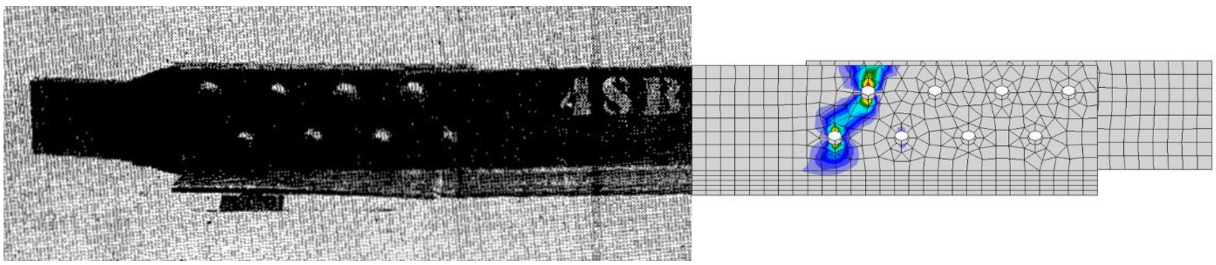
STRUCTURAL STEEL CONNECTION DESIGN FOR TENSILE RUPTURE BY ADVANCED INELASTIC ANALYSIS



Final Report
submitted to
IDEA StatiCa

by

Kayla Truman-Jarrell, Avery Burnham, and Mark D. Denavit
Department of Civil and Environmental Engineering
University of Tennessee, Knoxville

October 2022

ABSTRACT

Connections are critical in structural steel buildings for transferring forces from member to member. Connections must be designed for safety and to ensure they serve their intended function. Many resources are available to engineers for designing connections with common configurations and loads. But connection designers often encounter configurations and loading conditions for which there is little guidance. In these cases, design by advanced inelastic analysis can be particularly advantageous. IDEA StatiCa is a steel connection design software for design by advanced inelastic analysis. In this software, some limit states are captured in the same manner as standard strength equations, while others are not. The net-section tensile rupture limit state is among the most basic limit states not captured using standard strength equations. It is not necessary to use standard strength equations in design by advanced inelastic analysis if the analysis provides a comparable or higher level of reliability. To date, no rigorous reliability analysis has been performed to show IDEA StatiCa, and the underlying component-based finite element method, provides a comparable or higher level of reliability than provided by the standard strength equations. Such a reliability analysis is performed in this work for the limit state of tensile rupture. Data from hundreds of previously published experimental results exhibiting tensile rupture in a variety of connection types were examined and analyzed. Strengths from both standard equations and IDEA StatiCa were compared to the experimentally obtained strengths and to each other. A reliability analysis based on Monte Carlo simulations was conducted using results from the strength comparisons. Additionally, the sensitivity of the IDEA StatiCa strength to mesh parameters and plastic strain limit was quantified. The results indicate that IDEA StatiCa does, in most cases, provide a comparable or higher level of reliability than the standard strength equations. Cases where it does not are identified and options for modifications are recommended. Documentation of the level of safety provided by IDEA StatiCa for the tensile rupture limit state presented in this work will bring confidence to the overall approach and enable the wider use of this helpful tool.

Acknowledgements: This work was funded by IDEA StatiCa. The authors thank Dr. Martin Vild for his support and advice.

Cover Image: The cover image is of specimen 48B described by Greiner (1897). The left side of the image is a photograph by the experimentalists. The right side of the image is an IDEA StatiCa model showing plastic strain in the member when the maximum plastic strain is 5%.

TABLE OF CONTENTS

Chapter 1:
INTRODUCTION

Structural connections are crucial to safely transfer load from member to member. Connections in structural steel buildings take many forms and are subject to many different loading conditions. For some common connection types, detailed design guidance has been developed based on experimental and analytical research. For example, part 10 of the AISC *Steel Construction Manual* (2017) contains design recommendations for single plate shear connections. Similarly, AISC Design Guide 1 (2006) provides guidance and recommendations on the design of base plates and anchor rods. However, connection designers often encounter configurations and loading conditions for which there is little design guidance.

In the United States, the design of structural steel connections for buildings is governed by the *Specification for Structural Steel Buildings* published by the American Institute of Steel Construction (AISC). The latest edition of this standard was published in 2016 (AISC 2016). The next edition, to be published in 2022 is nearly final and drafts have been available for public review (AISC 2022a).

Design per the AISC *Specification* (AISC 2016) involves identifying applicable limit states, computing required strengths, computing available strengths, and ensuring that the available strength is greater than or equal to the required strength for each limit state. This process applies to common and uncommon connections alike, typically with calculations that can be completed by hand. The challenge for uncommon connections is that computing required and available strengths often rely on the use of unproven behavioral assumptions.

An alternative approach, made possible by advances in computer hardware and software, is to build a model capable of simulating the relevant behavioral effects, perform an analysis, and ensure that certain limits are met. This approach is permitted by the AISC *Specification* (AISC 2016) in the provisions of Appendix 1. Any method that uses inelastic analysis to design connections is permitted so long as it meets the general requirements of Section 1.3.1 of the AISC *Specification*.

Design by advanced inelastic analysis requires suitable analysis software. IDEA StatiCa is a steel connection design software based on the component-based finite element method (CBFEM). Connecting elements (e.g., plates, rolled shapes, hollow structural sections) are modeled with nonlinear shell elements that capture yielding, bolts are modeled as nonlinear springs, and welds are modeled as special constraints.

Extensive verification and validation studies have been performed on IDEA StatiCa to ensure accuracy and safety. The studies have mostly been conducted in comparison to European standards (Wald et al. 2020) however, comparisons to US standards are ongoing (Denavit and Truman-Jarrell 2021; Kasapoglu et al. 2021; Mahamid 2021). This study furthers the verification of IDEA StatiCa for US practice by examining how well the limit state of net-section tensile rupture is captured by IDEA StatiCa and identifying when IDEA StatiCa provides a comparable or higher level of reliability than the provisions of the AISC *Specification*.

The tensile rupture limit state was selected for investigation because it is the most basic and widely understood limit state that IDEA StatiCa captures in a fundamentally different manner than as prescribed in the AISC *Specification* (AISC 2016). The tensile yielding strength per IDEA StatiCa will never appreciably exceed the available strength from the AISC *Specification* given the constitutive relations used in the IDEA StatiCa model. Additionally, the limit states of bolt shear rupture, bearing, and tearout are evaluated in IDEA StatiCa using the equations from the AISC *Specification*. Tensile rupture, on the other hand is not directly evaluated in IDEA StatiCa. It is captured with a plastic strain limit. The default plastic strain limit in IDEA StatiCa is 5%. In the AISC *Specification*, tensile rupture is calculated based on the tensile strength of the steel, F_u, and employs a resistance factor of $\phi = 0.75$, neither of which are considered in IDEA StatiCa. In IDEA StatiCa the steel yields at the yield stress of the steel, F_y, times the resistance factor typically used for yielding, $\phi = 0.9$. Without explicit verification, it is unclear that these tradeoffs result in safe and accurate results.

The objective of this work is to rigorously evaluate how effectively IDEA StatiCa captures the tensile rupture limit state and to identify when IDEA StatiCa provides comparable or higher reliability when compared to the AISC *Specification* (AISC 2016) equations for tensile rupture. The work includes comparisons between previously published experimental results, results from design equations in the AISC *Specification*, and results from IDEA StatiCa.

This report is organized as follows. Chapter 2 describes the development of a database of previously published experimental data. Chapter 3 provides an overview of the methods used in the study. Specific connection types are evaluated in Chapters 4 through 9. The work is summarized, and conclusions are presented in Chapter 10.

Chapter 2:
EXPERIMENTAL DATABASE

To provide a trusted benchmark for comparing the results of various design approaches, a database of previously published well-documented results of physical experiments was compiled. An extensive review of the literature was performed to identify papers and reports that document tests relevant to the study, i.e., structural steel members loaded monotonically in tension to failure. Specimens were considered useful if they failed in tensile rupture or tensile yield. Specimens were not included if they were reported to have failed in the bolts, welds, tearout, block shear rupture, or any limit state other than tensile yield or tensile rupture.

The references used in the database are listed in Table 2.1 with the corresponding number of applicable specimens for the study. A total of 415 specimens were included in the database. The specimens were categorized into the following connection types: welded round HSS, welded rectangular HSS, bolted angles, welded angles, bolted plates, and welded plates. Table 2.2 provides the number of specimens found applicable for each connection type. It should be noted there were several experimental tests that were conducted using riveted connections. These connections were categorized as bolted specimens. Further discussion on these specimens is provided where necessary.

The information recorded in the database was dependent on the connection type. In general, the following parameters were documented for bolted connections: author(s), year, specimen name, cross-section name, measured dimensions, material grade, measured yield strength, measured ultimate strength, bolt material, bolt size, edge distances, bolt gage, bolt spacing, number of bolts per row, number of rows of bolts, experimental strength, and experimental failure mode. For welded specimens, the following information was typically recorded: author(s), year, specimen name, cross-section name, measured dimensions, material grade, measured yield strength, measured ultimate strength, weld strength, weld size, length of welds, experimental strength, and experimental failure mode.

Many specimens were reported in metric units. All lengths were converted to inches and reported to three decimal places. All forces were converted to kips and reported to one decimal place. All stresses were converted to ksi and reported to one decimal place.

Table 2.1: Specimen Enumeration by Reference

Reference	Specimen Count	Connection Type
Greiner (1897)	2	Riveted Angles
	2	Rivered Angles (Staggered)
McKibben (1907)	15	Riveted Angles
	9	Riveted Angles (Staggered)
Gibson and Wake (1942)	4	Welded Angles
Schutz and Newmark (1952)	26	Riveted Plates
	93	Riveted Plates (Staggered)
Munse (1959)	67	Riveted Plates
	23	Riveted Plates (Staggered)
Chesson and Munse (1963)	10	Bolted Angles
Regan and Salter (1984)	17	Welded Angles
Gonzalez (1989)	9	Welded Plates
	7	Welded Angles
Epstein (1992)	6	Bolted Angles
	7	Bolted Angles (Staggered)
Yeomans (1993)	10	Welded Rectangular HSS
	6	Welded Round HSS
Korol (1996)	7	Welded Rectangular HSS
Kulak and Wu (1997)	24	Bolted Angles
Cheng et al. (1998)	9	Welded Round HSS
Uzoegbo (1998)	14	Welded Angles
Petretta (2000)	7	Welded Angles
Mannem (2002)	19	Welded Plates
	16	Welded Angles
Bauer and Benaddi (2002)	6	Welded Angles
Willibald et al. (2006)	4	Welded Round HSS
Zhao et al. (2008)	29	Welded Rectangular HSS
Zhu et al. (2009)	9	Welded Angles
Može and Beg (2010)	16	Bolted Plates
Fang et al. (2013)	12	Welded Angles
Ke et al.(2018)	9	Bolted Angles
	9	Welded Angles
Dhanuskar and Gupta (2021a)	18	Welded Angles
Dhanuskar and Gupta (2021b)	8	Welded Angles

Table 2.2: Specimen Enumeration by Connection Type

Connection Type	Specimen Count
Welded Round HSS	19
Welded Rectangular HSS	46
Welded Angles	127
Bolted Angles	56
Bolted Angles with Staggered Bolts	28
Welded Plates	28
Bolted Plates	109
Bolted Plates with Staggered Bolts	116
Total	**529**

Chapter 3: METHODS

For each connection type examined in this study, two sets of connections are evaluated: an experimental set of connections derived from the experimental database and a reliability set of connections that is defined to cover a range of parameters and not directly based on any specific physical connections.

For the experimental set of connections, strengths from three different sources, 1) physical experiments, 2) AISC *Specification* equations, and 3) IDEA StatiCa, are compared. The comparisons help quantify how well IDEA StatiCa captures the limit state of tensile rupture. Of particular interest is the relationship between the material ratio, F_u/F_y, and the accuracy of IDEA StatiCa results. Given that the steel constitutive relation used in IDEA StatiCa does not have significant strain hardening (the hardening modulus is $E/1000$), and the AISC *Specification* strength equations are based on F_u, the material ratio may be an important parameter for characterizing the accuracy of IDEA StatiCa. The comparisons also provide data on variability for reliability analysis and the sensitivity of IDEA StatiCa results to the plastic strain limit and mesh parameters.

For the reliability set of connections, only strengths from the AISC *Specification* equations and IDEA StatiCa are compared since these connections are not based on physical tests. These comparisons are performed in the context of a reliability analysis to determine if IDEA StatiCa provides a comparable or higher level of reliability than the provisions of the AISC *Specification*.

This chapter presents general information on the determination of each strength as well as details of the reliability analysis. Details specific to each connection type are presented in the chapter on that connection type.

3.1 Experimental Results

The peak load recorded in the experiment, P_{EXP}, was taken as reported by the experimentalists for each specimen. As previously noted, the peak load recorded in the experiment and the observed failure mode were both included in the database for each specimen. Most experiments report the tensile rupture strength after significant yielding occurred and the load at which significant yielding occurred is not reported. An example of this is provided in Figure 3-1 from Dhanuskar and Gupta (2021b). Specimen 1-S1-250/90-B is represented by the solid black curve. The experimental strength was reported as 478.8 kN (107.6 kips), which is clearly beyond the point of yield. Another example is provided in Figure 3-2 with a plot from Fang et al. (2013) where the strengths extended beyond the linear portion of the load-deformation curve as well.

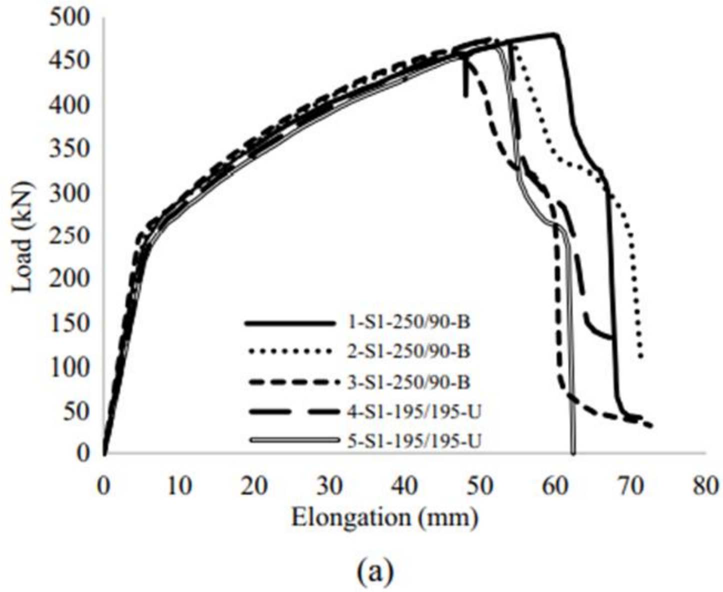


Figure 3-1: Load-deformation curve from Dhanuskar and Gupta (2021b)

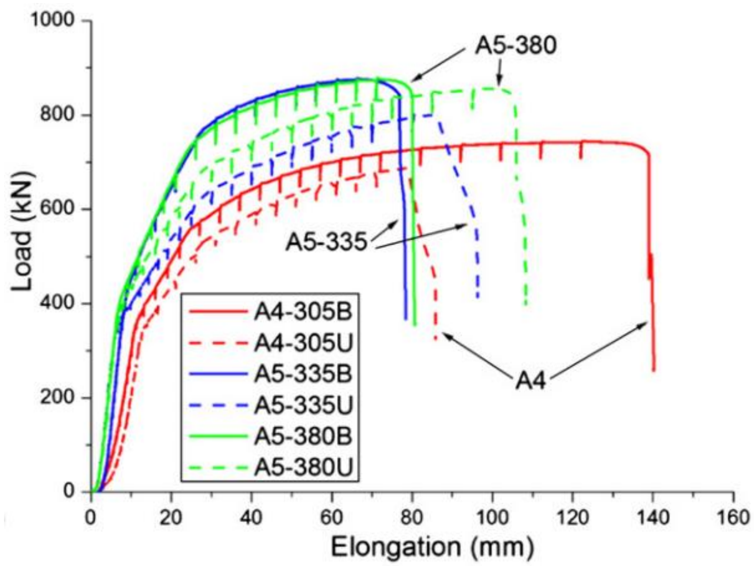


Figure 3-2: Load-deformation curve from Fang et al. (2013)

3.2 AISC Specification

The AISC *Specification for Structural Steel Buildings* (AISC 2016) governs the design of structural steel buildings in the United States. Strength design under the AISC *Specification* is performed using either the provisions for load and resistance factor design (LRFD) or the provisions for allowable strength design (ASD). This study focuses on LRFD exclusively. The governing strength equation for LRFD is

$$R_u \leq \phi R_n \tag{3-1}$$

where, R_u is the required strength using LRFD load combinations, R_n is the nominal strength, ϕ is the resistance factor, and ϕR_n is the available or design strength.

The nominal strength is the expected strength of the structure or component whereas the design strength is reduced by the resistance factor to account for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure. The AISC *Specification* (AISC 2016) includes equations for the nominal strength and lists resistance factors for each limit state.

This study focuses on two limit states: tensile yielding in the gross section and tensile rupture in the net section, both defined in Chapter D of the AISC *Specification* (AISC 2016). Other limit states were not evaluated because experimental specimens that failed by other limit states were not included in the experimental database.

The tensile yield strength, P_{YIELD}, is defined by Equation 3-2.

$$P_{YIELD} = F_y A_g \tag{3-2}$$

where F_y is the yield strength and A_g is the gross area of the member

The tensile rupture strength, $P_{RUPTURE}$, is defined by Equations 3-3 and 3-4.

$$P_{RUPTURE} = F_u A_e \tag{3-3}$$

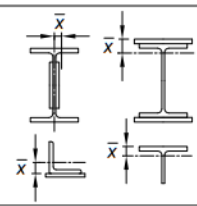
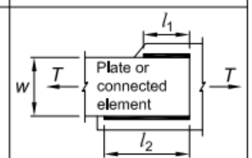
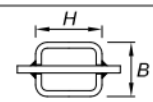
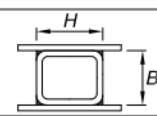
$$A_e = A_n U \tag{3-4}$$

where F_u is the ultimate strength, A_e is the effective net area of the member, A_n is the net area of the member, and U is the shear lag factor.

The shear lag factor, U, accounts for the non-uniform distribution of stresses over the cross-section. This factor results in a reduction for tension members connected by only some of the cross-sectional elements. The length of the connection and the centroid are factors that can impact the magnitude of U. The shear lag factor was determined based on Table D3.1 in the AISC *Specification* (2016) and replicated here as Figure 3-3. Each chapter will describe in detail the cases used from this table for each connection type.

The resistance factors are $\phi = 0.90$ for tensile yield and $\phi = 0.75$ for tensile rupture. However, resistance factors were not applied when comparing strengths for connections in the experimental database. Also, unless otherwise noted, measured dimensions and material properties were used in the calculation of P_{YIELD} and $P_{RUPTURE}$. The lesser of P_{YIELD} and $P_{RUPTURE}$ is denoted as P_{AISC}.

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).		$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)		$U = 1 - \dfrac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	–
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.		$U = \dfrac{3l^2}{3l^2 + w^2}\left(1 - \dfrac{\bar{x}}{l}\right)$	
5	Round HSS with a single concentric gusset plate through slots in the HSS.		$l \ge 1.3D$, $U = 1.0$ $D \le l < 1.3D$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{D}{\pi}$	
6	Rectangular HSS.	with a single concentric gusset plate	$l \ge H$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \ge H$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	
7	W-, M-, S- or HP-shapes, or tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with three or more fasteners per line in the direction of loading	$b_f \ge \frac{2}{3}d$, $U = 0.90$ $b_f < \frac{2}{3}d$, $U = 0.85$	–
		with web connected with four or more fasteners per line in the direction of loading	$U = 0.70$	–
8	Single and double angles. (If U is calculated per Case 2, the larger value is permitted to be used.)	with four or more fasteners per line in the direction of loading	$U = 0.80$	–
		with three fasteners per line in the direction of loading (with fewer than three fasteners per line in the direction of loading, use Case 2)	$U = 0.60$	–

B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); D = outside diameter of round HSS, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm); d = depth of section, in. (mm); for tees, d = depth of the section from which the tee was cut, in. (mm); l = length of connection, in. (mm); w = width of plate, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm).

[a] $l = \dfrac{l_1 + l_2}{2}$, where l_1 and l_2 shall not be less than 4 times the weld size.

Figure 3-3: AISC *Specification* Table D3.1 (AISC 2016)

3.3 IDEA StatiCa

IDEA StatiCa is a steel connection design software based on the component-based finite element method. Engineers using this software model a connection, apply loads obtained from a structural analysis of the overall structure (performed by hand or using other software), run an analysis of the connection, then review the results to determine the adequacy of the connection. The main results are the plastic strain in the members and connecting elements and the utilization ratios of the bolts and welds. The limiting plate plastic strain is set to 5.0% by default, however the limit can be adjusted by the user. The bolt and weld utilization ratios are calculated such that a value of 100% or below is passing and a value exceeding 100% is failing. If the connection does not pass all checks, then the connection should be adjusted, and the analysis rerun. In other modes, IDEA StatiCa has the capability to perform capacity design, approximate the design resistance of the connection, and evaluate fatigue.

The graphical user interface of IDEA StatiCa enables modeling of connections far quicker than general finite element software packages. Models consist of members, loads, and operations (including connecting elements, bolts, and welds). A variety of templates for common connection forms are available. Meshing is done automatically. Members and plates are modeled with shell elements with bilinear elastic-plastic constitutive relations. The yield stress is reduced by a resistance factor, with a default value of 0.9 for design by the AISC *Specification* and LRFD. The hardening slope is taken as $E/1000$, where E is the modulus of elasticity. This is a minimal value, used only to avoid the numerical issues that would arise with zero post-yield stiffness, and does not produce any significant strain hardening. Bolts and anchors are modeled as nonlinear springs and welds are modeled as special constraints.

Initial steps of for creating models in IDEA StatiCa are shown in Figure 3-4. Subsequent steps for defining each specific connection type are presented in Chapters 4 through 9.

Experimental Set:

1. Set all resistance factors to 1.0.
2. Define materials with measured values of F_y, F_u, and E (if reported).
3. Define cross sections with measured dimensions (e.g., height, width, thicknesses).
4. *continue with steps for individual connection type*.

Reliability Set:

1. Ensure all resistance factors are set to their default value.
2. Import materials with nominal properties.
3. Import cross sections with nominal dimensions.
4. *continue with steps for individual connection type*.

Figure 3-4: Initial steps for defining models in IDEA StatiCa

For this study, the maximum applied load that IDEA StatiCa deems safe, referred to as P_{IDEA}, was of the most interest. To calculate P_{IDEA}, applied loads were adjusted and the analysis rerun in an iterative process until the connection was right at the limit. For most analyses, the iterative determination of P_{IDEA} was performed using a script written in the Python programming language that communicated with IDEA StatiCa through its application programming interface (API). The tolerance for the plastic strain limit was set as 0.0005. For example, if the target plastic strain is 5.0% (i.e., the default value), the script would perform analyses in succession, updating the magnitude of loading until the maximum plastic strain was between 0.0495 and 0.0500 (4.95%

and 5.00%). When a result in this range was found, the applied load for that specific analysis was taken as P_{IDEA}. The results of approximately 20% of the specimens were manually checked in IDEA StatiCa. In addition to P_{IDEA}, the script records the connection name, mesh parameter set, plastic strain limit, plastic strain, bolt utilization, and weld utilization.

Automation of the analyses allowed investigation of varying plastic strain limits and mesh density. Four sets of mesh parameters were investigated as detailed in Table 3.1 where mesh parameter set 'B' represents the default parameters. Five plastic strain limits were evaluated: 1.0%, 2.5%, 5.0% (default), 7.5%, and 10%. For each specimen, 20 values of P_{IDEA} were computed (five plastic strain limits for each of the four mesh sets). Unless otherwise noted, the results throughout the entirety of this study are presented as mesh parameter set 'B' and 5% plastic strain limit.

Table 3.1: Mesh Parameter Set List

Code Setup Parameters	**Mesh Parameter Set**			
	A	**B***	**C**	**D**
Division of surface of the biggest circular hollow member	32	64	96	128
Division of arc of rectangular hollow member	3	3	6	12
Number of elements on biggest member web or flange	4	8	16	32
Number of elements on biggest web of RHS member	8	16	32	64
Minimal size of element	0.394 in. (10 mm)	0.394 in. (10 mm)	0.197 in. (5 mm)	0.098 in. (2.5 mm)
Maximal size of element	3.937 in. (100 mm)	1.969 in. (50 mm)	1.969 in. (50 mm)	0.984 in. (25 mm)

* default settings in IDEA StatiCa

3.3.1 Member Model Type

Only the connection region is modeled in IDEA StatiCa (note, however, other packages in the IDEA StatiCa software model members). Thus, it is necessary to define boundary conditions at the end of each member away from the connection. Given that analyses are performed in three-dimensions, there are 6 degrees-of-freedom at each member end, each of which can be fixed (i.e., zero displacement or rotation, non-zero reaction) or free (i.e., non-zero displacement, force or moment can be applied). IDEA StatiCa provides four options, termed "model types" for combinations of these boundary conditions: "N-Vy-Vz-Mz-My-Mz", "N-Vz-My", "N-Vy-Mz", and "N-Vy-Vz". The inclusion of 'Mx', 'My', or 'Mz' in the mode name indicates that member end is free to rotate about the 'x', 'y', or 'z' axes, respectively. The axes are defined with respect to the member with the 'x' axis defined along the length of the member and the 'y' and 'z' axes defined perpendicular to the longitudinal axis of the member. The inclusion of 'Vy', or 'Vz' in the mode name indicates that the member end is free to translate in the 'y', or 'z' directions, respectively. 'N' is included in all model names, indicating that in all cases the member end is free to translate along the longitudinal axis of the member. The absence of a degree-of-freedom from the model name indicates that the degree-of-freedom is restrained (i.e., zero displacement or rotation).

Determining the end conditions that best represents physical behavior can be critical to the end results. A small study was conducted to determine the appropriate end conditions for the tension members investigated in this work. The following specimens were used in the study: bolted angle S1 from Kulak and Wu (1997), welded plate P-B-2 from Gonzalez (1989), and welded tee T-L-1a from Gonzalez (1989). The Gonzalez specimens are actually double plates and double tee specimens, however for this study only single tension members were used. These specimens were chosen to investigate the effect of end conditions on eccentricity in two directions (as exists for the angle), eccentricity in one direction (as exists for the tee), and minimal eccentricity (as exists for the plate). For clarity, all tension members presented in this study are oriented on the same local axes. As shown in Figure 3-5, the *x*-axis runs along the length of the tension member, the *y*-axis runs along the width of the gusset plate, and the z-axis is perpendicular to the plane of the gusset plate.

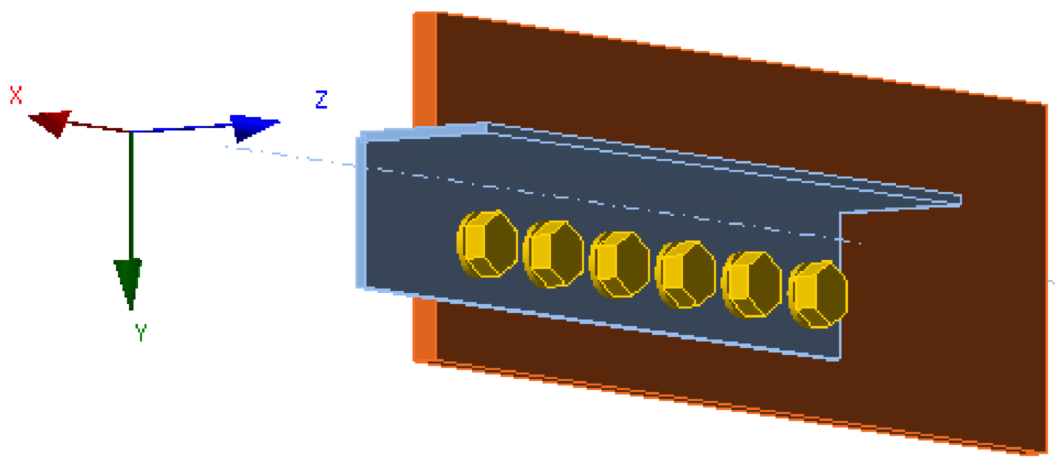


Figure 3-5: Local Axes on Bolted Angle

The end conditions and selection of bearing and nonbearing members were investigated in this study. In IDEA StatiCa, boundary conditions are applied to the end of the bearing member to restrict rigid body motion of the connection. There can only be one bearing member in IDEA StatiCa with all other members analyzed as nonbearing. The study presented below was conducted on the single angle tension member from Kulak and Wu (1997). The results are presented in Table 3.2. The failure mode in IDEA StatiCa was plate strain for all analyses, however the failing member did vary based on the bearing member selection. This is noted in the table under “Failure Detail”. Since the results do not change when the bearing member end conditions vary, it was concluded the end conditions of the bearing member do not impact the results. Therefore, the only significant end condition is that of the nonbearing member. Unless otherwise noted, all tension members are modeled as nonbearing members. Table 3.3 presents results from the remainder of the end condition study. The table contains results from the angle, tee, and plate tension member specimens. The gusset plate was set as the bearing member for all analyses presented in the table. Plate strain controlled for all analyses as well.

Table 3.2: Single Angle Bearing Member-End Condition Study

Index	Bearing Member	Model Type		IDEA StatiCa Results			
		Gusset Plate	Tension Member	Strength, kips	Plate Strain, %	Bolt/Weld Utiliz., %	Failure Detail
1	GP	[1]	[1]	27.3	4.8	46.2	GP
2	GP	[2]	[1]	27.3	4.8	46.2	GP
3	GP	[3]	[1]	27.3	4.8	46.2	GP
4	GP	[4]	[1]	27.3	4.8	46.2	GP
5	TM	[1]	[1]	46.4	4.9	27.6	TM
6	TM	[1]	[2]	46.4	4.9	27.6	TM
7	TM	[1]	[3]	46.4	4.9	27.6	TM
8	TM	[1]	[4]	46.4	4.9	27.6	TM

GP is gusset plate; TM is tension member; [1] is 'N-Vy-Vz-Mx- My-Mz'; [2] is N-Vz-My; [3] is N-Vy-Mz; [4] is N-Vy-Vz

Table 3.3: End Condition Study

Index	Tension Member Type	Tension Member Model Type	Strength, kips	Plate Strain, %	Bolt/Weld Utiliz., %	Failure Detail
1	Angle	[1]	27.3	4.8	46.2	GP
2	Angle	[2]	27.3	4.8	46.2	GP
3	Angle	[3]	63.4	4.9	46.3	TM
4	Angle	[4]	54.2	4.9	33.1	TM
5	Tee	[1]	17.4	5	87.6	TM
6	Tee	[2]	17.4	4.8	87.5	TM
7	Tee	[3]	65	5	85.9	TM
8	Tee	[4]	62	5	80.1	TM
9	Plate	[1]	41.3	4.6	79.6	TM
10	Plate	[2]	41.3	4.6	79.5	TM
11	Plate	[3]	41	4.5	79.7	TM
12	Plate	[4]	41.3	5	79.5	TM

GP is gusset plate; TM is tension member; [1] is 'N-Vy-Vz-Mx-My-Mz'; [2] is N-Vz-My; [3] is N-Vy-Mz; [4] is N-Vy-Vz

The angle specimen has two directions of eccentricity, and the strength varies significantly for all model types. The tee specimen has one direction of eccentricity, and two model types result in similar strengths. The plate specimen has minimal eccentricity, and two model types produce the same results with the remaining being not far off. It is concluded that end conditions become less significant as the eccentricity approaches zero; however, for specimens with multiple directions of eccentricity, the end conditions are very significant. For the angle and tee analyses, the model type [2] produced unrealistically low strengths. The angle and tee specimens indicate end conditions [3] and [4], 'N-Vy-Mz' and 'N-Vy-Vz', respectively, are more realistic of the physical experiments than the other options. For context, deformed shapes of the angle specimen with end conditions [3] and [4] are provided below in Figure 3-6 and Figure 3-7. All deformed shapes are shown with a deformation scale set to 10. The deformed shape of the angle with end condition N-Vy-Mz shows significant bending in the plate and tension member. Whereas, Figure 3-7 does not show significant bending in the tension member or gusset plate. IDEA StatiCa recommends engineers to model braced members using the 'N-Vy-Vz' model type. Tension members are commonly used as braced members in trusses. Per the recommendation, the 'N-Vy-Vz' model type was chosen for all subsequent analyses in this study.

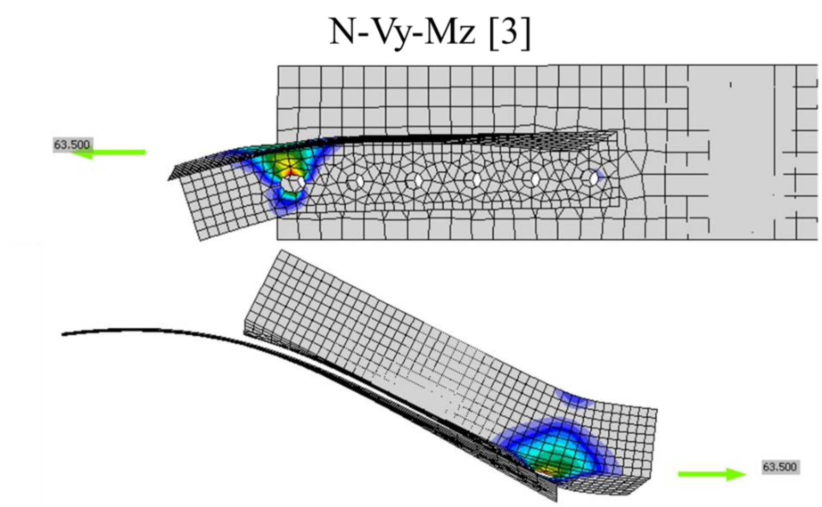


Figure 3-6: Deformed shapes of angle with end condition N-Vy-Mz

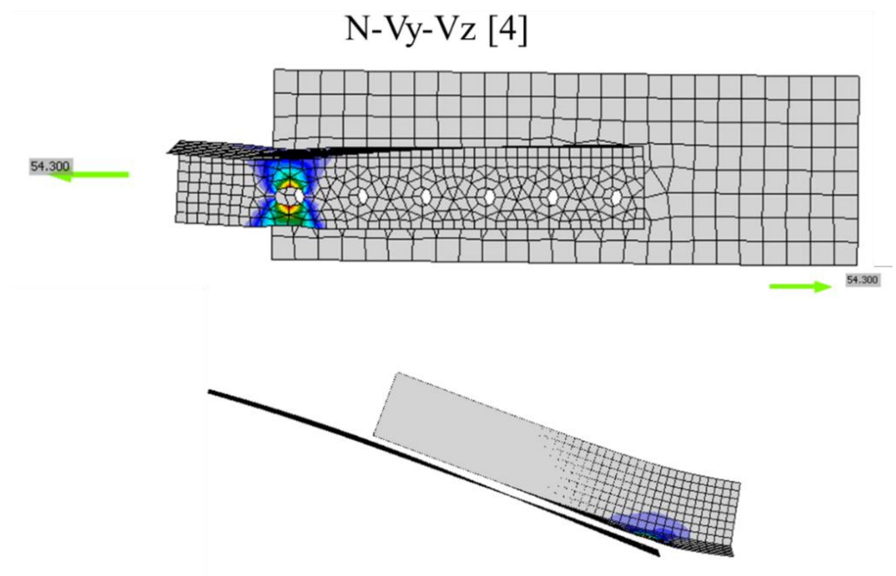


Figure 3-7: Deformed shapes of angle with end condition N-Vy-Vz

3.3.2 Mesh Study

A small study was conducted to better understand the sensitivity of strength results to mesh parameters in IDEA StatiCa. Similarly sized round and rectangular (square) HSS members were used for this study. Connections were welded as described in Chapter 4 for the round HSS and Chapter 5 for the rectangular HSS. Cross sections were a HSS6.000×0.250 (round) and HSS6×6×1/4 (square), both with ASTM A500 Gr. C material (F_y = 50 ksi, F_u = 62 ksi), 0.375 in. weld size, 8 in. weld length, and gusset plate thickness of 0.5 in. All analyses were conducted with 5.0% plastic strain limit.

This study was conducted using more increments than the mesh parameters outlined in Table 3.1. The rectangular HSS mesh is defined by the parameters 'number of elements on biggest web of RHS member' and 'division of arc of rectangular hollow member', whereas the round HSS is defined only by the parameter 'division of surface of the biggest circular hollow member'. Figure 3-8 provides the results of the study. The strengths are plotted against the number of elements around the perimeter of the HSS member. The results are provided for both the design strengths (including resistance factors) and the nominal strengths (not including resistance factors).

The results show a decrease in strength with increasing number of elements, but no distinct convergence. Plateaus in the results were observed where a minimal change in strength is noted for two or more increments and followed by decreases in strength for the following increments. The runtime of the analyses increases dramatically with number of elements and occasionally results in analysis error; therefore, it was not determined if the results would eventually converge.

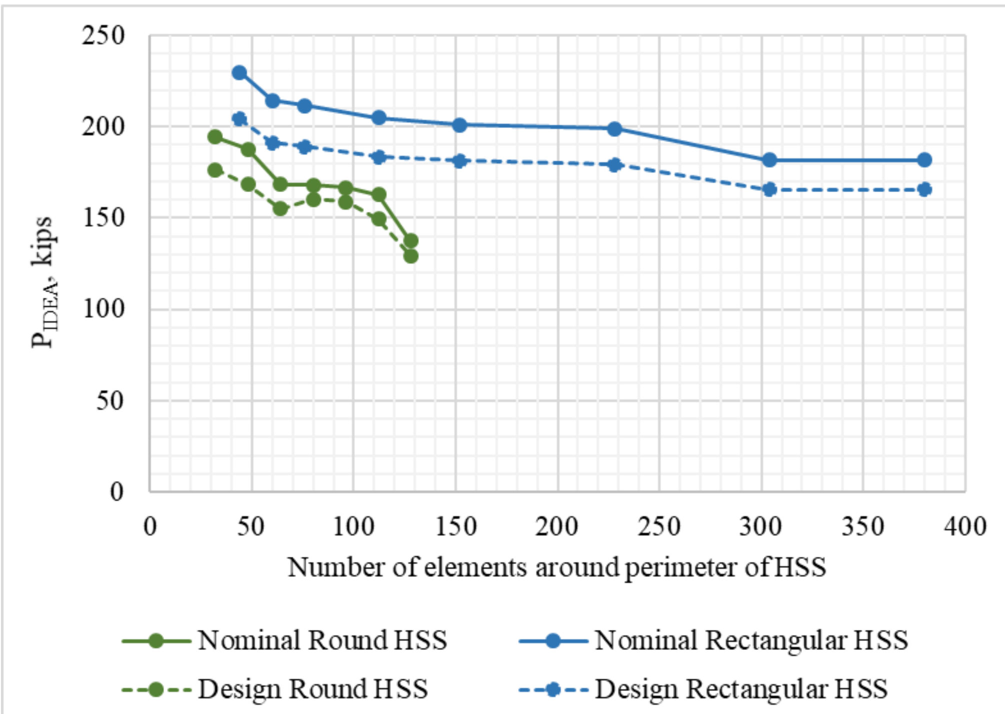


Figure 3-8: Mesh refinement for round and rectangular HSS

3.4 Presentation of Results

The presentation of comparisons between experimental results, IDEA StatiCa results and AISC *Specification* calculation results is the same for each of the various connection types examined in this work. Results are presented in a series of tables and figures.

The first table (e.g., Table 4.4) lists results of the AISC *Specification* calculations including yield strength, shear lag factor, controlling case for the shear lag factor, and rupture strength. For the connections with HSS members, results are presented for both the 2016 AISC *Specification* and the 2022 AISC *Specification*. For other connections, the results were the same for both editions of the specification. An entry of "N/A" appears for cases where the shear lag factor is undefined,

The second table (e.g., Table 4.5) lists the strength from the experiment, AISC *Specification* equation calculations, and IDEA StatiCa. The failure mode for each is also identified. The AISC strength reported in this table, $P_{AISC,}$ is the minimum of tensile yield and tensile rupture listed in the first table (e.g., Table 4.4). As noted previously, these strengths were computed using measured material and geometric properties and without resistance factors. The reported IDEA StatiCa strength, P_{IDEA}, is for the default mesh settings in the software and the default plastic strain limit (i.e., 5%).

After the two tables, a series of 7 figures is presented. The first two figures (e.g., Figure 4-6 and Figure 4-7) provide a general overview of the results. The first figure shows several strengths normalized by F_yA_g plotted versus their experimental set index (i.e., the order of the specimen in the database, which is somewhat arbitrary but grouped by reference). The second figure shows normalized IDEA StatiCa and AISC tensile rupture strengths vs normalized experimental strengths. In this figure, the colored lines are best fit lines to the data points of the same color.

The third and fourth figures (e.g., Figure 4-8 and Figure 4-9) explore trends in the results with respect to the material strength ratio F_u/F_y. Given that IDEA StatiCa uses the yield strength F_y and the AISC Specification equation for tensile rupture uses F_u, one may expect to observe trends in the results with the material strength ratio. When strengths are normalized by F_yA_g, as in the third figure, tensile rupture strength should increase with increasing F_u/F_y. When the IDEA StatiCa strength is normalized by the experimental strength, as in the fourth figure, the resulting ratio should decrease with increasing F_u/F_y.

The fifth and sixth figures (e.g., Figure 4-10 and Figure 4-11) explore the impact of the plastic strain limit in IDEA StatiCa. The fifth figure is like the first figure (e.g., Figure 4-6) but with multiple lines of IDEA StatiCa strength representing various plastic strain limits. The sixth figure presents the results from IDEA StatiCa only and with each strength normalized by the corresponding strength for the default plastic strain limit (i.e., 5%). All IDEA StatiCa results in this figure were obtained with default mesh parameters.

The seventh figure (e.g., Figure 4-12) explores the impact of mesh parameters. IDEA StatiCa strength for each specimen are normalized with respect to the IDEA StatiCa strength using the default mesh parameters and plotted versus experimental set index. Details of each mesh parameter set are listed in Table 3.1. All IDEA StatiCa results in this figure were obtained with the default plastic strain limit (i.e., 5%).

Lastly, a third table (e.g., Table 4.6) lists summary statistics of the test-to-predicted ratio. The test-to-predicted ratio is defined as $P_{EXP}/P_{RUPUTRE}$ for the AISC *Specification* calculations and P_{EXP}/P_{IDEA} for IDEA StatiCa. The test-to-predicted ratio for AISC is used in the reliability analysis for the professional factor random variable $\tilde{X}_R$. For the connections with HSS members, the test-to-predicted ratio for the 2022 edition of the AISC *Specification* is used.

3.5 Reliability Analysis

As previously mentioned, the AISC *Specification* (AISC 2016) allows design by advanced inelastic analysis when the analysis provides a comparable or higher level of reliability. Thus, analyses are performed in this work to evaluate the reliability of connections designed per the AISC *Specification* in comparison to those designed using IDEA StatiCa. Structural reliability is commonly quantified using the reliability index, β. The reliability index is a value that represents the probability of failure for a particular structural component. According to the commentary on the AISC *Specification*, current provisions provide a reliability index of approximately 4.0 for connections and 2.6 for members. For the development of the new provisions for rectangular HSS in the 2022 AISC *Specification*, Dowswell (2021) used a target reliability index of 4.0 for tensile rupture.

The methodology for the reliability analysis is outlined in the following discussion. To begin, a limit state function for tensile yield and tensile rupture can be represented by $\tilde{g}_Y$ and $\tilde{g}_R$, respectively.

$$\tilde{g}_Y = \tilde{R}_Y - \tilde{Q} \tag{3-5}$$

$$\tilde{g}_R = \tilde{R}_R - \tilde{Q} \tag{3-6}$$

where $\tilde{R}_Y$ is a random variable representing the resistance for the tensile yield limit state, $\tilde{R}_R$ is a random variable representing the resistance for the tensile rupture limit state, and $\tilde{Q}$ is a random variable representing the demand on the connection.

The random variable for the resistance of the tensile yield limit state is a function of $\tilde{A}_g$ and $\tilde{F}_y$, expressed as:

$$\tilde{R}_Y = \tilde{A}_g \tilde{F}_y \tag{3-7}$$

where $\tilde{A}_g$ is a random variable representing actual gross area and $\tilde{F}_y$ is a random variable representing the actual yield stress. These random variables are defined as:

$$\tilde{A}_g = \tilde{X}_A A_{gn} \tag{3-8}$$

$$\tilde{F}_y = \tilde{X}_{Fy} F_{yn} \tag{3-9}$$

where $\tilde{X}_A$ is a random variable representing the ratio between actual and nominal gross area, A_{gn} is the nominal gross area, $\tilde{X}_{Fy}$ is a random variable representing the ratio between actual and nominal yield stress, and F_{yn} is the nominal yield stress.

The random variable for the resistance of the tensile rupture limit state is a function of $\tilde{X}_R$, $\tilde{A}_e$, and $\tilde{F}_u$, expressed as:

$$\tilde{R}_R = \tilde{X}_R \tilde{A}_e \tilde{F}_u \tag{3-10}$$

where, $\tilde{X}_R$ is a random variable representing the ratio between actual strength and calculated tensile rupture strength (also referred to as the professional factor), $\tilde{A}_e$ is a random variable representing the actual effective net area, and $\tilde{F}_u$ is a random variable representing the actual ultimate tensile strength. $\tilde{A}_e$ and $\tilde{F}_u$ are defined as:

$$\tilde{A}_e = \tilde{X}_A \mathrm{A}_{\mathrm{en}} \tag{3-11}$$

$$\tilde{F}_u = \tilde{X}_{Fu} F_{un} \tag{3-12}$$

where $\tilde{X}_A$ is a random variable representing the ratio of actual and nominal effective net area, A_{en} is the nominal effective net area, $\tilde{X}_{Fu}$ is a random variable representing the ratio of actual to nominal ultimate tensile strength, and F_{un} is the nominal ultimate tensile strength.

The demand, $\tilde{Q}$, is taken as the sum of the dead load and the live load and can be expressed as:

$$\tilde{Q} = \tilde{X}_D P_{Dn} + \tilde{X}_L P_{Ln} \tag{3-13}$$

where $\tilde{X}_D$ is a random variable representing the ratio of the actual to nominal dead load, P_{Dn} is the nominal dead load, $\tilde{X}_L$ is the random variable representing the ratio of the actual to nominal live load, and P_{Ln} is the nominal live load.

Nominal dead and live loads, P_{Dn} and P_{Ln}, are calculated assuming the required strength equals the design strength of P_{IDEA}, standard load combinations, and a selected live-to-dead load ratio (P_{Ln}/P_{Dn}). The live-to-dead load ratio was taken as unity for all reliability analyses in this work.

$$P_u = \max(1.4P_{Dn},\ 1.2P_{Dn} + 1.6P_{Ln}) = \\ \phi P_{n,AISC} = \min\left(0.9A_{gn}F_{yn},\ 0.75A_{en}F_{un}\right) \tag{3-14}$$

$$P_u = \max(1.4P_{Dn},\ 1.2P_{Dn} + 1.6P_{Ln}) = P_{IDEA} \tag{3-15}$$

where P_{Dn} and P_{Ln} are calculated based on the ratio chosen, $\phi P_{n,AISC}$ is the design strength according to the AISC *Specification* (AISC 2016), and P_{IDEA} is the strength determined by IDEA StatiCa for a given connection (with resistance factors applied).

Failure occurs when the demand exceeds the resistance, $\tilde{R}_Y$ or $\tilde{R}_R$. Numerically, the probability of failure is expressed as the probability of either $\tilde{g}_Y$ or $\tilde{g}_R$ being less than zero:

$$P_f = P(\tilde{g}_Y < 0\ \cup\ \tilde{g}_R < 0) \tag{3-16}$$

The probability of failure is converted to the reliability index using:

$$\beta = -F_x^{-1}\left(P_f\right) \tag{3-17}$$

where F_x^{-1} is the inverse normal cumulative distribution.

The overall process for the reliability analyses was as follows: (1) determine statistical parameters of the professional factor ($\tilde{X}_R$) based on test-to-predicted ratios for each connection type, (2) determine statistical parameters of other random variables from the literature, (3) define set of controlled connections (i.e., the reliability set of connections), (4) calculate design strength of the connections using nominal properties per AISC *Specification* (AISC 2016) equations, (5) model connection set in IDEA StatiCa using nominal properties and resistance factors and determine maximum applied load, P_{IDEA}, (6) calculate reliability indices, β, for AISC and IDEA StatiCa respectively using a Python script which implements the above equations in a Monte Carlo simulation (1,000,000 trials were performed for each analysis).

Table 3.4 lists the statistical parameters for each random variable used in this analysis and the source of the information. The statistical parameters for the profession factor ($\tilde{X}_R$) varies from connection type to connection type and is determined from the comparisons to the experimental set of connections performed in this work. Specific values are presented in the chapter for each connection type.

Table 3.4: Summary statistics for random variables used in the reliability analysis

Variable		Mean	COV	Distribution	Source
$\tilde{X}_R$		varies*	varies*	Normal	this study
$\tilde{X}_D$		1.05	0.10	Normal	Ellingwood et al. (1980)
$\tilde{X}_L$		1.00	0.25	Extreme Value Type I	Ellingwood et al. (1980)
$\tilde{X}_A$		1.00	0.05	Normal	Ravindra and Galambos (1978)
$\tilde{X}_{Fu}$	A36 (plate)	1.39	0.07	Bivariate Lognormal	Liu et al. (2007)
	A572 Gr. 50 (plate)	1.16	0.07	Bivariate Lognormal	Liu et al. (2007)
	A500 Gr. C	1.3	0.1**	Bivariate Lognormal	AISC 341-22 (2022)
	A53 Gr. B	1.59	0.11	Bivariate Lognormal	Liu et al. (2007)
	A36 (angle)	1.34	0.07	Bivariate Lognormal	Liu et al. (2007)
$\tilde{X}_{Fy}$	A36 (plate)	1.23	0.04	Bivariate Lognormal	Liu et al. (2007)
	A572 Gr. 50 (plate)	1.26	0.07	Bivariate Lognormal	Liu et al. (2007)
	A500 Gr. C	1.2	0.1**	Bivariate Lognormal	AISC 341-22 (2022)
	A53 Gr. B	1.16	0.06	Bivariate Lognormal	Liu et al. (2007)
	A36 (angle)	1.22	0.04	Bivariate Lognormal	Liu et al. (2007)

COV: coefficient of variation; *mean and COV of the professional factor were computed from test-to-predicted ratio for each connection type, **assumed value

Chapter 4:
WELDED ROUND HSS

4.1 Description of Connection

Round HSS members welded to a gusset plate are evaluated in this chapter. A schematic of this connection is shown in Figure 4-1. The figure also defines the symbols used for the various dimensions of the connection. The typical connection includes a slotted round HSS tension member welded to a gusset plate. It should be noted some connections in the experimental set include a notched plate where the round HSS is not slotted. The hidden line in Figure 4-1 indicating the edge of the gusset plate would not be representative of those specific specimens.

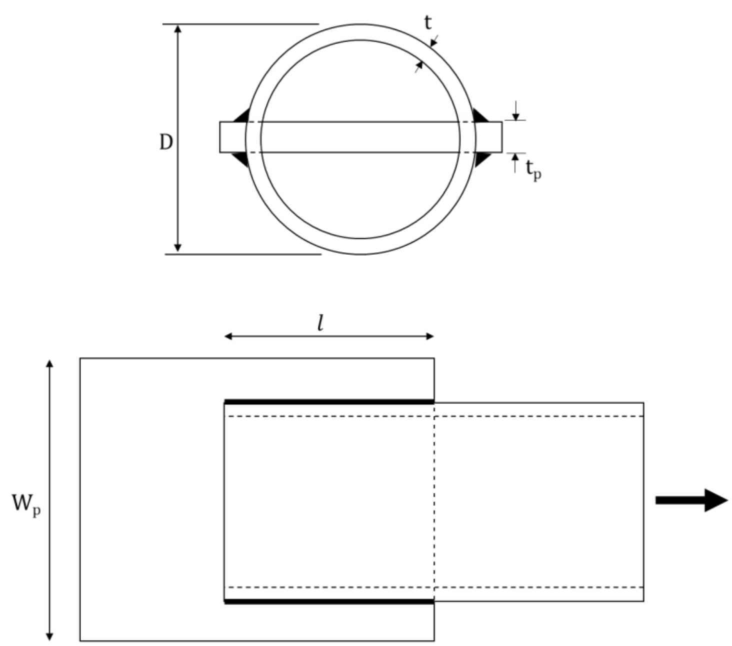


Figure 4-1: Schematic of Welded Round HSS Connection

For this connection, the AISC *Specification* calculations for the shear lag factor used in the evaluation of tensile rupture vary significantly from the 2016 edition (AISC 2016) to the 2022 edition (AISC 2022a) based on the work of Martinez-Saucedo and Packer (2009). The provisions of both editions are evaluated in this work. The 2016 shear lag factor calculations include the use of Case 5 in Table D3.1 of the AISC *Specification* shown in Figure 3-3. The equation is as follows:

$$l \geq 1.3D, \qquad U = 1.0 \tag{4-1}$$

$$D \leq l < 1.3, \qquad U = 1 - \frac{\bar{x}}{l} \tag{4-2}$$

$$\bar{x} = \frac{D}{\pi} \tag{4-3}$$

when the length of the weld, l, is less than the diameter of the HSS, D, the shear lag factor, U, is undefined.

For the 2022 edition of the AISC *Specification*, the equation is as follows:

$$U = \left[1 + \left(\frac{\bar{x}}{l}\right)^{3.2}\right]^{-10} \tag{4-4}$$

$$\bar{x} = \frac{R\sin\theta}{\theta} - \frac{1}{2}t_p \tag{4-5}$$

where l is the length of the weld and θ (in radians), R, and t_p are defined in Figure 4-2.

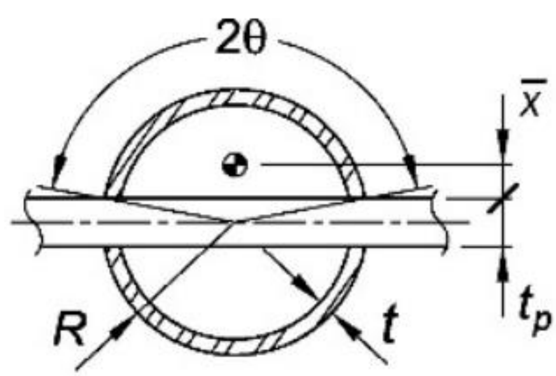


Figure 4-2: Figure from AISC *Specification* (AISC 2022a) defining parameters of round HSS

For the specimens with a notched plate (i.e., not a slotted HSS), the net area is equal to the gross area. Additionally, some specimens have weld across the thickness of the gusset plate (i.e., a weld return). The net area of specimens with a weld return was also taken as the gross area. For specimens with a slotted HSS and no weld return, the net area was taken as the gross area minus the removed area for the slots.

4.2 Comparison to Experimental Results

4.2.1 Description of Experimental Specimens

A total of 19 round HSS specimens from 3 references were identified for evaluation in this work as detailed in Table 4.1. Some of the specimens described in these references were not included. Willibald et al. (2006) tested 8 round HSS specimens, however, two of the specimens were subject to compression and an additional two specimens failed in block shear rupture. These four specimens were excluded from this study.

Table 4.1: Count of Welded Round HSS Connections by Reference

Reference	Specimen Count
Cheng et al. (1998)	9
Willibald et al. (2006)	4
Yeomans (1993)	6
Total	**19**

A description of each specimen is provided in Table 4.2 and Table 4.3. This information was used to model the specimens in IDEA StatiCa and to calculate the strengths according to the AISC *Specification* for the 2016 edition and the 2022 edition. The material grades were not provided for every specimen, however measured tensile yield strengths and ultimate strengths were reported and used for all calculations and modeling. Additionally, the weld strength was rarely provided. Unless otherwise noted in the text, E70XX weld strength was used. Yeomans (1993) did not provide the weld size but noted that welds were designed to not control. A weld size of 0.197 in. (5 mm) was used for the Yeomans specimens. For all round HSS specimens, the gross area was calculated based on the measured diameter and measured thicknesses. The column in the table labeled "Weld Return" refers to the indication of a weld around the edge of the plate, as shown in (B) of Figure 4-3. Note that weld returns were not included in IDEA StatiCa models.

Table 4.2: Welded Round HSS Connection Experimental Specimen Parameters

Index	Reference	Specimen	*F_y, ksi	*F_u, ksi	*D, in.	*t, in.
1	Willibald et al. (2006)	A2	72.2	78.3	6.572	0.191
2	Willibald et al. (2006)	B2	72.2	78.3	6.572	0.191
3	Willibald et al. (2006)	C1**	72.2	78.3	6.572	0.191
4	Willibald et al. (2006)	C2**	72.2	78.3	6.572	0.191
5	Yeomans (1993)	C-SEP-1	47.1	71.6	2.374	0.127
6	Yeomans (1993)	C-SEP-2	47.7	71.2	2.374	0.165
7	Yeomans (1993)	C-SEP-3	44.5	72.4	2.374	0.188
8	Yeomans (1993)	C-SEP-4	42.1	65.8	4.500	0.140
9	Yeomans (1993)	C-SEP-5	41.3	66.7	4.500	0.190
10	Yeomans (1993)	C-SEP-6	44.5	64.4	4.500	0.249
11	Cheng et al. (1998)	PWC1	57.0	65.1	3.962	0.246
12	Cheng et al. (1998)	PWC2	57.0	65.1	3.962	0.246
13	Cheng et al. (1998)	PWC3	57.0	65.1	3.962	0.246
14	Cheng et al. (1998)	PWC4	57.0	65.1	3.962	0.246
15	Cheng et al. (1998)	PWC5	54.4	65.4	3.967	0.177
16	Cheng et al. (1998)	PWC6	54.4	65.4	3.967	0.177
17	Cheng et al. (1998)	PWC7	54.4	65.4	3.967	0.177
18	Cheng et al. (1998)	SPEC1	50.5	62.5	8.568	0.291
19	Cheng et al. (1998)	SPEC2	50.5	62.5	8.568	0.291

*indicates measured value;
**indicates specimen with notched plate, see (C) in Figure 4-3

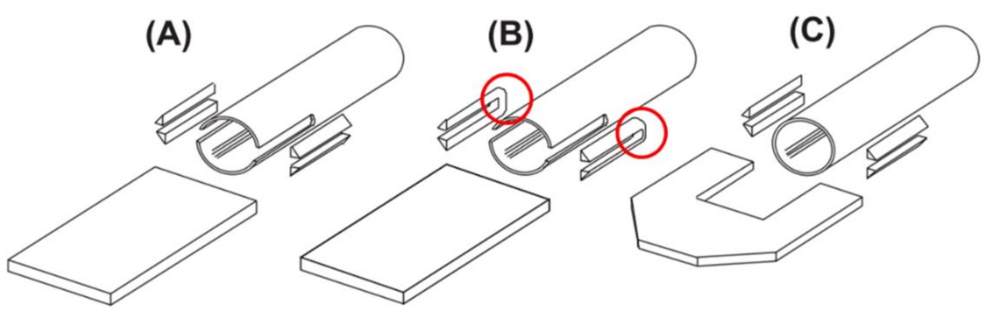


Figure 4-3: Detail of welds for HSS members from Willibald et al. (2006)

Table 4.3: Welded Round HSS Connection Weld Details

Index	Weld Size, in.	*l*, in	Cut Width, in.	Weld Return
1	0.390	7.560	1.060	No
2	0.351	8.190	1.060	Yes
3	0.551	6.478	1.060	No
4	0.551	7.677	1.060	No
5	-	3.150	0.390	Yes
6	-	3.150	0.484	Yes
7	-	2.953	0.602	Yes
8	-	5.906	0.602	Yes
9	-	5.906	0.787	Yes
10	-	5.709	0.787	Yes
11	0.234	6.630	0.390	Yes
12	0.234	6.630	0.390	Yes
13	0.234	6.630	0.390	Yes
14	0.234	6.630	0.390	Yes
15	0.195	5.850	0.390	Yes
16	0.195	5.850	0.390	Yes
17	0.195	5.850	0.390	Yes
18	0.390	13.455	0.780	Yes
19	0.390	10.725	0.780	Yes

The modeling of the round HSS tension members is different from the majority of the other connection types in the study because the tension members are slotted. For most of the other connection types, specimens are modeled with a plate being considered a ‘member’ in IDEA StatiCa. However, the HSS required additional operations which did not allow this approach. Therefore, a rigid wide-flange member was created, and the gusset plate was welded to this member and connected to the HSS on the other side as appropriate. An example for a typical round HSS is shown in Figure 4-4. The typical process for modelling the specimens is shown in Figure 4-5.

