

\[ d_h = d_b + (1/16) = 0.8125\text{in} \]

**W16x50 Supported Beam, Coped (A992):**

\[ F_y = 50\text{ksi} \quad F_u = 65\text{ksi} \]
\[ t_w = 0.38\text{in} \quad d = 16.3\text{in} \quad T = 13.625\text{in} \]
\[ d_c = 2.0\text{in} \quad L_c = 3.0\text{in} \]

Hole Type: Standard

\[ d_h = d_b + \frac{1}{16} = 0.8125\text{in} \]

Distance between vertical edge of beam and vertical centerline of bolts = 1.5in

Distance between horizontal edge of beam cope and centerline of first bolt hole = 2.5in

**Weldment:**

Two fillet welds manufactured from E70XX electrodes \((F_{EXX} = 70\text{ksi})\)

---

**Design Calculation:**

- Determine required number of bolts, plate length, and plate thickness

- Calculate the required number of connection bolts. Because eccentricity is a consideration, the effective number of bolts, \(C\), should be calculated:

  \[ \phi V_n = \phi (F_v A_b) C \]

Solving for \(C\):

\[ C = \frac{\phi V_n}{\phi (F_v A_b)} = \frac{25}{0.75(48*0.4418)} = 1.57 \]

where: \(\phi = 0.75\)

\[ A_b = \frac{\pi d_b^2}{4} = \frac{\pi (0.75)^2}{4} = 0.4418\text{in}^2 \]
• Calculate the bolt eccentricity for a single plate connection with flexible support condition and short-slotted holes is calculated as follows:

\[
e_b = \left\lfloor \left( \frac{2n}{3} \right) - a \right\rfloor \geq a
\]

\[
e_b = \left\lfloor \left( \frac{2(3)}{3} \right) - 3 \right\rfloor \geq 3 = 3.0\text{in}
\]

Based upon findings, **try three bolts**.

Using the calculated bolt eccentricity and three bolts, enter into Table 7.17 of the AISC LRFD 3rd Edition Manual (AISC 2003) and determine if three bolts are adequate.

– For three bolts in one vertical row with 3in of eccentricity, \(C = 1.75 \geq 1.57\) (OK)

• Length of the single plate

\[
L_p = (n - 1)s + 2L_{ev} = (3 - 1)*3 + (2*1.5)
\]

\[
L_p = 9.0\text{in}
\]

\[
L_p > \frac{T}{2} = \frac{13.625}{2} = 6.8\text{in} \quad \text{(OK)}
\]
• Thickness of single plate

As recommended in the AISC manual, the single plate must satisfy the following requirements:

\[
\begin{align*}
\frac{t_p}{K} & \geq \frac{1}{4} \text{ in} \quad \text{and} \quad t_p \geq \frac{d_e}{2} + \frac{1}{16} \geq t_p^\text{min} \\
\sqrt{\frac{F_y}{K}} & \geq \frac{1}{4} \text{ in} \quad \text{and} \quad t_p \geq \frac{0.75}{2} + \frac{1}{16} \geq t_p^\text{min}
\end{align*}
\]

where: \( K = 0.8 \) (from Part 9 of the AISC LRFD 3rd Edition Manual (AISC 2003))

Calculating the plate aspect ratio as \( \frac{L_p}{2a} \), \( t_p^\text{min} \) must satisfy the following requirement:

\[
0.25 \text{ in} \leq t_p \leq 0.4375 \text{ in}
\]

From design example 1, we know that after the bolt group capacity the shear yielding capacity of the plate controls. Assuming shear yielding of the plate will control, choose \( t_p \):

**Flexural Yielding**

\[
\phi R_n = \phi 0.6 F_y \left[ (n-1)s + L_e \right] t_p
\]

\[
\phi R_n = 0.9 \times 0.6 \times 36 \left[ (3-1)3 + 1.5 \right] t_p
\]

\[
t_p \geq \frac{25}{145.8} = 0.17 \text{ in}
\]

Choose \( t_p = 0.25 \text{ in} \) which satisfies the above requirements
- Verify applicable limit states for chosen parameters:

- Supported Beam Web

**Shear Strength**

\[
\phi V_n = \phi 0.6 F_s A_w = 0.9 \times 0.6 \times 50 \times 6.19 = 167.1 \text{kips} \geq 25 \text{kips (OK)}
\]

where:  
\[
\phi = 0.90
\]
\[
A_w = d t_w = 16.3 \times 0.38 = 6.19 \text{in}^2
\]

**Block Shear**

By inspection block shear of the single plate will control as the thickness of the beam web is greater, the beam is manufactured from a higher grade of steel, and the shear distances are greater than or equal to that of the single plate.

**Bearing and Tear Out**

Because the test beam is uncoped, the clear distance, \( L_c \), between two interior bolt holes will control. As such, \( L_c = s - d_h = 3 - 0.8125 = 2.19 \text{in} > 2d_b \).

For \( L_c > 2d_b \), bearing controls:

\[
\phi V_n = \phi 2.4 d_b t_w F_s = 0.75 \times 2.4 \times 0.75 \times 0.38 \times 65 = 33.3 \text{kips}
\]

where:  
\[
\phi = 0.75
\]

The bearing and tear out capacity of the supported beam is calculated as follows:

\[
n(\phi V_n) = 3 \times 33.3 = 99.9 \text{kips} \geq 25 \text{kips (OK)}
\]
• Connection Bolts

*Bolt Group Shear Capacity*

\[ \phi V_n = \phi (F_v A_h)C = 0.75 \times (48 \times 0.4418) \times 1.75 = 27.8 \text{kips} \geq 25 \text{kips (OK)} \]

where: \( \phi = 0.75 \)

\( C = 1.75 \)

• Single Plate

*Shear Yielding*

\[ \phi R_n = 0.9 \times 0.6 \times 36 \times (3-1) \times 3 + 1.5 \times 0.25 = 36.5 \text{kips} \geq 25 \text{kips (OK)} \]

where: \( \phi = 0.90 \)

*Shear Rupture*

\[ \phi V_n = \phi F_y A_n = 0.75 \times 0.6 \times 58 \times 1.59 = 41.6 \text{kips} \geq 25 \text{kips (OK)} \]

where: \( \phi = 0.75 \)

\[ A_n = \left( L_p - n \left( d_h + \frac{1}{16} \right) \right) t_p \]

\[ A_n = \left( 9 - \left( 3 \times \left( 0.8125 + \frac{1}{16} \right) \right) \right) 0.25 = 1.59 \text{in}^2 \]
**Block Shear**

- First check if the following inequality is true:

\[ F_u A_{nt} \geq 0.6 F_u A_{nv} \text{ where: } 58 \times 0.266 = 15.4 \text{kips} < 0.6 \times 58 \times 1.328 = 46.2 \text{kips} \]

- Using Equation 3.12, the block shear strength is calculated as follows:

\[ \phi V_n = \phi \left[ 0.6 F_u A_{nv} + F_y A_{gt} \right] \leq \phi \left[ 0.6 F_u A_{nv} + F_u A_{nt} \right] \]

\[ \phi V_n = 0.75 \left[ (0.6 \times 58 \times 1.328) + (36 \times 0.375) \right] \leq 0.75 \left[ (0.6 \times 58 \times 1.328) + (58 \times 0.266) \right] \]

\[ \phi V_n = 44.8 \text{kips} \leq 46.2 \text{kips} \text{ AND } \phi V_n = 44.8 \text{kips} \geq 25 \text{kips (OK)} \]

where: \( \phi = 0.75 \)

\[ A_{gv} = (L_p - L_{ev}) s_p = (9 - 1.5) \times 0.25 = 1.875 \text{in}^2 \]

\[ A_{gt} = (L_{ch}) s_p = (1.5) \times 0.25 = 0.375 \text{in}^2 \]

\[ A_{nv} = \left( (L_p - L_{ev}) - \left( (n - 0.5) \left( d_h + \frac{1}{16} \right) \right) \right) \times t_p \]

\[ A_{nt} = \left( L_{ch} - \left( 0.5 \left( d_h + \frac{1}{16} \right) \right) \right) \times t_p \]

\[ A_{nt} = \left( 1.5 - \left( 0.5 \left( \frac{13}{16} + \frac{1}{16} \right) \right) \right) \times 0.25 = 0.266 \text{in}^2 \]
\[
Z_b \left( \phi V_{df} \right) = 1.66(31.3) = 52.0 \text{kips}
\]

\[
\phi V_n = \min \left( n_p \phi V_{ev} \right) = 3(14.3) = 42.9 \text{kips}
\]

\[
n_p Z_{ehl} \left( \phi V_{eh} \right) = 3 \times 0.67 \times 14.3 = 28.7 \text{kips}
\]

\[
\phi V_n = 28.7 \text{kips} \geq 25 \text{kips} \text{(OK)}
\]

where:

\[
\phi V_{df} = 31.3 \text{kips (calculated below)}
\]

\[
\phi V_{eh} = 14.3 \text{kips (calculated below)}
\]

\[
\phi V_{ev} = 14.3 \text{kips (calculated below)}
\]

\[
Z_b = \frac{n_p}{\sqrt{1 + \left( \frac{6e}{s(n_p + 1)} \right)^2}} = \frac{3}{\sqrt{1 + \left( \frac{6(3)}{3(3+1)} \right)^2}} = 1.66
\]

\[
Z_{ehl} = \frac{s(n_p + 1)}{6e} = \frac{3(3+1)}{6(3)} = 0.67
\]

\[
e = \text{eccentricity of the connection defined as follows:}
\]

- Eccentricity Considerations (flexible support condition)

-For the single plate:

\[
e = a = 3.0 \text{in}
\]
• Bearing

\[
\phi V_{df} = \min \left\{ \phi(3.2)q_{d, f_{ui}} = 0.9 * 3.2 * 0.25 * 0.75 * 58 = 31.3 \text{kips} \right. \\
\phi(3.2)q_{w, f_{uw}} = 0.9 * 3.2 * 0.38 * 0.75 * 65 = 53.4 \text{kips} \\
\]

\[\phi V_{df} = 31.3 \text{kips} \]

where: \( \phi = 0.9 \)

\( f_{ui} \) = design tensile strength of the single plate

\( f_{uw} \) = design tensile strength of the supported beam web

• Tear out

- Horizontal Tear out

\[
\phi V_{eh} = \min \left\{ \phi(1.2)q_{e, L_c, f_{ui}} = 0.75 * 1.2 * 0.25 * 1.094 * 58 = 14.3 \text{kips} \right. \\
\phi(1.2)q_{w, L_c, f_{uw}} = 0.75 * 1.2 * 0.38 * 1.094 * 65 = 24.3 \text{kips} \\
\]

where: \( \phi = 0.75 \)

\[ L_{c, \text{single plate}} = \left( L_{eh} \right) - \frac{d_h}{2} = 1.5 - \frac{0.8125}{2} = 1.094 \text{in} \]

\[ L_{c, \text{beam}} = \left( L_{eh} \right) - \frac{d_h}{2} = 1.5 - \frac{0.8125}{2} = 1.094 \text{in} \]

