University of Cincinnati

Date: 7/14/2015

I, Prabhanjan B Wagh, hereby submit this original work as part of the requirements for the degree of Master of Science in Civil Engineering.

It is entitled:
An Investigation of Current Practice in the Design of all-Bolted Extended Double Angle Connections in a Beam-to-Girder Connection

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An Investigation of Current Practice in the Design of all-Bolted Extended Double Angle Connections in a Beam-to-Girder Connection

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By

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Abstract

This thesis presents a study of all-bolted extended double angles in a beam-to-girder connection in order to investigate the strength of the web of the girder that is supporting the beam. Extended connections have multiple advantages over the standard connections: Since coping is avoided, it reduces skilled labor and time resulting in reduced construction costs. Due to coping, reduced beam sections are exposed to possible failure due to lateral torsional buckling and local web buckling of the coped section.

A set of double angle extended beam-to-girder connections were designed for a range of angle thicknesses, girder web thicknesses, girder depths, beam depths, and bolt diameters using current AISC specifications. Only one sided beam-to-girder connections were considered in this study. Three dimensional non-linear finite element models of these connections were developed and studied. All of the connection components such as beam, girder, double angles and bolts were modelled using solid elements. A monotonically increasing load was applied on supported beam until failure. The results of this numerical modelling were compared with AISC expected ultimate shear strengths of the connections. Investigation of girder web strength was also carried out. Plastic mechanism at the girder web was not seen in any of the connection. Girder web showed enough strength for the effects due to eccentricity. AISC design equations were found to be conservative in predicting the ultimate strength of the connection. Design suggestions were made based on the behavior of component parts in the connection.

Keywords: Extended Double Angle Connections, Eccentricity, Beam-to-Girder Connection, Finite Element Modeling.
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“Farthest from your mind is the thought of falling back. In fact it isn’t there at all. And so you dig your hole carefully and deep, and wait, not for the mythical superman, but for the enemy you had beaten twice before and will again.” - Excerpt from the Curahee scrapbook for Easy Company, 2nd Battalion of the 506th Parachute Infantry Regiment of the 101st Airborne Division
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Chapter 1: Introduction

1.1 Overview

AISC (2010) Specification for Structural Steel Buildings (ANSI/AISC 360-10) - B3.6 classifies the steel connections based on the amount of restraint developed by them. Accordingly, they can be ‘Fully Restrained (FR) Moment Connections’ or ‘Partially Restrained (PR) Moment Connections or simple connections’. Simple connections have negligible resistance to rotation and are designed to transmit end shear only.

These simple shear connections are used to connect a beam-to-beam, a beam-to-girder, or column flanges to webs when simple support of the beam has been assumed. To design or construct these shear connections; double angles, a single angle, a single plate or tee sections can be used.

Here and henceforth, the term girder is used to indicate a primary beam or a supporting member. Similarly, the term beam is used to indicate a secondary or a supported member. The current practice of forming a simple shear connection between a beam and a girder involves cutting of a part of a beam flange to provide clearance for the girder flange. This method is termed as “Coping” however exposes the remaining beam sections to the possibility of a failure due to the lateral torsional buckling (LTB) and the local web buckling of the coped section. “Coping” also requires skilled labor, money, and time which subsequently scales up the cost of construction. These disadvantages can be overcome by making use of extended connections. Numerous experimental and finite element studies have been carried out in the past, which have addressed the use of extended shear tab and extended shear plate connections providing feasible alternative to “Coping”.

In this study, a case of double angles in an extended beam-to-girder connection was considered. The major advantages of using a double angle connection was that they can resist
larger end reactions. It is primarily because the supported member bolts are in a double shear and the eccentricity perpendicular to the beam axis need not to be considered for workable gages. However, for the gages larger than workable gages (3 in.) eccentricity cannot be neglected and has to be accounted for.

![Diagram of connections](image)

**Figure 1.1 All Bolted Double Angle Connection**

In extended connections, the eccentricity, \( e \), varies depending on the leg length of the supported leg, i.e. the outstanding leg length of the angle. Here, eccentricity was assumed to be the distance from the centerline of the supported bolts to the face of the supporting web. The flexural capacities of the connection and all limit states for bolts are dependent on the eccentricity. All the bolt limit states were treated as eccentrically loaded. Eccentricity produces both a rotation and a translation of one connection element with respect to other. (AISC, 2011) Consideration of both of these effects are included in the AISC Manual Part 7 tables for eccentrically loaded bolt groups. (AISC, 2011)

### 1.2 Outline

The following procedure was observed in order to achieve the objective of this study.
The goal of the proposed work was to investigate the current practices in the design of all bolted extended double angle connections in a beam-to-girder connection. This was achieved by investigating the strength of the web of the girder that was supporting the beam. The strength of the web was investigated by formulating “a simple to observe” procedure in a non-linear finite element modelling package for which a database of connections was created using current design provisions and previous research works.

All bolted extended double angle connections were designed with variables such as angle thickness, girder depth, girder web thickness, and beam depth with three bolt diameters of size 3/4”, 7/8” and 1”.

Three dimensional non-linear finite element model of a test carried out by Sherman et. al (2002) on an extended shear tab beam-to-girder connection was constructed. Experimental results were compared with analytical results to check the fidelity of the implemented computational techniques with Abaqus. The correct matching between Load vs Displacement curve of experimental and analytical results validated finite element modeling and helped to analyze the extended double angle connections with various connection parameters.

Connections with above variables were designed with the stiffener at the girder web. Total 30 connections for each combination, with and without a stiffener, were designed in four groups of variables mentioned above. Connections were designed for the eccentricity 6.75” within these combinations. The performance of these models with variables mentioned above was estimated by comparing curves of ‘Shear force at the connection vs Displacement along the bolt line’.
5) **Summary:**

This chapter deals with an introduction to extended connections, objective behind this research, and the basic outline to achieve this objective. Chapter 2 summarizes the literature review on extended connections and double angle shear connections. Chapter 3 outlines the detailed connection design procedure for all bolted extended double angles. Chapters 4 and 5 gives the procedure followed for the general finite element modeling and its application in the experimental verification model. Results of finite element modeling are discussed in chapter 6. Chapters 7 summarizes the research and its conclusions along with the recommendations for future studies.

The end goal of this research was to enhance the existing knowledge and understand the behavior of all bolted double angle extended shear connections. The final results of this study will provide a comprehensive investigation on the strength of the web of the girder that is supporting the beam in all bolted double angle extended connections.
Chapter 2: Literature Review

The onus of this section is to review and synthesize the available literature and completed studies and to document the previous works relevant to the topic of this research. It should be noted that the research works mentioned here are confined to the studies done in extended connections only, aligning with the scope of this thesis.

2.1 Previous Research:

Number of researchers have studied the design and behavior of extended connections in a beam-to-column connections and beam-to-beam connections. Few of them are discussed below.

2.1.1 Higgins A. et al (2005):

In 2005, a research group from the University of Florida conducted a study on “Design of all bolted extended double angle, single angle and tee shear connection”. This report covered the design of extended connections that involved beam and girders. Authors developed the design tables for six, seven, eight and nine inch leg angles; 3/4, 7/8, and 1 inch diameter ASTM A325/F1852 and ASTM A490 bolts, and ASTM A36 and ASTM A992 angle materials. These tables included design resistances for a wide range of angle and tee materials and bolts diameters as well as different connection types. Only simple shear connections were considered.

Assumptions in the designs were:

1. There was no end moment developed in the beam and that the beam supports will allow unrestrained rotations.
2. The connection was designed in order to prevent any slip between the faying surfaces.
3. The load passes through the center line of the girder (supporting member).
4. Prying action was ignored because the plastic hinge is developed at the girder web.
### Table 2.1: Design Table for All-Bolted Extended Double Angle Connections (Higgins et. al 2005)

<table>
<thead>
<tr>
<th>N (# of bolts)</th>
<th>Bolt Diameter</th>
<th>Angle Outstanding Leg</th>
<th>ASTM Design</th>
<th>Thread Condition</th>
<th>Hole Type</th>
<th>Angle Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>3/4</td>
<td>6</td>
<td>A325/ F1852</td>
<td>N</td>
<td>STD</td>
<td>5/16 3/8 1/2</td>
</tr>
<tr>
<td>SC Class A</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>STD</td>
<td>14.9 17.3 17.3</td>
</tr>
<tr>
<td>SC Class A</td>
<td></td>
<td></td>
<td></td>
<td>OVS</td>
<td>9.7</td>
<td>9.7 9.7 9.7</td>
</tr>
<tr>
<td>SC Class B</td>
<td></td>
<td></td>
<td></td>
<td>OVS</td>
<td>13.8</td>
<td>14.6 14.6 14.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SSLT</td>
<td>14.6</td>
<td>14.6 14.6 14.6</td>
</tr>
<tr>
<td>3</td>
<td>3/4</td>
<td>6</td>
<td>A325/ F1852</td>
<td>N</td>
<td>STD</td>
<td>5/16 3/8 1/2</td>
</tr>
<tr>
<td>SC Class A</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>STD</td>
<td>33.3 38.6 38.6</td>
</tr>
<tr>
<td>SC Class A</td>
<td></td>
<td></td>
<td></td>
<td>OVS</td>
<td>21.6</td>
<td>21.6 21.6 21.6</td>
</tr>
<tr>
<td>SC Class B</td>
<td></td>
<td></td>
<td></td>
<td>OVS</td>
<td>30.9</td>
<td>32.6 32.6 32.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SSLT</td>
<td>32.6</td>
<td>32.6 32.6 32.6</td>
</tr>
</tbody>
</table>

**Notes:**
- Angles are assumed to be A36 Steel
- Supporting and supported members are assumed to be A992 Steel
- Represents a bolt bearing limit state
- Represents a bolt shear limit state
- Represents a slip-critical limit state

Higgins A. et al (2005) assumed that most of the rotation in the connection occurs in the legs of the angle that are in the plane of the supporting member. It was assumed that the plastic
hinge was developed at the centerline of the girder (supporting member) which requires that the outstanding leg bolts be designed for an eccentric load. The following limit states were considered in the design of all bolted extended double angle connection.

a. Shear Yielding of the Angles  
   b. Shear Rupture of the Angles  
   c. Flexural Yielding of the Outstanding Angle Leg  
   d. Flexural Rupture of the Angles  
   e. Block Shear Rupture of the Angles  
   f. Bolt Bearing  
   g. Bolt Slip  
   h. Bolt Shear

Since coping is not involved in extended connection the lateral-torsional buckling, local web buckling and fatigue cracking of the beam web need not to be considered. Effects due to load eccentricity were considered in bolt limit states.

The authors also carried out a finite element study that was conducted on an extended single angle bolted connection. A comparison was made between the behaviors of the extended connection to that of a standard equal leg angle connection which would be used in a coped beam. The authors created the basic FE model where the position of the bolt line in the in-plane angle leg corresponded to that of a standard coped beam to girder connection. This model was then modified to replicate the protruded angle leg of an extended connection where it was not needed for the supported beam to be coped. Only a part of the girder web where the angle was connected to girder web and single angle was modeled.

The authors performed a static analysis solution scheme using the ADINA FE software package, Version 8.1. For fixed boundary conditions along the edges which were away from the portion of the girder web and where the angle was attached, the authors found out that finite element models of all-bolted extended single angle connection primarily lost their strength due to
the bolt bearing on the angle material at the location of the top hole in the angle leg. The authors remodeled the same connection with a pinned boundary condition along the edges of the girder web at the same location as before. The pinned boundary connection had a noticeable effect on the rotational behavior of the connection and also had an influence on the strength of the connection. With this changed boundary condition, the connection lost its strength due to bolt bearing on the girder web.

