Rotational Ductility of Extended Shear Tab Connections

BY

Tayseer Tasbahji

B.S. Middle East Technical University, 2016

THESIS

Submitted as partial fulfillment of the Requirements
For the Degree of Master of Science in Civil Engineering
In the Graduate College of the
University of Illinois at Chicago, 2018
Chicago, Illinois

Defense Committee:

Dr. Mustafa Mahamid, Advisor, Civil and Materials Engineering
Dr. Micheal McNallan, Chair, Civil and Materials Engineering
Dr. Eduard Karpov, Civil and Materials Engineering
ACKNOWLEDGEMENTS

I would like to especially show my gratitude to my parents, who never stopped motivating and supporting me to pursue my dreams.

I would also like to express the deepest appreciation to my advisor, Dr. Mustafa Mahamid, for his outstanding support and guidance. He is an icon of leadership and intellect and without his persistent help and supervision, this research would have not been possible.
# TABLE OF CONTENTS

ACKNOWLEDGEMENTS iv

LIST OF TABLES viii

LIST OF FIGURES ix

LIST OF SYMBOLS AND ABBREVIATIONS xii

SUMMARY xiii

CHAPTER 1. Introduction 1
  1.1 Overview 1
  1.2 Research Significance and Scope 2
  1.3 Literature Review 3

CHAPTER 2. Modeling 13
  2.1 Element Selection 13
  2.2 Material Properties 13
    2.2.1 Bolts Material Properties 13
    2.2.2 Material Properties of Shear Tabs, Beams and Columns 14
  2.3 Contact Properties 15
    2.3.1 Surface-to-Surface Interaction 16
    2.3.2 Tie Constraint 16
  2.4 Boundary Conditions 17
    2.4.1 Pin Supports 17
    2.4.2 Roller Supports 17
    2.4.3 Lateral Bracing 17
  2.5 Loading 18
    2.5.1 Bolt Pre-tensioning Force 18
    2.5.2 External Beam Force 19
  2.6 Loading Steps 19
  2.7 Meshing 20
  2.8 Solution 21

CHAPTER 3. finite element models validation 23
  3.1 Metzger Experiments Models 23
    3.1.1 Connections with Rigid Supports 23
  3.2 Sherman-Ghorbanpoor Experiments Models 27
    3.2.1 Unstiffened Connections with Flexible Supports 27
    3.2.2 Stiffened Connections with Flexible Supports 32
  3.3 Validation Conclusion 35

CHAPTER 4. Finite element Analysis Results and discussion 38
  4.1 General Behavior of Shear Tab Connections 38
  4.2 Models Analysis Results and Observations 39
| 4.2.1  | Extended Shear Tab Connections with Rigid Supports | 39 |
| 4.2.2  | Conventional Shear Tab Connections with Flexible Supports | 43 |
| 4.2.3  | Extended Shear Tab Connections with Flexible Supports | 45 |
| 4.2.4  | Extended Shear Tab Connections with Multiple Bolt Columns with Flexible Support | 45 |
| 4.3    | Parametric Study | 47 |
| 4.3.1  | a-distance | 48 |
| 4.3.2  | Plate Thickness | 49 |
| 4.3.3  | Skewed Angle | 51 |
| 4.4    | Results Discussion | 52 |
| 4.4.1  | Comparison Between Rigid and Flexible Supports | 52 |
| 4.4.2  | Difference Between Conventional and Extended Configurations | 54 |
| 4.4.3  | Effect of Double and Triple Vertical Line of Bolts | 55 |
| 4.4.4  | Stiffeners Influence on the Connection Rotational Behavior | 56 |

CHAPTER 5. Design Procedure and Results Discussion | 59 |
5.1 Design Procedure | 59 |
| 5.1.1 | Lead-in | 59 |
| 5.1.2 | Conventional Configuration Design Requirements | 59 |
| 5.1.3 | Extended Configuration Design Procedure | 63 |
| 5.2 | Conclusions and Recommendations | 69 |
| 5.3 | Future Work | 73 |

APPENDIX A. Snapshots of Connections Deformed Shapes | 74 |
| A.1 | Extended Shear Tab Connection with Rigid Supports (Metzger) | 75 |
| | Metzger Test No. 5 | 75 |
| | Metzger Test No. 7 | 76 |
| | Metzger Test No. 8 | 77 |
| A.2 | Extended Shear Tab Connection with Flexible Supports | 78 |
| | Three Bolted Connections | 78 |
| | Five Bolted Connections | 79 |
| | Eight Bolted Connections | 80 |
| A.3 | Conventional Shear Tab Connection with Flexible Supports | 81 |
| | Three Bolted Connections | 81 |
| | Five Bolted Connections | 82 |
| | Eight Bolted Connections | 83 |
| A.4 | Extended Shear Tab Connection with Flexible Supports with Multiple Bolt Columns | 84 |
| | Three Bolted Connections with Double Bolts Columns | 84 |
| | Three Bolted Connections with Triple Bolts Columns | 85 |
| | Five Bolted Connections with Double Bolts Columns | 86 |
| | Five Bolted Connections with Triple Bolts Columns | 87 |
| | Eight Bolted Connections with Double Bolts Columns | 88 |
| | Eight Bolted Connections with Triple Bolts Columns | 89 |

APPENDIX B. shear vs. rotation Curves | 90 |
| B.1 | Extended Shear Tab Connection with Flexible Supports | 90 |
B.1.1 Three Bolted Connections 90
B.1.2 Five Bolted Connections 90
B.1.3 Eight Bolted Connections 91

B.2 Conventional Shear Tab Connection with Flexible Supports 91
B.2.1 Three Bolted Connections 91
B.2.2 Five Bolted Connections 92
B.2.3 Eight Bolted Connections 92

B.3 Extended Shear Tab Connection with Flexible Supports with Multiple Bolt Columns 93
B.3.1 Three Bolted Connections 93
B.3.2 Five Bolted Connections 93
B.3.3 Eight Bolted Connections 94

B.4 Parameters Effects on Extended Shear Tab Connections with Flexible Supports 94
B.4.1 Three Bolted Connections 94
B.4.2 Five Bolted Connections 96
B.4.3 Eight Bolted Connections 97

REFERENCES 99
LIST OF TABLES

Table 2-1  – Material properties of tested experiments members 4
Table 3-1  – Shear strength and failure modes - Metzger Tests 25
Table 3-2  – Shear strength and failure modes - Sherman&Ghorbanpoor Tests 25
Table 4-1  – Metzger experimental tests members dimensions 29
Table 4-2  – Connection parameters studied 36
LIST OF FIGURES

Figure 2-1  – C3D8R element. 2
Figure 2-2  – Stress – Strain Behavior of Bolts and Rivets (Kulak 2001). 3
Figure 2-3  – Surface interactions between the bolts and other members. 5
Figure 2-4  – Plate to supporting member tie constraint. 5
Figure 2-5  – Pin supports locations. 6
Figure 2-6  – Roller supports and lateral bracings locations. 7
Figure 2-7  – Bolt pre-tensioning and applied external load. 8
Figure 2-8  – Mesh size. 9
Figure 2-9  – Meshing of Metzger connections models. 10
Figure 2-10 – Meshing of Sherman-Ghorbanpoor stiffened connections models. 11
Figure 2-11 – Meshing of Sherman-Ghorbanpoor unstiffened connections models. 11
Figure 3-1  – Shear-Beam end rotation curves for Test 5. 13
Figure 3-2  – Shear-Beam end rotation curves for Test 7. 13
Figure 3-3  – Shear-Beam end rotation curves for Test 8. 14
Figure 3-4  – Plastic hinge at the beam midspan. 14
Figure 3-5  – Bolt bearing at beam web. 15
Figure 3-6  – Web mechanism. 15
Figure 3-7  – Shear tab buckling. 16
Figure 3-8  – Shear-displacement curves for test 3U. 17
Figure 3-9  – Shear-displacement curves for test 4U. 18
Figure 3-10 – Shear-displacement curves for test 6U. 18
Figure 3-11 – Plate Twist.
Figure 3-12 – Web Mechanism.
Figure 3-13 – Plate bolt hole bearing.
Figure 3-14 – Bolt shear failure.
Figure 3-15 – Shear-displacement curves for test 3A.
Figure 3-16 – Shear-displacement curves for test 3B.
Figure 3-17 – Shear-displacement curves for test 4A.
Figure 3-18 – Shear-displacement curves for test 3D.
Figure 3-19 – Shear-displacement curves for test 6D.
Figure 3-20 – Shear-displacement curves for test 4B.
Figure 3-21 – Shear tab buckling.
Figure 4-1 – Shear-Beam end rotation curve for test 7.
Figure 4-2 – Plate buckling.
Figure 4-3 – Beam web buckling and web bolt hole bearing.
Figure 4-4 – Shear-Beam end rotation curve for test 5.
Figure 4-5 – Shear-Beam end rotation curve for test 8.
Figure 4-6 – General beam vertical deflection in test 5.
Figure 4-7 – Bolt group rotation relative to original position.
Figure 4-8 – Shear-Rotation plot for 3 bolted conventional flexible connection.
Figure 4-9 – Bolts shear plastic deformation.
Figure 4-10 – Plate deformation at first line of bolts.
Figure 4-11 – Shear-Rotation plots for five bolted flexible connections with multiple bolt columns.
Figure 4-12 – Bolt group rotational behavior relative to original position.
Figure 4-13 – a-distance effect of the rotational behavior of three bolted connection.

Figure 4-14 – Failure behavior for reduced tab thickness.

Figure 4-15 – Effect of plate thickness on the rotational behavior of five bolted flexible connection.

Figure 4-16 – Failure mechanism for 0.25" and 0.375" tab thickness.

Figure 4-17 – Rotational behavior of three bolted connection with varying connection orientations.