- Vertical Tear out

\[
\phi V_{ev} = \min \left\{ \phi(1.2)q_{e, L_c, f_{ui}} = 0.75 * 1.2 * 0.25 * 1.094 * 58 = 14.3 \text{kips} \right. \\
\phi(1.2)q_{w, L_c, f_{uw}} = 0.75 * 1.2 * 0.38 * 2.094 * 58 = 41.5 \text{kips} \\
\]

where: \( \phi = 0.75 \)

\[ L_{c, \text{single plate}} = \left( L_{ev} \right) - \frac{d_h}{2} = 1.5 - \frac{0.8125}{2} = 1.094 \text{in} \]

\[ L_{c, \text{beam}} = \left( L_{eh} \right) - \frac{d_h}{2} = 2.5 - \frac{0.8125}{2} = 2.094 \text{in} \]
Flexural Yielding

\[ \phi V_n = \phi \left( \frac{M_n}{a} \right) = 0.9 \left( \frac{182.25}{3} \right) = 54.7 \text{kips} \geq 40 \text{kips (OK)} \]

\[ M_n = F_y Z_x = 36 \times 5.0625 = 182.25 \text{kip} \times \text{in} \]

where: \( \phi = 0.90 \)

\[ Z_x = \frac{t_p L_p^2}{4} = \frac{0.375 \times 9^2}{4} = 5.0625 \text{in}^3 \]

- Weldment
  
  As recommended in the AISC manual, the weldment must satisfy the following requirement:

\[ D \geq 0.75 t_p \]

\[ D \geq 0.75 \times 0.25 \]

\[ D \geq 0.1875 \text{in} \]

**Choose** \( D = 0.1875 \text{in} \) to develop the plate.

**Size Verification**

Verify that the chosen weld size is adequate to develop the plate.

\[ D \geq \frac{\phi R_s}{C C_1 l} \]

\[ D = 3.0 \geq \frac{25}{2.224 \times 1.0 \times 9} = 1.25 \text{ (OK)} \]
where:  
\[ C = 2.224 \text{ (found from AISC Tables 8-5)} \]
\[ C_1 = 1.0 \text{ (found from AISC Tables 8-4)} \]
\[ D = \text{number in sixteenths of an inch in weld leg length} \]
\[ \phi R_n = \text{applied shear} \]

*Base Metal at Weld*

The shear rupture strength of the weld (represented as the shear strength per longitudinal inch of weld) is calculated for as follows:

\[
\phi R_n = \phi 0.6 F_u t_p = 0.75 \times 0.6 \times 58 \times 0.25 = 6.53 \text{kips/in}
\]

where:  
\[ \phi = 0.75 \]
\[ F_u = \text{ultimate tensile strength of the single plate} \]

*Weld Design Strength*

Written in terms to represent the weld strength per longitudinal inch of weld, the following calculation was made:

\[
\phi R_n = 1.392 D n_{welds} = 1.392 \times 3 \times 2 = 8.35 \text{kips/in}
\]

where:  
\[ n_{welds} = \text{number of longitudinal fillet welds} \]

From the previous calculations, the combined effect of shear and moment on the weld is defined as follows:

\[
\phi R_n = C C_1 D l \left( \frac{6.53}{8.35} \right) = 2.224 \times 1.0 \times 3 \times 9 \left( \frac{6.53}{8.35} \right) = 47.0 \text{kips}
\]
The shear strength of the weld was then found by multiplying the lesser of 6.53 kips/in and 8.35 kips/in by the longitudinal length of weld, \( l \), which yields a shear strength of 58.7 kips. Comparing 58.7 kips to 47.0 kips, the lesser of these two values, 47.0 kips will be the design strength for the weldment.

\[
\phi R_n = 47.0 \text{kips} \geq 25 \text{kips} \quad \text{(OK)}
\]

♦ Summary:

The following detail summarizes the plate dimensions selected. In addition, a tabulated form the limit states evaluated will be presented.

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<tr>
<td>Weldment</td>
<td>Weld Strength</td>
<td>75.2</td>
</tr>
<tr>
<td></td>
<td>Base Metal</td>
<td>58.8</td>
</tr>
<tr>
<td></td>
<td>Combined</td>
<td>47.0</td>
</tr>
<tr>
<td>Beam Web</td>
<td>Web Shear</td>
<td>167.1</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tear out</td>
<td>99.9</td>
</tr>
</tbody>
</table>
Connection Detail:

PL 1/4 x 4 1/2 x 0'-9" (A36)

Supporting Girder

CONNECTION DETAIL DESIGN EXAMPLE 2
APPENDIX B

TEST SUMMARY FOR:

TEST 1 – S1-RSS-3-A325-N
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S1-RSS-3-A325-N (Test 1A (Rigid, Slotted, Three-Bolt))
Test Date: April 9, 2003
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Three bolt single plate connection with a rigid support condition, short slotted holes, and snug-tight bolts.

Shear Tab
- Plate Size: PL 3/8" x 4 1/2" x 0'-9"  
  Fy = 39.6 ksi (Measured)
- Steel Grade: A36  
  Fu = 62.1 ksi (Measured)

Bolts
- Diameter: 0.75 in.
- Bolt Grade: A325  
  Ft = 106.4 ksi (Measured)
- Pretension: 10 ± kips (snug-tight)
  Thread Location: Included in shear plane

Beam
- Section: W16x50  
  Fy = 53.6 ksi (Measured)
- Steel Grade: A992  
  Fu = 69.3 ksi (Measured)

Column
- Section: W14x145
- Steel Grade: A992

Experimental Results
- Maximum Applied Shear at Connection = 78.8 kips
- Maximum Beam Rotation at Connection = 0.036 rad
- Maximum Vertical Beam Deflection at Connection = 0.40 in.
- Maximum Vertical Shear Tab Deflection = 0.14 in.
- Maximum Vertical Bolt Deflection = 0.30 in.

Test observations:
- Beam yielding at quarter points and midspan observed at 60kips
- Lateral braces were adjusted (tightened) at 65kips
- Shear tab yielding, additional beam yielding observed at 70kips
- Lateral buckling of beam between brace points observed at 75kips
- Severe yielding of beam at quarter points and midspan, yielding of shear tab, small yielding of beam web around bolts, and continued lateral torsional buckling of beam observed at 78kips.
- Test terminated due to lateral bucking of beam and achievement of 0.035 rad beam rotation at loads well above predictions.
NOTE: Standard holes are used in all test beams. Standard or short slotted holes are used in the shear tab.

Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Instrumentation
Connection Region Strain Gage Instrumentation
Test 1A – Connection Region at End of Test (bolts removed but not failed)
Test 1A – Connection Region at End of Test (bolts removed)
Test 1A – Beam Web After Test
Test 1A (Rigid, Slotted, Three-Bolt)
Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 90.9 kips

Test stopped prior to shear rupture of connection bolts
Test 1A (Rigid, Slotted, Three-Bolt)
Applied Shear vs. Component Rotation

Graph showing the relationship between Applied Shear at Connection (kips) and Rotation (rad) for Shear Tab and Bolts.
Test 1A (Rigid, Slotted, Three-Bolt)

Applied Shear vs. Bolt Eccentricity

\[ e_b = 1.6 \text{ in.} \]
Test 1A (Rigid, Slotted, Three-Bolt)
Applied Shear vs. Deflection

[Graph showing relationship between Applied Shear at Connection (kips) and Vertical Deflection at Connection (in.).]

- **Beam (Total)**
- **Bolts**
- **Shear Tab**

Legend:
- Δ Beam
- Δ Plate
## LIMIT STATE COMPARISON

(Test 1A: Three-Bolt, Rigid Support, Slotted Holes)

Calculations based on $F_y$ (ksi) = 39.6, with phi factors

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ($e_b=0$ in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>72.17</td>
<td>77.50</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>66.81</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ($e=3$in)</td>
<td>60.14</td>
<td>74.50</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>85.80</td>
<td>62.00</td>
<td>1.38</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>72.38</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ($e_w=3$in)</td>
<td>76.43</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>54.54</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>90.90</td>
<td>-----</td>
<td>-----</td>
</tr>
</tbody>
</table>

Note: Test Stopped Prior to Rupture of Connection Bolts.
Test Designation: Shear Tab Test #1
(Test1A: Three-Bolt, Rigid Support, Slotted Holes)

Test Properties and Values:
Young's Modulus, $E$ (ksi) = 29000
Yield Stress (ksi) = 39.6
Test Stopped (kips) = 80.2
(Connection did not fail)

<table>
<thead>
<tr>
<th>Strain Gage Location (in)</th>
<th>Load (kips)</th>
<th>Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>15</td>
<td>35</td>
</tr>
<tr>
<td>Corr. Strain ($\mu\varepsilon$)</td>
<td>18.4</td>
<td>32.1</td>
</tr>
<tr>
<td>Corr. Stress (ksi)</td>
<td>1.7</td>
<td>5.4</td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.
APPENDIX C

TEST SUMMARY FOR:

TEST 2 – S2-RST-3-A325-N
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S2-RST-3-A325-N (Test 2A (Rigid, Standard, Three-Bolt))
Test Date: April 28, 2003
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Three bolt single plate connection with a rigid support condition, standard holes, and snug-tight bolts.

Shear Tab
Plate Size: PL 3/8" x 4 1/2" x 0'-9"  Fy = 39.6 ksi (Measured)
Steel Grade: A36  Fu = 62.1 ksi (Measured)

Bolts
Diameter: 0.75 in.
Bolt Grade: A325
Ft = 106.4 ksi (Measured)
Pretension: 10 ± kips (snug-tight)
Thread Location: Included in shear plane

Beam
Section: W16x50  Fy = 53.6 ksi (Measured)
Steel Grade: A992  Fu = 69.3 ksi (Measured)

Column
Section: W14x145
Steel Grade: A992

Experimental Results
Maximum Applied Shear at Connection = 90.7 kips
Maximum Beam Rotation at Connection = 0.027 rad
Maximum Vertical Beam Deflection at Connection = 0.44 in.
Maximum Vertical Shear Tab Deflection = 0.15 in.
Maximum Vertical Bolt Deflection = 0.37 in.

Test observations:
- Beam yielding at quarter points and midspan observed at 62kips
- Shear tab yielding, additional beam yielding observed at 70kips
- Severe yielding of beam at quarter points and midspan, yielding of shear tab, small yielding of beam web around bolts, and small lateral torsional buckling of beam between brace points observed at 75kips.
- At approximately 75kips and 0.027 rad beam rotation, the test was paused and additional load was applied adjacent to the connection region using a hydraulic ram.
- Additional yielding of the shear tab, beam web were observed at 80kips
- Shear rupture of the connection bolts was observed at 90kips.
NOTE: Standard holes are used in all test beams. Standard or short slotted holes are used in the shear tab.

Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Instrumentation
Connection Region Strain Gage Instrumentation
Test 2A – Connection Region Before Test

Test 2A – Connection Region Before Test
Test 2A – Overall View of Test Setup
Test 2A – Connection Region During Test
Test 2A – Connection Region at End of Test

Test 2A – Connection Bolt at End of Test
Test 2A – Shear Tab at End of Test
Test 2A – Beam Web at End of Test
Test 2A (Rigid, Standard, Three-Bolt)

Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 90.9 kips

Test paused and shear load applied adjacent to connection up through shear rupture of the bolts.
Test 2A (Rigid, Standard, Three-Bolt)
Applied Shear vs. Component Rotation
Test 2A (Rigid, Standard, Three-Bolt)

Applied Shear vs. Bolt Eccentricity

- $e_b = 2.1$ in.
- $e_b = 3.0$ in.