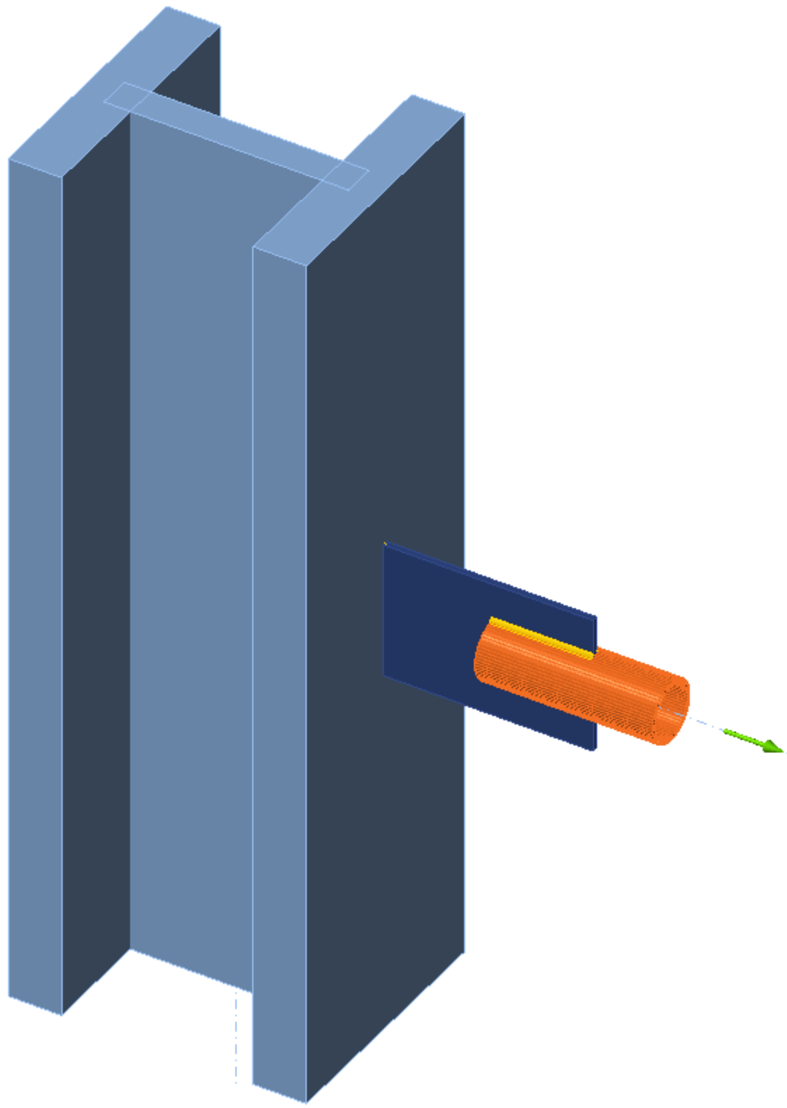


Figure 4-4: Typical Welded Round HSS Connection Model in IDEA StatiCa

4. Create members:
    a. Member 1: Rigid wide flange member
        i. Geometrical Type set to “Continuous”
        ii. Pitch of 90º
    b. Member 2: Round HSS tension member
        i. Geometrical Type set to “Ended”
        ii. β set to 180º
5. Create “Stiffening Plate” operation
    a. Select material based on specimen parameters
    b. Select thickness based on specimen parameters
    c. Input B1-width and B2-width based on specimen specific plate width
        i. B1, B2 are equal to half the plate width
    d. Input H1-height
        i. Unless otherwise noted in the text, the distance between the column and the HSS was always taken as 3.937 in. (100 mm)
        ii. H1 is calculated as (distance between the column and the HSS) + the weld length, l
    e. Origin is set to member with the member being the rigid wide flange (member 1)
    f. Plate set to “Top Flange 1”
    g. Type set to “Rib”
    h. Location set to “Rear”
    i. X-position set to 0 in. (0 mm)
    j. Rotation set to 90º
    k. Pitch set to 0.0º
    l. Welds set to “Butt welds”
        i. This weld is for connecting the plate to the rigid wide flange member
6. Create “Gusset Plate” operation
    a. Member is set to the HSS member
    b. Connected to is set to “Existing plate”
    c. Plate is set to the stiffening plate created in the previous step
    d. Gap is set the distance between the column and the HSS member (assumed to be 3.937 in. (100 mm) if not given)
    e. Alignment is set to “Center”
    f. Aligned plate is set to “No plate”
    g. Notched set to:
        i. “None” for all specimens except for indices 3 and 4
        ii. “Rectangle” for specimens with indices 3 and 4
    h. Connection type is set to “Welded”
    i. Welds are set to:
        i. “No weld” for all specimens except for specimens with indices 3 and 4
        ii. “Double Fillet Welds” with appropriate weld strength and size for specimens with notched plate, i.e., indices 3 and 4.
7. Create “Cut of Member” operation (necessary for all specimens except for those with notched plate, i.e., indices 3 and 4)
    a. Member is set to the HSS member
    b. Cut by is set to the stiffening plate created in step 3
    c. Cutting method is set to “Surface- all around”
    d. Offset is set to 0
    e. Welds are set to “Fillet Weld- Front Side” with the appropriate weld size and strength for each specimen

Figure 4-5: Modelling Process for Welded Round HSS Connections

4.2.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 4.4: AISC Calculation Results for Welded Round HSS Connections

Index	P_{YIELD}, kips	AISC 2016			AISC 2022		
		U	Controlling U Case	$P_{RUPTURE}$, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
1	276.1	0.723	Case 5	216.6	0.909	Case 5	272.1
2	276.1	0.745	Case 5	222.9	0.929	Case 5	278.0
3	276.1	N/A	N/A	N/A	0.855	Case 5	256.1
4	276.1	0.728	Case 5	217.8	0.913	Case 5	273.3
5	42.2	1.000	Case 5	64.1	0.942	Case 5	60.4
6	54.5	1.000	Case 5	81.3	0.950	Case 5	77.2
7	57.5	0.744	Case 5	69.6	0.949	Case 5	88.8
8	80.5	1.000	Case 5	126.1	0.933	Case 5	117.6
9	106.2	1.000	Case 5	171.4	0.942	Case 5	161.5
10	148.0	0.749	Case 5	160.3	0.936	Case 5	200.2
11	164.0	1.000	Case 5	187.3	0.965	Case 5	180.7
12	164.0	1.000	Case 5	187.3	0.965	Case 5	180.7
13	164.0	1.000	Case 5	187.3	0.965	Case 5	180.7
14	164.0	1.000	Case 5	187.3	0.965	Case 5	180.7
15	114.4	1.000	Case 5	137.6	0.947	Case 5	130.4
16	114.4	1.000	Case 5	137.6	0.947	Case 5	130.4
17	114.4	1.000	Case 5	137.6	0.947	Case 5	130.4
18	381.3	1.000	Case 5	472.3	0.956	Case 5	451.4
19	381.3	0.746	Case 5	352.2	0.911	Case 5	430.2

Table 4.5: Summary Strength Results for Welded Round HSS Connections

Index	Experimental		AISC 2016		AISC 2022		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{ASIC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
1	259.4	[2]	216.6	[2]	272.1	[2]	193.9	[3]
2	272.2	[2]	222.9	[2]	276.1	[1]	202.9	[3]
3	248.9	[2]	N/A	N/A	256.1	[2]	90.1	[3]
4	268.9	[2]	217.8	[2]	273.3	[2]	110.0	[3]
5	57.6	[2]	42.2	[1]	42.2	[1]	35.1	[3]
6	73.3	[2]	54.5	[1]	54.5	[1]	47.1	[3]
7	83.4	[2]	57.5	[1]	57.5	[1]	44.7	[3]
8	117.4	[2]	80.5	[1]	80.5	[1]	68.4	[3]
9	146.6	[2]	106.2	[1]	106.2	[1]	86.1	[3]
10	178.7	[2]	148.0	[1]	148.0	[1]	109.2	[3]
11	186.5	[2]	164.0	[1]	164.0	[1]	124.2	[3]
12	195.3	[1]	164.0	[1]	164.0	[1]	124.2	[3]
13	191.0	[1]	164.0	[1]	164.0	[1]	124.2	[3]
14	196.7	[1]	164.0	[1]	164.0	[1]	124.2	[3]
15	144.9	[1]	114.4	[1]	114.4	[1]	71.3	[3]
16	142.5	[1]	114.4	[1]	114.4	[1]	71.3	[3]
17	141.9	[1]	114.4	[1]	114.4	[1]	71.3	[3]
18	485.6	[1]	381.3	[1]	381.3	[1]	248.0	[3]
19	480.8	[2]	352.2	[2]	381.3	[1]	159.9	[3]

[1] tensile yield; [2] tensile rupture; [3] tension member plastic strain limit

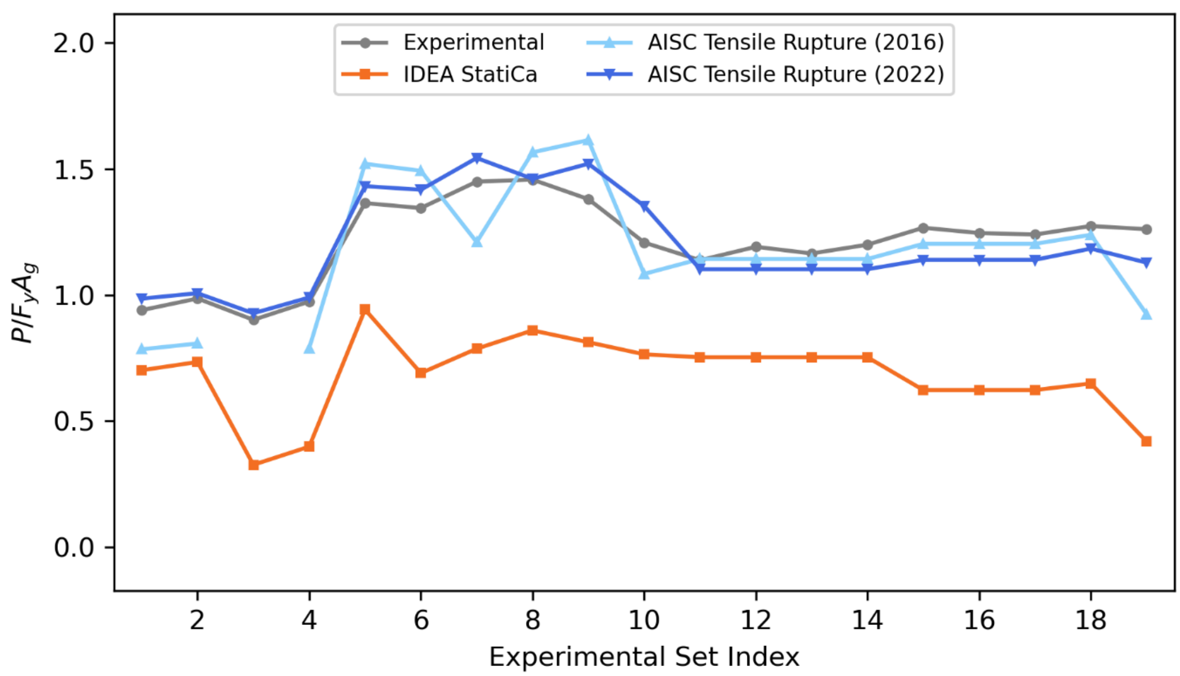


Figure 4-6: Normalized Strength Results for Welded Round HSS Connections

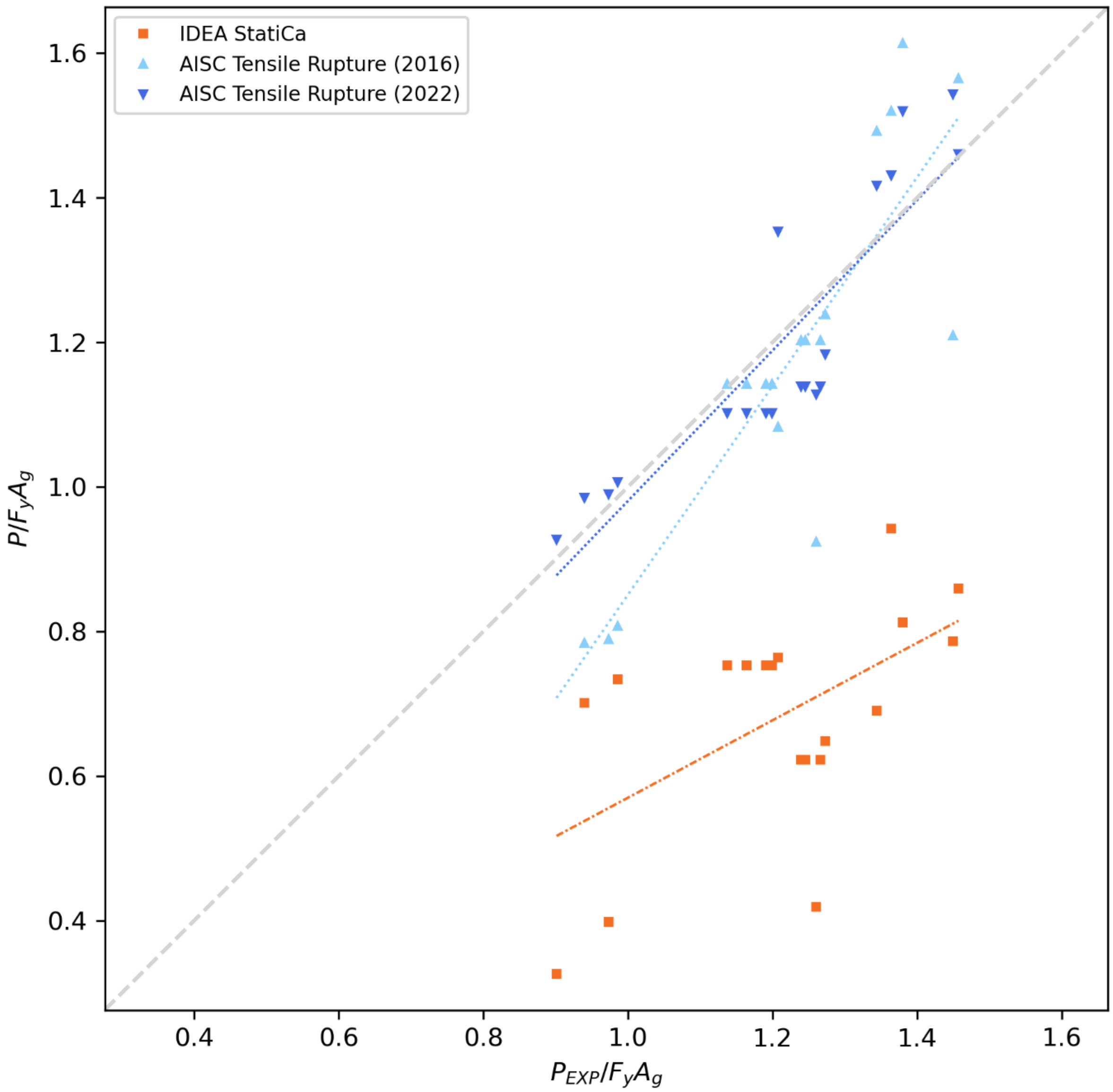


Figure 4-7: Normalized Strength vs. Normalized Experimental Strength for Welded Round HSS Connections

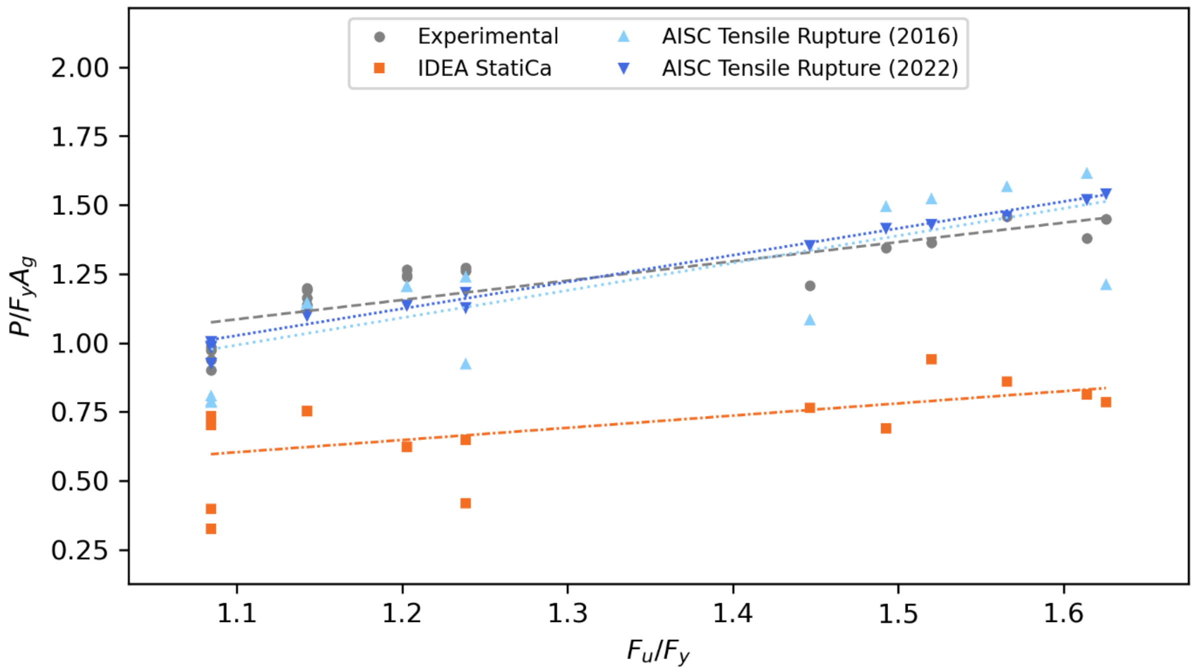


Figure 4-8: Normalized Strength vs. Material Strength Ratio for Welded Round HSS Connections

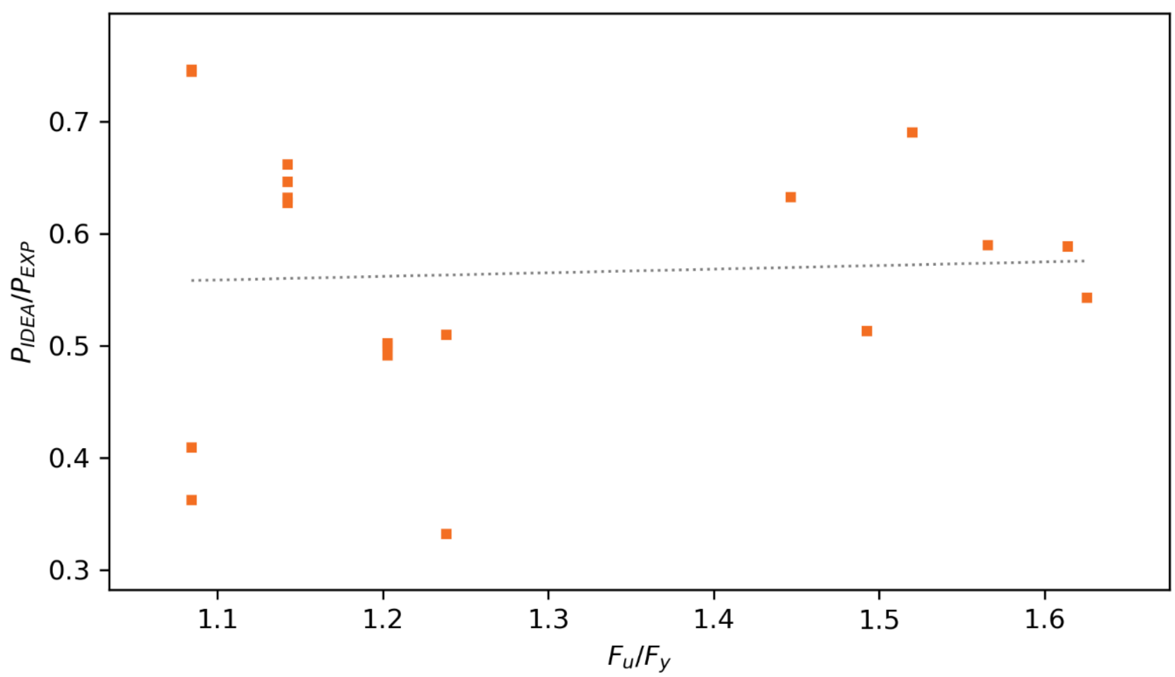


Figure 4-9: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Welded Round HSS Connections

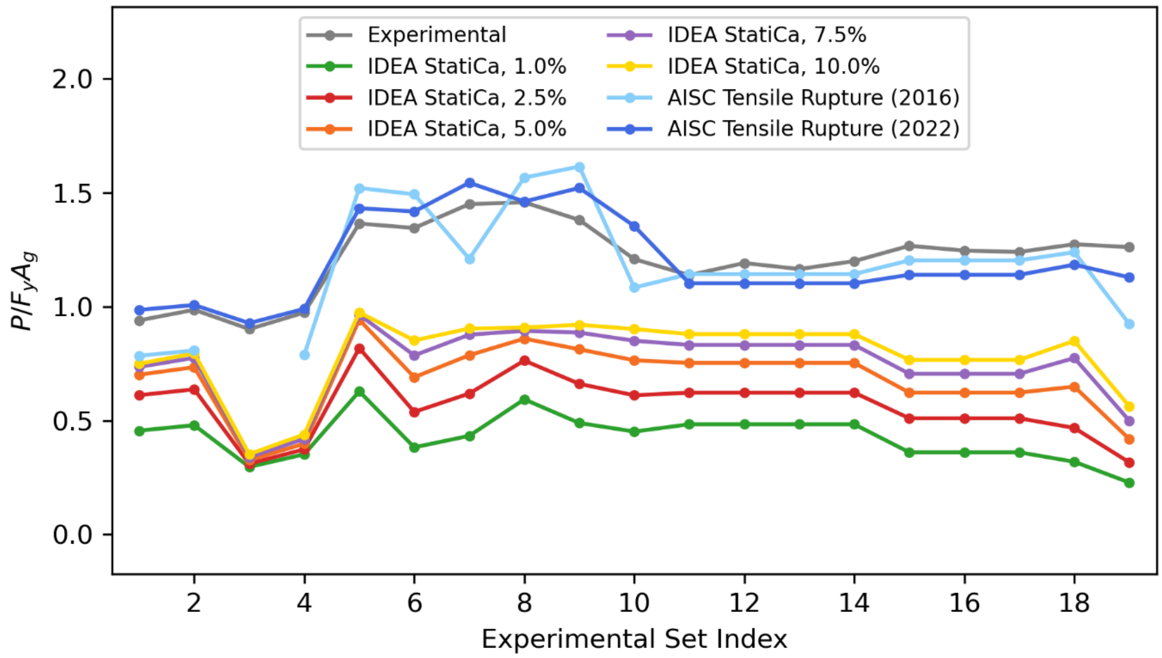


Figure 4-10: Normalized Strength Results for Welded Round HSS Connections Including Various Plastic Strain Limits for IDEA StatiCa

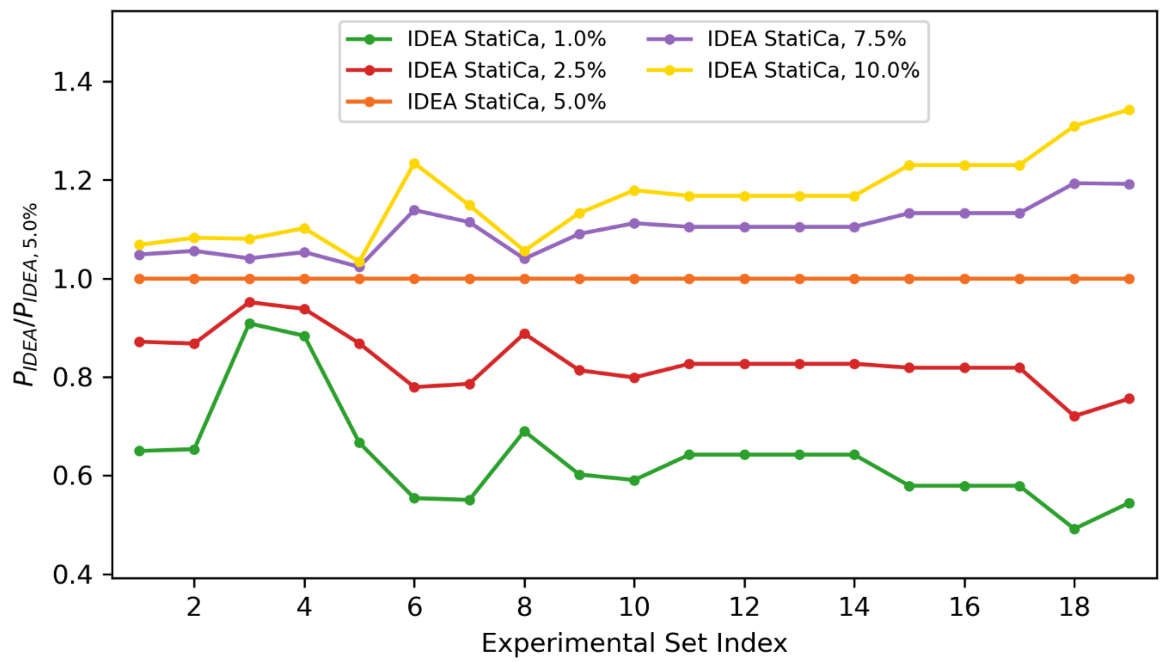


Figure 4-11: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Welded Round HSS Connections

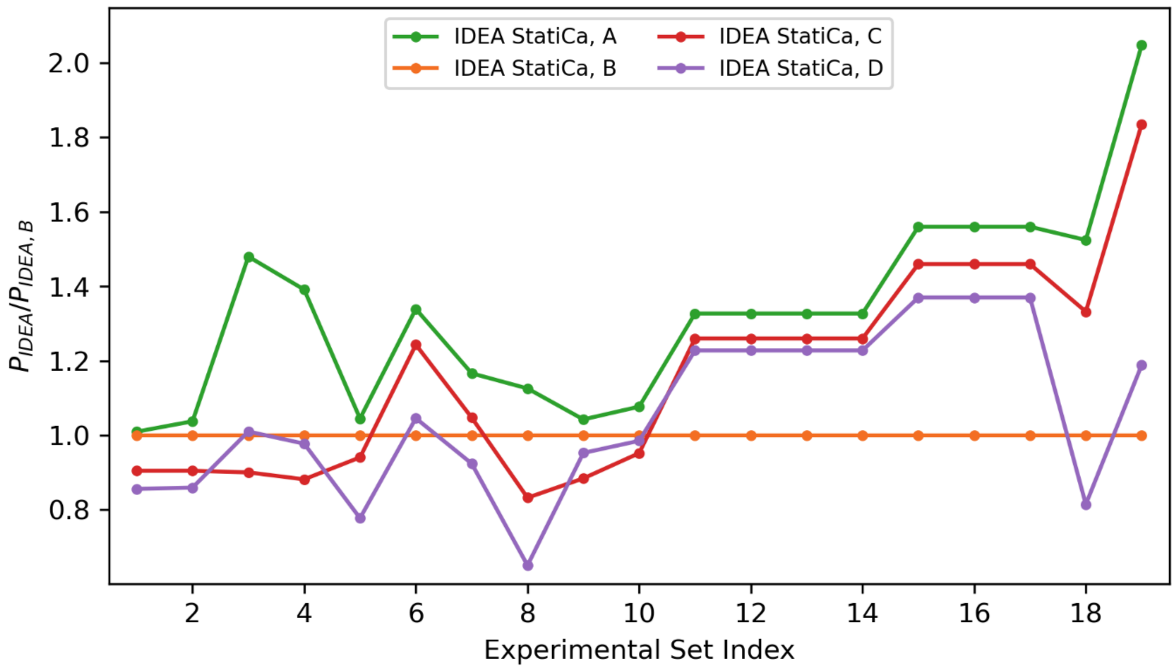


Figure 4-12: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Welded Round HSS Connections

Table 4.6: Summary Statistics of the Test-to-Predicted Ratio for Welded Round HSS Connections

Test-to-Predicted Ratio	Average	Standard Deviation	Coefficient of Variation
$P_{EXP}/P_{RUPTURE}$ (AISC 2016)	1.065	0.135	0.127
$P_{EXP}/P_{RUPTURE}$ (AISC 2022)	1.014	0.073	0.072
P_{EXP}/P_{IDEA}	1.860	0.458	0.246

4.2.3 Discussion

Most of the experimental specimens exhibited a tensile rupture failure, however as noted previously these failures most often occurred after significant yielding. As a result, tensile yield controlled the AISC strength calculations for most specimens (Table 4.5).

The maximum permitted applied load for IDEA StatiCa, P_{IDEA}, produced generally conservative (i.e., lower) results for the experimental set compared to the AISC *Specification* strength equations and the experimental results (Figure 4-6). The most extreme case is found in experimental index specimen 3 with a 65% lower strength than the AISC *Specification* (2022a) equations for tensile rupture. When compared to the experimental results, the largest difference was found in experimental index specimen 19 with a 67% difference from P_{IDEA} to P_{EXP}.

The results in Figure 4-7 show good agreement between the AISC tensile rupture strength and the experimental results, especially for the new 2022 provisions. The IDEA StatiCa results are less well correlated to the experimental results, however, the IDEA StatiCa results include the limit state of tensile yielding whereas the experimental results and AISC results in this figure do not. Nonetheless, there is a clear trend of the experimental specimens with higher normalized experimental strength seeing a higher normalized IDEA StatiCa strength.

With respect to the material strength ratio, F_u/F_y, all the strengths when normalized by F_yA_g are seen to increase with increases in F_u/F_y (Figure 4-8). This observation is as expected for the experimental and AISC tensile rupture strengths. The positive trend for the IDEA StatiCa results is unexpected since F_u is not utilized in the model and may simply be the result of variability among the data. The ratio P_{IDEA}/P_{EXP} has no discernable trend with F_u/F_y (Figure 4-9).

The variation of P_{IDEA} with the plastic strain limit was as expected with P_{IDEA} decreasing as the plastic strain limit decreases and vice versa (Figure 4-10 and Figure 4-11). The variation of P_{IDEA} with the plastic strain limit was smallest for specimens 3 and 4. These specimens also has notably smaller strengths for IDEA StatiCa than the experimental and AISC calculated strengths. These two specimens, as indicated by Table 4.2, have notched plates (as opposed to a slotted HSS). One potential reason for the decreased sensitivity to plastic strain limit for these specimens is that they have a more severe concentration of stress and strain in the connection. A visual of the stress concentration from IDEA StatiCa for specimen 3 is provided in Figure 4-13.

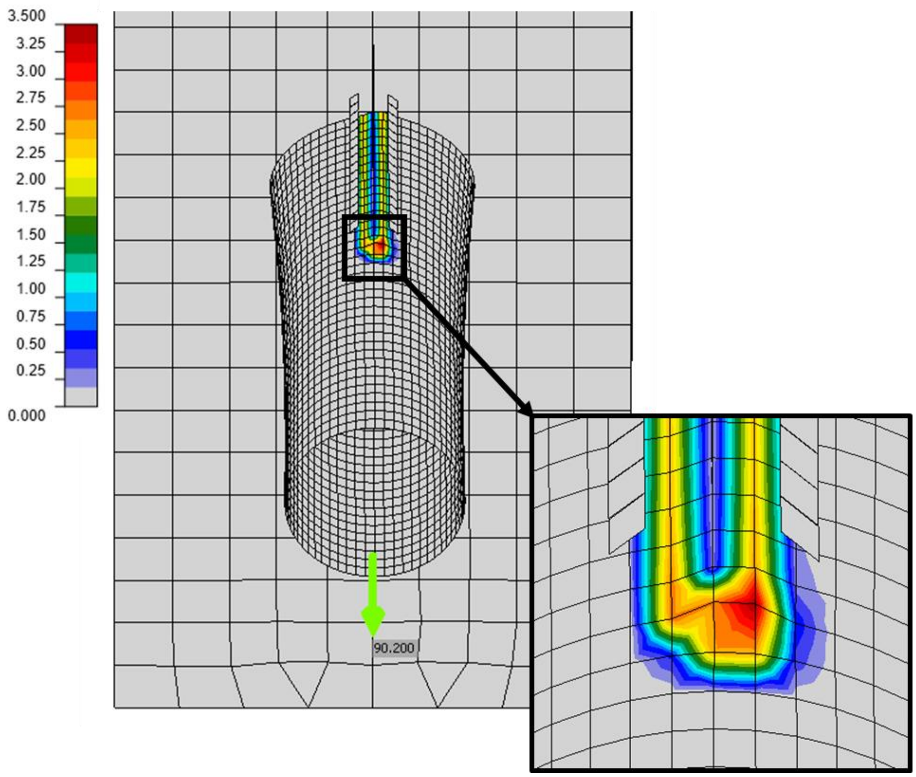


Figure 4-13: Round HSS experimental set index 3 plastic strain concentration, deformation scale: 10

The variation of P_{IDEA} with the mesh parameters was not as expected. Generally, for finite element analysis solutions, the results should follow a consistent pattern with the difference in results decreasing as the mesh is refined. However, this was not observed for this experimental set of specimens. Further investigation is required to determine why this pattern is not observed.

4.3 Reliability Analysis

4.3.1 Description of Reliability Set

The reliability analyses performed in this work applies to specific connections. To ensure broadly applicable results, a set of connections is defined with varying parameters. In contrast to the experimental set, the connections in the reliability set are not based on physical connections or test specimens. Parameters for the round HSS reliability set are outlined in Table 4.7. They include variations in diameter to thickness ratio, weld length to diameter ratio, gusset plate thickness, material grade, and cross-section thickness. It was desired to vary the material strength ratio (F_u/F_y); thus, material grades A53 Gr. B and A500 Gr. C were selected. For consistency with standards, specimens with A53 Gr. B material were selected to be pipes, and the specimens with A500 Gr. C were selected to be HSS members. It should be noted specimens with specimen indices 1-12 have a different set of varying thicknesses than those with specimen indices 13-24 for the reasons just discussed.

The modeling of the round HSS reliability set was, in general, the same as the experimental set previously described. There were a few differences regarding geometric and material properties. The reliability set is based on nominal properties and modeled as such. As previously mentioned, the round HSS contains ‘Pipe’ and ‘HSS’ cross-section types. The round HSS members selected in IDEA StatiCa were from the ‘PIPE (AISC 15.0 – A53)’ category for the pipe cross-section type and ‘HSS (AISC 15.0 – A500, A502, A618, A847)’ category for the HSS cross-section. The IDEA StatiCa models included reduced design thicknesses. Additionally, the material grade A500 Gr. C has a few options in IDEA StatiCa. The material ‘A500, Gr. C, shaped’ was selected for the reliability set to match current ASTM standards (2018).

Table 4.7: Welded Round HSS Connection Reliability Set Parameters

Index	Cross-Section Type	*D*, in.	*t*, in.	Weld Length, in.	Gusset Plate Thickness, in.	Material Grade	F_y, ksi	F_u, ksi
1	Pipe	6	0.864	6	0.5	A53 Gr. B	35	60
2	Pipe	6	0.432	6	0.5	A53 Gr. B	35	60
3	Pipe	6	0.280	6	0.5	A53 Gr. B	35	60
4	Pipe	6	0.864	12	0.5	A53 Gr. B	35	60
5	Pipe	6	0.432	12	0.5	A53 Gr. B	35	60
6	Pipe	6	0.280	12	0.5	A53 Gr. B	35	60
7	Pipe	6	0.864	6	1.0	A53 Gr. B	35	60
8	Pipe	6	0.432	6	1.0	A53 Gr. B	35	60
9	Pipe	6	0.280	6	1.0	A53 Gr. B	35	60
10	Pipe	6	0.864	12	1.0	A53 Gr. B	35	60
11	Pipe	6	0.432	12	1.0	A53 Gr. B	35	60
12	Pipe	6	0.280	12	1.0	A53 Gr. B	35	60
13	HSS	6	0.500	6	0.5	A500 Gr. C	50	62
14	HSS	6	0.250	6	0.5	A500 Gr. C	50	62
15	HSS	6	0.125	6	0.5	A500 Gr. C	50	62
16	HSS	6	0.500	12	0.5	A500 Gr. C	50	62
17	HSS	6	0.250	12	0.5	A500 Gr. C	50	62
18	HSS	6	0.125	12	0.5	A500 Gr. C	50	62
19	HSS	6	0.500	6	1.0	A500 Gr. C	50	62
20	HSS	6	0.250	6	1.0	A500 Gr. C	50	62
21	HSS	6	0.125	6	1.0	A500 Gr. C	50	62
22	HSS	6	0.500	12	1.0	A500 Gr. C	50	62
23	HSS	6	0.250	12	1.0	A500 Gr. C	50	62
24	HSS	6	0.125	12	1.0	A500 Gr. C	50	62

4.3.2 Results

Strength results for the reliability set of specimens are provided in Table 4.8. The nominal tensile yield and nominal tensile rupture strengths according to the AISC *Specification* (2022a) are provided, as well as the design strengths for each. The design strengths (including ϕ-factors) were used in the reliability analysis. Additionally, the maximum permitted applied loads from IDEA StatiCa are provided using all default settings (plastic strain limit of 5.0%, mesh parameter set 'B', and applicable resistance factors). These strengths were directly used in the reliability analysis for determining the nominal dead and live loads and ultimately the reliability index, β.

The maximum permitted applied load in IDEA StatiCa, P_{IDEA}, exceeded the design strength according to the AISC *Specification* equations for only a few specimens and only by small margins. The most extreme case resulted in a 3.5% larger strength of P_{IDEA} than ϕP_n, where ϕP_n is the minimum of ϕP_{YIELD} and $\phi P_{RUPTURE}$. The most conservative case was the specimen with reliability set specimen index 1 resulting in a P_{IDEA} 41% less than the corresponding AISC design strength. Overall, these results compare well with IDEA StatiCa being mostly on the conservative side.

Table 4.8: Summary Strength Results for Welded Round HSS Connection Reliability Set

Index	AISC 2022				IDEA StatiCa
	P_{YIELD}, kips	$P_{RUPTURE}$, kips	ϕP_{YIELD}, kips	$\phi P_{RUPTURE}$, kips	P_{IDEA}, kips
1	514.5	685.6	463.1	514.2	273.1
2	274.1	366.5	246.6	274.9	151.1
3	182.0	243.7	163.8	182.8	99.8
4	514.5	816.0	463.1	612.0	461.7
5	274.1	436.2	246.6	327.1	248.1
6	182.0	290.0	163.8	217.5	162.2
7	514.5	678.3	463.1	508.8	295.8
8	274.1	364.0	246.6	273.0	157.3
9	182.0	242.4	163.8	181.8	104.4
10	514.5	772.9	463.1	579.7	438.9
11	274.1	414.7	246.6	311.0	236.0
12	182.0	276.2	163.8	207.2	156.6
13	404.5	388.8	364.1	291.6	229.1
14	211.0	203.3	189.9	152.5	117.0
15	107.0	103.2	96.3	77.4	58.7
16	404.5	462.7	364.1	347.0	358.6
17	211.0	241.9	189.9	181.5	186.9
18	107.0	122.8	96.3	92.1	84.9
19	404.5	383.4	364.1	287.6	233.5
20	211.0	201.0	189.9	150.8	122.1
21	107.0	102.2	96.3	76.6	61.3
22	404.5	436.9	364.1	327.6	339.1
23	211.0	229.0	189.9	171.8	176.2
24	107.0	116.4	96.3	87.3	72.8

The results from the reliability analysis for the round HSS specimen set are presented in Figure 4-14. For specimens 1, 2, 3, 7, 8, 9, 13, 14, and 15, and design by IDEA StatiCa, no failures were noted in 1,000,000 simulations, resulting in a large reliability index, β. The minimum β for IDEA StatiCa was 3.50. The AISC *Specification* (2022a) equations resulted in a β range of 3.63 to 4.42. These results indicate that, for this set of specimens and parameters, IDEA StatiCa provides a comparable or sometimes higher level of reliability when compared to the AISC *Specification*.

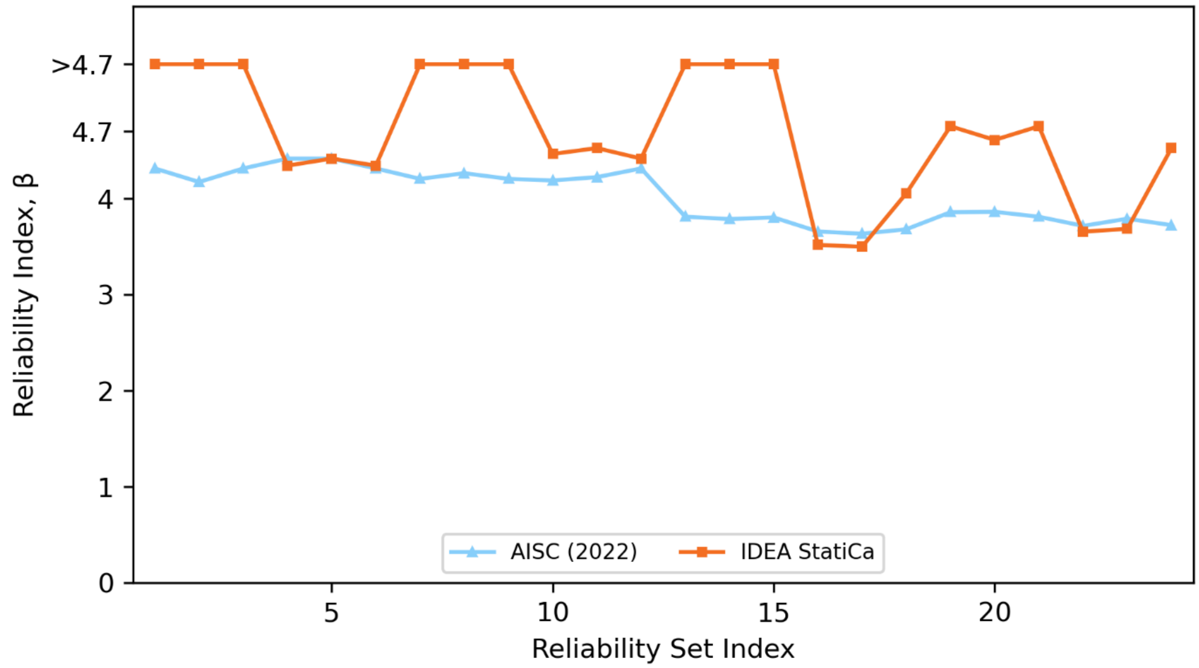


Figure 4-14: Reliability Index for Welded Round HSS Connections

Chapter 5:
WELDED RECTANGULAR HSS

5.1 Description of Connection

Square and rectangular HSS tension members welded to a gusset plate are evaluated in this chapter. A schematic of the connection with relevant terminology is provided in Figure 5-1. The typical connection includes a slotted rectangular HSS tension member welded to a gusset plate. For simplicity, square HSS are referred to as rectangular HSS. The dimension *H* is always parallel to the gusset plate with the dimension *B* being perpendicular to the gusset plate.

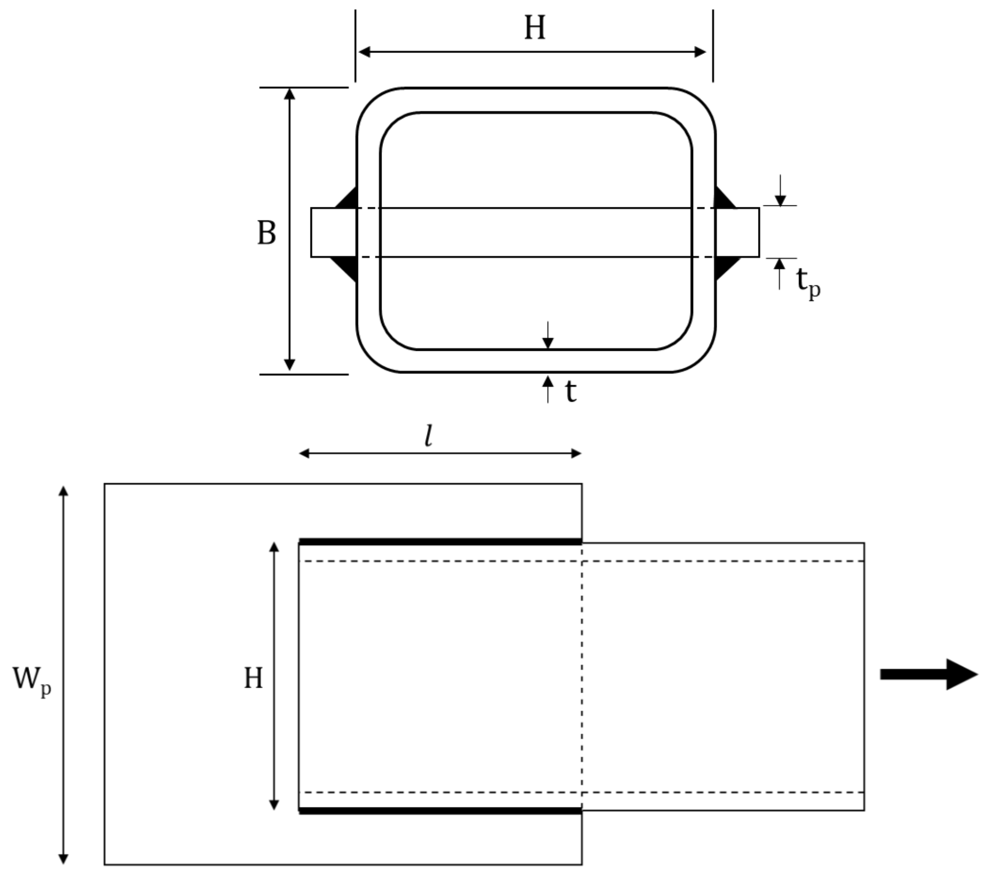


Figure 5-1: Schematic of Welded Rectangular HSS Connection

Similar to round HSS, the shear lag factor from the AISC *Specification* varies significantly from the 2016 edition of the standard (AISC 2016) to the 2022 edition (AISC 2022a), based on the work of Dowswell (2021). The provisions of both editions are evaluated in this work. The 2016 shear lag factor calculations include the use of Case 6 from Table D3.1 of the AISC *Specification* shown in Figure 3-3. There are two subcategories for Case 6: rectangular HSS with a single concentric gusset plate and rectangular HSS with two side gusset plates. All rectangular HSS specimens included in this study consist of only a single concentric gusset plate. The case with two side gusset plates was not evaluated in this work. The applicable equations for the shear lag factor are as follows:

$$l \geq H, \quad U = 1 - \frac{\bar{x}}{l} \tag{5-1}$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B + H)} \tag{5-2}$$

where l is the length of the weld, $\bar{x}$ is the eccentricity of the connection, and geometric dimensions B and H are consistent with Figure 5-1. Note that the shear lag factor is undefined when $l < H$.

In the 2022 edition of the AISC *Specification*, the applicable case is number 5. This case contains two subcategories: round HSS and rectangular HSS. The equation for a concentrically loaded rectangular HSS is as follows:

$$U = 1 - \frac{\bar{x}}{l} \tag{5-3}$$

$$\bar{x} = b - \frac{2b^2 + tH - 2t^2}{2H + 4b - 4t} \tag{5-4}$$

where l is the weld length, $\bar{x}$ is the eccentricity of the connection, and geometric dimensions b, H, and t are defined in Figure 5-2. Unlike the 2016 edition of the standard, the shear lag factor is defined for all weld lengths, l, even when $l < H$.

Some of the connections in the experimental database have a weld return. For the specimens with a weld return, the net area was taken as the gross area. For the specimens without a weld return, the net area was taken as the gross area minus the removed area for the slot on both sides.

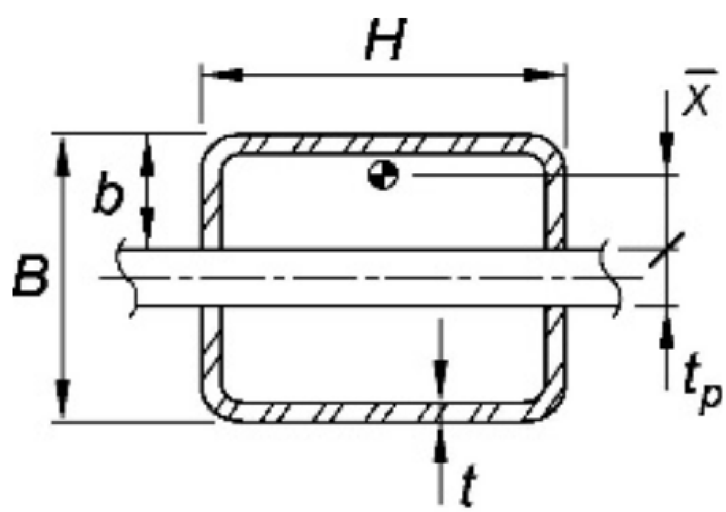


Figure 5-2: Figure from AISC *Specification* (2022) defining rectangular HSS parameters

5.2 Comparison to Experimental Results

5.2.1 Description of Experimental Specimens

A total of 46 rectangular HSS specimens from 3 references were identified for evaluation in this work as detailed in Table 5.1. Some of the specimens described in these references were not included. Zhao et al. (2008) reported a total of 30 specimens, but specimen R3 was not included in this work since it experienced a weld failure. Korol (1996) reported a total of 18 specimens, but specimens 4A, 4B, 5B, 6A, 6B, 7A, 7B, 8A, 8B, 9A, and 9B were not included in this work since they experienced a tearout failure. Yeomans (1993) reported a total of 18 rectangular HSS specimens, however 6 specimens failed in the gusset plate (S-SEP-1, R-SEP-1, R-SEP-2. R-SEP-4, R-SEP-11, and R-SEP-12), one specimen experienced bolt failure in the test setup (R-SEP-6), and one test did not complete (R-SEP-7). These 8 specimens were not included in this work.

Table 5.1: Count of Welded Rectangular HSS Connections by Reference

Reference	**Specimen Count**
Zhao et al. (2008)	29
Korol (1996)	7
Yeomans (1993)	10
Total	**46**

A detailed description of each specimen is provided in Table 5.2. Korol (1996) reported that the tubes were made of CSA 350W steel, but did not report measured yield and ultimate strengths. For this study, a yield strength of F_y = 50 ksi and ultimate strength of F_u = 65 ksi was used. Dowswell (2021) made the same assumption for the tests reported by Korol (1996). Yeomans (1993) did not report a weld size, but noted that the welds were designed not to fail. Butt welds were used in IDEA StatiCa for Yeomans specimens to avoid definition of weld size (butt welds are not modeled in IDEA StatiCa, rather the connecting elements are tied to each other directly). E70XX welds were assumed for all specimens. This is consistent with the weld strength reported by Zhao et al. (2008). The other studies did not report a specific weld strength. The ‘Weld Return’ column is the indication of a weld across the thickness of the gusset plate. The weld return was not included in IDEA StatiCa. For the AISC *Specification* calculations, the gross area of the sections was calculated based on measured geometric properties. Additionally, the inner radius of the specimens was taken as the thickness, unless otherwise noted.

Table 5.2: Welded Rectangular HSS Connection Experimental Specimen Parameters

Index	Reference	Specimen	*F_y, ksi	*F_u, ksi	*B, in.	*H, in,	*t, in.	Weld Size, in.	l, in.	Weld Return
1	Zhao (2008)	RL5G05P16	55.1	65.0	5.005	2.030	0.176	0.315	7.689	No
2	Zhao (2008)	RS5G05P16	55.1	65.0	2.024	5.014	0.176	0.315	7.677	No
3	Zhao (2008)	SM5G05P16	58.6	70.3	3.411	3.535	0.174	0.315	7.673	No
4	Zhao (2008)	SM5G05P16R	58.6	70.3	3.521	3.527	0.174	0.315	7.752	No
5	Zhao (2008)	RL4G05P16	55.1	65.0	5.011	2.032	0.176	0.315	6.122	No
6	Zhao (2008)	RS4G05P16	55.1	65.0	2.029	5.010	0.177	0.315	6.126	No
7	Zhao (2008)	SM4G05P16	58.6	70.3	3.520	3.533	0.173	0.315	6.146	No
8	Zhao (2008)	SM4G05P16R	58.6	70.3	3.527	3.522	0.174	0.315	6.185	No
9	Zhao (2008)	RL3G05P16	55.1	65.0	4.999	2.023	0.176	0.315	4.547	No
10	Zhao (2008)	RS3G05P16	55.1	65.0	2.022	5.009	0.178	0.315	4.626	No
11	Zhao (2008)	SM3G05P16	58.6	70.3	3.532	3.531	0.174	0.315	4.543	No
12	Zhao (2008)	SM3G05P16R	58.6	70.3	3.524	3.523	0.174	0.315	4.559	No
13	Zhao (2008)	SM3G05P12	58.6	70.3	3.519	3.524	0.174	0.315	4.697	No
14	Zhao (2008)	SM3G05P12R	58.6	70.3	3.523	3.525	0.174	0.315	4.638	No
15	Zhao (2008)	SM5G05P12	58.6	70.3	3.527	3.523	0.174	0.315	7.929	No
16	Zhao (2008)	SM5G05P12R	58.6	70.3	3.537	3.526	0.174	0.315	7.933	No
17	Zhao (2008)	SM3G05P20	58.6	70.3	3.497	3.502	0.174	0.315	4.500	No
18	Zhao (2008)	SM3G05P20R	58.6	70.3	3.519	3.548	0.175	0.315	4.480	No
19	Zhao (2008)	SM5G05P20	58.6	70.3	3.528	3.531	0.174	0.315	7.504	No
20	Zhao (2008)	SM5G05P20R	58.6	70.3	3.517	3.527	0.174	0.315	7.484	No
21	Zhao (2008)	SM3G25P16	58.6	70.3	3.519	3.528	0.174	0.315	4.618	No
22	Zhao (2008)	SM3G25P16R	58.6	70.3	3.516	3.537	0.174	0.315	4.598	No
23	Zhao (2008)	SM3G50P16	58.6	70.3	3.511	3.530	0.174	0.315	4.587	No
24	Zhao (2008)	SM3G50P16R	58.6	70.3	3.509	3.527	0.174	0.315	4.559	No
25	Zhao (2008)	SM5G50P16	58.6	70.3	3.516	3.531	0.174	0.315	7.717	No
26	Zhao (2008)	SM5G50P16R	58.6	70.3	3.515	3.532	0.173	0.315	7.689	No
27	Zhao (2008)	S4	53.7	63.8	3.517	3.491	0.174	0.433	6.272	Yes
28	Zhao (2008)	S4R	53.7	63.8	3.510	3.495	0.174	0.413	6.335	Yes
29	Zhao (2008)	S3	53.7	63.8	3.510	3.496	0.175	0.433	4.413	Yes
30	Korol (1996)	1A	NP	NP	1.969	4.921	0.252	0.315	6.299	No
31	Korol (1996)	1B	NP	NP	1.969	4.921	0.244	0.315	6.181	No
32	Korol (1996)	2A	NP	NP	3.465	3.465	0.242	0.315	6.181	No
33	Korol (1996)	2B	NP	NP	3.465	3.465	0.252	0.315	6.378	No
34	Korol (1996)	3A	NP	NP	4.961	1.969	0.242	0.315	6.142	No
35	Korol (1996)	3B	NP	NP	4.961	1.969	0.246	0.315	6.339	No
36	Korol (1996)	5A	NP	NP	3.504	3.504	0.236	0.315	3.858	No
37	Yeomans (1993)	S-SEP-2	54.1	66.7	1.969	1.969	0.134	NP	3.150	Yes
38	Yeomans (1993)	S-SEP-3	52.5	69.9	1.969	1.969	0.244	NP	2.953	Yes
39	Yeomans (1993)	S-SEP-4	44.7	63.7	3.543	3.543	0.146	NP	5.906	Yes
40	Yeomans (1993)	S-SEP-5	59.0	72.8	3.543	3.543	0.205	NP	5.906	Yes
41	Yeomans (1993)	S-SEP-6	51.1	74.0	3.543	3.543	0.241	NP	5.709	Yes
42	Yeomans (1993)	R-SEP-3	54.8	67.3	2.362	1.575	0.262	NP	2.953	Yes
43	Yeomans (1993)	R-SEP-5	63.2	76.9	1.575	2.362	0.160	NP	3.150	Yes
44	Yeomans (1993)	R-SEP-8	61.1	77.4	4.724	2.362	0.215	NP	5.709	Yes
45	Yeomans (1993)	R-SEP-9	54.1	70.8	4.724	2.362	0.253	NP	5.709	Yes
46	Yeomans (1993)	R-SEP-10	52.2	65.8	2.362	4.724	0.140	NP	5.906	Yes

* indicates measured value; NP: not provided

In general, the modeling of the rectangular HSS tension members was identical to the modeling of the round HSS. As mentioned in the previous chapter, a rigid wide-flange member was created for this analysis and the gusset plate was welded to this member and connected to the HSS on the other side of the gusset plate as appropriate. An example for a typical rectangular HSS model is shown in Figure 5-3. The typical process for modelling the specimens is shown in Figure 5-4.

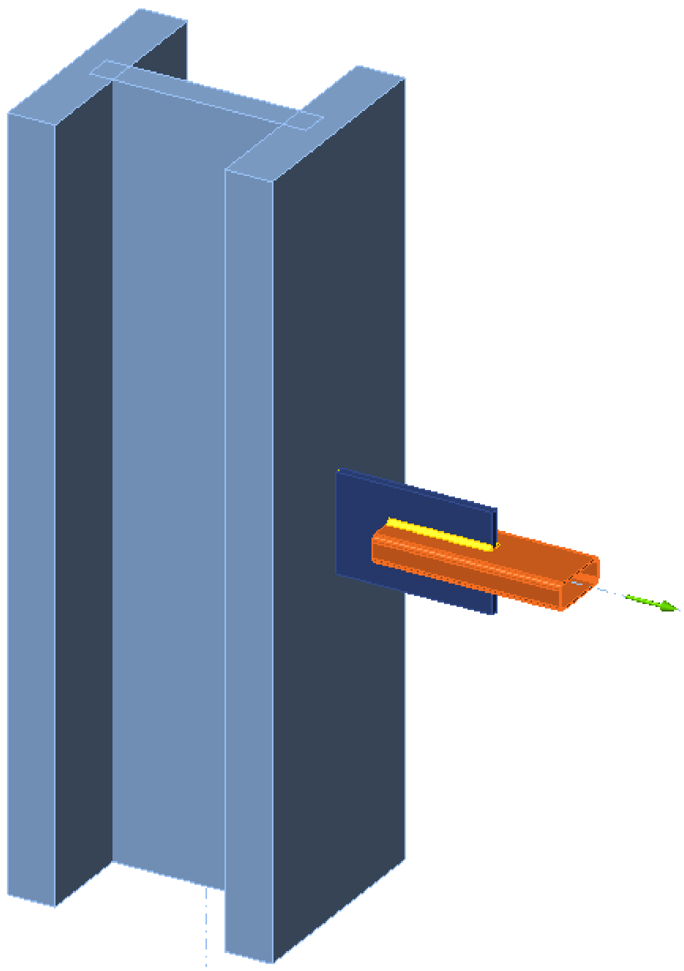


Figure 5-3: Typical Welded Rectangular HSS Connection Model in IDEA StatiCa

4. Create members:
   a. Member 1: Rigid wide flange member
      i. Geometrical Type set to "Continuous"
      ii. Pitch of 90º
   b. Member 2: Round HSS tension member
      i. Geometrical Type set to "Ended"
      ii. β set to 180º
5. Create "Stiffening Plate" operation
   a. Select material based on specimen parameters
   b. Select thickness based on specimen parameters
   c. Input B1-width and B2-width based on specimen specific plate width
      i. B1, B2 are equal to half the plate width
   d. Input H1-height
      i. Unless otherwise noted in the text, the distance between the column and the HSS was always taken as 3.937" (100 mm). Note: Korol (1996) indicated a gap of 2.756" (70 mm) between test setup (bolts) and HSS member. Therefore, 2.756" was used as the gap for all Korol (1996) specimens.
      ii. H1 is calculated as (distance between the column and the HSS) plus the weld length, l
   e. Origin is set to member with the member being the rigid wide flange (member 1)
   f. Plate set to "Top Flange 1"
   g. Type set to "Rib"
   h. Location set to "Rear"
   i. X-position set to 0.0 in. (0 mm)
   j. Rotation set to 90º
   k. Pitch set to 0.0º
   l. Welds set to "Butt welds"
      i. This weld is for connecting the plate to the rigid wide flange member
6. Create "Gusset Plate" operation
   a. Member is set to the HSS member
   b. Connected to is set to "Existing plate"
   c. Plate is set to the stiffening plate created in the previous step
   d. Gap is set the distance between the column and the HSS member (assumed to be 3.937 in. (100 mm) if not given)
   e. Alignment is set to "Center"
   f. Aligned plate is set to "No plate"
   g. Notched set to: "None"
   h. Connection type is set to "Welded"
   i. Welds are set to: "No weld"
7. Create "Cut of Member" operation
   a. Member is set to the HSS member
   b. Cut by is set to the stiffening plate created in step 3
   c. Cutting method is set to "Surface- all around"
   d. Offset is set to 0
   e. Welds are set to "Fillet Weld- Front Side" with the appropriate weld size and strength for each specimen

Figure 5-4: Modelling Process for Welded Rectangular HSS Connections

5.2.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 5.3: AISC Calculation Results for Welded Rectangular HSS Connections

Index	P_{YIELD}, kips	AISC 2016			AISC 2022		
		U	Controlling U Case	$P_{RUPTURE}$, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
1	129.6	0.790	Case 6	108.5	0.823	Case 5	113.1
2	129.4	0.887	Case 6	120.4	0.934	Case 5	126.8
3	134.8	0.832	Case 6	120.4	0.870	Case 5	125.9
4	136.6	0.830	Case 6	122.1	0.866	Case 5	127.5
5	129.8	0.736	Case 6	101.6	0.777	Case 5	107.2
6	130.3	0.858	Case 6	118.0	0.915	Case 5	125.8
7	135.9	0.785	Case 6	115.1	0.831	Case 5	121.8
8	136.4	0.786	Case 6	114.5	0.835	Case 5	121.6
9	129.1	0.646	Case 6	88.4	0.701	Case 5	96.0
10	130.7	N/A	N/A	N/A	0.889	Case 5	122.3
11	136.6	0.708	Case 6	104.3	0.771	Case 5	113.5
12	136.6	0.710	Case 6	104.7	0.772	Case 5	113.8
13	136.5	0.719	Case 6	108.3	0.770	Case 5	115.9
14	136.6	0.715	Case 6	107.9	0.766	Case 5	115.5
15	137.0	0.833	Case 6	125.4	0.864	Case 5	130.0
16	136.7	0.833	Case 6	125.4	0.863	Case 5	130.0
17	135.6	0.709	Case 6	101.2	0.781	Case 5	111.6
18	137.9	0.705	Case 6	102.6	0.778	Case 5	113.2
19	136.9	0.824	Case 6	118.6	0.868	Case 5	124.9
20	136.9	0.824	Case 6	118.8	0.867	Case 5	125.1
21	136.6	0.714	Case 6	105.1	0.776	Case 5	114.2
22	136.7	0.713	Case 6	105.2	0.774	Case 5	114.2
23	136.5	0.713	Case 6	105.9	0.770	Case 5	114.4
24	136.7	0.711	Case 6	105.3	0.771	Case 5	114.2
25	136.3	0.829	Case 6	121.6	0.866	Case 5	127.1
26	136.0	0.828	Case 6	121.8	0.865	Case 5	127.1
27	124.4	0.790	Case 6	116.8	0.833	Case 5	123.2
28	124.6	0.792	Case 6	117.4	0.835	Case 5	123.8
29	124.9	0.702	Case 6	104.3	0.764	Case 5	113.4
30	160.9	0.866	Case 6	162.2	0.924	Case 5	173.0
31	156.3	0.863	Case 6	157.1	0.922	Case 5	167.7
32	156.1	0.790	Case 6	143.6	0.839	Case 5	152.5
33	161.9	0.796	Case 6	150.2	0.844	Case 5	159.2
34	156.1	0.741	Case 6	134.7	0.785	Case 5	142.8
35	158.4	0.749	Case 6	138.2	0.792	Case 5	146.2
36	154.4	0.659	Case 6	118.8	0.737	Case 5	132.8
37	53.1	0.766	Case 6	50.2	0.843	Case 5	55.3
38	88.4	0.750	Case 6	88.3	0.867	Case 5	102.1
39	88.4	0.775	Case 6	97.7	0.827	Case 5	104.2
40	161.4	0.775	Case 6	154.3	0.842	Case 5	167.7
41	162.7	0.767	Case 6	180.9	0.849	Case 5	200.1
42	98.0	0.720	Case 6	86.6	0.831	Case 5	100.0
43	73.1	0.800	Case 6	71.1	0.888	Case 5	78.9
44	174.4	0.724	Case 6	160.2	0.795	Case 5	176.0
45	180.0	0.724	Case 6	170.5	0.819	Case 5	192.9
46	99.3	0.833	Case 6	104.4	0.881	Case 5	110.3

Table 5.4: Summary Strength Results for Welded Rectangular HSS Connections

Index	Experimental		AISC 2016		AISC 2022		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{AISC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
1	151.7	[2]	108.5	[2]	113.1	[2]	120.9	[3]
2	151.5	[2]	120.4	[2]	126.8	[2]	122.6	[3]
3	152.9	[2]	120.4	[2]	125.9	[2]	124.8	[3]
4	151.3	[2]	122.1	[2]	127.5	[2]	126.2	[3]
5	152.2	[2]	101.6	[2]	107.2	[2]	120.3	[3]
6	146.8	[2]	118.0	[2]	125.8	[2]	122.8	[3]
7	152.4	[2]	115.1	[2]	121.8	[2]	125.4	[3]
8	151.5	[2]	114.5	[2]	121.6	[2]	126.7	[3]
9	138.5	[2]	88.4	[2]	96.0	[2]	100.3	[3]
10	144.3	[2]	N/A	N/A	122.3	[2]	103.4	[3]
11	146.4	[2]	104.3	[2]	113.5	[2]	103.8	[3]
12	146.8	[2]	104.7	[2]	113.8	[2]	104.5	[3]
13	152.0	[2]	108.3	[2]	115.9	[2]	108.9	[3]
14	150.4	[2]	107.9	[2]	115.5	[2]	106.4	[3]
15	156.5	[2]	125.4	[2]	130.0	[2]	130.0	[3]
16	155.3	[2]	125.4	[2]	130.0	[2]	129.8	[3]
17	139.6	[2]	101.2	[2]	111.6	[2]	104.7	[3]
18	142.8	[2]	102.6	[2]	113.2	[2]	105.2	[3]
19	149.9	[2]	118.6	[2]	124.9	[2]	124.4	[3]
20	151.5	[2]	118.8	[2]	125.1	[2]	124.3	[3]
21	149.3	[2]	105.1	[2]	114.2	[2]	106.9	[3]
22	150.2	[2]	105.2	[2]	114.2	[2]	105.0	[3]
23	149.7	[2]	105.9	[2]	114.4	[2]	105.0	[3]
24	146.1	[2]	105.3	[2]	114.2	[2]	104.7	[3]
25	150.6	[2]	121.6	[2]	127.1	[2]	126.2	[3]
26	150.6	[2]	121.8	[2]	127.1	[2]	125.9	[3]
27	150.2	[1]	116.8	[2]	123.2	[2]	116.8	[3]
28	148.4	[1]	117.4	[2]	123.8	[2]	116.8	[3]
29	149.9	[1]	104.3	[2]	113.4	[2]	95.9	[3]
30	182.3	[2]	160.9	[1]	160.9	[1]	151.8	[3]
31	188.0	[2]	156.3	[1]	156.3	[1]	146.7	[3]
32	149.3	[2]	143.6	[2]	152.5	[2]	148.6	[3]
33	163.0	[2]	150.2	[2]	159.2	[2]	153.6	[3]
34	190.0	[2]	134.7	[2]	142.8	[2]	149.4	[3]
35	192.0	[2]	138.2	[2]	146.2	[2]	151.3	[3]
36	137.6	[2]	118.8	[2]	132.8	[2]	105.6	[3]
37	61.6	[2]	50.2	[2]	53.1	[1]	50.6	[3]
38	113.5	[2]	88.3	[2]	88.4	[1]	77.8	[3]
39	107.5	[2]	88.4	[1]	88.4	[1]	85.3	[3]
40	187.3	[2]	154.3	[2]	161.4	[1]	151.2	[3]
41	213.4	[2]	162.7	[1]	162.7	[1]	145.5	[3]
42	106.8	[2]	86.6	[2]	98.0	[1]	86.4	[3]
43	86.3	[2]	71.1	[2]	73.1	[1]	70.4	[3]
44	159.8	[2]	160.2	[2]	174.4	[1]	156.7	[3]
45	205.3	[2]	170.5	[2]	180.0	[1]	148.8	[3]
46	125.9	[2]	99.3	[1]	99.3	[1]	96.8	[3]