2.1.2 AISC Steel Construction Manual, 14th Ed. (AISC, 2011):

In the AISC Steel Construction Manual 14th Ed. (AISC, 2011) Part 10 covers the design methods for simple connections. Tables in this section includes all-bolted double angle connections, bolted/welded double angle connections, all-welded unstiffened seated connections, all-bolted stiffened seated connections, and bolted/welded stiffened seated connections. Table 10-1 is a design aid for all-bolted double-angle connections. Available strengths are tabulated for supported and supporting member material with $F_y = 50$ ksi and $F_u = 65$ ksi and angle material with $F_y = 36$ ksi and $F_u = 58$ ksi. Tables are provided for 2 through 12 rows of ¾ in., 7/8 in. and 1 in. diameter Group A and Group B bolts (as defined in the AISC specification Section J3.1) at 3-in. spacing and angle edge distance to be 1-1/4 in. These tables also provides beam web and supporting member available strengths, per in. of web thickness considering the limit state of bolt bearing on the beam web and on the support.

Eccentricity due to workable gages can be neglected. For extended connections, effects due to load eccentricity were considered in bolt limit states. Eccentricity produces both rotation and a translation of one connection element with respect to other (Higgins A., et. al 2005). The combined effect of rotation and translation is equivalent to a rotation about a point defined as the instantaneous center of rotation (ICR). To take non-linearity due to eccentricity in bolt groups into account, Chapter 7 of the AISC manual covers tables with ICR method coefficients. These tables
are designed for a bolt group containing 2 to 12 bolts and eccentricities up to 36 inches. All design steps for bolt limits were carried out as outlined in the chapter 7 of the AISC manual and taking into account basic mechanics.

Another analysis method i.e. the elastic vector method is useful for this type of eccentricity. The elastic vector method is simple to use but may be excessively conservative because it neglects the ductility of the bolt group and the potential for load distribution (AISC, 2011). Since the instantaneous center of rotation method includes the nonlinearity of the bolt deformation, this method was implemented to check all bolt limits.

Due to broad applicability of the AISC manual to the current design practice, all other limit states were designed according to respective sections in the manual.

2.1.3 Sherman D., Ghorbanpoor A. (2002):

Sherman and Ghorbanpoor (2002) developed a design procedure for the extended shear tab connections attached to a column web or girder. This research program consisted of thirty one full scale tests in two groups; the first group consisted of eight tests on unstiffened extended shear tabs and the second group consisted of twenty three tests on stiffened extended shear tabs. The extended shear tab was connected either to a column web or to a girder web. This study was conducted by varying the stiffness of the supported beam, size of the supporting member, weld configuration, and use of the standard holes with snug tight bolts or slotted holes with fully tightened bolts. In a stiffened extended connection, stiffening plates were welded on the top or bottom of the shear tab.

Two new limit states were observed for the unstiffened extended connections: column web mechanism and plate twisting. However, some of the stiffened extended shear tab connections lost their strength due to twisting in the shear tab. For stiffened connections the column web mechanism was not expected as the stiffener plate prevents the punching of the shear tab into the column or beam web.
Figure 2.1: Unstiffened Extended Shear Tab Connection (Sherman et al. 2002)

Figure 2.2: Stiffened Extended Shear Tab Connection (Sherman et al. 2002)

Figure 2.3: Observed Failure Modes (Sherman et al. 2002)

a) Column Web Mechanism
b) Plate Twisting
Sherman defined the failure of shear tab due to twisting when the maximum shear stress reached the shear yielding capacity of the material. This limit state is given as,

\[ V_{\text{max}} = V_t = 0.3LtF_y \]

Where, \( L \) is the shear tab depth, \( t \) is the shear tab thickness and \( F_y \) is the material’s yield stress.

### 2.1.4 Metzger (2006):

Metzger (2006), for Master’s thesis, tested eight full scale single plate shear connections, (four conventional shear plate and four extended shear plate connections) at Virginia Polytechnic Institute and State University. The test setup consisted of a test beam, supporting column, loading support frame, and lateral bracing frames. The test beam was welded to a column flange at one end with a shear tab connection and was supported by a roller at the other end. A992 steel material was used for the beam and the column while A572 Gr. 50 steel was used for the shear tabs. The shear tabs were welded to column flange and bolted to the beam web.

The experimental results were compared to the limit states determined based on the outline in the AISC Steel Construction Manual 13th Ed. (AISC, 2005). Metzger found that the AISC gives conservative capacity values than observed. Twisting was observed in one of the extended shear tab and the beam failed at the mid-span due to lateral torsional buckling. The author recommended to provide bracing at the connection when the eccentricity is large. In addition, Metzger also recommended to examine the bolt-diameter-to-plate thickness ratio in order to determine the maximum allowable plate thickness.

### 2.1.5 Thornton and Fortney (2011)

Thornton and Fortney (2011) reviewed the design procedure for extended single-plate connections outlined in the 13th edition of the Steel Construction Manual. Their focus primarily was to evaluate the need for stiffeners and the effect of small eccentricity due to the lapping of the
Literature Review

web with the beam web. The experiments done by Sherman and Ghorbanpoor (2002) were critically reviewed considering the lateral twist in the shear tabs. Thornton and Fortney (2011) found from the evaluation of the test data by Sherman and Ghorbanpoor (2002) that, the connection strengths computed by the AISC design procedures were very conservative compared to the beams they were supporting and did not represent practical connections. Additionally, they studied the effect of lateral torsional buckling on the strengths of the beam and the connections. Thornton and Fortney developed the equation by which the need for stiffeners could be established.

\[ R_n = \frac{1500\pi lt^3}{a^2} \]

*Equation 2.2*

\[ \eta = \frac{\phi R_n}{R_u} \]

*Equation 2.3*

where, \( \eta \) is the ratio between the connection capacity and demand, \( l \) depth of shear tab, \( t \) thickness of shear tab, \( a \) length of shear tab from support to first line of bolts and \( R_n \) the connection strength, and \( R_u \) is the factored shear.

They also studied the effect of torsion on the capacity of the beam and the connection. The top flange of the beam is always connected to the floor slab restraining the beam along the length from out-of-plane displacement. They proposed the following equation (LRFD) to take into account the torsional resistance of the entire system. All variables are in specification notations.

\[ M_{t,s} \leq \phi_y \left( 0.6 F_{3y} \right) \frac{R_u}{lt_p} \left[ lt_p^3 - \frac{2R_u^2(t_w+t_p)b_f}{(\phi_y F_{yb}) Lt_w^2} \right] \]

*Equation 2.4*

They suggested to use the above mentioned Equations 2.2, 2.3 and 2.4 on extended shear tab connections in order to determine the need of stiffeners, and to check the lateral torsional buckling capacity of the supported member.
2.1.6 **Guravich S. and Dawe J.L. (2003):**

Guravich and Dawe tested bolted double angle shear connections in combined shear and tension. A total of twenty six, two single bolt row connections, consisting of double angles bolted to a column flange and a beam web were tested. The test beam was rotated and held at 0.03 radians relative to the column during loading. A shear load was applied and was held constant at either 50 or 100 percent of the factored shear capacity of the connection while a tension load was applied and gradually incremented to failure. Guravich concluded that the bolted angle shear connections have a significant tension resistance which decreased as the shear load was increased.

2.2 **Summary**

This chapter discussed the past studies on extended connections. Higgins et al were the first to propose extended shear connections with double angles, single angles, and tee sections. Sherman and Ghorbanpoor tested and developed a design procedure for stiffened and unstiffened extended shear tab connections. Metzger studied the single shear plate connections with standard and extended configurations. Thornton and Fortney reviewed the design procedure for single plate extended connections and proposed the equations to check the need for stiffeners and to check the lateral torsional buckling capacity of the supported member. Guravich and Dawe tested bolted double angle shear connections in combined shear and tension and found out that the bolted angle shear connection had a significant tension capacity which decreases with the increasing shear load.

2.3 **Need for Research**

Most of the research done in the past was concerned with the beam-to-column connections. Beam-to-girder connections, where the connections are usually made between the girder web and the web of the beam, vary in behavior as compared to beam-to-column connections, where the beam web is connected to the column flange. The stiffness between the girder web and the column flange vary affecting the overall behavior of the connection. (Higgins A. et. al, 2005) Also, the
current AISC Steel Construction Manual, 14th Ed. (AISC, 2011) still lacks the specific design guides for all-bolted extended double angle connections where the eccentricity is larger than workable gages and which needs to be considered for ensuring strength of supporting side of the connections.
Chapter 3: Connection Design

3.1 Introduction

This chapter describes the design procedure for all-bolted extended double angle connections. The connection designs were done considering all the limit states explained in Chapter 2 by using current *AISC, Steel Construction Manual, 14th Ed.* (AISC, 2011) because of its broad applicability to current practice. The tables and design formulae available in the current steel construction manual were adopted to design the connection.

3.2 Floor Panel

A 20 feet x 20 feet floor plan was considered and used in this research. The assumed number of deck spans was three with the beam spaced at 6.67 feet. A 2 inch deep metal deck with a concrete topping was used. The average rib width provided was 6 inches with a rib spacing of 12 inches. These values were kept constant. The concrete used for the slab was 4,000 psi concrete with a weight of 150 pcf. The total slab depth was 5 inches.

![Figure 3.1: Floor Plan](image)
To design the beam and girder panel, a software tool “Floor Framing V14.0” developed by AISC Steel Solutions Center was used. This tool provides a range of beam and girder sizes for user defined parameters such as floor panel length and width, number of deck spans, loads such as dead load, live load, superimposed dead load, and construction live load etc.

![Image](Image)

*Figure 3.2: "Floor Framing V14.0" Steel Solutions Center AISC.*

### 3.3 Loading, Vibration and Deflection Criteria

The floor dead load combined with concrete slab was calculated to be 91.53 psf. A conservative live load of 200 psf was assumed. The superimposed dead load and construction live load were assumed to be 20 psf. Live load reduction was considered. This tool checked the beam, girder and beam – girder combination adequacy for strength, deflection and vibration criteria. No vibration criterion was applied to the floor panel. The allowable deflection ratios were kept at the default values of L/240 for the dead load (DL) and L/360 for superimposed dead load (SDL) and live load (LL).
3.4 Shear Studs and Composite Action

For shear stud design, 3/4 inch diameter and 3 inch height studs were used. This tool also allows the user to design for composite or non-composite sections. Also, the tool facilitates for the minimum and maximum percentage of composite action in the member. For this study, a minimum of 25% to a maximum of 100% composite action was considered. The default values of cambering were used in the tool.

3.5 Member Selection

“Floor Framing V14.0” allows the user to choose the range of the wide flange shapes from the default list for beam of size W24 to W12 and for girder W44 to W12. In extended connections the outstanding length of the angle is limited by the flange width of the girder. Maximum outstanding length of the angle available in current AISC Steel Construction Manual (AISC, 2011) is 8 inch. For this study the largest possible section combination of the angles and girder was chosen. This study focused on the maximum possible eccentricity within these combinations which comes to 6.75 in. after considering edge distance for angles to 1-1/4 in. Response of the extended connections were measured where uniformly distributed load was applied monotonically over the beam.

3.6 Design Variables:

The objective of studying these connections was to investigate the web strength of the girder that was supporting the beam. In order to achieve this objective connections were designed according to AISC Steel Construction Manual (AISC, 2011). The design variables considered were angle thickness, girder web thickness, girder depth, beam depth, and bolt diameters. Each connection mentioned in Table 3.1 was designed with three bolt diameters 3/4 inches, 7/8 inches, and 1 inch. All these connections were divided in to two phases. The first phase consisted of the connections without a stiffener and the second phase was designed with a stiffener plate of
thickness 0.5 inches at one side of the girder web. A total of 60 connections were designed in two phases with and without stiffener that comprised of 10 connections for each diameter. The response of all these connections was evaluated by conducting 3D nonlinear finite element analyses.

Table 3.1 enlists the connections designed with A325-X bolts.