Applied Shear at Connection (kips) vs. Eccentricity of Bolts, $e_b$ (in.)
**LIMIT STATE COMPARISON**  
(Test2A: Three-Bolt, Rigid Support, Standard Holes)

Calculations based on $F_y$ (ksi) = 39.6, with phi factors

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ($e_b$ = 0 in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>72.17</td>
<td>70.00</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>66.81</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ($e$ = 3 in)</td>
<td>60.14</td>
<td>76.30</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>85.80</td>
<td>70.00</td>
<td>1.23</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>72.38</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ($e_w$ = 3 in)</td>
<td>76.43</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>54.54</td>
<td>90.70</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>90.90</td>
<td>90.70</td>
<td>1.00</td>
</tr>
</tbody>
</table>

- Indicates the maximum observed test load
- Indicates a limit state which was not observed
Test Designation: Shear Tab Test #2
Test 2A (Rigid, Standard, Three-Bolt)

Test Properties and Values:
- Young's Modulus, E (ksi) = 29000
- Failure Load (kips) = 90.7
- Yield Stress (ksi) = 39.6

<table>
<thead>
<tr>
<th>Strain Gage Location (in)</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>Corresponding Strain (µε)</th>
<th>Corresponding Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.50</td>
<td>1765</td>
<td>2098</td>
<td>4484</td>
<td>2038</td>
<td>39.6</td>
<td>39.6</td>
</tr>
<tr>
<td>1.25</td>
<td>543</td>
<td>970</td>
<td>5753</td>
<td>8470</td>
<td>15.7</td>
<td>28.1</td>
</tr>
<tr>
<td>-1.25</td>
<td>31</td>
<td>212</td>
<td>-918</td>
<td>628</td>
<td>0.9</td>
<td>-26.6</td>
</tr>
<tr>
<td>-3.50</td>
<td>-2703</td>
<td>-6117</td>
<td>-14600</td>
<td>-19399</td>
<td>-39.6</td>
<td>-39.6</td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.
APPENDIX D

TEST SUMMARY FOR:

TEST 3 – S3-FSS-3-A325-N
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S3-FSS-3-A325-N (Test 3A (Flexible, Slotted, Three-Bolt))
Test Date: May 23, 2003
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Three bolt single plate connection with a flexible support condition, short slotted holes, and snug-tight bolts.

Shear Tab
- Plate Size: PL 3/8" x 4 1/2" x 0'-9"
- Steel Grade: A36
- Fy = 39.6 ksi (Measured)
- Fu = 62.1 ksi (Measured)

Bolts
- Diameter: 0.75 in.
- Bolt Grade: A325
- Ft = 106.4 ksi (Measured)
- Pretension: 10 ± kips (snug-tight)
- Thread Location: Included in shear plane

Beam
- Section: W16x50
- Steel Grade: A992
- Fy = 52.0 ksi (Measured)
- Fu = 68.9 ksi (Measured)

Girder
- Section: W18x50
- Steel Grade: A992
- Fy = 53.1 ksi (Measured)
- Fu = 70.9 ksi (Measured)

Experimental Results
- Maximum Applied Shear at Connection = 71.8 kips
- Maximum Beam End Rotation at Connection = 0.039 rad
- Maximum Girder Rotation at Connection = 0.114 rad
- Maximum Vertical Beam Deflection at Connection = 1.12 in.
- Maximum Vertical Girder Deflection at Connection = 0.16 in.

Test observations:
- Elastic rotation of the supporting girder observed at 30kips.
- Beam yielding at quarter points and midspan observed at 56kips.
- Severe yielding of beam at quarter points and midspan, observed at 60kips.
- Large rotation and small yielding of the top flange of the girder was observed at 62kips. The test was paused and braces were installed adjacent to the connection region to restrain the lateral movement of the girder top flange.
- At approximately 64kips and 0.038 rad beam rotation, the test was paused and additional load was applied adjacent to the connection region.
- Continued beam yielding and lateral torsional buckling of the beam between lateral brace points was observed.
- Shear rupture of the connection bolts was observed at 71.8kips.
- Failure of a lateral brace mechanism located at the 3/4 load point was observed at the time of bolt failure.
Detail of Single Plate (Shear Tab) Connection

NOTE: Standard holes are used in all test beams. Standard or short slotted holes are used in the shear tab.

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 3A – Overall View of Test Setup

Test 3A – Connection Region Before Test
Test 3A – Connection Region at End of Test

Test 3A – Connection Region at End of Test
Test 3A – Connection Region at End of Test
Test 3A – Shear Tab After Test
Test 3A – Top Bolt Hole in Shear Tab After Test

Test 3A – Center Bolt Hole in Shear Tab After Test
Test 3A – Bottom Bolt Hole in Shear Tab After Test

Test 3A – Top Bolt Hole in Beam Web After Test
Test 3A – Connection Bolts at End of Test

Test 3A – Connection Bolts at End of Test (fit back together)
Test 3A (Flexible, Slotted, Three-Bolt)
Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 90.9 kips

Beam
Girder
Girder Web
Shear Tab
Bolts

Applied Shear at Connection (kips)
Rotation at Connection (rad)
Test 3A (Flexible, Slotted, Three-Bolt)

Applied Shear vs. Deflection

Applied Shear at Connection (kips)

Vertical Deflection at Connection (in.)
Test 3A (Flexible, Slotted, Three-Bolt)

Applied Shear vs. Bolt Eccentricity

Applied Shear at Connection (kips)

Eccentricity of Bolts, \( e_b \) (in.)

\( e_b = (-) 2 \text{ in.} \)

\( e_b = (-) 1.3 \text{ in.} \)
### LIMIT STATE COMPARISON

(Test3A: Three-Bolt, Flexible Support, Slotted Holes)

Calculations based on Fy (ksi) = 39.6, with phi factors

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength (e_b=0 in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>72.17</td>
<td>70.90</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>66.81</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding (e=3in)</td>
<td>60.14</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>85.80</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>72.38</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal (e_w=3in)</td>
<td>76.43</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>54.54</td>
<td>71.80</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>90.90</td>
<td>71.80</td>
<td>1.27</td>
</tr>
</tbody>
</table>

- Indicates the maximum observed test load
- Indicates a limit state which was not observed
Single Plate Stress Analysis

Test Designation: Shear Tab Test #3
Test 3A (Flexible, Slotted, Three-Bolt)

Test Properties and Values:
Young's Modulus, $E$ (ksi) = 29000
Yield Stress (ksi) = 39.6
Failure Load (kips) = 71.8

<table>
<thead>
<tr>
<th>Strain Gage Location (in)</th>
<th>Load (kips)</th>
<th>Corresponding Strain ($\mu$ε)</th>
<th>Load (kips)</th>
<th>Corresponding Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
<td>30</td>
<td>50</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>30</td>
<td>50</td>
<td>70</td>
</tr>
<tr>
<td>3.50</td>
<td>87</td>
<td>451</td>
<td>1122</td>
<td>3622</td>
</tr>
<tr>
<td>1.25</td>
<td>32</td>
<td>101</td>
<td>378</td>
<td>2209</td>
</tr>
<tr>
<td>-1.25</td>
<td>7</td>
<td>13</td>
<td>41</td>
<td>204</td>
</tr>
<tr>
<td>-3.50</td>
<td>-21</td>
<td>-81</td>
<td>-15</td>
<td>501</td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.

Test 3A (Flexible, Slotted, Three-Bolt)
Single Plate Stress
APPENDIX E

TEST SUMMARY FOR:

TEST 4 – S4-FST-3-A325-N
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S4-FST-3-A325-N (Test 4A (Flexible, Standard, Three-Bolt))
Test Date: June 10, 2003
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Three bolt single plate connection with a flexible support condition, standard holes, and snug-tight bolts.

Shear Tab
- Plate Size: PL 3/8" x 4 1/2" x 0'-9"
- Fy = 39.6 ksi (Measured)
- Steel Grade: A36
- Fu = 62.1 ksi (Measured)

Bolts
- Diameter: 0.75 in.
- Bolt Grade: A325
- Ft = 106.4 ksi (Measured)
- Pretension: 10 ± kips (snug-tight)
- Thread Location: Included in shear plane

Beam
- Section: W16x50
- Fy = 52.5 ksi (Measured)
- Steel Grade: A992
- Fu = 69.2 ksi (Measured)

Girder
- Section: W18x50
- Fy = 53.1 ksi (Measured)
- Steel Grade: A992
- Fu = 70.9 ksi (Measured)

Experimental Results
- Maximum Applied Shear at Connection = 61.4 kips
- Maximum Beam End Rotation at Connection = 0.023 rad
- Maximum Girder Rotation at Connection = 0.059 rad
- Maximum Vertical Beam Deflection at Connection = 0.53 in.
- Maximum Vertical Girder Deflection at Connection = 0.13 in.

Test observations:
- Elastic rotation of the supporting girder observed at 30kips.
- At approximately 50kips the roller support was at midpoint of travel.
- Beam yielding at quarter points and midspan observed above 50kips.
- Severe yielding of beam at quarter points and midspan, observed at 60kips.
  Large rotation and small yielding of the top flange of the girder was also observed at this load.
- Reinitiating loading at 60kips, the connection experienced a sudden rupture of the bolts at 61.4kips.
W16 x 50 (A992)

3/4" A325-N Bolts
(See Note)

PL 3/8 x 4 1/2" x 0'-9" (A36)

3/4" A325-N Bolts
(See Note)

W18 x 50 (A992)

NOTE: Standard holes are used in all test beams. Standard or short slotted holes are used in the shear tab.

Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 4A – Overall View of Test Setup
Test 4A – Connection Region Before Test
Test 4A – Connection Region at End of Test
Test 4A – Connection Region at End of Test
Test 4A – Top Bolt Hole in Shear Tab After Test

Test 4A – Center Bolt Hole in Shear Tab After Test

437
Test 4A – Bottom Bolt Hole in Shear Tab After Test

Test 4A – Top Two Holes in Web Region After Test
Test 4A – Connection Bolts at End of Test

Test 4A – Connection Bolts at End of Test (fit back together)
Test 4A (Flexible, Standard, Three-Bolt)

Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 90.9 kips
Test 4A (Flexible, Standard, Three-Bolt)

Applied Shear vs. Deflection
Test 4A (Flexible, Standard, Three-Bolt)

Applied Shear vs. Bolt Eccentricity

Applied Shear at Connection (kips) vs. Eccentricity of Bolts, $e_b$ (in.)

$e_b = (-) 2$ in.
### LIMIT STATE COMPARISON

(Test4A: Three-Bolt, Flexible Support, Standard Holes)

Calculations based on Fy (ksi) = 39.6, with phi factors

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ((e_b = 0\text{ in}))</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>72.17</td>
<td>61.30</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>66.81</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ((e = 3\text{ in}))</td>
<td>60.14</td>
<td>61.20</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>85.80</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>72.38</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ((e_w = 3\text{ in}))</td>
<td>76.43</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>54.54</td>
<td>61.40</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>90.90</td>
<td>61.40</td>
<td>1.48</td>
</tr>
</tbody>
</table>

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Indicates the maximum observed test load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>Indicates a limit state which was not observed</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Results may be questionable due to preloading of test specimen.
APPENDIX F

TEST SUMMARY FOR:

TEST 5 – S5-FSS-3-A325-N-SR
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S5-FSS-3-A325-N-SR (Test 5A (Flexible, Slotted, Tie Plate, Three-Bolt))
Test Date: June 13, 2003
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Three bolt single plate connection with a flexible support condition, slotted holes, welded tie plate between top flange of girder and beam, and snug-tight bolts.