[1] tensile yield; [2] tensile rupture; [3] member strain

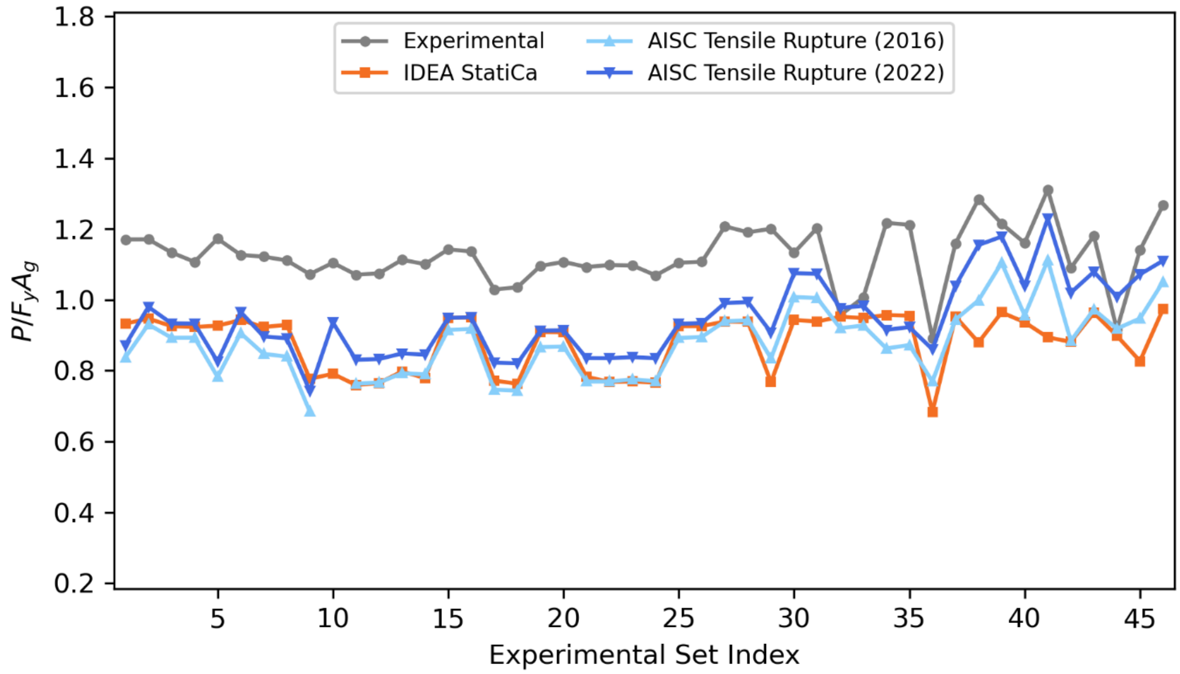


Figure 5-5: Normalized Strength Results for Welded Rectangular HSS Connections

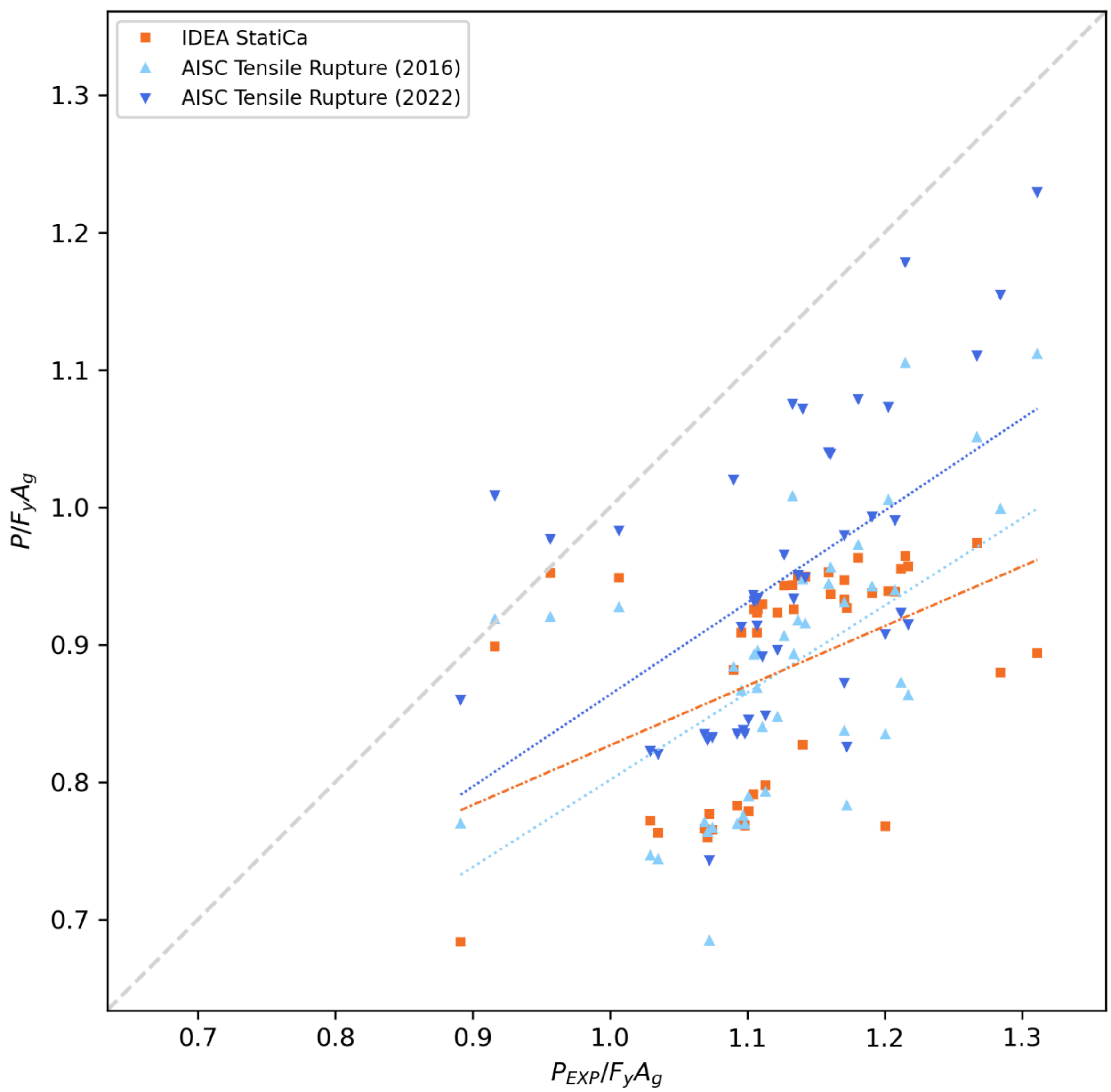


Figure 5-6: Normalized Strength vs. Normalized Experimental Strength for Welded Rectangular HSS Connections

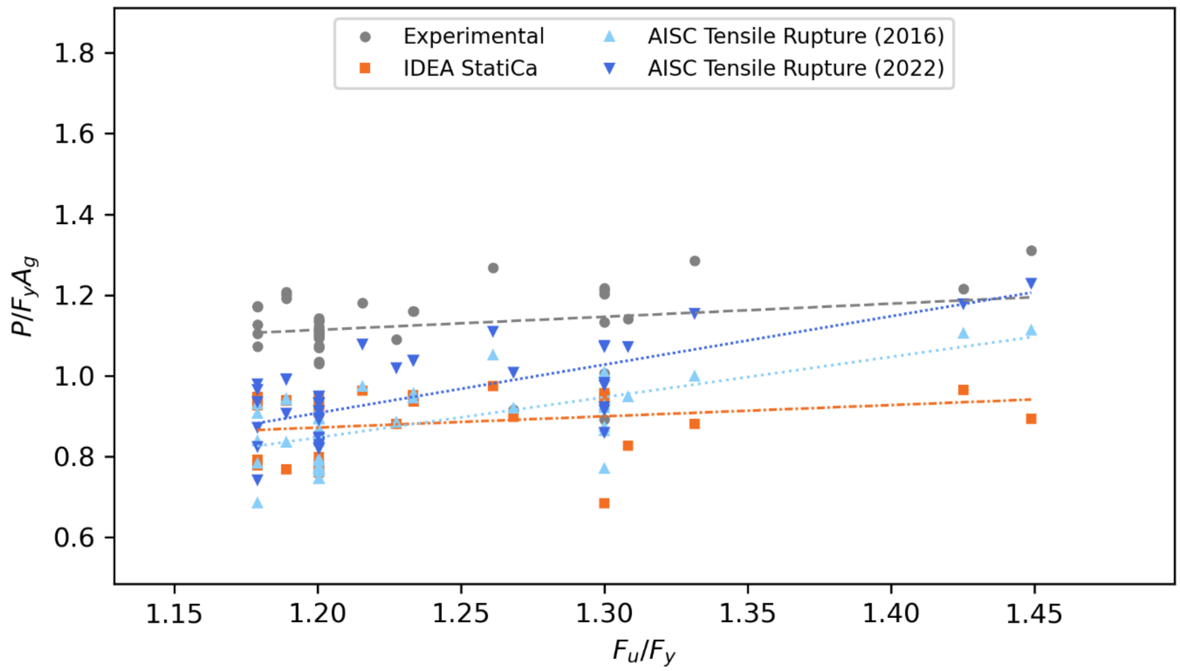


Figure 5-7: Normalized Strength vs. Material Strength Ratio for Welded Rectangular HSS Connections

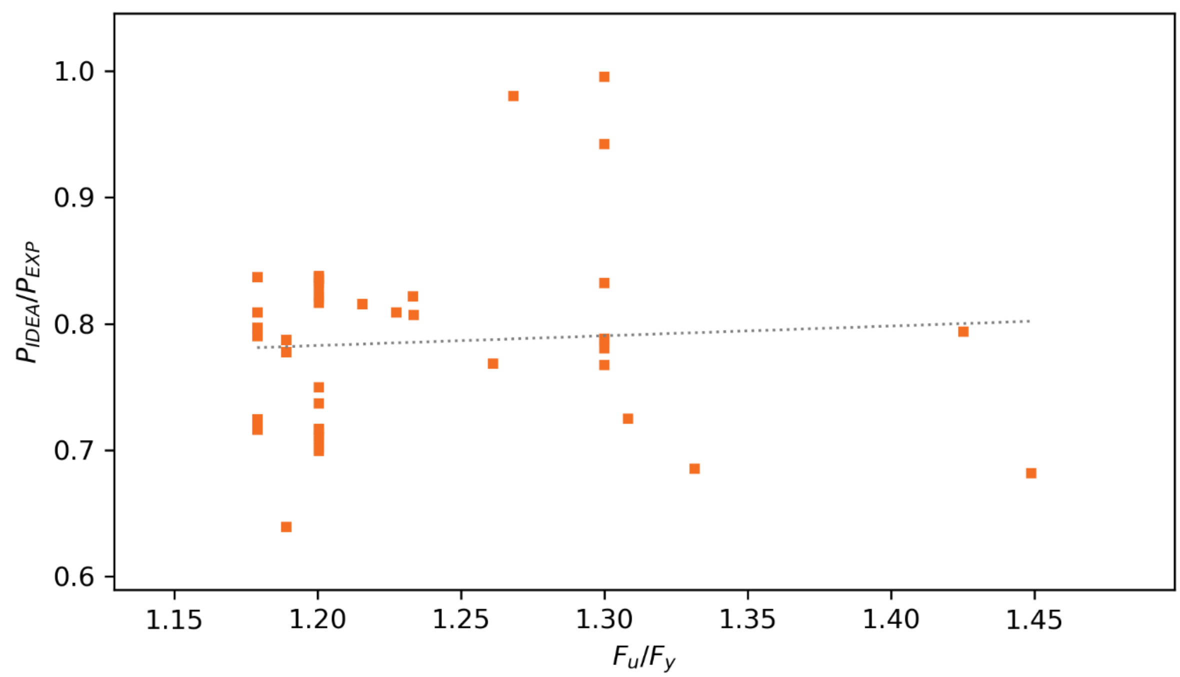


Figure 5-8: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Welded Rectangular HSS Connections

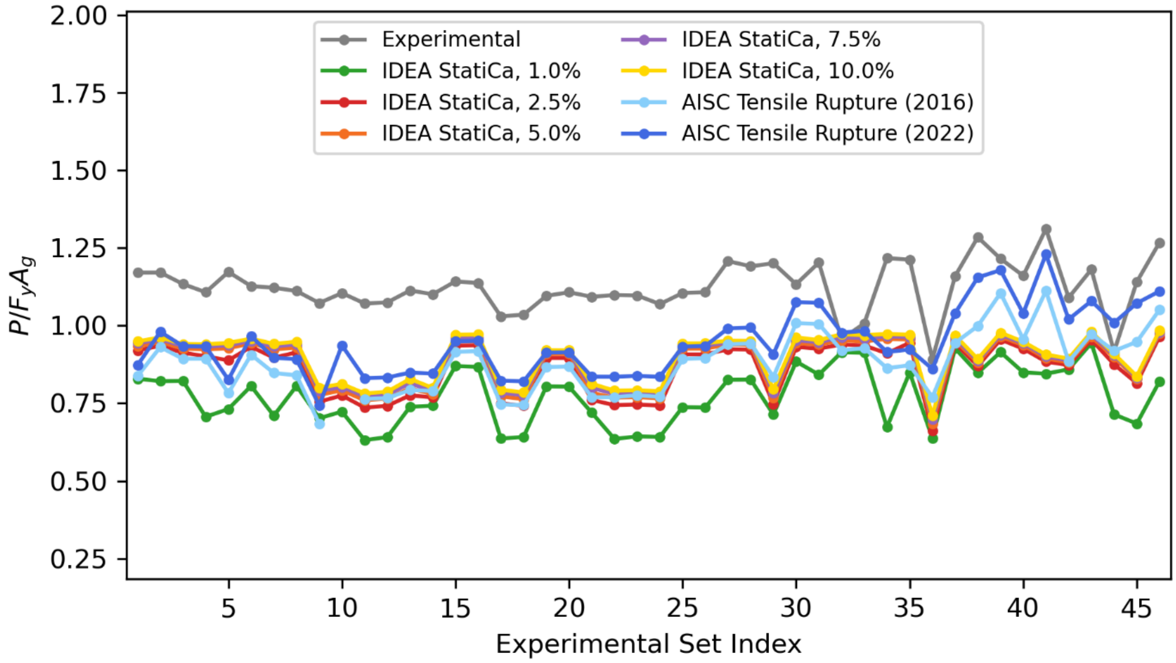


Figure 5-9: Normalized Strength Results for Welded Rectangular HSS Connections Including Various Plastic Strain Limits for IDEA StatiCa

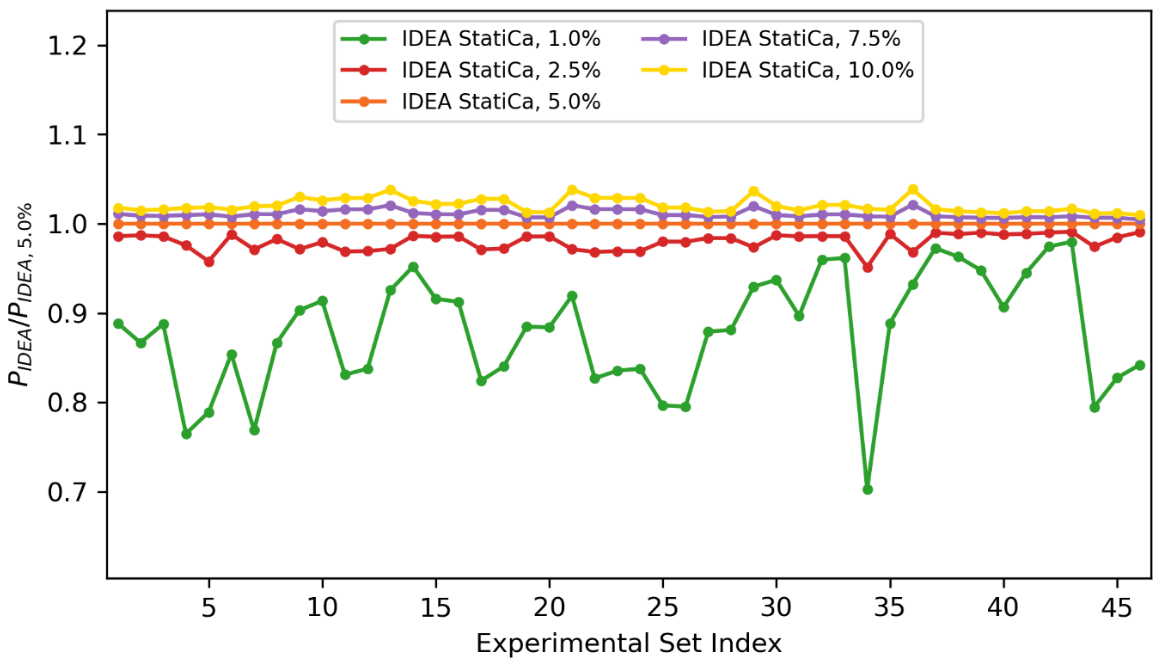


Figure 5-10: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Welded Rectangular HSS Connections

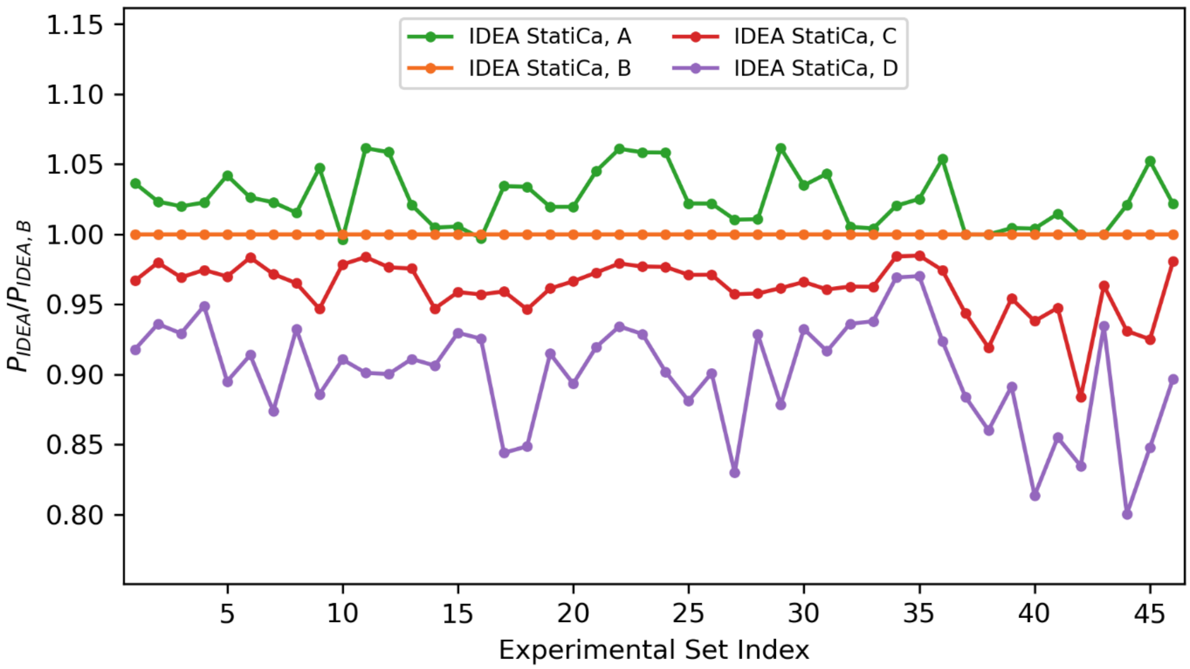


Figure 5-11: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Welded Rectangular HSS Connections

Table 5.5: Summary Statistics of the Test-to-Predicted Ratio for Welded Rectangular HSS Connections

Test-to-Predicted Ratio	Average	Standard Deviation	Coefficient of Variation
$P_{EXP}/P_{RUPTURE}$ (AISC 2016)	1.287	0.122	0.095
$P_{EXP}/P_{RUPTURE}$ (AISC 2022)	1.197	0.116	0.097
P_{EXP}/P_{IDEA}	1.284	0.116	0.090

5.2.3 Discussion

Three of the experimental specimens were reported as failed by tensile yielding with the remainder failing by tensile rupture (Table 5.4). Tensile rupture was the controlling limit state for most of the specimens for the AISC calculations as well.

There are a few instances where P_{IDEA} is greater than the controlling strength according to the 2022 AISC *Specification* strength results. The maximum difference appears with experimental index specimen 5, with a 12% difference. As for the most conservative case, P_{IDEA} provides a strength ~20% less than P_{AISC} in experimental specimen index 36. There is no specimen for which P_{IDEA} exceeded the experimental strength, P_{EXP}. The largest difference between P_{IDEA} and P_{EXP} occurred for specimen 29. This specimen indicated roughly a 36% difference from IDEA StatiCa's strength to the experimentally observed strength. Typically, IDEA StatiCa provided conservatism between 20-22% when compared to the experimental results.

The results in Figure 5-5 and Figure 5-6 show that the AISC tensile rupture and IDEA StatiCa strengths are similar. The strengths from the 2022 edition of the AISC *Specification* were consistently greater than those from the 2016 edition of the AISC *Specification*.

With respect to the material strength ratio F_u/F_y, all normalized strengths increase with increases in F_u/F_y (Figure 5-7). The trend for the AISC strength equations is stronger than for IDEA StatiCa or the experimental results. Correspondingly, the ratio of maximum applied load according to IDEA StatiCa, P_{IDEA}, to the strength observed in experiments, P_{EXP} does not exhibit any significant trends (Figure 5-8).

The variation of P_{IDEA} with the plastic strain limit exhibited the expected trend of increasing strength with increasing plastic strain limit (Figure 5-9 and Figure 5-10), however, except for the 1% strain limit, the P_{IDEA} did not vary much with plastic strain limit. For strength with a 1% strain limit was roughly 10% to 20% less than the for the default strain limit of 5%.

The variation of P_{IDEA} with mesh parameters show decreases in strength as the mesh was refined. However, the results do not appear to be converging. Specifically, the increment between 'C' and 'D' is larger in many cases than the increment between 'B' and 'C'.

5.3 Reliability Analysis

5.3.1 Description of Reliability Set

The reliability analyses performed in this work applies to specific connections. To ensure broadly applicable results, a set of connections is defined with varying parameters. In contrast to the experimental set, the connections in the reliability set are not based on physical connections or test specimens. Parameters for the rectangular HSS reliability set are outlined in Table 5.6. They include variations in the H/B ratios, weld lengths, cross-section thicknesses, and gusset plate thicknesses.

The connections were modeled in IDEA StatiCa largely the same as described previously for the experimental set with some differences. Connections in the reliability set utilize nominal material and geometric properties. The rectangular HSS members selected in IDEA StatiCa were from the 'HSS (AISC 15.0 – A500, A502, A618, A847)' section of the cross-section categories. These members contain reduced thicknesses consistent with practice. Additionally, the material grade A500 Gr. C has a few options in IDEA StatiCa. The material 'A500, Gr. C, *shaped*' was selected for the reliability set, as it is consistent with the current ASTM standard (2018).

5.3.2 Results

Strength results for the reliability set are provided in Table 5.7. The AISC strength calculations are based on the 2022 edition of the code (AISC 2022a). Nominal and design strengths are provided; however, the controlling design strength was used for the purpose of this reliability analysis (ϕP_{AISC} is the minimum of ϕP_{YIELD} and $\phi P_{RUPTURE}$). The IDEA StatiCa strength results, P_{IDEA}, are based upon all default settings (including LRFD resistance factors). When comparing P_{IDEA} to ϕP_{AISC}, IDEA StatiCa typically resulted in larger strengths than the AISC code equations. The most extreme cases resulted in a 25% higher strength in IDEA StatiCa than the AISC *Specification* equations. On average, P_{IDEA} exceeds ϕP_{AISC} by about 13%. It is possible the AISC tensile rupture ϕ-factor is a significant reason for these differences since large differences were not observed in the experimental set (where LRFD resistance factors were not used). The IDEA StatiCa, P_{IDEA}, was lower than ϕP_{AISC} for some specimens.

Table 5.6: Welded Rectangular HSS Connection Reliability Set Parameters

Index	*H*, in.	*B*, in.	*t*, in.	Gusset Plate Thickness, in.	Weld Length, in.	Material Grade	F_y, ksi	F_u, ksi
1	8	8	0.500	0.5	8	A500 Gr. C	50	62
2	8	8	0.250	0.5	8	A500 Gr. C	50	62
3	8	8	0.125	0.5	8	A500 Gr. C	50	62
4	8	8	0.500	0.5	16	A500 Gr. C	50	62
5	8	8	0.250	0.5	16	A500 Gr. C	50	62
6	8	8	0.125	0.5	16	A500 Gr. C	50	62
7	8	8	0.500	1.0	8	A500 Gr. C	50	62
8	8	8	0.250	1.0	8	A500 Gr. C	50	62
9	8	8	0.125	1.0	8	A500 Gr. C	50	62
10	8	8	0.500	1.0	16	A500 Gr. C	50	62
11	8	8	0.250	1.0	16	A500 Gr. C	50	62
12	8	8	0.125	1.0	16	A500 Gr. C	50	62
13	8	4	0.500	0.5	8	A500 Gr. C	50	62
14	8	4	0.250	0.5	8	A500 Gr. C	50	62
15	8	4	0.125	0.5	8	A500 Gr. C	50	62
16	8	4	0.500	0.5	16	A500 Gr. C	50	62
17	8	4	0.250	0.5	16	A500 Gr. C	50	62
18	8	4	0.125	0.5	16	A500 Gr. C	50	62
19	8	4	0.500	1.0	8	A500 Gr. C	50	62
20	8	4	0.250	1.0	8	A500 Gr. C	50	62
21	8	4	0.125	1.0	8	A500 Gr. C	50	62
22	8	4	0.500	1.0	16	A500 Gr. C	50	62
23	8	4	0.250	1.0	16	A500 Gr. C	50	62
24	8	4	0.125	1.0	16	A500 Gr. C	50	62
25	4	8	0.500	0.5	8	A500 Gr. C	50	62
26	4	8	0.250	0.5	8	A500 Gr. C	50	62
27	4	8	0.125	0.5	8	A500 Gr. C	50	62
28	4	8	0.500	0.5	16	A500 Gr. C	50	62
29	4	8	0.250	0.5	16	A500 Gr. C	50	62
30	4	8	0.125	0.5	16	A500 Gr. C	50	62
31	4	8	0.500	1.0	8	A500 Gr. C	50	62
32	4	8	0.250	1.0	8	A500 Gr. C	50	62
33	4	8	0.125	1.0	8	A500 Gr. C	50	62
34	4	8	0.500	1.0	16	A500 Gr. C	50	62
35	4	8	0.250	1.0	16	A500 Gr. C	50	62
36	4	8	0.125	1.0	16	A500 Gr. C	50	62

Table 5.7: Summary Strength Results for Welded Rectangular HSS Connection Reliability Set

Index	AISC 2022				IDEA StatiCa
	P_{YIELD}, kips	$P_{RUPTURE}$, kips	ϕP_{YIELD}, kips	$\phi P_{RUPTURE}$, kips	P_{IDEA}, kips
1	675.0	538.2	607.5	403.7	373.5
2	355.0	279.1	319.5	209.3	192.6
3	181.0	141.2	162.9	105.9	101.8
4	675.0	673.2	607.5	504.9	608.2
5	355.0	352.4	319.5	264.3	324.3
6	181.0	179.2	162.9	134.4	165.9
7	675.0	534.6	607.5	400.9	378.9
8	355.0	277.8	319.5	208.3	195.4
9	181.0	140.7	162.9	105.6	99.9
10	675.0	657.0	607.5	492.7	590.7
11	355.0	344.6	319.5	258.4	315.9
12	181.0	175.4	162.9	131.5	161.2
13	487.0	481.3	438.3	360.9	384.7
14	262.0	256.4	235.8	192.3	197.4
15	135.0	131.4	121.5	98.6	104.4
16	487.0	528.2	438.3	396.1	441.4
17	262.0	283.4	235.8	212.6	239.7
18	135.0	145.8	121.5	109.4	123.8
19	487.0	470.2	438.3	352.7	400.0
20	262.0	251.5	235.8	188.6	203.4
21	135.0	129.1	121.5	96.8	102.7
22	487.0	508.2	438.3	381.2	431.5
23	262.0	273.7	235.8	205.3	234.1
24	135.0	141.1	121.5	105.8	120.6
25	487.0	406.0	438.3	304.5	368.4
26	262.0	215.8	235.8	161.9	191.3
27	135.0	110.5	121.5	82.9	100.9
28	487.0	490.5	438.3	367.9	440.6
29	262.0	263.1	235.8	197.3	240.3
30	135.0	135.3	121.5	101.5	123.9
31	487.0	395.1	438.3	296.4	371.8
32	262.0	211.0	235.8	158.2	194.0
33	135.0	108.2	121.5	81.2	99.1
34	487.0	470.7	438.3	353.0	422.0
35	262.0	253.5	235.8	190.1	232.1
36	135.0	130.6	121.5	98.0	120.1

The reliability index, β, results are presented in Figure 5-12. The β-values for AISC fell within the range of 3.88 to 4.38. For IDEA StatiCa, the range was from 3.31 to 4.61. Specimen indices 1 through 24 appear to be more sensitive to the length of weld in the IDEA StatiCa results. Specimen indices 4, 5, 6, 10, 11, 12, 16, 17, 18, 22, 23, and 24 all have welds of 16 in. (longest weld in reliability set) and result in lower reliability for IDEA StatiCa. However, specimen indices 25 through 36 appear to be less influenced by weld length. These specimens have an H/B of 0.5, which results in a larger eccentricity.

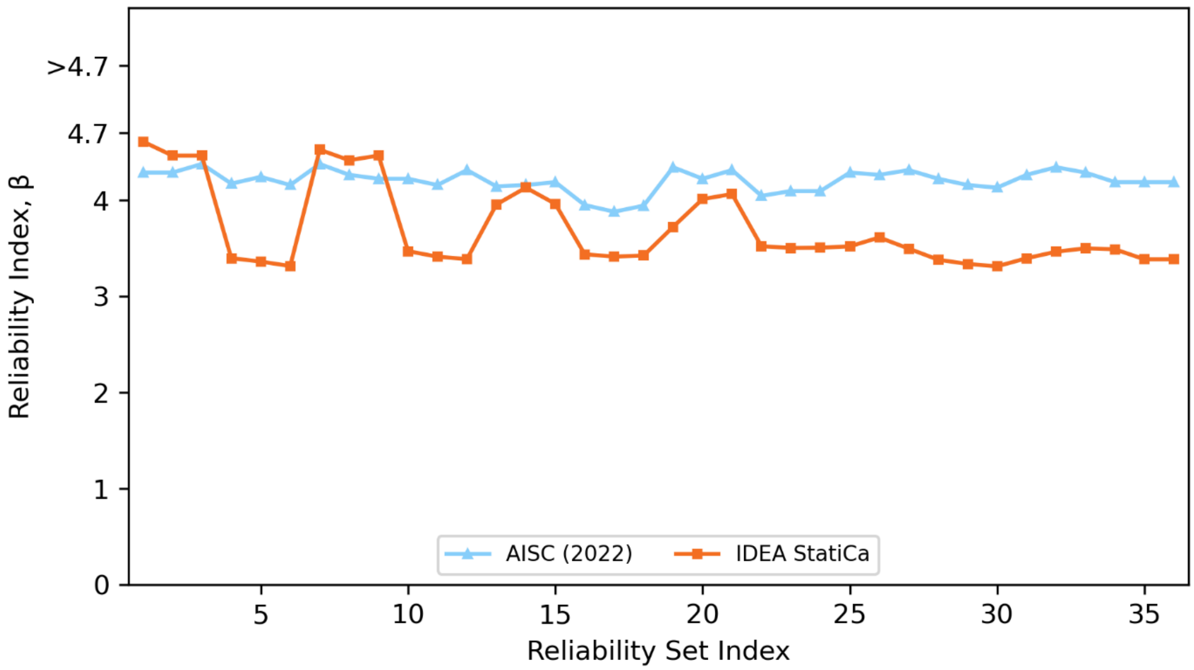


Figure 5-12: Reliability Index for Welded Rectangular HSS Connections

Chapter 6:
WELDED ANGLES

6.1 Description of Connection

Single and double angle tension members welded to a gusset plate are evaluated in this chapter. The weld configurations investigated in this chapter include longitudinal welds only and longitudinal welds combined with transverse welds. A schematic of the connection with relevant terminology is provided in Figure 6-1.

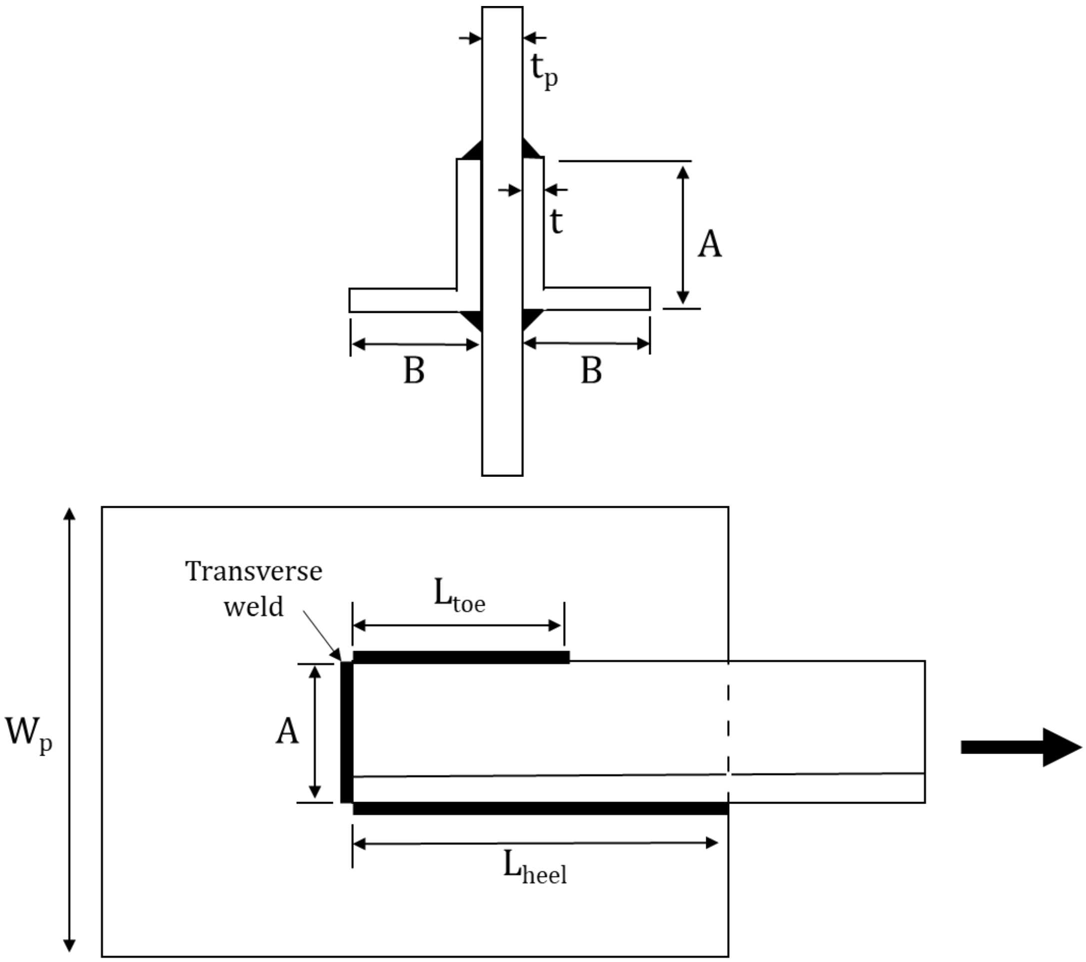


Figure 6-1: Schematic of Welded Angle Connection

The applicable shear lag factor cases for the evaluation of tensile rupture for welded angles were Cases 2 and 4 from Table D3.1 of the AISC *Specification* (also provided in Figure 3-3).

Case 2 applies when the load is transmitted to some, but not all cross-sectional elements by longitudinal welds in combination with transverse welds. The shear lag factor for Case 2 is:

$$U = 1 - \frac{\bar{x}}{l} \tag{6-1}$$

where $\bar{x}$ is the eccentricity of the connection (i.e., the distance between the centroid of the angle and the faying surface) and l is defined in Figure 6-2. When weld lengths are unequal, l is the average of the two lengths.

Case 4 applies when the load is transmitted by longitudinal welds only. The shear lag factor for Case 4 is:

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) \tag{6-2}$$

where $\bar{x}$ is the eccentricity of the connection and l and w are defined in Figure 6-2. When weld lengths are unequal, l is the average of the two longitudinal welds.

Additionally, Section D3 of the AISC *Specification* sets a lower bound for shear lag factors that is applicable to single and double angles. The section states that the shear lag factor does not need to be less than the ratio of the gross of the connected elements to the gross area of the member. This lower bound was evaluated, but never controlled for welded angle members.

The gross area and centroid were calculated based on measured dimensions neglecting leg-to-leg and toe fillets.

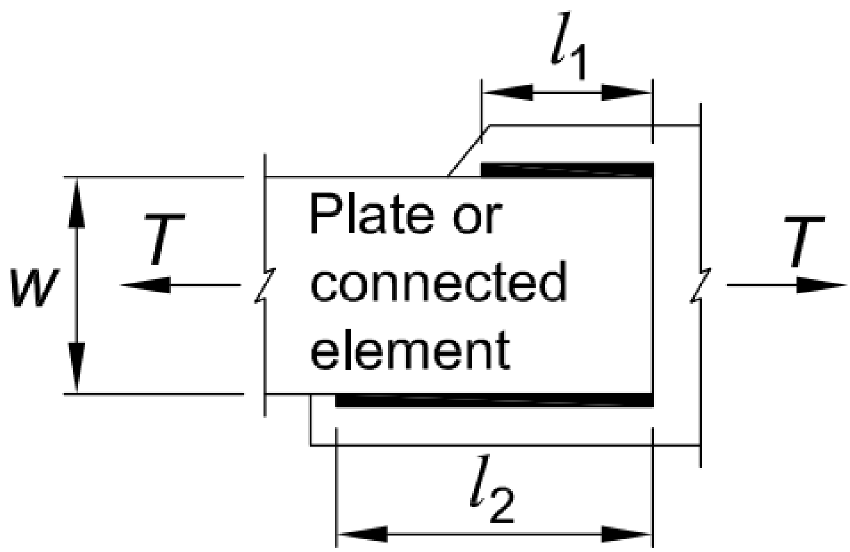


Figure 6-2: Example and dimensions for case 2 and 4 of Table D3.2 from the AISC *Specification*

6.2 Comparison to Experimental Results

6.2.1 Description of Experimental Specimens

A total of 127 welded angle specimens from 12 references were identified for evaluation in this work as detailed in Table 6.1. Some of the specimens described in these references were not included. Gonzalez (1989), also in Easterling and Gonzalez Giroux (1993), reported nine welded angle specimens, but specimens L-B-1b and L-T-1 were removed due to weld failures. Zhu et al.

(2009) reported 13 specimens, however four specimens were removed due to insufficient weld strength (specimens A1-300UL, A1-250US, A2-200BS, and A2-200UL). Mannem (2002) actually reported 22 experimental specimens, and specimens DEA3, DEA4, DEA5, DUEA1, UEA4, and UEA7 were removed from the current study due to weld failures. Pettretta, 1999 included 18 specimens, however 11 specimens were removed for a variety of reasons. Specimens A4-ii-s, C11-L-l, C12-L-ii, T21, T18, T20, T22, PS6-23, and PS6-MP24 were all removed due to the experimental failure mode being something other than tensile yield or rupture. Specimens X14-bx and X15-x were removed because the configuration of the angles is outside the scope of this chapter. Gibson and Wake (1942) actually reported 24 specimens. There were 14 specimens removed due to weld failures (single angles 1, 2, 4, 6, 8, 11, 12, 13, and 14; double angles 1, 8, 10, 11, 13), five specimens had configurations outside the scope of this section (single angles 5, 7, 9, and 15; double angles 5), and the machine reached full capacity before failure on double angle specimen 2. Dhanuskar and Gupta (2021a; b) has two separate publications, one published in May and one published in April. Some specimens were included in both papers, but unique specimens were presented in both as well. For the May 2021 paper, four specimens were excluded due to non-regular configurations (specimens 9 through 12). As for the April 2021 paper, five specimens were excluded because they were already accounted for in the May 2021 paper (specimens 16, 17, 18, 26, and 27). Specimens 5 through 8 failed in the welds, and were thus removed as well.

Table 6.1: Count of Welded Angle Connections by Reference

Reference	Specimen Count
Gonzalez (1989)	7
Zhu et al. (2009)	9
Ke et al. (2018)	9
Fang et al. (2013)	12
Regan and Salter (1984)	17
Mannem (2002)	16
Petretta (2000)	7
Bauer and Benaddi (2002)	6
Gibson and Wake (1942)	4
Dhanuskar and Gupta (2021b)	8
Dhanuskar and Gupta (2021a)	18
Uzoegbo (1998)	14
Total	**127**

The details for each experimental set specimen are provided in Table 6.2 and Table 6.3. As shown in the table, 25 specimens were double angles with the remainder being single angles. The dimensions ‘A’ and ‘B’ are consistent with Figure 6-1 where ‘A’ is the connected leg length and ‘B’ is the unconnected leg length. Unless otherwise noted, a weld strength of E70XX was assumed. If a different weld strength was reported, it was modeled as such. If the Table 6.3 does not show a value in the transverse weld columns, then transverse welds were not included on the specimen. If the length of the transverse weld was not provided, the weld was assumed to be continued along the entire length of the connected leg. There was not a reported case where the transverse weld was shorter than the connected leg. It should be noted, the names given to the specimens in Gibson and Wake (1942) consisted of two sets of numbers, one for single angles and one for double angles. There were duplicate specimen names between the two sets. For clarity, an ‘s’ or ‘d’ was added at the beginning of the specimens’ name in this work.

Table 6.2: Welded Angle Connection Experimental Specimen Parameters

Index	Reference	Specimen	# of angles	* F_y, ksi	*F_u, ksi	*A*, in.	*B*, in.
1	Gonzalez (1989)	L-L-1	2	54.1	81.1	2.011	1.991
2	Gonzalez (1989)	L-L-2	2	54.1	81.1	2.011	1.992
3	Gonzalez (1989)	L-L-3	2	54.1	81.1	2.009	1.989
4	Gonzalez (1989)	L-B-1a	2	47.8	71.3	3.972	2.978
5	Gonzalez (1989)	L-B-1c	2	54.1	81.1	2.01	1.992
6	Gonzalez (1989)	L-B-2	2	54.1	81.1	2.01	1.992
7	Gonzalez (1989)	L-B-3	2	54.1	81.1	2.01	1.994
8	Zhu et. al (2009)	A1-200BL	1	38.9	60.6	4.937	2.953
9	Zhu et. al (2009)	A1-200BS	1	38.9	60.6	2.953	4.937
10	Zhu et. al (2009)	A1-200UL	1	38.9	60.6	4.937	2.953
11	Zhu et. al (2009)	A1-250UL	1	38.9	60.6	4.937	2.953
12	Zhu et. al (2009)	A1-200US	1	38.9	60.6	2.953	4.937
13	Zhu et. al (2009)	A1-300US	1	38.9	60.6	2.953	4.937
14	Zhu et. al (2009)	A2-200BL	1	38.0	62.7	5.902	2.957
15	Zhu et. al (2009)	A2-200BS-d	1	38.0	62.7	2.957	5.902
16	Zhu et. al (2009)	A2-200US	1	38.0	62.7	2.957	5.902
17	Ke et al. (2018)	C1-220L	1	110.4	115.7	3.15	2.362
18	Ke et al. (2018)	C1-300L	1	110.4	115.7	3.15	2.362
19	Ke et al. (2018)	C1-380L	1	110.4	115.7	3.15	2.362
20	Ke et al. (2018)	C1-300S	1	110.4	115.7	2.362	3.150
21	Ke et al. (2018)	C2-220S	1	110.4	115.7	2.544	3.984
22	Ke et al. (2018)	C2-300S	1	110.4	115.7	2.559	3.937
23	Ke et al. (2018)	C2-380S	1	110.4	115.7	2.559	3.937
24	Ke et al. (2018)	D1-300L	1	42.4	65.7	3.101	2.395
25	Ke et al. (2018)	D2-300S	1	42.4	65.7	2.591	3.983
26	Fang et al. (2013)	A1-170B	1	39.7	59.8	1.181	2.362
27	Fang et al. (2013)	A1-170U	1	39.7	59.8	1.181	2.362
28	Fang et al. (2013)	A2-200B	1	70.2	100.5	1.969	2.953
29	Fang et al. (2013)	A2-200U	1	70.2	100.5	1.969	2.953
30	Fang et al. (2013)	A3-215B	1	41.6	60.2	1.969	2.362
31	Fang et al. (2013)	A3-215U	1	41.6	60.2	1.969	2.362
32	Fang et al. (2013)	A4-305B	1	32.8	54.4	2.953	4.921
33	Fang et al. (2013)	A4-305U	1	32.8	54.4	2.953	4.921
34	Fang et al. (2013)	A5-335B	1	40.3	51.5	2.953	5.906
35	Fang et al. (2013)	A5-335U	1	40.3	51.5	2.953	5.906
36	Fang et al. (2013)	A5-380B	1	40.3	51.5	2.953	5.906
37	Fang et al. (2013)	A5-380U	1	40.3	51.5	2.953	5.906
38	Regan and Salter (1984)	A1	1	43.9	68.0	0.984	0.984
39	Regan and Salter (1984)	B1	1	45.0	65.7	1.969	1.969
40	Regan and Salter (1984)	B2	1	46.7	67.2	1.969	1.969
41	Regan and Salter (1984)	B3	1	46.7	67.2	1.969	1.969
42	Regan and Salter (1984)	B4	1	47.6	68.6	1.969	1.969

Table 6.2 continued

Index	**Reference**	**Specimen**	**# of angles**	*** F_y, ksi**	***F_u, ksi**	**A, in.**	**B, in.**
43	Regan and Salter (1984)	C1	1	47.1	73.0	3.937	3.937
44	Regan and Salter (1984)	C2	1	49.0	74.7	3.937	3.937
45	Regan and Salter (1984)	D1	1	47.9	70.3	1.181	2.362
46	Regan and Salter (1984)	D2	1	46.7	68.3	1.181	2.362
47	Regan and Salter (1984)	D3	1	45.7	66.7	2.362	1.181
48	Regan and Salter (1984)	D4	1	46.7	68.3	2.362	1.181
49	Regan and Salter (1984)	D5	1	46.7	68.3	1.181	2.362
50	Regan and Salter (1984)	D6	1	46.4	67.6	1.181	2.362
51	Regan and Salter (1984)	E1	1	39.4	66.4	1.969	2.559
52	Regan and Salter (1984)	E2	1	38.4	68.5	2.559	1.969
53	Regan and Salter (1984)	F1	1	47.1	69.9	2.953	4.921
54	Regan and Salter (1984)	F2	1	48.3	71.5	4.921	2.953
55	Mannem (2002)	DEA1	2	52.1	74.1	4.004	3.992
56	Mannem (2002)	DEA2	2	52.1	74.1	4.004	3.992
57	Mannem (2002)	EA1	1	51.4	73.2	3.992	4.000
58	Mannem (2002)	EA2	1	51.4	73.2	3.992	4.000
59	Mannem (2002)	EA3	1	52.2	76.1	5.969	5.961
60	Mannem (2002)	EA4	1	52.2	76.1	5.969	5.961
61	Mannem (2002)	EAm1	1	51.4	73.2	3.992	4.000
62	Mannem (2002)	EAm2	1	51.4	73.2	3.992	4.000
63	Mannem (2002)	DUEA2	2	51.2	69.6	2.953	4.921
64	Mannem (2002)	UEA1	1	51.7	69.8	4.965	2.980
65	Mannem (2002)	UEA2	1	51.7	69.8	2.980	4.965
66	Mannem (2002)	UEA3	1	51.7	69.8	4.965	2.980
67	Mannem (2002)	UEA5	1	51.2	69.6	2.953	4.921
68	Mannem (2002)	UEA6	1	51.2	69.6	2.953	4.921
69	Mannem (2002)	UEA8	1	53.2	70.4	6.000	4.020
70	Mannem (2002)	UEA9	1	53.2	70.4	6.000	4.020
71	Petretta (2000)	A1-i	2	58.9	79.9	2.502	2.510
72	Petretta (2000)	A3-ii	2	58.9	79.9	2.504	2.510
73	Petretta (2000)	A6-iii	2	58.9	79.9	2.506	2.506
74	Petretta (2000)	B8-i-t	2	58.9	79.9	2.506	2.506
75	Petretta (2000)	B10-iii-t	2	58.9	79.9	2.504	2.514
76	Petretta (2000)	C12-L-ii-retest	2	51.5	76.0	2.437	3.482
77	Petretta (2000)	T19	2	50.8	73.1	2.516	2.518
78	Bauer and Benaddi (2002)	No. 1	2	57.0	77.0	1.496	1.496
79	Bauer and Benaddi (2002)	No. 2	2	50.6	71.4	2.008	2.008
80	Bauer and Benaddi (2002)	No. 3	2	46.1	66.9	2.008	2.520
81	Bauer and Benaddi (2002)	No. 4	2	50.0	72.4	2.520	2.520
82	Bauer and Benaddi (2002)	No. 5	2	49.2	70.6	2.008	2.992
83	Bauer and Benaddi (2002)	No. 6	2	50.5	76.4	2.992	2.992
84	Gibson and Wake (1942)	s3	1	38.8	64.4	2.500	2.500

Table 6.2 continued

Index	Reference	Specimen	# of angles	* F_y, ksi	*F_u, ksi	A, in.	B, in.
85	Gibson and Wake (1942)	s10	1	38.8	64.4	2.500	2.500
86	Gibson and Wake (1942)	d3	2	38.8	64.4	2.500	2.500
87	Gibson and Wake (1942)	d4	2	38.8	64.4	2.500	2.500
88	Dhanuskar and Gupta (2021b)	S1-250/90-B	1	52.5	70.9	3.961	3.961
89	Dhanuskar and Gupta (2021b)	S1-250/90-B	1	52.5	70.9	3.961	3.961
90	Dhanuskar and Gupta (2021b)	S1-250/90-B	1	52.5	70.9	3.961	3.961
91	Dhanuskar and Gupta (2021b)	S1-195/195-U	1	52.5	70.9	3.961	3.961
92	Dhanuskar and Gupta (2021b)	S1-195/195-U	1	52.5	70.9	3.961	3.961
93	Dhanuskar and Gupta (2021b)	S2-125/125-U	1	49.3	70.6	3.941	3.941
94	Dhanuskar and Gupta (2021b)	S2-125/125-U	1	49.3	70.6	3.941	3.941
95	Dhanuskar and Gupta (2021b)	S2-125/125-U	1	49.3	70.6	3.941	3.941
96	Dhanuskar and Gupta (2021a)	S1-150/100-U	1	59.8	82.4	2.539	2.524
97	Dhanuskar and Gupta (2021a)	S1-150/100-U	1	59.8	82.4	2.539	2.524
98	Dhanuskar and Gupta (2021a)	S1-150/100-U	1	59.8	82.4	2.539	2.524
99	Dhanuskar and Gupta (2021a)	S1-100/65-U	1	59.8	82.4	2.539	2.524
100	Dhanuskar and Gupta (2021a)	S2-225/85-B	1	48.9	63.5	2.594	2.547
101	Dhanuskar and Gupta (2021a)	S2-225/85-B	1	48.9	63.5	2.594	2.547
102	Dhanuskar and Gupta (2021a)	S2-225/85-B	1	48.9	63.5	2.594	2.547
103	Dhanuskar and Gupta (2021a)	S3-255/95-B	1	48.6	64.2	2.969	3.004
104	Dhanuskar and Gupta (2021a)	S3-255/95-B	1	48.6	64.2	2.969	3.004
105	Dhanuskar and Gupta (2021a)	S3-255/95-B	1	48.6	64.2	2.969	3.004
106	Dhanuskar and Gupta (2021a)	S3-225/135-U	1	48.6	64.2	2.969	3.004
107	Dhanuskar and Gupta (2021a)	S2-155/155-U	1	48.9	63.5	2.594	2.547
108	Dhanuskar and Gupta (2021a)	S2-155/155-U	1	48.9	63.5	2.594	2.547
109	Dhanuskar and Gupta (2021a)	S2-155/155-U	1	48.9	63.5	2.594	2.547
110	Dhanuskar and Gupta (2021a)	S3-255/135-U	1	48.6	64.2	2.969	3.004
111	Dhanuskar and Gupta (2021a)	S3-175/175-U	1	48.6	64.2	2.969	3.004
112	Dhanuskar and Gupta (2021a)	S3-175/175-U	1	48.6	64.2	2.969	3.004
113	Dhanuskar and Gupta (2021a)	S3-175/175-U	1	48.6	64.2	2.969	3.004
114	Uzoegbo (1998)	E1	1	46.4	75.4	2.953	4.921
115	Uzoegbo (1998)	E2	1	46.4	75.4	1.969	2.953
116	Uzoegbo (1998)	E3	1	46.4	75.4	1.969	2.559
117	Uzoegbo (1998)	E4	1	46.4	75.4	2.756	2.756
118	Uzoegbo (1998)	E5	1	46.4	75.4	4.921	2.953
119	Uzoegbo (1998)	E6	1	46.4	75.4	2.953	1.969
120	Uzoegbo (1998)	E7	1	46.4	75.4	2.560	1.969
121	Uzoegbo (1998)	U1	1	46.4	75.4	2.953	4.921
122	Uzoegbo (1998)	U2	1	46.4	75.4	1.969	2.953
123	Uzoegbo (1998)	U3	1	46.4	75.4	1.966	2.559
124	Uzoegbo (1998)	U4	1	46.4	75.4	2.756	2.756
125	Uzoegbo (1998)	U5	1	46.4	75.4	4.921	2.953
126	Uzoegbo (1998)	U6	1	46.4	75.4	2.953	1.969
127	Uzoegbo (1998)	U7	1	46.4	75.4	2.560	1.969

* indicates measured value

Table 6.3: Welded Angle Connection Experimental Specimen Weld Details

Index	Weld Material	Toe Weld Size, in.	L_{TOE}, in.	Heel Weld Size, in.	L_{HEEL}, in.	Transverse Weld Size, in.	$L_{TRANS.}$, in.
1	E70XX	0.188	4.500	0.375	4.500	-	-
2	E70XX	0.188	4.500	0.375	4.500	-	-
3	E70XX	0.188	4.500	0.375	4.500	-	-
4	E70XX	0.250	3.500	0.250	3.500	0.250	4.000
5	E70XX	0.188	3.000	0.438	3.000	0.188	2.000
6	E70XX	0.188	3.000	0.438	3.000	0.188	2.000
7	E70XX	0.188	3.000	0.438	3.000	0.188	2.000
8	E70XX	0.315	4.724	0.315	11.024	0.315	4.937
9	E70XX	0.315	3.150	0.315	12.598	0.315	2.953
10	E70XX	0.315	7.874	0.315	7.874	0.315	4.937
11	E70XX	0.315	9.843	0.315	9.843	0.315	4.937
12	E70XX	0.315	7.874	0.315	7.874	0.315	2.953
13	E70XX	0.315	11.811	0.315	11.811	0.315	2.953
14	E70XX	0.315	4.724	0.315	11.024	0.315	5.902
15	E70XX	0.315	2.756	0.315	12.992	0.315	2.957
16	E70XX	0.315	7.874	0.315	7.874	0.315	2.957
17	NP	0.276	8.661	0.276	8.661	0.276	3.150
18	NP	0.276	11.811	0.276	11.811	0.276	3.150
19	NP	0.276	14.961	0.276	14.961	0.276	3.150
20	NP	0.276	11.811	0.276	11.811	0.276	2.362
21	NP	0.276	8.661	0.276	8.661	0.276	2.559
22	NP	0.276	11.811	0.276	11.811	0.276	2.559
23	NP	0.276	14.961	0.276	14.961	0.276	2.559
24	NP	0.276	11.811	0.276	11.811	0.276	3.150
25	NP	0.276	11.811	0.276	11.811	0.276	2.559
26	NP	0.236	2.756	0.236	10.630	0.236	1.181
27	NP	0.236	6.693	0.236	6.693	0.236	1.181
28	NP	0.236	3.543	0.236	12.205	0.236	1.969
29	NP	0.236	7.874	0.236	7.874	0.236	1.969
30	NP	0.236	3.937	0.236	12.992	0.236	1.969
31	NP	0.236	8.465	0.236	8.465	0.236	1.969
32	NP	0.394	4.724	0.394	19.291	0.394	2.953
33	NP	0.394	12.008	0.394	12.008	0.394	2.953
34	NP	0.394	5.118	0.394	21.260	0.394	2.953
35	NP	0.394	13.189	0.394	13.189	0.394	2.953
36	NP	0.394	6.693	0.394	23.228	0.394	NP
37	NP	0.394	14.961	0.394	14.961	0.394	NP
38	NP	0.157	7.874	0.157	7.874	0.157	0.984
39	NP	0.157	7.874	0.157	7.874	0.157	1.969
40	NP	0.157	7.874	0.157	7.874	0.157	1.969
41	NP	0.157	7.874	0.157	7.874	0.157	1.969
42	NP	0.157	7.874	0.157	7.874	0.157	1.969

Table 6.3 continued

Index	Weld Material	Toe Weld Size, in.	L_{TOE}, in.	Heel Weld Size, in.	L_{HEEL}, in.	Transverse Weld Size, in.	$L_{TRANS.}$, in.
43	NP	0.236	15.748	0.236	15.748	0.236	3.937
44	NP	0.236	15.748	0.236	15.748	0.236	3.937
45	NP	0.157	6.890	0.157	6.890	0.157	1.181
46	NP	0.157	6.890	0.157	6.890	0.157	1.181
47	NP	0.157	5.906	0.157	5.906	0.157	2.362
48	NP	0.157	5.906	0.157	5.906	0.157	2.362
49	NP	0.157	6.890	0.157	6.890	0.157	1.181
50	NP	0.157	6.890	0.157	6.890	0.157	1.181
51	NP	0.236	9.843	0.236	9.843	0.236	1.969
52	NP	0.236	8.858	0.236	8.858	0.236	2.559
53	NP	0.236	15.748	0.236	15.748	0.236	2.953
54	NP	0.236	14.764	0.236	14.764	0.236	4.921
55	E760XX#	0.264	3.937	0.709	3.937	0.264	4.004
56	E760XX#	0.264	15.748	0.709	15.748	-	-
57	E760XX#	0.268	5.512	0.472	5.512	0.268	3.992
58	E760XX#	0.268	5.315	0.630	5.315	-	-
59	E760XX#	0.382	8.268	0.709	8.268	0.382	5.969
60	E760XX#	0.382	8.465	0.709	8.465	-	-
61	E760XX#	0.268	5.906	0.551	5.906	0.268	3.992
62	E760XX#	0.268	5.906	0.551	5.906	0.268	3.992
63	E760XX#	0.256	4.921	0.551	4.921	0.256	2.980
64	E760XX#	0.256	4.921	0.551	4.921	0.256	4.965
65	E760XX#	0.256	5.315	0.551	5.315	0.335	2.980
66	E760XX#	0.256	6.102	0.551	6.102	-	-
67	E760XX#	0.260	9.843	0.630	9.843	-	-
68	E760XX#	0.260	7.480	0.630	7.480	-	-
69	E760XX#	0.319	9.055	0.630	9.055	-	-
70	E760XX#	0.319	11.811	0.630	11.811	-	-
71	ER480S	0.236	3.543	0.394	3.543	-	-
72	ER480S	0.236	4.724	0.236	4.724	-	-
73	ER480S	0.236	6.299	0.236	6.299	-	-
74	ER480S	0.236	3.543	0.394	3.543	0.236	2.506
75	ER480S	0.236	6.299	0.236	6.299	0.236	2.504
76	ER480S	0.236	4.724	0.394	4.724	-	-
77	ER480S	0.236	7.480	0.236	3.937	-	-
78	NP	0.197	3.425	0.197	3.425	-	-
79	NP	0.197	4.409	0.197	4.409	-	-
80	NP	0.197	4.803	0.197	4.803	-	-
81	NP	0.197	5.354	0.197	5.354	-	-
82	NP	0.197	5.433	0.197	5.433	-	-
83	NP	0.197	6.654	0.197	6.654	-	-
84	E-6012	0.250	2.875	0.375	4.375	-	-

Table 6.3 continued

Index	Weld Material	Toe Weld Size, in.	L_{TOE}, in.	Heel Weld Size, in.	L_{HEEL}, in.	Transverse Weld Size, in.	$L_{TRANS.}$, in.
85	E-6012	0.250	4.875	0.250	2.375	0.250	2.500
86	E-6012	0.250	2.875	0.375	4.375	-	-
87	E-6012	0.250	2.875	0.500	3.500	-	-
88	NP	0.236	3.642	0.236	9.961	-	-
89	NP	0.236	3.720	0.236	9.843	-	-
90	NP	0.236	3.642	0.236	9.823	-	-
91	NP	0.236	7.677	0.236	7.677	-	-
92	NP	0.236	7.677	0.236	7.677	-	-
93	NP	0.236	4.961	0.236	4.961	-	-
94	NP	0.236	4.941	0.236	4.941	-	-
95	NP	0.236	4.921	0.236	4.921	-	-
96	NP	0.236	3.937	0.236	5.925	-	-
97	NP	0.236	4.035	0.236	6.024	-	-
98	NP	0.236	3.976	0.236	5.945	-	-
99	NP	0.236	2.579	0.236	3.957	-	-
100	NP	0.236	3.386	0.236	8.839	-	-
101	NP	0.236	3.366	0.236	8.917	-	-
102	NP	0.236	3.425	0.236	8.819	-	-
103	NP	0.197	3.799	0.197	10.059	-	-
104	NP	0.197	3.780	0.197	10.039	-	-
105	NP	0.197	3.780	0.197	10.098	-	-
106	NP	0.197	5.295	0.197	10.039	-	-
107	NP	0.236	6.102	0.236	6.102	-	-
108	NP	0.236	6.102	0.236	6.102	-	-
109	NP	0.236	6.102	0.236	6.102	-	-
110	NP	0.197	6.890	0.197	6.890	-	-
111	NP	0.197	6.890	0.197	6.890	-	-
112	NP	0.197	6.890	0.197	6.890	-	-
113	NP	0.197	6.890	0.197	6.890	-	-
114	E480XX#	0.197	9.843	0.197	9.843	0.197	2.953
115	E480XX#	0.197	5.906	0.197	5.906	0.197	1.969
116	E480XX#	0.197	5.906	0.197	5.906	0.197	1.969
117	E480XX#	0.197	5.906	0.197	5.906	0.197	2.756
118	E480XX#	0.197	9.843	0.197	9.843	0.197	4.921
119	E480XX#	0.197	5.906	0.197	5.906	0.197	2.953
120	E480XX#	0.197	5.906	0.197	5.906	0.197	2.559
121	E480XX#	0.197	6.693	0.197	13.386	0.197	2.953
122	E480XX#	0.197	3.937	0.197	7.874	0.197	1.969
123	E480XX#	0.197	3.937	0.197	7.874	0.197	1.969
124	E480XX#	0.197	3.937	0.197	7.874	0.197	2.756
125	E480XX#	0.197	6.693	0.197	13.386	0.197	4.921
126	E480XX#	0.197	3.937	0.197	7.874	0.197	2.953
127	E480XX#	0.197	3.937	0.197	7.874	0.197	2.559

NP is not provided; # indicates metric units

The modeling of the welded angles was different from that outlined in the previous two chapters, but similar to the remainder of the connection types. These specimens were modeled using two or three members, depending if the specimen consisted of one or two angles. The gusset plate was modeled as a member in addition to the angle specimen. The specimens were then attached using a variety of weld operations. An example of a modeled specimen with two angles is provided in Figure 6-3. The typical process for modelling the specimens is shown in Figure 6-4.