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

1) Bolt bearing on the supported beam (AISC Spec. J3-6a)
2) Bolt bearing on the girder (AISC Spec. J3-6a)
3) Block shear rupture of angle (AISC Spec. J4-5)
4) Block shear rupture of beam web (AISC Spec. J4-5)
5) Shear yielding of angle (AISC Spec. J4-3)
6) Shear rupture of angle (AISC Spec. J4-4)
7) Flexural rupture of angle (AISC Manual Eq. 9-4)
8) Flexural yielding of outstanding leg of angle (AISC Spec. J4.5)
9) Bolt shear (AISC Spec. J3-1)
10) Gross shear on beam web (AISC Spec. J4-3)
11) Shear and tension from eccentric loading at girder web bolts (AISC Spec. J3.7, J3-2, J3-3a)
12) Combined Shear yielding, shear buckling and flexural yielding of angle

All design calculations used expected material strength values instead nominal capacity values to calculate the connection strength. A detailed design calculation is presented in Appendix A.
### Table 3.1 Connection Details

<table>
<thead>
<tr>
<th>Variable</th>
<th>Beam</th>
<th>Girder</th>
<th>Angle</th>
<th>Eccentricity (in.)</th>
<th>Bolt Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle Thickness</td>
<td>W16x45</td>
<td>W21x93</td>
<td>2L8x4x7/16</td>
<td>6.75</td>
<td>A325-X</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2L8x4x9/16</td>
<td>6.75</td>
<td>A325-X</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2L8x4 x 3/4</td>
<td>6.75</td>
<td>A325-X</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2L8x4x1</td>
<td>6.75</td>
<td>A325-X</td>
</tr>
<tr>
<td>Girder web thickness</td>
<td>W16x45</td>
<td>W21x93</td>
<td>2L8x4x7/16</td>
<td>6.75</td>
<td>A325-X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>W21x73</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>W21x48</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Girder depth</td>
<td>W16x45</td>
<td>W24x76</td>
<td>2L8x4x7/16</td>
<td>6.75</td>
<td>A325-X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>W21x73</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>W18x65</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filler Beam depth</td>
<td>W16x45</td>
<td>W21x73</td>
<td>2L8x4x7/16</td>
<td>6.75</td>
<td>A325-X</td>
</tr>
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<td></td>
<td>W18x46</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>W21x44</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Chapter 4: Methodology

The term *finite element* was first coined and used by Clough in 1960. (Tirupathi R. B., Ashok D. B., 2002). The finite element method has become a powerful tool for the numerical solution of a wide range of engineering problems. The applications of the finite element method range from deformation and stress analysis of automotive, aircraft, building, and bridge structures to field analysis of heat flux, fluid flow, magnetic flux, seepage and other flow problems. With the advances in computer technology and CAD systems, modeling of complex problems has become relatively easy.

In this method of analysis, a complex region defining a continuum is discretized into simple geometric shapes called elements. The material properties and the governing relationships are considered over these elements and expressed in terms of unknown values at element nodes. The integration of these element nodes duly considering the loading and constraints results in a set of equations, the solution of which provides the approximate behavior of the initially complex continuum. This chapter describes the basic procedure and basis which is necessary to explain the chosen parameters which will be used for the 3D FE modelling in Abaqus. For modelling purpose, Ruffley’s (2011) work is used as a reference for this study.

4.1 Choice of Solver

A variety of finite element studies addressing the complex behavior of steel structures subjected to loads has been carried out in the past. Researchers have used commercially available finite element modelling packages to carry out deformation and stress analysis. Among them, Abaqus Unified FEA product suite developed by Dassault Systems SIMULIA Corporation has been proven to produce reliable results.
Methodology

Abaqus/CAE was used as a pre-processor and Abaqus/Standard solver was selected for the analysis of the finite element models for this study. With Abaqus/CAE we can efficiently create, edit, monitor, diagnose, and visualize advanced Abaqus analyses (Abaqus 6.10, 2011a). Abaqus/Standard is a general-purpose analysis product that can solve a wide range of linear and nonlinear problems involving the static, dynamic, thermal and electrical response of components. Abaqus/Standard solves a system of equations implicitly at each solution “increment.” (Abaqus 6.10, 2011a).

4.2 Parts

The first function in Abacus/CAE module is Part. The Part function module was used to create and import individual instances in the assembly of the structure by sketching their geometry. These parts were then imported in the Assembly module to create instances of them. Eventually, these instances were positioned as required in a global coordinate system to create an assembly. In this study, components labeled as Part consisted I-beams, angles and bolts. They were 3 dimensional, deformable, and solid extrusions.

4.3 Bolt

A325 bolts of diameter 3/4”, 7/8” and, 1” were used in this study. The methodology used to model the bolt was similar to the one implemented by Ruffley (Ruffley D.J., 2011) Wurzelbacher (Wurzelbacher K.P., 2012) and Kennedy (Kennedy R., 2014) at the University of Cincinnati. Ruffley recommended to idealize the bolt head and nut as cylinders instead of hex heads. Wurzelbacher compared the behavior of simplified bolts recommended by Ruffley and refined the bolts with threads. Wurzelbacher concluded that Ruffley’s simplified bolts are easy to model and appropriate to use. There is a small difference between a refined bolt and simplified bolt. The washer and nut were modeled as a single part. For this single entity, the washer’s nominal
outer diameter as the diameter and the nut height plus the maximum washer thickness as the total thickness. (Ruffley D.J., 2011)

To model the bolt, the values of the parameters cited above were used from AISC table 7-14, “Dimensions of high strength fasteners” (AISC 2011). Ruffley proposed a method to account for the threads in the bolt by including them in the grip. A cut section was created to model the threads instead of actually modelling the threads. The effective area that is the net tensile area for threaded regions can be calculated by Equation 4.1.

\[ A_{net} = \frac{\pi}{4} \left[ d_{th} - \frac{0.9743}{n} \right]^2 \]  \hspace{1cm} \text{Equation 4.1}

A cut section from the core of the bolt in the threaded region was removed to model the net section area. The diameter of the cut section is given by the following formula as solved by Ruffley (Ruffley D.J., 2011).

\[ d_{void} = \sqrt{1.9486 \frac{d_{th}}{n} - \frac{0.94926}{n^2}} \]  \hspace{1cm} \text{Equation 4.2}

Figure 4.1: Meshing and Partitioning Details for A325, 3/4” dia. Bolt

4.4 Material Properties

Followed by the Part module, the Property module was used to create sections and material definitions and these were assigned to the regions of parts. A section definition is an entity containing information about the properties of a part or a region of a part. Some typical material
Methodology

property values were assumed. Poisson’s ratio was assumed to be 0.3. The modulus of elasticity was assumed to be 29,000 ksi. The thermal coefficient was set to $6.6 \times 10^{-6}$ strain per degree Fahrenheit. A36 steel was considered for angle sections. The expected material strength values for A36 steel were used instead of nominal capacity values. A992 steel was considered for girder and beam. Coupon data for A992 was taken from the test recorded at the University of Cincinnati. A325 bolts were used for all models. For A325, the expected ultimate strength of 136 ksi was used in lieu of the ultimate strength of 120 ksi as per the recommendations by Ruffley (Ruffley D.J., 2011). Since true stress and true strain values are required as an input in Abaqus, these values were found out by the Equations 4.3 and 4.4.

\[
\text{True Stress} = \text{Engineering stress} \times (1 + \text{Engineering strain}) \quad \text{Equation 4.3}
\]

\[
\text{True Strain} = \ln x (1 + \text{Engineering Strain}) \quad \text{Equation 4.4}
\]

All stress-strain values from the above data were converted to true stress and true plastic strain using the equations mentioned above and were used for finite element analysis.

\[
\begin{array}{cccccc}
\text{A36 Atlas - Expected} & \text{A992 Coupon} \\
\hline
\text{Engineering} & \text{True} & \text{Engineering} & \text{True} \\
\text{Stress} & \text{Stress} & \text{Stress} & \text{Stress} & \text{Stress} & \text{Stress} \\
(\text{ksi}) & (\text{ksi}) & (\text{ksi}) & (\text{ksi}) & (\text{ksi}) \\
0.0 & 0.00000 & 0.0 & 0.00000 & 0.0 & 0.00000 \\
54.0 & 0.00186 & 54.1 & 0.00000 & 55.0 & 0.00190 & 55.1 & 0.00190 \\
54.0 & 0.01407 & 54.8 & 0.01208 & 60.5 & 0.05000 & 63.5 & 0.04879 \\
62.7 & 0.02575 & 64.4 & 0.02320 & 65.7 & 0.10000 & 72.3 & 0.09531 \\
66.3 & 0.03481 & 68.6 & 0.03185 & 67.0 & 0.15000 & 77.1 & 0.13976 \\
68.4 & 0.04982 & 71.8 & 0.04614 & 67.4 & 0.20000 & 80.9 & 0.18232 \\
69.3 & 0.07962 & 74.8 & 0.07403 & 66.5 & 0.25000 & 83.1 & 0.22314 \\
69.6 & 0.11970 & 77.9 & 0.11037 & 61.7 & 0.30000 & 80.2 & 0.26236 \\
68.9 & 0.15970 & 79.9 & 0.14540 & 44.3 & 0.35000 & 59.8 & 0.30010 \\
66.2 & 0.19980 & 79.4 & 0.17942 & \hline
56.2 & 0.23190 & 69.2 & 0.20617
\end{array}
\]

Table 4.1: A36 Expected and A992 Coupon True Stress-Strain data
Table 4.2: Material data for A325 bolts

<table>
<thead>
<tr>
<th></th>
<th>Engineering</th>
<th></th>
<th>True</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress</td>
<td>(ksi)</td>
<td>Strain</td>
<td>(ksi)</td>
</tr>
<tr>
<td></td>
<td>(ksi)</td>
<td></td>
<td>(e)</td>
</tr>
<tr>
<td>92.00</td>
<td>0.00317</td>
<td>92.29</td>
<td>0.00317</td>
</tr>
<tr>
<td>120.00</td>
<td>0.13683</td>
<td>136.42</td>
<td>0.12824</td>
</tr>
</tbody>
</table>

4.5 Elements

A wide range of elements are available in Abaqus empowering the user to solve a variety of problems. The number of nodes and the integration type were defined by according to Ruffley’s recommendations. All elements used were 3D solid elements with reduced integration. There are five aspects of an element influencing its behavior. They are:

4.5.1 Family:

The element families that are most commonly used in stress analysis are continuum (brick), shell, beam, rigid, truss etc. For this study, Continuum (solid) elements were used throughout the models.

![Commonly Used Element Families](Abaqus 6.10)

Figure 4.2: Commonly Used Element Families (Abaqus 6.10)
4.5.2 **Degrees of Freedom:**

Each node in a solid element has three translational degrees of freedom and three rotational degrees of freedom.

4.5.3 **Number of Nodes:**

Ruffley recommended to use 8-node and 20 node solid elements. For this study two types of 3-D solid elements 8 node (C3D8R) and 20 node (C3D20R), were used.

![Linear Element](image1) ![Quadratic Element](image2)

(8-node brick, C3D8R) (20-node brick, C3D20R)

*Figure 4.3: Linear Brick and Quadratic Brick Elements*

4.5.4 **Formulation:**

An element’s formulation refers to the mathematical theory used to define the element’s behavior. In the absence of adaptive meshing, all of the stress/displacement elements in Abaqus are based on the Lagrangian or the material description of behavior: the material associated with an element remains associated with the element throughout the analysis, and the material cannot flow across element boundaries.

4.5.5 **Integration:**

Quadratic reduced-integration elements are not susceptible to locking, even when subjected to complicated states of stress. Therefore, these elements are generally the best choice
for most general stress/displacement simulations, except in large-displacement simulations involving very large strains and in some types of contact analyses.