Shear Tab
Plate Size: PL 3/8" x 4 1/2" x 0'-9"
Steel Grade: A36
Fy = 39.6 ksi (Measured)

Bolts
Diameter: 0.75 in.
Bolt Grade: A325
Ft = 106.4 ksi (Measured)
Pretension: 10 ± kips (snug-tight)
Thread Location: Included in shear plane

Beam
Section: W16x50
Steel Grade: A992
Fy = 52.0 ksi (Measured)

Girder
Section: W18x50
Steel Grade: A992
Fy = 53.1 ksi (Measured)

Tie Plate
Plate Size: PL 3/8" x 6" x 6"
Steel Grade: A36

Experimental Results
Maximum Applied Shear at Connection = 75.6 kips
Maximum Beam End Rotation at Connection = 0.031 rad
Maximum Girder Rotation at Connection = 0.060 rad
Maximum Vertical Beam Deflection at Connection = 1.14 in.
Maximum Vertical Girder Deflection at Connection = 0.20 in.

Test observations:
- Elastic rotation of the girder was observed at 27kips.
- Girder rotation, bending of the welded tie plate, movement of the bolts in the holes and beam yielding at the quarter points and midspan observed at 50kips.
- Continued yielding of the beam was observed at 60kips.
- At 60kips, the test was paused and additional load was applied adjacent to the connection. The loading of the specimen was continued.
- Shear rupture of the connection bolts was observed at 75.6kips. Failure of a lateral brace mechanism located at the 3/4 load point was observed at the time of bolt failure.
NOTE: Standard holes are used in all test beams. Standard or short slotted holes are used in the shear tab.

Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 5A – Overall View of Test Setup
Test 5A – Connection with Simulated Slab Restraint Before Test
Test 5A – Connection Region Before Test
Test 5A – Connection Region at End of Test
Test 5A – Top Bolt Hole in Shear Tab After Test

Test 5A – Center Bolt Hole in Shear Tab After Test

453
Test 5A – Bottom Bolt Hole in Shear Tab After Test

Test 5A – Connection Bolts at End of Test
Test 5A – Connection Bolts at End of Test (fit back together)
Test 5A (Flexible, Slotted, Slab-Restraint, Three-Bolt)
Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 90.9 kips

Beam
Girder
Girder Web
Shear Tab
Bolts

Applied Shear at Connection (kips)
Rotation at Connection (rad)
Test 5A (Flexible, Slotted, Slab Restraint, Three-Bolt)

Applied Shear vs. Deflection

- Beam
- Shear Tab
- Bolts
- Girder
Test 5A (Flexible, Slotted, Slab Restraint, Three-Bolt)

Applied Shear vs. Bolt Eccentricity

Applied Shear at Connection (kips)

Eccentricity of Bolts, $e_b$ (in.)

$e_b = 0.1$ in.
<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ($e_b=0$ in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>72.17</td>
<td>74.30</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>66.81</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ($e=3$in)</td>
<td>60.14</td>
<td>74.70</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>85.80</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>72.38</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ($e_w=3$in)</td>
<td>76.43</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>54.54</td>
<td>75.60</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>90.90</td>
<td>75.60</td>
<td>1.20</td>
</tr>
</tbody>
</table>

**LIMIT STATE COMPARISON**
(Test5A: Three-Bolt, Flexible Support, Slotted Holes, Slab Restraint)

Calculations based on $F_y$ (ksi) = 39.6, with phi factors

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ($e_b=0$ in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>72.17</td>
<td>74.30</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>66.81</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ($e=3$in)</td>
<td>60.14</td>
<td>74.70</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>85.80</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>72.38</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ($e_w=3$in)</td>
<td>76.43</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>54.54</td>
<td>75.60</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>90.90</td>
<td>75.60</td>
<td>1.20</td>
</tr>
</tbody>
</table>

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td><strong>Indicates the maximum observed test load</strong></td>
<td></td>
</tr>
<tr>
<td><strong>-----</strong></td>
<td><strong>Indicates a limit state which was not observed</strong></td>
</tr>
</tbody>
</table>
Test Designation: Shear Tab Test #5
Test 5A (Flexible, Slotted, Slab-Restraint, Three-Bolt)

Test Properties and Values:
Young's Modulus, E (ksi) = 29000
Yield Stress (ksi) = 39.6
Failure Load (kips) = 75.6

<table>
<thead>
<tr>
<th>Strain Gage Location (in)</th>
<th>Load (kips)</th>
<th>Corresponding Strain (µε)</th>
<th>Corresponding Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>15</td>
<td>35</td>
<td>55</td>
</tr>
<tr>
<td>3.50</td>
<td>31</td>
<td>139</td>
<td>200</td>
</tr>
<tr>
<td>1.25</td>
<td>-129</td>
<td>-256</td>
<td>-244</td>
</tr>
<tr>
<td>-1.25</td>
<td>-99</td>
<td>-156</td>
<td>-190</td>
</tr>
<tr>
<td>-3.50</td>
<td>-185</td>
<td>-470</td>
<td>-599</td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.
APPENDIX G

TEST SUMMARY FOR:

TEST 6 – S6-FST-2-A325-N
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S6-FST-2-A325-N (Test 6A (Flexible, Standard, Two-Bolt))  
Test Date: July 17, 2003  
Sponsor: W & W Steel Company  
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.  
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University  

Two bolt single plate connection with a flexible support condition, standard holes, and snug-tight bolts.

Shear Tab  
Plate Size: PL 3/8" x 4 1/2" x 0'-6"  
Steel Grade: A36  
Fy = 39.6 ksi (Measured)  
Fu = 62.1 ksi (Measured)

Bolts  
Diameter: 0.75 in.  
Bolt Grade: A325  
Ft = 106.4 ksi (Measured)  
Pretension: 10 ± kips (snug-tight)  
Thread Location: Included in shear plane

Beam  
Section: W16x50  
Steel Grade: A992  
Fy = 54.2 ksi (Measured)  
Fu = 69.6 ksi (Measured)

Girder  
Section: W18x50  
Steel Grade: A992  
Fy = 53.1 ksi (Measured)  
Fu = 70.9 ksi (Measured)

Experimental Results  
Maximum Applied Shear at Connection = 44.2 kips  
Maximum Beam End Rotation at Connection = 0.012 rad  
Maximum Girder Rotation at Connection = 0.083 rad  
Maximum Vertical Beam Deflection at Connection = 0.78 in.  
Maximum Vertical Girder Deflection at Connection = 0.09 in.

Test observations:
- At service load conditions, 18kips, girder experienced some elastic rotation.  
- At 30kips, first appearance of concentrated yielding near brace at midspan.  
- At 40kips, the test was paused and a visible increase in girder rotation as well as slight curvature of beam was noted. Data indicated a leveling slope in connection rotation, suggesting approach of failure  
- Testing resumed and shear rupture of the connection bolts was observed at 44kips. The beam was unyielded from test.
NOTE: Standard holes are used in all test beams. Standard or short slotted holes are used in the shear tab.

Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 6A – Connection Region at End of Test

Test 6A – Top Bolt Hole in Shear Tab at End of Test
Test 6A – Bottom Bolt Hole in Shear Tab at End of Test

Test 6A – Connection Bolts at End of Test
Test 6A – Connection Bolts at End of Test (fit back together)
Test 6A (Flexible, Standard, Two-Bolt)

Applied Shear vs. Rotation

- Applied Shear at Connection (kips)
- Rotation at Connection (rad)

Predicted Ultimate Strength of Bolts = 60.6 kips

Graph lines:
- Beam
- Bolts
- Shear Tab
- Girder Web
- Girder

Legend:
- Beam
- Bolts
- Shear Tab
- Girder
- Girder Web
Note: The pot measuring the vertical deflection of the bolts malfunctioned. Therefore, the bolt data was omitted from this plot.
Test 6A (Flexible, Standard, Two-Bolt)

Applied Shear vs. Bolt Eccentricity

$e_b = (-) 4.4 \text{ in.}$
<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ((e_b = 0\ \text{in}))</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>48.11</td>
<td>43.30</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>44.54</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ((e = 3\ \text{in}))</td>
<td>26.73</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>54.36</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>50.11</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ((e_w = 3\ \text{in}))</td>
<td>39.60</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>36.36</td>
<td>44.20</td>
<td>0.82</td>
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<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>60.60</td>
<td>44.20</td>
<td>1.37</td>
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<table>
<thead>
<tr>
<th><strong>LIMIT STATE COMPARISON</strong></th>
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</thead>
<tbody>
<tr>
<td>(Test6A: Two-Bolt, Flexible Support, Standard Holes)</td>
</tr>
</tbody>
</table>

Calculations based on \(F_y\ (\text{ksi}) = 39.6\), with phi factors

Indicates the maximum observed test load

----- Indicates a limit state which was not observed
Test Designation: Shear Tab Test #6  
(Test 6A: Two-Bolt, Flexible Support, Standard Holes)

Test Properties and Values:
Young's Modulus, \( E \) (ksi) = 29000  
Failure Load (kips) = 44.2
Yield Stress (ksi) = 39.6

<table>
<thead>
<tr>
<th>Strain Gage Location (in)</th>
<th>Load (kips)</th>
<th>Corresponding Strain (( \mu \varepsilon ))</th>
<th>Corresponding Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>71</td>
<td>279</td>
<td>440</td>
</tr>
<tr>
<td>0</td>
<td>-58</td>
<td>-37</td>
<td>71</td>
</tr>
<tr>
<td>-2</td>
<td>-150</td>
<td>-206</td>
<td>-180</td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.
APPENDIX H

TEST SUMMARY FOR:

TEST 7 – S7-FSS-2-A325-N
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S7-FSS-2-A325-N (Test 7A (Flexible, Slotted, Two-Bolt))
Test Date: August 29, 2003
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Two bolt single plate connection with a flexible support condition, slotted holes, and snug-tight bolts.

**Shear Tab**
- Plate Size: PL 3/8" x 4 1/2" x 0'-6"
- Fy = 39.6 ksi (Measured)
- Steel Grade: A36
- Fu = 62.1 ksi (Measured)

**Bolts**
- Diameter: 0.75 in.
- Bolt Grade: A325
- Ft = 106.4 ksi (Measured)
- Pretension: 10 ± kips (snug-tight)
- Thread Location: Included in shear plane

**Beam**
- Section: W16x50
- Fy = 53.6 ksi (Measured)
- Steel Grade: A992
- Fu = 69.3 ksi (Measured)

**Girder**
- Section: W18x50
- Fy = 53.1 ksi (Measured)
- Steel Grade: A992
- Fu = 70.9 ksi (Measured)

**Experimental Results**
- Maximum Applied Shear at Connection = 45.45 kips
- Maximum Beam End Rotation at Connection = 0.011 rad
- Maximum Girder Rotation at Connection = 0.099 rad
- Maximum Vertical Beam Deflection at Connection = 1.11 in.
- Maximum Vertical Girder Deflection at Connection = 0.07 in.