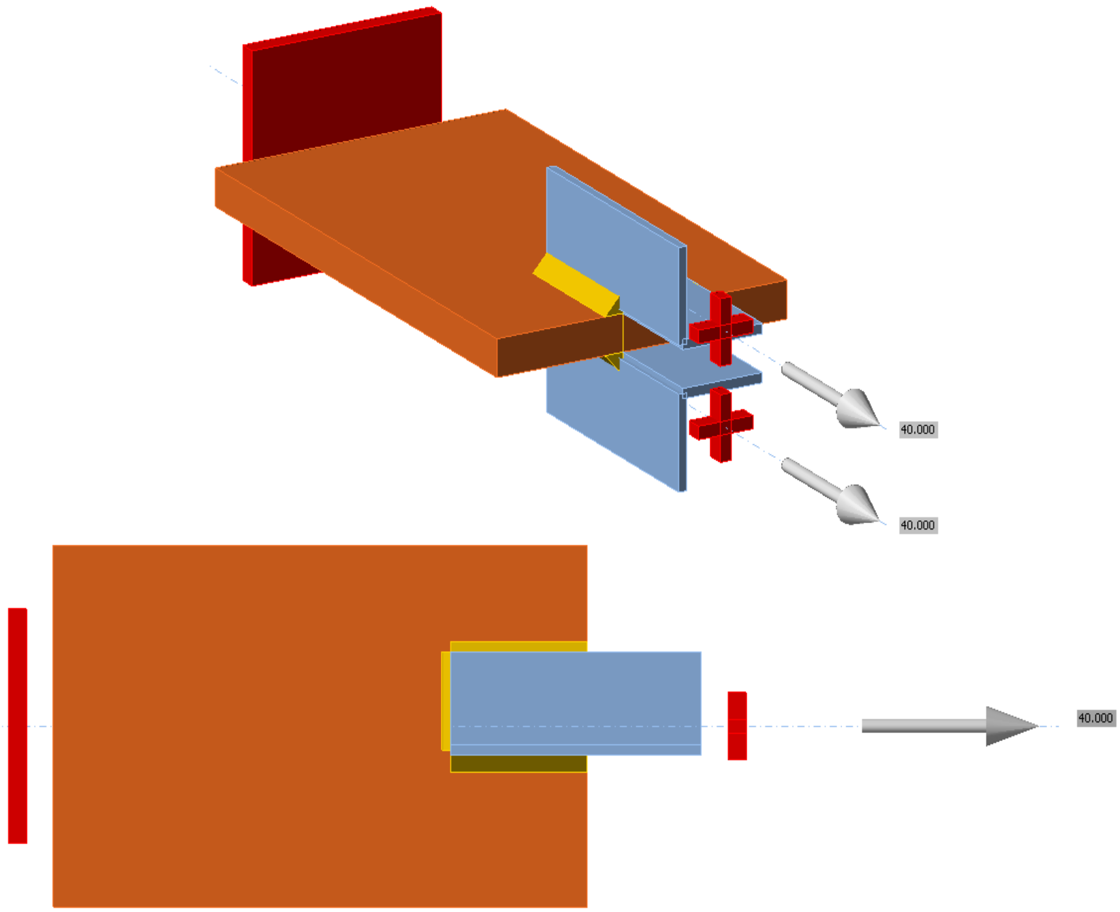


Figure 6-3: Typical Welded Angle Connection Model in IDEA StatiCa

4. Create Members
    a. Member 1: Gusset plate
        i. Geometrical type set to “Ended”
        ii. B set to 180º
        iii. Offset ex set to the negative of the connection length (longest longitudinal weld length)
    b. Member 2, 3 (if applicable)
        i. Geometrical type set to “Ended”
        ii. Align set to “To member plate”
        iii. Aligned plate set to “[member name] | Bottom flange 1”
        iv. Related plate set to “[gusset plate member name] Bottom flange 1”
        v. Model type set to “N-Vy-Vz”
5. Create weld operations:
    a. Continuous welds:
        i. Placement set to “Edge to surface”
        ii. Type set to “Weld”
        iii. First plate: Member or plate set to “[angle member name] | Bottom flange 1”
        iv. Select correct edge index
        v. Second plate: Plate set to “[gusset plate name] | Bottom flange 1”
        vi. Input desired weld size and type (fillet- front side for all specimens)
        vii. Weld type set to “Continuous”
    b. Partial welds:
        i. Placement set to “Edge to surface”
        ii. Type set to “Weld”
        iii. First plate: Member or plate set to “[angle member name] | Bottom flange 1”
        iv. Select correct edge index
        v. Second plate: Plate set to “[gusset plate name] | Bottom flange 1”
        vi. Input desired weld size and type (fillet- front side for all specimens)
        vii. Weld type set to “Partial”
        viii. Weld offset 2 set to the absolute value of the offset in Step 2 minus the partial weld length
    c. Repeat as necessary until all welds are modeled

Figure 6-4: Modelling Process for Welded Angle Connections

6.2.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 6.4: AISC Calculation Results for Welded Angle Connections

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
1	82.3	0.819	Case 4	101.0
2	82.3	0.819	Case 4	101.0
3	81.8	0.819	Case 4	100.5
4	160.7	0.791	Case 2	189.7
5	83.4	0.810	Case 2	101.3
6	82.7	0.810	Case 2	100.4
7	81.1	0.810	Case 2	98.6
8	115.9	0.911	Case 2	164.7
9	115.9	0.785	Case 2	141.9
10	115.9	0.911	Case 2	164.7
11	115.9	0.929	Case 2	167.9
12	115.9	0.785	Case 2	141.9
13	115.9	0.857	Case 2	154.8
14	132.7	0.917	Case 2	200.6
15	132.7	0.730	Case 2	159.7
16	132.7	0.730	Case 2	159.7
17	182.8	0.928	Case 2	177.9
18	181.8	0.947	Case 2	180.5
19	181.2	0.958	Case 2	182.1
20	181.8	0.914	Case 2	174.2
21	217.3	0.846	Case 2	192.8
22	214.9	0.889	Case 2	200.3
23	215.5	0.912	Case 2	206.2
24	68.9	0.946	Case 2	101.1
25	80.2	0.888	Case 2	110.5
26	31.0	0.869	Case 2	40.5
27	31.0	0.869	Case 2	40.5
28	101.9	0.873	Case 2	127.3
29	101.9	0.873	Case 2	127.3
30	40.2	0.914	Case 2	53.1
31	40.2	0.914	Case 2	53.1
32	96.6	0.860	Case 2	137.8
33	96.6	0.860	Case 2	137.8
34	159.6	0.837	Case 2	170.6
35	159.6	0.837	Case 2	170.6
36	159.6	0.856	Case 2	174.5
37	159.6	0.856	Case 2	174.5
38	15.3	0.960	Case 2	22.8
39	33.1	0.928	Case 2	44.9
40	34.4	0.928	Case 2	45.9
41	34.4	0.928	Case 2	45.9
42	35.1	0.928	Case 2	46.9

Table 6.4 continued

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
43	112.2	0.930	Case 2	161.6
44	116.7	0.930	Case 2	165.4
45	37.4	0.873	Case 2	47.9
46	36.5	0.873	Case 2	46.5
47	35.7	0.951	Case 2	49.5
48	36.4	0.951	Case 2	50.7
49	36.5	0.873	Case 2	46.5
50	36.2	0.873	Case 2	46.0
51	52.4	0.915	Case 2	80.6
52	51.0	0.939	Case 2	85.3
53	112.2	0.895	Case 2	148.9
54	115.0	0.954	Case 2	162.5
55	215.1	0.722	Case 2	220.7
56	215.1	0.911	Case 4	278.5
57	106.5	0.800	Case 2	121.4
58	106.5	0.667	Case 4	101.2
59	231.2	0.803	Case 2	270.6
60	231.2	0.692	Case 4	233.5
61	106.5	0.814	Case 2	123.4
62	106.5	0.814	Case 2	123.4
63	203.2	0.667	Case 2	184.4
64	102.9	0.866	Case 2	120.3
65	103.0	0.690	Case 2	95.8
66	102.9	0.731	Case 4	101.5
67	101.6	0.809	Case 4	111.8
68	101.6	0.743	Case 4	102.6
69	166.0	0.783	Case 4	172.2
70	166.0	0.848	Case 4	186.6
71	141.2	0.683	Case 4	130.9
72	140.8	0.775	Case 4	148.1
73	141.8	0.841	Case 4	161.9
74	142.8	0.797	Case 2	154.4
75	143.5	0.885	Case 2	172.4
76	150.3	0.701	Case 4	155.5
77	121.4	0.820	Case 4	143.4
78	60.4	0.819	Case 4	58.6
79	73.2	0.814	Case 4	73.5
80	75.6	0.793	Case 4	73.2
81	91.7	0.810	Case 4	93.5
82	89.4	0.786	Case 4	83.0
83	110.5	0.822	Case 4	120.7
84	56.8	0.687	Case 4	64.8

Table 6.4 continued

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
85	56.8	0.796	Case 2	75.1
86	113.7	0.687	Case 4	129.6
87	113.7	0.637	Case 4	120.2
88	107.6	0.755	Case 4	109.7
89	107.6	0.754	Case 4	109.5
90	107.6	0.751	Case 4	109.2
91	107.6	0.788	Case 4	114.5
92	107.6	0.788	Case 4	114.5
93	93.4	0.647	Case 4	86.5
94	93.4	0.645	Case 4	86.3
95	93.4	0.644	Case 4	86.1
96	64.8	0.786	Case 4	70.2
97	64.8	0.791	Case 4	70.7
98	64.8	0.788	Case 4	70.3
99	64.8	0.651	Case 4	58.1
100	53.8	0.833	Case 4	58.3
101	53.8	0.834	Case 4	58.4
102	53.8	0.834	Case 4	58.3
103	55.3	0.830	Case 4	60.7
104	55.3	0.829	Case 4	60.6
105	55.3	0.830	Case 4	60.7
106	55.3	0.849	Case 4	62.1
107	53.8	0.833	Case 4	58.3
108	53.8	0.833	Case 4	58.3
109	53.8	0.833	Case 4	58.3
110	55.3	0.828	Case 4	60.6
111	55.3	0.828	Case 4	60.6
112	55.3	0.828	Case 4	60.6
113	55.3	0.828	Case 4	60.6
114	110.5	0.832	Case 2	149.3
115	67.3	0.830	Case 2	90.8
116	61.6	0.858	Case 2	85.8
117	76.0	0.864	Case 2	106.6
118	110.5	0.932	Case 2	167.3
119	67.3	0.913	Case 2	100.0
120	61.6	0.908	Case 2	90.9
121	110.5	0.835	Case 2	149.9
122	67.3	0.830	Case 2	90.8
123	61.6	0.858	Case 2	85.8
124	76.0	0.864	Case 2	106.6
125	110.5	0.933	Case 2	167.5
126	67.3	0.913	Case 2	100.0
127	61.6	0.908	Case 2	90.9

Table 6.5: Summary Strength Results for Welded Angle Connections

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
1	100.0	[2]	82.3	[1]	84.3	[3]
2	101.0	[2]	82.3	[1]	84.4	[3]
3	100.8	[2]	81.8	[1]	83.9	[3]
4	197.4	[2]	160.7	[1]	142.5	[3]
5	100.0	[2]	83.4	[1]	81.1	[3]
6	92.4	[2]	82.7	[1]	80.3	[3]
7	97.6	[2]	81.1	[1]	78.8	[3]
8	176.7	[2]	115.9	[1]	80.7	[3]
9	175.8	[2]	115.9	[1]	67.9	[3]
10	170.9	[2]	115.9	[1]	78.9	[3]
11	175.8	[2]	115.9	[1]	78.9	[3]
12	149.5	[2]	115.9	[1]	67.6	[3]
13	170.0	[2]	115.9	[1]	67.7	[3]
14	222.6	[2]	132.7	[1]	95.0	[3]
15	210.4	[2]	132.7	[1]	85.3	[3]
16	179.0	[2]	132.7	[1]	75.5	[3]
17	178.9	[2]	177.9	[2]	112.9	[3]
18	188.6	[1]	180.5	[2]	112.3	[3]
19	191.1	[2]	181.2	[1]	112.0	[3]
20	168.2	[2]	174.2	[2]	103.5	[3]
21	187.9	[2]	192.8	[2]	122.8	[3]
22	185.0	[2]	200.3	[2]	121.6	[3]
23	207.9	[2]	206.2	[2]	122.0	[3]
24	109.7	[1]	68.9	[1]	45.8	[3]
25	114.9	[2]	80.2	[1]	48.2	[3]
26	46.1	[1]	31.0	[1]	20.2	[3]
27	45.4	[2]	31.0	[1]	19.7	[3]
28	109.9	[2]	101.9	[1]	59.1	[3]
29	104.1	[2]	101.9	[1]	58.2	[3]
30	55.1	[2]	40.2	[1]	27.2	[3]
31	56.2	[1]	40.2	[1]	26.3	[3]
32	159.4	[2]	96.6	[1]	57.2	[3]
33	147.3	[2]	96.6	[1]	57.0	[3]
34	189.3	[2]	159.6	[1]	89.7	[3]
35	170.2	[2]	159.6	[1]	89.6	[3]
36	187.3	[2]	159.6	[1]	89.7	[3]
37	182.8	[2]	159.6	[1]	89.7	[3]
38	23.2	[2]	15.3	[1]	9.0	[3]
39	45.0	[2]	33.1	[1]	21.1	[3]
40	47.4	[2]	34.4	[1]	21.9	[3]
41	45.0	[2]	34.4	[1]	26.0	[3]
42	50.8	[2]	35.1	[1]	22.3	[3]

Table 6.5 continued

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
43	163.4	[2]	112.2	[1]	72.8	[3]
44	172.4	[2]	116.7	[1]	75.6	[3]
45	49.9	[2]	37.4	[1]	20.5	[3]
46	50.4	[2]	36.5	[1]	20.1	[3]
47	51.7	[2]	35.7	[1]	22.9	[3]
48	51.3	[2]	36.4	[1]	23.4	[3]
49	52.2	[2]	36.5	[1]	25.0	[3]
50	50.6	[2]	36.2	[1]	19.9	[3]
51	83.2	[2]	52.4	[1]	30.7	[3]
52	82.7	[2]	51.0	[1]	32.0	[3]
53	150.0	[2]	112.2	[1]	66.7	[3]
54	160.1	[2]	115.0	[1]	79.4	[3]
55	223.7	[2]	215.1	[1]	174.3	[3]
56	265.5	[2]	215.1	[1]	219.5	[3]
57	129.7	[2]	106.5	[1]	68.8	[3]
58	123.0	[2]	101.2	[2]	68.8	[3]
59	281.0	[2]	231.2	[1]	149.1	[3]
60	275.4	[2]	231.2	[1]	149.1	[3]
61	129.7	[2]	106.5	[1]	68.8	[3]
62	125.0	[2]	106.5	[1]	68.8	[3]
63	208.0	[2]	184.4	[2]	153.4	[3]
64	126.3	[2]	102.9	[1]	74.7	[3]
65	109.0	[2]	95.8	[2]	61.8	[3]
66	125.4	[2]	101.5	[2]	74.6	[3]
67	125.0	[2]	101.6	[1]	61.0	[3]
68	116.9	[2]	101.6	[1]	61.0	[3]
69	206.8	[2]	166.0	[1]	114.5	[3]
70	202.6	[2]	166.0	[1]	114.4	[3]
71	180.3	[2]	130.9	[2]	133.5	[3]
72	185.7	[2]	140.8	[1]	143.2	[3]
73	180.1	[2]	141.8	[1]	144.7	[3]
74	185.5	[2]	142.8	[1]	132.4	[3]
75	188.8	[2]	143.5	[1]	146.5	[3]
76	196.7	[2]	150.3	[1]	131.9	[3]
77	175.8	[2]	121.4	[1]	123.1	[3]
78	79.4	[1]	58.6	[2]	62.1	[3]
79	93.1	[2]	73.2	[1]	75.0	[3]
80	110.2	[2]	73.2	[2]	77.4	[3]
81	127.2	[2]	91.7	[1]	93.8	[3]
82	126.1	[2]	83.0	[2]	91.0	[3]
83	160.3	[2]	110.5	[1]	112.9	[3]
84	77.3	[2]	56.8	[1]	35.3	[3]

Table 6.5 continued

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{EXP}, kips	Failure Mode
85	75.8	[2]	56.8	[1]	35.1	[3]
86	170.7	[2]	113.7	[1]	116.3	[3]
87	172.8	[2]	113.7	[1]	107.0	[3]
88	107.6	[2]	107.6	[1]	70.4	[3]
89	106.4	[2]	107.6	[1]	70.4	[3]
90	103.7	[2]	107.6	[1]	70.4	[3]
91	106.9	[2]	107.6	[1]	69.7	[3]
92	104.8	[2]	107.6	[1]	69.7	[3]
93	81.4	[2]	86.5	[2]	60.4	[3]
94	79.8	[2]	86.3	[2]	60.4	[3]
95	78.8	[2]	86.1	[2]	60.4	[3]
96	76.5	[2]	64.8	[1]	41.8	[3]
97	80.4	[2]	64.8	[1]	41.8	[3]
98	77.6	[2]	64.8	[1]	41.8	[3]
99	76.5	[2]	58.1	[2]	41.7	[3]
100	63.8	[2]	53.8	[1]	35.5	[3]
101	64.8	[2]	53.8	[1]	35.5	[3]
102	64.7	[2]	53.8	[1]	35.5	[3]
103	68.4	[2]	55.3	[1]	36.7	[3]
104	65.7	[2]	55.3	[1]	36.8	[3]
105	66.6	[2]	55.3	[1]	36.7	[3]
106	67.1	[2]	55.3	[1]	36.7	[3]
107	61.0	[2]	53.8	[1]	34.6	[3]
108	59.0	[2]	53.8	[1]	34.6	[3]
109	64.9	[2]	53.8	[1]	34.6	[3]
110	65.8	[2]	55.3	[1]	36.0	[3]
111	62.3	[2]	55.3	[1]	36.0	[3]
112	67.2	[2]	55.3	[1]	36.0	[3]
113	64.1	[2]	55.3	[1]	36.0	[3]
114	123.9	[2]	110.5	[1]	71.9	[3]
115	80.9	[2]	67.3	[1]	50.9	[3]
116	80.0	[2]	61.6	[1]	48.4	[3]
117	98.9	[2]	76.0	[1]	57.3	[3]
118	139.4	[2]	110.5	[1]	90.8	[3]
119	105.7	[2]	67.3	[1]	60.7	[3]
120	91.1	[2]	61.6	[1]	54.7	[3]
121	122.3	[2]	110.5	[1]	74.8	[3]
122	78.7	[2]	67.3	[1]	53.5	[3]
123	71.9	[2]	61.6	[1]	51.8	[3]
124	81.6	[2]	76.0	[1]	61.9	[3]
125	138.9	[2]	110.5	[1]	96.3	[3]
126	84.5	[2]	67.3	[1]	63.8	[3]
127	77.3	[2]	61.6	[1]	58.1	[3]

[1] tensile yield; [2] tensile rupture; [3] member strain

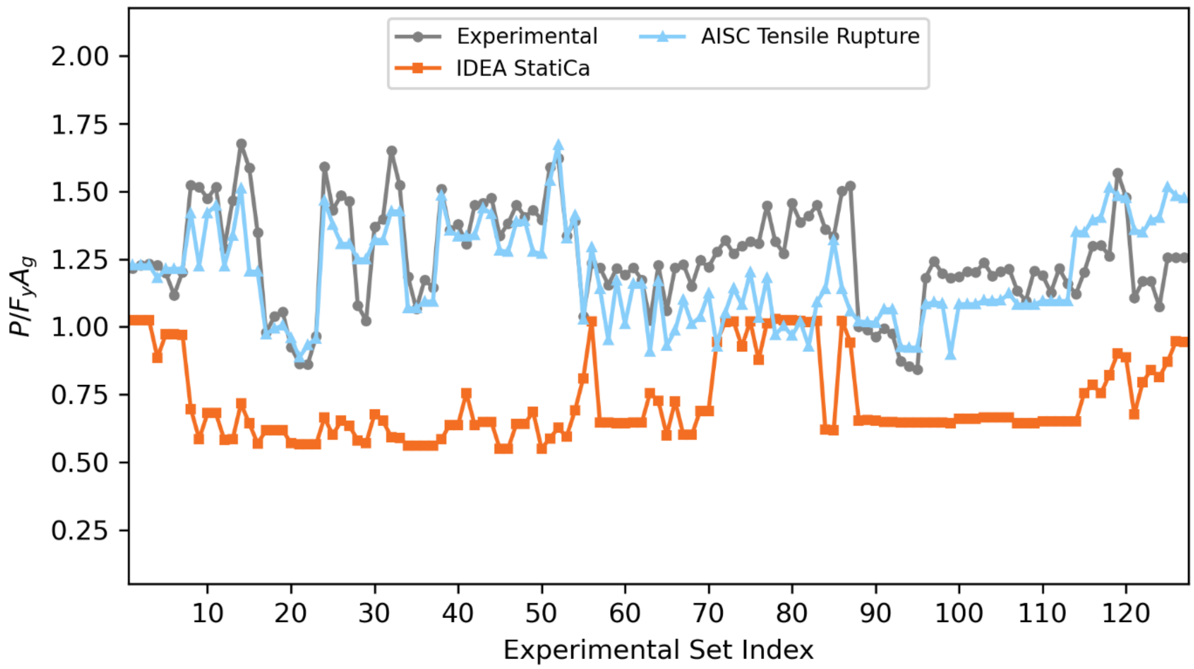


Figure 6-5: Normalized Strength Results for Welded Angle Connections

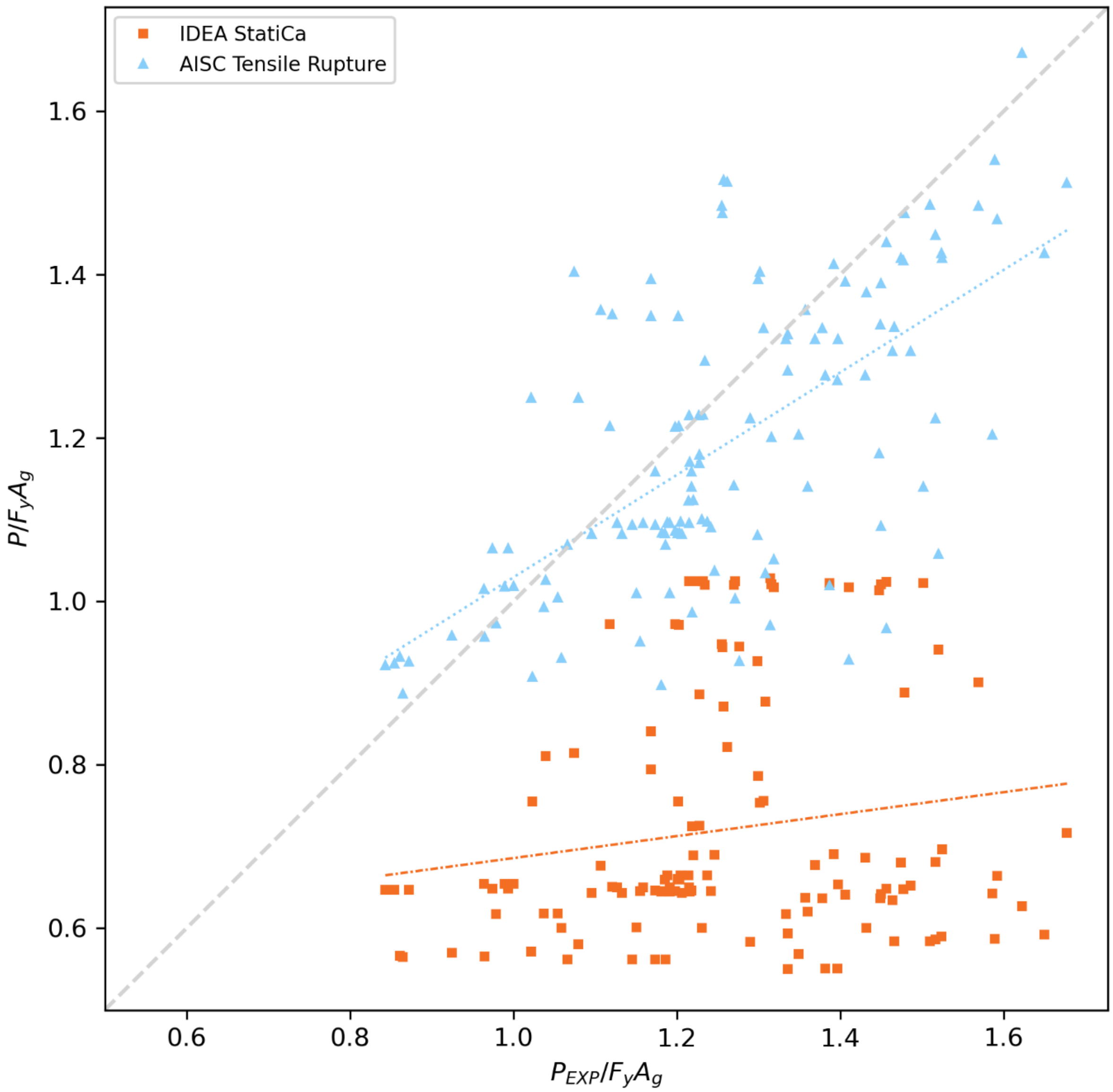


Figure 6-6: Normalized Strength vs. Normalized Experimental Strength for Welded Angle Connections

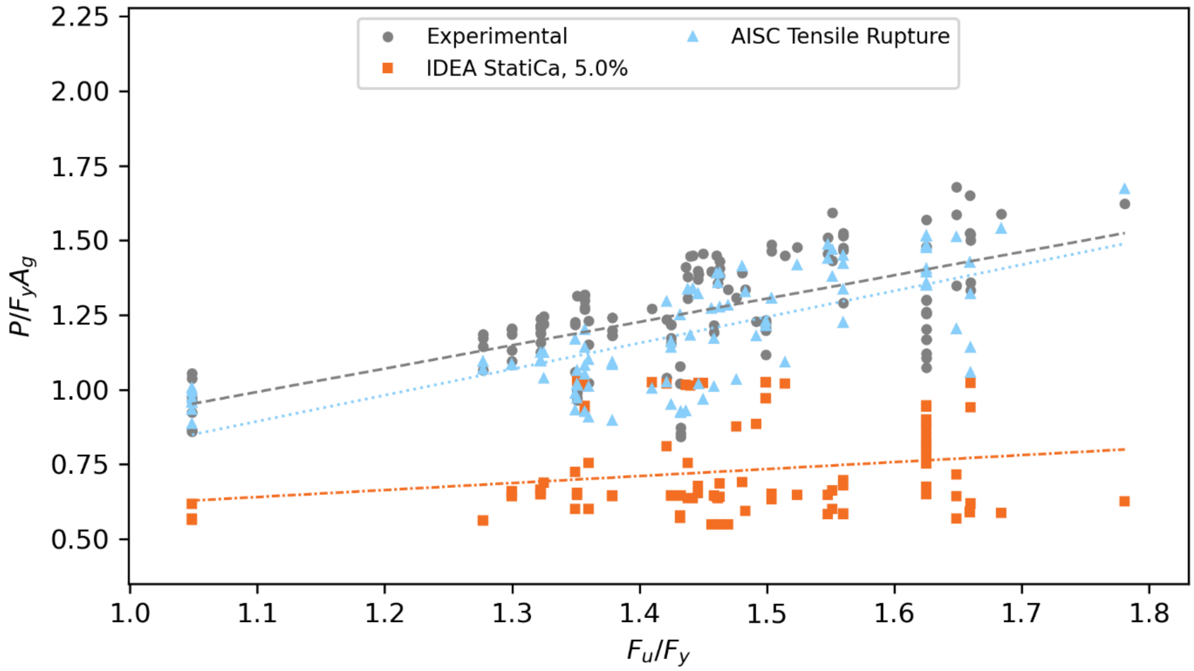


Figure 6-7: Normalized Strength vs. Material Strength Ratio for Welded Angle Connections

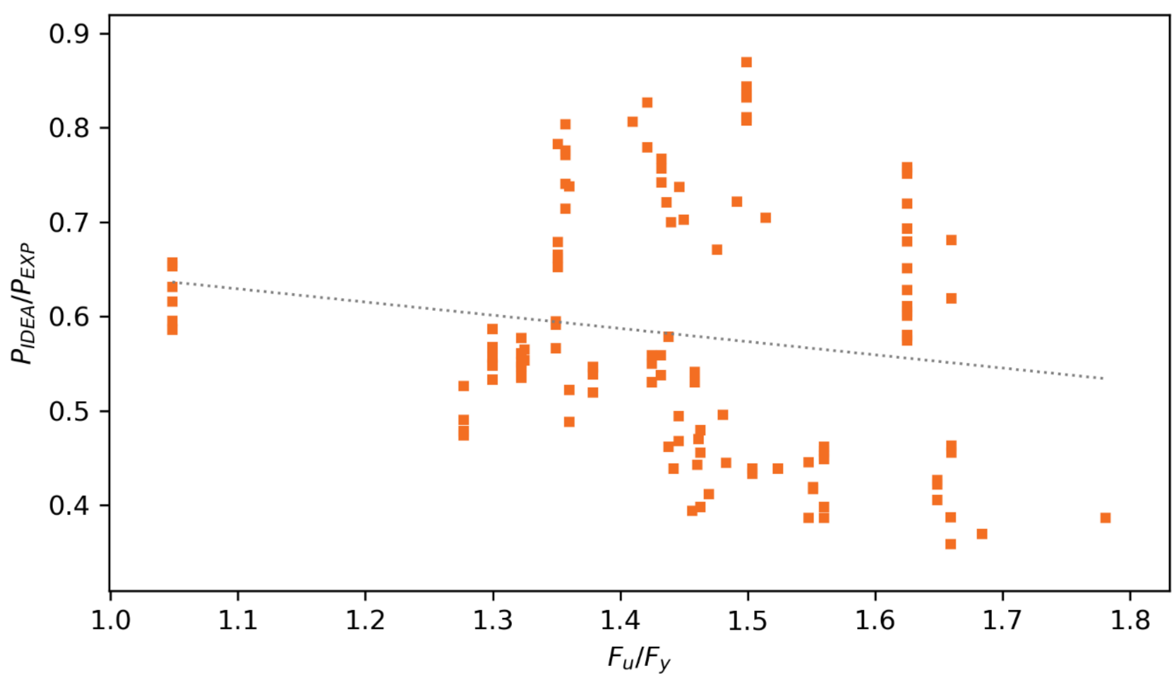


Figure 6-8: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Welded Angle Connections

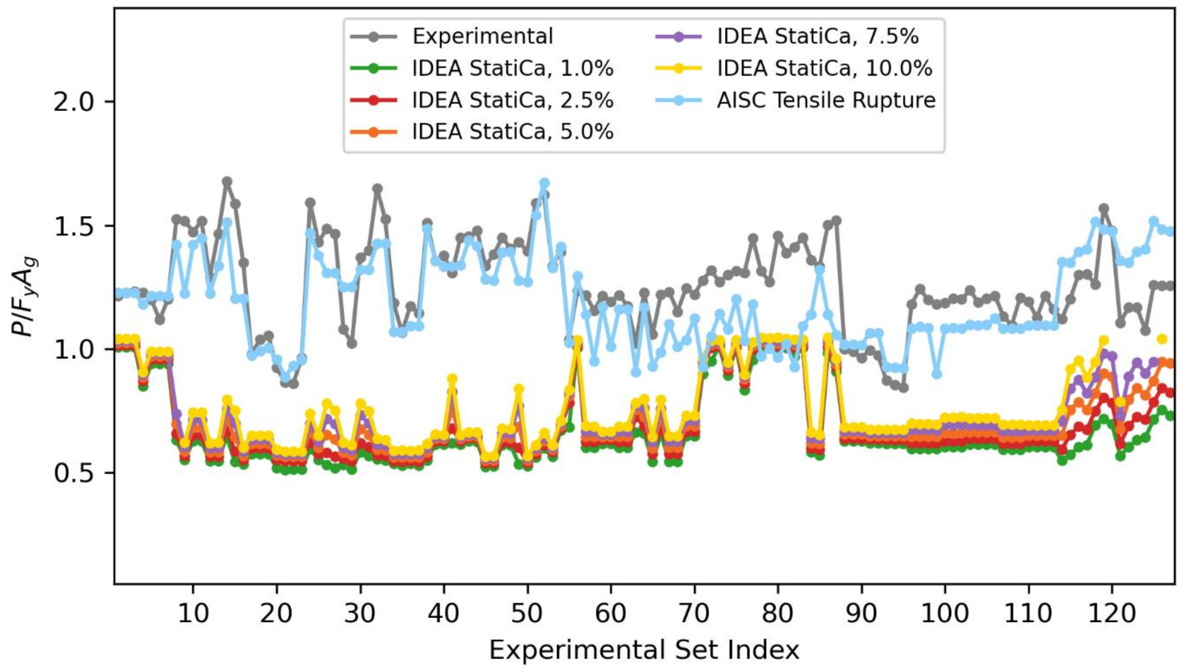


Figure 6-9: Normalized Strength Results for Welded Angle Connections Including Various Plastic Strain Limits for IDEA StatiCa

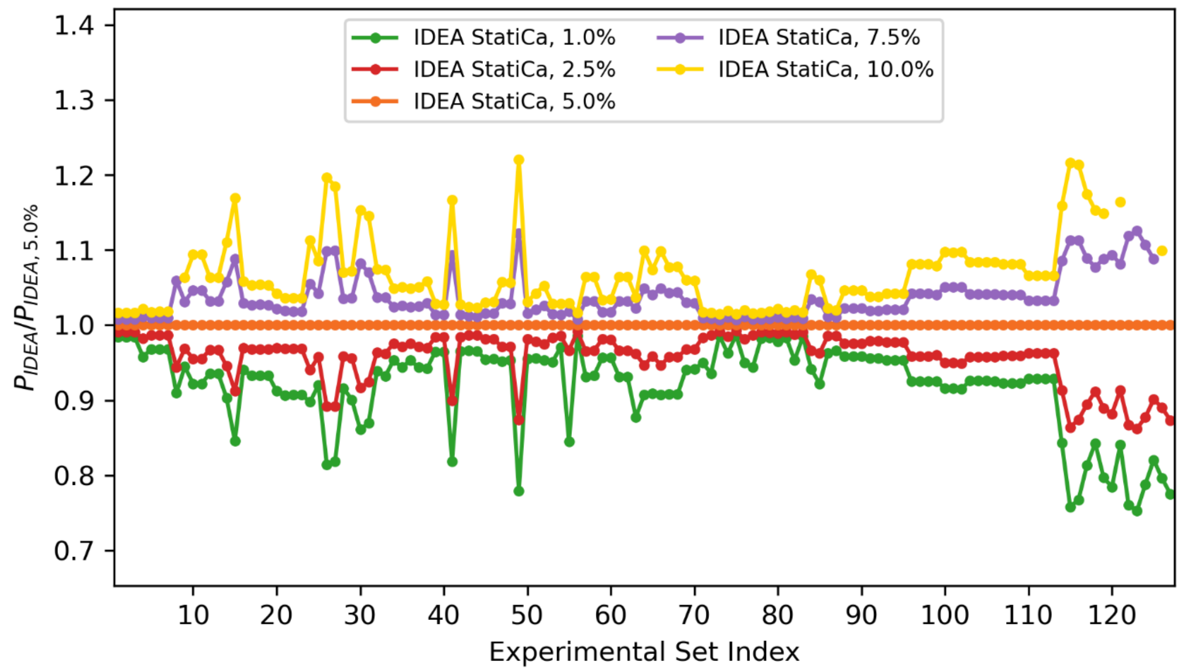


Figure 6-10: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Welded Angle Connections

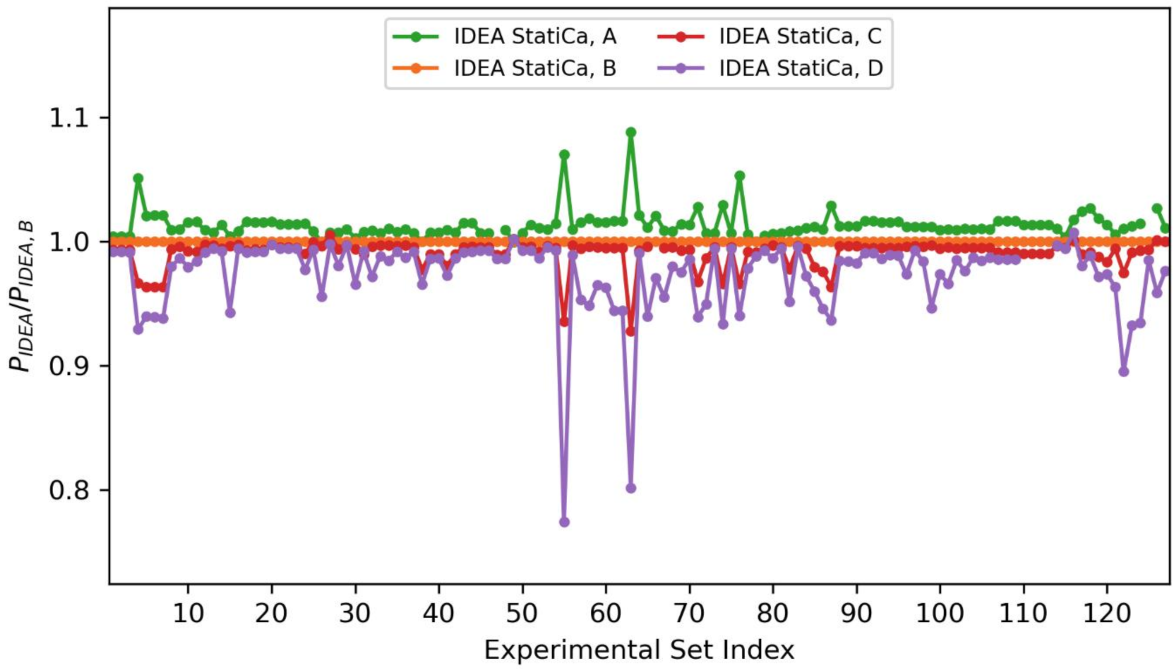


Figure 6-11: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Welded Angle Connections

Table 6.6: Summary Statistics of the Test-to-Predicted Ratio for Welded Angle Connections

Test-to-Predicted Ratio	Average	Standard Deviation	Coefficient of Variation
$P_{EXP}/P_{RUPTURE}$ (AISC)	1.065	0.138	0.130
P_{EXP}/P_{IDEA}	1.804	0.400	0.222

6.2.3 Discussion

Most of the experimental specimens were reported to have failed by tensile rupture, with the others failing by tensile yield. The proportion of specimens for which tensile rupture controlled the AISC strength calculations was smaller (Table 6.5).

The AISC tensile rupture strength calculations compared well the experimental strengths for most of the specimens (Figure 6-5). However, for the cases where tensile rupture was most critical (i.e., when P_{EXP} was less then F_yA_g), the AISC strength equations slightly overestimated the strength (Figure 6-6). The IDEA StatiCa strengths were low in comparison both the experiments and the AISC strength equations for most of the specimens. P_{IDEA} was at most 10% greater than the P_{AISC} and never greater than P_{EXP}. The most conservative case results from experimental set specimen 45 with about a 45% difference from P_{AISC}. One potential cause for the conservative results for IDEA StatiCa is that first order analyses were used, and the bending moment caused by the eccentricity between the centroid of the gusset plate and the centroid of the angle detracted from the axial strength. Evidence for this possibility is seen in the difference in results between single and double angle specimens. Second-order (i.e., geometric nonlinear) analyses may result a significantly closer comparison between IDEA StatiCa and the other strengths.

With respect to the material strength ratio F_u/F_y, clear increases with increasing F_u/F_y are seen for the experimental and AISC strengths (Figure 6-7). As expected, when normalized by F_yA_g, the IDEA StatiCa strength does not significantly vary with F_u/F_y. Correspondingly, the ratio P_{IDEA}/P_{EXP} is seen to decrease with increasing F_u/F_y (Figure 6-8).

The variation of P_{IDEA} with plastic strain limit exhibited the expected trend of increasing strength with increasing plastic strain limit and vice versa (Figure 6-9 and Figure 6-10). There are some gaps in the data where the Python script was unable to determine the maximum permitted applied load. The results in Figure 6-10 are notably symmetric with, for example, the increase in strength using a 10% plastic strain limit being approximately the same as the decrease in strength using a 1% plastic strain limit.

The variation of P_{IDEA} with mesh parameters showed minimal changes for most specimens (Figure 6-11), indicating convergence of the mesh. However, there are a few cases where the change from mesh set ‘C’ to the mesh set ‘D’ is quite significant. For example, this is seen in specimens with experimental indices 55, 63 and 122.

6.3 Reliability Analysis

6.3.1 Description of Reliability Set

The reliability set consists of selected connections based on varying desired parameters. For the welded angle set it was desired to vary the following: A/B ratio (connected leg length to unconnected leg length), thickness, and weld configuration. There were three weld configurations that were selected for these specimens, 2 longitudinal 4 in. welds, 2 longitudinal 8 in. welds, and a balanced weld configuration. The reliability set specimen parameters are outlined in Table 6.7.

Table 6.7: Welded Angle Connection Reliability Set Parameters

Index	***A*, in.**	***B*, in.**	***t*, in.**	**Material Grade**	**F_y, ksi**	**F_u, ksi**	**L_{toe}, in.**	**L_{heel}, in.**	**Weld Size, in.**	**Balanced/ Unbalanced**
1	4.0	6.0	0.375	A36 (Angle)	36	58	4.00	4.00	0.313	Unbalanced
2	4.0	6.0	0.500	A36 (Angle)	36	58	4.00	4.00	0.313	Unbalanced
3	4.0	6.0	0.375	A36 (Angle)	36	58	8.00	8.00	0.313	Unbalanced
4	4.0	6.0	0.500	A36 (Angle)	36	58	8.00	8.00	0.313	Unbalanced
5	4.0	6.0	0.375	A36 (Angle)	36	58	2.50	8.25	0.313	Balanced
6	4.0	6.0	0.500	A36 (Angle)	36	58	3.50	10.75	0.313	Balanced
7	6.0	4.0	0.375	A36 (Angle)	36	58	4.00	4.00	0.313	Unbalanced
8	6.0	4.0	0.500	A36 (Angle)	36	58	4.00	4.00	0.313	Unbalanced
9	6.0	4.0	0.375	A36 (Angle)	36	58	8.00	8.00	0.313	Unbalanced
10	6.0	4.0	0.500	A36 (Angle)	36	58	8.00	8.00	0.313	Unbalanced
11	6.0	4.0	0.375	A36 (Angle)	36	58	3.50	7.25	0.313	Balanced
12	6.0	4.0	0.500	A36 (Angle)	36	58	4.75	9.50	0.313	Balanced
13	4.0	4.0	0.375	A36 (Angle)	36	58	4.00	4.00	0.313	Unbalanced
14	4.0	4.0	0.500	A36 (Angle)	36	58	4.00	4.00	0.313	Unbalanced
15	4.0	4.0	0.375	A36 (Angle)	36	58	8.00	8.00	0.313	Unbalanced
16	4.0	4.0	0.500	A36 (Angle)	36	58	8.00	8.00	0.313	Unbalanced
17	4.0	4.0	0.375	A36 (Angle)	36	58	2.50	6.25	0.313	Balanced
18	4.0	4.0	0.500	A36 (Angle)	36	58	5.00	11.75	0.313	Balanced

6.3.2 Results

The strength results for the reliability set of welded angle specimens is provided in Table 6.8. The nominal and design strengths for the AISC *Specification* (2016) is provided; however, only the controlling design strength was used in the reliability analysis. As for the IDEA StatiCa strength results, P_{IDEA} was based on the use of all default settings (mesh parameter set 'B', 5.0% plastic strain limit, and all resistance

factors). When comparing P_{IDEA} to the controlling design strength according to the AISC equations (ϕP_{AISC}), P_{IDEA} resulted in a larger strength in only two specimens with a 10% and 7% increased strength for reliability set specimen indices 1 and 2, respectively. For all other specimens, IDEA StatiCa indicated decreased strengths compared to the AISC code equations with an average of 26% decreased strength.

Table 6.8: Summary Strength Results for Welded Angle Connection Reliability Set

Index	AISC				IDEA StatiCa
	P_{YIELD}, kips	$P_{RUPTURE}$, kips	ϕP_{YIELD}, kips	$\phi P_{RUPTURE}$, kips	P_{IDEA}, kips
1	130.0	87.0	117.0	65.3	72.2
2	171.0	116.0	153.9	87.0	92.9
3	130.0	146.6	117.0	110.0	71.8
4	171.0	191.4	153.9	143.5	92.4
5	130.0	113.3	117.0	85.0	72.4
6	171.0	180.0	153.9	135.0	92.8
7	130.0	130.5	117.0	97.9	73.5
8	171.0	174.0	153.9	130.5	93.4
9	130.0	155.8	117.0	116.8	82.1
10	171.0	203.6	153.9	152.7	104.1
11	130.0	130.5	117.0	97.9	83.9
12	171.0	192.1	153.9	144.1	105.9
13	103.0	89.3	92.7	66.9	60.0
14	135.0	116.0	121.5	87.0	76.2
15	103.0	131.5	92.7	98.6	59.9
16	135.0	171.2	121.5	128.4	75.9
17	103.0	96.2	92.7	72.2	61.3
18	135.0	173.7	121.5	130.2	77.0

The reliability index, β, results are provided in Figure 6-12. Connections with specimen indices 3, 4, 6, 11, 12, 15, 16, and 18 all returned very large values for β indicating zero failures in 1,000,000 simulations. In general, for this particular set of reliability specimens IDEA StatiCa provided more reliable results than the AISC *Specification* (2016). Connections with specimen indices 1 and 2 were the only specimens where the AISC strength equations resulted in more reliable results than IDEA StatiCa. These specimens have the smaller *A*/*B* ratio resulting in a larger eccentricity. These two specimens also have the smallest unbalanced weld length. The larger eccentricity coupled with the smaller unbalanced weld length could be reason for this difference. IDEA StatiCa resulted in a β-value range of 3.36 to a large value. Whereas AISC resulted in a range of 3.63 to 3.77.

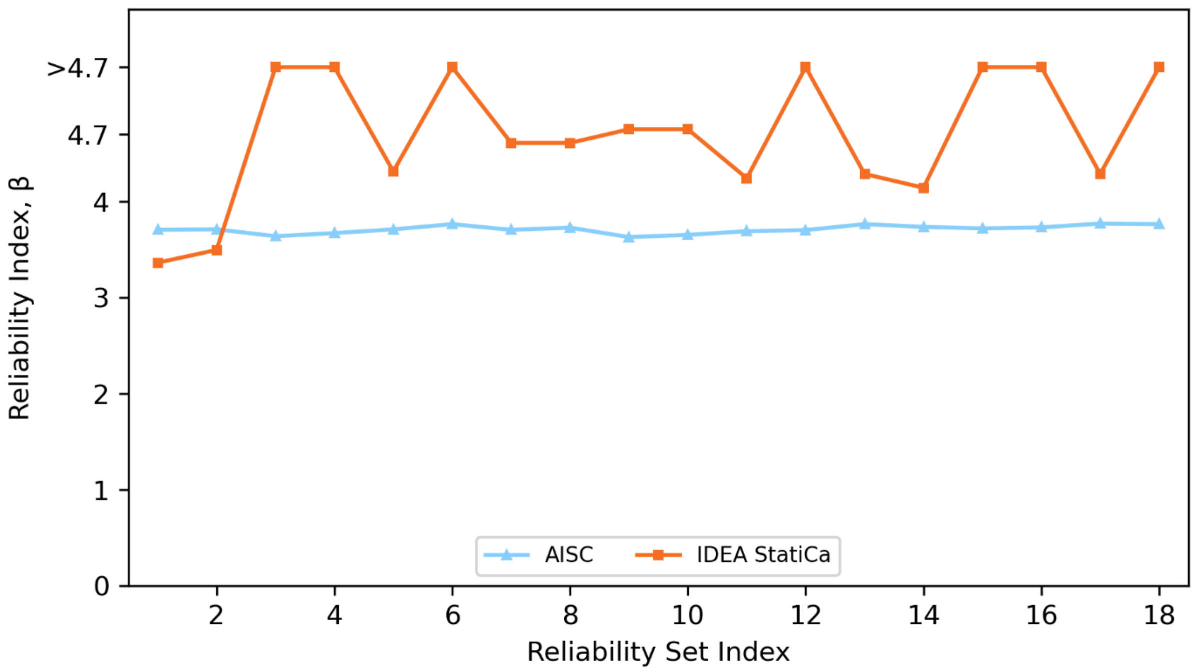


Figure 6-12: Reliability Index for Welded Angle Connections

Chapter 7:
BOLTED ANGLES

7.1 Description of Connection

Single and double angle tension members bolted or riveted to a gusset plate are evaluated in this chapter. A typical schematic with relevant terminology is shown in Figure 7-1. For the double angle specimens, the angles are identical in cross section and material. Connections with both regular and staggered bolt patterns are evaluated in this chapter, but in separate sections.

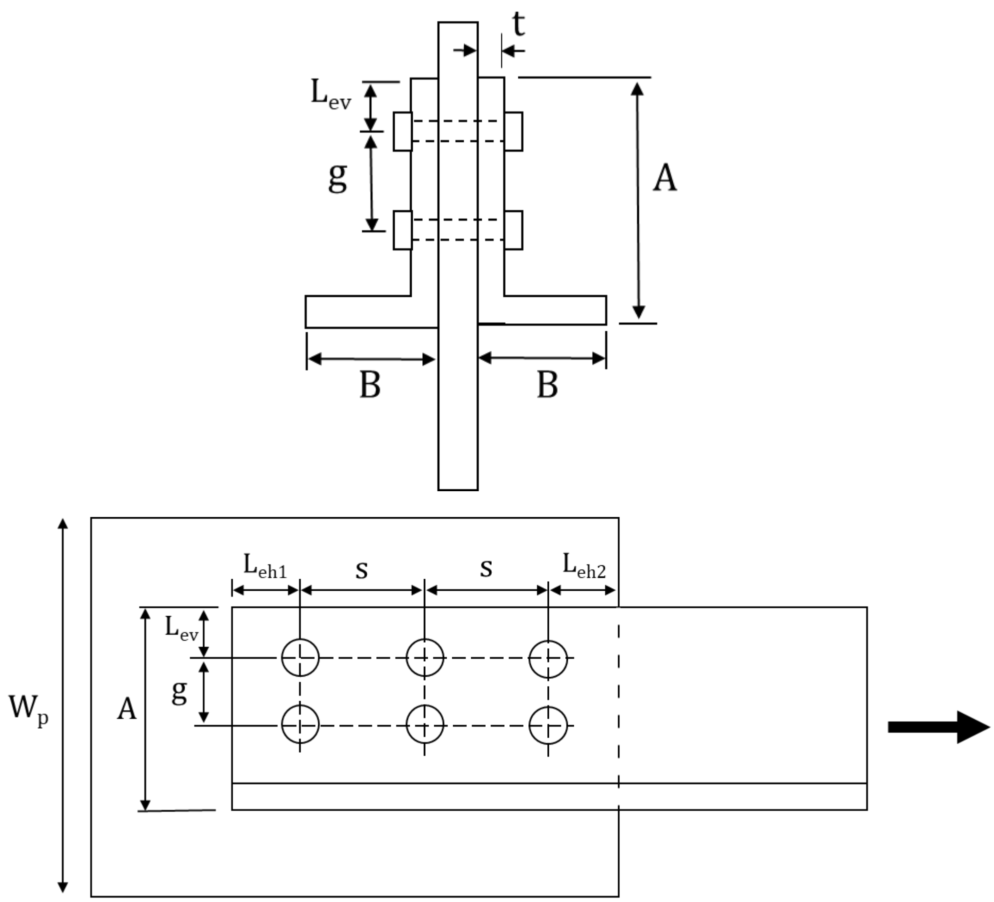


Figure 7-1: Schematic of Bolted Angle Connection

For these connections, the applicable shear lag factor cases outlined in Section D3 of the AISC *Specification* (2016) and shown in Figure 3-3 include cases 2 and 8 as well as the lower limit defined in Section D3. The lower limit on the shear lag factor (the area of connected elements divided by the gross area) was checked, but never controlled. Case 2 is applicable to tension members where the load is transmitted to some but not all of the cross-section elements through fasteners (or welds). Shear lag factor for Case 2 is:

$$U = 1 - \frac{\bar{x}}{l} \tag{7-1}$$

where $\bar{x}$ is the eccentricity of the connection (i.e., the distance between the centroid of the angle to the faying surface), and l is the length of the connection (i.e., center of first bolt to center of last bolt along the longitudinal axis of the member).

Case 8 is applicable to specimens with three or more fasteners in the direction of loading. For this case U = 0.60 for single and double angles with three fasteners per line the direction of loading and U = 0.80 for single and double angles with four or more fasteners per line the direction of loading. The shear lag factor was taken as the larger of Case 2 and Case 8 as permitted by the AISC *Specification*.

The gross area and centroid were calculated based on measured dimensions neglecting leg-to-leg and toe fillets. For all bolt and rivet holes, an additional 1/16 in. for damage was added to the diameter when computing net area in accordance with Section B4.3b of the AISC *Specification* (2016). Note, however, that IDEA StatiCa does not consider the additional 1/16 in.

7.2 Comparison to Experimental Results

7.2.1 Description of Experimental Specimens

A total of 56 welded angle specimens (with regular bolt patterns) from 5 references were identified for evaluation in this work as detailed in Table 7.1. Some of the specimens described in these references were not included. A majority of specimens reported in Epstein (1992) were removed from the experimental set for a variety of reasons. A total of 22 specimens were removed due to failures classified as block shear rupture (specimens 1, 2, 3, 4, 9, 10, 11, 17, 18, 19, 21, 25, 26, 27, 29, 30, 32, 33, 34, 35, 37, and 38) and three specimens failed in a combination of block shear and bolt shear (specimens 6, 14, and 22). Additionally, seven specimens had staggered bolt configurations and are examined in the following section (specimens 5, 12, 13, 20, 28, 31, and 36). McKibben (1907) contained a variety of different bolt and angle configurations, resulting in the removal of 45 total specimens. There were 39 specimens with non-regular angle configurations (4, 5, 6, 10, 11, 12, 16, 17, 18, 19, 20, 21, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 52, 53, 54, 58, 59, and 60) and 6 specimens with staggered bolt patterns (46, 47, 48, 49, 50, and 51). Lastly, only two specimens were considered applicable to this study from Greiner (1897). The remaining specimens were examined in the following section since they included staggered bolt patterns (47 B, 48 B) or due to non-regular angle configurations (45 AX, 46 AX, 49 BX, and 50 BX).

Table 7.1: Count of Bolted Angle Connections by Reference

Reference	Specimen Count
Kulak and Wu (1997)	24
Ke et al. (2018)	9
Epstein (1992)	6
McKibben (1907)	15
Greiner (1897)	2
Total	**56**

A detailed description of each specimen is provided in Table 7.2 and Table 7.3. The terminology of these tables is consistent with Figure 7-1. The first table provides information only on the angle members whereas the second table provides information solely pertaining to the fasteners. For specimens with only one row of bolts, the bolt gage dimension, *g*, is not defined and is indicated by 'N/A'. This is likewise for specimens with only one bolt per row and the bolt spacing, 's'. If provided, the diameter of the bolt or rivet hole was taken as reported. Otherwise, bolt holes were taken as indicated by Table J3.3 of the AISC *Specification* (2016). For rivets, the diameter of the hole was taken as the rivet diameter unless specifically reported by the experimentalist.

In IDEA StatiCa, the gusset plates and angles were all modeled as members. The bearing member was always selected as the gusset plate for all bolted angles. An example modeled in IDEA StatiCa is provided in Figure 7-2. The typical process for modelling the specimens is shown in Figure 7-3.

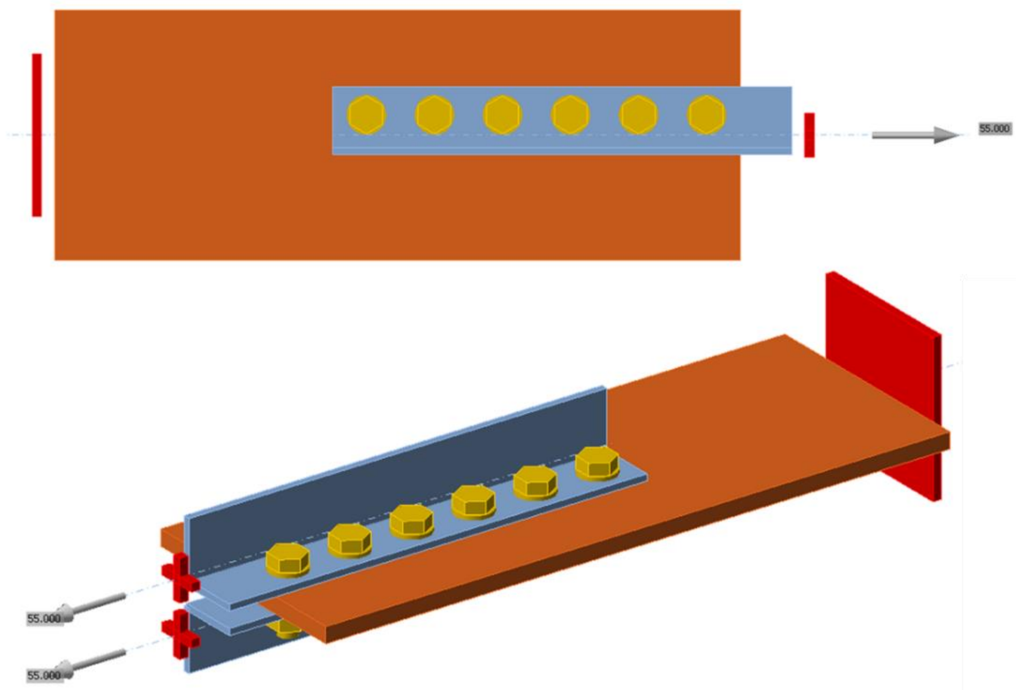


Figure 7-2: Typical Bolted Angle Connection Model in IDEA StatiCa

Table 7.2: Bolted Angle Connection Experimental Specimen Parameters

Index	Reference	Specimen	# of angles	*F_y, ksi	*F_u, ksi	*A, in.	*B, in.	*t, in.
1	Kulak and Wu (1997)	S1	1	49.3	76.0	4.016	4.016	0.257
2	Kulak and Wu (1997)	S2	1	48.8	76.5	4.016	4.016	0.256
3	Kulak and Wu (1997)	S3	1	48.3	76.0	4.016	3.976	0.257
4	Kulak and Wu (1997)	S4	1	46.7	68.6	2.996	2.945	0.193
5	Kulak and Wu (1997)	S5	1	47.4	69.8	4.055	2.906	0.254
6	Kulak and Wu (1997)	S6	1	47.0	69.2	2.933	4.055	0.252
7	Kulak and Wu (1997)	S7	1	47.1	69.3	4.055	2.937	0.253
8	Kulak and Wu (1997)	S8	1	48.4	70.8	2.988	2.000	0.378
9	Kulak and Wu (1997)	S9	1	49.4	70.7	3.028	1.988	0.187
10	Kulak and Wu (1997)	S10	1	49.3	70.4	3.043	1.984	0.185
11	Kulak and Wu (1997)	S11	1	49.1	70.7	3.031	1.988	0.186
12	Kulak and Wu (1997)	D1-1	2	49.3	76.0	4.016	4.016	0.256
13	Kulak and Wu (1997)	D1-2	2	48.9	76.5	4.016	4.016	0.256
14	Kulak and Wu (1997)	D1-3	2	48.3	76.0	4.016	3.976	0.257
15	Kulak and Wu (1997)	D2	2	46.7	68.6	3.004	2.945	0.194
16	Kulak and Wu (1997)	D3-1	2	47.4	69.8	4.055	2.909	0.254
17	Kulak and Wu (1997)	D3-2	2	47.0	69.2	4.055	2.929	0.253
18	Kulak and Wu (1997)	D4-1	2	47.5	69.8	2.921	4.055	0.255
19	Kulak and Wu (1997)	D4-2	2	47.0	69.2	2.937	4.055	0.253
20	Kulak and Wu (1997)	D5	2	47.2	69.0	4.055	2.933	0.253
21	Kulak and Wu (1997)	D6	2	48.4	70.8	2.984	2.000	0.375
22	Kulak and Wu (1997)	D7	2	49.4	70.7	3.020	1.988	0.185
23	Kulak and Wu (1997)	D8	2	49.3	70.4	3.039	1.992	0.185
24	Kulak and Wu (1997)	D9	2	49.1	70.7	3.035	1.992	0.185
25	Ke et al. (2018)	A1-60L	1	110.4	115.7	3.178	2.391	0.319
26	Ke et al. (2018)	A1-75L	1	110.4	115.7	3.129	2.374	0.317
27	Ke et al. (2018)	A1-90L	1	110.4	115.7	3.172	2.359	0.317
28	Ke et al. (2018)	A1-75S	1	110.4	115.7	2.374	3.129	0.315
29	Ke et al. (2018)	A2-60S	1	110.4	115.7	2.480	3.898	0.318
30	Ke et al. (2018)	A2-75S	1	110.4	115.7	2.559	3.898	0.313
31	Ke et al. (2018)	A2-90S	1	110.4	115.7	2.520	3.898	0.319
32	Ke et al. (2018)	B1-75L	1	42.4	65.7	3.071	2.362	0.235
33	Ke et al. (2018)	B2-75S	1	42.4	65.7	2.559	3.976	0.220
34	Epstein (1992)	7	2	51.6	74.8	6.000	6.000	0.313
35	Epstein (1992)	8	2	52.0	74.6	6.000	6.000	0.313
36	Epstein (1992)	15	2	46.5	64.9	6.000	4.000	0.313
37	Epstein (1992)	16	2	48.1	65.7	6.000	4.000	0.313
38	Epstein (1992)	23	2	45.6	69.3	6.000	3.500	0.313
39	Epstein (1992)	24	2	46.8	69.7	6.000	3.500	0.313
40	McKibben (1907)	1	1	33.2	59.3	3.500	3.000	0.375
41	McKibben (1907)	2	1	33.2	59.3	3.500	3.000	0.375
42	McKibben (1907)	3	1	33.2	59.3	3.500	3.000	0.375

Table 7.2 continued

Index	Reference	Specimen	# of angles	*F_y, ksi	*F_u, ksi	*A, in.	*B, in.	*t, in.
43	McKibben (1907)	7	1	31.6	54.0	4.000	3.000	0.375
44	McKibben (1907)	8	1	31.6	54.0	4.000	3.000	0.375
45	McKibben (1907)	9	1	31.6	54.0	4.000	3.000	0.375
46	McKibben (1907)	13	1	31.6	54.0	4.000	3.000	0.375
47	McKibben (1907)	14	1	31.6	54.0	4.000	3.000	0.375
48	McKibben (1907)	15	1	31.6	54.0	4.000	3.000	0.375
49	McKibben (1907)	22	2	34.8	61.0	3.000	3.000	0.313
50	McKibben (1907)	23	2	34.8	61.0	3.000	3.000	0.313
51	McKibben (1907)	24	2	34.8	61.0	3.000	3.000	0.313
52	McKibben (1907)	55	2	34.8	61.0	3.000	3.000	0.313
53	McKibben (1907)	56	2	34.8	61.0	3.000	3.000	0.313
54	McKibben (1907)	57	2	34.8	61.0	3.000	3.000	0.313
55	Greiner (1897)	43 A	1	38.9	59.4	3.500	3.500	0.375
56	Greiner (1897)	44 A	1	38.9	59.4	3.500	3.500	0.375

* indicates measured value

Table 7.3: Bolted Angle Connection Experimental Specimen Fastener Details

Index	**Bolts/ Rivets**	**Bolt Material**	***d*, in.**	**# of rows of fasteners**	**# of fasteners per row**	***s*, in.**	**L_{eh}, in.**	***g*, in.**	**L_{ev}, in.**
1	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.516
2	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.516
3	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.516
4	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.246
5	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.555
6	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.183
7	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.555
8	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.238
9	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.278
10	Bolts	A490	0.875	1	4	3.000	1.500	N/A	1.293
11	Bolts	A490	0.875	1	2	3.000	1.500	N/A	1.281
12	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.516
13	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.516
14	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.516
15	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.254
16	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.555
17	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.555
18	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.171
19	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.187
20	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.555
21	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.234
22	Bolts	A490	0.875	1	6	3.000	1.500	N/A	1.270
23	Bolts	A490	0.875	1	4	3.000	1.500	N/A	1.289
24	Bolts	A490	0.875	1	2	3.000	1.500	N/A	1.285
25	Bolts	Gr. 12.9	0.866	1	5	2.362	2.362	N/A	1.575
26	Bolts	Gr. 12.9	0.866	1	5	2.953	2.362	N/A	1.575
27	Bolts	Gr. 12.9	0.866	1	5	3.543	2.362	N/A	1.575
28	Bolts	Gr. 12.9	0.866	1	5	2.953	2.362	N/A	1.181
29	Bolts	Gr. 12.9	0.866	1	5	2.362	2.362	N/A	1.280
30	Bolts	Gr. 12.9	0.866	1	5	2.953	2.362	N/A	1.280
31	Bolts	Gr. 12.9	0.866	1	5	3.543	2.362	N/A	1.280
32	Bolts	Gr. 12.9	0.866	1	5	2.953	2.362	N/A	1.575
33	Bolts	Gr. 12.9	0.866	1	5	2.953	2.362	N/A	1.280
34	Bolts	A490X	0.750	2	3	3.000	1.500	2.500	1.250
35	Bolts	A490X	0.750	2	4	3.000	1.500	2.500	1.250
36	Bolts	A490X	0.750	2	3	3.000	1.500	2.500	1.250
37	Bolts	A490X	0.750	2	4	3.000	1.500	2.500	1.250
38	Bolts	A490X	0.750	2	3	3.000	1.500	2.500	1.250
39	Bolts	A490X	0.750	2	4	3.000	1.500	2.500	1.250
40	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
41	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
42	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500

Table 7.3 continued

Index	Bolts/ Rivets	Bolt Material	*d*, in.	# of rows of fasteners	# of fasteners per row	*s*, in.	L_{eh}, in.	*g*, in.	L_{ev}, in.
43	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
44	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
45	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
46	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
47	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
48	Rivets	NP	0.875	1	6	2.625	1.500	N/A	1.500
49	Rivets	NP	0.875	1	6	3.000	1.500	N/A	1.250
50	Rivets	NP	0.875	1	6	3.000	1.500	N/A	1.250
51	Rivets	NP	0.875	1	6	3.000	1.500	N/A	1.250
52	Rivets	NP	0.875	1	6	3.000	1.500	N/A	1.250
53	Rivets	NP	0.875	1	6	3.000	1.500	N/A	1.250
54	Rivets	NP	0.875	1	6	3.000	1.500	N/A	1.250
55	Rivets	NP	0.875	1	5	2.750	1.500	N/A	1.750
56	Rivets	NP	0.875	1	5	2.750	1.500	N/A	1.750