Ruffley recommended the use of C3D20R elements in the regions where significant inelastic deformation was expected. 20 node elements were used in double angles as they were assumed to undergo a significant plastic deformation. Element type sensitivity analyses done by Ruffley showed that C3D8R elements with reduced integration were equally accurate. For bolts, beams and stiffener C3D8R elements were used.

### 4.6 Partitioning and Meshing

Since, actual mesh size affects the failure load, appropriate mesh sizes were needed for modelling. In this study, they will be assigned according to the area of importance. Meshing and partitioning of the elements of the connection such as the double angles, beam, girder and bolts were done according to the recommendations from the past research done at University of Cincinnati by Ruffley (2011).

![Partition Details for W16x45](image)

*Figure 4.4: Partition Details for W16x45*

![Mesh Details for W16x45](image)

*Figure 4.5: Mesh Details for W16x45*
Methodology

4.7 Step:

An analysis in Abaqus is performed by dividing the problem history into steps. For each event of analysis, a step is defined. To carry out the simulation, two analysis steps were identified. Therefore, the overall analysis consisted of three steps.

4.7.1 An Initial Step-

It is the default time step, in which the boundary conditions that constrain the ends of the beam and girder elements and, interactions were defined.

4.7.2 Pretension Step-

The pretension step is the step after the first initial step. In this step the temperature change required to produce the prescribed pretension force in the bolts was defined.

Figure 4.6: Partition Details for W21x73

Figure 4.7: Mesh Details for W21x73
4.7.3 Loading Step-

The pretension step was followed by the loading step where the loading on the components was defined.

4.7.4 Step Properties:

Time period of each step was one. Step parameters used for finite element models are explained in Table 4.3.

<table>
<thead>
<tr>
<th></th>
<th>Pretension</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum number of increments</td>
<td>100</td>
<td>10000</td>
</tr>
<tr>
<td>Initial increment size</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>Minimum increment size</td>
<td>1.00E-05</td>
<td>1.00E-05</td>
</tr>
<tr>
<td>Maximum increment size</td>
<td>0.1</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Non-linear geometric effects were enabled in each step to consider local instability and large deformation effects. Un-symmetric matrix storage and solution scheme was used with other default solution technique and load variation. It significantly improved the convergence history.

4.8 Interactions:

This module is used to assign the behavior between the instances of a part. Once the surfaces of the instance have been created, it is imperative to specify which pairs of surfaces can interact with each other during the analysis. These contact pairs can either be assigned manually or can be automatically found using “find contact pairs”.

An interaction property must be assigned after these pairs are defined. For these models, normal and tangential behaviors of instances were considered. The normal behavior imposes a simple hard contact, which prevents surfaces from penetrating into each other. A penalty friction
Methodology

formulation was used with a friction coefficient of 0.3 in the tangential interaction. Tangential behavior refers to the slip or friction between two surfaces. Ruffley recommended the use of design coefficient or friction coefficient of friction while using finite element results to predict behavior.

4.9 **Constraints:**

Constraints partially or fully eliminate degrees of freedom of a group of nodes and couple their motion to the motion of a master nodes. Different types of constraints are available. These include tie, rigid body, display body, coupling, MPC constraint, etc. For modeling purpose of this study, tie constraint and kinematic coupling constraints were used.

Surface based ties were used to replicate fillet welds to connect stiffener to the girder web. Tie constraints keep the slave nodes locked in position relative to the master node. In a tie constraint, the movement of the nodes on the slave surface follows the movement of the nodes on master surface.

Kinematic coupling constraints were used to define girder boundary conditions. In kinematic coupling constraints a surface of slave nodes were typically tied to a single master node reference points which were typically placed at the center of the surface. Kinematic coupling constraint keeps plane section plane.

4.10 **Loading:**

To simulate the uniformly distributed load (UDL) over the beam, a load was created as surface traction and applied over the surface of the top flange of the beam being supported.

4.11 **Boundary Conditions**

In this section boundary conditions are discussed. Half of the assembly from the floor plan was modelled.
4.11.1 **Beam:**

Half the length of beam was modeled and a symmetry boundary condition was imposed to reduce computational time. For the beam, the symmetry plane is the Y-Z plane. For surfaces, at the end of beam, the symmetry boundary conditions were set to zero displacement in the X-axis. The beam is bolted to the double angles on one end and symmetry boundary conditions are applied at the other end.

4.11.2 **Girder:**

Symmetry boundary conditions were also imposed on girder ends. Only one third of the girder was modeled with bolt holes to connect the double angles to the girder web. This boundary condition was modeled by creating a reference point to which all the degrees of freedom on the surface of the girder cross section kinematically constrained through kinematic coupling constrain option provided by Abaqus.

The girder end was restrained in the Y and the Z-direction but was kept free to rotate. The beam and girder were restrained against out of plane translation.

4.11.3 **Bolts:**

All the bolt heads were restrained in the X, Y and, Z direction. This boundary condition was applied in the initial and the pretension step. However, at the loading step these restraints were de-activated so that the bolts could deform as the function of loads applied. The details of the boundary conditions are explained in following Figure 4.8.
4.12 Bolt Pre-tensioning:

The pre-tensioning of bolts is described in detail by Ruffley (Ruffley, 2011). This method was followed throughout for pre-tensioning. For this study, pre-tensioning was done in Abaqus by applying temperature gradient to the bolt shank. Bolts were pre-tensioned to a value slightly over that specified in the AISC specifications. Equation 4.5 represents the temperature gradient required for minimum pre-tension force in the bolts.

\[ \Delta T = \frac{P_b}{A_b E \alpha} \]

\(\Delta T =\) Temperature gradient

\(E =\) Young’s modulus of Elasticity for steel, 29,000 ksi

\(\alpha = 6.6 \times 10^{-6}\) strain per degree Fahrenheit

\(A_b =\) nominal area of bolt
Methodology

A negative value of the temperature gradient output from the above equation is used in Abaqus to shorten the bolt length until minimum pre-tension load in the bolts is reached.

4.13 Verification Model

Before starting the finite element analysis, it was necessary to calibrate the modeling strategy used with Abaqus. The primary objective of this verification was to check whether the Abaqus could imitate the behavior and failure modes of the connection. For this, the experimental work done by Sherman and Ghorbanpoor (2002) on the “Design of Extended Shear Tabs” at University of Wisconsin-Milwaukee was selected. One of the full scale test was modelled in Abaqus and the results were compared with the load-displacement curves. Details of the connection are provided in following Figure 4.9 and Table 4.4.

Figure 4.9: Setup for Beam and Support Member (Sherman et al 2002)

Table 4.4: Connection Details (Sherman et al 2002)

<table>
<thead>
<tr>
<th>Support Member</th>
<th>Secondary Beam</th>
<th>Tab Thk. t (in.)</th>
<th>Weld Size (in.)</th>
<th>Web h/tw</th>
<th># of bolts</th>
<th>tab Length (in.)</th>
<th>Weld Bolt Distance (in.)</th>
<th>Bracing</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14x53</td>
<td>W12x87</td>
<td>3/8</td>
<td>5/16</td>
<td>30.8</td>
<td>3</td>
<td>9</td>
<td>6.85</td>
<td>NO</td>
</tr>
</tbody>
</table>
Methodology

Parts were modelled in Abaqus according to their actual measurements used in the experiment. C3D8R element type was used for all the parts except for shear tab. For shear tab C3D20R element type was used. Seed size for the parts were assigned as discussed before. Seed size of 0.3 was assigned for the region which experienced a large inelastic deformations such as the region near the connection. For the regions away from the connection, seed size was increased to 1, 2 and 5 based on the distance from the connection since its size didn’t have any effect on the ultimate capacity of the connection.

4.13.1 Material Properties

Sherman and Ghorbanpoor provided the actual tension coupon test data for the shear tabs and support member webs. These material properties were used in the model. Shear tab was ASTM Grade 36 steel. All support members and beam were ASTM A572 grade 50 steel. The modulus of elasticity of steel was assumed to be 29,000 ksi. True plastic stress and strains value of the members were used as requisite in Abaqus which were obtained from the actual engineering stress and strain values. These properties are tabulated in following table.

Table 4.5: Material Properties for A36 and A572-50 (Sherman et.al 2002)

<table>
<thead>
<tr>
<th></th>
<th>A36</th>
<th>A572-50</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Engineering</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Stress</td>
<td>Strain</td>
</tr>
<tr>
<td></td>
<td>(ksi)</td>
<td>(e)</td>
</tr>
<tr>
<td></td>
<td>42.6</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>66.5</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>54.2</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>70.8</td>
<td>0.38</td>
</tr>
</tbody>
</table>

4.13.2 Assembly

All bolt holes were short slotted with snug tight bolts. All bolts used were A325-X 3/4 inches diameter. Bolts were pre-tensioned as per the procedure explained in the previous sections. Tab had a 3-inches pitch and 1-1/2 inches edge distance. All girders were 10-feet long. All these
parts were assembled together to simulate the experiment. Tie constraint was used for welded connection between tab and the supporting web. Contact surfaces were defined and found out by using the function in Abaqus “find contact surfaces”. Load was applied as a concentrated load at a point on a supported beam as it was applied in the experiment. Similar boundary conditions were applied to imitate the exact experiment conditions.

![Diagram of un-stiffened extended shear tab assembly](image)

*Figure 4.10: Un-stiffened Extended Shear Tab Assembly*

### 4.13.3 Results

The connections were expected to fail primarily due to bolt shear as per AISC equations. However, the test with W14x53 girder, showed the bolt bearing as the primary mode of failure followed by bolt shear and shear rupture at a higher load than the AISC critical limit loads.

#### 4.13.3.1 Shear Force vs Vertical Displacement:

Figure 4.11 shows the relationship between shear force and vertical displacement of the connection along its bolt line. It’s clearly seen in the results that Abaqus can closely match the experimental response of the connection.
**4.13.3.2 Failure Mode:**

The FE model successfully predicted the bolt bearing failure mode and the results are in good agreement with those from the experimental investigations.

**4.13.4 Conclusion:**

Table 4.6 compares the capacities and failure modes that were observed in the experiment and Abaqus model. Abaqus predicted the capacities within 10% of the experimental values. Also, Abaqus was able to capture the modes of failure accurately as those observed in the experiment.
This reinforces the rationale behind using non-linear finite element modeling package instead of actual experimental testing.

Table 4.6: Comparison of Experimental capacity and FEA capacity

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Measured Capacity, kips</th>
<th>FEA capacity, kips</th>
<th>Measured failure mode</th>
<th>FEA failure mode*</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-U</td>
<td>58.7</td>
<td>53.87</td>
<td>BB</td>
<td>BB, CWM</td>
<td>8.58</td>
</tr>
</tbody>
</table>

**BB: Bolt bearing, CWM: Column Web Mechanism**
Chapter 5: Finite Element models

3-D FE models of the designed connections were constructed following the procedure defined in Chapter 4. A view of the assembly is shown in following Figure 5.1. For all the models, the beam span is 10’ and the girder span is 6.67’.

Figure 5.1: Assembly Details (Connections With and Without Stiffener)
Boundary conditions were applied to the beam and girder as explained in Section 4.11. Loading was applied at the top flange of the beam as surface traction. For connections with a stiffener at the girder web, the welds between the stiffener and the web girder and flanges were modelled using tied constraints. The material properties, element types, partitioning etc. for these models were discussed in Sections 4.4, 4.5, and 4.6 respectively. For this study, static general FE analysis was performed in Abaqus/ Standard.

5.1 Data Extraction:

Results were extracted using Abaqus/CAE post-processor. Stresses and strains were visually monitored to decide whether a part had yielded or failed. Reaction forces at the connection were measured using a free body cut. To measure the vertical displacement of the connection due to loading, a deflection response of the centroid of the connection was measured. This deflection response was plotted against the measured reaction forces and a plot of shear force vs displacement at a connection was. All these plots are summarized in Appendix B.