Figure 4-18 – Comparative behavior of the bolt behavior in extended and conventional shear tabs connections.
LIST OF SYMBOLS AND ABBREVIATIONS

E  Modulus of Elasticity (Young’s Modulus)

ν  Poisson’s Ratio

σ_u  Ultimate Strength

σ_y  Yield Strength
SUMMARY

By AISC Specifications, simple shear connections of beams shall be designed to resist the required shear strength and allow the connected member to accommodate flexible end beam rotation of simple beams. Single plate shear tab connections should be configured so the failure occurs by plate yielding along with bolt group rotation and bolt holes elongation, to avoid weld sudden rupture and bolts failure. The AISC specifications requires an end beam rotation of 0.03 radians with some limiting inelastic deformation in the connection is allowed to accommodate this requirement. Conventional geometry and plate thickness limitations are assigned for shear tab connections to satisfy the ductility requirements. However, in cases where connections cannot satisfy conventional dimensional limitations, extended tab configurations are more general and allow greater geometric variations. Extended shear tab connections in many instances offer practical solutions to the construction process, e.g. by augmenting a clear distance of the support, coping of the supported member is not required any more. Such case includes welding the tab to columns webs.

The use of extended tab connections in this case comes to a position of concern when the connection flexibility is not satisfied. Higher rigidity of the connection could result in an unanticipated moment development that the column has not been designed to resist or causes sudden rupture of the bolts or weld. Under other condition for unstiffened tabs welded to column webs (flexible support) substantial distortions of column web could result in yield line mechanism failure. This excess deformation in the column web allows
an unlimiting connection rotation around the weld centerline, thus limiting the end beam rotation.

To address these concerns a parametric study is performed. Plate thickness, distance between bolt line and weld line, skewed angle, number of bolts and number of bolt columns are the variable parameters considered. The parameters were evaluated in terms of shear strength, corresponding beam end rotation, support web rotation, unanticipated moment and connection failure mechanism. This study helps analyze the effects of parameter variation on the rotational ductility of the connection, define parameter ranges, specify design constraints and nominate parameters for further evaluation. Stiffening the column web to limit its rotation is also studied as a practical solution to the problem.
INTRODUCTION

1.1 Overview

Steel frames are composed of beams and columns that are connected using bolted or welded connections designed to provide a safe structure. The connections are of three types, termed as rigid, semi-rigid or simple connections. The last type, simple connections, are designed to transmit shear force only and restrict moment development in the joint. Single plate shear tab connections are one of the most frequently used type of these connections. This type of assembly is composed of a plate fillet welded to the supporting member and bolted to the supported members web.

The design of shear tab connections is separated to conventional and extended configuration; the conventional configuration contains a single vertical row of bolts only with the number of bolts ranging between 2 and 12, and the distance from the bolt line to the weld line (a-distance) must be equal to or less than 3.5 inches. Any deviation from these limitations falls into what is called an extended configuration.

The supporting member plays an important role in controlling the actual behavior of the connection. It can be classified to a rigid support or a flexible support. This classification is based on the capability of the connection to follow the rotation of the supported member. A rigid support restrains the connection from following the rotation of the supported member, while on the other hand, a flexible support will provide lower stiffness and allow more rotation in comparison to the rigid support. Column web is one example of flexible supports.
1.2 Research Significance and Scope

The AISC specifications requires an end beam rotation of 0.03 radians with some limiting inelastic deformation in the connection is allowed to resemble a simple connection. Conventional geometry and plate thickness limitations are assigned for shear tab connections to satisfy the ductility requirements. However, in cases where connections cannot satisfy conventional dimensional limitations, extended tab configurations are more general and allow greater geometric variations. Extended shear tab connections in many instances offer practical solutions to the construction process, e.g. by augmenting a clear distance of the support, coping of the supported member is not required any more. Such case includes welding the tab to columns webs.

The use of extended tab connections in this case comes to a position of concern when the connection flexibility is not satisfied. Higher rigidity of the connection could result in an unanticipated moment development that the column has not been designed to resist or causes sudden rupture of the bolts or weld. Under other conditions for unstiffened tabs welded to column webs (flexible supports) substantial distortions of column web could result in yield line mechanism failure. This excess deformation in the column web allows an unlimiting connection rotation around the weld centerline, thus limiting the end beam rotation.

Through studying more than 80 finite element models, this research aims to:

1. Study the effect of the support stiffness on the rotational behavior of extended shear tab connections.
2. Analyze the effects of different plate parameters on the rotational ductility of the connection.

3. Determine the effect of stiffening column webs on limiting the connection rotation.

4. Provide some design recommendations to take in consideration for satisfying the rotational ductility requirement of extended shear tab connection.

1.3 Literature Review

Numerous studies were done to study the behavior of shear tab connections and to develop a simple design procedure. The following section provides concise summaries of the past research done.

Richard et al. (1980-1982) studied the behavior of the single plate connections with A325 and A490 bolt types. First, single bolt-single shear tests with various dimensional parameters were executed. From this the author concluded that bolt shear and tension tearing should be avoided as a failure mode in order for the connection to provide sufficient ductility. Numerical finite element models were prepared and analyzed to obtain moment-rotation curves of this framing connection. Based on the data, it was concluded that the curves are dependent on the shear when the eccentricity is less than the bolt group height and independent on shear in the opposite case. At the end of this study, the authors proposed a design procedure for the single plate framing connections with ASTM A325 and ASTM A490 bolts based on the numerical and experimental results obtained from his study. In 1982, the author studied the connections with A307 bolts. Single bolt shear tests and full-scale experimental tests were executed. The author proposed a detailed procedure to design
single plate framing connections using A307 type of bolts and recommended to use slotted holes for this type of bolts.

Cheng et al. (1984) studied the behavior of coped beams with various parameters (e.g. coping span and depth) using finite element analysis. The failure mechanisms observed were local web buckling and lateral torsional buckling of the coped region. Later, the author developed a design check equation to circumvent these failure modes and validated the equations with experimental work.

Astaneh et al. (1989-1993) investigated the behavior of single plate shear connections. The authors proposed that shear connections should be designed for shear strength and rotational flexibility. Several experiments were conducted to indicate the limit states of this type of connections. Bolt type (A325-N or A490-N), beam steel grade (A36 or grade 50) and edge distance were the investigated variable parameters. As expected, it was revealed that bolts grade A490 showed higher strength resistance and influenced the failure mode of the connection. Although the stiffer bolts failed simultaneously in shear with weld fracture, bolts grade A325 had failed due to sudden shear fracture after developing significant permanent. Further, it was determined that lesser number of bolts in a connection exhibits larger rotational ductility and developed smaller moment. Based on his observations, the author concluded five limit states associated with single plate connections, being plate yielding, bolt fracture, bearing failure of bolt holes, fracture of plate net section and weld fracture. Lastly, the author proposed a design procedure based on the obtained results. In 1993, the author showed that shear connections should have sufficient rotational ductility, in addition to its adequate shear capacity, to accommodate simple beam end rotation and prevent moment development in the connection. The authors
studied the end support shear-rotation relationship for simply supported beam and proposed a trilinear curve to present the actual rotational behavior based on the span-to-depth ratio and the beam shape factor. The rotation-shear curve included the elastic, inelastic and full plastic-hinge formation behaviors. Later, the author executed experimental tests based on the proposed behavior and developed a design procedure considering all limit states, that the ductile limit states precede the brittle limit states to allow the connection to exhibit adequate rotational ductility. This proposed design procedure was later adapted by the American Institute of Steel Construction.

Sherman and Ghorbanpoor (2002) studied the behavior of extended shear tab connections connected to flexible supports. Between three faces and total of 31 experiments the extended shear tabs were either welded to column web or girder web. The capacity of the connections was studied as a function of width-to thickness ratio of the supporting member web, dimensions of the shear tab, type of bolt holes, number of bolts and span-to-depth ratio of the supported beam. Later the researches investigated the effect of using snug tightened bolts in short slotted holes, the effect of using one stiffener plate instead of two and the behavior of deep plates with up to eight bolts. The authors observed that shear tabs twist in the majority of the unstiffened tests with the stiffeners thickness not improving the performance of the stiffened connection. Thus, they recommended the use of stiffeners with a similar thickness as that of the plate. In addition, the authors identified web mechanism of the supporting member web as a new limit state for flexible extended shear tab connections. Finally, the author proposed a new design procedure for the extended shear tab connections. This study formed the basis of the AISC manual 13th
edition (AISC 2005) design procedure for the extended configuration of the simple plate connections.

Ashakul (2004) used finite element analysis to investigate several parameters including a-distance, plate thickness, plate steel grade and the plate position relative to the beam neutral axis and their effect of the bolt shear rupture. The author concluded that significant horizontal forces could concentrate on the bolt group if the plate materials and thicknesses did not satisfy ductility criteria, which will lead to reduction in the bolt group shear strength. Also, the bolt group was found to be independent on the a-distance. The author proposed a relationship to calculate plate shear yielding strength based on shear stress distribution along the tab.

Creech (2005) performed full scale experimental tests on shear tab connections, studying bolt holes types, standard and short-slotted, and the support type, flexible and rigid, in addition to slab effect on the behavior of the connection. The researcher concluded that:

1. The AISC design procedure for single-plate shear connections is overly conservative.
2. The eccentricity should be considered in the design procedure for two and three bolted connections.
3. Plates in this type of connections should satisfy the thickness requirements specified in AISC Manual (AISC, 2001) and should be manufactured of lower steel grade.
4. The type of hole has no significant effect on ultimate capacity of the connection.
5. The slab restrains and affect the connection rotational behavior.
**Goodrich (2005)** conducted experimental and numerical tests on extended shear tab connections. The author concluded from the experimental results that AISC manual (AISC, 2001) design procedure is conservative and the connections can carry twice the design load. The main failure mode observed was tab buckling. The author mentioned that the finite element analysis executed was dependent on many assumptions and the results are not reliable.

**Metzger (2006)** studied experimentally conventional and extended shear tab connections framed to rigid support in Virginia Tech. Through a comparative study, Metzger in her thesis concluded that the AISC 13th Edition (AISC, 2005) design procedure is conservative in predicting the ultimate strength capacity. Two experimental tests failed in weld fracture, however, the author indicated that the failure could be due to fabrication issues.