**Test observations:**
- At service load conditions, 18kips, the connection exhibited slight rotation. The first appearance of concentrated yield marks at midspan of top flange was noted.
- At 30kips, an increase in concentrated yielding at midspan of beam as well as an increase in rotation at connection was observed.
- At 40kips, yielding of the shear tab and localized yielding on web near all stiffeners was noted.
- Shear rupture of the connection bolts was observed at 45kips. Upon examination of the connection, failure of the top portion of the shear tab weld was noted. Although the beam underwent localized yielding, overall, the beam was relatively unyielded.
NOTE: Standard holes are used in all test beams. Standard or short slotted holes are used in the shear tab.

**Detail of Single Plate (Shear Tab) Connection**

**Elevation of Test Setup**
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 7A – Overall View of Test Setup
Test 7A – Connection Region Before Test

Test 7A – Connection Region at End of Test
Test 7A – Connection Region at End of Test

Test 7A – Top Bolt Hole in Shear Tab at End of Test
Test 7A – Bottom Bolt Hole in Shear Tab at End of Test

Test 7A – Failure of Shear Tab Weld at End of Test
Test 7A – Connection Bolts at End of Test

Test 7A – Connection Bolts at End of Test (fit back together)
Test 7A (Flexible, Slotted, Two-Bolt)  
Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 60.6 kips

Note: The pot measuring the rotation at the top of the shear tab malfunctioned. Data was approximated to reflect shear tab rotation based upon a rotation about the centerline of the shear tab.
Test 7A (Flexible, Slotted, Two-Bolt)

Applied Shear vs. Deflection

- Beam
- Bolts
- Shear Tab
- Girder

Vertical Deflection at Connection (in.)

Applied Shear at Connection (kips)

- Δ Beam
- Δ Plate
- Δ Girder
Test 7A (Flexible, Slotted, Two-Bolt)
Applied Shear vs. Bolt Eccentricity

- Eccentricity of Bolts, $e_b$ (in.): $(-) 2.4$ in.

Graph showing the relationship between Applied Shear at Connection (kips) and Eccentricity of Bolts, $e_b$ (in.).
<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ((e_b = 0 \text{ in}))</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>48.11</td>
<td>44.40</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>44.54</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ((e = 3\text{ in}))</td>
<td>26.73</td>
<td>44.70</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>54.36</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>50.11</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ((e_w = 3\text{ in}))</td>
<td>39.60</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>36.36</td>
<td>45.45</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>60.60</td>
<td>45.45</td>
<td>1.33</td>
</tr>
</tbody>
</table>

Indicates the maximum observed test load

----- Indicates a limit state which was not observed
APPENDIX I

TEST SUMMARY FOR:

TEST 8 – S8-FSS-2-A325-N-SR
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S8-FSS-2-A325-N-SR (Test 8A (Flexible, Slotted, Tie Plate, Two-Bolt))
Test Date: September 8, 2003
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Two bolt single plate connection with a flexible support condition, slotted holes, welded tie plate between top flange of girder and beam, and snug-tight bolts.

Shear Tab
- Plate Size: PL 3/8" x 4 1/2" x 0'-6"  
  - Fy = 39.6 ksi (Measured)
- Steel Grade: A36
  - Fu = 62.1 ksi (Measured)

Bolts
- Diameter: 0.75 in.
- Bolt Grade: A325
  - Ft = 106.4 ksi (Measured)
- Pretension: 10 ± kips (snug-tight)
Thread Location: Included in shear plane

Beam
- Section: W16x50
  - Fy = 53.6 ksi (Measured)
- Steel Grade: A992
  - Fu = 69.3 ksi (Measured)

Girder
- Section: W18x50
  - Fy = 53.1 ksi (Measured)
- Steel Grade: A992
  - Fu = 70.9 ksi (Measured)

Tie Plate
- Plate Size: PL 3/8" x 6" x 6"
- Steel Grade: A36

Experimental Results
- Maximum Applied Shear at Connection = 47.93 kips
- Maximum Beam End Rotation at Connection = 0.013 rad
- Maximum Girder Rotation at Connection = 0.032 rad
- Maximum Vertical Beam Deflection at Connection = 1.08 in.
- Maximum Vertical Girder Deflection at Connection = 0.15 in.

Test observations:
- At 11kips, a shift at the connection was noted, resulting in an increase in beam deflection and connection rotation.
- At service load conditions, 18kips, there was slight rotation at connection with minimal girder rotation due to restraints.
- At 30kips, a visible rotation at the connection was noted.
- The connection sustained loading up to 46kips. After which a weakening of the connection was noted as the load dropped slightly.
- Connection sustained an increase in load. Failure occurred at 48kips when the bottom bolt sheared and the top portion of the shear tab weld failed. The top bolt was reasonably straight and the beam was relatively unyielded.
Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 8A – Overall View of Test Setup
Test 8A – Connection Region at End of Test

Test 8A – Connection Region at End of Test
Test 8A – Top Bolt Hole in Shear Tab at End of Test

Test 8A – Bottom Bolt Hole in Shear Tab at End of Test
Test 8A – Failure of Shear Tab Weld at End of Test

Test 8A – Connection Bolts at End of Test
Test 8A – Connection Bolts at End of Test (fit back together)
Test 8A (Flexible, Slotted, Slab Restraint, Two-Bolt)

Applied Shear vs. Rotation

Note: The pot measuring the rotation at the top of the shear tab malfunctioned. Therefore, the shear tab data was omitted from this plot.

Predicted Ultimate Strength of Bolts = 60.6 kips

Beam
Bolts
Girder Web
Girder

Applied Shear at Connection (kips)
Rotation at Connection (rad)
Test 8A (Flexible, Slotted, Slab Restraint, Two-Bolt)

Applied Shear vs. Deflection

- Beam
- Shear Tab
- Girder
- Bolts

Applied Shear at Connection (kips) vs. Vertical Deflection at Connection (in.)

Graph with data points for each component (Beam, Shear Tab, Girder, Bolts) showing their respective vertical deflections under applied shear forces.
Test 8A (Flexible, Slotted, Slab Restraint, Two-Bolt)
Applied Shear vs. Bolt Eccentricity

Applied Shear at Connection (kips)

Eccentricity of Bolts, $e_b$ (in.)

$e_b = (-) 2.2$ in.
<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ($e_b = 0$ in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>48.11</td>
<td>45.30</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>44.54</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ($e = 3$in)</td>
<td>26.73</td>
<td>Pot Malfunction</td>
<td>Pot Malfunction</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>54.36</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>50.11</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ($e_w = 3$in)</td>
<td>39.60</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>36.36</td>
<td>47.93</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>60.60</td>
<td>47.93</td>
<td>1.26</td>
</tr>
</tbody>
</table>

*Indicates the maximum observed test load

*-----* Indicates a limit state which was not observed
Test Designation: Shear Tab Test #8
(Test8A: Two-Bolt, Flexible Support, Slab-Restraint, Slotted Holes)

Test Properties and Values:
Young's Modulus, $E$ (ksi) = 29000
Failure Load (kips) = 47.9
Yield Stress (ksi) = 39.6

<table>
<thead>
<tr>
<th>Strain Gage Location (in)</th>
<th>Load (kips)</th>
<th>Corresponding Strain ($\mu \varepsilon$)</th>
<th>Corresponding Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10 20 30 40 46</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>120 163 215 388 634</td>
<td>3.5 4.7 6.2 11.3 18.4</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>-6 -84 -147 -80 270</td>
<td>-0.2 -2.4 -4.3 -2.3 7.8</td>
<td></td>
</tr>
<tr>
<td>-2</td>
<td>-110 -221 -213 210 2239</td>
<td>-3.2 -6.4 -6.2 6.1 39.6</td>
<td></td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S9-RSS-7-A325-N (Test 9A (Rigid, Slotted, Seven-Bolt))
Test Date: September 21, 2004
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Seven bolt single plate connection with a rigid support condition, slotted holes, and snug-tight bolts.

Shear Tab
- Plate Size: PL 3/8" x 4 1/2" x 1'-9"
- Steel Grade: A36
- Fy = 44.4 ksi (Measured)
- Fy = 66.25 ksi (Measured)

Bolts
- Diameter: 0.75 in.
- Bolt Grade: A325
- Ft = 106.4 ksi (Measured)
- Pretension: 10 ± kips (snug-tight)
- Thread Location: Included in shear plane

Beam
- Section: W27x84
- Steel Grade: A992
- Fy = 54.1 ksi (Measured)
- Fy = 70.4 ksi (Measured)

Experimental Results
- Maximum Applied Shear at Connection = 166.53 kips
- Maximum Beam End Rotation at Connection = 0.028* rad
- Maximum Vertical Beam Deflection at Connection = 0.44 in.
- Maximum Vertical Shear Tab Deflection = 0.293 in.
- Maximum Vertical Bolt Deflection = 0.411 in.

* Due to the separation of the test column from the strong wall, the rotation of the test beam was approximated producing the listed value. (See Appendix J1 for details.)

Test observations:
- At service load conditions, 104kips, the connection exhibited slight rotation.
- At 125kips, the first appearance of concentrated yield marks at midspan of beam was noted.
- At 142kips, an increase in concentrated yield marks at midspan of beam. Also, concentrated yield marks noted on single plate between the bolt line and weld line.
- At 165kips, nonlinearity of the load-deflection curve was noted. The test beam had sustained the maximum loading and a plastic hinge had formed at the midspan.
- Loading continued, but the sustained load dropped to 140kips. At which time, additional load was applied to the connection via the hydraulic ram.
- With additional load from the hydraulic ram, the connection sustained up to 160kips, which was less than the maximum applied shear at the connection experienced prior to the additional load of the hydraulic ram, before the test was terminated due to lateral torsional buckling failure at the midspan of the test beam and failure of the lateral brace mechanisms located at the one-third and two-third load points.
NOTE: Standard holes are used in all test beams. Short slotted holes are used in the shear tab.

Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 9A – Overall View of Test Setup

Test 9A – Connection Region Before Test
Test 9A – Connection Region at End of Test
Test 9A – Bolt Holes in Shear Tab at End of Test (with Bolts Removed)
Test 9A – Failure of Lateral Braces and Plastic Hinge at Midspan of Test Beam at End of Test (Photo from Connection Region Toward Supported End)

Test 9A – Deformation in Test Beam at End of Test (Lateral Braces Removed)
Test 9A – Plastic Hinge at Midspan of Beam at End of Test

Test 9A – Connection Bolts at End of Test
Test 9A (Rigid, Slotted, Seven-Bolt)
Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 212.1 kips

Failure of Lateral Bracing Occurred (No Connection Failure)
Test 9A (Rigid, Slotted, Seven-Bolt)
Applied Shear vs. Component Rotation

Rotation (rad)

Applied Shear at Connection (kips)

Bolts
Shear Tab

Shear Tab Rot.
Bolt Rot.
Shear @ Conn.
Test 9A (Rigid, Slotted, Seven-Bolt)
Applied Shear vs. Bolt Eccentricity

$e_b = 5.5$ in.

Applied Shear at Connection (kips)

Eccentricity of Bolts, $e_b$ (in.)
Test 9A (Rigid, Slotted, Seven-Bolt)

Applied Shear vs. Deflection

Applied Shear at Connection (kips) vs. Vertical Deflection at Connection (in.)