4. Create members:
    a. Member 1: Gusset plate
        i. Geometrical Type set to “Ended”
        ii. Offset ex set to the negative of the connection length
            1. Connection length is 2 times L_{eh} + (the number of bolts per row minus one) times the bolt spacing
    b. Member 2, 3 (if applicable): Angle(s)
        i. β set to 180º
        ii. Aligned set to “To member plate”
        iii. Aligned plate set to “[angle name] | bottom flange 1”
        iv. Related plate set to “[gusset plate name] | bottom flange 1”
        v. Model type set to “N-Vy-Vz”
5. Create “Bolt Grid” operation:
    a. Fastener set to “Bolts”
    b. Items count set to the 2 for single angles and 3 for double angles
    c. Item 1, 2, 3, set to each member created in step 1
    d. Type set to the diameter and bolt type indicated by report
        i. Bolt type typically selected as A490 bolts due to the removal of bolt failures
    e. Coord. System set to “Orthogonal”
    f. Rows, Positions set to dimension indicated by reports
    g. Grid set to “Regular”
    h. Shear force transfer set to “Bearing – tension/shear interaction”

Figure 7-3: Modelling Process for Bolted Angle Connections

7.2.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 7.4: AISC Calculation Results for Bolted Angle Connections

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
1	98.4	0.927	Case 2	122.5
2	97.0	0.927	Case 2	122.6
3	96.4	0.928	Case 2	122.4
4	51.8	0.947	Case 2	59.5
5	80.7	0.953	Case 2	96.5
6	80.1	0.915	Case 2	92.0
7	80.3	0.953	Case 2	95.8
8	84.4	0.964	Case 2	93.3
9	44.7	0.969	Case 2	49.2
10	44.2	0.949	Case 2	47.5
11	44.2	0.845	Case 2	42.6
12	196.3	0.927	Case 2	244.4
13	194.5	0.927	Case 2	245.8
14	192.1	0.928	Case 2	243.9
15	104.1	0.947	Case 2	119.5
16	161.8	0.953	Case 2	193.4
17	160.6	0.953	Case 2	191.9
18	162.6	0.915	Case 2	186.2
19	160.0	0.915	Case 2	183.7
20	161.2	0.953	Case 2	191.3
21	167.2	0.964	Case 2	184.7
22	88.3	0.969	Case 2	97.2
23	88.6	0.948	Case 2	95.3
24	88.1	0.845	Case 2	85.0
25	184.6	0.933	Case 2	146.5
26	181.2	0.947	Case 2	145.5
27	182.4	0.956	Case 2	148.1
28	180.1	0.915	Case 2	139.8
29	212.8	0.861	Case 2	160.7
30	212.5	0.890	Case 2	166.4
31	214.7	0.908	Case 2	171.2
32	51.8	0.949	Case 2	61.8
33	59.0	0.891	Case 2	68.7
34	376.9	0.731	Case 2	339.4
35	379.8	0.820	Case 2	380.1
36	281.5	0.847	Case 2	272.7
37	291.2	0.898	Case 2	292.7
38	261.8	0.873	Case 2	281.2
39	268.7	0.915	Case 2	296.5
40	76.2	0.937	Case 2	106.8
41	76.2	0.937	Case 2	106.8
42	76.2	0.937	Case 2	106.8

Table 7.4 continued

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
43	78.5	0.940	Case 2	107.1
44	78.5	0.940	Case 2	107.1
45	78.5	0.940	Case 2	107.1
46	78.5	0.940	Case 2	107.1
47	78.5	0.940	Case 2	107.1
48	78.5	0.940	Case 2	107.1
49	123.7	0.942	Case 2	168.4
50	123.7	0.942	Case 2	168.4
51	123.7	0.942	Case 2	168.4
52	123.7	0.942	Case 2	168.4
53	123.7	0.942	Case 2	168.4
54	123.7	0.942	Case 2	168.4
55	96.5	0.908	Case 2	113.7
56	96.5	0.908	Case 2	113.7

Table 7.5: Summary Strength Results for Bolted Angle Connections

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
1	115.3	[2]	98.4	[1]	54.2	[3]
2	117.1	[2]	97.0	[1]	53.5	[3]
3	109.5	[2]	96.4	[1]	53.1	[3]
4	62.2	[2]	51.8	[1]	28.1	[3]
5	100.4	[2]	80.7	[1]	46.2	[3]
6	91.0	[2]	80.1	[1]	40.1	[3]
7	97.3	[2]	80.3	[1]	46.0	[3]
8	93.3	[2]	84.4	[1]	43.1	[3]
9	52.5	[2]	44.7	[1]	26.3	[3]
10	53.9	[2]	44.2	[1]	26.2	[3]
11	44.6	[2]	42.6	[2]	23.9	[3]
12	218.8	[2]	196.3	[1]	163.3	[3]
13	224.2	[2]	194.5	[1]	162.0	[3]
14	222.6	[2]	192.1	[1]	160.5	[3]
15	110.7	[2]	104.1	[1]	92.8	[3]
16	188.4	[2]	161.8	[1]	145.1	[3]
17	191.1	[2]	160.6	[1]	143.5	[3]
18	179.2	[2]	162.6	[1]	143.0	[3]
19	175.8	[2]	160.0	[1]	140.8	[3]
20	192.7	[2]	161.2	[1]	143.9	[3]
21	183.2	[2]	167.2	[1]	145.3	[3]
22	92.8	[2]	88.3	[1]	77.1	[3]
23	96.5	[2]	88.6	[1]	76.2	[3]
24	77.5	[2]	85.0	[2]	52.2	[3]
25	142.1	[2]	146.5	[2]	91.6	[3]
26	147.5	[2]	145.5	[2]	90.1	[3]
27	148.4	[2]	148.1	[2]	90.7	[3]
28	121.8	[2]	139.8	[2]	79.0	[3]
29	132.6	[2]	160.7	[2]	89.9	[3]
30	137.8	[2]	166.4	[2]	91.8	[3]
31	140.5	[2]	171.2	[2]	91.1	[3]
32	88.8	[2]	51.8	[1]	29.0	[3]
33	69.9	[2]	59.0	[1]	29.4	[3]
34	237.1	[2]	339.4	[2]	198.4	[3]
35	297.7	[2]	379.8	[1]	231.2	[3]
36	218.6	[2]	272.7	[2]	177.1	[3]
37	243.5	[2]	291.2	[1]	211.3	[3]
38	236.5	[2]	261.8	[1]	174.7	[3]
39	255.2	[2]	268.7	[1]	206.5	[3]
40	89.5	[2]	76.2	[1]	41.2	[3]
41	96.1	[2]	76.2	[1]	41.2	[3]
42	95.2	[2]	76.2	[1]	41.2	[3]

Table 7.5 continued

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{EXP}, kips	Failure Mode
43	85	[2]	78.5	[1]	43.7	[3]
44	84	[2]	78.5	[1]	43.7	[3]
45	85.6	[2]	78.5	[1]	43.7	[3]
46	88.6	[2]	78.5	[1]	43.7	[3]
47	90.4	[2]	78.5	[1]	43.7	[3]
48	91	[2]	78.5	[1]	43.7	[3]
49	134.1	[2]	123.7	[1]	111.2	[3]
50	136.5	[2]	123.7	[1]	111.2	[3]
51	139.1	[2]	123.7	[1]	111.2	[3]
52	140	[2]	123.7	[1]	111.2	[3]
53	140.7	[2]	123.7	[1]	111.2	[3]
54	140.6	[2]	123.7	[1]	111.2	[3]
55	99	[2]	96.5	[1]	51.0	[3]
56	91	[2]	96.5	[1]	51.0	[3]

[1] tensile yield; [2] tensile rupture; [3] plate strain

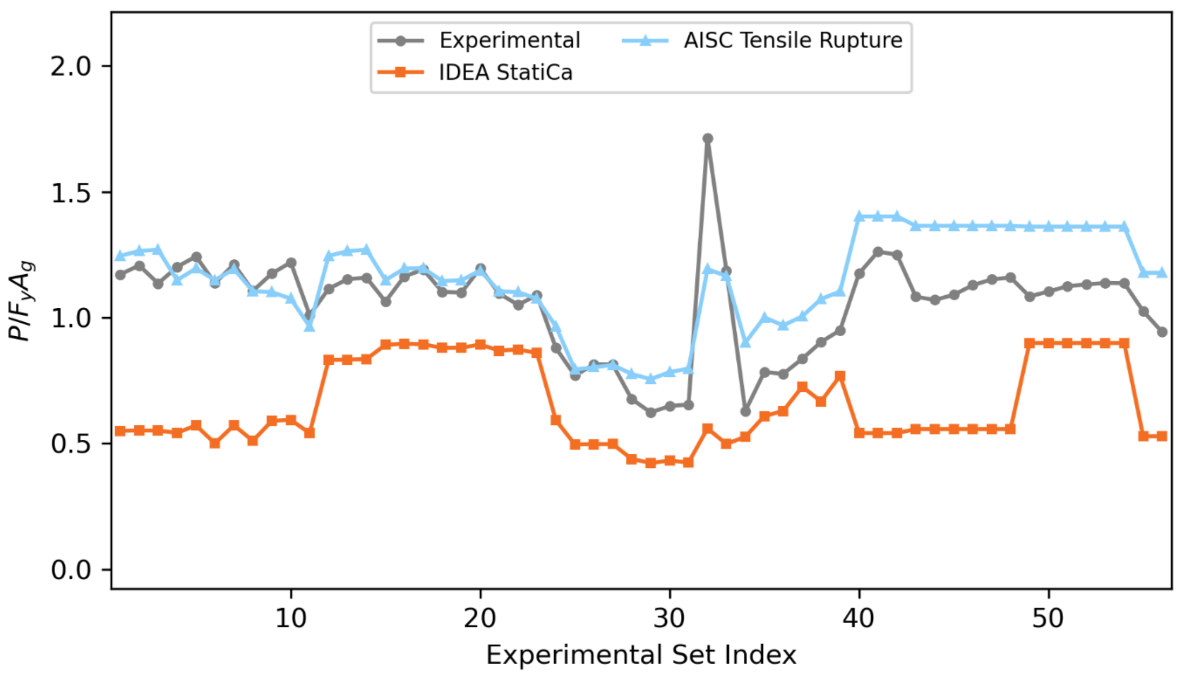


Figure 7-4: Normalized Strength Results for Bolted Angle Connections

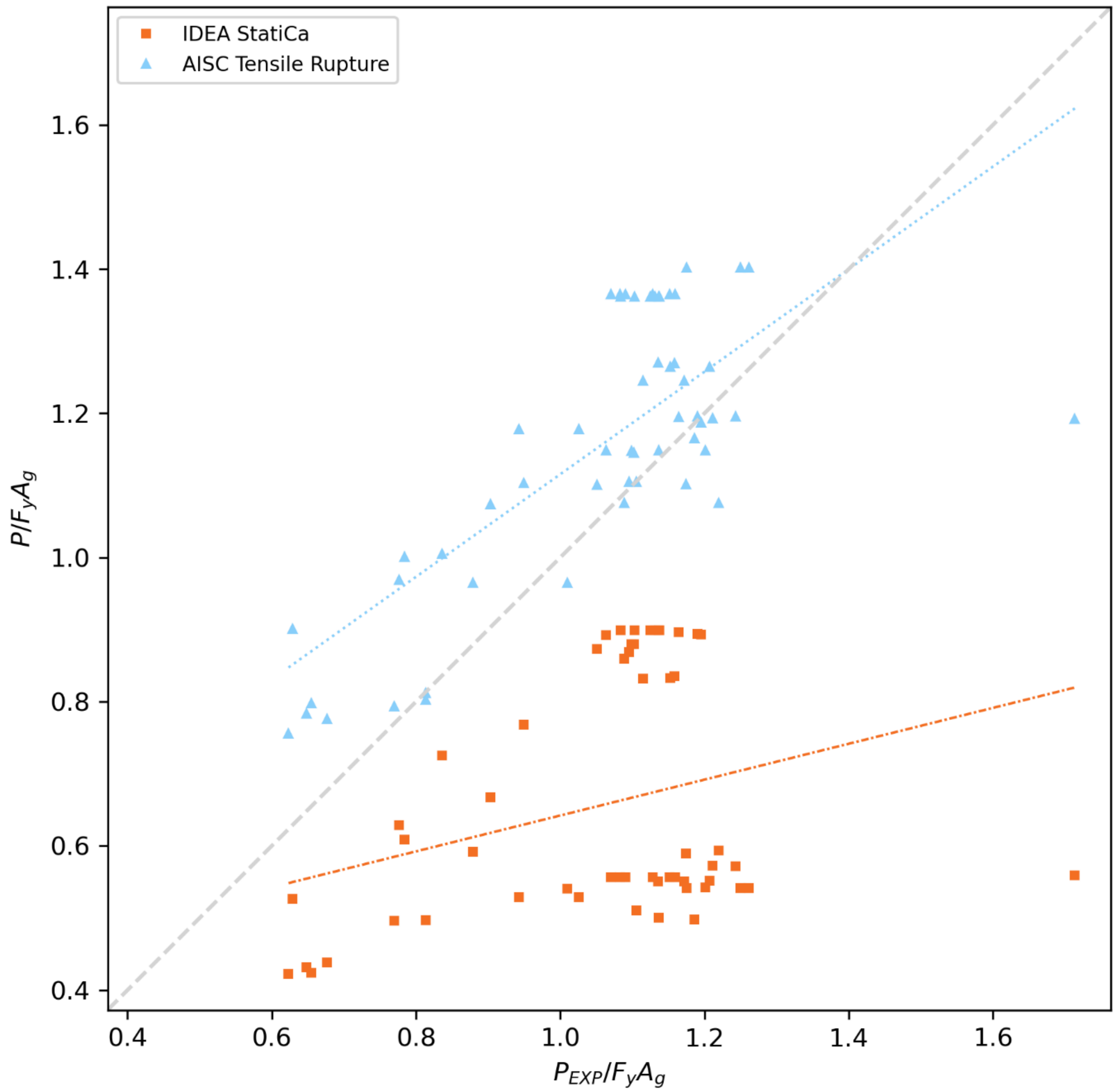


Figure 7-5: Normalized Strength vs. Normalized Experimental Strength for Bolted Angle Connections

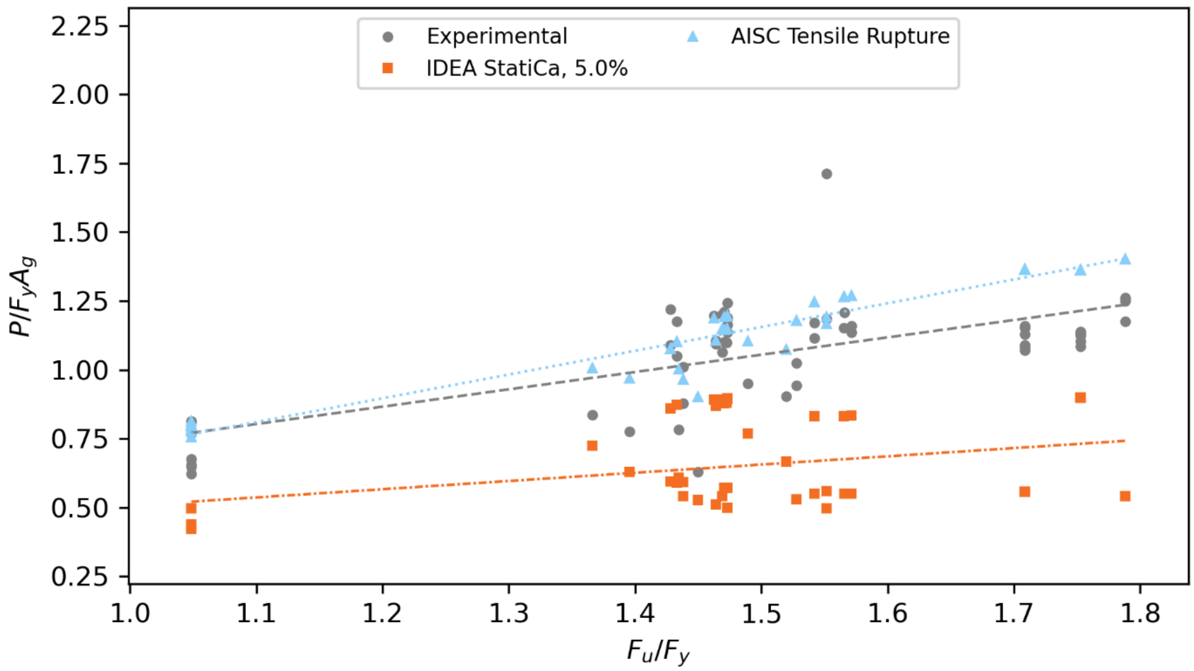


Figure 7-6: Normalized Strength vs. Material Strength Ratio for Bolted Angle Connections

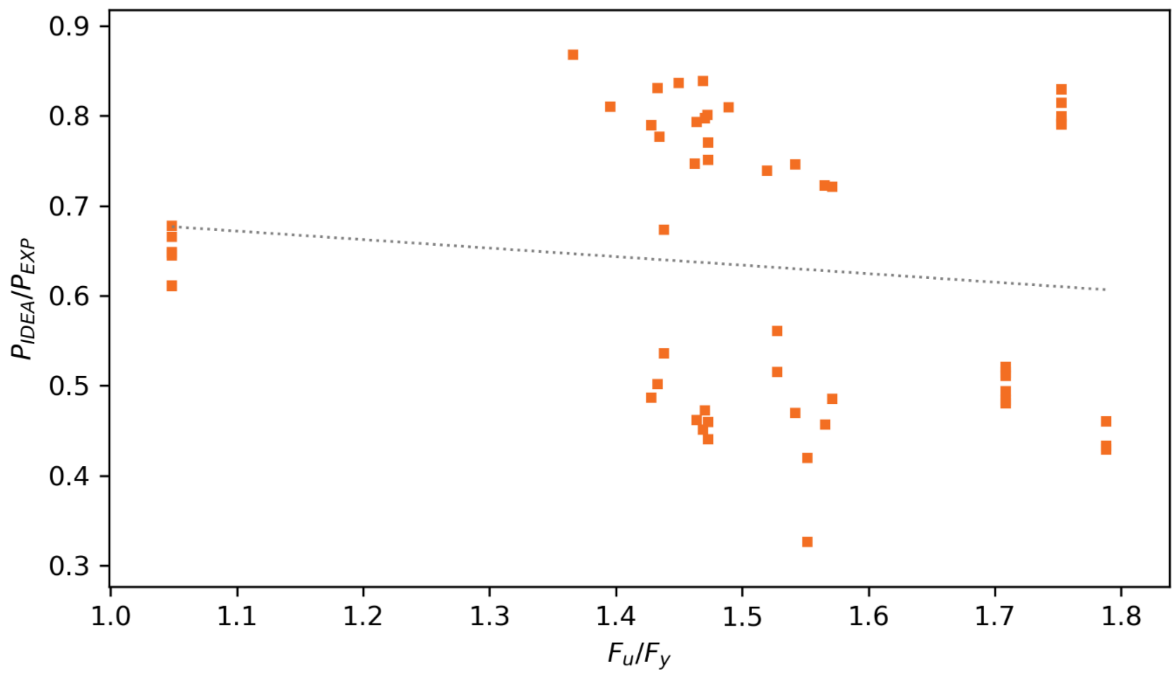


Figure 7-7: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Bolted Angle Connections

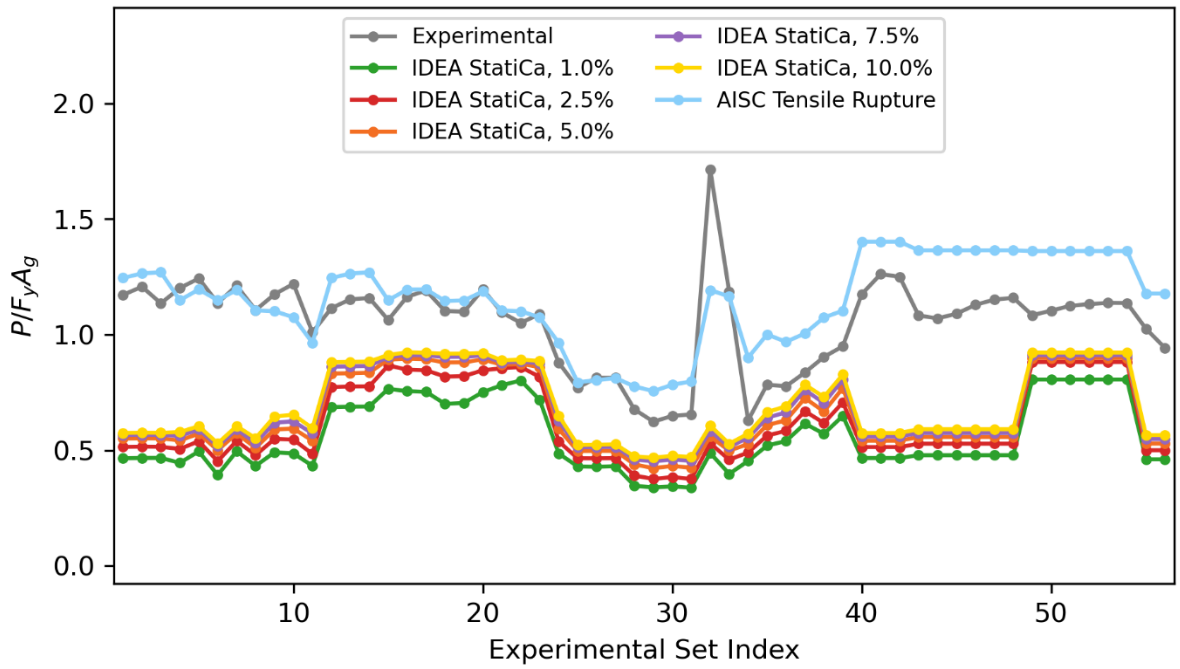


Figure 7-8: Normalized Strength Results for Bolted Angle Connections Including Various Plastic Strain Limits for IDEA StatiCa

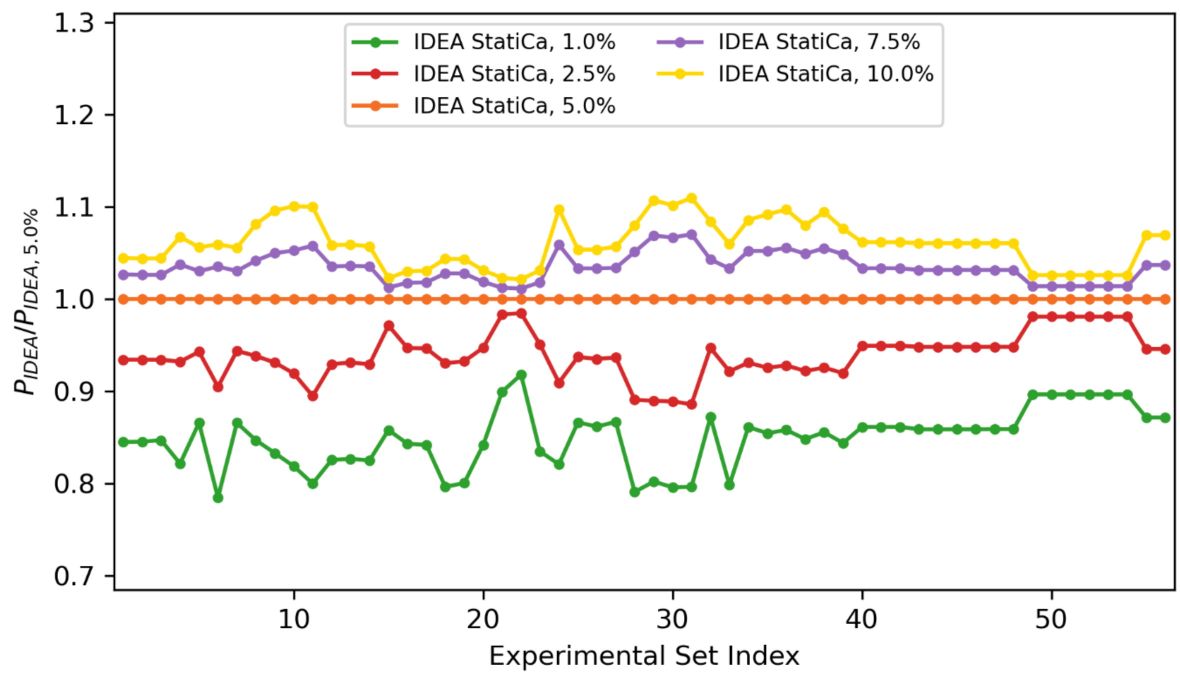


Figure 7-9: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Bolted Angle Connections

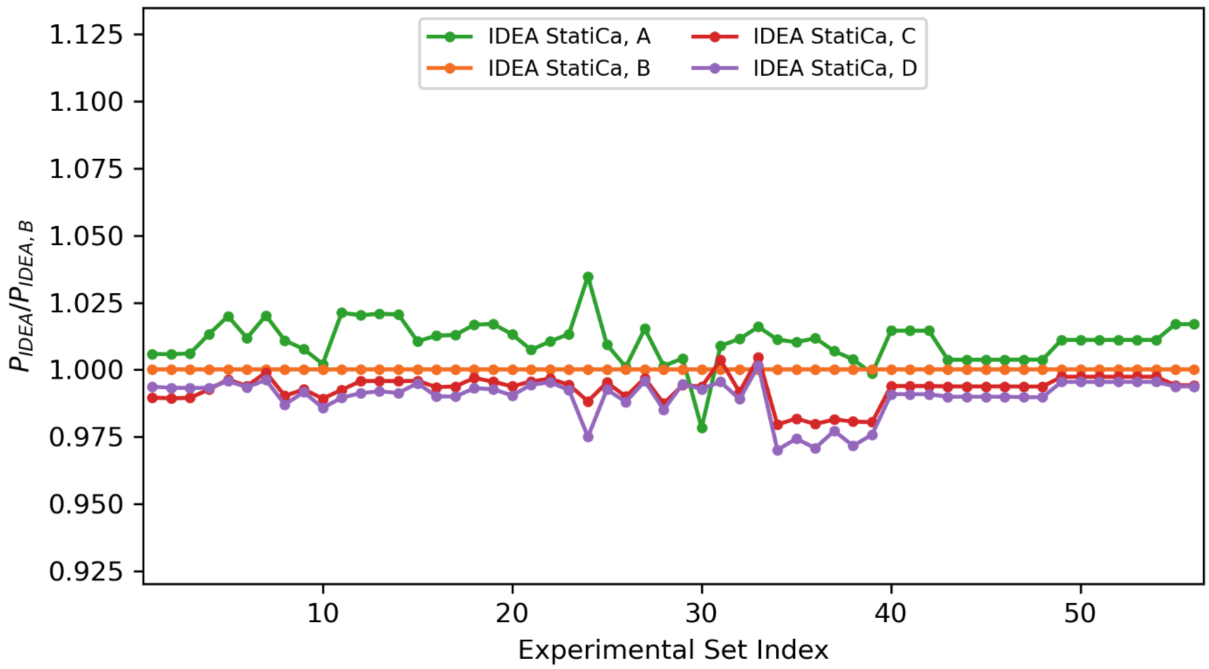


Figure 7-10: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Bolted Angle Connections

Table 7.6: Summary Statistics of the Test-to-Predicted Ratio for Bolted Angle Connections

Test-to-Predicted Ratio	Average	Standard Deviation	Coefficient of Variation
$P_{EXP}/P_{RUPTURE}$ (AISC)	0.916	0.116	0.127
P_{EXP}/P_{IDEA}	1.679	0.441	0.263

7.2.3 Discussion

Tensile rupture was the reported failure mode for all experiments of the bolted specimens in this set. However, tensile yield controlled the AISC strength calculations for most of the specimens (Table 7.5). When comparing P_{IDEA} to P_{AISC}, the most conservative case was specimen 33. This specimen resulted in a maximum permitted applied load in IDEA StatiCa of 50% less than the strength according to the AISC *Specification* equations. The least conservative case indicates only a 10% difference between P_{IDEA} and P_{AISC}. As with the welded angles one potential cause of the relatively low strength from IDEA StatiCa is the moment developed by the eccentricity between the angle and the gusset plate. It is anticipate that significantly greater strengths would be observed from geometrically nonlinear IDEA StatiCa analyses.

The strengths according to the IDEA StatiCa analyses were less than the strength from the experiment and the strength from the AISC equations for all specimens in the set (Figure 7-4). The AISC strength and the experimental strength were more comparable. However, for the cases where tensile rupture was most critical (i.e., when P_{EXP} was less then F_yA_g), the AISC strength equations overestimated the strength (Figure 7-5).

With respect to the material strength ratio F_u/F_y, clear increases with increasing F_u/F_y are seen for the experimental and AISC strengths (Figure 7-6). The increase in IDEA StatiCa strength normalized by F_yA_g with increasing F_u/F_y is less than for the experimental strength and AISC strength. Correspondingly, the ratio P_{IDEA}/P_{EXP} is seen to decrease with increasing F_u/F_y (Figure 7-7).

The variation of P_{IDEA} with plastic strain limit exhibited the expected trend of increasing strength with increasing plastic strain limit and vice versa (Figure 7-8 and Figure 7-9). When comparing the results from the 10% and 5% plastic strain limits to P_{AISC}, the 10% plastic strain limit only provides a 6% less conservative strength than the 5% plastic strain limit in the most extreme case.

The variation of P_{IDEA} with mesh parameters showed very minor changes (Figure 7-10). In IDEA StatiCa, the mesh around the bolt holes in IDEA StatiCa always remains constant at 8 elements, regardless of the mesh parameters set by the user. Since the greatest plastic strains were observed near the bolt holes, adjusting the mesh elsewhere has a minimal effect on the resulting strength.

7.3 Comparison to Experimental Results: Staggered Bolt Configuration

7.3.1 Description of Experimental Specimens

Bolted angle connection specimens with staggered bolt patterns are examined in this section. For this set, most specimens included a configuration like that shown in Figure 7-11. However, due to the disparate nature of the bolt configurations observed in the literature, a detailed description of each specimen is not provided.

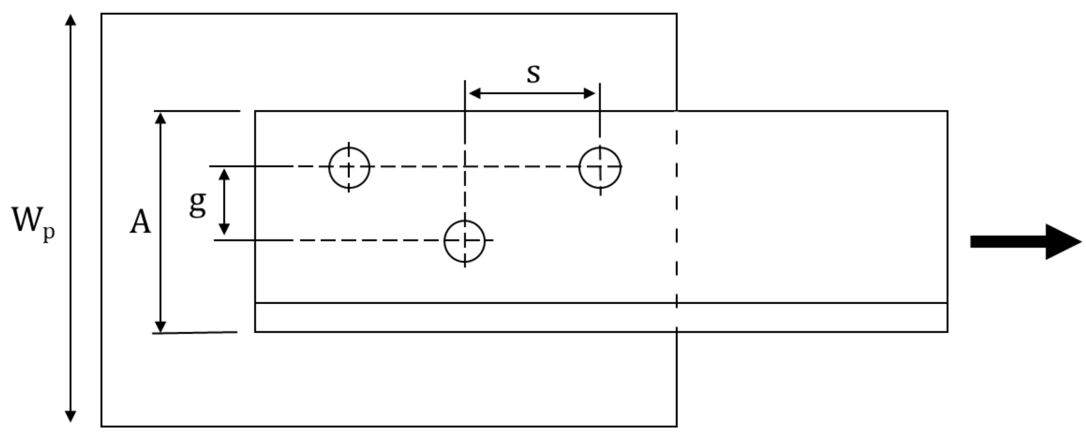


Figure 7-11: Schematic of Bolted Angle (with Staggered Bolts) Connection

A total of 28 bolted angle specimens (with staggered bolt patterns) from 4 references were identified for evaluation in this work as detailed in Table 7.7.

Table 7.7: Count of Bolted Angle (with Staggered Bolts) Connections by Reference

Reference	**Specimen Count**
Epstein (1992)	7
Greiner (1897)	2
McKibben (1907)	9
Chesson and Munse (1963)	10
Total	**28**

Key details of each specimen are provided in Table 7.8 and Table 7.9. The dimensions defined in Figure 7-11 are consistent with those used in Table 7.8.

The modeling of these specimens in IDEA StatiCa is identical to the bolted angle specimens previously described. The primary difference for staggered bolt configurations is found in the calculations for the strength according to the AISC *Specification* (2016). The calculations for the net area, A_n, include additional guidance than the specimens with normal bolt configurations. In addition to subtracting out the area removed for the bolt holes, a term is added to the width of the angle for every diagonal in accordance with Section B4.3b of the AISC *Specification* (2016). The quantity added is $s^2/4g$ where ‘s’ is the transverse center-to-center spacing between fastener gage lines and ‘g’ is the longitudinal center-to-center spacing of any two consecutive holes. These dimensions are consistent with those in Figure 7-11. Consistent with common practice, the net area was calculated for every applicable failure path and the minimum value was taken as the controlling net area.

Table 7.8: Bolted Angle (with Staggered Bolts) Connection Experimental Specimen Parameters

Index	Reference	Specimen	# of angles	*F_y, ksi	*F_u, ksi	*A, in	*B, in	*t, in
1	Epstein (1992)	5	2	49.3	73.6	6.000	6.000	0.313
2	Epstein (1992)	12	2	55.5	80.0	6.000	4.000	0.313
3	Epstein (1992)	13	2	50.5	70.2	6.000	4.000	0.313
4	Epstein (1992)	20	2	50.3	68.5	6.000	3.500	0.313
5	Epstein (1992)	28	2	50.4	70.1	5.000	5.000	0.313
6	Epstein (1992)	31	2	45.2	68.2	5.000	3.500	0.313
7	Epstein (1992)	36	2	42.2	61.1	5.000	3.000	0.313
8	Greiner (1897)	47 B	1	38.9	59.4	6.000	4.000	0.375
9	Greiner (1897)	48 B	1	38.9	59.4	6.000	4.000	0.375
10	McKibben (1907)	16	1	34.8	61.0	6.000	4.000	0.375
11	McKibben (1907)	17	1	34.8	61.0	6.000	4.000	0.375
12	McKibben (1907)	18	1	34.8	61.0	6.000	4.000	0.375
13	McKibben (1907)	46	1	34.8	61.0	6.000	4.000	0.375
14	McKibben (1907)	47	1	34.8	61.0	6.000	4.000	0.375
15	McKibben (1907)	48	1	34.8	61.0	6.000	4.000	0.375
16	McKibben (1907)	49	1	34.8	61.0	6.000	4.000	0.375
17	McKibben (1907)	50	1	34.8	61.0	6.000	4.000	0.375
18	McKibben (1907)	51	1	34.8	61.0	6.000	4.000	0.375
19	Chesson and Munse (1963)	SA-1-DB	4	45.2	67.0	3.500	3.500	0.438
20	Chesson and Munse (1963)	SB-1-DR	4	43.3	66.4	5.000	3.000	0.375
21	Chesson and Munse (1963)	SB-2-DR	4	43.3	66.4	5.000	3.000	0.375
22	Chesson and Munse (1963)	SB-1-PR	4	43.3	66.4	5.000	3.000	0.375
23	Chesson and Munse (1963)	SB-2-PR	4	43.3	66.4	5.000	3.000	0.375
24	Chesson and Munse (1963)	SD-1-DR	4	42.5	65.4	5.000	3.000	0.375
25	Chesson and Munse (1963)	SD-2-DR	4	42.5	65.4	5.000	3.000	0.375
26	Chesson and Munse (1963)	SD-1-PR	4	42.5	65.4	5.000	3.000	0.375
27	Chesson and Munse (1963)	SD-2-PR	4	42.5	65.4	5.000	3.000	0.375
28	Chesson and Munse (1963)	SE-1-DB	4	40.4	66.7	5.000	5.000	0.375

*Indicates measured value

Table 7.9: Bolted Angle (with Staggered Bolts) Connection Experimental Specimen Fastener Details

Index	Bolts/ Rivets	d, in.	Total # of Fasteners	# of Fasteners in Last Column
1	Bolts	0.750	5	1
2	Bolts	0.750	5	1
3	Bolts	0.750	5	1
4	Bolts	0.750	5	1
5	Bolts	0.750	5	1
6	Bolts	0.750	5	1
7	Bolts	0.750	5	1
8	Rivets	0.875	8	1
9	Rivets	0.875	8	1
10	Rivets	0.875	9	1
11	Rivets	0.875	9	1
12	Rivets	0.875	9	1
13	Rivets	0.875	9	1
14	Rivets	0.875	9	1
15	Rivets	0.875	9	1
16	Rivets	0.875	9	1
17	Rivets	0.875	9	1
18	Rivets	0.875	9	1
19	Bolts	0.750	22	1
20	Rivets	0.875	22	1
21	Rivets	0.875	22	1
22	Rivets	0.875	22	1
23	Rivets	0.875	22	1
24	Rivets	0.875	8	2
25	Rivets	0.875	8	2
26	Rivets	0.875	8	2
27	Rivets	0.875	8	2
28	Bolts	0.750	28	1

7.3.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 7.10: AISC Calculation Results for Bolted Angle (with Staggered Bolts) Connections

Index	P_{YIELD}, kips	A_n, in^2	U	Controlling U Case	$P_{RUPTURE}$, kips
1	360.1	3.176	0.600	Case 8	280.5
2	336.0	2.551	0.694	Case 2	283.3
3	305.8	2.551	0.694	Case 2	248.6
4	288.8	2.395	0.746	Case 2	244.6
5	305.2	2.581	0.600	Case 8	217.1
6	231.3	2.112	0.721	Case 2	207.7
7	202.8	1.956	0.773	Case 2	184.8
8	140.2	2.974	0.821	Case 2	144.9
9	140.2	2.974	0.821	Case 2	144.9
10	125.6	3.094	0.906	Case 2	171.0
11	125.6	3.094	0.906	Case 2	171.0
12	125.6	3.094	0.906	Case 2	171.0
13	125.6	3.234	0.949	Case 2	187.3
14	125.6	3.234	0.949	Case 2	187.3
15	125.6	3.234	0.949	Case 2	187.3
16	125.6	3.026	0.882	Case 2	162.9
17	125.6	3.026	0.882	Case 2	162.9
18	125.6	3.026	0.882	Case 2	162.9
19	494.3	2.021	0.877	Case 2	475.0
20	495.2	2.155	0.893	Case 2	511.3
21	495.2	2.155	0.893	Case 2	511.3
22	495.2	2.155	0.893	Case 2	511.3
23	495.2	2.155	0.893	Case 2	511.3
24	510.0	1.938	0.778	Case 2	394.2
25	510.0	1.938	0.778	Case 2	394.2
26	510.0	1.938	0.778	Case 2	394.2
27	510.0	1.938	0.778	Case 2	394.2
28	583.2	2.958	0.870	Case 2	686.7

Table 7.11: Summary Strength Results for Bolted Angle (with Staggered Bolts) Connections

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
1	204.9	[2]	280.5	[2]	208.5	[3]
2	247.1	[2]	283.3	[2]	236.1	[3]
3	189.1	[2]	248.6	[2]	209.0	[3]
4	238.5	[2]	244.6	[2]	221.0	[3]
5	169.6	[2]	217.1	[2]	183.7	[3]
6	208.8	[2]	207.7	[2]	179.2	[3]
7	163.0	[2]	184.8	[2]	149.7	[3]
8	128.0	[2]	140.2	[1]	79.7	[3]
9	129.3	[2]	140.2	[1]	79.7	[3]
10	131.0	[2]	125.6	[1]	74.0	[3]
11	125.7	[2]	125.6	[1]	74.0	[3]
12	128.2	[2]	125.6	[1]	74.0	[3]
13	154.6	[2]	125.6	[1]	76.6	[3]
14	150.6	[2]	125.6	[1]	76.6	[3]
15	153.3	[2]	125.6	[1]	76.6	[3]
16	150.6	[2]	125.6	[1]	75.5	[3]
17	144.1	[2]	125.6	[1]	75.5	[3]
18	152.2	[2]	125.6	[1]	75.5	[3]
19	559.0	[2]	475.0	[2]	439.4	[3]
20	513.0	[2]	495.2	[1]	426.3	[3]
21	527.0	[2]	495.2	[1]	426.3	[3]
22	492.4	[2]	495.2	[1]	426.3	[3]
23	498.2	[2]	495.2	[1]	426.3	[3]
24	470.7	[2]	394.2	[2]	349.1	[3]
25	466.7	[2]	394.2	[2]	349.1	[3]
26	451.8	[2]	394.2	[2]	349.1	[3]
27	418.0	[2]	394.2	[2]	349.1	[3]
28	842.0	[2]	583.2	[1]	526.1	[3]

[1] tensile yield; [2] tensile rupture; [3] plate strain

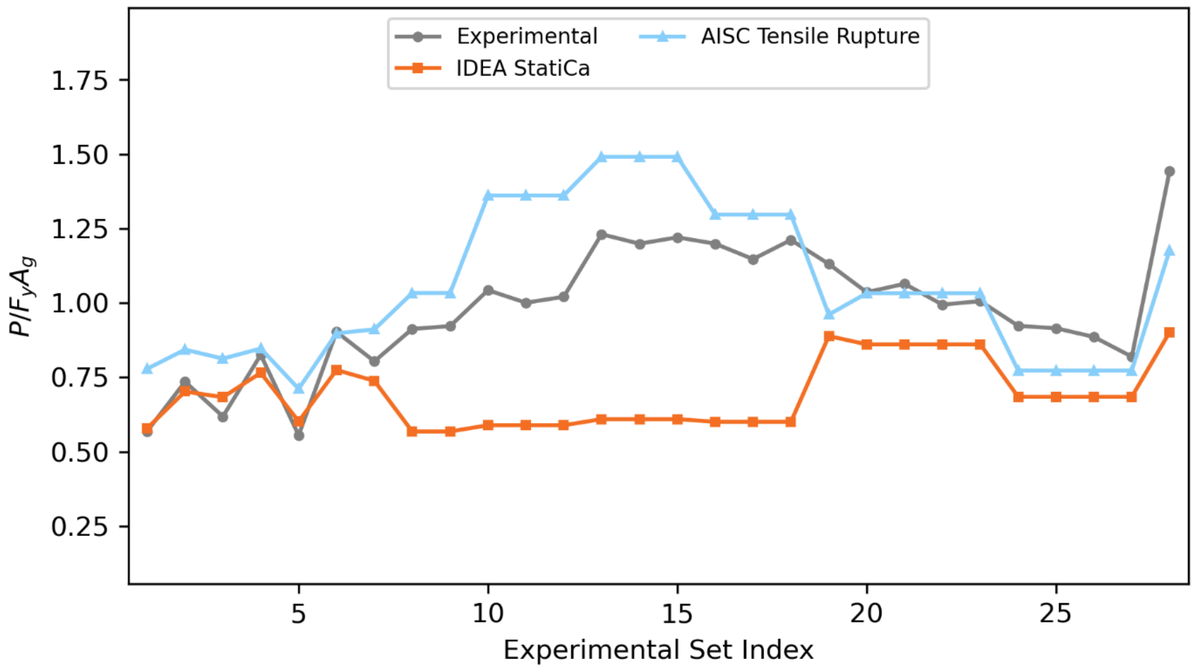


Figure 7-12: Normalized Strength Results for Bolted Angle (with Staggered Bolts) Connections

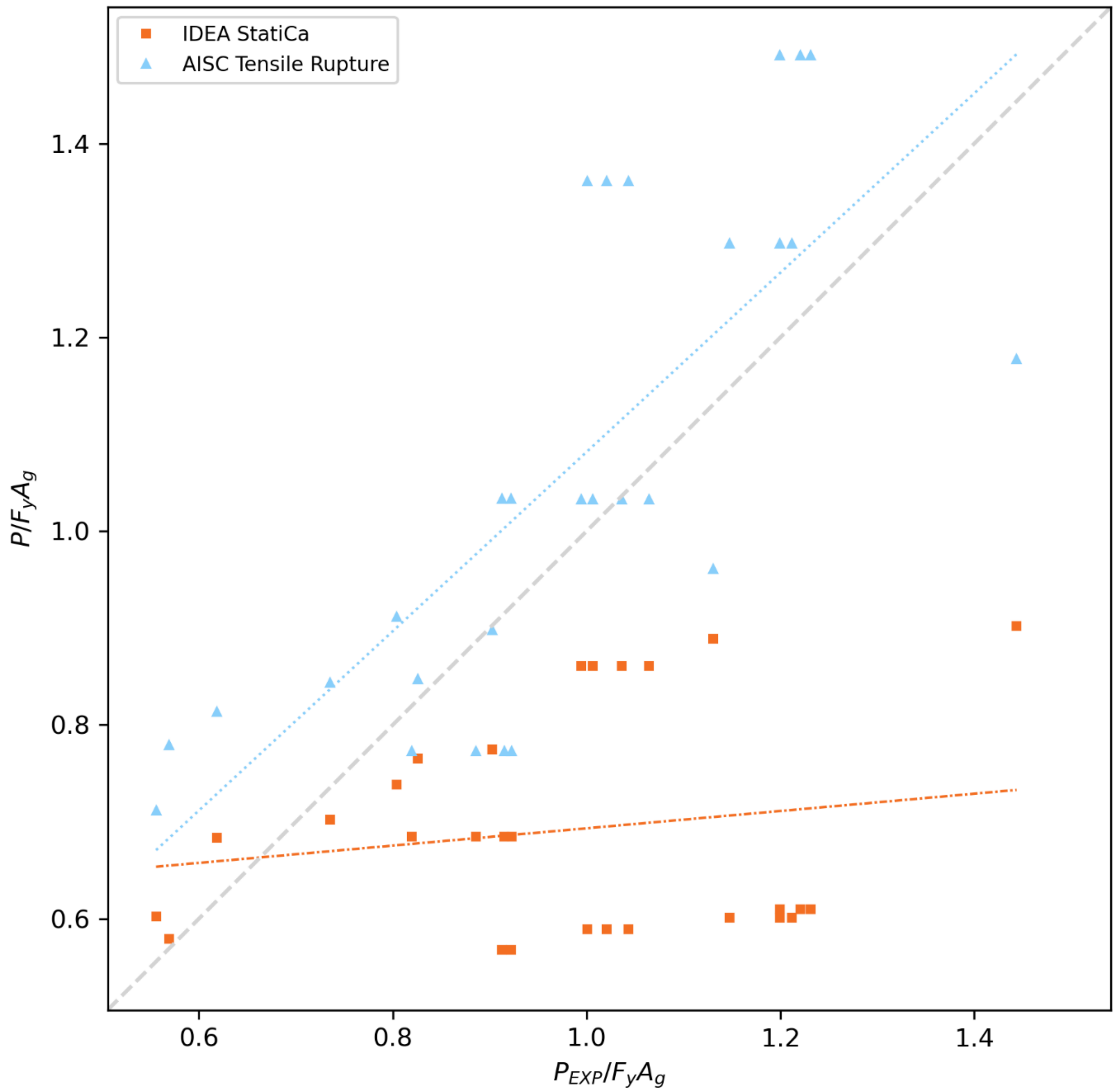


Figure 7-13: Normalized Strength vs. Normalized Experimental Strength for Bolted Angle (with Staggered Bolts) Connections

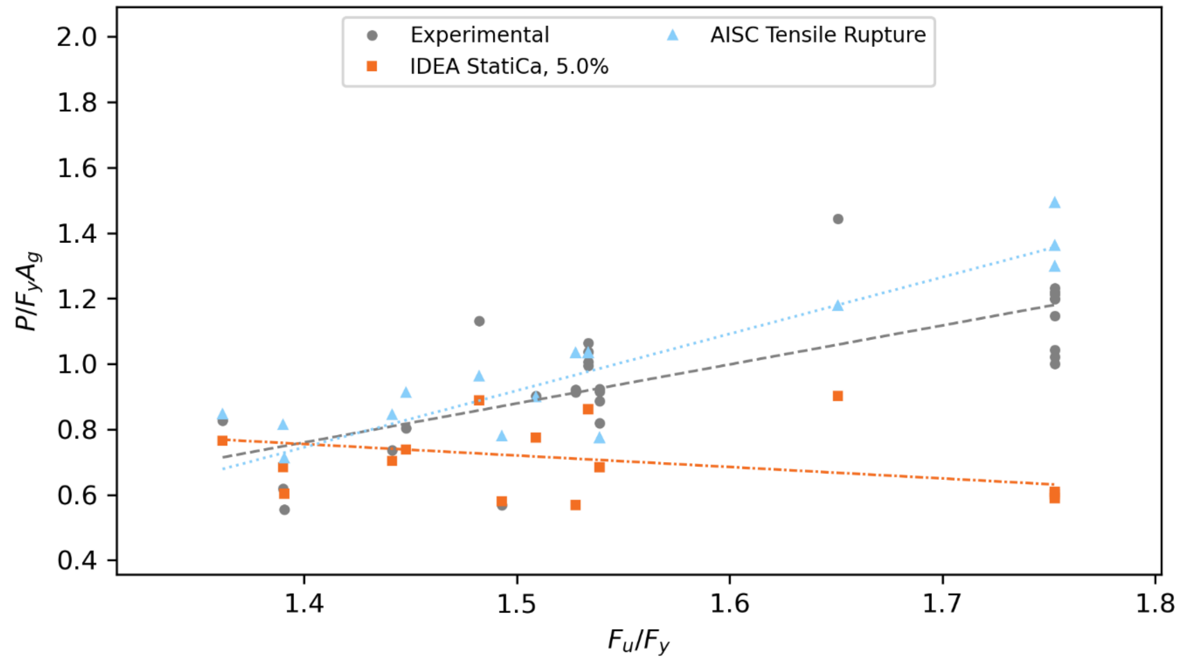


Figure 7-14: Normalized Strength vs. Material Strength Ratio for Bolted Angle (with Staggered Bolts) Connections

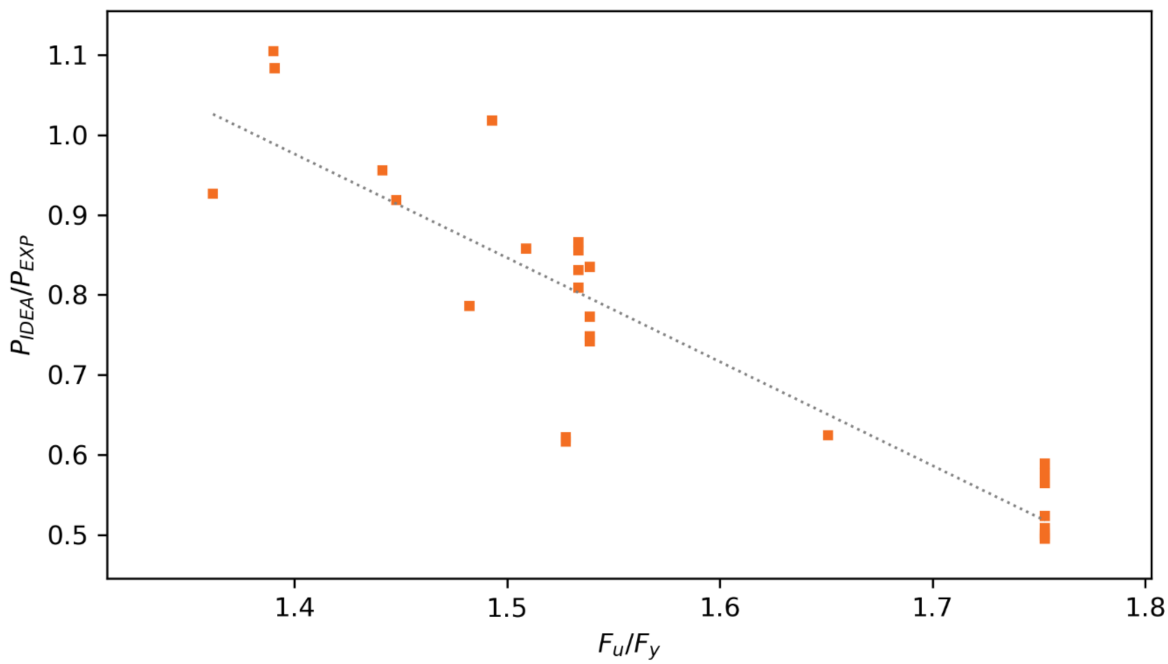


Figure 7-15: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Bolted Angle (with Staggered Bolts) Connections

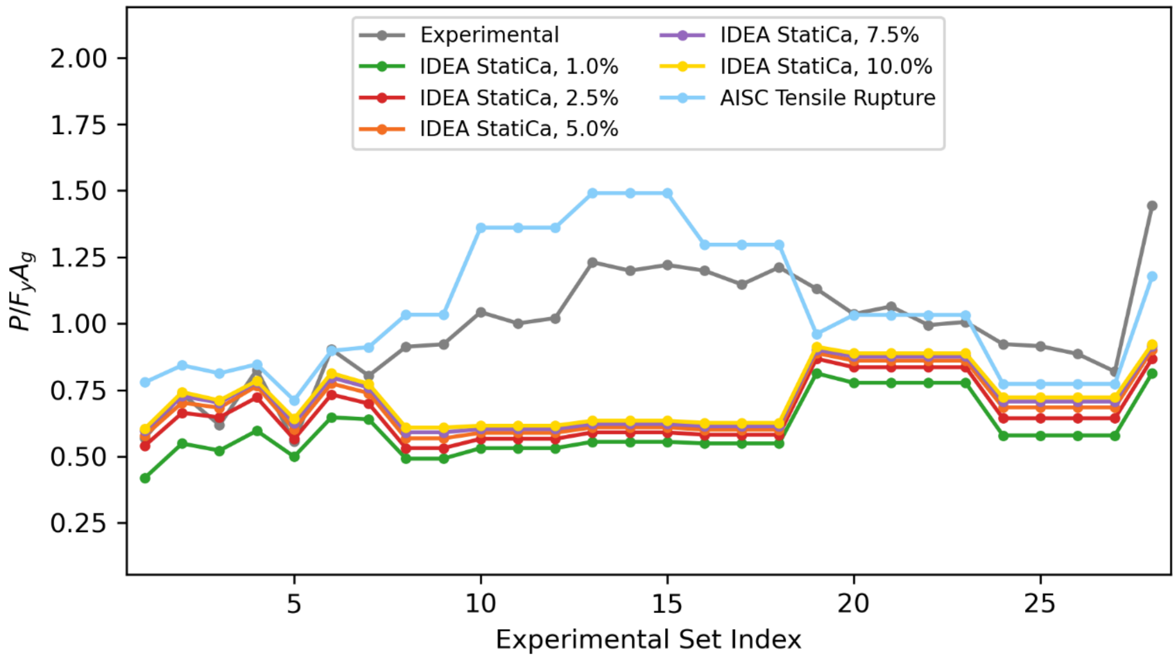


Figure 7-16: Normalized Strength Results for Bolted Angle (with Staggered Bolts) Connections Including Various Plastic Strain Limits for IDEA StatiCa

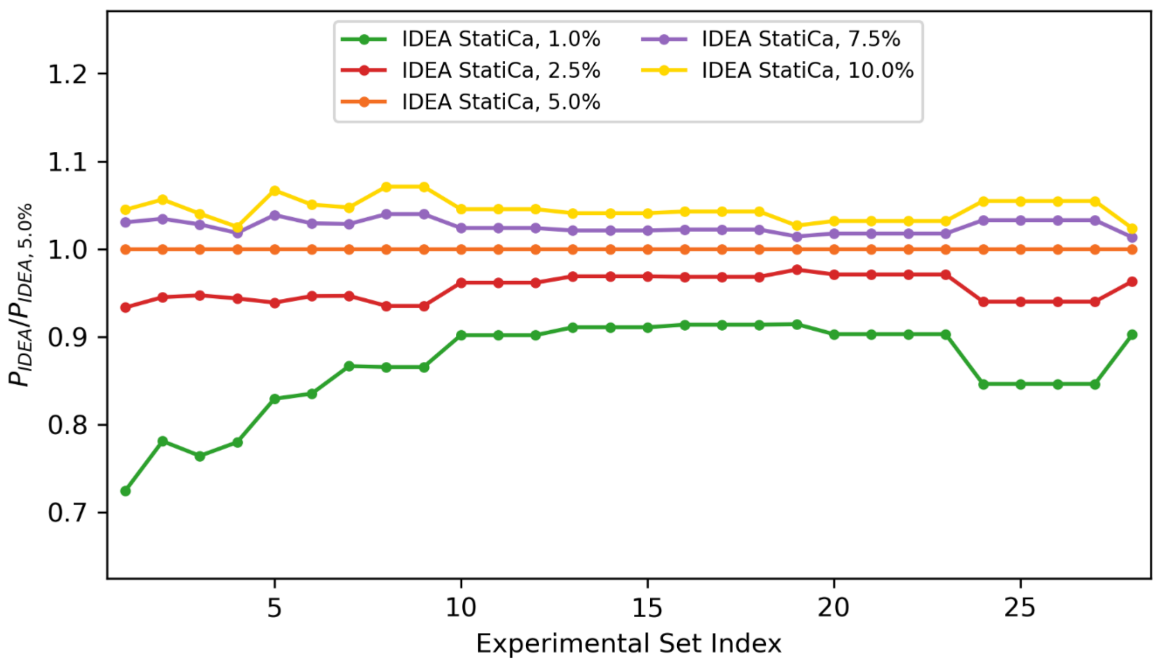


Figure 7-17: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Bolted Angle (with Staggered Bolts) Connections

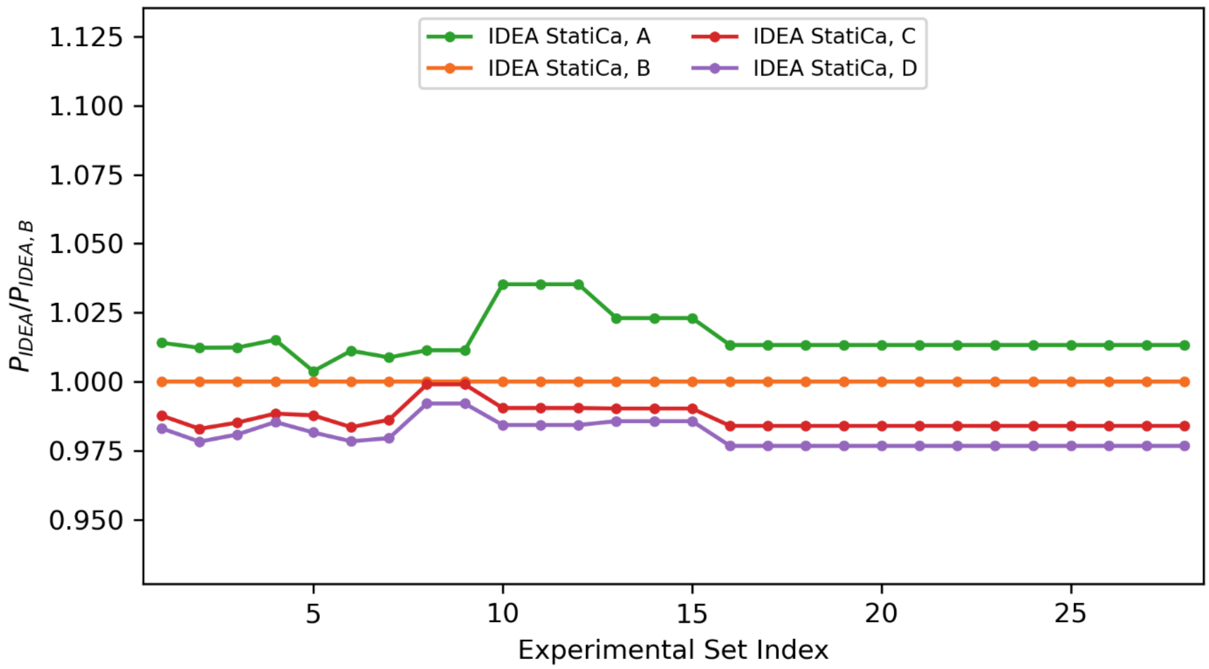


Figure 7-18: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Bolted Angle (with Staggered Bolts) Connections

Table 7.12: Summary Statistics of the Test-to-Predicted Ratio for Bolted Angle (with Staggered Bolts) Connections

Test-to-Predicted Ratio	**Average**	**Standard Deviation**	**Coefficient of Variation**
$P_{EXP}/P_{RUPTURE}$ (AISC)	0.935	0.150	0.161
P_{EXP}/P_{IDEA}	1.441	0.373	0.259

7.3.3 Discussion

The results for the bolted angles with staggered bolt patterns are similar to those with regular bolt patterns. Again, single angles exhibit relatively low strengths in IDEA StatiCa. However, for three specimens tested by Epstein (1992), the IDEA StatiCa strength is slightly higher than that from the experiment. For these three specimens, the AISC tensile rupture strength is greater than the IDEA StatiCa strength.

7.4 Reliability Analysis

7.4.1 Description of Reliability Set

The reliability specimen set was created based on a variety of desired parameters. For this particular set, the parameters were selected based upon varying the following: *A*/*B* ratio (where *A* is the connected leg length and *B* is the non-connected leg length), thickness, bolts per row, and bolt spacing. The spacing was categorized into two groups: minimum and greater than minimum. The spacing was dependent upon the connected leg length and based upon Table 1-7A of the AISC *Steel Construction Manual* (2017) and Section J3 of the AISC *Specification* (2016). The general parameters for each connection are provided in Table 7.13 and the spacing details are provided in Table 7.14.

Table 7.13: Bolted Angle Connection Reliability Set Parameters

Index	***A*, in.**	***B*, in.**	***t*, in.**	**Material Grade**	**F_y, ksi**	**F_u, ksi**	**Bolt Diameter, in.**	**Rows of Bolts**	**Bolts per Row**
1	4.0	3.0	0.375	A36 (Angle)	36	58	0.75	1	4
2	4.0	3.0	0.375	A36 (Angle)	36	58	0.75	1	4
3	4.0	3.0	0.500	A36 (Angle)	36	58	0.75	1	4
4	4.0	3.0	0.500	A36 (Angle)	36	58	0.75	1	4
5	4.0	3.0	0.375	A36 (Angle)	36	58	0.75	1	6
6	4.0	3.0	0.375	A36 (Angle)	36	58	0.75	1	6
7	4.0	3.0	0.500	A36 (Angle)	36	58	0.75	1	6
8	4.0	3.0	0.500	A36 (Angle)	36	58	0.75	1	6
9	3.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	4
10	3.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	4
11	3.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	4
12	3.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	4
13	3.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	6
14	3.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	6
15	3.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	6
16	3.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	6
17	4.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	4
18	4.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	4
19	4.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	4
20	4.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	4
21	4.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	6
22	4.0	4.0	0.375	A36 (Angle)	36	58	0.75	1	6
23	4.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	6
24	4.0	4.0	0.500	A36 (Angle)	36	58	0.75	1	6

Table 7.14: Bolted Angle Connection Reliability Set Spacing Parameters

Index	***s*, in.**	**L_{ev}, in.**	**L_{eh}, in.**
1	2.00	1.50	1.00
2	4.00	1.50	2.00
3	2.00	1.50	1.00
4	4.00	1.50	2.00
5	2.00	1.50	1.00
6	4.00	1.50	2.00
7	2.00	1.50	1.00
8	4.00	1.50	2.00
9	2.00	1.25	1.00
10	4.00	1.25	2.00
11	2.00	1.25	1.00
12	4.00	1.25	2.00
13	2.00	1.25	1.00
14	4.00	1.25	2.00
15	2.00	1.25	1.00
16	4.00	1.25	2.00
17	2.00	1.50	1.00
18	4.00	1.50	2.00
19	2.00	1.50	1.00
20	4.00	1.50	2.00
21	2.00	1.50	1.00
22	4.00	1.50	2.00
23	2.00	1.50	1.00
24	4.00	1.50	2.00

7.4.2 Results

For this reliability set of bolted angles, the strength results are provided in Table 7.15. Consistent with previous chapters, the nominal and design strengths are provided for both tensile yield and tensile rupture in this table. However, for the purpose of the reliability analysis only the controlling design strength was used (ϕP_{AISC} is the minimum of ϕP_{YIELD} and $\phi P_{RUPTURE}$). As for the strength according to IDEA StatiCa, P_{IDEA}, it was determined using all default settings (mesh parameter set 'B', 5% plastic strain limit, and all LRFD resistance factors applied). For this reliability set, IDEA StatiCa never exceeded the strength according to the AISC equations. On average, IDEA StatiCa provided a strength 47% less than the AISC *Specification* equations.

Table 7.15: Summary Strength Results for Bolted Angle Connection Reliability Set

Index	AISC				IDEA StatiCa
	P_{YIELD}, kips	$P_{RUPTURE}$, kips	ϕP_{YIELD}, kips	$\phi P_{RUPTURE}$, kips	P_{IDEA}, kips
1	89.6	109.2	80.7	81.9	44.1
2	89.6	117.3	80.7	88.0	45.0
3	117.0	140.8	105.3	105.6	54.5
4	117.0	152.0	105.3	114.0	55.6
5	89.6	115.7	80.7	86.8	45.1
6	89.6	120.5	80.7	90.4	45.4
7	117.0	149.7	105.3	112.3	55.9
8	117.0	156.4	105.3	117.3	56.3
9	89.6	100.3	80.7	75.2	39.6
10	89.6	112.1	80.7	84.1	40.3
11	117.0	130.5	105.3	97.9	50.1
12	117.0	145.2	105.3	108.9	50.8
13	89.6	109.5	80.7	82.1	40.3
14	89.6	117.4	80.7	88.1	40.6
15	117.0	141.6	105.3	106.2	51.0
16	117.0	152.4	105.3	114.3	51.2
17	103.0	119.2	92.7	89.4	48.2
18	103.0	133.0	92.7	99.8	50.4
19	135.0	154.3	121.5	115.8	60.4
20	135.0	173.2	121.5	129.9	63.2
21	103.0	130.3	92.7	97.7	50.5
22	103.0	138.6	92.7	103.9	50.9
23	135.0	169.5	121.5	127.1	63.7
24	135.0	180.8	121.5	135.6	64.2

The reliability index, β, results are provided in Figure 7-19. The β-values are plotted against the specimen index for the reliability set. For the IDEA StatiCa results, the analysis indicated no failures in any of the 1,000,000 Monte Carlo trials, thus the β-values for IDEA StatiCa are large (i.e., greater than 4.75) and off the plot in Figure 7-19. The AISC β-values ranged from 3.20 to 3.57. These results are consistent with both the experimental set strength results and strength results of the reliability set for bolted angles. In conclusion, IDEA StatiCa consistently provided more reliable results when compared to the AISC *Specification* equations.