**Rahman et al. (2007)** presented a three dimensional model to study the behavior of the unstiffened extended shear tab connections and validate the experimental results done by Sherman 13 and Ghorbanpoor (2002). The study focused on two configurations: three bolts unstiffened beam to column web configuration, and five bolts unstiffened beam-to-column web configuration. The model was created using the finite element software ANSYS. Four element types were used in the model: Eight node brick element to model the shear tab, supporting member and the beam. While ten node tetrahedral element was used to model the bolts. Both elements have the ability to track the elastic and plastic behavior and appropriate for modeling the steel members. Additionally, the pretensioning forces in bolts were simulated using pretensioning elements, these elements were defined on the pretensioning section which divides each bolt into two parts. Finally, contact surfaces were created in order to transfer forces from the beam web to bolts and from bolts to the shear
tab using contact elements. Three load steps were used: Applying the pretensioning forces in bolts, transformation of pretensioning stresses into strain, and applying the external load on the beam. The bolt material used was ASTM A325-X, ASTM A36 used for the shear tab material, and finally, ASTM A572 Gr. 50 used for the column and beam material. The weld lines between the shear tab and the column web were modeled as continuous since the weld failure was not a critical failure mode. The author did not consider the shear-twist curves in the finite element analysis to determine the capacity of the connection since only one of the experimental tests has the twist as primary failure mode. The connection capacity was determined using the shear-displacement curves. These curves were obtained from FEA showed good agreement with the results obtained from the experiments. In order to determine ultimate shear capacities, yield points and failure modes of the connections, shear-displacement; shear-twist; and shear-rotation curves were investigated. It was observed that failure modes obtained from the FEA and experiments were the same. Rahman et al. concluded that the presented model in this study is a powerful tool in addressing the failure of the unstiffened extended shear tab connection in the plastic region.

Mahamid et al. (2007) addressed in detail the failure modes and analyses of the stiffened extended shear tab connection. Finite element models were created using ANSYS, these models were compared and verified with experimental study done by Sherman and Ghorbanpoor (2002). The author created models for eight different connection configurations: beam-to-column connections (2, 8, 10 and 12 bolts), and beam-to-girder connections (3, 6, 10 and 12 bolts). The author used four element types to model the connections: Pretensioning elements to model the bolt’s pretensioning force, contact elements with surface-to-surface and flexible-to-flexible interaction properties to define
the interaction between bolts-beam web holes; bolts-shear tab holes; and beam web-shear tab, eight node brick elements to model the beams; columns; girders; and shear tabs, three dimensional tetrahedral elements to model the bolts. Material properties of ASTM A36 was used for shear tabs, ASTM A572 Grade 50 for beams; columns; and girders, and A325-X for bolts. Additionally, bilinear curves were used to simulate the stress-strain responses by identifying the experimental results of yield strength, ultimate strength, and elongation.

The twist of the connection plate, vertical displacement along the connection bolt line, shear load eccentricity relative to the connection bolt line, failure modes and nonlinearity were studied for each model so as to analyze the behavior of the stiffened extended shear tab connections. Moreover, the author observed good agreement between the finite element and experimental results, nevertheless, the zero strain locations were slightly off because of the use of the linear regression approach which is considered as approximate but acceptable method. Also, it was observed that the connection response becomes close to the rigid connections behavior as the number of bolts increases. Additionally, the twist was identified as secondary failure mode and detected at the bottom of the shear tab for the case where the beam framed into girder because of the fact that the tab is stiffened by the girder’s top flange and not stiffened to the bottom flange. 15 Twist was also detected along the shear tab for the case of deep connections when the beam is framed into column. The author considered the nonlinearity as a crucial aspect in the extended shear tab connections’ behavior since it helps in identifying the failure modes of these connections. Nonlinearity was implemented in the models using several parameters: the use of contact elements, material properties nonlinearity, and geometric nonlinearity. The author studied the nonlinear behavior by monitoring the plastic deformation and stresses at different locations.
in the connections, also by analyzing the shear-displacement curves. The author observed five different failure modes in these connections: bolt shear, shear yielding of the plate, bolt bearing, twist in the shear tab, and web mechanism in the girder web. Mahamid et al. concluded that his model is accurate and unique in examining the behavior of the stiffened extended shear tab connections.

**Muir and Hewitt (2009)** investigated the behavior of unstiffened extended shear tab connections connected. The authors concluded that these connections generate additional moment on the supporting member that should not be neglected in the supporting member design. The authors found that plate buckling was not a primary failure mode and highlighted the plate behavior to act as fuse by yielding before bolts or weld failure. At last, the authors proposed a general design procedure for extended tab shear connections.

**Marosi (2011)** studied single and double vertical bolt rows shear tab connections with up to ten bolts per column to update the Canadian Institute of Steel Construction (CISC) handbook, that its design procedure is limited to one column of bolts with up to seven bolts. Experimental tests with one and two vertical rows of bolts were done with the number of bolts ranging between three to ten bolts per row. Also, the study included retrofit weld tests.

In addition to proposing a new design procedure, the author concluded that the weld retrofit connections reached the target rotations and shear capacity of their corresponding bolted connections. The author observed that weld retrofit connections behaved similarly to bolted connections for single bolt column tests but outperformed their bolted counterparts double bolt columns.
Muir and Thornton (2011) revised design procedure for single-plate shear connections outlined in the 14th edition of the AISC Manual (AISC, 2010). The author showed that in previous editions an additional factor of safety was specified for all bolted connections designs to account for uneven force distribution between the bolts by reducing their nominal strength by 20%. The authors indicated that this design procedure in the AISC Manual 13th edition (AISC, 2010) need to be re-evaluated since this reduction is no longer appropriate to neglect the connection eccentricity because of the increase in the nominal bolt shear values provided in AISC’s specification (AISC, 2010).

Yim and Krauthammer (2012) presented mechanical models to define structural properties shear tab connections. The authors reached their result through finite element analysis and studies of single plate shear connections subjected to monotonic, cyclic, and blast loadings. The authors approved that the proposed models are applicable to use for frame analyses and can accurately predict the characterization curves for the connections with low cost and short time. However, they noted that the models are dependent on detailed connection geometrical and material properties of the components and the connections configuration.

Suleiman et al. (2013) used numerical simulation software ABAQUS to investigate plate twisting of extended shear tab connections. The main focus of the study was the effect of concrete slab on changing the behavior of flexible extended tab connections. In the tests, the lateral translation movement of the supported beams compression flanges was restricted to simulate the effect of the floor slab. The authors concluded that plate twist is significantly reduced with the existence of slab and it could be neglected as a failure mode. Nevertheless, the connection capacity wasn’t affected by the restriction of the slab.
Abou-zidan and Liu (2015) investigated the behavior of the unstiffened extended shear tab connections through finite element analysis. The authors studied different parameters related to bolts including number of bolts, and distance from weld line to bolt line center. In addition, plate thickness and the supporting member web slenderness ratio were also studied. The authors concluded that for connections with 2 to 6 bolts the current design procedure is very conservative in predicting bolt shear fracture strength but gives reasonable results otherwise.

Al-Hijaj and Mahamid (2016) studied the torsional behavior of skewed extended shear tab connections in a series of research phases. With more than 300 models the authors studied the connections with rigid and flexible supports. The authors determined from the analysis results that the supporting member contributes in resisting the additional torsional at the elastic level only, and the contribution reduces significantly as the shear force increases and becomes neglected at the plastic level. In addition, the effect of the connection orientation was found to be insignificant on the torsional and bending behavior of these connections. Later, the authors studied several plate dimensions and their effect on the behavior and shear capacity of skewed extended shear tab connections. These parameters included plate thickness, a-distance, skewed angle and number of bolts. From these results, the authors proposed a modification to the design procedure for skewed configuration of extended shear tab connections.
MODELING

To proceed with case studies and behavioral simulation, numerical analysis is to be operated on three-dimensional finite element models. In this research, the finite element software package ABAQUS CAE version 6.13 was found to be a best suit to serve the purpose of this study. In this section, the details of the modeling process are introduced, including models’ properties and characteristics from computational simulation standpoint.

The initial shear tab connection models share similar characteristics to those previously tested in the experiments by Sherman-Ghorbanpoor (2002) and Metzger (2006). In the following chapter, the results obtained from the finite element analysis are later compared with the reported experimental results for validation.

1.4 Element Selection

The element type selected for the entire model is a three-dimensional eight node brick element with reduced integration, referred to as C3D8R in ABAQUS library. Each node has three translational degrees of freedom, while the integration points were compacted to one point at the center of the element as shown in Figure 2-1. The integration point serves in preventing elements locking at boundaries, therefore better tolerance to distortion is left to allow the model to capture any inelastic behavior.

1.5 Material Properties

1.5.1 Bolts Material Properties
As in the experiments executed by Sherman-Ghorpanboor (2002) and Metzger (2006), bolts (A325-X) were used for the finite element models. An elastic perfectly-plastic constitutive relationship was used. The modulus of elasticity (E), Poisson’s ratio (ν) and ultimate strength (σ_u) used are 29,000 ksi, 0.3 and 120 ksi, respectively. The ultimate strength (σ_u) for A325-X bolts used was taken from a report by Kulak (2001) that studied the mechanical properties of different bolts and rivets types. Figure 2-2 shows coupon stress versus strain relationships for different fastener materials done by Kulak (2001).