- Beam (Total)
- Bolts
- Shear Tab

Diagram inset shows:
- \( \Delta \) Bolt
- \( \Delta \) Plate
- \( \Delta \) Beam
<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ($e_b=0$ in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>188.81</td>
<td>165.40</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>166.30</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ($e=3$in)</td>
<td>367.13</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>225.69</td>
<td>165.90</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>173.15</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ($e_w=3$in)</td>
<td>234.66</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>169.68</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>212.10</td>
<td>-----</td>
<td>-----</td>
</tr>
</tbody>
</table>

----- Indicates a limit state which was not observed
Single Plate Stress Analysis

Test Designation:  Shear Tab Test #9
Test 9A (Rigid, Slotted, Seven-Bolt)

Test Properties and Values:
Young's Modulus, E (ksi) = 29000  Test Stopped (kips) = 166.53
Yield Stress (ksi) = 44.4  (Connection did not fail)

<table>
<thead>
<tr>
<th>Strain Gage Location (Zero at single plate N.A.) (in)</th>
<th>Load (kips) 40</th>
<th>80</th>
<th>120</th>
<th>160</th>
<th>Load (kips) 40</th>
<th>80</th>
<th>120</th>
<th>160</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.50</td>
<td>-52</td>
<td>202</td>
<td>790</td>
<td>1688</td>
<td>-1.5</td>
<td>5.9</td>
<td>22.9</td>
<td>44.4</td>
</tr>
<tr>
<td>3.50</td>
<td>-160</td>
<td>-384</td>
<td>-361</td>
<td>-380</td>
<td>-4.6</td>
<td>-11.1</td>
<td>-10.5</td>
<td>-11.0</td>
</tr>
<tr>
<td>-3.50</td>
<td>-251</td>
<td>-515</td>
<td>-780</td>
<td>-974</td>
<td>-7.3</td>
<td>-14.9</td>
<td>-22.6</td>
<td>-28.2</td>
</tr>
<tr>
<td>-9.50</td>
<td>-540</td>
<td>-1192</td>
<td>-2789</td>
<td>-6391</td>
<td>-15.7</td>
<td>-34.6</td>
<td>-44.4</td>
<td>-44.4</td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.
Note:  At Failure, the top and two bottom most strain gages indicated the steel had yielded.
APPENDIX J1

RAW DATA

AND

MODIFICATION EXPLANATION FOR:

TEST 9 – S9-RSS-7-A325-N
Single Plate (Shear Tab) Connection Test Modification Summary

Test ID: S9-RSS-7-A325-N (Test 9A (Rigid, Slotted, Seven-Bolt))
Test Date: September 21, 2004
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

General

The following is a summary of the need for and the modifications which were made to Test 9. Raw test data as well as any applicable modification data will be included herein.

Modification Description

Post test investigation revealed that the test column had separated at one location from the strong wall by approximately ½in. This separation consequently affected the applied shear versus rotation data plots. Noticeably, a counterclockwise or negative beam rotation occurred which is not typical of this test setup. This behavior can be seen in Figure J1 (applied shear versus rotation behavior of the test beam of Test 9), which is presented after this section. The data series labeled ‘Original Rotation Plot’ shows the unadjusted beam end rotation behavior.

As seen in Figure 5.1 for a rigid supported test, typical test beam rotation behavior begins with the application of load. In comparison to Figure J1 ‘Original Rotation Plot’, initially the beam end rotation increases slightly and then tends back toward zero rotation. Therefore, it is believed that the test column separation began at some rate with the
application of load. However, the amount of rotation of the column due to loading was not measured during testing.

Two methods of approximating the supported test beam rotation were utilized. First, from Section 4.2.1 Specimen Design – *Beam End Rotations*, the theoretical formula for determining beam end rotation was used to estimate beam end rotation at various load increments. Investigation indicated that in the previous two and three bolt tests as well as Test 10 (seven bolt test with flexible support condition at a lesser applied shear at the connection) the beam end rotation formula approximated the observed beam end rotation with reasonable accuracy. Second, as mentioned above the separation between the column and strong wall at one point was approximately $\frac{1}{2}$in. From this measurement and knowing the height of the test column to the point of measurement, an approximation of the test column rotation could be made. Using both methods a reasonable approximation of the test column rotation, which can be seen in Figure J1 ‘Adjustment Curve’, was generated. A rough calculation to determine the amount of separation between the test column and strong wall using the maximum rotation value from Figure J1 ‘Adjustment Curve’ generated a displacement of 0.73in. Although this value is greater than the post test observed value of $\frac{1}{2}$in of separation, arguably this value is reasonable as there are potentially multiple points of separation in the test setup and gaps may have lessened after the applied load was removed.

To approximate the actual beam end rotation experienced by the supported test beam, the ‘Original Rotation Plot’ data was added to the ‘Adjustment Curve’ data. The resultant of which is the third data series depicted in Figure J1 and labeled as ‘Final Rotation Plot’. The
‘Final Rotation Plot’ data series represents the final applied shear vs. rotation plot for the beam end rotation of the supported beam used in Test 9.

**Supplemental and Other Raw Data**

In addition to Figure J1 (applied shear versus rotation behavior), Figure J2 provides the raw data for the applied shear versus rotation of the single plate and bolts of Test 9. Lastly, Figure J3, presents the ‘Adjustment Curve’ from Figure J1. It should be noted that the ‘Adjustment Curve’ is a compilation of various modification equations used at various applied shears at the connection.
FIGURE J1
Test 9A (Rigid, Slotted, Seven-Bolt)
Applied Shear vs. Rotation (Original, Adjustment, Final)

Failure of Lateral Bracing Occurred (No Connection Failure)

Predicted Ultimate Strength of Bolts = 212.1 kips

Original Rotation Plot
Adjustment Curve
Final Rotation Plot

Applied Shear at the Connection (kips)
Rotation (rad)
FIGURE J2
Test 9A (Rigid, Slotted, Seven-Bolt)
Applied Shear vs. Component Rotation

Note: This graph illustrates the unaltered test data.
FIGURE J3
Rotation Modification Factor

Connection Rotation Due to Column Movement (rad) vs. Applied Shear at the Connection (kips)
APPENDIX K:

TEST 10 – S10-FSS-7-A325-N-SR
Single Plate (Shear Tab) Connection Test Summary Sheet

Test ID: S10-FSS-A325-N-SR (Test 10A (Flexible, Slotted, Tie Plate, Seven-Bolt))
Test Date: October 5, 2004
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

Connection Description
Seven bolt single plate connection with a flexible support condition, slotted holes, welded tie plate between top flange of girder and beam, and snug-tight bolts.

Shear Tab
- Plate Size: PL 3/8" x 4 1/2" x 1'-9"
- Steel Grade: A36
  - $F_y = 44.4$ ksi (Measured)
  - $F_u = 66.25$ ksi (Measured)

Bolts
- Diameter: 0.75 in.
- Bolt Grade: A325
  - $F_t = 106.4$ ksi (Measured)
- Pretension: 10 ± kips (snug-tight)
  - Thread Location: Included in shear plane

Beam
- Section: W27x84
- Steel Grade: A992
  - $F_y = 54.1$ ksi (Measured)
  - $F_u = 70.4$ ksi (Measured)

Girder
- Section: W30x99
- Steel Grade: A992
  - $F_y = 59.6$ ksi (Measured)
  - $F_u = 75$ ksi (Measured)

Tie Plate
- Plate Size: PL 1/2" x 8" x 10"
- Steel Grade: A36

Experimental Results
- Maximum Applied Shear at Connection = 202.46 kips
- Maximum Beam End Rotation at Connection = 0.027* rad
- Maximum Girder Rotation at Connection = 0.030* rad
- Maximum Vertical Beam Deflection at Connection = 0.86 in.
- Maximum Vertical Girder Deflection at Connection = 0.25 in.

* Due to the shift in the connection, the beam and girder rotation values were approximated producing the values presented above. (See Appendix K1 for details.)

Test observations:
- At 95kips, a significant shift in the connection occurred.
- At 125kips, the load-deflection curve began to exhibit nonlinear behavior.
- Between 125kips and 160kips, a popping noise was noted at the addition of every few kips.
- At 140kips, the appearance of concentrated yield marks at midspan of beam was noted. At this loading, additional load was applied to the connection via the hydraulic ram in a manner to attain a "linear" applied shear vs. rotation curve. Thus, attempting to achieve the target load-rotation relationship prior to failure of the beam.
- At 160kips, the concentrated yield marks was observed on the single plate between the bolt line and weld line.
- Connection sustained an increase in load. Failure occurred at 202 kips when the connection bolts experienced a sudden rupture.
Detail of Single Plate (Shear Tab) Connection

Elevation of Test Setup
Elevation of Test Setup Showing Instrumentation

Connection Region Strain Gage Instrumentation
Test 10A – Overall View of Test Setup
Test 10A – Connection Region Before Test
Test 10A – Connection Region at End of Test
Test 10A – Upper Four Bolt Holes in Shear Tab at End of Test
Test 10A – Lower Three Bolt Holes in Shear Tab at End of Test
Test 10A – Flange Yielding at Midspan of Beam (with Six of the Sheared Seven Bolts) at End of Test

Test 10A – Connection Bolts at End of Test
Test 10A – Connection Bolts at End of Test (fit back together)
Test 10A (Flexible, Slotted, Slab Restraint, Seven-Bolt)

Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 212.1 kips

Beam
Bolts
Girder
Girder Web
Shear Tab
Beam
Girder, Shear Tab, & Bolt Rotation @ Conn.
Shear @ Conn.

Applied Shear at Connection (kips)
Rotation at Connection (rad)
Test 10A (Flexible, Slotted, Slab Restraint, Seven-Bolt)

Applied Shear vs. Deflection

- Beam
- Shear Tab
- Bolts
- Girder

Applied Shear at Connection (kips)

Vertical Deflection at Connection (in.)
Test 10A (Flexible, Slotted, Slab Restraint, Seven-Bolt)
Applied Shear vs. Bolt Eccentricity

Applied Shear at Connection (kips)

Eccentricity of Bolts, $e_b$ (in.)

$e_b = 0.5$ in.

$e_b = 1.0$ in.
### LIMIT STATE COMPARISON

(Test10A: Seven-Bolt, Flexible Support, Slotted Holes, Slab Restraint)

Calculations based on $F_y$ (ksi) = 44.4, with phi factors

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Predicted LRFD Strength ($e_b=0$ in)</th>
<th>Estimated Test Value</th>
<th>Pred./Obs. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Plate</td>
<td>Shear Yielding</td>
<td>188.81</td>
<td>199.90</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>Shear Rupture</td>
<td>166.30</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Flexural Yielding ($e=3$in)</td>
<td>367.13</td>
<td>200.00</td>
<td>1.84</td>
</tr>
<tr>
<td></td>
<td>Bearing/Tearout</td>
<td>225.69</td>
<td>201.80</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>Block Shear</td>
<td>173.15</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>Base Metal ($e_w=3$in)</td>
<td>234.66</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bolts</td>
<td>Bolt Shear Strength</td>
<td>169.68</td>
<td>202.46</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>Bolt Str. (Shear Tests)</td>
<td>212.10</td>
<td>202.46</td>
<td>1.05</td>
</tr>
</tbody>
</table>

- Indicates the maximum observed test load
- ----- Indicates a limit state which was not observed
Test Designation: Shear Tab Test #10
Test 10A (Flexible, Slotted, Slab-Restraint, Seven-Bolt)

Test Properties and Values:
Young's Modulus, E (ksi) = 29000
Yield Stress (ksi) = 44.4
Failure Load (kips) = 202.5

<table>
<thead>
<tr>
<th>Strain Gage Location (in)</th>
<th>Load (kips)</th>
<th>Load (kips)</th>
<th>Corresponding Strain (µε)</th>
<th>Corresponding Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50</td>
<td>90</td>
<td>140</td>
<td>190</td>
</tr>
<tr>
<td>9.50</td>
<td>127</td>
<td>464</td>
<td>1176</td>
<td>5730</td>
</tr>
<tr>
<td>3.50</td>
<td>35</td>
<td>-166</td>
<td>-1227</td>
<td>-3.7</td>
</tr>
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<td>-3.50</td>
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<td>-9.50</td>
<td>-257</td>
<td>634</td>
<td>-1378</td>
<td>-7.5</td>
</tr>
</tbody>
</table>

Indicates the stress calculated from the strain gages exceeded the yield stress of the tab.
Note: At Failure, the top and two bottom most strain gages indicated the steel had yielded.