![Illustration](image_url)
Reaction forces due to stresses in the elements were measured and compared with the nominal strengths which were calculated using the AISC equations. To check the response of the girder web, elements and nodes were selected at the contact surface between angles and the girder web. This region is shown in the Figure 5.3.

*Figure 5.3: Element and Node Sets to Calculate Contact Stress Reaction Force between Angles and Girder Web*
Chapter 6: Results and Discussion

In this chapter the failure modes observed in the finite element models are discussed. Analyses of all models converged. The methodology used by Kennedy (Kennedy, 2014) to identify the failure in the finite element models was referenced in this study. The materials were defined in Abaqus in such a way that once the value of the strains exceeded the defined value of fracture strain, the stress is maintained instead of dropping to zero. The moment of failure was defined at this instant when the strain exceeded the value of the material’s fracture strain. (Kennedy, 2014) Equivalent plastic strain (PPEQ) was used to identify the failure. Three grades of steel material were used in this study viz. A36 for angles, A992 for I sections and A325 for bolts. For these grades of steel, the values of ultimate plastic strain, corresponding to the engineering ultimate stress, were determined. The values were 0.1795, 0.1454, and 0.12354 for A992, A36, and A325 respectively. If the value of the equivalent plastic strains at any element in the section exceeded beyond the values mentioned above, it was considered as a failure in the section. Observed failure modes were compared with the expected failures estimated as per the AISC design equations. All these failure modes are summarized in the Appendix B.

6.1 Identification of Limit States:

A number of limit states governed the capacity and behavior of the extended double angle connections. The most important limit states were tearing of the angles, tension bolts fracture, bolt bearing on the beam web, and bolt shear.

6.1.1 Tearing of Angles:

Angle tear was predicted at the first row of bolt holes of the angle leg which was connected to the girder. This failure in the angles can be seen in the Figures 6.1, 6.2, 6.3 and 6.4. One of the nodes near the first row of the tension bolt holes of the cross section of the angles reached the
Materials fracture strain. Inelastic strains near to yield strain were also visible in the first row of tension bolts. Typically, tearing failure near the bolt holes occurred first followed by tensile rupture failure of the outstanding angle legs.

Tearing of angles was the controlling limit state in most of the connections with angle thickness of 7/16”. As angle thickness increases, inelastic strain in the angles diminishes.

Figure 6.1: Inelastic Strains in Double Angles 7/16” Thick

Figure 6.2: Inelastic Strains in Double Angles 9/16” Thick
6.1.2 Tension Bolt Fracture:

Tension bolts are those which connect angles to the girder web. Connections with angle thicknesses greater than 7/16” (9/16”, 3/4", and 1”) i.e. way much larger than design requirement have tension bolt fracture as the governing limit state. In all other connections, tension bolt fracture is either second or third mode of failure. Only first row of tension bolts had reached the definition of failure. The cross section of first row of tension bolts saw a higher demand of inelastic strain than the following three rows of bolts. At fracture, some elements of the bolt shank have PEEQ at or exceeding the upper limit of the scale, which implied that element’s stress was beyond ultimate.

Figure 6.3: Inelastic Strains in Double Angles 3/4” Thick

Figure 6.4: Inelastic Strains in Double Angles 1” Thick
stress. With such large strain demands, it was believed that the tension bolts had failed and responsible for the loss of the strength of the connection.

6.1.3 Bolt Bearing:

The bolt bearing failure mode was identified by examining the equivalent plastic strains around the bolt holes. The failure was noted when a node or a set of nodes making up a cross section around the hole had an equivalent plastic strain (PEEQ) which exceeded the materials plastic strain. The cross section of the bolt, not shown in the following figure, was found to have partially yielded; however the level of extension of yielding in the beam web holes was more extensive. Following Figure 6.6 shows the bearing at the beam web.

![Figure 6.5: 3/4" Dia. Tension Bolts Contours show Von Mises stresses and Inelastic Strain](image)

![Figure 6.6: Bearing at the W16x45 Beam Web](image)
The girder web and the angles yielded due to the bearing on the tension bolts. The girder web particularly appeared to have inelastic deformations near the first row of tension bolts. Figure 6.7 shows the equivalent plastic strain in girder.

![Image of equivalent plastic strain](image)

**Figure 6.7: Inelastic Strains in W21x93 Girder**

### 6.1.4 Bolt Shear Capacity:

Figure 6.8 shows PEEQ for tension bolts and shear bolts. Shear bolts connect extended double angles to the supported beam. Shear bolts had not yet reached $F_a$ at any point. Inelastic strain in the shear bolts appeared primarily due to the bending of the bolts. Shear bolts are the only parts which transfer the applied load from the beam web to the extended double angles. Additional inelastic strain was visible in the beam web at the bolt holes due to the bearing which caused inelastic strain to develop in the shear plane of the bolt, though inelastic strain was not significant through the cross section of the bolt.
Results and Discussion

A free body cut past the shear bolts was taken to calculate the force transferred by shear bolts in the double angles. In the Figure 6.9, the free body cut past the shear bolts shows a force of 87.12 kips which was 91% of the bolt shear design capacity. Some of the force was likely to transfer to the angle by friction from the beam web. Since, inelastic deformations in the shear plane of the bolts were small and bearing on the beam, and angle was causing the material to strain hardening, there was reserve capacity in the shear fasteners.
6.2 Strength of the Girder Web:

Strength of the girder web was determined by computing the resultant reaction force due to contact stresses between angles and the web of the girder. The elements and nodes at the web were selected as discussed in Section 5.1 to calculate the resultant reaction force. Figure 6.10 shows the resultant reaction force and the moment at the girder web. Resultant with a double arrow head represents the moment and the one with the single arrow head is the resultant reaction force. Moment is measured in kip-inch and the resultant reaction force in kips. These resultants are summarized for each connection in Appendix B. In all the connections, reaction force which was transferred to the girder web through contact between angles and tension bolts was well within the limits of the design shear capacity of the girder. Also, the inelastic strains at the girder web were just above the yield strains as can be seen in Figure 6.7. This implies that there was a sufficient capacity in the girder web.

Figure 6.10: Free Body Cut at the Contact between Angles and Girder Web Including Tension Bolts: Resultant Reaction Force (Girder-W21x93; Angle Thickness 9/16")
6.3 Effects of Varying Section Properties on Behavior, Ultimate Connection Strength, and Girder Web Strength:

In Appendix B, the curves of shear force vs vertical displacement along the bolt line and, the resultant reaction forces transferred to the girder web are summarized.

6.3.1 Effects Due to Varying Angle Thicknesses:

The angle thickness, $t_a$, is considered to be one of the most important angle dimensions. It was chosen to be varied from 7/16 inch to 1 inch. Changes in the angle thickness clearly marked the influence on the behavior, ultimate strength and controlling limit state of the connection. Increasing the angle thickness caused an increase in the ultimate strength of the connection. It can be clearly seen from the curves of the shear force vs vertical displacement in Figures B.1, B.2, B.3, and B.4 for varying angle thicknesses and three different bolt diameters in Appendix B. Also, the reaction force carried by the girder web increased as the angle thickness was increased. However, this reaction force was well within the shear capacity of the girder web. Inelastic strains at the girder web were just past the yield strains. This implies that there was a sufficient capacity in the girder web.

There was no significant increase in the ultimate strength of the connection when a stiffener was provided at one side of the girder web. But it significantly increased the resultant load carried by the girder web. It was important to note that in the FE models, as the angle thickness was increased from designed 7/16 inch to larger thicknesses, there was a change in the primary mode of failure of the connections for all diameters. For finite element models, with 7/16 inch angle thickness, the primary mode of failure was tearing of the angle near the top row of bolt holes in the leg connected to the girder web. For all the other angle thicknesses the primary mode of failure was the tension bolt shear. Only first row of the tension bolts reached the definition of failure. Maximum plastic strains in the other rows of the tension bolts and shear bolts connecting the beam
to angles were in the strain hardening range. Similar behavior was observed in the connections with the stiffener at the girder web.

6.3.2 Effects Due to Varying Girder Web Thickness:

It was chosen to increase the girder web slenderness ratio from 32.3 to 53.6. For all the girder web thicknesses, the girder depth was chosen to be 21 inches and the supported beam W16x45 was kept constant. There was no appreciable gain or decrease in the connection strength as the slenderness ratio of the girder web increased for all the bolt diameters. However, as the diameter of the bolt was increased, the ultimate strength of the connection increased by a small margin.

Including the stiffener at the girder web had no significant effect on the ultimate strength of the connection. But it caused an increase in the load carried by the girder web than the connections without the stiffener. However, this increase in load was not proportional to the web slenderness ratio. Also, this reaction force is well within the shear capacity of the girder web.

All finite element models with varying web slenderness ratio lost their connection strength primarily due to tearing of the angles near top row of bolt holes in the leg connecting to girder web, with a single exception in a connection with a girder of size W21x48 with bolt diameters 3/4 inch and no stiffener. The primary failure mode in this connection was bolt bearing at the supported beam web.

6.3.3 Effects Due to Varying Girder Depth:

Extended double angle connections were designed with three different girder depths of 24 inch, 21 inch and 18 inch. The ultimate connection strength decreased as the girder depth decreased from 24 inch to 18 inch for the connections with bolt diameters 3/4 inch with and without stiffener. However, this decrease in strength was not significant. There was no significant increase or
decrease in the ultimate connection capacity for the connections with bolt diameters 7/8 inch and 1 inch. Similar behavior was observed in the connections with stiffener at the girder web.

There was no appreciable change in load carried by the girder web in connections without stiffener. Including stiffener at the girder web increased the load carried by the girder web but not significantly in all connections designed with all three different diameters. But this change in load carried by the girder was not proportional to the change in girder depths. Also, inelastic strains at the girder web were just past the yield strains. This implies that there was a sufficient capacity in the girder web.

All finite element models with varying girder depths with and without stiffener lost their connection strength in a similar way as that with the connections with varying girder web thicknesses. Tearing of the angle near top row of bolt holes in the leg connecting to girder web was the primary mode of failure in all FE models with varying girder depths.

### 6.3.4 Effects Due to Varying Beam Depths:

There was a slight increase connection strength as the beam depth was varied from 16 inch to 21 inch with bolt diameters 7/8 inch and 1 inch. However, this increase in strength was attributed more to the increase in diameter of the bolts. For the connections with 3/4 inch bolt diameter the ultimate connection strength was decreased as the depth increased. This same behavior was observed in both types of connections with and without stiffener.

There was no appreciable increase or decrease in the load carried by the girder web as the beam depth was increased. But inclusion of stiffener at the girder web significantly increased the load carried by the girder web in all diameters. This change in load carried by girder web was increased with small margin as the beam depth increased for the bolt diameter of 1 inch.
Results and Discussion

With the exceptions of three finite element models with bolt diameter of 3/4 inch failed primarily due to tearing of the angle near top row of bolt holes in the leg connecting to girder web. Similar trend was observed in the connections with stiffener at the girder web. All these results are summarized in Appendix B.
Conclusions and Recommendations

Chapter 7: Conclusions and Recommendations

The major objective of this study was to investigate the strength of the web of the girder in all-bolted extended double angles in a beam-to-girder connection. Non-linear FE models were developed and studied to achieve this objective. This chapter summarizes the results and findings from the thesis. Section 7.1 discusses the conclusions. Section 7.2 gives thoughts to be considered for future research.

7.1 Conclusions:

The following conclusions can be drawn from this study:

The design steps proposed by Higgins et. al (2005) were considered an appropriate starting point for all-bolted extended double angle connections. AISC 14th Ed. (2011) design equations were followed to calculate the girder web strength. The finite element analysis indicates that the AISC 14th Ed. (2011) design equations conservatively predict the ultimate strength of the double angle extended configuration. However, they can still be used to design extended double angle connection.