![Stress-Strain Behavior of Bolts and Rivets (Kulak 2001)](image)

**Figure 0-2 Stress – Strain Behavior of Bolts and Rivets (Kulak 2001)**

1.5.2 **Material Properties of Shear Tabs, Beams and Columns**

To simulate the materials nonlinearity, an elastoplastic bilinear model with linear hardening was used for the model major member components. The material properties used for models were the same as those obtained by Sherman-Ghorbanpoor (2002) and Metzger (2006) experimentally. Refer to Table 2-1 for the numerical values of the properties.
Table 0-1 Material Properties of Tested Experiments Members

<table>
<thead>
<tr>
<th>Member</th>
<th>$E$ (ksi)</th>
<th>$\nu$</th>
<th>$\sigma_y$ (ksi)</th>
<th>$\sigma_u$ (ksi)</th>
<th>% Elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sherman &amp; Ghorbanpoor</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/8 in. Plate</td>
<td>29,000</td>
<td>0.26</td>
<td>42.6</td>
<td>66.5</td>
<td>34</td>
</tr>
<tr>
<td>½ in. Plate</td>
<td>29,000</td>
<td>0.26</td>
<td>40.5</td>
<td>63.6</td>
<td>36</td>
</tr>
<tr>
<td>W14x53</td>
<td>29,000</td>
<td>0.30</td>
<td>54.2</td>
<td>70.8</td>
<td>38</td>
</tr>
<tr>
<td>W24x55</td>
<td>29,000</td>
<td>0.30</td>
<td>55.1</td>
<td>70.1</td>
<td>38</td>
</tr>
<tr>
<td>W8x31</td>
<td>29,000</td>
<td>0.30</td>
<td>55.2</td>
<td>75.3</td>
<td>31</td>
</tr>
<tr>
<td>W14x90</td>
<td>29,000</td>
<td>0.30</td>
<td>56.7</td>
<td>71.7</td>
<td>37</td>
</tr>
<tr>
<td>Metzger</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/8 in. Plate</td>
<td>29,000</td>
<td>0.26</td>
<td>69.3</td>
<td>96.3</td>
<td>20</td>
</tr>
<tr>
<td>½ in. Plate</td>
<td>29,000</td>
<td>0.26</td>
<td>68.2</td>
<td>97.7</td>
<td>22</td>
</tr>
<tr>
<td>W18x55</td>
<td>29,000</td>
<td>0.30</td>
<td>58.9</td>
<td>77.6</td>
<td>27</td>
</tr>
<tr>
<td>W24x62</td>
<td>29,000</td>
<td>0.30</td>
<td>58.0</td>
<td>77.1</td>
<td>27</td>
</tr>
<tr>
<td>W30x108</td>
<td>29,000</td>
<td>0.30</td>
<td>61.5</td>
<td>79.3</td>
<td>31</td>
</tr>
</tbody>
</table>

1.6 Contact Properties

The interaction between the model parts are defined in two different contact types. Using the right contact type plays a considerable vital role in the solution of the problem.
and defining the right properties leads to the right convergence of results. The contact types used in the models are defined in this section.

1.6.1 Surface-to-Surface Interaction

This type of interaction is used between the bolts and the other parts of the model. This hard contact allows the separation of the contacting surfaces to simulate the real behaviour of the model and not restrain the parts movement. A friction coefficient of 0.03 is used between all steel-on-steel surfaces. Figure 2-3 shows the bolt head, shank and nut defined as master surfaces, while slave surfaces were assigned to the beam and plate since the bolts are stiffer.

1.6.2 Tie Constraint

This contact type was used to simulate welding between the tab and the supporting member. Here, the supporting member was considered as a master surface, while the slave surface was assigned to the plate side. For that, the degrees of freedom of slave surface nodes will follow the degrees of freedom of the master surface nodes. Figure 2-4 demonstrates the tie constraint defined in a model.
1.7 Boundary Conditions

To achieve accurate results, the boundary conditions are applied as used experimentally in the tests done by Sherman-Ghorbanpoor (2002) and Metzger (2006). Three types of boundary conditions were applied: Pin supports, roller supports and lateral bracings.

1.7.1 Pin Supports

For pin supports the translational degrees of freedom in x, y and z directions are restrained. Figure 2-5 shows the supports location for the connections supporting members.

1.7.2 Roller Supports

Roller supports are assigned to allow free translational motion in one direction. In this case, the lateral movement in x direction was allowed at the far end of the beam to simulate the behaviour of simply supported beam. Figure 2-6 shows the beam bottom flange restrained nodes.

1.7.3 Lateral Bracing

To simulate the behaviour of lateral bracing, constraints for the lateral movement in z direction was applied along the beam flanges. This lateral DOF restrain is used to restrain lateral torsional buckling of the beam as applied by Sherman-Ghorpanboor (2002) and
Metzger (2006) in their experimental work. Figure 2-6 shows the location of the lateral bracing applied on one of the models.

![Figure 0-6 Roller supports and lateral bracings locations](image)

1.8 Loading

In addition to supports and bracings, force loading is another boundary condition to be defined for the completion of the modelling process. As shown in Figure 2-7, two types of loading were applied: bolt pre-tensioning and external beam load.

1.8.1 Bolt Pre-tensioning Force

Bolt pre-tensioning was applied at each bolt using bolt force option. This load is important to keep the beam web and plate in contact under the applied external loads. The minimum bolt pretensioning force from the AISC manual is 28 kips for ¾ in. diameter
A325 bolts (group A) which is the same value that was used in the experiments done by Sherman-Ghorpanboor (2002) and Metzger (2006).

1.8.2 **External Beam Force**

The external forces applied on the connection using actuators were applied to the models as equivalent pressure load on the top flange at the same loading point as the experiments. The location of load application was selected to produce the same reaction and rotation at the beam ends as if the beam was uniformly loaded, which represents the actual loading case in beams (Sherman-Ghorpanboor, 2002).

![Bolt Pre-tensioning Force](image1.png)  ![Applied External Load](image2.png)

**Figure 0-7 Bolt pre-tensioning and applied external load**

1.9 **Loading Steps**

The boundary conditions were applied in three consecutive time steps. The initial time step used is to define the boundary conditions in the model, next, bolt pre-tensioning force was applied to each bolt to initiate the contact between the beam web and plate. Finally, the external hydraulic load is applied after establishing the contact surfaces and boundary conditions in the models. ABAQUS predefines the initial time step, while the
next steps were assigned as general static. One second initial time with 0.01 and 0.1 seconds as minimum and maximum increment size were assigned respectively.

1.10 Meshing

All parts were chosen as dependent parts in order to perform meshing at a part level. First, the model parts were partitioned to separate parts to discretize the model. Then, fine mesh was used at regions with higher interest and that are expected to have a high stress level. These regions include the plate, contact area between the plate and supporting member, and contact area between bolts and beam web and bolts and plate. Coarse mesh was used at regions with lower deflections and interest such as beam far end, the top and bottom of columns, and ends of girders. Figures 2-8 to 2-11 illustrates mesh sizes used in the model. An ABAQUS built in option called “Verifying Mesh” was used to detect poorly meshed parts with poor aspect ratios. All model passed the check with no errors.

![Figure 0-8 Mesh size](image)
1.11 Solution

The analysis of the models reaches the desired solution when the run terminates due to excessive distortion to be present in elements with high strains. This occurs when the strain increment has exceeded fifty times the strain to cause first yield, or when excessive distortion at some integration points in solid (continuum) elements. The unloading path wasn’t considered; as investigating the post failure stage will increase significantly the running time of the models without serving the objective of this study. In addition, the model’s behavior plots were very comparable to those reached during the experiments.

Figure 0-9 Meshing of Metzger connections models
Figure 0-11 Meshing of Sherman-Ghorbanpoor unstiffened connections models

Figure 0-10 Meshing of Sherman-Ghorbanpoor stiffened connections models
FINITE ELEMENT MODELS VALIDATION

The finite element models must be validated with experimental results to confirm their accuracy. Two experimental research works by Sherman-Ghorbanpoor and Metzger (2006) were used to compare the connection behavior.

1.12 Metzger Experiments Models

1.12.1 Connections with Rigid Supports

In Metzger models the shear tab was welded to the flange of the supporting column; rigid support. The shear-beam end rotation curves were used to compare the results between the finite element models and the experiments, then the failure mechanism was compared to assure the validity of the models. Table 3-1, at the end of the chapter, shows a comparison between the ultimate shear forces and failure modes obtained. Secondary failure modes are kept between parentheses.

1.12.1.1 Shear-Rotation Curves

To assure a fair comparison, the rotation measurements were taken following the same method as that done by Metzger. In the experiments, the beam end rotation was measured using two linear potentiometers, the first placed over the center of the bolt group, while the second potentiometer was placed 6 inches apart from the first toward the opposite end of the beam. As shown in Figures 3-1 to 3-3, there is good agreement between the experimental results and FEA results. However, due to the differences in applying lateral bracings to the beam between FEA and experimentally, the models showed a stiffer
behavior than the actual experiment. Additionally, this difference could also be attributed to the column’s reduced capacity, as the same column was used for several experiments; based on the column deflected shape, the FEA shows that there is an expected local buckling in the column’s web and flange. Thus, due to a better control over the finite element models, no initial stresses and possible plastic deformations existed in the columns. Nevertheless, the error did not exceed 10% in all three models.

**Figure 0-1 Shear-Beam end rotation curves for Test 5**

**Figure 0-2 Shear-Beam end rotation curves for Test 7**
1.12.1.2 Failure Modes

Figures 3-4 to 3-7 show the failure modes obtained from the finite element models and experiments for Metzger connection. As could be observed, similar primary modes for all three tests were obtained. In addition, secondary failure modes were also detected, like plate twisting in extended configurations and bolt shear failure in test 7. Note that “PEEQ” is the equivalent plastic strain.

Figure 0-3 Shear-Beam end rotation curves for Test 8

Figure 0-4 Plastic hinge at the beam midspan
Figure 0-5 Bolt bearing at beam web

Figure 0-6 Weld rupture
1.13 Sherman-Ghorbanpoor Experiments Models

In these tests, the shear tab is welded to the web of the supporting member. Due to the expected flexibility of the supporting member, the tests were carried twice, with and without stiffeners. For that, the validation was carried on both type of experiments.

Table 3-2 shows a comparison between the ultimate shear forces and failure modes obtained from finite element models and experimental results.

1.13.1 Unstiffened Connections with Flexible Supports

1.13.1.1 Shear-Displacement Curves
Modeling the unstiffened connections showed a good agreement with the experimental results with a maximum error of 11% at the ultimate shear force. However, some deviation in the behavior was observed between the two curves in some tests. These irregularities in behavior are due to several difficulties in controlling the test environment in the experiments such as: uncertainty in the material properties, difficulties in determining the effectiveness of the lateral bracing during the experiments and uncertainty in the actual bolt forces. In addition, the use of tie constraint instead of modelling the welds in order to have time-efficient models and excluding the effect of residual stresses due to welding and rolling process is thought to have affected the numerical results. Overall, the finite element models are found to have a more realistic results in both the linear and nonlinear ranges. The figures 3-8 to 3-10 show the connections’ shear-vertical displacement curves of the FEA and experiments.