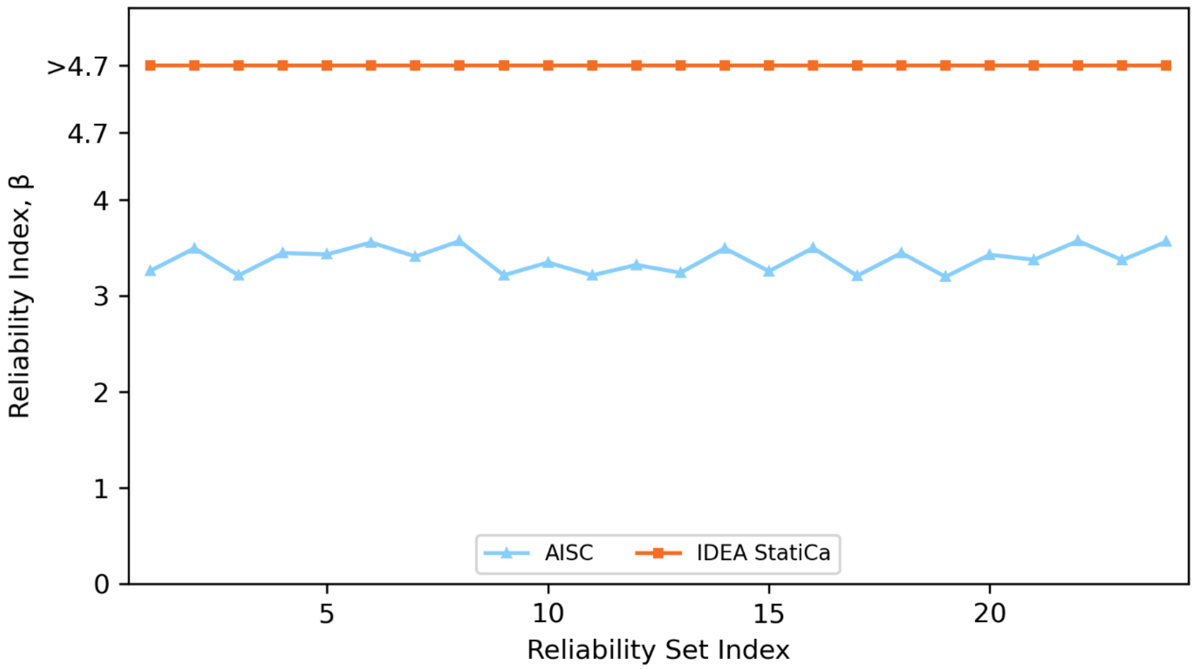


Figure 7-19: Reliability Index for Bolted Angle Connections

Chapter 8:
WELDED PLATES

8.1 Description of Connection

Single and double plate tension members welded to a gusset plate are evaluated in this chapter. A typical schematic with relevant terminology is shown in Figure 8-1. For clarity, the dimension t always refers to the thickness of the tension member and t_p always refers to the thickness of the plate to which the tension member is connected, referred to as the gusset plate. The failure always occurred in the tension member in the physical tests.

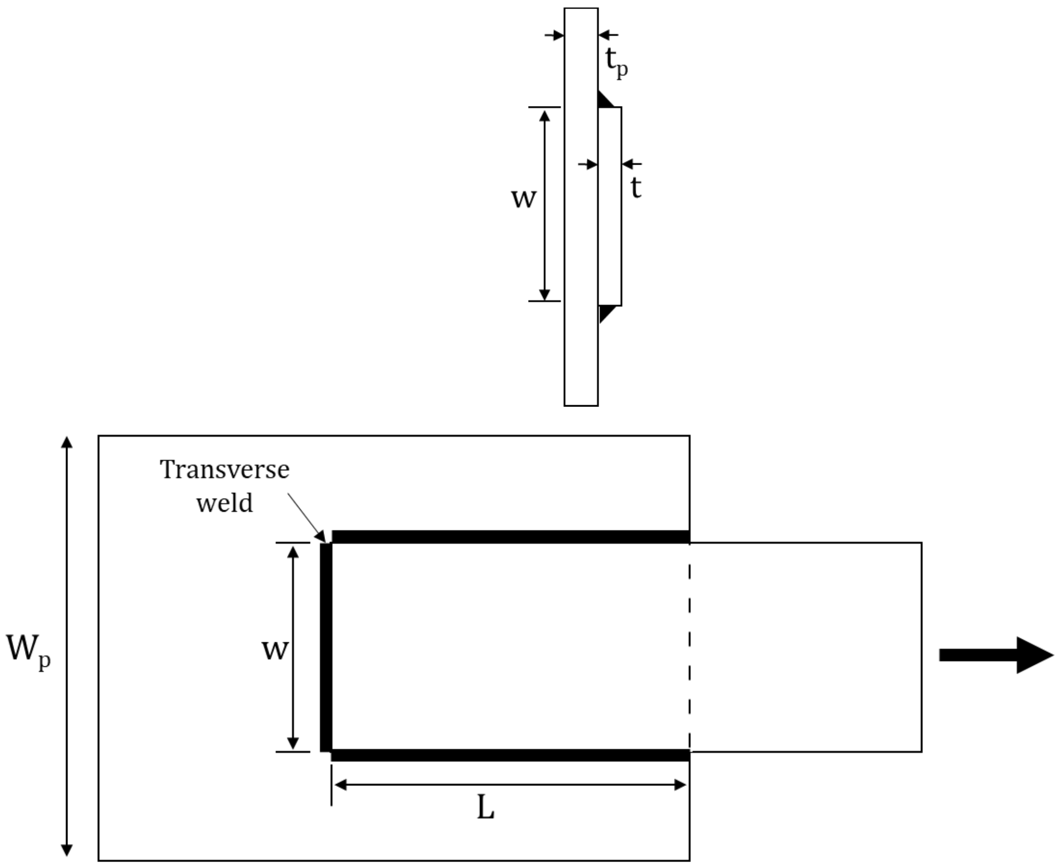


Figure 8-1: Schematic of Welded Plate Connection

The shear lag factor cases applicable to this connection are Cases 1 and 4. Case 1 is applicable when transverse welds are used. The shear lag factor equals 1.0 for Case 1. Case 4 is applicable when only longitudinal welds are used. The shear lag factor for Case 4 is:

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) \tag{8-1}$$

where l and w are defined in Figure 6-2 and $\bar{x}$ is eccentricity of the connection (one half the tension member thickness, t).

The gross area was calculated based on measured dimensions.

8.2 Comparison to Experimental Results

8.2.1 Description of Experimental Specimens

A total of 28 welded plate specimens from 2 references were identified for evaluation in this work as detailed in Table 8.1. Some of the specimens described in these references were not included. Gonzalez (1989) reported 11 specimens total, however specimens P-L1-1a and P-B-1a were removed because it reported the testing machine reached its full capacity before failure could occur. Note that details reported by Gonzalez (1989) are also reported by Easterling and Gonzalez Giroux (1993). Mannem (2002) reported 27 total specimens, but specimens OP120-S-a and P120-1-b failed in the welds. Specimens OP120-s-b, DP120-s, P120-s, P75-s, P75-s-b, and DP75-s were all removed because they included only one longitudinal weld. These specimens were deemed a special case and removed from the study.

Table 8.1: Count of Welded Plate Connections by Reference

Reference	Specimen Count
Gonzalez (1989)	9
Mannem (2002)	19
Total	**28**

The parameters associated with each specimen are provided in Table 8.2 and Table 8.3. The measured F_y and F_u of the Mannem (2002) specimens were provided as a result of three coupon tests. The average of the three was taken as the measured F_y and F_u. Mannem did not indicate a weld size. Mannem did report a metric weld strength of E760XX (E110XX in imperial units) and noted the welds were designed not to fail. A weld size of 0.315 in. (8 mm) and strength of E760XX (metric) was used for modelling in IDEA StatiCa. An entry of 'NP' in Table 8.3 indicates that the weld size was not provided in the literature. The dashes, '-', indicate there is not a weld in that particular location.

Table 8.2: Welded Plate Connection Experimental Specimen Parameters

Index	Reference	Specimen	# of plates	*F_y, ksi	*F_u, ksi	*t, in.	*w, in.
1	Gonzalez (1989)	P-L1-1b	2	51.9	73.0	0.260	3.020
2	Gonzalez (1989)	P-L1-2	2	51.9	73.0	0.259	3.024
3	Gonzalez (1989)	P-L1-3	2	51.9	73.0	0.258	3.025
4	Gonzalez (1989)	P-L2-1	2	51.9	73.0	0.260	3.020
5	Gonzalez (1989)	P-L2-2	2	51.9	73.0	0.259	3.025
6	Gonzalez (1989)	P-L2-3	2	51.9	73.0	0.257	3.023
7	Gonzalez (1989)	P-B-1b	2	51.9	73.0	0.258	3.024
8	Gonzalez (1989)	P-B-2	2	51.9	73.0	0.257	3.024
9	Gonzalez (1989)	P-B-3	2	51.9	73.0	0.259	3.023
10	Mannem (2002)	OP120-1-a	1	30.5	62.6	0.505	4.724
11	Mannem (2002)	OP120-1-b	1	30.5	62.6	0.505	4.720
12	Mannem (2002)	OP120-T	1	30.5	62.6	0.505	4.724
13	Mannem (2002)	P120-1-a	1	53.2	73.5	0.511	4.748
14	Mannem (2002)	P120-1-c	1	53.2	73.5	0.511	4.756
15	Mannem (2002)	P120-1.5	1	53.2	73.5	0.511	4.756
16	Mannem (2002)	DP120-1	2	53.2	73.5	0.511	4.803
17	Mannem (2002)	P120-T-a	1	53.2	73.5	0.511	4.780
18	Mannem (2002)	P120-T-b	1	53.2	73.5	0.511	4.736
19	Mannem (2002)	DP120-2	2	53.2	73.5	0.511	4.732
20	Mannem (2002)	P75-0.87	1	53.2	73.5	0.511	3.000
21	Mannem (2002)	P75-1.0	1	53.2	73.5	0.511	3.051
22	Mannem (2002)	P75-1.6	1	53.2	73.5	0.511	3.063
23	Mannem (2002)	P75-2.0	1	53.2	73.5	0.511	2.992
24	Mannem (2002)	P75-T	1	53.2	73.5	0.511	2.972
25	Mannem (2002)	P75-e3/4	1	53.2	73.5	0.511	2.953
26	Mannem (2002)	P75-e5/4	1	53.2	73.5	0.511	2.972
27	Mannem (2002)	UP75-3/4	1	53.2	73.5	0.511	2.906
28	Mannem (2002)	P250-1	1	53.2	73.5	0.511	9.862

* indicates measured value

Table 8.3: Welded Plate Connection Experimental Specimen Weld Details

Index	Longitudinal Weld Size, in.	*L*, in.	Transverse Weld Size, in.	Transverse Weld Length, in.
1	0.250	4.250	-	-
2	0.250	4.250	-	-
3	0.250	4.250	-	-
4	0.250	5.000	-	-
5	0.250	5.000	-	-
6	0.250	5.000	-	-
7	0.250	3.000	0.250	3.000
8	0.250	3.000	0.250	3.000
9	0.250	3.000	0.250	3.000
10	NP	4.528	-	-
11	NP	4.528	-	-
12	NP	2.165	NP	4.724
13	NP	4.724	-	-
14	NP	4.724	-	-
15	NP	7.087	-	-
16	NP	4.724	-	-
17	NP	3.937	NP	4.780
18	NP	1.969	NP	4.736
19	NP	9.252	-	-
20	NP	2.559	-	-
21	NP	2.953	-	-
22	NP	4.724	-	-
23	NP	6.024	-	-
24	NP	1.181	NP	2.953
25	NP	2.953	-	-
26	NP	2.953	-	-
27	NP	3.740	-	-
28	NP	9.843	-	-

NP: not provided

The modeling process for the welded plate connections in IDEA StatiCa is identical to that for the welded angle connections. The tested plate and gusset plate are both modeled as a member. A typical model is shown in Figure 8-2. For clarity, the tested plate is referred to as ‘plate’ and the non-tested plate is referred to as the ‘gusset plate’. The typical process for modelling the specimens is shown in Figure 8-3.

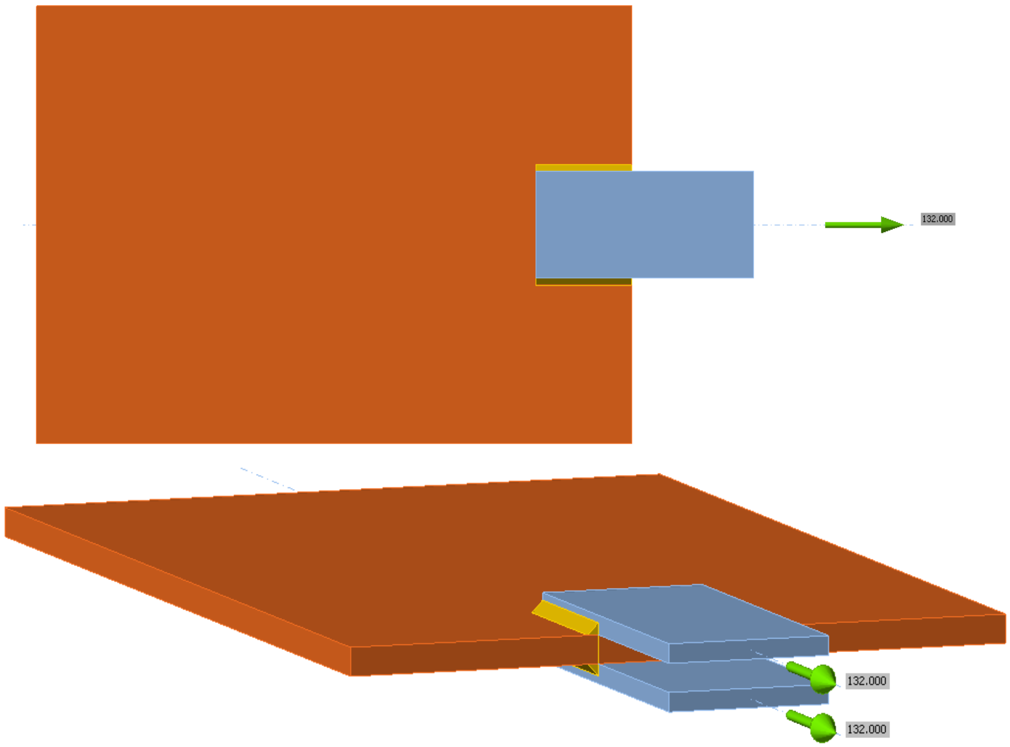


Figure 8-2: Typical Welded Plate Connection Model in IDEA StatiCa

4. Create members:
    a. Member 1: Gusset plate
        i. Geometrical type set to “Ended”
        ii. B set to 180º
        iii. Offset ex set to the negative of the connection length (longest longitudinal weld length)
    b. Member 2, 3 (if applicable)
        i. Geometrical type set to “Ended”
        ii. Align set to “To member plate”
        iii. Aligned plate set to “[member name] | Bottom flange 1”
        iv. Related plate set to “[gusset plate member name] Bottom flange 1”
        v. Model type set to “N-Vy-Vz”
5. Create weld operations:
    a. Continuous welds:
        i. Placement set to “Edge to surface”
        ii. Type set to “Weld”
        iii. First plate: Member or plate set to “[plate member name] | Bottom flange 1”
        iv. Select correct edge index
        v. Second plate: Plate set to “[gusset plate name] | Bottom flange 1”
        vi. Input desired weld size and type (fillet- front side for all specimens)
        vii. Weld type set to “Continuous”
    b. Partial welds:
        i. Placement set to “Edge to surface”
        ii. Type set to “Weld”
        iii. First plate: Member or plate set to “[plate member name] | Bottom flange 1”
        iv. Select correct edge index
        v. Second plate: Plate set to “[gusset plate name] | Bottom flange 1”
        vi. Input desired weld size and type (fillet- front side for all specimens)
        vii. Weld type set to “Partial”
        viii. Weld offset 2 set to the absolute value of the offset in Step 2 minus the partial weld length
    c. Repeat as necessary until all welds are modeled

Figure 8-3: Modelling Process for Welded Plate Connections

8.2.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 8.4: AISC Calculation Results for Welded Plate Connections

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
1	81.5	0.830	Case 4	95.1
2	81.3	0.830	Case 4	94.8
3	81.1	0.830	Case 4	94.6
4	81.5	0.868	Case 4	99.5
5	81.4	0.868	Case 4	99.4
6	80.7	0.868	Case 4	98.5
7	81.0	1.000	Case 1	113.9
8	80.7	1.000	Case 1	113.4
9	81.3	1.000	Case 1	114.3
10	72.6	0.693	Case 4	103.4
11	72.6	0.693	Case 4	103.4
12	72.6	1.000	Case 1	149.3
13	128.9	0.708	Case 4	126.1
14	129.1	0.707	Case 4	126.2
15	129.1	0.838	Case 4	149.6
16	260.9	0.704	Case 4	253.6
17	129.8	1.000	Case 1	179.3
18	128.6	1.000	Case 1	177.7
19	257.0	0.894	Case 4	317.6
20	81.5	0.617	Case 4	69.5
21	82.9	0.674	Case 4	77.1
22	83.2	0.830	Case 4	95.4
23	81.3	0.885	Case 4	99.3
24	80.7	1.000	Case 1	111.5
25	80.2	0.685	Case 4	75.9
26	80.7	0.683	Case 4	76.2
27	78.9	0.727	Case 4	79.3
28	267.8	0.730	Case 4	270.1

Table 8.5: Summary Strength Results for Welded Plate Connections

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kip	**Failure Mode**	**P_{AISC}, kips**	**Controlling Limit State**	**P_{IDEA}, kips**	**Failure Mode**
1	107.4	[2]	81.5	[1]	83.6	[3]
2	112.0	[2]	81.3	[1]	83.4	[3]
3	115.0	[2]	81.1	[1]	83.1	[3]
4	111.8	[2]	81.5	[1]	83.6	[3]
5	111.6	[2]	81.4	[1]	83.4	[3]
6	108.8	[2]	80.7	[1]	82.7	[3]
7	102.4	[2]	81.0	[1]	83.0	[3]
8	112.2	[2]	80.7	[1]	82.7	[3]
9	111.4	[2]	81.3	[1]	83.3	[3]
10	119.4	[2]	72.6	[1]	75.0	[3]
11	114.2	[2]	72.6	[1]	75.0	[3]
12	127.9	[2]	72.6	[1]	75.0	[3]
13	168.6	[2]	126.1	[2]	128.2	[3]
14	164.8	[2]	126.2	[2]	96.7	[3]
15	170.0	[2]	129.1	[1]	128.2	[3]
16	341.9	[2]	253.6	[2]	267.3	[3]
17	171.5	[2]	129.8	[1]	128.9	[3]
18	173.1	[2]	128.6	[1]	127.7	[3]
19	321.7	[2]	257.0	[1]	263.3	[3]
20	107.5	[2]	69.5	[2]	80.9	[3]
21	109.9	[2]	77.1	[2]	82.4	[3]
22	112.4	[1]	83.2	[1]	82.6	[3]
23	109.9	[2]	81.3	[1]	80.7	[3]
24	105.9	[2]	80.7	[1]	77.8	[3]
25	109.9	[2]	75.9	[2]	78.2	[3]
26	107.9	[2]	76.2	[2]	82.1	[3]
27	106.8	[2]	78.9	[1]	78.4	[3]
28	327.3	[2]	267.8	[1]	244.2	[3]

[1] tensile yield; [2] tensile rupture; [3] member strain

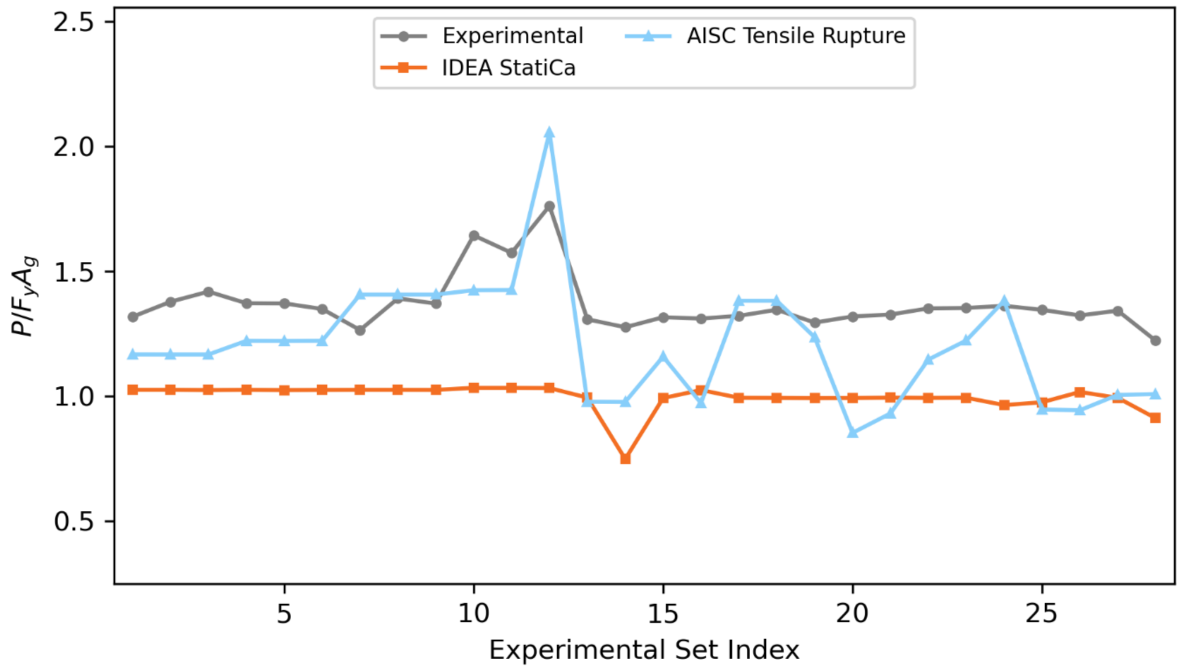


Figure 8-4: Normalized Strength Results for Welded Plate Connections

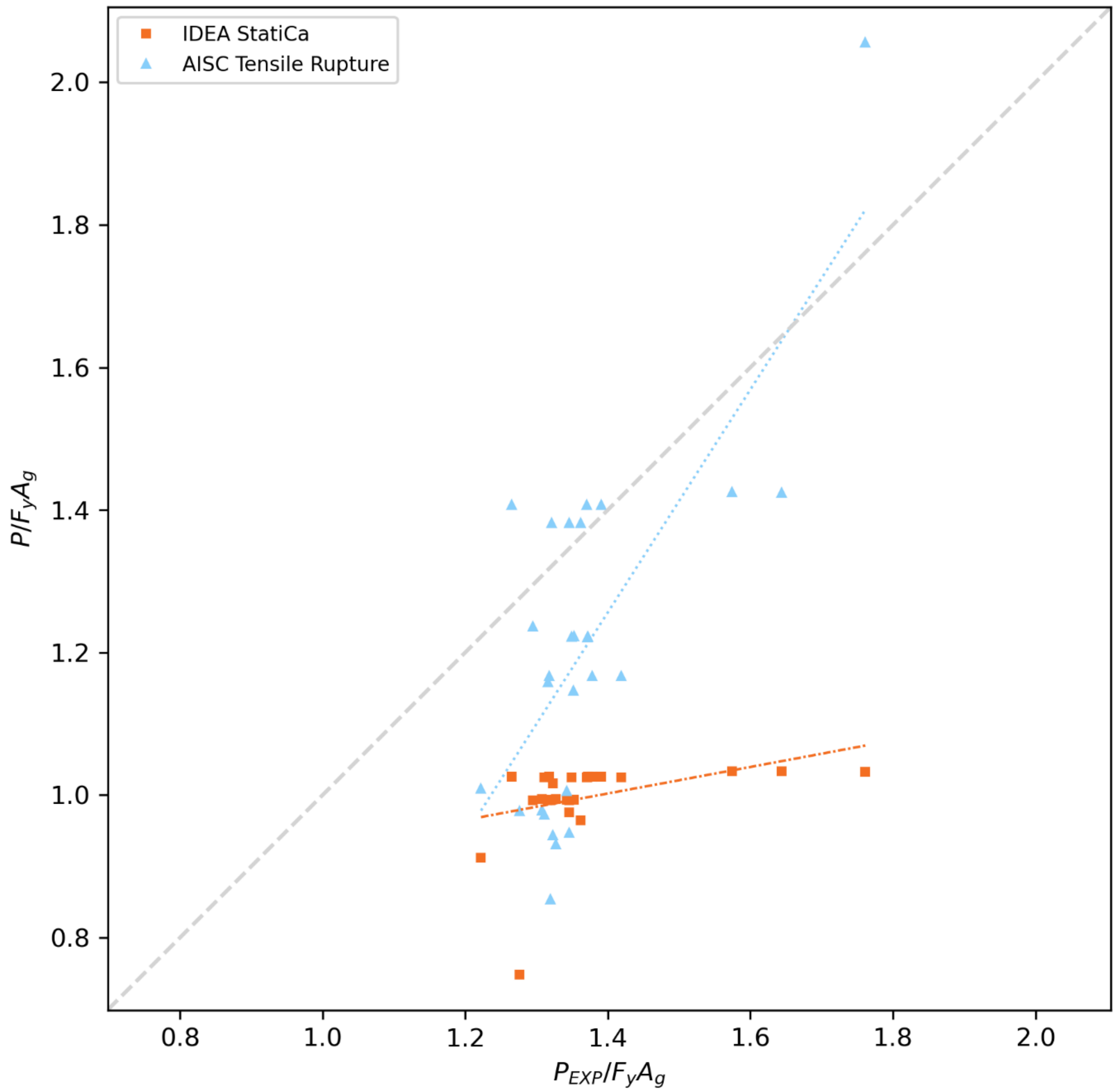


Figure 8-5: Normalized Strength vs. Normalized Experimental Strength for Welded Plate Connections

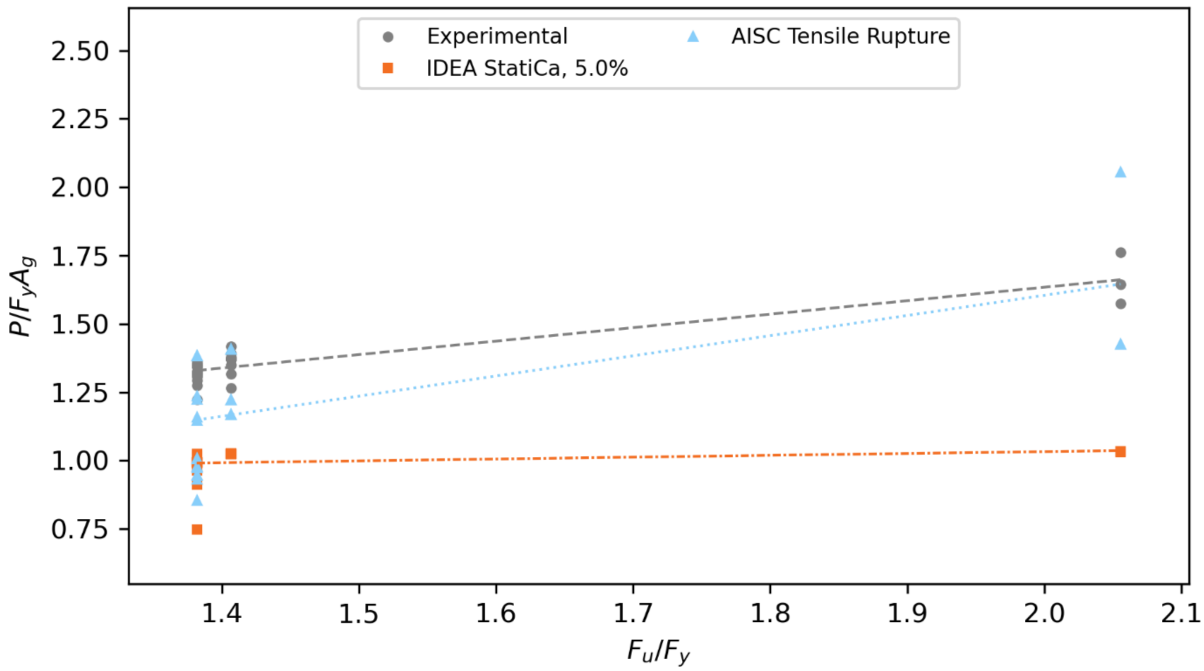


Figure 8-6: Normalized Strength vs. Material Strength Ratio for Welded Plate Connections

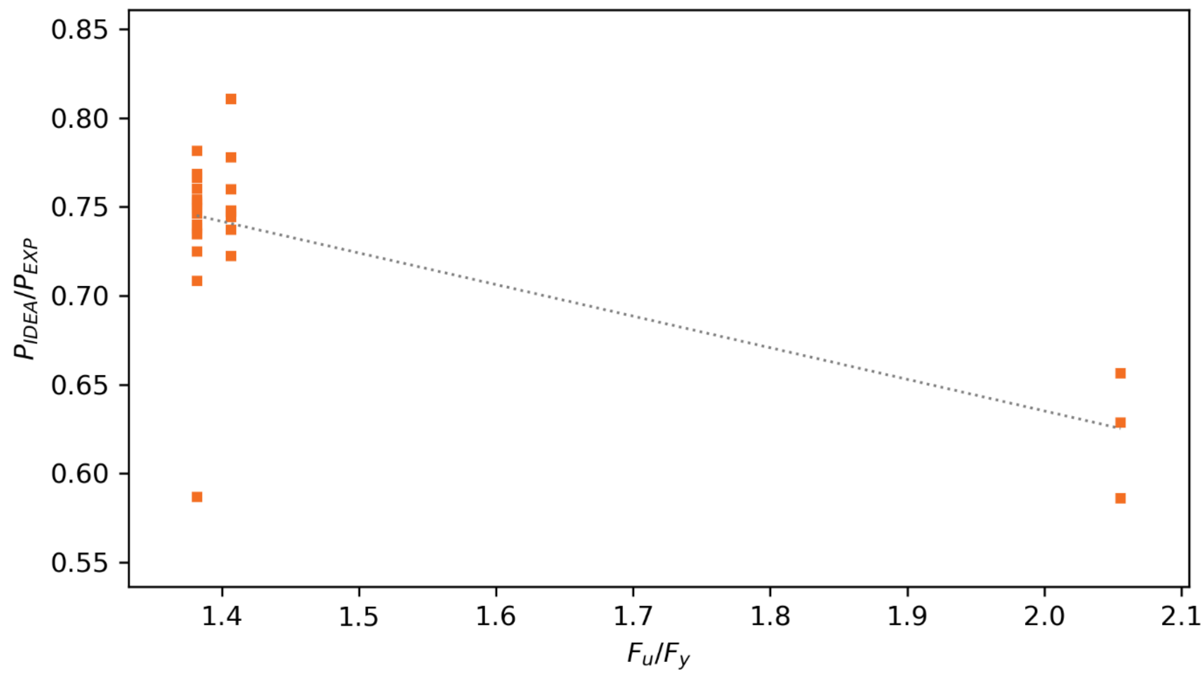


Figure 8-7: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Welded Plate Connections

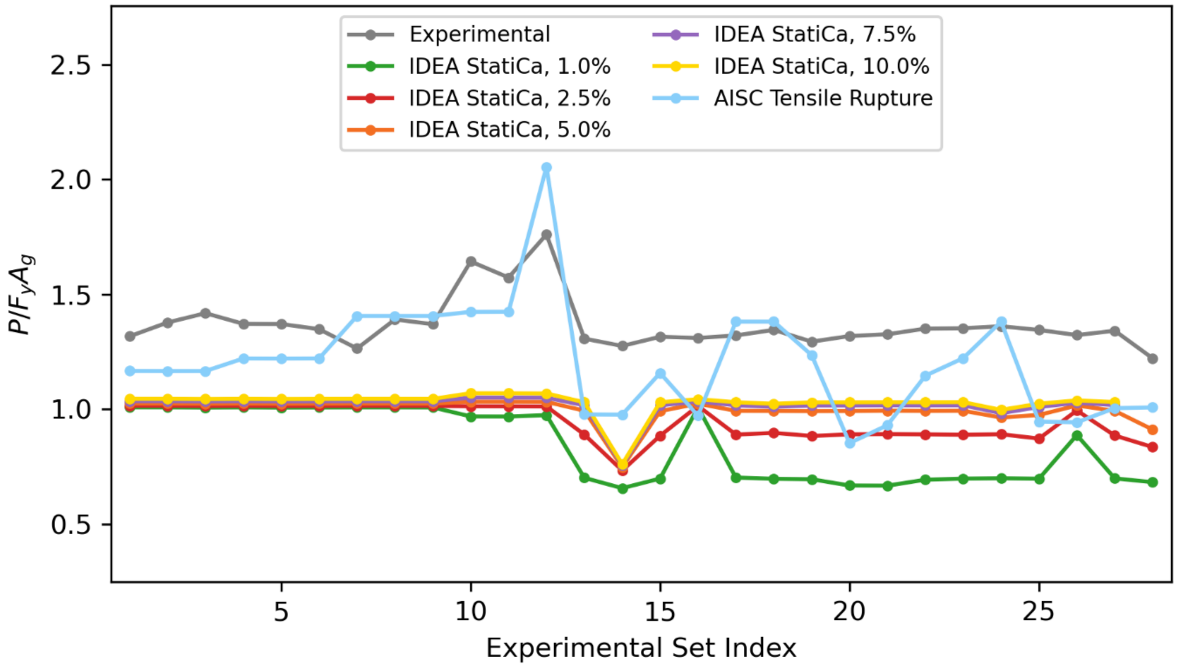


Figure 8-8: Normalized Strength Results for Welded Plate Connections Including Various Plastic Strain Limits for IDEA StatiCa

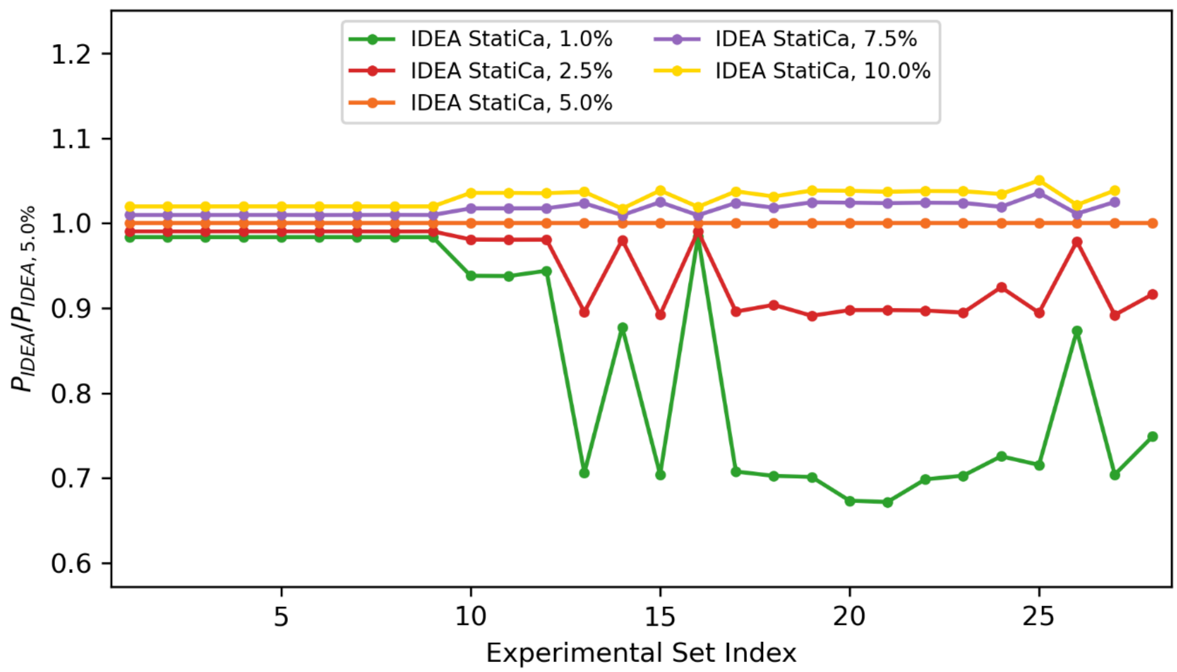


Figure 8-9: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Welded Plate Connections

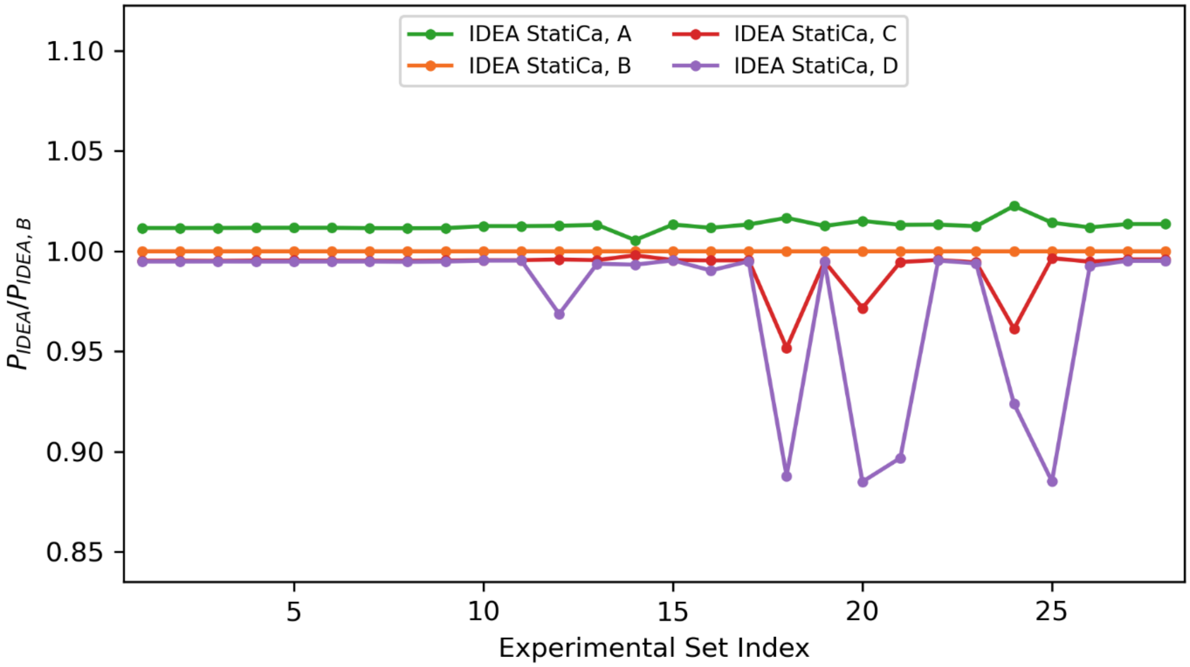


Figure 8-10: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Welded Plate Connections

Table 8.6: Summary Statistics of the Test-to-Predicted Ratio for Welded Plate Connections

Test-to-Predicted Ratio	Average	Standard Deviation	Coefficient of Variation
$P_{EXP}/P_{RUPTURE}$ (AISC 2016)	1.163	0.175	0.150
P_{EXP}/P_{IDEA}	1.376	0.115	0.084

8.2.3 Discussion

Tensile rupture was the reported failure mode for all but one of the experiments in this set. However, tensile yield controlled the AISC strength calculations for most of the specimens (Table 8.5).

There were several cases in this experimental set where the maximum permitted applied load in IDEA StatiCa, P_{IDEA}, resulted in a larger value than indicated by the AISC *Specification* equations, P_{AISC}. The most severe case was a result of the specimen with the experimental index 20. The P_{IDEA} for this specimen was around 16% higher than P_{AISC}. The most conservative specimen, experimental set index 14, resulted in an IDEA StatiCa strength of around 23% less than the AISC *Specification* equations suggest. In comparison to the experimental strengths, P_{EXP}, IDEA StatiCa strength never exceeded the experimental for this particular set of experimental specimens.

The IDEA StatiCa strength was close to F_yA_g for all specimens except 14 (Figure 8-4). This is the maximum strength that can be achieved by IDEA StatiCa and indicates that software is predicting minimal effects from shear lag and strain and stress concentrations. Physically, none of the specimens experienced tensile rupture before approximately 120% of F_yA_g. However, the AISC equations would predict tensile rupture to control for some cases.

With respect to the material strength ratio F_u/F_y, clear increases with increasing F_u/F_y are seen for the experimental and AISC strengths (Figure 8-6). As expected, when normalized by F_yA_g, the IDEA StatiCa strength does not significantly vary with F_u/F_y. Correspondingly, the ratio P_{IDEA}/P_{EXP} is seen to decrease with increasing F_u/F_y (Figure 8-7).

The variation of P_{IDEA} with plastic strain limit exhibited the expected trend of increasing strength with increasing plastic strain limit and vice versa (Figure 8-8 and Figure 8-9). Given that nearly all of the specimens achieved a strength of F_yA_g, increasing the plastic strain limit had only a minor effect on the strength. The IDEA StatiCa strengths only increased by 1% to 2% when changing the plastic strain limit from 5% to 10%. Reducing the plastic strain limit, however, does decrease the strength.

The variation of P_{IDEA} with mesh parameters also showed minimal changes for most specimens (Figure 8-10) Also, for most specimens, as the mesh parameters became more refined (i.e., smaller elements), the difference in strengths from mesh-to-mesh decreases. For some specimens (i.e., 12, 18, 20, 21, 24, and 25) convergence is not observed and the difference in strengths from mesh-to-mesh increases as the mesh is refined. For the most severe case with specimen 25, the largest decrease in strength from one mesh setting to the next was ~11%.

8.3 Reliability Analysis

8.3.1 Description of Reliability Set

The reliability set specimens were created based on selected parameters and ratios. The specific parameters are outlined in Table 8.7. It was desired to vary the following: width to thickness ratio, material grade, length of longitudinal welds, and the option of a transverse weld. The option of the transverse weld was to vary the shear lag factor case between case 1 and 4 from D3.1 of the AISC *Specification* (2016). The modeling of these specimens in IDEA StatiCa was practically identical to that outlined earlier in this chapter, except that nominal properties were used instead of measured properties. The welds were modeled as 5/16 in. welds with a weld strength of EXX110. This was to reduce the impact of weld failures.

Table 8.7: Welded Plate Connection Reliability Set Parameters

Index	*W*, in.	*t*, in.	Material Grade	F_y, ksi	F_u, ksi	Length of Weld, in.	Transverse weld?
1	5	0.750	A36 (Plate)	36	58	2.5	No
2	5	0.500	A36 (Plate)	36	58	2.5	No
3	5	0.375	A36 (Plate)	36	58	2.5	No
4	5	0.750	A36 (Plate)	36	58	5.0	No
5	5	0.500	A36 (Plate)	36	58	5.0	No
6	5	0.375	A36 (Plate)	36	58	5.0	No
7	5	0.750	A36 (Plate)	36	58	10.0	No
8	5	0.500	A36 (Plate)	36	58	10.0	No
9	5	0.375	A36 (Plate)	36	58	10.0	No
10	5	0.750	A36 (Plate)	36	58	2.5	Yes
11	5	0.500	A36 (Plate)	36	58	2.5	Yes
12	5	0.375	A36 (Plate)	36	58	2.5	Yes
13	5	0.750	A36 (Plate)	36	58	5.0	Yes
14	5	0.500	A36 (Plate)	36	58	5.0	Yes
15	5	0.375	A36 (Plate)	36	58	5.0	Yes
16	5	0.750	A36 (Plate)	36	58	10.0	Yes
17	5	0.500	A36 (Plate)	36	58	10.0	Yes
18	5	0.375	A36 (Plate)	36	58	10.0	Yes
19	5	0.750	A572 Gr. 50 (Plate)	50	65	2.5	No
20	5	0.500	A572 Gr. 50 (Plate)	50	65	2.5	No
21	5	0.375	A572 Gr. 50 (Plate)	50	65	2.5	No
22	5	0.750	A572 Gr. 50 (Plate)	50	65	5.0	No
23	5	0.500	A572 Gr. 50 (Plate)	50	65	5.0	No
24	5	0.375	A572 Gr. 50 (Plate)	50	65	5.0	No
25	5	0.750	A572 Gr. 50 (Plate)	50	65	10.0	No
26	5	0.500	A572 Gr. 50 (Plate)	50	65	10.0	No
27	5	0.375	A572 Gr. 50 (Plate)	50	65	10.0	No
28	5	0.750	A572 Gr. 50 (Plate)	50	65	2.5	Yes
29	5	0.500	A572 Gr. 50 (Plate)	50	65	2.5	Yes
30	5	0.375	A572 Gr. 50 (Plate)	50	65	2.5	Yes
31	5	0.750	A572 Gr. 50 (Plate)	50	65	5.0	Yes
32	5	0.500	A572 Gr. 50 (Plate)	50	65	5.0	Yes
33	5	0.375	A572 Gr. 50 (Plate)	50	65	5.0	Yes
34	5	0.750	A572 Gr. 50 (Plate)	50	65	10.0	Yes
35	5	0.500	A572 Gr. 50 (Plate)	50	65	10.0	Yes
36	5	0.375	A572 Gr. 50 (Plate)	50	65	10.0	Yes

8.3.2 Results

The strength results from the reliability set of specimens is provided in Table 8.8. The strength according to the AISC *Specification* (2016) is provided for tensile yield and tensile rupture. The nominal and design strength are provided for both limit states. The strength according to IDEA StatiCa is provided as well with all default settings. More specifically, all resistance factors were used for this analysis. The strength results according to IDEA StatiCa provided larger strengths than the AISC *Specification* equations for much of this reliability set of specimens. The most significant cases were a result of specimens with specimen indices 19, 20, and 21. These specimens resulted in P_{IDEA} greater than the available strength calculated according to the AISC *Specification* (minimum of ϕP_{YIELD} and $\phi P_{RUPTURE}$) by more than 50%. These specimens have the smallest weld length to plate width ratio of the reliability set with a ratio of 0.5. In comparison to the experimental set, the smallest longitudinal weld length to plate width ratio without a transverse weld was 0.85 for the specimen with experimental index 20. For this case, IDEA StatiCa resulted in a 16% larger strength than the AISC *Specification* equations, which was the largest in the entire welded plate experimental set. However, IDEA StatiCa was still 25% less than the experimentally observed strength for experimental index 20. This could indicate that the AISC *Specification* equations are conservative for shorter weld lengths.

Table 8.8: Summary Strength Results for Welded Plate Connection Reliability Set

Index	**AISC**				**IDEA StatiCa**
	P_{YIELD}, kips	**$P_{RUPTURE}$, kips**	**ϕP_{YIELD}, kips**	**$\phi P_{RUPTURE}$, kips**	**P_{IDEA}, kips**
1	135.0	79.2	121.5	59.4	77.3
2	90.0	55.9	81.0	41.9	53.9
3	67.5	43.1	60.8	32.3	41.7
4	135.0	150.9	121.5	113.2	122.7
5	90.0	103.3	81.0	77.5	83.5
6	67.5	78.5	60.8	58.9	62.8
7	135.0	193.2	121.5	144.9	122.4
8	90.0	130.5	81.0	97.9	83.5
9	67.5	98.5	60.8	73.9	62.8
10	135.0	217.5	121.5	163.1	122.7
11	90.0	145.0	81.0	108.8	83.5
12	67.5	108.8	60.8	81.6	62.8
13	135.0	217.5	121.5	163.1	122.7
14	90.0	145.0	81.0	108.8	83.5
15	67.5	108.8	60.8	81.6	62.8
16	135.0	217.5	121.5	163.1	122.6
17	90.0	145.0	81.0	108.8	83.5
18	67.5	108.8	60.8	81.6	62.8
19	187.5	88.8	168.8	66.6	103.8
20	125.0	62.7	112.5	47.0	72.5
21	93.8	48.3	84.4	36.2	55.8
22	187.5	169.1	168.8	126.8	154.7
23	125.0	115.8	112.5	86.8	114.9
24	93.8	88.0	84.4	66.0	86.5
25	187.5	216.6	168.8	162.4	154.5
26	125.0	146.3	112.5	109.7	114.8
27	93.8	110.4	84.4	82.8	86.5
28	187.5	243.8	168.8	182.8	155.5
29	125.0	162.5	112.5	121.9	114.7
30	93.8	121.9	84.4	91.4	86.5
31	187.5	243.8	168.8	182.8	155.6
32	125.0	162.5	112.5	121.9	114.9
33	93.8	121.9	84.4	91.4	86.5
34	187.5	243.8	168.8	182.8	155.4
35	125.0	162.5	112.5	121.9	114.8
36	93.8	121.9	84.4	91.4	86.5

The results of the reliability analysis are provided in Figure 8-11. For IDEA StatiCa, most of the cases compare very well to the reliability for the AISC *Specification* (2016) equations. However, there are several of cases were IDEA StatiCa provides a lower level of reliability in comparison to AISC. Specimens with index 1, 2, 3, 19, 20, 21, 22, 23, and 24 have a reliability index significantly lower for IDEA StatiCa than the corresponding reliability index for AISC. All of these specimens have a weld length to plate width ratio of 0.5, which is the smallest for this reliability set. These β-value results are consistent with the reliability set strength results discussed previously. The lowest reliability index for IDEA StatiCa was approximately 2.20 and 3.17 for AISC.

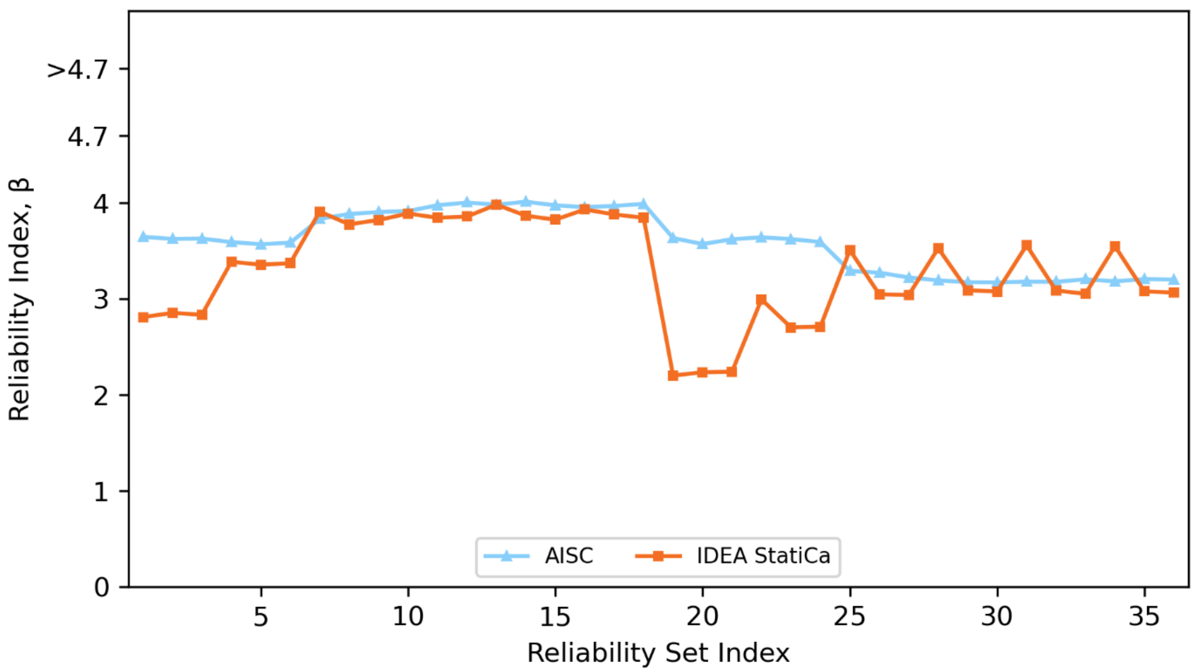


Figure 8-11: Reliability Index for Welded Plate Connections

Chapter 9:
BOLTED PLATES

9.1 Description of Connection

Single and double plate tension members bolted or riveted to a gusset plate are evaluated in this chapter. A typical schematic with relevant terminology is shown in Figure 9-1. For the double plate specimens, the plates are identical in cross section and material. Connections with both regular and staggered bolt patterns are evaluated in this chapter, but in separate sections.

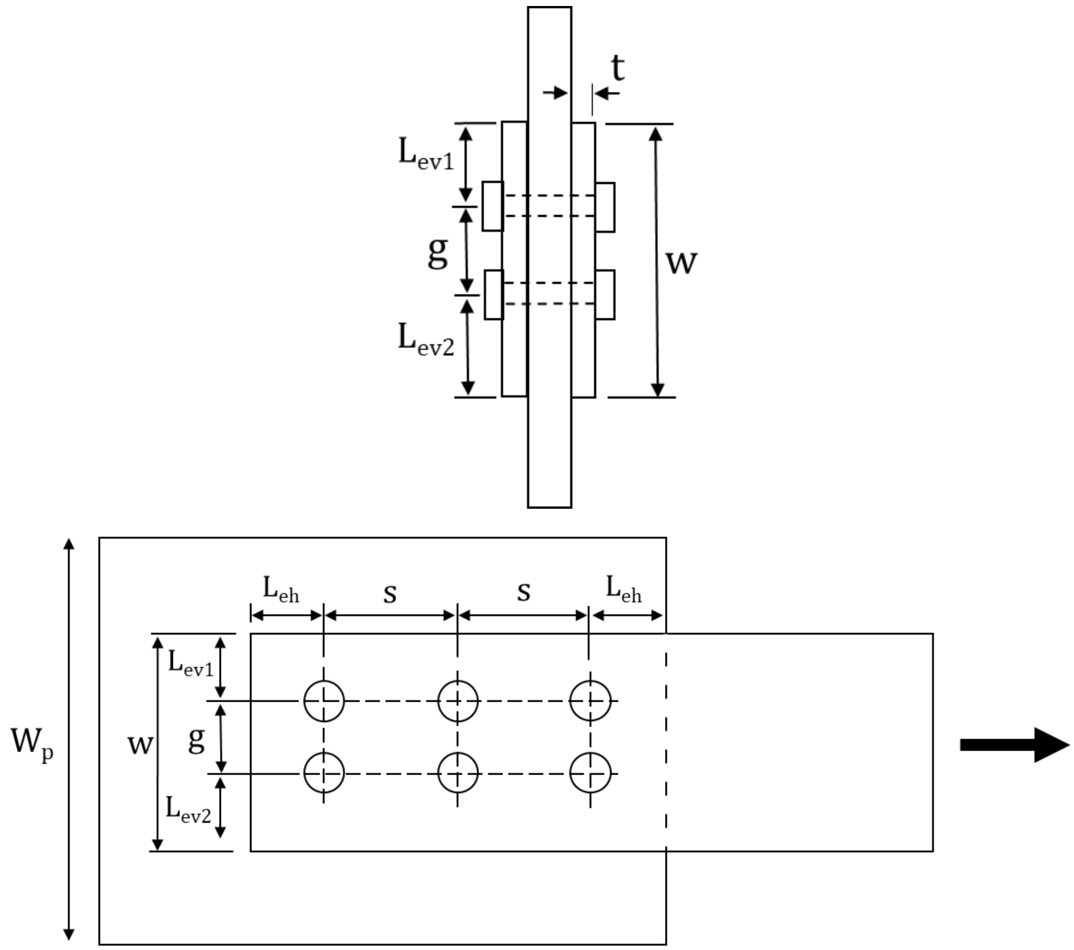


Figure 9-1: Schematic of Bolted Plate Connection

The shear lag factor was taken as unity (i.e., U = 1.0) for all specimens in this chapter since Case 1 of AISC *Specification* Table D3.1 (Figure 3-3) applied. The gross area was calculated based on measured dimensions.

9.2 Comparison to Experimental Results

9.2.1 Description of Experimental Specimens

A total of 109 bolted plate specimens (with regular bolt patterns) from 3 references were identified for evaluation in this work as detailed in Table 9.1. Some of the specimens described in these references were not included. Može and Beg (2010) reported 38 total specimens however eight specimens were removed due to bolt failures (B109, B110, B118, B202, B206, B208, B209, and B210) and ten specimens were removed due to 'splitting failures' believed to be the equivalent of bolt tearout. These specimens were reported as B102, B103, B111, B112, B116, B117, B119, B120, B121, and B211. Additionally, specimens B108, B115, B124, and B125 contained an eccentric bolt hole pattern and were removed. Munse (1959) reported 131 total riveted specimens tested under tensile loading. A large majority of these specimens included a staggered bolt pattern. There were 81 total specimens with a normal fastener configuration. Among these, 14 specimens were removed. Specimens 51-4-1, 51-4-2, 51-4-3, 50-X6-1, 50-X6-2, 50-X6-1A, 50-X6-2A, 50-X6-3A, 50-X7-1, 50-X7-2, and 50-X7-3 experienced rivet failures, and specimens 50-X7-1A, 50-X7-2A, and 50-X7-3A experienced plate tearing. As for Schutz and Newmark (1952), 130 specimens were reported, with only 26 consisting of a regular bolt pattern. Of these 26 specimens, only two were removed. Specimen SE1A was removed for bolt failure, and specimen SE1B was outside the minimum tolerance for edge distance modeling in IDEA StatiCa.

Additionally, a series of tests reported by Davis et al. (1940) was evaluated but is not included in this work. The specimens they tested were a lap plate configuration. When evaluated in IDEA StatiCa, these connections experienced severe bending due to the eccentricity between the plates since IDEA StatiCa defaults to first-order analyses. As a result, the strengths from IDEA StatiCa were low. Subsequent second order (i.e., geometric nonlinear) analyses in IDEA StatiCa revealed a more physically realistic response and strengths closer to the experimental strength. Further investigation of the use of second order analysis in IDEA StatiCa is recommended.

Table 9.1: Count of Bolted Plate Connections by Reference

Reference	**Specimen Count**
Može and Beg (2010)	16
Munse (1959)	67
Schutz & Newmark (1952)	26
Total	**109**

The parameters associated with each specimen are provided in Table 9.2 and Table 9.3. The terminology of both tables is consistent with Figure 9-1. For specimens with only one row of bolts, the gage dimension, *g*, is not defined and is indicated by 'N/A'. This is likewise for specimens with only one bolt per row and the bolt spacing, 's'. If provided, the diameter of the bolt or rivet hole was taken as reported. Otherwise, bolt holes were taken as indicated by Table J3.3 of the AISC *Specification*. For rivets, the diameter of the hole was taken as the rivet diameter unless specifically reported by the experimentalist. Additionally, for the AISC *Specification* calculations of the net area, an additional 1/16 in. was added to the hole diameter in accordance with AISC *Specification* Section B4.3b.