Finite element analysis showed that the girder web had enough strength to account for the effects due to eccentric load. Plasticity, at the girder web was not observed. This was in contrast to the primary assumption made in the past study by Higgins et.al (2005). The author had assumed that the plastic hinge was formed at the girder web, and hence the prying action could be ignored (Higgins et.al 2005). It was justified by Kishi and Chen (1990) by the power model relationship between moment in the connection and end rotation. This model was verified on beam-to-column connection and was subsequently adapted for the beam-to-girder connection. A suitable strength and stiffness model is required to predict the behavior of extended double angles in a beam-to-girder connection.
Conclusions and Recommendations

Finite element models for the connections designed with 7/16” angle thickness lost their strength initially by tearing of the double angles near the first row of tension bolts and then later by either the failure due to bolt bearing at the beam web or failure of the first row tension bolt shear. For FE models with an angle thickness larger than 7/16”, this sequence of failure changed to tension bolt fracture first and then was followed by either the tearing of the double angles near the top row of tension bolt holes or failure due to bolt bearing at the beam web. This was in contradiction to the results obtained by using AISC design equations where bolt bearing at the beam web was the primary mode of failure and was followed by the failure due to bolt shear.

For all the other FE models which were designed with varying girder web thicknesses, girder depth and beam depth the primary mode of failure was tearing of the double angles near the first row of tension bolts and later, the connections lost strength due to either the tension bolt fracture or the flexural yielding of the double angles at the bottom of outstanding leg. Under these circumstances also the AISC design equations had predicted a different sequence of failure.

Even adding a stiffener at the girder web failed to offer reconciliation as the connection strength did not increase significantly. Nevertheless, it helped to increase the capacity of the girder which was evident by the small amount of inelastic strains observed in the girder web at the connection. It also reduced the deformation due to the pull from double angles. Thus, it is recommended to provide stiffener at the girder web to prevent the possible failure of the girder web due to excessive deformation.

It is evident from the finite element study that the all-bolted extended double angles in a beam-to-girder connection can be a possible alternative to a standard connection where coping of the supported beam is required.
7.2 Future Research

All of the past experimental research in extended connections was done primarily in extended plate or extended shear tab connection. This study provided the platform for the experimental work for extended double angles in a beam-to-girder connection.

The finite element models differed for the mode of failures than those predicted from AISC equations. This can further be verified by testing the extended double angle connections in combination with finite element models in Abaqus/Explicit considering the inertial effects and the failure criteria for steel.

This thesis covered connections which were only on one side of the girder. In future, similar research can be performed for an interior girder where there is one connection on either side of the girder web.

A suitable strength and stiffness model to predict the behavior of the extended double angle connections in a beam-to-girder connection can be established based on the experimental results.

In the current AISC Steel Construction Manual, 14th Ed. (AISC, 2011), the largest hot-rolled outstanding leg of the angle available is 8 inch. Now, the largest available outstanding angle leg length is 12 inch. (Zuo, 2012). This increases the possibility of using wider shapes of the girder and larger connected beams with multiple columns of bolts. With these, the all-bolted double angle extended connections can be widely used in a beam-to-girder connection.
References:


Appendix A: Extended Connection Design

Design all bolted extended double angle connection for W16x45 beam connected to W21x93 having a factored shear reaction of 30.2 kips. The connection is to be of 3/4 in. diameter A325 bolts in a bearing-type connection with threads excluded from the shear plane. Use A36 steel for angles and A992 steel for beam. Use expected material strength data for A36 and tension coupon test data for A992 from Section 4.4.

Solution:

<table>
<thead>
<tr>
<th>W16x45</th>
<th>W21x93</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_w = 0.345$ in.</td>
<td>$t_w = 0.580$ in.</td>
</tr>
<tr>
<td>$b_f = 7.04$ in.</td>
<td>$b_f = 8.42$ in.</td>
</tr>
<tr>
<td>$d = 16.1$ in.</td>
<td>$d = 21.6$ in.</td>
</tr>
<tr>
<td>$t_{fb} = 0.565$ in.</td>
<td>$t_{fb} = 0.93$ in.</td>
</tr>
</tbody>
</table>

For A992,

- $F_y = 55$ ksi
- $F_u = 67.4$ ksi

A325-X 3/4 in. diameter bolts; End shear = 30.2 kips; Eccentricity = 6.75 in.

Figure A.1: Schematic Representation of Connection
Appendix A: Extended Connection Design

a) **Necessary Outstanding Leg Length:**

Following Equation A.1 is used to calculate the necessary outstanding leg length of the angles. The value of 3.5 was chosen in such a way that the outstanding angle leg will clear the flange width of the supporting girder. (Higgins et al 2005)

\[
e_a = \frac{b_f}{2} + 3.5''
\]

\[
e_a = \frac{8.42''}{2} + 3.5'' = 7.71''
\]

Take, \(e_a = 8''\)

b) **Bearing Strength of One Bolt on the Beam Web:**

Bearing strength of the connection is influenced by the spacing between bolts and the edge distance. The edge distance in the angles is assumed to be 1-1/4 inch and the bolt spacing is 3 inch. Edge distance in the beam web is \(2 \frac{5}{16}\) inch. This spacing between the bolts and edge distance are the minimum for connections utilizing 3/4 inch diameter to 1 inch diameter bolts.

A325-X 3/4 in. diameter bolts; \(S \geq 3.0d_b\) and \(L_e = 2.0d_b\)

![Figure A.2: Bolt Bearing on Beam Web](image)
According to AISC Specification J3-6a, \( \phi = 0.75 \)

\[
\phi R_n = \phi 1.2 \ell_c t_w F_u \leq \phi 2.4 d t_w F_u
\]

\[
\phi R_n = 0.75 \times 1.2 \times \left[ 2.3125'' - \frac{0.8125''}{2} \right] \times 0.345'' \times (67.4^{ksi})
\]

\[ \phi R_n = 39.89 \text{ kips/bolt} \]

and \( \phi R_n = \phi 2.4 d t_w F_u \)

\[ = 0.75 \times 2.4 \times 0.75'' \times 0.345'' \times (67.4^{ksi}) \]

\[ \phi R_n = 31.4 \text{ kips/bolt} \]

Therefore, \( \phi R_n = 31.4 \text{ kips/bolt} \) Controls

For double shear, Nominal Shear Strength in Bearing-Type Connections,

\[ F_{n\nu} = 68 \text{ ksi} \quad \text{AISC Spec. Table J3.2} \]

\[ \phi R_n = \phi F_{n\nu} m A_b \]

\[ \phi R_n = 0.75 \times 68 \times 2 \times (0.4418^{in^2}) \]

\[ \phi R_n = 45.06 \text{ kips/bolt} \]

Therefore, bearing controls over shear.

Number of bolts required, for a single row of bolts, following Equation from Salmon C.G. et al. (2008) Equation 4.12.29 is used,

\[
\sqrt[6]{\frac{6 M_u}{\phi R_n P}}
\]

\[ n = \sqrt[6]{\frac{6 \times (30.2^{kips}) \times (6.75'')}{(31.4^{kips/bolt}) \times 3''}} \]

\[ n = 3.61 \]

Take \( n = 4 \) number of bolts.

All bolt limit states are treated as eccentrically loaded. Eccentricity produces a rotation and a translation of one connection element with respect to the other. The combined effect of rotation and translation is equivalent to a rotation about a point defined as the instantaneous
center of rotation (ICR). To determine the effects due to eccentricity on bolt bearing capacity of the bolt group instantaneous center of rotation (ICR) method is used.

By AISC *Steel Construction Manual, 14th Ed.* (2011) Chapter 7, Table 7-6, for 4 bolts in one vertical row with spacing 3 in. and eccentricity \( e = 6.75 \) in.

ICR coefficient is \( C = 1.57 \);

Strength of bolt group,

\[
\phi r_n = C \times \phi R_n \\
= 1.57 \times (31.4 \text{kips/bolt}) = 49.3 \text{kips} > 30.2 \text{kips} \quad \text{(OK)}
\]

Without resistance factor,

\( R_n = 65.74 \) kips

c) **Angle Thickness:**

According to AISC J3-6a, bearing will not control angles thickness unless end distance is \( L_e < 1.5d_b \) or bolt spacing \( S < 3d_b \) because \( 2.4F_u \) was used to compute bearing strength. End distance provided \( L_e = 1.25 \) in. > 1.5 \( d_b = 1.13 \) in.; Bolt spacing provided \( S = 3 \) in. > 3.0 \( d_b \)

Since \( 3.0 \ d_b = 3 \times 3/4" = 2.25" < S \)

Therefore, the thickness of the angle might be controlled by either their shear rupture or shear yield strength in accordance with AISC Specification J 4.2,

\[
\left[ \phi (0.6F_y) A_{gy}, \text{ and } \phi (0.6F_y) A_{ny} \right] \geq P_u
\]

For the angles connected to the W16x45,

\[
\left[ \phi (0.6F_y) A_{gy}, \text{ and } \phi (0.6F_y) A_{ny} \right] \geq P_u
\]

\[
\phi (0.6F_y) A_{gy} \geq P_u
\]

\[
1 \times 0.6 \times (54^\text{ksi}) \times 2t \times 11.5" \geq 30.2 \text{kips}
\]
d) **Check for Block Shear Rupture:**

In block shear rupture, the failure plane undergoes both shear and tension. In standard connection, the connection element and the net coped section are susceptible to block shear rupture. Since, coping is not required for supported beam in extended connections, block shear rupture will not occur. (Driver et al 2004) The angles are the only element that needs to be checked for block shear rupture in an extended connections. Here, check block shear rupture limit state for supported beam is also carried out.

By AISC specification J4.5, \( R_n = (0.6F_{uA_{nv}} + F_u U_{bs} A_{nt}) \leq (0.6F_{yA_{gy}} + F_u U_{bs} A_{nt}) \)
Appendix A: Extended Connection Design

Figure A.3: Block Shear Rupture of Angle

a) Block Shear Rupture of Angle:

Now,

\[ A_{gv} = \text{Shear yield area} = t_a \times [L_{ev} + (n-1) \times 3] \]
\[ = \frac{7}{16} \times [1.25" + (4-1) \times 3] \]
\[ A_{gv} = 4.48 \text{ in}^2 \]

\[ A_{nv} = \text{Shear fracture area} = t_a \times \left[ (d - L_{ev}) - (n - 0.5) \left( d_h + \frac{1}{8} \right) \right] \]
\[ = \frac{7}{16} \times \left[ (11.5" - 1.25") - (4 - 0.5) \left( \frac{3}{4} + \frac{1}{8} \right) \right] \]
\[ = \frac{7}{16} \times [10.25" - 3.5(0.875")] \]
\[ = \frac{7}{16} \times (10.25" - 3.0625") \]
\[ A_{nv} = 3.145 \text{ in}^2 \]
Appendix A: Extended Connection Design

\[ A_{nt} = \text{Tension fracture area} = t_a \times \left[ L_{ch} - 0.5 \times \left( d_b + \frac{1}{8} \right) \right] \]
\[ = \frac{7''}{16} \times \left[ 1.25'' - 0.5 \times \left( \frac{3''}{4} + \frac{1''}{8} \right) \right] \]
\[ = \frac{7''}{16} \times [1.25'' - 0.5 \times (0.875'')] \]
\[ A_{nt} = 0.355 \text{ in}^2 \]

By AISC Spec. J4.3, \( U_{bs} = 1.0 \)

Now,
\[ R_n = (0.6F_u A_{nv} + F_u U_{bs} A_{nt}) \times 2 \]
\[ = [0.6 \times (69.6^{\text{ksi}}) \times 3.145^{\text{in}^2} + (69.6^{\text{ksi}}) \times 1 \times (0.355^{\text{in}^2})] \times 2 \]
\[ R_n = 313.2 \text{kips} \]

and
\[ R_n = (0.6F_y A_{gy} + F_u U_{bs} A_{nt}) \times 2 \]
\[ = [0.6 \times (54^{\text{ksi}}) \times (4.48^{\text{in}^2}) + (69.6^{\text{ksi}}) \times 1 \times (0.355^{\text{in}^2})] \times 2 \]
\[ = 339.72 \text{kips} \geq 313.2 \text{kips} \]

Therefore, \( R_n = 313.2 \text{kips} \) controls.