Figure 0-8 Shear-displacement curves for test 3U
Figure 0-9 Shear-displacement curves for test 4U

Figure 0-10 Shear-displacement curves for test 6U
1.13.1.2 Failure Modes

The behavior of the models was investigated throughout the loading steps. The failure modes were later compared to those observed during the experiments, and it was determined that the finite element simulation and experiments shared the same primary failure modes. However, secondary failure modes were better captured in the models through the presence of the equivalent failure plastic strains. Plate twist in test 1-U and bolt bearing in test 4-U are examples of the secondary failure modes.

It is important to identify the secondary failure modes as they could become more critical with the usage of different parameters, and to make sure the connection will provide sufficient strength and ductility. Figures 3-11 to 3-14 show some of the failure modes mentioned. Note that “U3” is the lateral displacement in the z-axis.

Figure 0-11 Plate Twist
Figure 0-14 Bolt shear failure

Figure 0-13 Plate bolt hole bearing

Figure 0-12 Web Mechanism
1.13.2 Stiffened Connections with Flexible Supports

1.13.2.1 Shear-Displacement Curves

As can be seen in figures 3-15 to 3-20, FEA shear-vertical displacement curves showed a good agreement with the experimental results with an error less than 15% at failure.

Figure 0-15 Shear-displacement curves for test 3A

Figure 0-16 Shear-displacement curves for test 3B
Figure 0-17 Shear-displacement curves for test 4A

Figure 0-18 Shear-displacement curves for test 3D
Figure 0-20 Shear-displacement curves for test 4B

Figure 0-19 Shear-displacement curves for test 6D
1.13.2.2 Failure Modes

Similarly, the finite element models’ primary failure modes agree with the experimental results. Figure 3-21 shows shear tab buckling as a failure mode obtained from the FEA and experiments for the stiffened shear tab connections.

![Figure 0-21 Shear tab buckling](image)

1.14 Validation Conclusion

To recapitulate, the proposed finite element models showed sufficiently accurate simulation for the behavior of extended shear tab connections with different configurations and support types. All primary failure modes were observed with a better understanding of the overall behavior. Moreover, the secondary failure modes were captured. These failure modes could be critical and affect the connection behavior when changing certain parameters. With an error of less than 15%, the proposed FEA model is considered adequate and could be used to further investigated the behavior of extended shear tab connections and perform parametric study to understand the relation between connection rotational ductility and the parameters.
Table 0-1 Shear strength and failure modes - Metzger Tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Experimental</th>
<th>Finite Element Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_{exp}$ (kips)</td>
<td>Failure Modes</td>
</tr>
<tr>
<td>6B2C – 4.5 – 1/2</td>
<td>89.7</td>
<td>G</td>
</tr>
<tr>
<td>7B1C – 9 – 3/8</td>
<td>98.0</td>
<td>I, B, H</td>
</tr>
<tr>
<td>10B2C – 10.5 – 1/2</td>
<td>94.6</td>
<td>L, C</td>
</tr>
</tbody>
</table>

Table 0-2 Shear strength and failure modes - Sherman&Ghorbanpoor Tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Experimental</th>
<th>Finite Element Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_{exp}$ (kips)</td>
<td>Failure Modes</td>
</tr>
<tr>
<td>Unstiff.</td>
<td>3U</td>
<td>54.8</td>
</tr>
<tr>
<td>Unstiff.</td>
<td>4U</td>
<td>98.7</td>
</tr>
<tr>
<td>Unstiff.</td>
<td>6U</td>
<td>138.0</td>
</tr>
<tr>
<td>Stiff.</td>
<td>3A</td>
<td>53.2</td>
</tr>
<tr>
<td>Stiff.</td>
<td>3B</td>
<td>53.1</td>
</tr>
<tr>
<td>Stiff.</td>
<td>3D</td>
<td>51.1</td>
</tr>
<tr>
<td>Stiff.</td>
<td>4A</td>
<td>103</td>
</tr>
<tr>
<td>Stiff.</td>
<td>4B</td>
<td>107</td>
</tr>
<tr>
<td>Stiff.</td>
<td>6B</td>
<td>124.5</td>
</tr>
</tbody>
</table>
Description of failure modes notations:

A – Bolt Shear
B – Bolt Bearing on the Plate
C – Shear Rupture
D – Web Mechanism
E – Twist
F – Bolt Fracture
G – Shear Yield
H – Web Shear
I – Weld
1.15 General Behavior of Shear Tab Connections

Before describing the behavior of extended shear tab connections and the results of this research study, this sub-section is dedicated to provide a description for the deformation mechanism and behavior of conventionally configured shear tab connections connected to rigid support. This type of connections could be considered as a basis from which other configurations divert, and mentioning its behavior gives the reader a better understanding to the deviations occurring in the extended configuration.

The following behavior description is concluded from previous research which the current AISC design procedure is based on. Astaneh (1989) divided the behavior of extended shear tab connections to three stages “At first the tab acts as a short cantilever beam with moment being dominant. As load increases the shear tab acts as a deep shear beam with the shear yielding effect dominant, if both the bolts and weld do not fail, due to excessive deformation, the shear tab acts similarly to the diagonal member of a truss and carries the applied shear by a combination of shear and diagonal tension effects.” From the statement and an overlook over the analytical behavior of previously tested specimens, it could be stated that the shear tab acts as a fuse to dissipate the excessive forces exerted on the connection. As shear forces increase considerable yielding occurs in the tab, almost uniformly distributed along its depth between the bolt and weld lines. While the rotation of the beam is accommodated by the rotation of the bolts group; bearing on the plate and beam web. As forces increase, bolt holes expand permanently to allow free rotation and release of excess moment.
1.16 Models Analysis Results and Observations

To investigate the rotational ductility of extended shear tab connections, different models with various cases have been studied. In this section, behavior analysis of the connections is described as observed for extended connections with rigid and flexible supports. The models for the former case were adopted from experimental tests executed earlier by Metzger. While the connections with flexible supports, connections connected to column webs, were studied for conventional configuration and extended configuration with single bolts column and multiple bolts column. Finally, the effects of plate thickness, eccentricity and connection orientation on the overall behavior were also studied.

1.16.1 Extended Shear Tab Connections with Rigid Supports

To study extended shear tab connections with rigid support, experimental results executed by Metzger (2006) were used. Metzger studied connections with both conventional and extended configurations, however, to serve the purpose of this study only the latter had been considered; tests 5 through 8. Test 6 was not considered due to experimental issues resulting in unreliable results. Table 4-1 contains member dimensions of all connections as listed by Metzger. All connections where designed on the criteria that failure occurs in the beam while reaching the required rotational ductility at the connection end.
Table 0-1 Metzger experimental tests members dimensions

<table>
<thead>
<tr>
<th>Test</th>
<th>Bolt Rows</th>
<th>Bolt Columns</th>
<th>a-distance (in.)</th>
<th>Plate Dimensions (in.)</th>
<th>Beam Section</th>
<th>Column Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>3</td>
<td>2</td>
<td>4.5</td>
<td>½” x 7 ½” x 8 ½”</td>
<td>W18x55</td>
<td>W21x62</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>1</td>
<td>9</td>
<td>3/8” x 10 ½” x 20 ½”</td>
<td>W24x62</td>
<td>W21x62</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>2</td>
<td>10.5</td>
<td>½” x 13 ½” x 14 ½”</td>
<td>W24x62</td>
<td>W21x62</td>
</tr>
</tbody>
</table>

Figure 0-1 Shear-Beam end rotation curve for test 7

The connection of test 7 was designed with a single bolt column with an extended a-distance. Experimentally, the connection reached a high capacity of 97 kips and a beam end rotation of 0.034 until failure due to lateral buckling of the beam at midspan. However, in the numerical study, A rotation almost doubling the specifications requirements was reached before shear failure of bolts. Notwithstanding, the plate showed instability with a
combination of twisting, buckling and yielding as shown in Figure 4-1 to 3. The connection rotation was accommodated by the overall rotation of the bolt group; yielding of the first two bolts and the 7th bolt at the bottom. Further, inelastic rotation proceeded by the rotation of the plate; due to the high stiffness of the support, the rotation of the plate was restricted causing local buckling at midspan accompanied by lateral twist. Overall the inelastic rotation was relatively controlled by the plate deformation.

The connections in Tests 5 and 8 have two vertical rows of bolts. Test 8 plate has extended plate configuration while that of Test 5 maintained the conventional dimensions. Test 8 had reached 97 kips and 0.035 radians of rotation before beam web buckling at midspan, as shown in Figure 4-5. By analysing the numerical model, not much inelastic rotational behavior was observed. However, secondary...
failure modes were detected, including bolts shear and plate yielding at the bottom end close to the weld. The rotational behavior of the connection was accommodated by the rotation of the bolt group with no plastic deformation. Further rotation was caused by the buckling of the plate due to twist and high tensile inelastic stresses at the upper side of the plate. No plastic deformation was observed in the bolts or bolt holes.

Figure 0-4 Shear-Beam end rotation curve for test 5

Figure 0-5 Shear-Beam end rotation curve for test 8
With a rigid support, the connection in test 5 rotated around the bolt line to almost reach 0.03 radians, as shown in Figure 4-5. Observing the behavior of the overall system, while the beam deflected the bolts shifted in the holes to allow free rotation of the beam end. As the load increased, the plate started to deform gradually with some rotation resulting in tension stresses between the bolt holes and the weld line (a-distance). The bolt group rotated along with the plate and beam as shown in Figures 4-6 and 4-7. No plastic deformation was observed in any of the bolts. However, the plate showed some minor buckling and yielding close to the weld line.

1.16.2 Conventional Shear Tab Connections with Flexible Supports

Understanding the behavior of conventional tab connections with flexible supports gives a better understanding of the issues that could be faced and what may lead to the need for extended configuration at such cases. To fit members with the same section sizes, the beam flanges had to be coped for three bolted connection. Looking at this case, at first as load increases bolt group rotates in the plate holes and continues bearing until the
connection reaches failure. Observing the behavior, support rotation could be considered as a second mode of failure. The column web started deforming with rotation of the plate and bolt group; while just very minor plate vertical deformation followed the rotation of connection. At failure, the plate and beam showed minimal to no elongation at bolt holes.