Table 9.2: Bolted Plate Connection Experimental Specimen Parameters

Index	Reference	Specimen	# of plates	*F_y, ksi	*F_u, ksi	*t, in.	*W, in.
1	Može and Beg (2010)	B101	1	122.8	128.4	0.394	2.402
2	Može and Beg (2010)	B104	1	122.8	128.4	0.394	2.823
3	Može and Beg (2010)	B105	1	122.8	128.4	0.394	2.835
4	Može and Beg (2010)	B106	1	122.8	128.4	0.394	3.189
5	Može and Beg (2010)	B107	1	122.8	128.4	0.394	3.169
6	Može and Beg (2010)	B113	1	122.8	128.4	0.394	3.555
7	Može and Beg (2010)	B114	1	122.8	128.4	0.394	3.543
8	Može and Beg (2010)	B122	1	122.8	128.4	0.394	4.677
9	Može and Beg (2010)	B123	1	122.8	128.4	0.394	3.098
10	Može and Beg (2010)	B201	1	122.8	128.4	0.400	3.827
11	Može and Beg (2010)	B203	1	122.8	128.4	0.400	4.563
12	Može and Beg (2010)	B204	1	122.8	128.4	0.400	4.567
13	Može and Beg (2010)	B205	1	122.8	128.4	0.400	4.886
14	Može and Beg (2010)	B207	1	122.8	128.4	0.400	5.394
15	Može and Beg (2010)	B212	1	122.8	128.4	0.400	5.685
16	Može and Beg (2010)	B213	1	122.8	128.4	0.400	4.764
17	Munse (1959)	50-1A	2	37.8	63.4	0.313	8.800
18	Munse (1959)	50-1B	2	37.8	63.4	0.313	8.800
19	Munse (1959)	50-1C	2	37.8	63.4	0.313	8.800
20	Munse (1959)	50-2A	2	37.0	59.7	0.250	10.330
21	Munse (1959)	50-2B	2	37.0	59.7	0.250	10.330
22	Munse (1959)	50-2C	2	37.0	59.7	0.250	10.330
23	Munse (1959)	50-3A	2	35.2	61.2	0.188	13.040
24	Munse (1959)	50-3B	2	35.2	61.2	0.188	13.040
25	Munse (1959)	50-3C	2	35.2	61.2	0.188	13.040
26	Munse (1959)	50-4A	2	36.0	62.0	0.375	10.020
27	Munse (1959)	50-4B	2	36.0	62.0	0.375	10.020
28	Munse (1959)	50-4C	2	36.0	62.0	0.375	10.020
29	Munse (1959)	50-5A	2	37.8	63.4	0.313	11.480
30	Munse (1959)	50-5B	2	37.8	63.4	0.313	11.480
31	Munse (1959)	50-5C	2	37.8	63.4	0.313	11.480
32	Munse (1959)	50-6A	2	39.2	62.3	0.250	13.640
33	Munse (1959)	50-6B	2	39.2	62.3	0.250	13.640
34	Munse (1959)	50-6C	2	39.2	62.3	0.250	13.640
35	Munse (1959)	50-7A	2	35.3	60.1	0.188	17.240
36	Munse (1959)	50-7B	2	35.3	60.1	0.188	17.240
37	Munse (1959)	50-7C	2	35.3	60.1	0.188	17.240
38	Munse (1959)	50-8A	2	35.9	64.9	0.438	11.260
39	Munse (1959)	50-8B	2	35.9	64.9	0.438	11.260
40	Munse (1959)	50-8C	2	35.9	64.9	0.438	11.260
41	Munse (1959)	50-9A	2	36.2	62.7	0.375	12.620
42	Munse (1959)	50-9B	2	36.2	62.7	0.375	12.620

Table 9.2 continued

Index	Reference	Specimen	# of plates	*F_y, ksi	*F_u, ksi	*t, in.	*W, in.
43	Munse (1959)	50-9C	2	36.2	62.7	0.375	12.620
44	Munse (1959)	50-10A	2	37.6	62.9	0.313	14.500
45	Munse (1959)	50-10B	2	37.6	62.9	0.313	14.500
46	Munse (1959)	50-10C	2	37.6	62.9	0.313	14.500
47	Munse (1959)	50-11A	2	37.1	60.1	0.250	17.320
48	Munse (1959)	50-11B	2	37.1	60.1	0.250	17.320
49	Munse (1959)	50-11C	2	37.1	60.1	0.250	17.320
50	Munse (1959)	51-1-1	1	38.7	64.5	0.438	9.480
51	Munse (1959)	51-1-2	1	38.7	64.5	0.438	9.480
52	Munse (1959)	51-1-3	1	38.7	64.5	0.438	9.480
53	Munse (1959)	51-1-1A	1	32.0	63.2	0.438	9.480
54	Munse (1959)	51-1-2A	1	32.6	63.7	0.438	9.480
55	Munse (1959)	51-1-3A	1	31.6	62.9	0.438	9.480
56	Munse (1959)	51-2-1	1	40.2	67.7	0.375	10.340
57	Munse (1959)	51-2-2	1	40.2	67.7	0.375	10.340
58	Munse (1959)	51-2-3	1	40.2	67.7	0.375	10.340
59	Munse (1959)	51-2-1A	1	37.4	64.2	0.375	10.340
60	Munse (1959)	51-2-2A	1	40.5	64.7	0.375	10.340
61	Munse (1959)	51-2-3A	1	37.5	62.9	0.375	10.340
62	Munse (1959)	51-3-1	1	39.7	65.5	0.313	11.500
63	Munse (1959)	51-3-2	1	39.7	65.5	0.313	11.500
64	Munse (1959)	51-3-3	1	39.7	65.5	0.313	11.500
65	Munse (1959)	51-3-1A	1	39.4	67.2	0.313	11.500
66	Munse (1959)	51-3-2A	1	38.4	66.2	0.313	11.500
67	Munse (1959)	51-3-3A	1	38.3	66.9	0.313	11.500
68	Munse (1959)	51-4-1A	1	31.8	66.8	0.563	11.840
69	Munse (1959)	51-4-2A	1	31.4	66.7	0.563	11.840
70	Munse (1959)	51-4-3A	1	32.0	66.1	0.563	11.840
71	Munse (1959)	51-5-1	1	35.4	66.8	0.500	13.440
72	Munse (1959)	51-5-2	1	35.4	66.8	0.500	13.440
73	Munse (1959)	51-5-3	1	35.4	66.8	0.500	13.440
74	Munse (1959)	51-5-1A	1	38.2	62.1	0.500	13.440
75	Munse (1959)	51-5-2A	1	32.5	63.4	0.500	13.440
76	Munse (1959)	51-5-3A	1	31.4	61.8	0.500	13.440
77	Munse (1959)	51-6-1	1	37.6	64.1	0.438	15.720
78	Munse (1959)	51-6-2	1	37.6	64.1	0.438	15.720
79	Munse (1959)	51-6-3	1	37.6	64.1	0.438	15.720
80	Munse (1959)	51-6-1A	1	33.6	60.5	0.438	15.720
81	Munse (1959)	51-6-2A	1	36.0	63.2	0.438	15.720
82	Munse (1959)	51-6-3A	1	32.1	61.6	0.438	15.720
83	Munse (1959)	50-X6-3	2	47.1	64.7	0.250	8.220
84	Schutz and Newmark (1952)	AD1A	1	39.2	65.2	0.313	4.750

Table 9.2 continued

Index	Reference	Specimen	# of plates	*F_y, ksi	*F_u, ksi	*t, in.	*W, in.
85	Schutz and Newmark (1952)	AD1B	1	39.2	65.2	0.313	4.750
86	Schutz and Newmark (1952)	AD2A	1	39.2	65.2	0.313	3.500
87	Schutz and Newmark (1952)	AD2B	1	39.2	65.2	0.313	3.500
88	Schutz and Newmark (1952)	AD3A	1	39.2	65.2	0.313	7.500
89	Schutz and Newmark (1952)	AD3B	1	39.2	65.2	0.313	7.500
90	Schutz and Newmark (1952)	JA1A	1	39.2	65.2	0.313	6.000
91	Schutz and Newmark (1952)	JA1B	1	39.2	65.2	0.313	6.000
92	Schutz and Newmark (1952)	AE1A	1	39.2	65.2	0.313	1.500
93	Schutz and Newmark (1952)	AE1B	1	39.2	65.2	0.313	1.500
94	Schutz and Newmark (1952)	AP1A	1	39.2	65.2	0.313	4.500
95	Schutz and Newmark (1952)	AP1B	1	39.2	65.2	0.313	4.500
96	Schutz and Newmark (1952)	S1A	1	39.2	65.2	0.313	4.500
97	Schutz and Newmark (1952)	S1B	1	39.2	65.2	0.313	4.500
98	Schutz and Newmark (1952)	S2A	1	45.6	63.7	0.250	4.500
99	Schutz and Newmark (1952)	S2B	1	45.6	63.7	0.250	4.500
100	Schutz and Newmark (1952)	S3A	1	37.4	63.6	0.375	4.500
101	Schutz and Newmark (1952)	S3B	1	37.4	63.6	0.375	4.500
102	Schutz and Newmark (1952)	SE2A	1	35.1	65.5	0.500	6.000
103	Schutz and Newmark (1952)	SE2B	1	35.1	65.5	0.500	6.000
104	Schutz and Newmark (1952)	SE3A	1	45.6	63.7	0.250	3.000
105	Schutz and Newmark (1952)	SE3B	1	45.6	63.7	0.250	3.000
106	Schutz and Newmark (1952)	AC1A	1	39.2	65.2	0.313	10.500
107	Schutz and Newmark (1952)	AC1B	1	39.2	65.2	0.313	10.500
108	Schutz and Newmark (1952)	V1A	1	39.2	65.2	0.313	7.500
109	Schutz and Newmark (1952)	V1B	1	39.2	65.2	0.313	7.500

* indicates measured value

Table 9.3: Bolted Plate Connection Experimental Specimen Fastener Details

Index	**Bolts/ Rivets**	**Bolt Material**	***d*, in.**	**# of fasteners per row**	**# of rows of fasteners**	***s*, in.**	**L_{eh}, in.**	***g*, in.**	**L_{ev1}, in.**	**L_{ev2}, in.**
1	Bolts	12.9	1.063	1	1	N/A	3.555	N/A	1.122	1.280
2	Bolts	12.9	1.063	1	1	N/A	2.374	N/A	1.406	1.417
3	Bolts	12.9	1.063	1	1	N/A	3.520	N/A	1.370	1.465
4	Bolts	12.9	1.063	1	1	N/A	2.976	N/A	1.571	1.618
5	Bolts	12.9	1.063	1	1	N/A	3.567	N/A	1.559	1.610
6	Bolts	12.9	1.063	1	1	N/A	2.941	N/A	1.724	1.831
7	Bolts	12.9	1.063	1	1	N/A	3.543	N/A	1.724	1.819
8	Bolts	12.9	1.063	1	1	N/A	4.181	N/A	2.303	2.374
9	Bolts	10.9	0.866	1	1	N/A	3.987	N/A	1.521	1.577
10	Bolts	10.9	0.866	1	2	N/A	2.806	1.918	0.907	1.002
11	Bolts	10.9	0.866	1	2	N/A	1.899	2.239	1.134	1.190
12	Bolts	10.9	0.866	1	2	N/A	2.825	2.230	1.134	1.203
13	Bolts	10.9	0.866	1	2	N/A	2.797	2.551	1.143	1.191
14	Bolts	10.9	0.866	1	2	N/A	2.863	2.551	1.398	1.444
15	Bolts	10.9	0.866	1	2	N/A	2.872	2.844	1.389	1.452
16	Bolts	10.9	0.866	1	2	N/A	1.899	2.258	1.257	1.249
17	Rivets	A141	0.750	2	3	3.500	1.500	2.650	1.750	1.750
18	Rivets	A141	0.750	2	3	3.500	1.500	2.650	1.750	1.750
19	Rivets	A141	0.750	2	3	3.500	1.500	2.650	1.750	1.750
20	Rivets	A141	0.750	2	3	3.500	1.750	3.440	1.750	1.750
21	Rivets	A141	0.750	2	3	3.500	1.750	3.440	1.750	1.750
22	Rivets	A141	0.750	2	3	3.500	1.750	3.440	1.750	1.750
23	Rivets	A141	0.750	2	3	3.500	2.500	4.770	1.750	1.750
24	Rivets	A141	0.750	2	3	3.500	2.500	4.770	1.750	1.750
25	Rivets	A141	0.750	2	3	3.500	2.500	4.770	1.750	1.750
26	Rivets	A141	0.875	2	3	3.750	1.750	3.130	1.880	1.880
27	Rivets	A141	0.875	2	3	3.750	1.750	3.130	1.880	1.880
28	Rivets	A141	0.875	2	3	3.750	1.750	3.130	1.880	1.880
29	Rivets	A141	0.875	2	3	3.750	2.000	3.860	1.880	1.880
30	Rivets	A141	0.875	2	3	3.750	2.000	3.860	1.880	1.880
31	Rivets	A141	0.875	2	3	3.750	2.000	3.860	1.880	1.880
32	Rivets	A141	0.875	2	3	3.750	2.500	4.940	1.880	1.880
33	Rivets	A141	0.875	2	3	3.750	2.500	4.940	1.880	1.880
34	Rivets	A141	0.875	2	3	3.750	2.500	4.940	1.880	1.880
35	Rivets	A141	0.875	2	3	3.750	3.250	6.740	1.880	1.880
36	Rivets	A141	0.875	2	3	3.750	3.250	6.740	1.880	1.880
37	Rivets	A141	0.875	2	3	3.750	3.250	6.740	1.880	1.880
38	Rivets	A141	1.000	2	3	4.000	1.750	3.630	2.000	2.000
39	Rivets	A141	1.000	2	3	4.000	1.750	3.630	2.000	2.000
40	Rivets	A141	1.000	2	3	4.000	1.750	3.630	2.000	2.000
41	Rivets	A141	1.000	2	3	4.000	2.000	4.310	2.000	2.000
42	Rivets	A141	1.000	2	3	4.000	2.000	4.310	2.000	2.000

Table 9.3 continued

Index	Bolts/ Rivets	Bolt Material	*d*, in.	# of fasteners per row	# of rows of fasteners	*s*, in.	L_{eh}, in.	*g*, in.	L_{ev1}, in.	L_{ev2}, in.
43	Rivets	A141	1.000	2	3	4.000	2.000	4.310	2.000	2.000
44	Rivets	A141	1.000	2	3	4.000	2.500	5.250	2.000	2.000
45	Rivets	A141	1.000	2	3	4.000	2.500	5.250	2.000	2.000
46	Rivets	A141	1.000	2	3	4.000	2.500	5.250	2.000	2.000
47	Rivets	A141	1.000	2	3	4.000	3.000	6.660	2.000	2.000
48	Rivets	A141	1.000	2	3	4.000	3.000	6.660	2.000	2.000
49	Rivets	A141	1.000	2	3	4.000	3.000	6.660	2.000	2.000
50	Rivets	A141	0.750	2	3	3.500	2.000	3.160	1.580	1.580
51	Rivets	A141	0.750	2	3	3.500	2.000	3.160	1.580	1.580
52	Rivets	A141	0.750	2	3	3.500	2.000	3.160	1.580	1.580
53	Rivets	A141	0.750	2	3	3.500	2.000	3.160	1.580	1.580
54	Rivets	A141	0.750	2	3	3.500	2.000	3.160	1.580	1.580
55	Rivets	A141	0.750	2	3	3.500	2.000	3.160	1.580	1.580
56	Rivets	A141	0.750	2	3	3.500	2.000	3.450	1.720	1.720
57	Rivets	A141	0.750	2	3	3.500	2.000	3.450	1.720	1.720
58	Rivets	A141	0.750	2	3	3.500	2.000	3.450	1.720	1.720
59	Rivets	A141	0.750	2	3	3.500	2.000	3.450	1.720	1.720
60	Rivets	A141	0.750	2	3	3.500	2.000	3.450	1.720	1.720
61	Rivets	A141	0.750	2	3	3.500	2.000	3.450	1.720	1.720
62	Rivets	A141	0.750	2	3	3.500	2.000	3.830	1.920	1.920
63	Rivets	A141	0.750	2	3	3.500	2.000	3.830	1.920	1.920
64	Rivets	A141	0.750	2	3	3.500	2.000	3.830	1.920	1.920
65	Rivets	A141	0.750	2	3	3.500	2.000	3.830	1.920	1.920
66	Rivets	A141	0.750	2	3	3.500	2.000	3.830	1.920	1.920
67	Rivets	A141	0.750	2	3	3.500	2.000	3.830	1.920	1.920
68	Rivets	A141	1.000	2	3	4.000	2.500	3.950	1.970	1.970
69	Rivets	A141	1.000	2	3	4.000	2.500	3.950	1.970	1.970
70	Rivets	A141	1.000	2	3	4.000	2.500	3.950	1.970	1.970
71	Rivets	A141	1.000	2	3	4.000	2.500	4.480	2.240	2.240
72	Rivets	A141	1.000	2	3	4.000	2.500	4.480	2.240	2.240
73	Rivets	A141	1.000	2	3	4.000	2.500	4.480	2.240	2.240
74	Rivets	A141	1.000	2	3	4.000	2.500	4.480	2.240	2.240
75	Rivets	A141	1.000	2	3	4.000	2.500	4.480	2.240	2.240
76	Rivets	A141	1.000	2	3	4.000	2.500	4.480	2.240	2.240
77	Rivets	A141	1.000	2	3	4.000	2.500	5.240	2.620	2.620
78	Rivets	A141	1.000	2	3	4.000	2.500	5.240	2.620	2.620
79	Rivets	A141	1.000	2	3	4.000	2.500	5.240	2.620	2.620
80	Rivets	A141	1.000	2	3	4.000	2.500	5.240	2.620	2.620
81	Rivets	A141	1.000	2	3	4.000	2.500	5.240	2.620	2.620
82	Rivets	A141	1.000	2	3	4.000	2.500	5.240	2.620	2.620
83	Rivets	A141	0.875	1	3	0.000	2.500	2.740	1.370	1.370
84	Rivets	NP	0.375	3	4	1.500	0.750	1.188	0.594	0.594

Table 9.3 continued

Index	Bolts/ Rivets	Bolt Material	*d*, in.	# of fasteners per row	# of rows of fasteners	*s*, in.	L_{eh}, in.	*g*, in.	L_{ev1}, in.	L_{ev2}, in.
85	Rivets	NP	0.375	3	4	1.500	0.750	1.188	0.594	0.594
86	Rivets	NP	0.375	3	4	1.500	0.750	0.875	0.438	0.438
87	Rivets	NP	0.375	3	4	1.500	0.750	0.875	0.438	0.438
88	Rivets	NP	0.375	3	4	1.500	0.750	1.875	0.938	0.938
89	Rivets	NP	0.375	3	4	1.500	0.750	1.875	0.938	0.938
90	Rivets	NP	0.375	3	4	1.500	0.750	1.500	0.750	0.750
91	Rivets	NP	0.375	3	4	1.500	0.750	1.500	0.750	0.750
92	Rivets	NP	0.375	3	1	1.500	0.750	N/A	0.750	0.750
93	Rivets	NP	0.375	3	1	1.500	0.750	N/A	0.750	0.750
94	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
95	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
96	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
97	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
98	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
99	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
100	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
101	Rivets	NP	0.375	3	3	1.500	0.750	1.500	0.750	0.750
102	Rivets	NP	0.500	3	3	2.000	0.750	2.000	1.000	1.000
103	Rivets	NP	0.500	3	3	2.000	0.750	2.000	1.000	1.000
104	Rivets	NP	0.250	3	3	1.000	0.750	1.000	0.500	0.500
105	Rivets	NP	0.250	3	3	1.000	0.750	1.000	0.500	0.500
106	Rivets	NP	0.375	3	7	1.500	0.750	1.500	0.750	0.750
107	Rivets	NP	0.375	3	7	1.500	0.750	1.500	0.750	0.750
108	Rivets	NP	0.375	3	5	1.500	0.750	1.500	0.750	0.750
109	Rivets	NP	0.375	3	5	1.500	0.750	1.500	0.750	0.750

NP is not provided

The modelling process of the bolted plate specimens was similar to the bolted angles. The gusset plates and tested plates were all modeled as members. The rivets were always modeled as bolts. It should be noted, most plate specimens were tested in the physical experiments as one half of a double-strap, butt-type joint. Of these experiments, some failing specimens were the center plate, and some were the outer plates. The IDEA StatiCa models represented the same test set up. The bearing member was not always selected as the gusset plate in this case due to the unique testing setup in the experiments.

In the cases where the experimentalists used the double-strap, butt-type joint, the center plate was set as the bearing member whether it was the tested or non-tested plate. This is because only one member can be chosen as the bearing member. For all other cases, the bearing member was always taken as the gusset plate. An example model in IDEA StatiCa is provided in Figure 9-2. The procedure described here is for when the failing plate was on the outside. The procedure for when the failing plate was on the outside was similar. The typical process for modelling the specimens is shown in Figure 9-3.

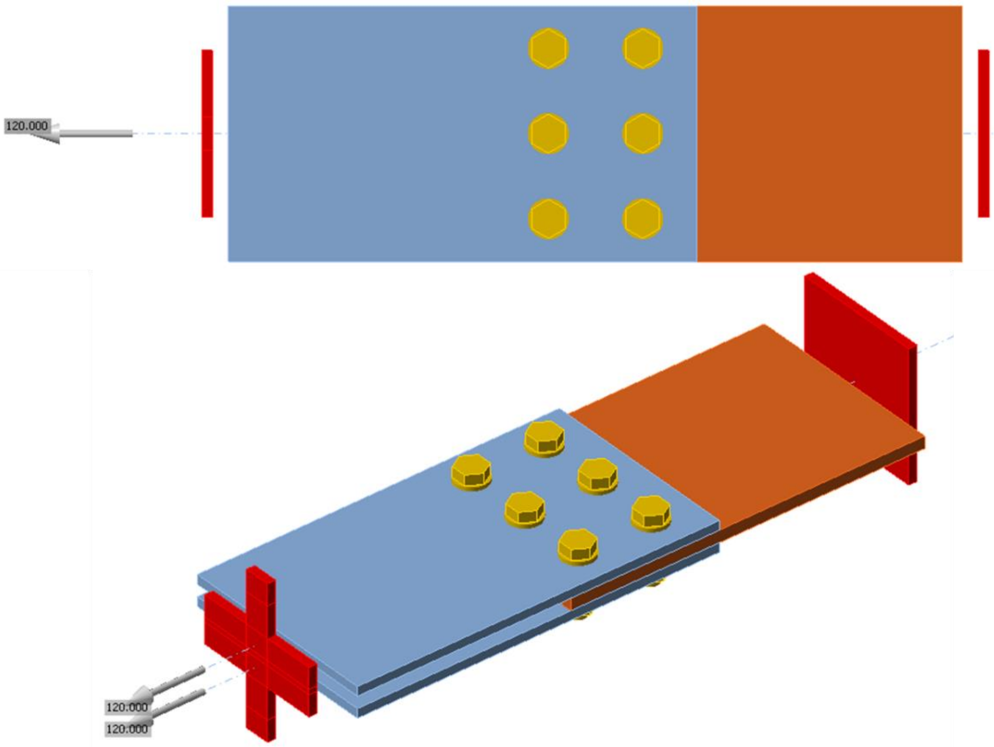


Figure 9-2: Typical Bolted Plate Connection Model in IDEA StatiCa

4. Create members:
    a. Member 1: Gusset plate
        i. Geometrical Type set to “Ended”
        ii. Offset ex set to the negative of the connection length
            1. Connection length is 2 times L_{eh} + (the number of bolts per row minus one) times the bolt spacing
    b. Member 2, 3 (if applicable): Tested plate(s)
        i. β set to 180º
        ii. Aligned set to “To member plate”
        iii. Aligned plate set to “[tested plate name] | bottom flange 1”
        iv. Related plate set to “[gusset plate name] | bottom flange 1”
        v. Model type set to “N-Vy-Vz”
5. Create Bolt Grid operation:
    a. Fastener set to “Bolts”
    b. Items count set to the number of plates (2 or 3)
    c. Item 1, 2, 3, set to each member created in step 1
    d. Type set to diameter and type indicated by report
    e. Coord. System set to “Orthogonal”
    f. Rows, Positions set to dimension indicated by reports
    g. Grid set to “Regular”
    h. Shear force transfer set to “Bearing – tension/shear interaction”

Figure 9-3: Modelling Process for Bolted Plate Connections

9.2.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 9.4: AISC Calculation Results for Bolted Plate Connections

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
1	116.1	1.00	Case 1	58.5
2	136.5	1.00	Case 1	79.8
3	137.0	1.00	Case 1	80.4
4	154.2	1.00	Case 1	98.3
5	153.2	1.00	Case 1	97.3
6	171.9	1.00	Case 1	116.8
7	171.3	1.00	Case 1	116.3
8	226.1	1.00	Case 1	173.6
9	149.8	1.00	Case 1	105.7
10	187.8	1.00	Case 1	93.0
11	223.9	1.00	Case 1	130.7
12	224.1	1.00	Case 1	131.0
13	239.8	1.00	Case 1	147.3
14	264.7	1.00	Case 1	173.4
15	279.0	1.00	Case 1	188.3
16	233.8	1.00	Case 1	141.1
17	207.9	1.00	Case 1	252.1
18	207.9	1.00	Case 1	252.1
19	207.9	1.00	Case 1	252.1
20	191.1	1.00	Case 1	235.6
21	191.1	1.00	Case 1	235.6
22	191.1	1.00	Case 1	235.6
23	172.1	1.00	Case 1	243.3
24	172.1	1.00	Case 1	243.3
25	172.1	1.00	Case 1	243.3
26	270.5	1.00	Case 1	335.1
27	270.5	1.00	Case 1	335.1
28	270.5	1.00	Case 1	335.1
29	271.2	1.00	Case 1	343.4
30	271.2	1.00	Case 1	343.4
31	271.2	1.00	Case 1	343.4
32	267.3	1.00	Case 1	337.3
33	267.3	1.00	Case 1	337.3
34	267.3	1.00	Case 1	337.3
35	228.2	1.00	Case 1	325.2
36	228.2	1.00	Case 1	325.2
37	228.2	1.00	Case 1	325.2
38	353.7	1.00	Case 1	458.4
39	353.7	1.00	Case 1	458.4
40	353.7	1.00	Case 1	458.4
41	342.6	1.00	Case 1	443.6
42	342.6	1.00	Case 1	443.6

Table 9.4 continued

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
43	342.6	1.00	Case 1	443.6
44	340.8	1.00	Case 1	444.7
45	340.8	1.00	Case 1	444.7
46	340.8	1.00	Case 1	444.7
47	321.3	1.00	Case 1	424.7
48	321.3	1.00	Case 1	424.7
49	321.3	1.00	Case 1	424.7
50	321.0	1.00	Case 1	397.5
51	321.0	1.00	Case 1	397.5
52	321.0	1.00	Case 1	397.5
53	265.4	1.00	Case 1	389.5
54	270.4	1.00	Case 1	392.5
55	262.1	1.00	Case 1	387.6
56	311.8	1.00	Case 1	401.2
57	311.8	1.00	Case 1	401.2
58	311.8	1.00	Case 1	401.2
59	290.0	1.00	Case 1	380.5
60	314.1	1.00	Case 1	383.5
61	290.8	1.00	Case 1	372.8
62	285.3	1.00	Case 1	371.0
63	285.3	1.00	Case 1	371.0
64	285.3	1.00	Case 1	371.0
65	283.2	1.00	Case 1	380.6
66	276.0	1.00	Case 1	375.0
67	275.3	1.00	Case 1	378.9
68	423.6	1.00	Case 1	650.2
69	418.2	1.00	Case 1	649.3
70	426.2	1.00	Case 1	643.4
71	475.8	1.00	Case 1	684.9
72	475.8	1.00	Case 1	684.9
73	475.8	1.00	Case 1	684.9
74	513.4	1.00	Case 1	636.7
75	436.8	1.00	Case 1	650.0
76	422.0	1.00	Case 1	633.6
77	517.2	1.00	Case 1	702.9
78	517.2	1.00	Case 1	702.9
79	517.2	1.00	Case 1	702.9
80	462.2	1.00	Case 1	663.4
81	495.2	1.00	Case 1	693.0
82	441.5	1.00	Case 1	675.5
83	193.6	1.00	Case 1	174.9
84	58.1	1.00	Case 1	61.2

Table 9.4 continued

Index	P_{YIELD}, kips	U	Controlling U Case	$P_{RUPTURE}$, kips
85	58.1	1.00	Case 1	61.2
86	42.8	1.00	Case 1	35.7
87	42.8	1.00	Case 1	35.7
88	91.8	1.00	Case 1	117.2
89	91.8	1.00	Case 1	117.2
90	73.4	1.00	Case 1	86.6
91	73.4	1.00	Case 1	86.6
92	18.4	1.00	Case 1	21.7
93	18.4	1.00	Case 1	21.7
94	55.1	1.00	Case 1	65.0
95	55.1	1.00	Case 1	65.0
96	55.1	1.00	Case 1	65.0
97	55.1	1.00	Case 1	65.0
98	51.3	1.00	Case 1	50.8
99	51.3	1.00	Case 1	50.8
100	63.0	1.00	Case 1	76.0
101	63.0	1.00	Case 1	76.0
102	105.3	1.00	Case 1	141.2
103	105.3	1.00	Case 1	141.2
104	34.2	1.00	Case 1	32.9
105	34.2	1.00	Case 1	32.9
106	128.5	1.00	Case 1	151.6
107	128.5	1.00	Case 1	151.6
108	91.8	1.00	Case 1	108.3
109	91.8	1.00	Case 1	108.3

Table 9.5: Summary Strength Results for Bolted Plate Connections

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	**Failure Mode**	**P_{AISC}, kips**	**Controlling limit state**	**P_{IDEA}, kips**	**Failure Mode**
1	58.9	[2]	58.5	[2]	71.5	[3]
2	80.9	[2]	79.8	[2]	88.3	[3]
3	79.8	[2]	80.4	[2]	88.9	[3]
4	100.0	[2]	98.3	[2]	99.4	[3]
5	98.9	[2]	97.3	[2]	99.3	[3]
6	116.0	[2]	116.8	[2]	105.1	[3]
7	114.7	[2]	116.3	[2]	104.9	[3]
8	177.1	[2]	173.6	[2]	107.6	[3]
9	108.6	[2]	105.7	[2]	85.2	[3]
10	102.7	[2]	93.0	[2]	116.9	[3]
11	144.6	[2]	130.7	[2]	146.8	[3]
12	143.4	[2]	131.0	[2]	148.4	[3]
13	154.9	[2]	147.3	[2]	159.3	[3]
14	177.4	[2]	173.4	[2]	169.7	[3]
15	191.3	[2]	188.3	[2]	172.2	[3]
16	152.4	[2]	141.1	[2]	154.2	[3]
17	271.1	[2]	207.9	[1]	181.0	[3]
18	269.2	[2]	207.9	[1]	181.0	[3]
19	270.3	[2]	207.9	[1]	181.0	[3]
20	246.6	[2]	191.1	[1]	171.8	[3]
21	244.0	[2]	191.1	[1]	171.8	[3]
22	245.5	[2]	191.1	[1]	171.8	[3]
23	225.6	[2]	172.1	[1]	160.5	[3]
24	240.9	[2]	172.1	[1]	160.5	[3]
25	232.8	[2]	172.1	[1]	160.5	[3]
26	358.3	[2]	270.5	[1]	235.3	[3]
27	355.8	[2]	270.5	[1]	235.3	[3]
28	360.3	[2]	270.5	[1]	235.3	[3]
29	358.3	[2]	271.2	[1]	244.1	[3]
30	358.6	[2]	271.2	[1]	244.1	[3]
31	361.5	[2]	271.2	[1]	244.1	[3]
32	347.0	[2]	267.3	[1]	250.2	[3]
33	351.9	[2]	267.3	[1]	250.2	[3]
34	337.6	[2]	267.3	[1]	250.2	[3]
35	288.8	[2]	228.2	[1]	196.0	[3]
36	295.3	[2]	228.2	[1]	196.0	[3]
37	285.2	[2]	228.2	[1]	196.0	[3]
38	442.7	[2]	353.7	[1]	305.2	[3]
39	444.5	[2]	353.7	[1]	305.2	[3]
40	442.2	[2]	353.7	[1]	305.2	[3]
41	436.7	[2]	342.6	[1]	303.1	[3]
42	448.9	[2]	342.6	[1]	303.1	[3]

Table 9.5 continued

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling limit state	P_{IDEA}, kips	Failure Mode
43	454.8	[2]	342.6	[1]	303.1	[3]
44	465.2	[2]	340.8	[1]	314.0	[3]
45	471.5	[2]	340.8	[1]	314.0	[3]
46	460.8	[2]	340.8	[1]	314.0	[3]
47	393.3	[2]	321.3	[1]	301.0	[3]
48	392.2	[2]	321.3	[1]	301.0	[3]
49	399.5	[2]	321.3	[1]	301.0	[3]
50	267.9	[2]	321.0	[1]	275.5	[3]
51	245.0	[2]	321.0	[1]	275.5	[3]
52	244.6	[2]	321.0	[1]	275.5	[3]
53	228.8	[2]	265.4	[1]	238.7	[3]
54	248.9	[2]	270.4	[1]	242.9	[3]
55	238.9	[2]	262.1	[1]	235.9	[3]
56	279.2	[2]	311.8	[1]	274.4	[3]
57	280.0	[2]	311.8	[1]	274.4	[3]
58	247.2	[2]	311.8	[1]	274.4	[3]
59	261.9	[2]	290.0	[1]	265.0	[3]
60	231.3	[2]	314.1	[1]	275.6	[3]
61	241.8	[2]	290.8	[1]	265.4	[3]
62	266.6	[2]	285.3	[1]	261.2	[3]
63	267.9	[2]	285.3	[1]	261.2	[3]
64	244.2	[2]	285.3	[1]	261.2	[3]
65	229.0	[2]	283.2	[1]	259.8	[3]
66	242.6	[2]	276.0	[1]	253.5	[3]
67	251.0	[2]	275.3	[1]	253.1	[3]
68	457.2	[2]	423.6	[1]	370.8	[3]
69	425.3	[2]	418.2	[1]	366.4	[3]
70	409.0	[2]	426.2	[1]	372.8	[3]
71	469.6	[2]	475.8	[1]	426.8	[3]
72	461.1	[2]	475.8	[1]	426.8	[3]
73	409.5	[2]	475.8	[1]	426.8	[3]
74	400.0	[2]	513.4	[1]	456.6	[3]
75	369.8	[2]	436.8	[1]	393.6	[3]
76	376.0	[2]	422.0	[1]	380.9	[3]
77	386.7	[2]	517.2	[1]	479.2	[3]
78	385.6	[2]	517.2	[1]	479.2	[3]
79	389.0	[2]	517.2	[1]	479.2	[3]
80	369.3	[2]	462.2	[1]	431.7	[3]
81	375.4	[2]	495.2	[1]	460.9	[3]
82	367.8	[2]	441.5	[1]	414.1	[3]
83	168.0	[2]	174.9	[2]	141.0	[3]
84	78.1	Plate	58.1	[1]	48.0	[3]

Table 9.5 continued

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling limit state	P_{IDEA}, kips	Failure Mode
85	79.1	Plate	58.1	[1]	48.0	[3]
86	48.0	Plate	35.7	[2]	30.8	[3]
87	48.5	Plate	35.7	[2]	30.8	[3]
88	135.7	Plate	91.8	[1]	89.3	[3]
89	133.0	Plate	91.8	[1]	89.3	[3]
90	104.6	Plate	73.4	[1]	65.6	[3]
91	103.2	Plate	73.4	[1]	65.6	[3]
92	25.7	Plate	18.4	[1]	15.5	[3]
93	25.7	Plate	18.4	[1]	15.5	[3]
94	69.1	Plate	55.1	[1]	48.7	[3]
95	69.0	Plate	55.1	[1]	48.7	[3]
96	79.0	Plate	55.1	[1]	48.7	[3]
97	80.2	Plate	55.1	[1]	48.7	[3]
98	45.3	Plate	50.8	[2]	45.0	[3]
99	46.3	Plate	50.8	[2]	45.0	[3]
100	88.8	Plate	63.0	[1]	55.8	[3]
101	89.2	Plate	63.0	[1]	55.8	[3]
102	165.6	Plate	105.3	[1]	93.9	[3]
103	160.7	Plate	105.3	[1]	93.9	[3]
104	38.2	Plate	32.9	[2]	29.5	[3]
105	38.6	Plate	32.9	[2]	29.5	[3]
106	175.9	Plate	128.5	[1]	117.4	[3]
107	178.2	Plate	128.5	[1]	117.4	[3]
108	131.9	Plate	91.8	[1]	83.0	[3]
109	131.7	Plate	91.8	[1]	83.0	[3]

[1] tensile yield; [2] tensile rupture; [3] member strain

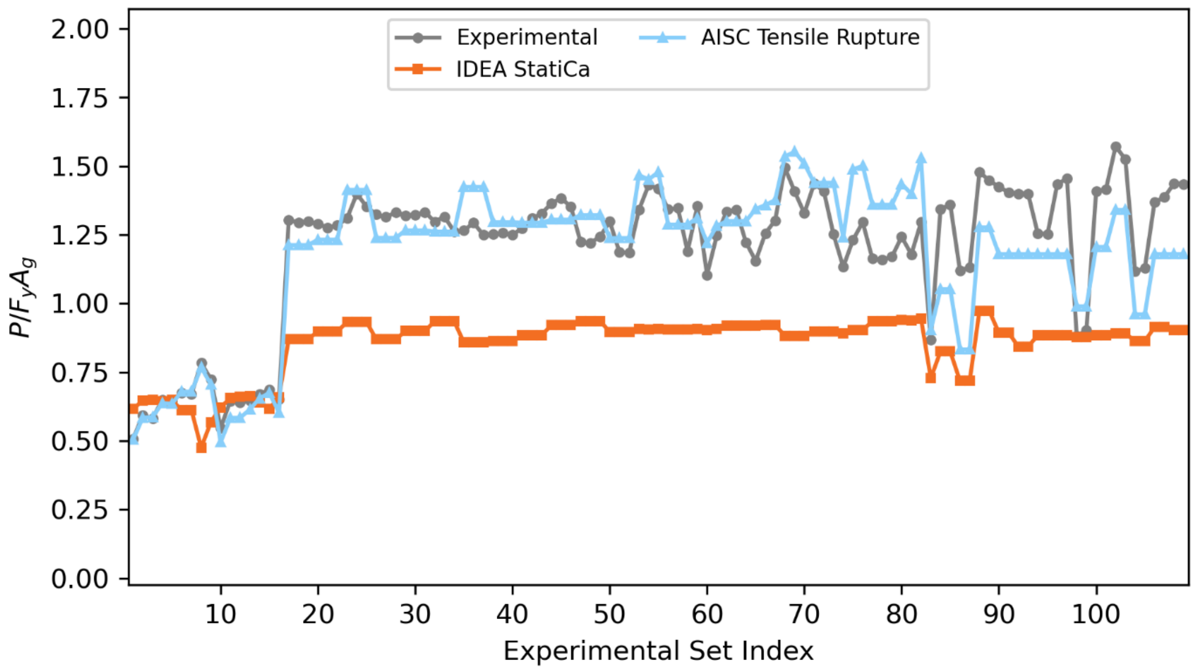


Figure 9-4: Normalized Strength Results for Bolted Plate Connections

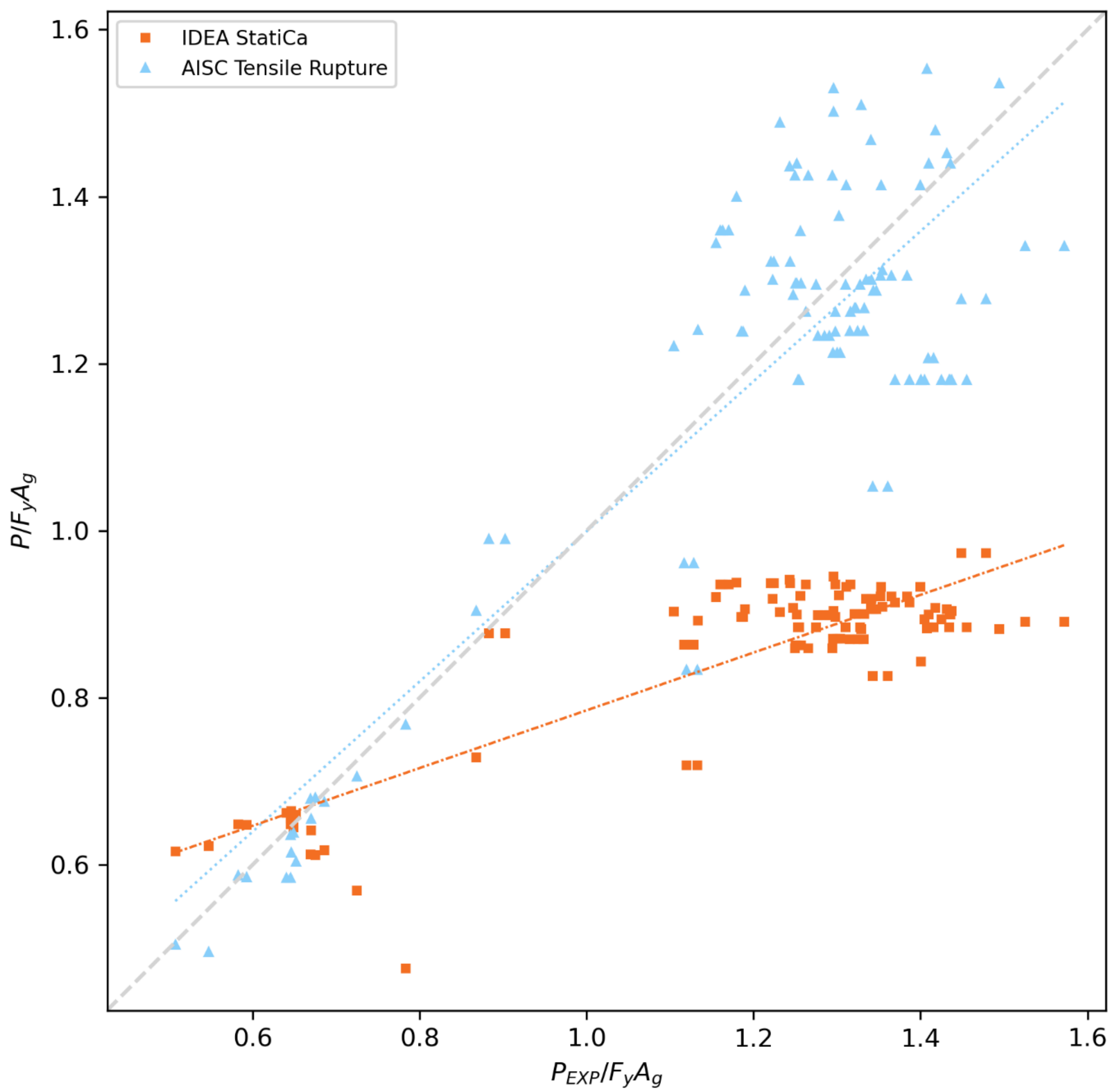


Figure 9-5: Normalized Strength vs. Normalized Experimental Strength for Bolted Plate Connections

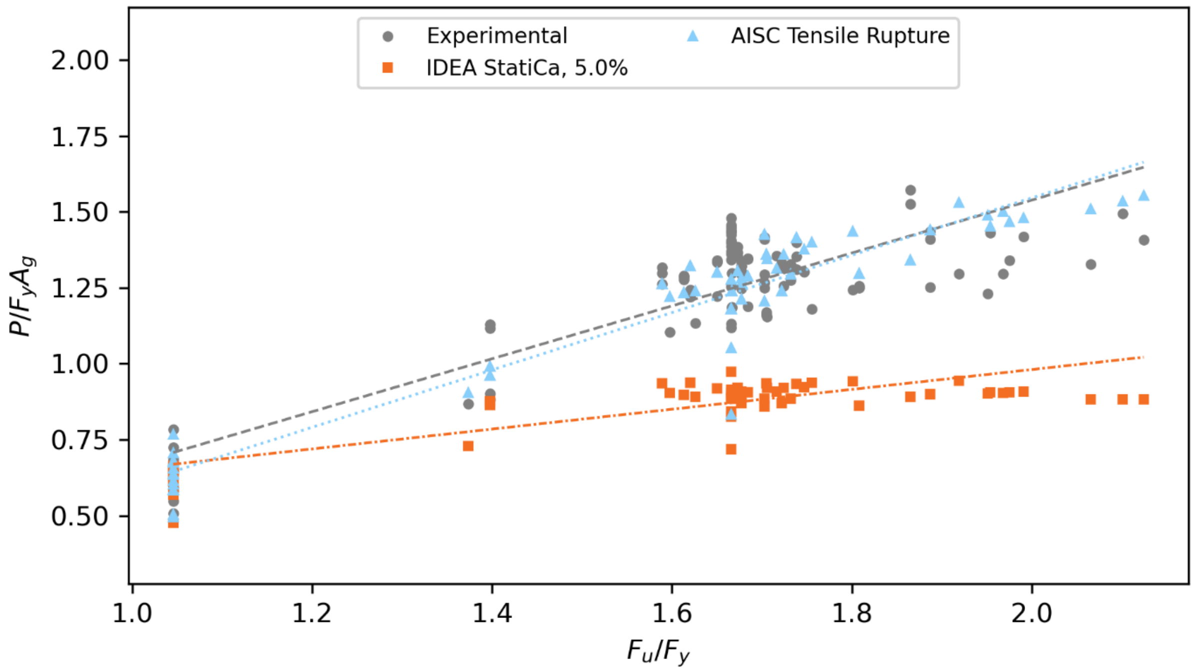


Figure 9-6: Normalized Strength vs. Material Strength Ratio for Bolted Plate Connections

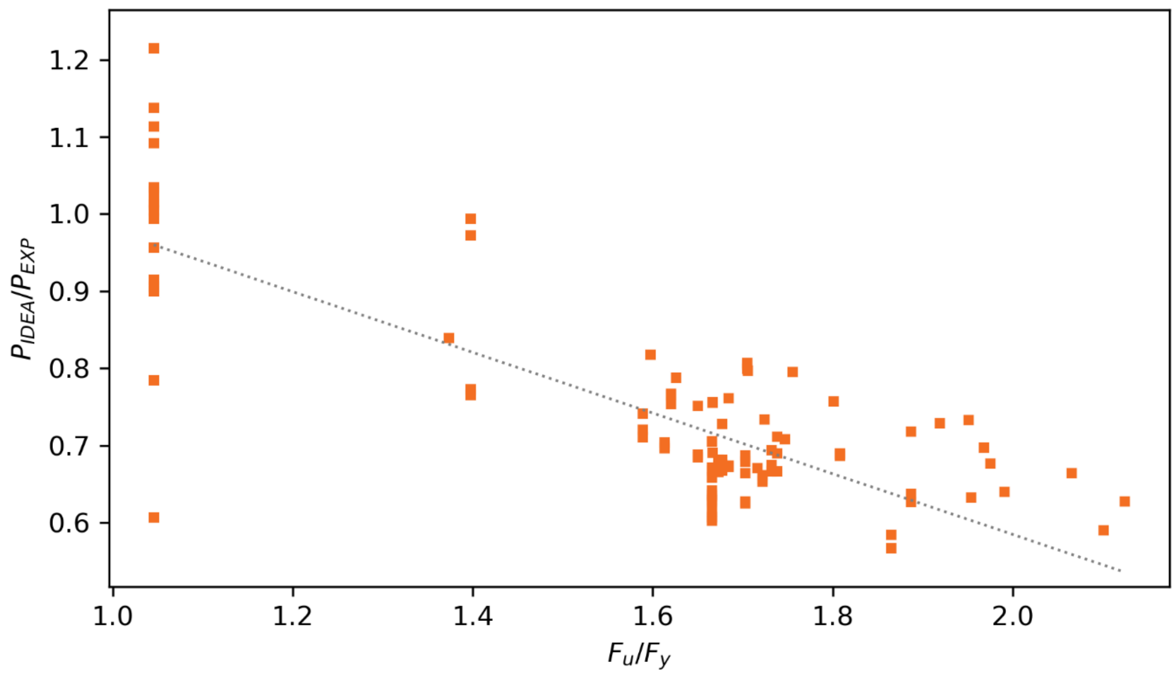


Figure 9-7: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Bolted Plate Connections

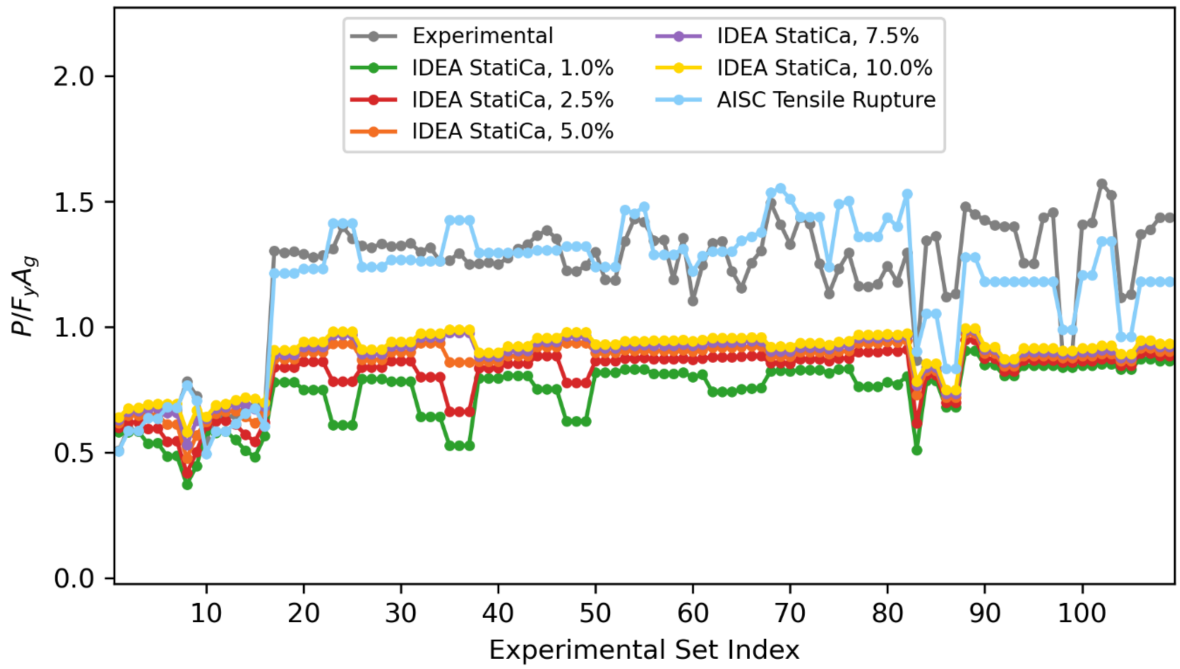


Figure 9-8: Normalized Strength Results for Bolted Plate Connections Including Various Plastic Strain Limits for IDEA StatiCa

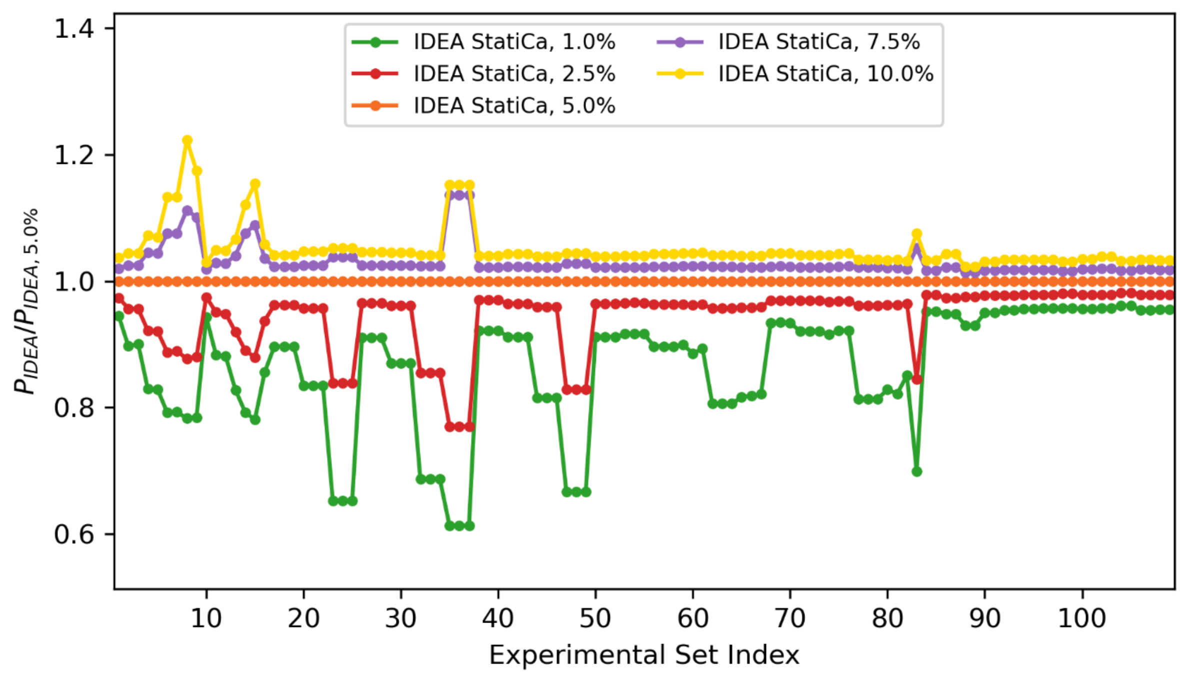


Figure 9-9: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Bolted Plate Connections

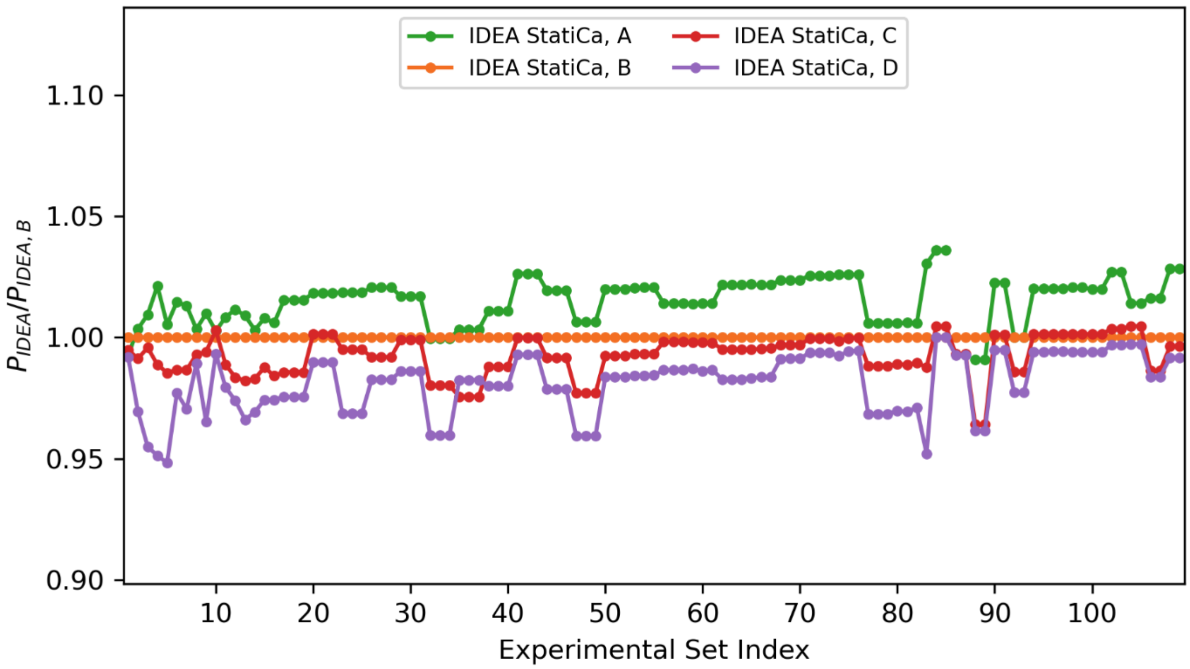


Figure 9-10: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Bolted Plate Connections

Table 9.6: Summary Statistics of the Test-to-Predicted Ratio for Bolted Plate Connections

Test-to-Predicted Ratio	Average	Standard Deviation	Coefficient of Variation
$P_{EXP}/P_{RUPTURE}$ (AISC)	1.027	0.115	0.112
P_{EXP}/P_{IDEA}	1.392	0.205	0.147

9.2.3 Discussion

Tensile yielding was not reported as a failure mode for any of the specimens (the specimens tested by Schutz & Newmark (1952) were reported as "Plate Failure"). However, tensile yield controlled the AISC strength calculations for most of the specimens (Table 9.5). There were some instances where IDEA StatiCa provided strengths larger than the strength calculated according to the AISC *Specification* equations and strengths observed in the experimental tests. The most severe case resulted in roughly a 22% greater strength in IDEA StatiCa to both AISC and the experimentally observed strength. Outside of these specimens, P_{IDEA} consistently provided strengths around 10% less than P_{AISC}.

Apart from specimens 1 through 16, the strength according to IDEA StatiCa provided a relatively consistent level of conservatism compared to both the experimentally observed strengths and the strengths according to the AISC *Specification* equations. Specimens 1 through 16 are all from Može and Beg (2010); 9 of these specimens provided a larger strength in IDEA StatiCa than the experimentally observed strength. Note that only 12 specimens in the entire study had an IDEA StatiCa strength greater than that from the experiment. It is worth noting for the 16 specimens tested by Može and Beg (2010), tensile rupture controlled the AISC *Specification* strength calculations and the experimental results by a significant margin. These specimens were fabricated from high strength steel (F_y = 122.8 ksi) with a relatively low F_u/F_y ratio (1.05). Whereas almost all other specimens had tensile yield occurring before rupture. This is shown in Figure 9-4 where specimens with indices 1 through 16 have normalized strengths typically well below 1.0. These are cases where IDEA StatiCa is identifying tensile rupture, and not tensile yield.

With respect to the material strength ratio F_u/F_y, clear increases with increasing F_u/F_y are seen for the experimental, AISC, and IDEA StatiCa strengths (Figure 9-6). This trend is not expected for the IDEA StatiCa strengths, but may have been inducted by the Može and Beg (2010) experiments which has a relatively low F_u/F_y ratio. Nonetheless, the ratio P_{IDEA}/P_{EXP} is seen to decrease with increasing F_u/F_y as expected (Figure 9-7).

The variation of P_{IDEA} with plastic strain limit exhibited the expected trend of increasing strength with increasing plastic strain limit and vice versa (Figure 9-8 and Figure 9-9). For most specimens the increase in strength with increasing plastic strain limit was small.

The variation of P_{IDEA} with mesh parameters showed only minor changes (Figure 7-10). In IDEA StatiCa, the mesh around the bolt holes in IDEA StatiCa always remains constant at 8 elements, regardless of the mesh parameters set by the user. Since the greatest plastic strains were observed near the bolt holes, adjusting the mesh elsewhere has a minimal effect on the resulting strength.

9.3 Comparison to Experimental Results: Staggered Bolt Configuration

9.3.1 Description of Experimental Specimens

Bolted plate connection specimens with staggered bolt patterns are examined in this section. A wide variety of staggered bolt plate connections with various bolt configurations was identified in the literature therefore a detailed description of each specimen is not provided. The schematic shown in Figure 9-11 is only provided for the purpose of defining dimensions, such as ‘*s*’ and ‘*g*’.

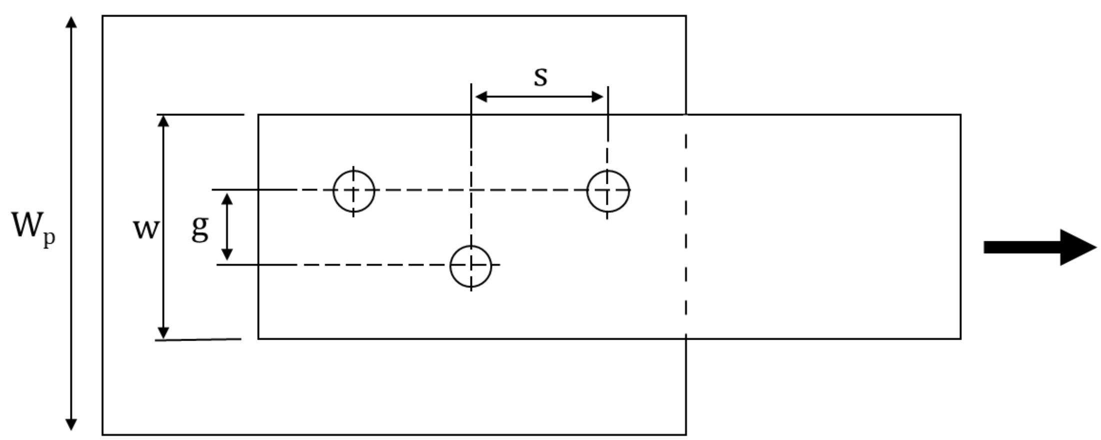


Figure 9-11: Schematic of Bolted Plate (with Staggered Bolts) Connection

A total of 116 bolted plate specimens (with staggered bolt patterns) from 2 references were identified for evaluation in this work as detailed in Table 9.7. As described in the previous section, connections tested by Davis et al. (1940), were not included.

Table 9.7: Count of Bolted Plate (with Staggered Bolts) Connections by Reference

Reference	Specimen Count
Munse (1959)	23
Schutz and Newmark (1952)	93
Total	**116**

Modeling these specimens in IDEA StatiCa is identical to the bolted plate specimens previously outlined. The main difference included the calculations for the strength according to IDEA StatiCa, specifically the calculation of the net area, A_n. In addition to subtracting out the area removed for the bolt holes, a term is added to the width of the angle for every diagonal in accordance with Section B4.3b of the AISC *Specification* (2016). The quantity added is $s^2/4g$ where ‘s’ is the transverse center-to-center spacing between fastener gage lines and ‘g’ is the longitudinal center-to-center spacing of any two consecutive holes. These dimensions are consistent with those in Figure 9-11. Consistent with common practice, the net area was calculated for every applicable failure path and the minimum value was taken as the controlling net area. This was particularly significant for the staggered bolted plates presented in this section as there are a wide variety of bolt patterns included in these specimens. The specimens were provided from two sources and enumerated in Table 9.7. Detailed description of each specimen is provided in Table 9.8 with the details specific to the fasteners provided in Table 9.9.

Table 9.8: Bolted Plate (with Staggered Bolts) Connection Experimental Specimen Parameters

Index	Reference	Specimen	# of plates	*Fy, ksi	*Fu, ksi	*t, in	*w, in
1	Munse (1959)	49-1A	1	36.2	67.4	0.760	6.460
2	Munse (1959)	49-1B	1	36.2	67.4	0.760	6.460
3	Munse (1959)	49-2A	1	36.9	66.4	0.650	7.400
4	Munse (1959)	49-2B	1	36.9	66.4	0.650	7.400
5	Munse (1959)	49-3A	1	34.5	63.0	0.500	8.820
6	Munse (1959)	49-3B	1	34.5	63.0	0.500	8.820
7	Munse (1959)	49-4A	1	39.5	66.6	0.380	11.170
8	Munse (1959)	49-4B	1	39.5	66.6	0.380	11.170
9	Munse (1959)	49-5A	1	37.8	62.9	0.320	13.050
10	Munse (1959)	49-7A	1	30.0	60.7	0.820	9.970
11	Munse (1959)	49-7B	1	30.0	60.7	0.820	9.970
12	Munse (1959)	49-8A	1	34.6	65.5	0.640	12.300
13	Munse (1959)	49-8B	1	34.6	65.5	0.640	12.300
14	Munse (1959)	49-9A	1	35.0	62.9	0.500	14.820
15	Munse (1959)	49-9B	1	35.0	62.9	0.500	14.820
16	Munse (1959)	49-10B	1	38.9	65.0	0.460	16.630
17	Munse (1959)	49-1XA	1	30.0	60.2	0.730	6.470
18	Munse (1959)	49-1XB	1	30.0	60.2	0.730	6.470
19	Munse (1959)	49-1XXB	1	30.0	60.2	0.730	6.470
20	Munse (1959)	49-4XA	1	36.0	62.0	0.380	11.180
21	Munse (1959)	49-4XB	1	36.0	62.0	0.380	11.180
22	Munse (1959)	49-6XB	1	27.0	57.5	1.000	8.510
23	Munse (1959)	49-9XB	1	34.9	63.7	0.500	14.820
24	Schutz and Newmark (1952)	AA1A	1	39.2	65.2	0.313	6.000
25	Schutz and Newmark (1952)	AA1B	1	39.2	65.2	0.313	6.000
26	Schutz and Newmark (1952)	AA3A	1	39.2	65.2	0.313	6.000
27	Schutz and Newmark (1952)	AA3B	1	39.2	65.2	0.313	6.000
28	Schutz and Newmark (1952)	AA4A	1	39.2	65.2	0.313	6.000
29	Schutz and Newmark (1952)	AA4B	1	39.2	65.2	0.313	6.000
30	Schutz and Newmark (1952)	AA5A	1	39.2	65.2	0.313	6.000
31	Schutz and Newmark (1952)	AA5B	1	39.2	65.2	0.313	6.000
32	Schutz and Newmark (1952)	AA6A	1	39.2	65.2	0.313	6.000
33	Schutz and Newmark (1952)	AA6B	1	39.2	65.2	0.313	6.000
34	Schutz and Newmark (1952)	AC2A	1	39.2	65.2	0.313	10.500
35	Schutz and Newmark (1952)	AC2B	1	39.2	65.2	0.313	10.500
36	Schutz and Newmark (1952)	AC3A	1	39.2	65.2	0.313	10.500
37	Schutz and Newmark (1952)	AC3B	1	39.2	65.2	0.313	10.500
38	Schutz and Newmark (1952)	AC4A	1	39.2	65.2	0.313	10.500
39	Schutz and Newmark (1952)	AC4B	1	39.2	65.2	0.313	10.500
40	Schutz and Newmark (1952)	C3A	1	39.2	65.2	0.313	4.500
41	Schutz and Newmark (1952)	C3B	1	39.2	65.2	0.313	4.500
42	Schutz and Newmark (1952)	C8A	1	39.2	65.2	0.313	4.500

Table 9.8 continued

Index	Reference	Specimen	# of plates	*Fy, ksi	*Fu, ksi	*t, in	*w, in
43	Schutz and Newmark (1952)	C8B	1	39.2	65.2	0.313	4.500
44	Schutz and Newmark (1952)	FM1A	1	39.2	65.2	0.313	6.000
45	Schutz and Newmark (1952)	FM1B	1	39.2	65.2	0.313	6.000
46	Schutz and Newmark (1952)	FM1C	1	39.2	65.2	0.313	6.000
47	Schutz and Newmark (1952)	FM2A	1	39.2	65.2	0.313	6.000
48	Schutz and Newmark (1952)	FM2B	1	39.2	65.2	0.313	6.000
49	Schutz and Newmark (1952)	FM2C	1	39.2	65.2	0.313	6.000
50	Schutz and Newmark (1952)	JA2A	1	39.2	65.2	0.313	6.000
51	Schutz and Newmark (1952)	JA2B	1	39.2	65.2	0.313	6.000
52	Schutz and Newmark (1952)	JA3A	1	39.2	65.2	0.313	6.000
53	Schutz and Newmark (1952)	JA3B	1	39.2	65.2	0.313	6.000
54	Schutz and Newmark (1952)	JA4A	1	39.2	65.2	0.313	6.000
55	Schutz and Newmark (1952)	JA4B	1	39.2	65.2	0.313	6.000
56	Schutz and Newmark (1952)	JA5A	1	39.2	65.2	0.313	6.000
57	Schutz and Newmark (1952)	JA5B	1	39.2	65.2	0.313	6.000
58	Schutz and Newmark (1952)	M1A	1	39.2	65.2	0.313	4.500
59	Schutz and Newmark (1952)	M1B	1	39.2	65.2	0.313	4.500
60	Schutz and Newmark (1952)	N1A	1	39.2	65.2	0.313	4.500
61	Schutz and Newmark (1952)	N1B	1	39.2	65.2	0.313	4.500
62	Schutz and Newmark (1952)	N2A	1	39.2	65.2	0.313	4.500
63	Schutz and Newmark (1952)	N2B	1	39.2	65.2	0.313	4.500
64	Schutz and Newmark (1952)	N3A	1	39.2	65.2	0.313	4.500
65	Schutz and Newmark (1952)	N3B	1	39.2	65.2	0.313	4.500
66	Schutz and Newmark (1952)	N4A	1	39.2	65.2	0.313	4.500
67	Schutz and Newmark (1952)	N4B	1	39.2	65.2	0.313	4.500
68	Schutz and Newmark (1952)	P1A	1	39.2	65.2	0.313	4.500
69	Schutz and Newmark (1952)	P1B	1	39.2	65.2	0.313	4.500
70	Schutz and Newmark (1952)	P1C	1	39.2	65.2	0.313	4.500
71	Schutz and Newmark (1952)	P1D	1	39.2	65.2	0.313	4.500
72	Schutz and Newmark (1952)	P1E	1	39.2	65.2	0.313	4.500
73	Schutz and Newmark (1952)	P1F	1	39.2	65.2	0.313	4.500
74	Schutz and Newmark (1952)	P2A	1	39.2	65.2	0.313	4.500
75	Schutz and Newmark (1952)	P2B	1	39.2	65.2	0.313	4.500
76	Schutz and Newmark (1952)	P3A	1	39.2	65.2	0.313	4.500
77	Schutz and Newmark (1952)	P3B	1	39.2	65.2	0.313	4.500
78	Schutz and Newmark (1952)	R2A	1	39.2	65.2	0.313	4.500
79	Schutz and Newmark (1952)	R2B	1	39.2	65.2	0.313	4.500
80	Schutz and Newmark (1952)	R3A	1	39.2	65.2	0.313	4.500
81	Schutz and Newmark (1952)	R3B	1	39.2	65.2	0.313	4.500
82	Schutz and Newmark (1952)	ST1A	1	39.2	65.2	0.313	4.000
83	Schutz and Newmark (1952)	ST1B	1	39.2	65.2	0.313	4.000
84	Schutz and Newmark (1952)	ST2A	1	39.2	65.2	0.313	4.300

Table 9.8 continued

Index	Reference	Specimen	# of plates	*Fy, ksi	*Fu, ksi	*t, in	*w, in
85	Schutz and Newmark (1952)	ST2B	1	39.2	65.2	0.313	4.300
86	Schutz and Newmark (1952)	ST3A	1	39.2	65.2	0.313	4.875
87	Schutz and Newmark (1952)	ST3B	1	39.2	65.2	0.313	4.875
88	Schutz and Newmark (1952)	ST4A	1	39.2	65.2	0.313	6.000
89	Schutz and Newmark (1952)	ST4B	1	39.2	65.2	0.313	6.000
90	Schutz and Newmark (1952)	ST5B	1	39.2	65.2	0.313	4.000
91	Schutz and Newmark (1952)	ST6A	1	39.2	65.2	0.313	4.300
92	Schutz and Newmark (1952)	ST6B	1	39.2	65.2	0.313	4.300
93	Schutz and Newmark (1952)	ST7A	1	39.2	65.2	0.313	4.875
94	Schutz and Newmark (1952)	ST7B	1	39.2	65.2	0.313	4.875
95	Schutz and Newmark (1952)	ST8A	1	39.2	65.2	0.313	6.000
96	Schutz and Newmark (1952)	ST8B	1	39.2	65.2	0.313	6.000
97	Schutz and Newmark (1952)	V2A	1	39.2	65.2	0.313	7.500
98	Schutz and Newmark (1952)	V2B	1	39.2	65.2	0.313	7.500
99	Schutz and Newmark (1952)	V3A	1	39.2	65.2	0.313	7.500
100	Schutz and Newmark (1952)	V3B	1	39.2	65.2	0.313	7.500
101	Schutz and Newmark (1952)	V4A	1	39.2	65.2	0.313	7.500
102	Schutz and Newmark (1952)	V4B	1	39.2	65.2	0.313	7.500
103	Schutz and Newmark (1952)	V5A	1	39.2	65.2	0.313	7.500
104	Schutz and Newmark (1952)	V5B	1	39.2	65.2	0.313	7.500
105	Schutz and Newmark (1952)	V6A	1	39.2	65.2	0.313	7.500
106	Schutz and Newmark (1952)	V6B	1	39.2	65.2	0.313	7.500
107	Schutz and Newmark (1952)	V7A	1	39.2	65.2	0.313	7.500
108	Schutz and Newmark (1952)	V7B	1	39.2	65.2	0.313	7.500
109	Schutz and Newmark (1952)	X2A	1	39.2	65.2	0.313	7.500
110	Schutz and Newmark (1952)	X2B	1	39.2	65.2	0.313	7.500
111	Schutz and Newmark (1952)	Y1A	1	39.2	65.2	0.313	7.500
112	Schutz and Newmark (1952)	Y1B	1	39.2	65.2	0.313	7.500
113	Schutz and Newmark (1952)	Y2A	1	39.2	65.2	0.313	7.500
114	Schutz and Newmark (1952)	Y2B	1	39.2	65.2	0.313	7.500
115	Schutz and Newmark (1952)	Y3A	1	39.2	65.2	0.313	7.500
116	Schutz and Newmark (1952)	Y3B	1	39.2	65.2	0.313	7.500

*indicates measured value

Table 9.9: Bolted Plate (with Staggered Bolts) Connection Experimental Specimen Fastener Details

Index	Bolts/Rivets	d, in	Total # of Fasteners	# Fasteners in Last Column
1	Rivets	0.750	5	2
2	Rivets	0.750	5	2
3	Rivets	0.750	5	2
4	Rivets	0.750	5	2
5	Rivets	0.750	5	2
6	Rivets	0.750	5	2
7	Rivets	0.750	5	2
8	Rivets	0.750	5	2
9	Rivets	0.750	5	2
10	Rivets	1.000	5	2
11	Rivets	1.000	5	2
12	Rivets	1.000	5	2
13	Rivets	1.000	5	2
14	Rivets	1.000	5	2
15	Rivets	1.000	5	2
16	Rivets	1.000	5	2
17	Rivets	0.750	5	2
18	Rivets	0.750	5	2
19	Rivets	0.750	5	2
20	Rivets	0.750	5	2
21	Rivets	0.750	5	2
22	Rivets	1.000	5	2
23	Rivets	1.000	5	2
24	Rivets	0.375	12	2
25	Rivets	0.375	12	2
26	Rivets	0.375	12	2
27	Rivets	0.375	12	2
28	Rivets	0.375	12	2
29	Rivets	0.375	12	2
30	Rivets	0.375	12	2
31	Rivets	0.375	12	2
32	Rivets	0.375	12	2
33	Rivets	0.375	12	2
34	Rivets	0.375	21	3
35	Rivets	0.375	21	3
36	Rivets	0.375	21	3
37	Rivets	0.375	21	3
38	Rivets	0.375	21	3
39	Rivets	0.375	21	3
40	Rivets	0.375	8	2
41	Rivets	0.375	8	2
42	Rivets	0.375	8	2

Table 9.9 continued

Index	Bolts/Rivets	d, in	Total # of Fasteners	# Fasteners in Last Column
43	Rivets	0.375	8	2
44	Rivets	0.375	14	2
45	Rivets	0.375	14	2
46	Rivets	0.375	14	2
47	Rivets	0.375	14	2
48	Rivets	0.375	14	2
49	Rivets	0.375	14	2
50	Rivets	0.375	12	2
51	Rivets	0.375	12	2
52	Rivets	0.375	12	2
53	Rivets	0.375	12	2
54	Rivets	0.375	12	2
55	Rivets	0.375	12	2
56	Rivets	0.375	12	1
57	Rivets	0.375	12	1
58	Rivets	0.375	8	2
59	Rivets	0.375	8	2
60	Rivets	0.375	8	3
61	Rivets	0.375	8	3
62	Rivets	0.375	8	2
63	Rivets	0.375	8	2
64	Rivets	0.375	8	2
65	Rivets	0.375	8	2
66	Rivets	0.375	8	2
67	Rivets	0.375	8	2
68	Rivets	0.375	8	1
69	Rivets	0.375	8	1
70	Rivets	0.375	8	1
71	Rivets	0.375	8	1
72	Rivets	0.375	8	1
73	Rivets	0.375	8	1
74	Rivets	0.375	8	1
75	Rivets	0.375	8	1
76	Rivets	0.375	8	1
77	Rivets	0.375	8	1
78	Rivets	0.375	10	1
79	Rivets	0.375	10	1
80	Rivets	0.375	13	1
81	Rivets	0.375	13	1
82	Rivets	0.375	8	2
83	Rivets	0.375	8	2
84	Rivets	0.375	8	2

Table 9.9 continued

Index	Bolts/Rivets	d, in	Total # of Fasteners	# Fasteners in Last Column
85	Rivets	0.375	8	2
86	Rivets	0.375	10	2
87	Rivets	0.375	10	2
88	Rivets	0.375	11	2
89	Rivets	0.375	11	2
90	Rivets	0.375	8	2
91	Rivets	0.375	8	2
92	Rivets	0.375	8	2
93	Rivets	0.375	10	2
94	Rivets	0.375	10	2
95	Rivets	0.375	13	1
96	Rivets	0.375	13	1
97	Rivets	0.375	15	2
98	Rivets	0.375	15	2
99	Rivets	0.375	15	2
100	Rivets	0.375	15	2
101	Rivets	0.375	15	2
102	Rivets	0.375	15	2
103	Rivets	0.375	15	2
104	Rivets	0.375	15	2
105	Rivets	0.375	15	2
106	Rivets	0.375	15	2
107	Rivets	0.375	15	2
108	Rivets	0.375	15	2
109	Rivets	0.375	22	2
110	Rivets	0.375	22	2
111	Rivets	0.375	15	2
112	Rivets	0.375	15	2
113	Rivets	0.375	15	2
114	Rivets	0.375	15	2
115	Rivets	0.375	15	2
116	Rivets	0.375	15	2

9.3.2 Results

Results are presented in this section in the manner described in Section 3.4.