![Graph showing Single Plate Stress Analysis](image-url)
APPENDIX K1

RAW DATA

AND

MODIFICATION EXPLANATION FOR:

TEST 10 – S10-FSS-7-A325-N-SR
Single Plate (Shear Tab) Connection Test Modification Summary

Test ID: S10-FSS-A325-N-SR (Test 10A (Flexible, Slotted, Tie Plate, Seven-Bolt))
Test Date: October 5, 2004
Sponsor: W & W Steel Company
Tested By: Emmett Sumner, Ph. D., P.E. and Dustin Creech, E.I.
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University

General

The following is a summary of the need for and the modifications which were made to Test 10. Raw test data as well as any applicable modification data will be included herein.

Modification Description

At approximately 95kips, a significant shift in the connection occurred such that the linear potentiometers were so severely jostled that rotation measurements were adversely affected. Figure K1, which is presented after this section, shows the raw applied shear versus component rotation at the connection for Test 10. As such, the actual rotation behavior of the connection components could only be predicted. In an attempt to compensate for the shift in the connection, Test 5, which was a three bolt test with flexible support condition and simulated slab restraint, was referenced in order to predict the behavior of Test 10 after the shift.

In Test 5, a shift in the connection occurred at approximately 60kips. The behavior of the shift occurring in Test 5 indicated that the simulated slab restraint restrained the movement of the upper test components including: the test beam, single plate, bolts, and girder web. Test 5 data indicated that the girder rotated clockwise with the girder top flange
rotating slightly away from the strong wall while the girder bottom flange moved significantly toward the strong wall. The most significant increase in horizontal displacement of the connection occurred at the bottom flange of the girder. However, in Test 5 relatively no change was recorded between the potentiometer positioned at the bottom of the test beam. This seems to be the case with Test 10 as well because the potentiometer measuring movement of the bottom flange of the beam was unchanged after the shift. From Test 5, we see that the horizontal displacements of the bottom girder web and bottom single plate were affected. Of the bottom girder web and bottom of the single plate, the most significant change occurred at the single plate. Yet the shift in the linear potentiometers of the bottom single plate and girder web were proportionately smaller than the shift of the girder bottom flange. Also, the trend of Test 5 indicated that the component rotations increased in the following order: the beam, bolts, girder, girder web, and the single plate. From Figure K1, we see where the component rotations are with respect to each other prior to the shift (i.e. from left to right the magnitude of rotations increases as follows: the girder, the bolts, the beam, and then the single plate and girder web are approximately the same with the single plate rotation beginning to exceed the girder web rotation). Because of this, we can assume that post shift the components will be in relatively the same order and trending to that of Test 5 as previously mentioned. Taking each of these considerations into account, the applied shear versus rotation plot presented as Figure 5.10 was generated.

Figure K2, which is shown at the end of this section, depicts the raw applied shear at the connection versus the component deflections. Unfortunately, the shift, which affected the rotation behavior, also affected the vertical deflection measurements. In particular, the
measured vertical deflection of the test beam was affected the most. After the shift in the connection from approximately 80kips to 124kips, the string potentiometer measuring connection deflection of the beam indicated that the connection did not deflect. However, after 124kips the deflection of the beam continues in a typical load-deflection behavior. One can infer from this shape that the potentiometer measuring beam deflection jammed such that a constant deflection was measured. Confirming this assumption, the vertical deflection potentiometers of the girder and at the third point of the beam indicated increasing deflections. In addition, the potentiometer measuring vertical deflection of the single plate appeared to malfunction until approximately 180kips, at which time the behavior appears to be typical.

Compensating for the shift, Figure 5.13 shows the approximated component deflection behavior. A reasonable amount of potential connection shift was allowed. To account for the jammed connection pot for the beam, a parabolic approximation with respect to the applied load was generated. Because deflection plots are based upon absolute measurements and do not show relative deflections of each component, the bolts automatically reflect the modified beam connection deflection. For the single plate, again a parabolic approximation was used from just after the shift in the connection to approximately 180kips, at which point the parabolic approximation connects to the raw vertical single plate data.
FIGURE K1
Test 10A (Flexible, Slotted, Slab Restraint, Seven-Bolt)
Applied Shear vs. Rotation

Predicted Ultimate Strength of Bolts = 212.1 kips

A shift occurred in the connection at approximately 95 kips
A shift occurred in the connection at approximately 95 kips. The potentiometer measuring beam vertical displacement at the connection appears to have become stuck from 80-120 kips. Vertical beam and shear tab potentiometers, which are measured relative to the beam, reflect the vertical beam behavior.
APPENDIX L

TENSILE COUPON TEST RESULTS
Test Designation: Coupon Test
Coupon Number: A1
Date: 1/22/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.557
Width (in): 1.504

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
End Level: To Yield
Range 2 Rate: 0.6 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 53.1 ksi
.2% Offset Yield: 53.1 ksi
.5 in/in Yield: 53.1 ksi
Ultimate Strength: 70.9 ksi
Modulus of Elasticity: 29.9 ksi
% Elongation: 28%

Tension Test of Materials
(In accordance with ASTM A370-03)
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: A2
Date: 1/22/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.581
Width (in): 1.502

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
End Level: To Yield
Range 2 Rate: 0.6 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 54.0 ksi
.2% Offset Yield: 53.9 ksi
.5in/in Yield: 53.8 ksi
Ultimate Strength: 70.9 ksi
Modulus of Elasticity: 30.0 ksi
% Elongation: 28%

<table>
<thead>
<tr>
<th>Stress (ksi)</th>
<th>Strain (in/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>550</td>
<td>0.0000</td>
</tr>
<tr>
<td>0</td>
<td>0.0050</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stress (ksi)</th>
<th>Strain (in/in)</th>
</tr>
</thead>
<tbody>
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<td>0</td>
<td>0.0000</td>
</tr>
<tr>
<td>80</td>
<td>0.5000</td>
</tr>
</tbody>
</table>
**Tension Test of Materials**
(In accordance with ASTM A370-03)

**Test Designation:** Coupon Test  
**Coupon Number:** B1  
**Date:** 1/22/2004  
**Gage Length (in):** 8.00  
**Total Length (in):** 8.00  
**Length between Shoulders (in):** 9.00  
**Thickness (in):** 0.605  
**Width (in):** 1.507

**Test Setup:**  
**Procedure:** Tensile Test  
**Range 1**  
- **Rate:** 0.05 in/min  
- **End Level:** To Yield  
**Range 2**  
- **Rate:** 0.6 in/min  
- **End Level:** Sample Break

**Test Data:**  
- **.1\% Offset Yield:** 52.3 ksi  
- **.2\% Offset Yield:** 52.3 ksi  
- **.5\% in/in Yield:** 52.2 ksi  
- **Ultimate Strength:** 69.0 ksi  
- **Modulus of Elasticity:** 29.7 ksi  
- **% Elongation:** 28%
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: B2
Date: 1/22/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.610
Width (in): 1.508

Test Setup:
Procedure: Tensile Test
Range 1
   Rate: 0.05 in/min
   End Level: To Yield
Range 2
   Rate: 0.6 in/min
   End Level: Sample Break

Test Data:
.1% Offset Yield: 52.7 ksi
.2% Offset Yield: 52.7 ksi
.5in/in Yield: 52.8 ksi
Ultimate Strength: 69.4 ksi
Modulus of Elasticity: 29.7 ksi
% Elongation: 29%

---

![Graph](https://via.placeholder.com/150)

---

![Graph](https://via.placeholder.com/150)
**Tension Test of Materials**
(In accordance with ASTM A370-03)

Test Designation: Coupon Test  
Coupon Number: C1  
Date: 1/22/2004  
Gage Length (in): 8.00  
Total Length (in): 8.00  
Length between Shoulders (in): 9.00  
Thickness (in): 0.634  
Width (in): 1.515

**Test Setup:**
Procedure: Tensile Test  
Range 1  
Rate: 0.05 in/min  
End Level: To Yield  
Range 2  
Rate: 0.6 in/min  
End Level: Sample Break

**Test Data:**
- .1% Offset Yield: 53.6 ksi  
- .2% Offset Yield: 53.7 ksi  
- .5 in/in Yield: 53.8 ksi  
- Ultimate Strength: 69.5 ksi  
- Modulus of Elasticity: 29.3 ksi  
- % Elongation: 29%
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: C2
Date: 1/22/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.623
Width (in): 1.508

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
End Level: To Yield
Range 2 Rate: 0.6 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 53.5 ksi
.2% Offset Yield: 53.4 ksi
.5in/in Yield: 53.2 ksi
Ultimate Strength: 69.1 ksi
Modulus of Elasticity: 31.0 ksi
% Elongation: 29%

![Stress-Strain Diagram](chart1.png)
![Stress-Strain Diagram](chart2.png)
**Test Designation:** Coupon Test  
**Coupon Number:** D1  
**Date:** 1/22/2004  
**Gage Length (in):** 8.00  
**Total Length (in):** 8.00  
**Length between Shoulders (in):** 9.00  
**Thickness (in):** 0.594  
**Width (in):** 1.513

**Test Setup:**  
- **Procedure:** Tensile Test  
- **Range 1:** Rate: 0.05 in/min  
  End Level: To Yield  
- **Range 2:** Rate: 0.6 in/min  
  End Level: Sample Break

**Test Data:**  
- **.1% Offset Yield:** 52.1 ksi  
- **.2% Offset Yield:** 52.2 ksi  
- **.5in/in Yield:** 52.3 ksi  
- **Ultimate Strength:** 68.0 ksi  
- **Modulus of Elasticity:** 30.1 ksi  
- **% Elongation:** 29%
**Tension Test of Materials**
(In accordance with ASTM A370-03)

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<tr>
<td>Gage Length (in): 8.00</td>
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<tr>
<td>Total Length (in): 8.00</td>
</tr>
<tr>
<td>Length between Shoulders (in): 9.00</td>
</tr>
<tr>
<td>Thickness (in): 0.618</td>
</tr>
<tr>
<td>Width (in): 1.506</td>
</tr>
</tbody>
</table>

**Test Setup:**
- Procedure: Tensile Test
- Range 1: Rate: 0.05 in/min, End Level: To Yield
- Range 2: Rate: 0.6 in/min, End Level: Sample Break