For factored reaction capacity,
\[ \phi R_n = 0.75 \times (313.2^{\text{kips}}) = 234.9 \text{kips} > 30.2 \text{kips} \] (OK)
b) Block Shear Rupture on Beam Web:

![Figure A.4: Block Shear Rupture on Beam Web](image)

The distances between the girder top flange bottom and center of the first bolt hole is assumed to 2.5 inch. To align the top flange of the girder and the filler beam at the same level, location of the first bolt hole in the filler beam is calculated as follows:

\[
X = (2.5" + t_{fg}) - t_{fb}
\]

\[
= (2.5" + 0.93") - 0.565" = 2.865"
\]

Gross Shear Area,

\[
A_{gv} = 0.345" \times (9" + 2.865")
\]

\[
A_{gv} = 4.09 \text{ in}^2
\]

Net Shear Area,

\[
A_{nv} = 0.345" \times \left[ 11.865" - (4 - 0.5) \left( \frac{3"}{4} + \frac{1}{8} \right) \right]
\]

\[
= 0.345" \times [11.865" - (3.5)(0.875")]
\]

\[
= 0.345" \times (8.8025")
\]

\[
A_{nv} = 3.037 \text{ in}^2
\]
Appendix A: Extended Connection Design

Tension Fracture Area,

\[
A_{nt} = Tension \ fracture \ area = t_w \times \left[ L_{ch} - 0.5 \times \left( d_b + \frac{1}{8} \right) \right]
\]

\[
= 0.345'' \times \left[ 2.3125'' - 0.5 \times \left( \frac{3''}{4} + \frac{1}{8} \right) \right]
\]

\[
A_{nt} = 0.6468 \text{ in}^2
\]

Now, \( R_n = (0.6 F_u A_{nv} + F_u U_{bs} A_{nt}) \)

\[
= 0.6 \times (67.4^{\text{ksi}}) \times (3.037^{\text{in}^2}) + (67.4^{\text{ksi}}) \times 1 \times (0.6468^{\text{in}^2})
\]

\[
R_n = 166.41 \text{ kips}
\]

And \( R_n = (0.6 F_y A_{gy} + F_y U_{bs} A_{nt}) \)

\[
= 0.6 \times (55^{\text{ksi}}) \times (4.09^{\text{in}^2}) + (67.4^{\text{ksi}}) \times 1 \times (0.6468^{\text{in}^2})
\]

\[
= 178.56 \text{ kips} \geq 166.41 \text{ kips}
\]

Therefore, \( R_n = 166.41 \text{ kips} \)

For factored reaction capacity,

\[
\phi R_n = 0.75 \times (166.41^{\text{kips}}) = 124.81 \text{ kips} > 30.2 \text{ kips} \quad \text{(OK)}
\]

e) Shear Yielding of Angle:

Shear yielding is a ductile limit state and is calculated from AISC Specification J4.3. The shear area is used to calculate the shear yielding capacity of the connection. Double angles have twice the shear area of a single angle.

According to AISC Specification J4-3,

\[
R_n = 0.6 F_y A_{gy}
\]

\[
= 0.6 \times (54^{\text{ksi}}) \times 0.4375'' \times 11.5'' \times 2
\]

\[
R_n = 326.025 \text{ kips}
\]
Appendix A: Extended Connection Design

Factored capacity, $\phi = 1.0$

$$\phi R_n = 1 \times (326.025^{\text{kips}}) = 326.025 \text{ kips} > 30.2 \text{ kips} \quad \text{(OK)}$$

f) Shear Rupture of Angle:

For shear rupture the failure plane is along the line of the bolts in the supported angle legs. Therefore, a reduced or net area is used to calculate the shear rupture strength of the connection (Higgins et al 2005). According to AISC Specification J4-4,

$$R_n = 0.6 F_u A_{nv}$$

$$A_{nv} = t_a \times \left[ h_a - n \left( d_b + \frac{1}{16} \right) \right]$$

$$R_n = 0.6 \times (69.6^{\text{kpsi}}) \times \frac{7''}{16} \times \left[ 11.5'' - 4 \left( \frac{3''}{4} + \frac{1}{16} \right) \right]$$

$$R_n = 150.76 \text{ kips}$$

Factored capacity, $\phi=0.75$

$$\phi R_n = 0.75 \times (150.76^{\text{kips}}) = 113.07 \text{ kips} > 30.2 \text{ kips} \quad \text{(OK)}$$

g) Flexural Rupture of Angle:

To calculate the flexural rupture capacity of the section, the net elastic section modulus is conservatively used (Higgins et al 2005). The following equation for the net elastic section modulus was derived from AISC Steel Construction Manual, 3rd Ed. (2001).

$$S_{net} = \text{net elastic section modulus}$$

$$S_{net} = \frac{t_a}{6} \times \left[ \frac{s^2 n(n^2 - 1) \left( d_b + \frac{1}{16} + \frac{1}{16} \right)}{h_a} \right]$$

$$S_{net} = \frac{7}{16} \times \left[ \frac{(3'')^2 (4^2 - 1) \left( \frac{3''}{4} + \frac{1}{16} + \frac{1}{16} \right)}{11.5} \right]$$
Appendix A: Extended Connection Design

\[
S_{\text{net}} = 0.07292" \times \left[ (132.25\text{in}^2) - \frac{(540\text{in}^2)(0.875")}{11.5"} \right] \\
= 0.07292" \times \left[ (132.25\text{in}^2) - (41.09\text{in}^2) \right]
\]

\[
S_{\text{net}} = 6.65\text{ in}^3
\]

For double angle, \( S_{\text{net}} = 6.65\text{ in}^3 \times 2 = 13.3\text{ in}^3 \)

Flexural rupture strength, \( M_n = S_{\text{net}} F_u \)

\[
= 13.3\text{ in}^3 \times (69.6 \text{ ksi}) \\
= 925.68 \text{ kips-in}
\]

Eccentricity, \( e = 6.75" \)

Flexural rupture strength,

\[
R_n = \frac{M_n}{e} = \frac{925.68 \text{ kips}}{6.75"} \\
R_n = 137.14 \text{ kips}
\]

Factored rupture strength,

\[
\phi = 0.75 \\
\phi R_n = 0.75 \times 137.14 \text{ kips} = 102.86 \text{ kips} > 30.2 \text{ kips} \quad \text{(OK)}
\]

h) Flexural Yielding of Outstanding Angle:

In extended double angle connections, the outstanding angle leg is checked for flexural yielding limit state. This limit state is not considered for standard connections since, the load eccentricity is within specified gage distance (3 in.). For a conservative approximation, the elastic section modulus is used while estimating the flexural yielding capacity of the connection. (Higgins et al 2005)
Elastic section modulus, 
\[ s = \frac{t_a h^2}{6} = \frac{\left(\frac{7}{16}\right)(11.5'')}^2}{6} = 6.75'' \]
\[ s = 9.64 \text{ in}^3 \]

for double angles, 
\[ s = 2 \times 9.64 \text{ in}^3 = 19.28 \text{ in}^3 \]

Flexural yielding, 
\[ M_n = sF_y \]
\[ = 19.28 \text{ in}^3 \times 54 \text{ksi} \]
\[ = 1041.12 \text{kips-in.} \]

Here, eccentricity, \( e = 6.75'' \)

Therefore, flexural rupture strength, 
\[ R_n = \frac{M_n}{e} = \frac{1041.12 \text{kips}}{6.75''} \]
\[ R_n = 154.24 \text{kips} \]

Factored rupture strength, 
\[ \phi = 0.9 \]
\[ \phi R_n = 0.9 \times 154.24 \text{kips} \]
\[ = 138.82 \text{kips} > 30.2 \text{kips} \]

OK

i) Bolt Shear Check:

Bolt shear limit state can be determined by AISC Specification J3.1. Double angle connections have two shear planes. Following equation determines the bolt shear capacity.

\[ r_n = F_{nv} A_b N_s N_b \]
\[ = (68^{\text{ksi}}) \times (0.4418^{\text{in}^2}) \times 2 \times 4 \]
\[ r_n = 240.34 \text{kips} \]

For one bolt, 
\[ r_n = \frac{240.34 \text{kips}}{4} = 60.1 \text{kips} \]

Considering eccentricity, \( e = 6.75'' \)
Appendix A: Extended Connection Design

For 4 number of bolts, ICR coefficient, \( C = 1.57 \)

\[
R_n = Cr_n = 1.57 \times 60.1^{\text{kips}} \\
= 94.36 \text{kips}
\]

Factored bolt shear capacity,

\[
\phi R_n = 0.75 \times 94.36^{\text{kips}} \\
= 70.77 \text{kips} > 30.2 \text{kips} \quad \text{(OK)}
\]

j) Gross Shear of Beam Web:

Gross shear on beam web can be computed according to AISC Specification J4-3.

Where, \( A_w \) is the gross area of the supported beam web subjected to shear.

\[
R_n = 0.6 \times F_y \times A_w \\
= 0.6 \times (55^{\text{ksi}} \times 16.1'' \times 0.345'' \\
R_n = 183.3 \text{kips}
\]

Factored gross shear capacity of beam web,

\[
\phi = 1.0 \\
\phi R_n = 1.0 \times 183.3^{\text{kips}} \\
= 183.3 \text{kips} > 30.2 \text{kips} \quad \text{(OK)}
\]

k) Shear Yielding, Shear Buckling and Flexural Yielding of Angle:

\[
\left( \frac{V_u}{\phi V_n} \right)^2 + \left( \frac{M_u}{\phi_b M_n} \right)^2 \leq 1.0
\]

Now,

\[
V_u = 30.2 \text{kips} \\
\phi V_n = 326.03 \text{kips} \\
M_u = V_u \times e = 30.2^{\text{kips}} \times 6.75'' \\
= 203.85 \text{kips-in.}
\]
Appendix A: Extended Connection Design

\[ \phi_b = 0.90 \]
\[ \phi_b M_n = \phi_b F_p Z_p \]
\[ Z_p = \frac{t_s l_a^2}{4} = \frac{7''(11.5'')^2}{16} \]
\[ = 14.46 \text{ in}^3 \]
\[ \phi_b M_n = 0.90 \times (54 \text{ ksi}) \times 14.46 \text{ in}^3 \times 2 \]
\[ = 1405.5 \text{ kips-in.} \]

Now,

\[ \left( \frac{30.2 \text{kips}}{326.03 \text{kips}} \right)^2 + \left( \frac{203.85 \text{kips}}{1405.5 \text{kips}} \right)^2 \leq 1.0 \]
\[ 0.0086 + 0.021 = 0.0296 < 1.0 \quad (\text{OK}) \]

**l) Bearing Strength of Bolt on the Girder Web:**

The bolt bearing capacity at the girder web is calculated according to AISC specification J3-6a.

\[ r_n = 2.4 dt_w F_u \]
\[ r_n = 2.4 \times 0.75'' \times 0.580'' \times (67.4 \text{ ksi}) \]
\[ r_n = 70.37 \text{ kips/bolt} \]

For bolt group, \( R_n = (70.37 \text{ kips/bolt}) \times 8 \)
\[ = 562.92 \text{ kips} \]

Factored capacity,

\[ \phi = 0.75 \]
\[ \phi R_n = 0.75 \times 562.96 \text{ kips} \]
\[ = 422.19 \text{ kips} > 30.2 \text{ kips} \quad (\text{OK}) \]