![Figure 0-8 Shear-Rotation plot for 3 bolted conventional flexible connection](image)

**Figure 0-8 Shear-Rotation plot for 3 bolted conventional flexible connection**

Five and eight bolted connections showed similar behavior to three bolted connection except that the rotation of column web was much less; this could be attributed to the higher stiffness of the support column and the rotational rigidity increase from the increased number of bolts. Anyhow, with load increase, bolt holes elongation and plate plastic deformation close to weld line started simultaneously and continued until bolts shear failure.

By considering the rotation of the connection as the total sum of beam end and column web rotation, three bolted connection reaches a rotation of almost 0.038, Figure 4-8, while five and eight bolted connections do not reach the specified rotation even at failure.
1.16.3 Extended Shear Tab Connections with Flexible Supports

Extended shear tab connections follow the same behavior for all bolt rows numbers. Three modes of failure were observed. First, fracture of bolts after excessive shear plastic deformation. Second mode of failure is plate bearing and holes elongation in the inner face of the plate and beam. Note that for all connections, mises stress had the same magnitude around the bolt holes in the plate and the beam, however, the plastic expansion was much more significant in the tab with a ratio of 3:1. Finally, the column web reaches high plastic deformation making it a third mode of failure.

In addition, plastic deformation at plate ends close to the weld line were also observed. Due to yielding at the top of the plate accompanied by the its twist causes additional forces at the bottom leading to local buckling in the bottom edge.

1.16.4 Extended Shear Tab Connections with Multiple Bolt Columns with Flexible Support

The plot behavior of extended tab connections with multiple bolts is similar to that of single bolt column. First, the column web starts to rotate, and is first to show plastic deformation, as the load increases the stresses in the bolts increases and start deforming plastically at the first line, this plastic deformation is accompanied by the vertical deformation of the plate at the bolt line, until the bolts reach the maximum capacity and fail, as shown in Figure 4-9. Throughout the behavior, plate twist causes additional stresses along the first line of bolts, and due to the relative stiffness along the plate before and after that line, minor plate bending is observed.
In all the cases, the vertical yield deformation of the plate and the bolt deformation was mainly concentrated on the first line of bolts while the rest of the bolt columns behaved along with the beam web, rotating around the center of the first line, leading to additional stress concentrations. From the unbalanced stress distribution, it could be concluded that the center of rotation of the bolt group is located at the centre of the first column and the a-distance is defined from first line of bolts to the weld line. This behavior could be observed from the comparative Figures 4-10 to 4-12 of three bolted connections tests.

Figure 0-11 Shear-Rotation plots for five bolted flexible connections with multiple bolt columns

Figure 0-10 Plate deformation at first line of bolts
### 1.17 Parametric Study

#### Table 0-2 Connection parameters studied

<table>
<thead>
<tr>
<th>No. of Bolts in Connection</th>
<th>Three</th>
<th>Five</th>
<th>Eight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate Thickness</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4</td>
<td>5/16</td>
<td>5/16</td>
<td></td>
</tr>
<tr>
<td>5/16</td>
<td>3/8</td>
<td>3/8</td>
<td></td>
</tr>
<tr>
<td>3/8</td>
<td>7/16</td>
<td>7/16</td>
<td></td>
</tr>
<tr>
<td>1/2</td>
<td>1/2</td>
<td>1/2</td>
<td></td>
</tr>
<tr>
<td>5/8</td>
<td>5/8</td>
<td>5/8</td>
<td></td>
</tr>
<tr>
<td>a-distance</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td>10.5</td>
<td>10.5</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>11.0</td>
<td>11.0</td>
<td></td>
</tr>
<tr>
<td>8.0</td>
<td>11.5</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td>8.5</td>
<td>12.0</td>
<td>12.0</td>
<td></td>
</tr>
<tr>
<td>Skewed Angle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>15</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>

Figure 0-12 Bolt group rotational behavior relative to original position
The objective of this section is to investigate plate parameters affecting the rotational ductility of extended shear tab connections and the overall behavior. Three parameters are considered including the tab thickness, a – distance, the distance from the bolt line to the weld line, and the connection orientation. The parameters were studied for three different connection configurations including three bolted connections, five bolted and eight bolted connections. Table 4-2 shows the values considered for each parameter.

1.17.1 a-distance

![Figure 0-13 a-distance effect of the rotational behavior of three bolted connection](image)

The a-distance is an important parameter to be considered while designing extended plates. This distance determines the slenderness of the plate and controls its tendency to twist or buckle. Thus, initially a limitation was specified for conventional design. The a-distance has been studied for three, five and eight bolted connections. The widths used are 7, 7.5, 8 and 8.5 inches for three bolted connections, while for five and eight bolted
connections the numbers ranged between 10.5 and 12.0 inches with 0.5 variance as well. Over all the rotation of extended shear tab connections wasn’t affected by the a-distance. All connections showed bolt shear controlling the failure mechanism with bolt hole expansion and plate rotation due to column web deformation. From the plot in Figure 4-13, it could be seen that the beam end rotation behaved the same along the elastic range while the column web rotation deviated allowing connections with smaller a distance to bear higher forces during the inelastic behavior.

1.17.2 Plate Thickness

Plate thickness is another plate geometry that is expected to affect the behavior of the plate. Conceptually, the plate thickness is indirectly proportional to the slenderness of the connection, meaning that as the thickness increases its tendency to buckle or twist reduces. In addition, the thickness of the plate appeared to affect the overall expansion and deformation of plate holes at bearing with the bolts. Five different plate thicknesses were chosen for each connection type. With thicknesses ranging from 0.25 to 0.625 with 0.125 in increments.
The connection rotational behavior wasn’t affected by the plate thickness, however the failure mechanism was of concern. Due to low plate thickness, plate rupture and excessive elongation could occur in the connection at bolt holes. As could be seen in Figure 4-15, the connections with thicker plates followed the same deformation pattern and failure mechanism expected, however, by reducing the plate thickness, earlier failure occurred due to excessive yielding at holes causing additional force concentrations and bearing stresses leading to early failure and rupture. The difference in behavior could be observed by examining the plastic deformation in the figures 4-14 and 4-16.

Figure 0-15 Effect of plate thickness on the rotational behavior of five bolted flexible connection
The effect of connection orientation on the rotational behavior of extended shear tab connections was studied. Instead of attaching the plate and beam perpendicularly, the plate was oriented with an angle of 5, 10, 15 and 20 degrees. Due to the orientation of the plate, the beam flange could come to conjunction with the supporting member components in a conventional configuration, making plate extension a good solution for the case. In a previous study by Mustafa et al. (2016), it was found that the connection orientation has a significant effect on the torsional stability of the plate. However, by numerical analysis, the over-all rotational behavior of the connection is not affected but by a minor reduction in the column web rotation as could be shown in Figure 4-17. The reduction is attributed to the moment component redistribution in both directions due to the formed angle.
1.18 Results Discussion

1.18.1 Comparison Between Rigid and Flexible Supports

The capability of the connection to follow the rotation of the supported member classifies the support as rigid or flexible. A rigid support restrains the connection from following the rotation of the supported member. Conversely, although a flexible support should still provide relative stiffness to the connection, but it allows a rotational pliability to the connection. Throughout this research, column strong axis was considered as a rigid support while the weak axis was considered as a flexible support.

It was found that to achieve the targeted end rotations relative to the column face at the connection, the absolute rotation of the beams needed to be much larger in the case
of flexible support conditions due to the rotation of the columns. In all cases, for single or multiple columns of bolts, the connections surpassed the predicted shear resistance and the targeted beam end rotations.

It can be affirmed that the support condition had little effect on the failure modes of tab connections since all bolted connection tests suffered little to extensive shear deformation of the plate which carried on to high shear stresses and bolt failure. However, it should be stated that plate buckling is observed to be less evident when the connection is attached to flexible support, while for connections attached to rigid supports plastic buckling of the bottom edge of the shear tab is found to be a critical limit state.

The rotational behavior of the connection with respect to the shear force were similar for both support types as appeared in previous research. Nevertheless, with flexible supports, most of the rotation during the inelastic and plastic behavior of the connection was accommodated by the column’s web instead of the plate and beam-end that are directly connected to the bolt group. This behavior leads extensive plastic deformations in the column web leading to web mechanism failure that should be considered during the design procedure.

Although negative bending moments were expected for both support conditions which are usually neglected in the column design, it was found that the main difference between the shear tab connection to rigid and flexible supports lies in the magnitude of the flexural bending that is generated in the column weak axes in the case of flexible supports.
1.18.2 Difference Between Conventional and Extended Configurations

In both conventional and extended shear tab connections significant localized deformation within the supporting member was observed. The deformation was characterized by extensive yielding in the column web at both end corners of the tab. Also, both connection configurations in all tests reached almost the same shear strength. However, extended configuration showed much higher rotational ductility as some of the conventional connections didn’t even reach 0.03 radians.

The behavior required of the tab, to act as a shear fuse to release concentrated stresses wasn’t observed in the conventional configurations attached to flexible supports. The plate rotated with the column web, allowing plastic deformation in the web, this stress release affected the behaviour and the failure modes of the connection. The bolts rotated until shear failure and web mechanism failure. It can be observed from figure 4-18 that extended tabs showed much higher vertical deformation and bolt plastic yielding before failure.

![Figure 4-18 Comparative behavior of the bolt behavior in extended and conventional shear tabs connections](image)
Thus, for extended shear tab configuration, higher single bolt shear strength values should be considered when determining the shear resistance of the bolt group. This could be done by eliminating the reduction factor applied to account for uneven distribution, as it is determined to be overly conservative by previous research.

1.18.3 Effect of Double and Triple Vertical Line of Bolts

The main objective of comparing different number of bolt columns was to observe the rotational behavior and shear resistance of multiple column bolted test and to indicate whether this configuration allows enough rotation to be considered as a ductile connection. The shear tab dimensions and properties were kept constant in all tests to focus on the effect of the bolt group only.