Table 9.10: AISC Calculation Results for Bolted Plate (with Staggered Bolts) Connections

Index	**P_{YIELD}, kips**	**A_n, in^2**	***U***	**Controlling U Case**	**$P_{RUPTURE}$, kips**
1	177.7	3.675	1.0	Case 1	247.7
2	177.7	3.675	1.0	Case 1	247.7
3	177.5	3.754	1.0	Case 1	249.2
4	177.5	3.754	1.0	Case 1	249.2
5	152.1	3.598	1.0	Case 1	226.6
6	152.1	3.598	1.0	Case 1	226.6
7	167.7	3.623	1.0	Case 1	241.3
8	167.7	3.623	1.0	Case 1	241.3
9	157.9	3.653	1.0	Case 1	229.8
10	245.3	6.425	1.0	Case 1	390.0
11	245.3	6.425	1.0	Case 1	390.0
12	272.4	6.512	1.0	Case 1	426.5
13	272.4	6.512	1.0	Case 1	426.5
14	259.4	6.348	1.0	Case 1	399.3
15	259.4	6.348	1.0	Case 1	399.3
16	297.6	6.668	1.0	Case 1	433.4
17	141.7	3.544	1.0	Case 1	213.4
18	141.7	3.544	1.0	Case 1	213.4
19	141.7	3.544	1.0	Case 1	213.4
20	152.9	3.631	1.0	Case 1	225.1
21	152.9	3.631	1.0	Case 1	225.1
22	229.8	6.395	1.0	Case 1	367.7
23	258.6	6.348	1.0	Case 1	404.3
24	73.4	1.604	1.0	Case 1	104.7
25	73.4	1.604	1.0	Case 1	104.7
26	73.4	1.604	1.0	Case 1	104.6
27	73.4	1.604	1.0	Case 1	104.6
28	73.4	1.604	1.0	Case 1	104.6
29	73.4	1.604	1.0	Case 1	104.6
30	73.4	1.604	1.0	Case 1	104.6
31	73.4	1.604	1.0	Case 1	104.6
32	73.4	1.604	1.0	Case 1	104.6
33	73.4	1.604	1.0	Case 1	104.6
34	128.5	2.504	1.0	Case 1	163.4
35	128.5	2.504	1.0	Case 1	163.4
36	128.5	2.817	1.0	Case 1	183.8
37	128.5	2.817	1.0	Case 1	183.8
38	128.5	2.876	1.0	Case 1	187.6
39	128.5	2.876	1.0	Case 1	187.6
40	55.1	1.135	1.0	Case 1	74.0
41	55.1	1.135	1.0	Case 1	74.0
42	55.1	1.135	1.0	Case 1	74.0

Table 9.10 continued

Index	P_{YIELD}, kips	A_n, in^2	U	Controlling U Case	$P_{RUPTURE}$, kips
43	55.1	1.135	1.0	Case 1	74.0
44	73.4	1.604	1.0	Case 1	104.7
45	73.4	1.604	1.0	Case 1	104.7
46	73.4	1.604	1.0	Case 1	104.7
47	73.4	1.604	1.0	Case 1	104.7
48	73.4	1.604	1.0	Case 1	104.7
49	73.4	1.604	1.0	Case 1	104.7
50	73.4	1.565	1.0	Case 1	102.1
51	73.4	1.565	1.0	Case 1	102.1
52	73.4	1.604	1.0	Case 1	104.6
53	73.4	1.604	1.0	Case 1	104.6
54	73.4	1.565	1.0	Case 1	102.1
55	73.4	1.565	1.0	Case 1	102.1
56	73.4	1.741	1.0	Case 1	113.6
57	73.4	1.741	1.0	Case 1	113.6
58	55.1	1.135	1.0	Case 1	74.0
59	55.1	1.135	1.0	Case 1	74.0
60	55.1	0.998	1.0	Case 1	65.1
61	55.1	0.998	1.0	Case 1	65.1
62	55.1	1.024	1.0	Case 1	66.8
63	55.1	1.024	1.0	Case 1	66.8
64	55.1	1.135	1.0	Case 1	74.0
65	55.1	1.135	1.0	Case 1	74.0
66	55.1	0.998	1.0	Case 1	65.1
67	55.1	0.998	1.0	Case 1	65.1
68	55.1	1.024	1.0	Case 1	66.8
69	55.1	1.024	1.0	Case 1	66.8
70	55.1	1.024	1.0	Case 1	66.8
71	55.1	1.024	1.0	Case 1	66.8
72	55.1	1.024	1.0	Case 1	66.8
73	55.1	1.024	1.0	Case 1	66.8
74	55.1	1.161	1.0	Case 1	75.7
75	55.1	1.161	1.0	Case 1	75.7
76	55.1	1.272	1.0	Case 1	83.0
77	55.1	1.272	1.0	Case 1	83.0
78	55.1	0.998	1.0	Case 1	65.1
79	55.1	0.998	1.0	Case 1	65.1
80	55.1	0.998	1.0	Case 1	65.1
81	55.1	0.998	1.0	Case 1	65.1
82	49.0	0.978	1.0	Case 1	63.8
83	49.0	0.978	1.0	Case 1	63.8
84	52.6	1.072	1.0	Case 1	69.9

Table 9.10 continued

Index	P_{YIELD}, kips	A_n, in^2	U	Controlling U Case	$P_{RUPTURE}$, kips
85	52.6	1.072	1.0	Case 1	69.9
86	59.7	1.252	1.0	Case 1	81.7
87	59.7	1.252	1.0	Case 1	81.7
88	73.4	1.604	1.0	Case 1	104.7
89	73.4	1.604	1.0	Case 1	104.7
90	49.0	0.978	1.0	Case 1	63.8
91	52.6	1.072	1.0	Case 1	69.9
92	52.6	1.072	1.0	Case 1	69.9
93	59.7	1.252	1.0	Case 1	81.7
94	59.7	1.252	1.0	Case 1	81.7
95	73.4	1.627	1.0	Case 1	106.1
96	73.4	1.627	1.0	Case 1	106.1
97	91.8	1.715	1.0	Case 1	111.9
98	91.8	1.715	1.0	Case 1	111.9
99	91.8	1.963	1.0	Case 1	128.1
100	91.8	1.963	1.0	Case 1	128.1
101	91.8	2.074	1.0	Case 1	135.3
102	91.8	2.074	1.0	Case 1	135.3
103	91.8	1.780	1.0	Case 1	116.1
104	91.8	1.780	1.0	Case 1	116.1
105	91.8	1.871	1.0	Case 1	122.1
106	91.8	1.871	1.0	Case 1	122.1
107	91.8	2.035	1.0	Case 1	132.7
108	91.8	2.035	1.0	Case 1	132.7
109	91.8	2.074	1.0	Case 1	135.3
110	91.8	2.074	1.0	Case 1	135.3
111	91.8	1.744	1.0	Case 1	113.8
112	91.8	1.744	1.0	Case 1	113.8
113	91.8	1.963	1.0	Case 1	128.1
114	91.8	1.963	1.0	Case 1	128.1
115	91.8	2.074	1.0	Case 1	135.3
116	91.9	2.074	1.0	Case 1	135.3

Table 9.11: Summary Strength Results for Bolted Plate (with Staggered Bolts) Connections

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
1	180.7	[2]	177.7	[1]	161.9	[3]
2	168.6	[2]	177.7	[1]	161.9	[3]
3	211.7	[2]	177.5	[1]	165.2	[3]
4	214.2	[2]	177.5	[1]	165.2	[3]
5	195.0	[2]	152.1	[1]	145.1	[3]
6	196.0	[2]	152.1	[1]	145.1	[3]
7	188.2	[2]	167.7	[1]	159.8	[3]
8	202.0	[2]	167.7	[1]	159.8	[3]
9	191.2	[2]	157.9	[1]	150.0	[3]
10	341.1	[2]	245.3	[1]	229.8	[3]
11	334.4	[2]	245.3	[1]	229.8	[3]
12	370.0	[2]	272.4	[1]	257.2	[3]
13	369.0	[2]	272.4	[1]	257.2	[3]
14	331.2	[2]	259.4	[1]	246.8	[3]
15	328.6	[2]	259.4	[1]	246.8	[3]
16	357.0	[2]	297.6	[1]	285.3	[3]
17	211.6	[2]	141.7	[1]	130.2	[3]
18	209.2	[2]	141.7	[1]	130.2	[3]
19	229.4	[2]	141.7	[1]	128.4	[3]
20	214.3	[2]	152.9	[1]	147.1	[3]
21	211.9	[2]	152.9	[1]	147.1	[3]
22	377.0	[2]	229.8	[1]	210.2	[3]
23	350.3	[2]	258.6	[1]	246.2	[3]
24	111.8	[2]	73.4	[1]	71.7	[3]
25	109.9	[2]	73.4	[1]	71.7	[3]
26	115.0	[2]	73.4	[1]	71.7	[3]
27	115.0	[2]	73.4	[1]	71.7	[3]
28	109.7	[2]	73.4	[1]	71.7	[3]
29	113.2	[2]	73.4	[1]	71.7	[3]
30	110.3	[2]	73.4	[1]	71.7	[3]
31	113.4	[2]	73.4	[1]	71.7	[3]
32	111.0	[2]	73.4	[1]	71.7	[3]
33	111.1	[2]	73.4	[1]	71.7	[3]
34	183.8	[2]	128.5	[1]	121.1	[3]
35	190.3	[2]	128.5	[1]	121.1	[3]
36	186.4	[2]	128.5	[1]	126.7	[3]
37	192.5	[2]	128.5	[1]	126.7	[3]
38	195.9	[2]	128.5	[1]	127.3	[3]
39	192.8	[2]	128.5	[1]	127.3	[3]
40	79.9	[2]	55.1	[1]	49.0	[3]
41	81.0	[2]	55.1	[1]	49.0	[3]
42	80.4	[2]	55.1	[1]	52.6	[3]

Table 9.11 continued

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	Failure Mode	P_{AISC}, kips	Controlling Limit State	P_{IDEA}, kips	Failure Mode
43	83.1	[2]	55.1	[1]	52.6	[3]
44	110.2	[2]	73.4	[1]	71.7	[3]
45	110.3	[2]	73.4	[1]	71.7	[3]
46	112.6	[2]	73.4	[1]	71.7	[3]
47	112.6	[2]	73.4	[1]	71.7	[3]
48	111.3	[2]	73.4	[1]	71.7	[3]
49	111.7	[2]	73.4	[1]	71.7	[3]
50	108.1	[2]	73.4	[1]	71.0	[3]
51	107.2	[2]	73.4	[1]	71.0	[3]
52	115.3	[2]	73.4	[1]	71.7	[3]
53	112.8	[2]	73.4	[1]	71.7	[3]
54	105.6	[2]	73.4	[1]	68.5	[3]
55	105.8	[2]	73.4	[1]	68.5	[3]
56	112.6	[2]	73.4	[1]	72.4	[3]
57	116.9	[2]	73.4	[1]	72.4	[3]
58	80.4	[2]	55.1	[1]	52.8	[3]
59	82.5	[2]	55.1	[1]	52.8	[3]
60	79.8	[2]	55.1	[1]	48.6	[3]
61	81.0	[2]	55.1	[1]	48.6	[3]
62	85.0	[2]	55.1	[1]	48.9	[3]
63	84.0	[2]	55.1	[1]	48.9	[3]
64	83.5	[2]	55.1	[1]	51.6	[3]
65	84.8	[2]	55.1	[1]	51.6	[3]
66	81.1	[2]	55.1	[1]	52.5	[3]
67	81.5	[2]	55.1	[1]	52.5	[3]
68	79.4	[2]	55.1	[1]	49.0	[3]
69	78.9	[2]	55.1	[1]	49.0	[3]
70	77.0	[2]	55.1	[1]	49.0	[3]
71	79.7	[2]	55.1	[1]	49.0	[3]
72	81.1	[2]	55.1	[1]	49.0	[3]
73	81.4	[2]	55.1	[1]	49.0	[3]
74	82.1	[2]	55.1	[1]	50.6	[3]
75	81.3	[2]	55.1	[1]	50.6	[3]
76	87.3	[2]	55.1	[1]	52.3	[3]
77	85.5	[2]	55.1	[1]	52.3	[3]
78	86.8	[2]	55.1	[1]	52.2	[3]
79	84.8	[2]	55.1	[1]	52.2	[3]
80	86.4	[2]	55.1	[1]	51.9	[3]
81	85.8	[2]	55.1	[1]	51.9	[3]
82	68.4	[2]	49.0	[1]	46.1	[3]
83	68.9	[2]	49.0	[1]	46.1	[3]
84	79.1	[2]	52.6	[1]	49.8	[3]

Table 9.11 continued

Index	Experimental		AISC		IDEA StatiCa	
	P_{EXP}, kips	**Failure Mode**	**P_{AISC}, kips**	**Controlling Limit State**	**P_{IDEA}, kips**	**Failure Mode**
85	78.6	[2]	52.6	[1]	49.8	[3]
86	92.8	[2]	59.7	[1]	57.3	[3]
87	92.1	[2]	59.7	[1]	57.3	[3]
88	112.0	[2]	73.4	[1]	71.6	[3]
89	114.3	[2]	73.4	[1]	71.6	[3]
90	68.4	[2]	49.0	[1]	43.7	[3]
91	77.2	[2]	52.6	[1]	48.2	[3]
92	77.7	[2]	52.6	[1]	48.2	[3]
93	89.5	[2]	59.7	[1]	56.4	[3]
94	91.2	[2]	59.7	[1]	56.4	[3]
95	105.7	[2]	73.4	[1]	69.4	[3]
96	110.6	[2]	73.4	[1]	69.4	[3]
97	129.3	[2]	91.8	[1]	83.8	[3]
98	126.7	[2]	91.8	[1]	83.8	[3]
99	132.9	[2]	91.8	[1]	88.7	[3]
100	138.3	[2]	91.8	[1]	88.7	[3]
101	135.1	[2]	91.8	[1]	90.0	[3]
102	139.9	[2]	91.8	[1]	90.0	[3]
103	138.7	[2]	91.8	[1]	85.0	[3]
104	131.0	[2]	91.8	[1]	85.0	[3]
105	140.7	[2]	91.8	[1]	86.2	[3]
106	139.0	[2]	91.8	[1]	86.2	[3]
107	143.4	[2]	91.8	[1]	89.5	[3]
108	141.9	[2]	91.8	[1]	89.5	[3]
109	137.0	[2]	91.8	[1]	89.6	[3]
110	137.6	[2]	91.8	[1]	89.6	[3]
111	132.8	[2]	91.8	[1]	84.6	[3]
112	136.8	[2]	91.8	[1]	84.6	[3]
113	133.6	[2]	91.8	[1]	88.4	[3]
114	136.1	[2]	91.8	[1]	88.4	[3]
115	140.4	[2]	91.8	[1]	89.8	[3]
116	139.4	[2]	91.9	[1]	89.8	[3]

[1] tensile yield; [2] tensile rupture; [3] plate strain

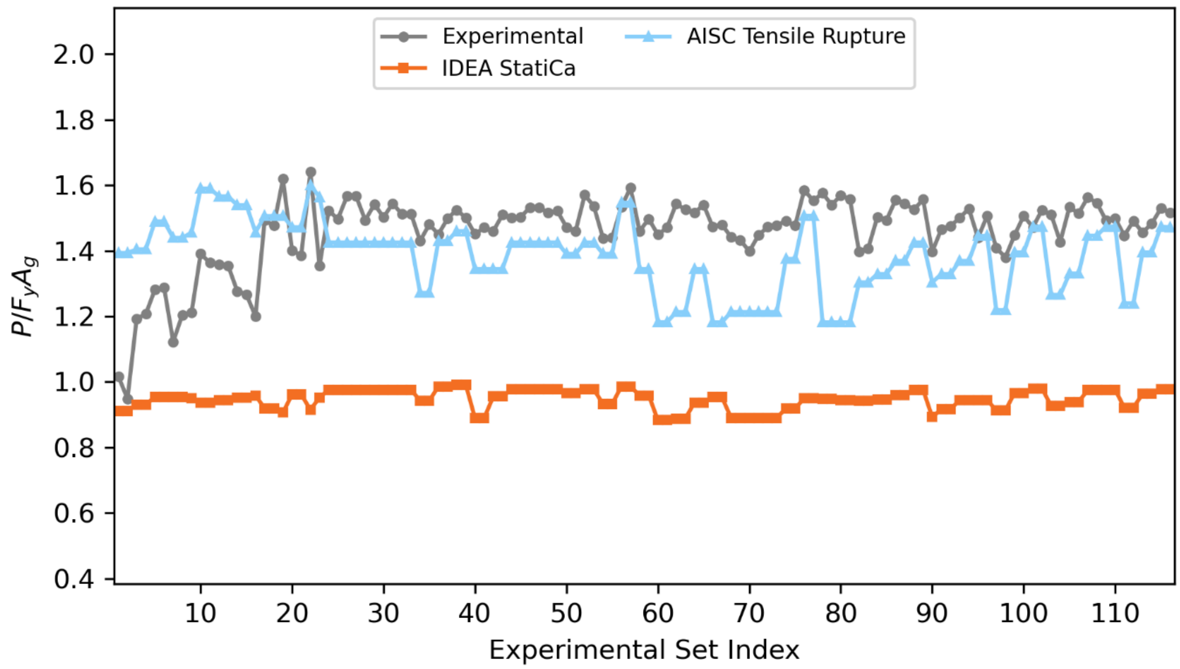


Figure 9-12: Normalized Strength Results for Bolted Plate (with Staggered Bolts) Connections

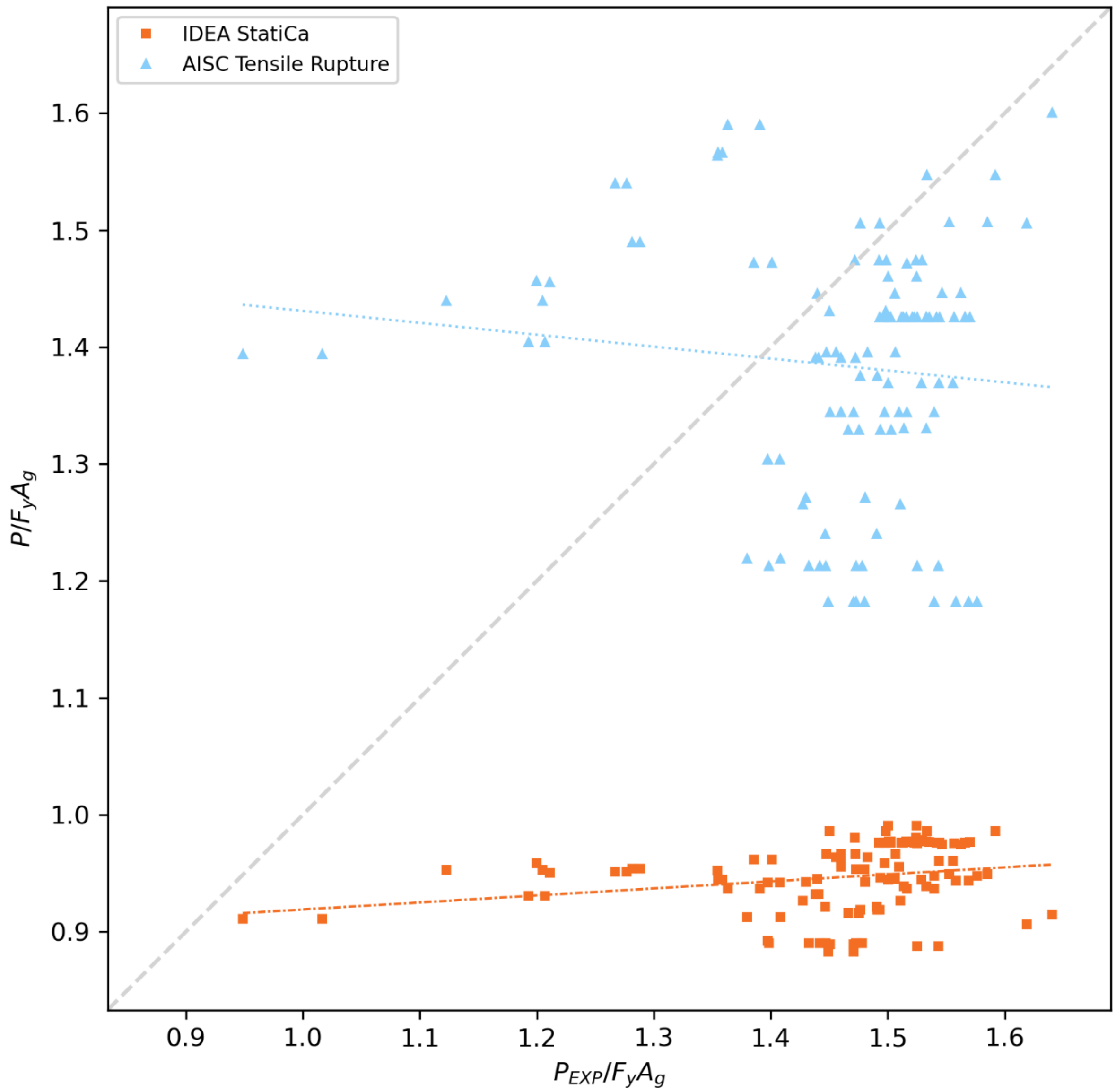


Figure 9-13: Normalized Strength vs. Normalized Experimental Strength for Bolted Plate (with Staggered Bolts) Connections

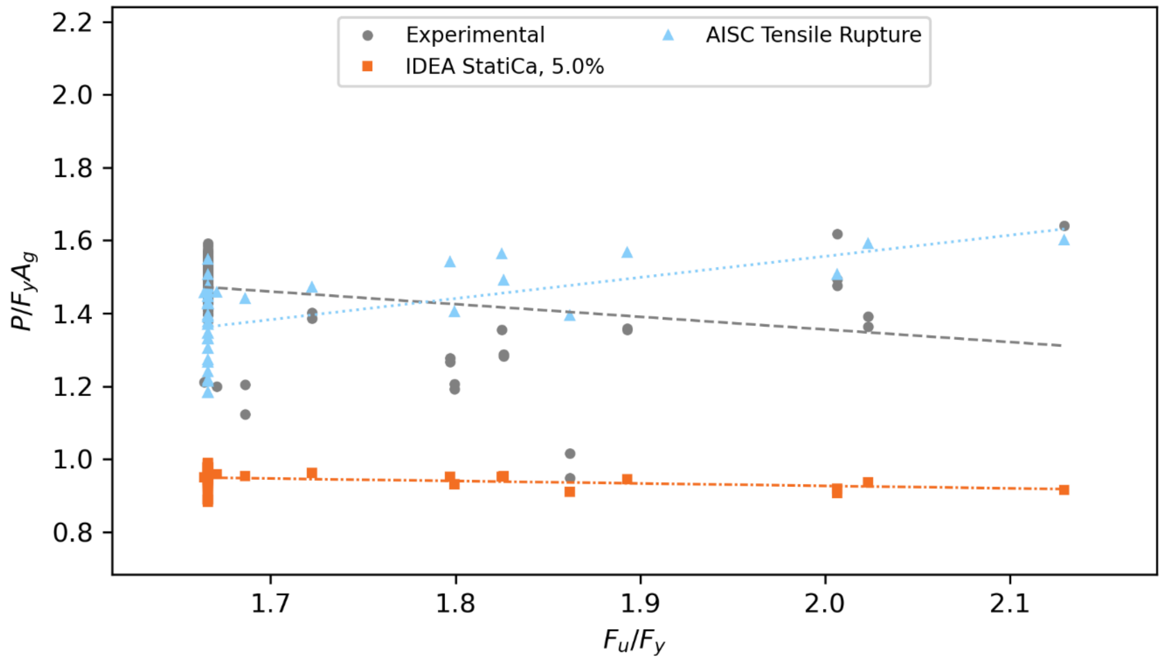


Figure 9-14: Normalized Strength vs. Material Strength Ratio for Bolted Plate (with Staggered Bolts) Connections

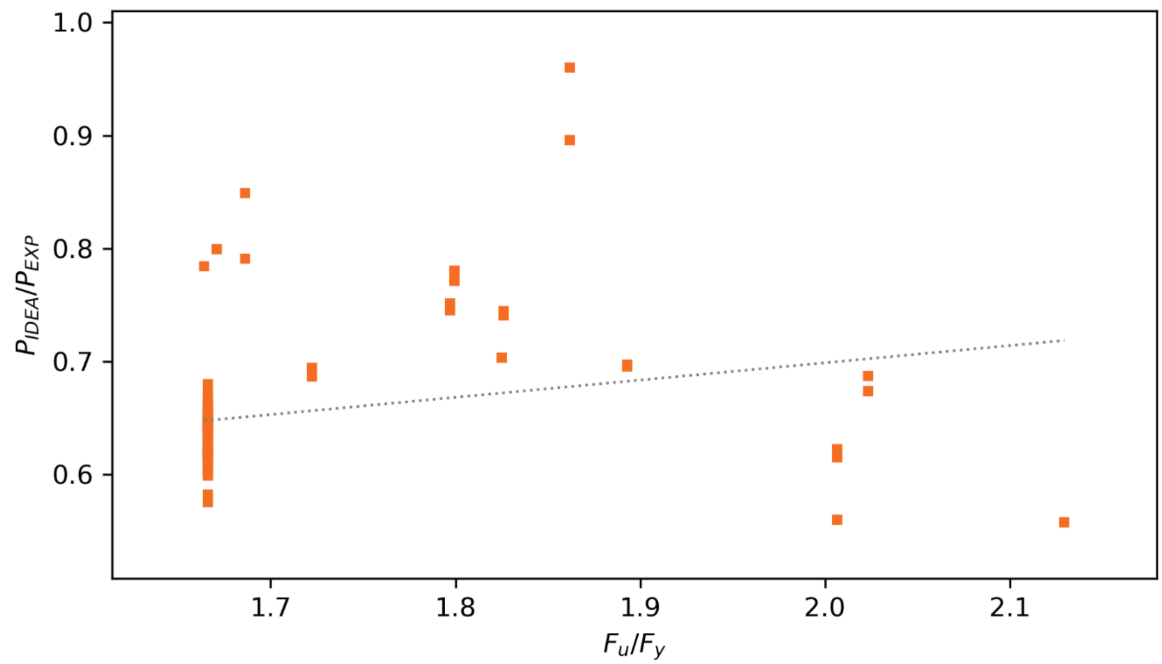


Figure 9-15: Ratio of IDEA StatiCa Strength to Experimental vs. Material Strength Ratio for Bolted Plate (with Staggered Bolts) Connections

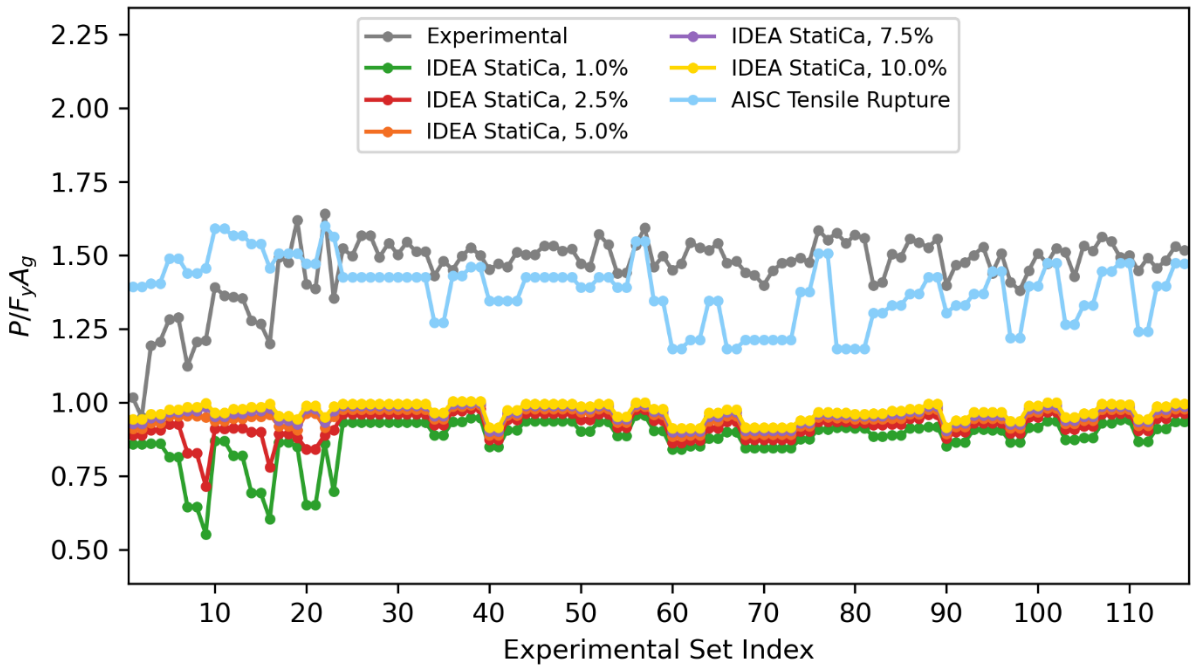


Figure 9-16: Normalized Strength Results for Bolted Plate (with Staggered Bolts) Connections Including Various Plastic Strain Limits for IDEA StatiCa

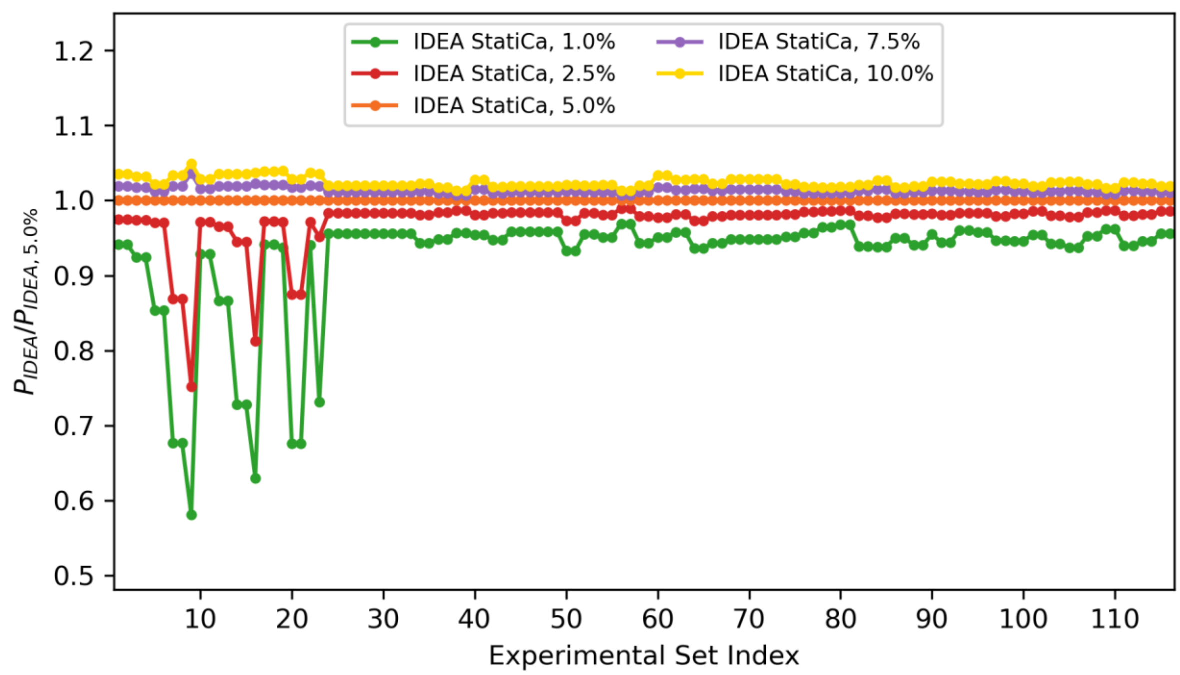


Figure 9-17: Ratio of IDEA StatiCa Strength for Various Plastic Strain Limits to IDEA StatiCa Strength for Default Plastic Strain Limit for Bolted Plate (with Staggered Bolts) Connections

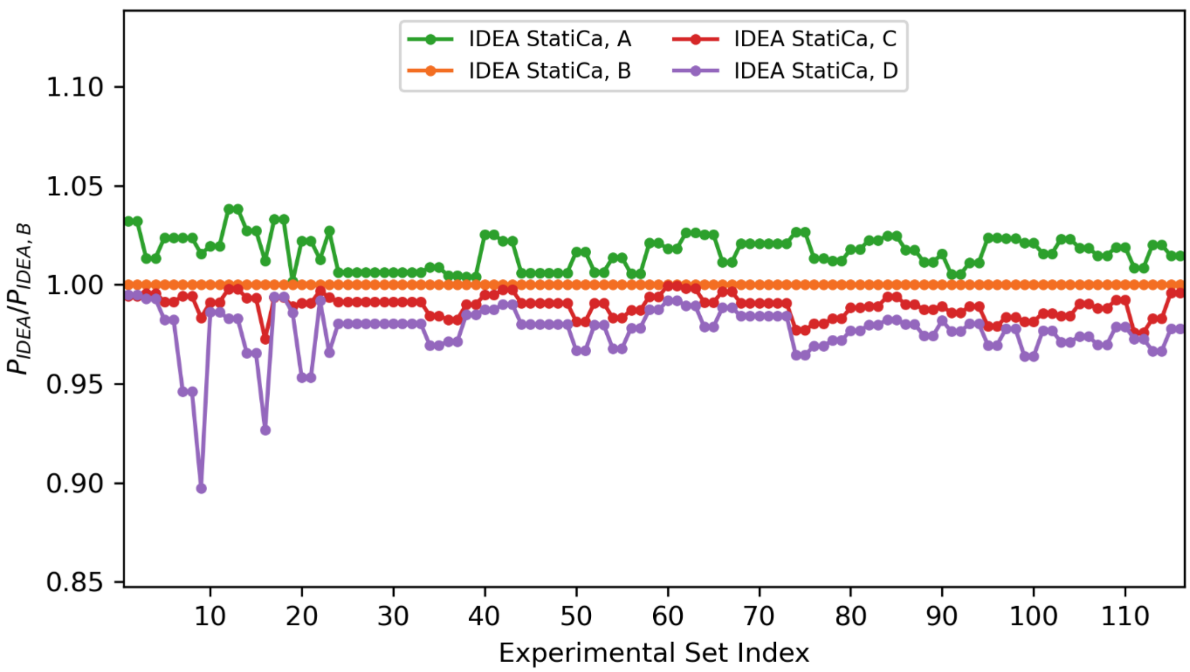


Figure 9-18: Ratio of IDEA StatiCa Strength for Various Mesh Parameters to IDEA StatiCa Strength for Default Mesh Parameters for Bolted Plate (with Staggered Bolts) Connections

Table 9.12: Summary Statistics of the Test-to-Predicted Ratio for Bolted Plate (with Staggered Bolts) Connections

Test-to-Predicted Ratio	**Average**	**Standard Deviation**	**Coefficient of Variation**
$P_{EXP}/P_{RUPTURE}$ (AISC)	1.061	0.124	0.117
P_{EXP}/P_{IDEA}	1.542	0.120	0.078

9.3.3 Discussion

The results for the bolted plates with staggered bolt patterns are largely similar to those with regular bolt patterns. One difference is that the AISC strengths appear relatively uncorrelated to the experimental strengths (Figure 9-13).

9.4 Reliability Analysis

9.4.1 Description of Reliability Set

For the bolted plate reliability set, it was desired to vary the following parameters: material grade, bolt diameter, and bolt configuration. The width of the plate was directly dependent upon the selected bolt gage and L_{ev}. The bolt spacings were arranged into two categories, a 'minimum' and 'greater than minimum' spacing type. The bolt spacings for the 'minimum' spacing type were selected following the minimum spacing requirements in Section J3 of the AISC *Specification* (2016). The bolt spacings for the 'greater than minimum' spacing type were selected based upon spacings typically used in practice. All bolt hole diameters were selected based upon Section J3 of the AISC *Specification* as well. The parameters for this reliability set are provided in two separate tables. Table 9.13 provides general parameters, whereas Table 9.14 provides information specifically on the bolt spacings. The terminology is consistent with that expressed earlier in this chapter. The material grades are specifically expressed for plates. The literature signified separate factors for the average and coefficient of variation values of $\tilde{X}_{Fy}$ and $\tilde{X}_{Fu}$ for plates and angles. These values are listed in Table 3.4.

Table 9.13: Bolted Plate Connection Reliability Set Parameters

Index	W, in.	t, in.	Material Grade	F_y, ksi	F_u, ksi	Bolt Diameter, in.	Rows of Bolts	Bolts per Row
1	2.000	0.5	A36 (PL)	36	58	0.750	1	2
2	2.000	0.5	A36 (PL)	36	58	0.750	1	4
3	4.000	0.5	A36 (PL)	36	58	0.750	2	2
4	4.000	0.5	A36 (PL)	36	58	0.750	2	4
5	4.000	0.5	A36 (PL)	36	58	0.750	1	2
6	4.000	0.5	A36 (PL)	36	58	0.750	1	4
7	7.000	0.5	A36 (PL)	36	58	0.750	2	2
8	7.000	0.5	A36 (PL)	36	58	0.750	2	4
9	3.000	0.5	A36 (PL)	36	58	1.125	1	2
10	3.000	0.5	A36 (PL)	36	58	1.125	1	4
11	6.000	0.5	A36 (PL)	36	58	1.125	2	2
12	6.000	0.5	A36 (PL)	36	58	1.125	2	4
13	4.000	0.5	A36 (PL)	36	58	1.125	1	2
14	4.000	0.5	A36 (PL)	36	58	1.125	1	4
15	7.375	0.5	A36 (PL)	36	58	1.125	2	2
16	7.375	0.5	A36 (PL)	36	58	1.125	2	4
17	2.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	1	2
18	2.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	1	4
19	4.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	2	2
20	4.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	2	4
21	4.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	1	2
22	4.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	1	4
23	7.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	2	2
24	7.000	0.5	A572 Gr. 50 (PL)	50	65	0.750	2	4
25	3.000	0.5	A572 Gr. 50 (PL)	50	65	1.125	1	2
26	3.000	0.5	A572 Gr. 50 (PL)	50	65	1.125	1	4
27	6.000	0.5	A572 Gr. 50 (PL)	50	65	1.125	2	2
28	6.000	0.5	A572 Gr. 50 (PL)	50	65	1.125	2	4
29	4.000	0.5	A572 Gr. 50 (PL)	50	65	1.125	1	2
30	4.000	0.5	A572 Gr. 50 (PL)	50	65	1.125	1	4
31	7.375	0.5	A572 Gr. 50 (PL)	50	65	1.125	2	2
32	7.375	0.5	A572 Gr. 50 (PL)	50	65	1.125	2	4

PL: plate

Table 9.14: Bolted Plate Connection Reliability Set Spacing Parameters

Index	***s*, in.**	***g*, in.**	**L_{ev}, in.**	**L_{eh}, in.**
1	2.000	2.000	1.000	1.000
2	2.000	2.000	1.000	1.000
3	2.000	2.000	1.000	1.000
4	2.000	2.000	1.000	1.000
5	3.000	3.000	2.000	2.000
6	3.000	3.000	2.000	2.000
7	3.000	3.000	2.000	2.000
8	3.000	3.000	2.000	2.000
9	3.000	3.000	1.500	1.500
10	3.000	3.000	1.500	1.500
11	3.000	3.000	1.500	1.500
12	3.000	3.000	1.500	1.500
13	3.375	3.375	2.000	2.000
14	3.375	3.375	2.000	2.000
15	3.375	3.375	2.000	2.000
16	3.375	3.375	2.000	2.000
17	2.000	2.000	1.000	1.000
18	2.000	2.000	1.000	1.000
19	2.000	2.000	1.000	1.000
20	2.000	2.000	1.000	1.000
21	3.000	3.000	2.000	2.000
22	3.000	3.000	2.000	2.000
23	3.000	3.000	2.000	2.000
24	3.000	3.000	2.000	2.000
25	3.000	3.000	1.500	1.500
26	3.000	3.000	1.500	1.500
27	3.000	3.000	1.500	1.500
28	3.000	3.000	1.500	1.500
29	3.375	3.375	2.000	2.000
30	3.375	3.375	2.000	2.000
31	3.375	3.375	2.000	2.000
32	3.375	3.375	2.000	2.000

9.4.2 Results

The strength results used in the reliability analysis are provided in Table 9.15. For the strengths according to the AISC *Specification* (2016), the nominal and design strengths are provided for both tensile yield and tensile rupture; however, only the design strengths were used for the purpose of the reliability analysis. The strengths according to IDEA StatiCa, $\mathrm{P}_{\mathrm{IDEA}}$, were the result of analyses utilizing all default settings in the program. More specifically, the resistance factors were used with the default mesh settings (mesh parameter set ‘B’) and a 5% plastic strain limit. For this specific reliability specimen set, there were some specimens in which IDEA StatiCa provided a larger maximum permitted applied load, P_{IDEA}, than the available strength according to the AISC equations, the most extreme case resulted in only a 19% increase in strength compared to P_{AISC}. On the more conservative side, IDEA StatiCa provided a maximum decrease of 17% from P_{AISC}.

Table 9.15: Summary Strength Results for Bolted Plate Connection Reliability Set

Index	AISC				IDEA StatiCa
	P_{YIELD}, kips	$P_{RUPTURE}$, kips	ϕP_{YIELD}, kips	$\phi P_{RUPTURE}$, kips	P_{IDEA}, kips
1	36.0	32.6	32.4	24.5	22.7
2	36.0	32.6	32.4	24.5	22.2
3	72.0	65.3	64.8	48.9	46.2
4	72.0	65.3	64.8	48.9	45.2
5	72.0	90.6	64.8	68.0	54.0
6	72.0	90.6	64.8	68.0	53.7
7	126.0	152.3	113.4	114.2	95.6
8	126.0	152.3	113.4	114.2	95.5
9	54.0	48.9	48.6	36.7	34.9
10	54.0	48.9	48.6	36.7	34.9
11	108.0	97.9	97.2	73.4	72.0
12	108.0	97.9	97.2	73.4	71.8
13	72.0	77.9	64.8	58.5	51.2
14	72.0	77.9	64.8	58.5	51.2
15	132.8	137.8	119.5	103.3	95.1
16	132.8	137.8	119.5	103.3	95.2
17	50.0	36.6	45.0	27.4	30.5
18	50.0	36.6	45.0	27.4	29.5
19	100.0	73.1	90.0	54.8	62.1
20	100.0	73.1	90.0	54.8	60.0
21	100.0	101.6	90.0	76.2	71.7
22	100.0	101.6	90.0	76.2	72.7
23	175.0	170.6	157.5	128.0	126.3
24	175.0	170.6	157.5	128.0	128.7
25	75.0	54.8	67.5	41.1	47.2
26	75.0	54.8	67.5	41.1	47.1
27	150.0	109.7	135.0	82.3	97.6
28	150.0	109.7	135.0	82.3	97.4
29	100.0	87.3	90.0	65.5	69.7
30	100.0	87.3	90.0	65.5	69.5
31	184.4	154.4	165.9	115.8	129.2
32	184.4	154.4	165.9	115.8	129.3

The resulting reliability indices, β, are provided for each corresponding specimen in Figure 9-19 for AISC and IDEA StatiCa. The reliability indices compare well from AISC to IDEA StatiCa. IDEA StatiCa does appear to provide a higher level of reliability for the first 16 specimens rather than the second set of 16 (reliability specimens with specimen indices 17-32). The second set of specimens consisted of A572 Gr. 50 material, resulting in a larger F_u/F_y ratio.

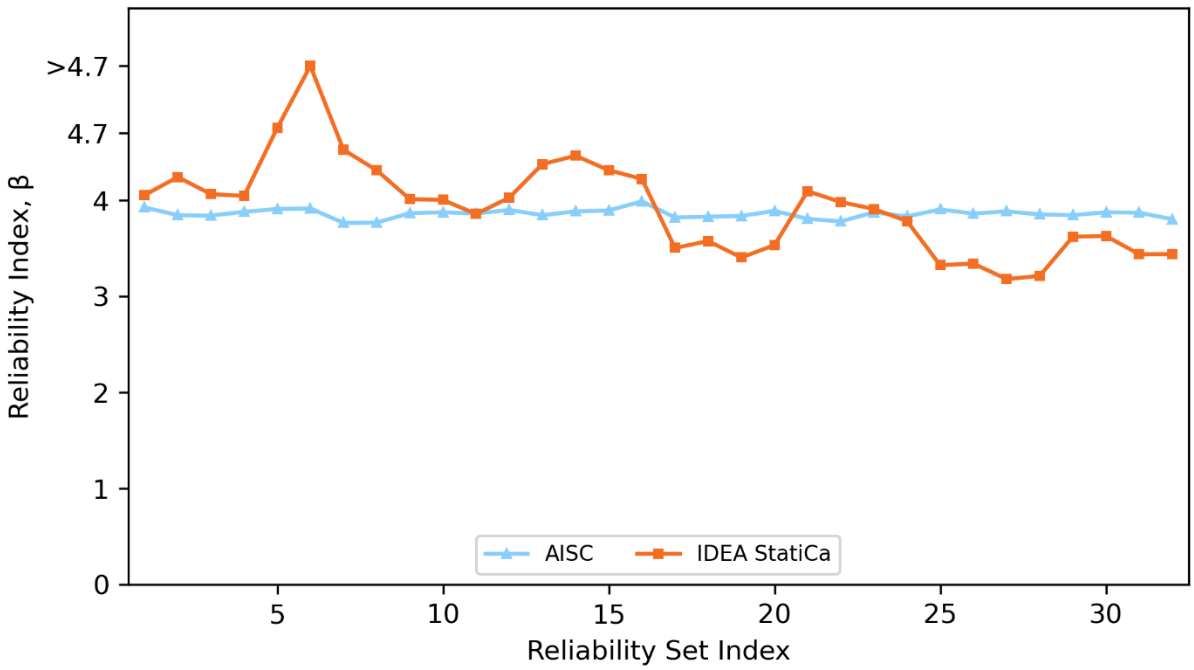


Figure 9-19: Reliability Index for Bolted Plate Connections

Chapter 10: CONCLUSIONS

Inelastic analysis can be a powerful tool for steel connection design that overcomes limitations of traditional hand calculation-based design procedures. However, as with all design methods, validation must be performed to ensure the resulting connections are safe. IDEA StatiCa is a connection design software that employs the component-based finite element method. As part of a larger verification and validation effort, this work compares results from IDEA StatiCa to results from previously published experimental results and to results from design equations for the limit state of net-section tensile rupture. Specifically, this work seeks to answer the following questions: (1) how well does IDEA StatiCa capture net-section tensile rupture? and (2) does IDEA StatiCa provide a comparable or higher level of reliability than the provisions of the AISC *Specification*?

Hundreds of previously published experiments of tension members that failed in either tensile rupture or tensile yield were examined. Each specimen was modeled and analyzed in IDEA StatiCa, and then strengths were compared to the results of design equations in the AISC *Specification* and experimental results. The sensitivity of the IDEA StatiCa results to mesh density and plastic strain limit was also investigated. Using statistical data from the comparisons to experiments, a reliability analysis was performed to quantify the probability of failure for connections designed using the various methods.

The results show that IDEA StatiCa captures the limit state of tensile rupture generally well with accurate or conservative expected strength in comparison to experimental results and design equations. Detailed findings of this work include:

- Using measured material and geometric properties without resistance factors applied, the strength from IDEA StatiCa was less than or equal to the experimentally observed strength for all but 12 specimens out of 529 (9 of which were fabricated with high strength steel, F_y = 122.8 ksi) and less than or equal to the expected tensile rupture strength computed using design equations for all but for 30 specimens out of 529.
- Using nominal material and geometric properties with resistance factors applied, unconservative errors of up to 56% were observed for IDEA StatiCa in comparison to the design strength for plate specimens with relatively short welds, up to 25% for rectangular HSS specimens, and up to 20% in other cases.
- The AISC *Specification* equations appear well calibrated with reliability indices between 3.17 and 4.42 for the connections investigated.
- The level of reliability provided by IDEA StatiCa was more variable across the connections investigated. In many cases the reliability for IDEA StatiCa was greater than for the AISC *Specification* equations. However, in some cases, IDEA StatiCa resulted in reliability indices as low as 2.05. The cases of lower reliability were the same as those where IDEA StatiCa exhibited higher strengths than the design strengths. In general, where tensile yield controls, IDEA StatiCa resulted in a greater reliability than the provisions of the AISC *Specification*.
- Given that the constitutive relation used in IDEA StatiCa is based on the yield strength, F_y, and tensile rupture strength is more correlated to the tensile strength, F_u, IDEA StatiCa

tends to produce more conservative results for connections with members that have a higher material strength ratio, F_u/F_y.

- The strength from IDEA StatiCa was particularly low for connections with significant eccentricity between the tension member and gusset plate (e.g., bolted and welded single angles). Physically, the eccentricity is reduced as the connection deforms under tension, however IDEA StatiCa utilizes first-order (i.e., geometrically linear) analyses. Second-order (i.e., geometrically nonlinear) analyses would likely provide increased strengths for these connections.
- Results from IDEA StatiCa often do not converge with mesh refinement as is typically expected with finite element analyses. However, refining the mesh in IDEA StatiCa typically resulted in minor decreases in strength (i.e., less than 5% difference upon halving element size for most specimens). Some cases, particularly the HSS specimens, showed greater mesh dependence.
- Increasing the plastic strain limit increases the IDEA StatiCa strength and vice versa. The sensitivity of strength to plastic strain limit varied by connection.

Based on these results, use of IDEA StatiCa for capturing tensile rupture in structural steel connection design is typically acceptable. However, cases of unconservative error with respect to the current provisions of the AISC *Specification* identified in this work indicate that modifications to the program may be necessary. The challenge when implementing these modifications will be to improve results for the unconservative cases while not adding further conservatism to the already conservative cases. Potential modifications could include a different ϕ factor or plastic strain limit. However, both of these modifications would further increase the conservatism in most cases where IDEA StatiCa is already conservative as well as affect results for other limit states. Alternatively, changes to the modeling of discontinuities, welds, or bolts may also be necessary.

Further investigation is needed to verify the true strength of the connections which exhibited large differences between IDEA StatiCa and specification equations to determine if the differences are the result of unconservatism in IDEA StatiCa or conservatism in the specification equations and identify the most appropriate course of action. Such studies could include a fundamental characterization and quantification of the interrelationship between stress concentrations, analysis options, plastic strain limits, and fracture. Investigation of other similar limit states, such as block shear rupture, should also be conducted.

REFERENCES

AISC. 2016. *Specification for Structural Steel Buildings*. Chicago, Illinois: American Institute of Steel Construction.

AISC. 2017. *Steel Construction Manual*. Chicago, Illinois: American Institute of Steel Construction.

AISC. 2022a. *Public Review Draft of Specification for Structural Steel Buildings*. Chicago, Illinois: American Institute of Steel Construction.

AISC. 2022b. *Public Review Draft of AISC 341-22 Seismic Provisions for Structural Steel Buildings*. Chicago, Illinois: American Institute of Steel Construction.

ASTM. 2018. *ASTM A500/A500M-21a: Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*. West Conshohocken, PA: American Society of Testing Materials.

Bauer, D. B., and A. Benaddi. 2002. "Shear lag in double angle truss connections." *Advances in Steel Structures (ICASS'02)*, 181–188. Elsevier.

Cheng, J. J. R., G. L. Kulak, and H.-A. Khoo. 1998. "Strength of slotted tubular tension members." *Canadian Journal of Civil Engineering*, 25 (6): 982–991.

Chesson, E., and W. H. Munse. 1963. "Riveted and Bolted Joints: Truss-Type Tensile Connections." *Journal of the Structural Division*, 89 (1): 67–106. American Society of Civil Engineers. https://doi.org/10.1061/JSDEAG.0000891.

Davis, R. E., G. B. Woodruff, and H. E. Davis. 1940. "Tension Tests of Large Riveted Joints." *Transactions of the American Society of Civil Engineers*, ASCE, 105 (1): 1193–1245. https://doi.org/10.1061/TACEAT.0005216.

Denavit, M. D., and K. Truman-Jarrell. 2021. "Single Plate Shear Connections." Accessed June 25, 2022. https://www.ideastatica.com/support-center/single-plate-shear-connections-aisc.

Dhanuskar, J. R., and L. M. Gupta. 2021a. "Shear Lag Effect in Welded Single Angle Tension Member." *International Journal of Steel Structures*, 21 (3): 935–949. https://doi.org/10.1007/s13296-021-00482-1.

Dhanuskar, J. R., and L. M. Gupta. 2021b. "Behaviour of a single angle tension member welded at single leg and both legs." *Asian Journal of Civil Engineering*, 22 (6): 1157–1171. https://doi.org/10.1007/s42107-021-00372-1.

Dowswell, B. 2021. "Analysis of the Shear Lag Factor for Slotted Rectangular HSS Members." *Engineering Journal*, AISC, 58 (3): 155–164.

Easterling, W. S., and L. Gonzalez Giroux. 1993. "Shear lag effects in steel tension members." *Engineering Journal*, AISC, 30 (3): 77–89.

Ellingwood, B., T. Galambos, J. MacGregor, and C. A. Cornell. 1980. *Development of a probability based load criterion for American National Standard A58: Building code requirements for minimum design loads in buildings and other structures*. NBS Special Publication. 577. Washington, D.C.: National Bureau of Standards.

Epstein, H. I. 1992. "An experimental study of block shear failure of angles in tension." *Engineering journal*, AISC, 29 (2): 75–84.

Fang, C., A. C. C. Lam, and M. C. H. Yam. 2013. "Influence of shear lag on ultimate tensile capacity of angles and tees." *Journal of Constructional Steel Research*, 84 (1): 49–61. https://doi.org/10.1016/j.jcsr.2013.02.006.

Fisher, J. M., and L. A. Kloiber. 2006. *Design Guide 1: Base Plate and Anchor Rod Design*. Chicago, Illinois: American Institute of Steel Construction.

Gibson, G. T., and B. T. Wake. 1942. "An investigation of welded connections for angle tension members." *The Welding Journal*, 21 (1): 44–49.

Gonzalez, L. 1989. "Investigation of the shear lag coefficient for welded tension members." Master's Thesis. Blacksburg, Virginia: Virginia Tech.

Greiner, J. E. 1897. "Recent tests of bridge members." *Transactions of the American Society of Civil Engineers*, 38 (2): 41–67.

Kasapoglu, B., R. A. Giorjao, A. Nassiri, and H. Sezen. 2021. *Verification of the results from IDEA StatiCa for steel connections according to the U.S. design codes*. The Ohio State University.

Ke, K., Y. H. Xiong, M. C. H. Yam, A. C. C. Lam, and K. F. Chung. 2018. "Shear lag effect on ultimate tensile capacity of high strength steel angles." *Journal of Constructional Steel Research*, 145 (1): 300–314. https://doi.org/10.1016/j.jcsr.2018.02.015.

Korol, R. M. 1996. "Shear lag in slotted HSS tension members." *Can. J. Civ. Eng.*, 23 (6): 1350–1354. https://doi.org/10.1139/l96-943.

Kulak, G. L., and E. Y. Wu. 1997. "Shear Lag in Bolted Angle Tension Members." *Journal of Structural Engineering*, ASCE, 123 (9): 1144–1152. https://doi.org/10.1061/(ASCE)0733-9445(1997)123:9(1144).

Liu, J., R. Sabelli, R. L. Brockenbrough, and T. P. Fraser. 2007. "Expected Yield Stress and Tensile Strength Ratios for Determination of Expected Member Capacity in the 2005 AISC Seismic Provisions." *Engineering Journal*, AISC, 44 (1): 15–25.

Mannem, R. 2002. "Shear lag effects on welded steel angles and plates." PhD Thesis. St. John's, NL, Canada: Memorial University of Newfoundland.

Martinez-Saucedo, G., and J. A. Packer. 2009. "Static Design Recommendations for Slotted End HSS Connections in Tension." *Journal of Structural Engineering*, ASCE, 135 (7): 797–805. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000016.

McKibben, F. P. 1907. "Tension tests of steel angles." *Proceedings of the annual meeting*, ASTM, 267–274. Philadelphia, Pennsylvania.

Može, P., and D. Beg. 2010. "High strength steel tension splices with one or two bolts." *Journal of Constructional Steel Research*, 66 (8–9): 1000–1010. https://doi.org/10.1016/j.jcsr.2010.03.009.

Munse, W. H. 1959. *The effect of bearing pressure on the static strength of riveted connections.* Engineering Experiment Station Bulletin No. 454. Urbana, Illinois: University of Illinois.

Mahamid, M. 2021. “Chevron Brace Connection in a braced frame.” Accessed June 25, 2022. https://www.ideastatica.com/support-center/chevron-brace-connection-in-a-braced-frame-aisc.

Petretta, M. 2000. “An investigation of the shear lag effect in welded angle tensile connections.” Master’s Thesis. Toronto, Ontario, Canada: University of Toronto.

Ravindra, M. K., and T. V. Galambos. 1978. “Load and Resistance Factor Design for Steel.” *Journal of the Structural Division,* ASCE, 104 (9): 1337–1353.

Regan, P. E., and P. R. Salter. 1984. “Tests on welded-angle tension members.” *The Structural Engineer*, The Institute of Structural Engineers, 62B (2): 25–30.

Schutz, F. W., and N. M. Newmark. 1952. *The Efficiency of Riveted Structural Joints*. Structural Research Series. 30. Urbana, Illnois: University of Illinois.

Uzoegbo, H. C. 1998. “Shear lag in steel angles: An investigation of the South African standards.” *Journal of Constructional Steel Research,* 1 (46): 162.

Wald, F., L. Šabatka, M. Bajer, P. Jehlička, J. Kabeláč, M. Kožich, M. Kuříková, and M. Vild. 2020. *Component–Based Finite Element Design of Steel Connections*. Czech Technical University in Prague.

Willibald, S., J. A. Packer, and G. Martinez-Saucedo. 2006. “Behaviour of gusset plate connections to ends of round and elliptical hollow structural section members.” *Can. J. Civ. Eng.,* 33 (4): 373–383. https://doi.org/10.1139/l05-052.

Yeomans. 1993. *Slotted End Plate Connection in CHS and RHS*. TS and MD Technical Development Report. P004.S.08-1. Corby, England: British Steel.

Zhao, R. G., R. F. Huang, H. A. Khoo, and J. J. R. Cheng. 2008. “Experimental study on slotted rectangular and square hollow structural section (HSS) tension connections.” *Canadian Journal of Civil Engineering*, 35 (11): 1318–1330. https://doi.org/10.1139/L08-069.

Zhu, H. T., M. C. H. Yam, A. C. C. Lam, and V. P. Iu. 2009. “The shear lag effects on welded steel single angle tension members.” *Journal of Constructional Steel Research*, 65 (5): 1171–1186. https://doi.org/10.1016/j.jcsr.2008.10.004.