**Test Data:**
- .1% Offset Yield: 52.7 ksi
- .2% Offset Yield: 52.6 ksi
- .5 in/in Yield: 52.5 ksi
- Ultimate Strength: 68.3 ksi
- Modulus of Elasticity: 29.6 ksi
- % Elongation: 29%

---

![Graph 1](image1)

![Graph 2](image2)
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: E1
Date: 1/20/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.604
Width (in): 1.512

Test Setup:
Procedure: Tensile Test
Range 1  Rate: 0.05 in/min
End Level: To Yield
Range 2  Rate: 0.4 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 51.7 ksi
.2% Offset Yield: 51.8 ksi
.5in/in Yield: 50.8 ksi
Ultimate Strength: 68.9 ksi
Modulus of Elasticity: 29.0 ksi
% Elongation: 29%

---

Stress (ksi) vs. Strain (in/in)

---

Stress (ksi) vs. Strain (in/in)
Test Designation: Coupon Test
Coupon Number: E2
Date: 1/22/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.618
Width (in): 1.505

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
   End Level: To Yield
Range 2 Rate: 0.6 in/min
   End Level: Sample Break

Test Data:
.1% Offset Yield: 52.1 ksi
.2% Offset Yield: 52.1 ksi
.5 in/in Yield: 52.1 ksi
Ultimate Strength: 68.9 ksi
Modulus of Elasticity: 29.5 ksi
% Elongation: 28%
**Test Designation:** Coupon Test  
**Coupon Number:** F1  
**Date:** 1/20/2004  
**Gage Length (in):** 8.00  
**Total Length (in):** 8.00  
**Length between Shoulders (in):** 9.00  
**Thickness (in):** 0.609  
**Width (in):** 1.506

**Test Setup:**  
**Procedure:** Tensile Test  
**Range 1**  
Rate: 0.05 in/min  
End Level: To Yield  
**Range 2**  
Rate: 0.4 in/min  
End Level: Sample Break

**Test Data:**  
.1% Offset Yield: 54.8 ksi  
.2% Offset Yield: 54.6 ksi  
.5in/in Yield: 53.7 ksi  
Ultimate Strength: 70.1 ksi  
Modulus of Elasticity: 28.5 ksi  
% Elongation: 30%

---

**Tension Test of Materials**  
(In accordance with ASTM A370-03)
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: F2
Date: 1/20/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.604
Width (in): 1.510

Test Setup:
Procedure: Tensile Test
Range 1
   Rate: 0.05 in/min
   End Level: To Yield
Range 2
   Rate: 0.4 in/min
   End Level: Sample Break

Test Data:
.1% Offset Yield: 55.0 ksi
.2% Offset Yield: 53.7 ksi
.5 in/in Yield: 53.2 ksi
Ultimate Strength: 69.1 ksi
Modulus of Elasticity: 28.6 ksi
% Elongation: 28%

**Graphs**

- Stress-strain curve (Initial loading)
- Stress-strain curve (Post-yield behavior)

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<th>Stress (ksi)</th>
<th>Strain (in/in)</th>
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<table>
<thead>
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<th>Stress (ksi)</th>
<th>Strain (in/in)</th>
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<tbody>
<tr>
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<td>0.3000</td>
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<td>0.4000</td>
</tr>
<tr>
<td>80</td>
<td>0.5000</td>
</tr>
</tbody>
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**Tension Test of Materials**  
(In accordance with ASTM A370-03)

<table>
<thead>
<tr>
<th>Test Designation: Coupon Test</th>
<th>Test Data:</th>
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<tbody>
<tr>
<td>Coupon Number: G1</td>
<td>.1% Offset Yield: 38.5 ksi</td>
</tr>
<tr>
<td>Date: 1/22/2004</td>
<td>.2% Offset Yield: 39.3 ksi</td>
</tr>
<tr>
<td>Gage Length (in): 8.00</td>
<td>.5in/in Yield: 39.5 ksi</td>
</tr>
<tr>
<td>Total Length (in): 8.00</td>
<td>Ultimate Strength: 61.9 ksi</td>
</tr>
<tr>
<td>Length between Shoulders (in): 9.00</td>
<td>Modulus of Elasticity: 27.0 ksi</td>
</tr>
<tr>
<td>Thickness (in): 0.380</td>
<td>% Elongation: 27%</td>
</tr>
<tr>
<td>Width (in): 1.506</td>
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</tr>
</tbody>
</table>

**Test Setup:**  
Procedure: Tensile Test

Range 1  
Rate: 0.05 in/min  
End Level: To Yield

Range 2  
Rate: 0.6 in/min  
End Level: Sample Break

**Graph:**

- Top graph: Shows stress-strain relationship with curves indicating different test stages.
- Bottom graph: Another stress-strain curve, possibly showing different material properties or test conditions.
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: G2
Date: 1/22/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.380
Width (in): 1.508

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
End Level: To Yield
Range 2 Rate: 0.6 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 39.1 ksi
.2% Offset Yield: 40.2 ksi
.5in/in Yield: 40.1 ksi
Ultimate Strength: 62.0 ksi
Modulus of Elasticity: 27.3 ksi
% Elongation: 21%
**Tension Test of Materials**  
(In accordance with ASTM A370-03)

<table>
<thead>
<tr>
<th>Test Designation: Coupon Test</th>
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<tr>
<td>Gage Length (in): 8.00</td>
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<tr>
<td>Total Length (in): 8.00</td>
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<tr>
<td>Length between Shoulders (in): 9.00</td>
</tr>
<tr>
<td>Thickness (in): 0.380</td>
</tr>
<tr>
<td>Width (in): 1.509</td>
</tr>
</tbody>
</table>

**Test Setup:**

- Procedure: Tensile Test
- Range 1: Rate: 0.05 in/min  
  End Level: To Yield
- Range 2: Rate: 0.4 in/min  
  End Level: Sample Break

**Test Data:**

- .1% Offset Yield: 37.8 ksi
- .2% Offset Yield: 39.5 ksi
- .5in/in Yield: 39.2 ksi
- Ultimate Strength: 61.5 ksi
- Modulus of Elasticity: 26.5 ksi
- % Elongation: 31%

---

![Stress-Strain Graph](image1)

![Stress-Strain Graph](image2)

---

563
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: H2
Date: 11/5/2003
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.380
Width (in): 1.506

Test Setup:
Procedure: Tensile Test
Range 1
Rate: 0.10 in/min
End Level: To Yield
Range 2
Rate: 1.0 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 38.1 ksi
.2% Offset Yield: 39.3 ksi
.5in/in Yield: 38.7 ksi
Ultimate Strength: 62.8 ksi
Modulus of Elasticity: 18.7 ksi
% Elongation: 29%

---

Strain (in/in) vs. Stress (ksi)

---

Strain (in/in) vs. Stress (ksi)
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: X1
Date: 10/13/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.592
Width (in): 1.501

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
   End Level: To Yield
Range 2 Rate: 0.6 in/min
   End Level: Sample Break

Test Data:
.1% Offset Yield: 55.0 ksi
.2% Offset Yield: 55.0 ksi
.5in/in Yield: 55.0 ksi
Ultimate Strength: 70.7 ksi
Modulus of Elasticity: 29.6 ksi
% Elongation: 28%

---

---
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: X2
Date: 10/13/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.620
Width (in): 1.501

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
End Level: To Yield
Range 2 Rate: 0.6 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 53.0 ksi
.2% Offset Yield: 53.1 ksi
.5in/in Yield: 53.2 ksi
Ultimate Strength: 70.0 ksi
Modulus of Elasticity: 30.0 ksi
% Elongation: 28%
Test Designation: Coupon Test  
Coupon Number: Y1  
Date: 10/13/2004  
Gage Length (in): 8.00  
Total Length (in): 8.00  
Length between Shoulders (in): 9.00  
Thickness (in): 0.683  
Width (in): 1.502

Test Setup:  
Procedure: Tensile Test  
Range 1 Rate: 0.05 in/min  
End Level: To Yield  
Range 2 Rate: 0.6 in/min  
End Level: Sample Break

Test Data:  
.1% Offset Yield: 60.3 ksi  
.2% Offset Yield: 60.2 ksi  
.5in/in Yield: 60.0 ksi  
Ultimate Strength: 75.0 ksi  
Modulus of Elasticity: 30.3 ksi  
% Elongation: 25%

---

Tension Test of Materials  
(In accordance with ASTM A370-03)

---

![Stress-Strain Diagram](image1)

![Stress-Strain Diagram](image2)
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: Y2
Date: 10/13/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.651
Width (in): 1.500

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
End Level: To Yield
Range 2 Rate: 0.6 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 58.6 ksi
.2% Offset Yield: 59.0 ksi
.5in/in Yield: 58.7 ksi
Ultimate Strength: 75.0 ksi
Modulus of Elasticity: 30.1 ksi
% Elongation: 24%

---

Graph 1:
- Stress (ksi) vs. Strain (in/in)
- Range 1 and Range 2
- Linear region for Elasticity
- Non-linear region for Yield and Ultimate Strength

Graph 2:
- Stress (ksi) vs. Strain (in/in)
- Initial linear region
- Non-linear region for Elasticity and Yield

---

568
Tension Test of Materials
(In accordance with ASTM A370-03)

Test Designation: Coupon Test
Coupon Number: Z1
Date: 10/13/2004
Gage Length (in): 8.00
Total Length (in): 8.00
Length between Shoulders (in): 9.00
Thickness (in): 0.367
Width (in): 1.500

Test Setup:
Procedure: Tensile Test
Range 1 Rate: 0.05 in/min
End Level: To Yield
Range 2 Rate: 0.6 in/min
End Level: Sample Break

Test Data:
.1% Offset Yield: 44.5 ksi
.2% Offset Yield: 44.6 ksi
.5in/in Yield: 44.5 ksi
Ultimate Strength: 66.2 ksi
Modulus of Elasticity: 28.4 ksi
% Elongation: 28%

---

---
**Tension Test of Materials**  
(In accordance with ASTM A370-03)

<table>
<thead>
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<th>Test Designation: Coupon Test</th>
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<tr>
<td>Thickness (in): 0.366</td>
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<tr>
<td>Width (in): 1.503</td>
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**Test Setup:**

Procedure: Tensile Test

- **Range 1**
  - Rate: 0.05 in/min
  - End Level: To Yield

- **Range 2**
  - Rate: 0.6 in/min
  - End Level: Sample Break

**Test Data:**

- .1% Offset Yield: 43.9 ksi
- .2% Offset Yield: 44.1 ksi
- .5in/in Yield: 44.1 ksi
- Ultimate Strength: 66.3 ksi
- Modulus of Elasticity: 29.7 ksi
- % Elongation: 28%

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![Stress-Strain Diagram](image)

- Stress (ksi) vs. Strain (in/in)
- **Range 1**
  - 0.0000 to 0.0050
  - 0.0000 to 0.0010
  - 0.0000 to 0.0020
  - 0.0000 to 0.0030
  - 0.0000 to 0.0040
  - 0.0000 to 0.0050

- **Range 2**
  - 0.0000 to 0.0050
  - 0.0000 to 0.0010
  - 0.0000 to 0.0020
  - 0.0000 to 0.0030
  - 0.0000 to 0.0040
  - 0.0000 to 0.0050