In the conducted tests, all connections surpassed the rotational requirement by AISC. It was observed in the analysis that increasing the number of bolt vertical rows to two had some influence on the connection shear capacity by almost doubling it with 3 bolted connections. The relative strength ratio decreases as the number of bolts per column increases. However, further adding a third vertical line of bolts has very little influence on the overall connection behavior and capacity when the other parameters are kept constant. Important to realize, both bolted arrangements attained their targeted beam end rotation with similar behavior. Although the behavior deviated hardly as the number of bolts per column increased, the results obtained suggest that both double and triple bolt line patterns allowed enough rotation in order for the connections to fail in a ductile and safe manner avoiding sudden collapses.
In the configurations with multiple columns of bolts the deformations were more concentrated around the holes in the first line of bolts closest to the supporting column which led to fractures occurring along that line of bolt holes before other parts of the shear tab saw significant plastic deformations. The rest of the tab width that is in direct contact with the beam web is strictly restrained by increasing the number of bolt columns and doesn’t contribute to the end rotation of the beam. The effect of this restriction is more critical than just a movement barrier; with the increased capacity of the connection from the added bolt line, the demand of rotation is increased, causing higher stress concentration and excessive bolt hole deformations at the first line. This results in bolt hole rupture.

As can be seen, with the center of rotation of the bolt group being located at the first line middle bolt, and due to excessive plastic hole expansions on the first line of bolts, the tab design procedure for multiple columns of bolts should be followed with the consideration that the plate shear capacity is concentrated in the a-distance defined between the tab edge at the support to the first line of bolts. Additionally, the bolt forces should not be reduced for multiple lines of bolts to prevent bolt hole fracture, plate failure, and allow bolt deformation instead.

1.18.4 Stiffeners Influence on the Connection Rotational Behavior

The flexible support connections are studied with and without stiffeners. The finite element analysis has indicated that the stiffeners did add rigidity to the connection that the behaviour was similar to that of connections with rigid supports. Weak stiffeners did carry part of the load as lateral deformation was obviously detected. In fact, the analysis tends to
confirm that adding web stiffeners do attract flexural moment to the column, as the bending moment in the weak axis could be almost doubled in the weak axis bending.

The stiffness that the added plates create in the columns diminishes the flexible behavior of the support attracting unwanted eccentricity in the support. On this indication, for connections framed to column webs with stiffeners, it is recommended to consider the behaving distance of the tab, a – distance, to be spanning from the first bolt line to the outer edge of the stiffeners, because of the stiffness brought to the connection acts behind that point. Hence, for moment calculations, the design eccentricity should be considered as the distance between the centerline of the bolt group and the tip of the column flanges. However, if stiffeners are not used, local failure modes in the column web, e.g. web mechanism failure, should be considered in the design, while local buckling and compression instability should be considered in the tab.

The moment applied overstresses the column web since it is relatively thin. Abolitz and Warner (1965) investigated the supporting column web bending using the yield line method. This method is used to determine the ultimate loads based on equilibrium between internal energy, amount of work done by the column web, and external energy, single bracket moment. The equations are derived based on the assumption that the side ends of the column web are fixed by the flanges.

Note that if the moment applied is less than \( X \), the column web will not be overstressed. However, if it exceeded the capacity, a column with thicker web could be used, a doubler plate could be back welded, or stabilizer plates could be added to reduce the moment and stiffen the web.
\[ k = \frac{2T}{l} + \frac{2l}{T} + 8 \]

\[ X = k \phi m l \]

In specification notation,

\[ \phi_b X = k \phi m l \geq X_u = R_u a \]

\[ \frac{X}{\Omega_b} = k \frac{\phi m l}{\Omega_b} \geq X_a = R_a a \]

where,

\[ X = \text{Ultimate bracket moment to which the plate may be subjected, kips - in.} \]

\[ k = \text{coefficient depends on L/T ratio} \]

\[ \phi = \text{reduction factor (0.6 - 0.9)} \]

\[ m = f S = \text{nominal moment in the column web, kips-in per inch} \]

\[ f = 0.75 F_y = \text{nominal bending stress of the column web, ksi} \]

\[ F_y = \text{yield stress of the column web, ksi} \]

\[ S = \frac{t^2}{4} = \text{plastic section modulus, in.}^2 \text{ per unit width} \]

\[ T = \text{horizontal width of the column web, in.} \]

\[ \phi_b = 0.90 \]

\[ \Omega_b = 1.67 \]
1.19 Design Procedure

This section is aimed to provide the reader with a summary of the latest design method for single plate shear connections specified by AISC (14th Edition). The method was compared to those of previous editions of AISC specifications (3rd and 13th) to study the development timeline and build a better understanding of the latest method. Updates in the newest AISC (15th Edition) were also added.

1.19.1 Lead-in

A single plate connection is made with a plate welded to the support from both end sides of the plate and bolted to the supported member (e.g. beam). The available strength of this type of connection is determined from the applicable limit states for the bolts (Part 7), weld (Part 8) and connecting elements (Part 9). There exist two possible design procedures:

a. Conventional Configuration: For connections that satisfy a given set of dimensional configurations could be designed using a simplified procedure.

b. Extended Configuration: All other connection configurations.

Both the conventional and extended configurations permit the use of Group A or Group B bolts. The procedure is valid for bolts that are snug-tightened, pretensioned, or slip-critical. In both the conventional and extended configuration, the design recommendations are equally applicable to plate and beam web material with \( F_y = 36 \text{ ksi or 50 ksi} \).
1.19.2.1 Dimensional limitations

The dimensional limitations according to which the design of single plate shear connection follows the conventional configuration method are similar in all four editions of AISC specifications, except for minor differences. The limitations listed in AISC 14th Edition are as follows:

a. Only a single vertical row of blts is permitted. The number of bolts in the connection, \( n \), must be between 2 and 12.

b. The distance from the bolt line to the weld line, \( a \), must be equal or less than 3 \( \frac{1}{2} \) in.

c. Standard holes (STD) or short-slotted holes transverse to the direction of the supported member reaction (SSLT) are permitted to be used as noted in Table 10-9.

d. The vertical edge distance, \( L_{ev} \), must satisfy AISC specification Table J3.4 requirements.

e. The horizontal edge distance, \( L_{eh} \), should be greater than or equal to \( 2d \) for both plates and beam web, where \( d \) is the bolt diameter.

Either the plate thickness, \( t_p \), or the beam web thickness, \( t_w \), must satisfy the maximum requirement given in Table 10-9. (This has been updated from the 13th Edition which states that \( t_p \) or \( t_w \leq \frac{d}{2} + 1/16 \) for all \( n \) and \( e \).)

1.19.2.2 Design Checks

- Weld:
Welds in single plate shear connections are typically not explicitly designed. A weld thickness equal to $\frac{5}{8}t_p$ is recommended. This condition provides that the plate will yield prior to weld fracture in pure moment, pure shear, or a combination of both shear and moment. This limit also ensures that the weld will not have excessive capacity beyond the capacity of the plate. This was an update proposed by Thornton (2006) where previous AISC editions (3rd) were based on Astaneh’s original weld thickness limitation $\frac{3}{4}t_p$ that ensured plate yield prior to weld yield.

- Eccentricity

The bolt strength in shear connections is dependent on the eccentricity, $e$. The AISC 3rd edition expanded on Astaneh’s work (1989c) and calculated an eccentricity depended on the number of bolts, $a$-distance, hole type and support type (rigid or flexible). The eccentricity calculations in the AISC 13th Edition have been altered to reflect newer research completed by Creech (2005). Calculation of eccentricity is no longer dependent on the support type. For standard holes, no eccentricity is calculated for connections in the conventional configuration with less than 10 bolts; 20% bolt group strength reduction in AISC 2005 doesn’t apply to single plate shear connections. For 10, 11 and 12 bolt connections the eccentricity provides strength reduction of greater than 20%; therefore, eccentricity is taken into account. However, the calculated eccentricity coefficient is multiplied by (1.25). In the latest AISC specifications (14th and 15th editions) the eccentricity calculation had been added again and should be calculated as given by Table 10-9.
• **Bolts Strength**

Bolts are checked for shear strength in accordance with AISC specification section J3.6 assuming the eccentricity, $e$, shown in Table 10-9 and the effective number of bolts from the Table 7-6.

• **Plate Check**

Conventional configuration procedures require edge distances to be consistent with chapter J Table J3.4 of the AISC specification. The AISC 3rd edition finds the plate strength is based on *shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate*, and the *bearing capacity of the plate*. The AISC 13th and 14th editions for conventional configuration recommends the same checks as the AISC 3rd edition. The 14th edition adds that the plate bearing and tearout are checked in accordance with AISC specifications section J3.10 assuming the reaction is applied concentrically. The strength at the individual bolt hole is calculated as the minimum of the bearing and tearout strength of the hole. The tear out is calculated in the direction of the applied force. The individual bolt hole strengths and for each bolt are then added to obtain the bearing and tearout capacity of the connection.

All reviewed specifications state that plate buckling will not control for the conventional configuration.
1.19.3 \textit{Extended Configuration Design Procedure}

1.19.3.1 \textbf{Dimensional Limitations}

\begin{itemize}
  \item[a.] The number of bolts, \(n\), is not limited.
  \item[b.] The distance from the weld line to the bolt line, \(a\), is not limited.
  \item[c.] The use of holes must satisfy AISC specification section J3.2 requirements.
  \item[d.] The horizontal and the vertical edge distance, \(L_{eh}\) and \(L_{ev}\), must satisfy AISC specification Table J3.4 requirements.
\end{itemize}

1.19.3.2 \textbf{Design Checks}

In addition to the design checks of the conventional configuration, the following additional checks should be considered,

\begin{itemize}
  \item \textbf{Weld}

      The weld thickness limit for extended configuration applies as in the conventional configuration.
  \item \textbf{Eccentricity}

      Single plate connections designed using extended configuration procedure are required to consider an eccentricity equal to the distance for the centroid of the bolt group to the support. This is updated from the 13\textsuperscript{th} edition where \(e = a\) of the connection, where \(a\) is defined as the distance to first raw of bolts. Exceptional methods could also be used as those proposed by Sherman and Ghorbanpoor (2003).
  \item \textbf{Bolts Strength}
\end{itemize}
All checks apply as in the conventional configuration, Short slotted holes can be used with the extended configuration with the bolts designed as bearing. Any slip of the bolts is a serviceability issue and does not affect the connection strength (Muir and Hewitt, 2009)

- Plate Checks

Several design checks were added to the 13th and 14th editions to create dimensional flexibility in the extended design configuration. These include:

i. Ensuring that the moment capacity of the plate is less than the moment capacity of the bolt group.

ii. Checking plate flexure using von-Mises shear reduction,

iii. Checking for plate buckling using double coped beam procedure found in AISC Manual (2005b), which is based on work done by Muir and Thornton.