**m) Shear and Tension from Eccentric Loading at Girder Web Bolts:**

For eccentric bearing type connections, the combined shear and tension limit state on tension bolts is computed according to AISC specification J3.7.
Appendix A: Extended Connection Design

Factored tension,

\[ T_u = \frac{6M_u P(d - P)}{d^3} \]

\[ = \frac{6 \times (30.2 \text{kips}) \times (6.75 \text{ in}) \times 3'' \times (11.5'' - 3'')}{2 \times (11.5 \text{ in})^3} \]

\[ T_u = 10.26 \text{ kips/bolt} \]

Shear component per bolt,

\[ V_u = \frac{P_u}{\text{No. of bolts}} = \frac{30.2 \text{kips}}{8} = 3.775 \text{ kips/bolt} \]

Design strength \( \phi R_n \) of bolts in shear and tension, AISC J3.7

In tension, the design strength \( \phi R_{nt} \) is, \( \phi R_{nt} = \phi F_{nt} A_b \)

Where, \( \phi F'_{nt} = \phi \left( 1.3 F_{nt} - \frac{F_{nt} f_v}{\phi F_{nv}} \right) \leq \phi F_{nt} \)

where, \( F_{nv} = 68 \text{ ksi} \quad F_{nt} = 90 \text{ ksi} \quad \phi = 0.75 \quad A_b = 0.4418 \text{ in}^2 \)

\[ \phi F'_{nt} A_b = 87.8 \text{ksi} \times A_b - 1.32 \times V_u \leq 67.5 A_b \]

\[ = (38.8 - 4.983) \text{kips/bolt} \leq 29.83 \text{kips/bolt} \]

\[ = 33.82 \text{kips/bolt} \leq 29.83 \text{kips/bolt} \]

\[ = 29.83 \text{kips/bolt} \geq 10.26 \text{kips/bolt} \]

For shear the maximum nominal shear stress is, \( \phi R_{nv} = \phi F_{nv} A_b \)

Where, \( \phi F'_{nv} = \phi \left( 1.3 F_{nv} - \frac{F_{nv} f_t}{\phi F_{nt}} \right) \leq \phi F_{nv} \)

where, \( F_{nv} = 68 \text{ ksi} \quad F_{nt} = 90 \text{ ksi} \quad \phi = 0.75 \quad A_b = 0.4418 \text{ in}^2 \)

\[ \phi F'_{nv} A_b = 66.3 \text{ksi} \times A_b - 0.76 \times f_t \leq 51 A_b \]

\[ f_t = \frac{T_u}{\sum A_b} \]
Appendix A: Extended Connection Design

Now required total area based on the maximum nominal stress in shear can be given by,

\[
\sum A_b = \frac{\sum V_u + 0.67 \sum T_u}{66.3^{\text{ksi}}} = \frac{30.2^{\text{kips}} + (0.67 \times 8 \times 10.26^{\text{kips/bolt}})}{66.3^{\text{ksi}}}
\]

\[
\sum A_b = 1.28 \text{ in}^2
\]

\[
f_t = \frac{T_u}{\sum A_b} = (10.26^{\text{kips/bolt}} \times 8)
\]

\[
f_t = 64.13 \text{ ksi}
\]

Now,

\[
\phi F_{nv} A_b = 66.3 \times A_b - 0.76 \times f_t A_b \leq 51 A_b
\]

\[
= (29.29 - 21.5)^{\text{kips/bolt}} \leq 22.53 \text{ kips/bolt}
\]

\[
= 7.79 \text{ kips/bolt} \leq 22.53 \text{ kips/bolt}
\]

\[
\phi R_{nv} = 7.79 \text{ kips/bolt} \geq 3.775 \text{ kips/bolt} \quad (\text{OK})
\]

**Table A.1: Summary of Calculated Capacities at Various Limit States:**

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Limit states</th>
<th>Nominal (kips)</th>
<th>Factored (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Bearing Strength at beam web</td>
<td>65.74</td>
<td>49.30</td>
</tr>
<tr>
<td>b</td>
<td>Bearing Strength at girder web</td>
<td>562.96</td>
<td>422.24</td>
</tr>
<tr>
<td>c</td>
<td>Block shear rupture for angle</td>
<td>313.20</td>
<td>234.90</td>
</tr>
<tr>
<td>d</td>
<td>Block shear rupture for beam web</td>
<td>166.41</td>
<td>124.81</td>
</tr>
<tr>
<td>e</td>
<td>Shear yielding of angle</td>
<td>326.03</td>
<td>326.03</td>
</tr>
<tr>
<td>f</td>
<td>Shear rupture of angle</td>
<td>150.76</td>
<td>113.07</td>
</tr>
<tr>
<td>g</td>
<td>Flexural rupture of angle</td>
<td>137.14</td>
<td>102.86</td>
</tr>
<tr>
<td>h</td>
<td>Flexural yielding of angle</td>
<td>154.24</td>
<td>138.82</td>
</tr>
<tr>
<td>i</td>
<td>Bolt shear</td>
<td>94.36</td>
<td>70.77</td>
</tr>
<tr>
<td>j</td>
<td>Gross shear on beam web</td>
<td>183.30</td>
<td>183.30</td>
</tr>
<tr>
<td></td>
<td><strong>Connection capacity</strong></td>
<td><strong>65.74</strong></td>
<td><strong>49.30</strong></td>
</tr>
</tbody>
</table>

**Connection Details:**

**Bolts:** A325, 4 numbers of 3/4 in. diameter bolts; **Angle:** 2L8x4x7/16-11 1/2”

**Beam:** W16x45  **Girder:** W21x93
Appendix A: Extended Connection Design

Figure A.5: Connection Details

W 21 x 93

A325 - 3/4" dia.
4 no. of bolts

1.250 in.
1.250 in.

3.000 in.

11.500 in.

2.3125 in.

2L 8 x 4 x 7/16

W 16 x 45
Appendix B

This appendix summarizes the finite element results for all connections designed with three different diameters with and without stiffener.

Table B.1: Comparison of Measured and Computed Capacities (Bolt dia. 3/4 in. No stiffener)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Connection ID</th>
<th>Nominal Strength</th>
<th>Failure modes</th>
<th>Force transferred in</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>AISC (kips)</td>
<td>FEA (kips)</td>
<td>% difference</td>
<td>AISC</td>
</tr>
<tr>
<td>Angle thickness</td>
<td>EC1.2L-7/16</td>
<td>65.74</td>
<td>87.27</td>
<td>28.15</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td>EC1.2L-9/16</td>
<td>65.74</td>
<td>89.05</td>
<td>30.12</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td>EC1.2L-3/4</td>
<td>65.74</td>
<td>93.74</td>
<td>35.12</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td>EC1.2L-1</td>
<td>65.74</td>
<td>99.77</td>
<td>41.13</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Avg.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Girder web thickness</td>
<td>ECGw1-W21x93</td>
<td>65.74</td>
<td>87.27</td>
<td>28.15</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td>ECGw1-W21x73</td>
<td>65.74</td>
<td>88.93</td>
<td>29.99</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td>ECGw1-W21x48</td>
<td>65.74</td>
<td>79.65</td>
<td>19.14</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Avg.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Girder depth</td>
<td>ECG1-W24 x 76</td>
<td>65.74</td>
<td>90.7</td>
<td>31.91</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td>ECG1-W21 x 73</td>
<td>65.74</td>
<td>88.93</td>
<td>29.99</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td>ECG1-W18 x 65</td>
<td>65.74</td>
<td>86.41</td>
<td>27.18</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Avg.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam depth</td>
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<td>65.74</td>
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**BB= Bolt bearing; AT= Tearing of angle; BS= Bolt shear**
Following Figure represents the Shear force vs Displacement curves for connections with variables Angle thicknesses, girder web thicknesses, girder depths and, beam depths.

**Figure B.1:** Shear Force vs Displacement (Bolt dia. 3/4”, No Stiffener) - Angle thickness

**Figure B.2:** Shear Force vs Displacement (Bolt dia. 3/4”, No Stiffener) - Girder Web Thickness
Figure B.3: Shear Force vs Displacement (Bolt dia. 3/4", No Stiffener) - Girder Depth

Figure B.4: Shear Force vs Displacement (Bolt dia. 3/4", No Stiffener) - Beam Depth
### Table B.2: Comparison of Measured and Computed Capacities (Bolt dia. 3/4 in. with stiffener)

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**BB = Bolt bearing; AT = Tearing of angle; BS = Bolt shear**

Avg. 39.79

Avg. 26.36

Avg. 28.96

Avg. 28.96

Avg. 28.72
Appendix B

Figure B.5: Shear Force vs Displacement (Bolt dia. 3/4”, With Stiffener) - Angle thickness

Figure B.6: Shear Force vs Displacement (Bolt dia. 3/4”, With Stiffener) - Girder Web Thickness
Figure B.7: Shear Force vs Displacement (Bolt dia. 3/4”, With Stiffener) - Girder Depth

Figure B.8: Shear Force vs Displacement (Bolt dia. 3/4”, With Stiffener) - Beam Depth
### Table B.3: Comparison of Measured and Computed Capacities (Bolt dia. 7/8 in. No Stiffener)

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**BB= Bolt bearing; AT= Tearing of angle of angle; BS= Bolt shear**

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Figure B.9: Shear Force vs Displacement (Bolt dia. 7/8”, No Stiffener) - Angle Thickness

Figure B.10: Shear Force vs Displacement (Bolt dia. 7/8”, No Stiffener) - Girder Web Thickness
Figure B.11: Shear Force vs Displacement (Bolt dia. 7/8", No Stiffener) - Girder Depth

Figure B.12: Shear Force vs Displacement (Bolt dia. 7/8", No Stiffener) - Beam Depth
Appendix B

Table B.4: Comparison of Measured and Computed Capacities (Bolt dia. 7/8 in. with stiffener)

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<th>Tension in girder bolts (kips)</th>
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**BB= Bolt bearing; AT= Tearing of angle; BS= Bolt shear**
Appendix B

Figure B.13: Shear Force vs Displacement (Bolt dia. 7/8\”, With Stiffener) - Angle Thickness

Figure B.14: Shear Force vs Displacement (Bolt dia. 7/8\”, With Stiffener) - Girder Web Thickness
Figure B.15: Shear Force vs Displacement (Bolt dia. 7/8”, With Stiffener) - Girder Depth

Figure B.16: Shear Force vs Displacement (Bolt dia. 7/8”, With Stiffener) - Beam Depth
### Table B.5: Comparison of Measured and Computed Capacities (Bolt dia. 1 in. without stiffener)

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**BB= Bolt bearing; AT= Tearing of angle; BS= Bolt shear**
Figure B.17: Shear Force vs Displacement (Bolt dia. 1'', No Stiffener) - Angle Thickness

Figure B.18: Shear Force vs Displacement (Bolt dia. 1'', No Stiffener) - Girder Web Thickness
Figure B.19: Shear Force vs Displacement (Bolt dia. 1", No Stiffener) - Girder Depth

Figure B.20: Shear Force vs Displacement (Bolt dia. 1", No Stiffener) - Beam Depth
### Table B.6: Comparison of Measured and Computed Capacities (Bolt dia. 1 in. with stiffener)

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**BB= Bolt bearing; AT= Tearing of angle; BS= Bolt shear**
Figure B.21: Shear Force vs Displacement (Bolt dia. 1" , With Stiffener) - Angle Thickness

Figure B.22: Shear Force vs Displacement (Bolt dia. 1" , With Stiffener) - Girder Web Thickness
Appendix B

Figure B.23: Shear Force vs Displacement (Bolt dia. 1”, With Stiffener) - Girder Depth

Figure B.24: Shear Force vs Displacement (Bolt dia. 1”, With Stiffener) - Beam Depth