These checks are accounted for in the AISC specifications (14th edition) through the following:

a. Determine the maximum plate thickness permitted such that the plate moment strength does not exceed the moment strength of the bolt group shear strength:

\[
t_{max} = \frac{6M_{max}}{F_y l^2}
\]

where,

\[
F_y = \text{specified minimum yield stress of plate, ksi}
\]

\[
M_{max} = \frac{F_nv}{0.90} (A_b C')
\]
The shear strength of an individual bolt from AISC specification Table J3.2, ksi, divided by a factor of 0.90 to remove the 10% reduction for uneven force distribution in end-loaded bolt groups (Kulak, 2002). The joint in question is not end-loaded.

\[ F_{nv}^{0.90} \]

\( A_b \) = area on an individual bolt, in²

\( C' \) = coefficient from Part 7 for the moment-only case (instantaneous center of rotation at the centroid of the bolt group)

\( l \) = depth of plate, in

Exceptions,

- For a single vertical row of bolts only, the foregoing criterion need not be satisfied if either the beam web or the plate satisfies the thickness requirement of Table 10-9 and both satisfy \( l_{eh} \geq 2d_b \)

- For a double vertical row of bolts only, the foregoing criterion need not be satisfied if both the beam web and the plate satisfy the thickness requirements of Table 10-9 and \( l_{eh} \geq 2d_b \).

Note that,

- The moment calculation equation is an updated version of the AISC 13th Edition where \( M_{max} = 1.25F_v(A_bC') \); multiplied by 1.25 to remove the 20% reduction for uneven distribution in end-loaded bolts. The joint is not end-loaded.

- The foregoing check is made at the nominal strength level, since the check is to ensure ductility, not strength.
b. Check the plate for limit states of shear yielding, shear rupture, block shear rupture, and flexural rupture. Check the beam web for the same limit states, as applicable.

c. Check the plate for the limit states of combined shear and moment:

\[
\left( \frac{V_r}{V_C} \right)^2 + \left( \frac{M_r}{M_C} \right)^2 \leq 1.0
\]

\[M_C = \phi_b M_n \text{(LRFD)} \text{ or } M_n / \Omega_b \text{ (ASD)}, \text{ kip-in}\]

\[M_n = Z_{pl} F_y, \text{ kip-in}\]

\[M_r = M_u \text{(LRFD)} \text{ or } M_a \text{ (ASD)} \]

\[= V_r \alpha, \text{ kip-in}\]

\[V_C = \phi_c V_n \text{ (LRFD)} \text{ or } V_n / \Omega_v \text{ (ASD)}, \text{ kips}\]

\[V_n = 0.6 F_y A_g, \text{ kips}\]

\[V_r = V_u \text{ (LRFD)} \text{ or } V_a \text{ (ASD)}, \text{ kips}\]

\[Z_{pl} = \text{plastic section modulus of the shear plate, } in^3\]

\[A_g = \text{gross sectional area of the shear plate, } in^2\]

\[\alpha = \text{distance from the support to the first line of bolts, in}\]

\[\phi_b = 0.90\]

\[\phi_v = 1.00\]

\[\Omega_b = 1.67\]

\[\Omega_v = 1.50\]
Note, this is an updated procedure to that in the 13th edition where the available flexural yielding strength of the plate is calculated based upon a critical stress, 

\[ M_n = F_{cr}Z \]

where, \( F_{cr} = \sqrt{F_y^2 - 3f_v^2} \) based on von Mises shear reduction.

d. Check the plate for limit state of buckling using the double-coped beam procedure given in part 9. This check assumes the beam supported near the end of the plate as indicated in Step 6. For other conditions, see Thornton and Fortney (2011).

e. Ensure that the supported beam is braced at points of support.

The design procedure for extended single plate shear connection permits the column to be designed for an axial force without eccentricity. In some cases, economy may be gained by considering alternative design procedures that allow transfer of some moment into the column. A percentage of the column’s weak axis flexural strength, such as 5% may be used as a mechanism to reduce the required eccentricity on the bolt group provided that this weak axis flexural strength may be justified at the roof level.
Design Procedure of Extended Shear Tab Connections

Estimate the required number of bolts
- Use Table 7-1 (AISC Manual) for single bolt shear strength
- Use Table 7-7 (AISC Manual) for bolt group and number of bolts estimation
- Estimate the a-distance needed for the connection

Determine Maximum Plate Thickness Using Eq. 10-3 (AISC Manual). Then select an estimated shear tab thickness less than the maximum.

Check the shear capacity of the connection and limit states
- Bolt bearing using AISC Specifications Eq. J3-6a
- Plate shear yielding using AISC Specifications Eq. J4-3
- Plate shear rupture using AISC Specifications Eq. J4-4
- Plate block shear rupture using AISC Specifications Eq. J4-5
- Plate local buckling using AISC Manual Eq. 9-18
- Flexural rupture of the plate using AISC Manual Eq. 15-3 and Table 15-3
- Strength of the column web using AISC Manual Eq. 9-2

Check weld capacity using AISC Manual Part 10

Check if stabilizer plates are needed
- Available strength to resist lateral displacement AISC Manual Eq. 10-6, Eq. 10-7a, Eq. 10-7b

Check column web bending strength using the equations proposed in this thesis on Page 57
Notes and Design Recommendations:

- If stabilizer plates are used, it is recommended to use an a-distance from the first line of bolts to the supporting member flange. This results in less critical and reduced plate dimensions.

- Stabilizer plate and column web strength are also recommended for conventional connection design.

- If column strength is adequate and no stabilizer plates are added, it is recommended to design the plate with shear yielding preceding flexural yielding to satisfy the rotational requirements.

- If multiple bolt columns are used, it is recommended to take the a-distance from the first line of bolts to weld line and not from the bolts center line.

1.20 Conclusions and Recommendations

Nonlinear finite element models were prepared to analyze and study the rotational behavior of extended shear tab connections. The models were validated against experimental data previously done by Sherman-Ghorpanboor (2002) and Metzger (2006) for orthogonal extended shear tab connections. A special attention was given to connections attached to column webs, flexible supports. The following observations and conclusions can be made from this investigation based on the finite element results:

1. The results from finite element models showed good agreement with the experimental results with an error less than 15%. The proposed models could be reliable to predict the plastic behavior and capacity of the framed connections.
2. Secondary failure modes were easily obtained with the numerical models. Due to the low plastic deformations of these failure modes, they could be hard to detect through experimental work and visual inspection.

3. For connections with rigid support, the column contributes significantly in resisting the moment, while possessing minimal deformation and rotation.

4. Due to shear yielding the extended plate material losses stiffness leading to local buckling of the plate when the connection is framed to rigid support.

5. For extended connections with flexible support, the column web rotates with the connection, increasing its rotational ductility but changing the plate behavior with effects on connection failure modes.

6. Flexible supports do not seem to allow significant greater connection rotation than rigid supports, however, most of the inelastic rotation is accommodated by the supporting member, column web, while on the other hand rigid supports restrict the motion to the plate.

7. The tests on bolted connections revealed that extended shear tab connections shear-rotation curves showed similar behavior regardless of the support condition they are attached to.

8. All connections with rigid supports have showed sufficient rotational ductility. Inelastic behavior was concentrated on plate yielding following beam failure as designed by Metzger, with no observed deformation in bolts or bolt holes.

9. The increase in number of bolts in conventional flexible connections, restrained relative bolt slip in bolt holes allowing less rotation.
10. Conventional connections with flexible supports allowed rotation of beam end by the rotation of bolt group, plate and column web. While showing no vertical deformation along line of bolts with plate yielding. Stiffening of the connection could be a good solution for conventional connections too, as no vertical yielding was observed in the plate resulting in reduced rotational ductility and increase in stress concentrations on bolts resulting in shear failure before reaching rotational demand.

11. Connections with fewer bolts developed smaller moment and exhibited larger rotational ductility.

12. The moment capacity of the plate should be designed to be lower than the moment capacity of the bolt group to assure yielding of the plate. The design thickness of the plate should be designed according to the minimum requirement of AISC to assure plate hole elongation and rotation of the connection.

13. In addition, as suggested by previous studies, a negative axis bending moment was induced in the support columns. These moments are generally neglected in the design of the columns, which seems as an appropriate assumption for core columns but does not seem conservative for perimeter columns, especially when acting as a flexible support. It was found that the bending moment imposed to the columns could be very well predicted using AISC 2011 reference design eccentricity, and that in a slightly conservative manner.

14. With regards to the bolt configuration, the difference between the number of vertical bolt rows in the connection had very little influence on the ultimate connection capacity and in all cases the predicted connection strength was surpassed. The findings indicate that the addition of a third vertical line of bolts, while keeping the other shear tab
parameters constant has little to no effect on the overall observed connection resistance in comparison with a two bolts column configuration. In addition, in terms of rotation allowance, both configurations similarly exceeded their targeted rotation values.

15. Connections with multiple bolt columns showed excessive yielding at the first line of bolts while the inner bolts showed minimal deformation due to tight attachment to beam web. Thus, the a-distance could be considered from the first line of bolt to the weld line instead of weld line to bolt group center.

16. Connections framed to column weak axis with the addition of plate stiffeners showed the same behavior to connections with rigid supports. The rotation of the plate was mostly accommodated by the bolt group and the plate. Thus, while designing the connection with stiffeners, it is recommended to consider the a-distance from the first bolt line to the end of the stiffeners, outer flange edges.

17. The connection shear capacity is independent of connection orientation. Further, the connection orientation had no effect on the rotational behavior of the connection or its relative twist.

18. As the a-distance increases the connection shear capacity decreases due to the increase of the applied moment on the connection.

19. The plate thickness slightly affects connection shear capacity.

20. The connection’s shear capacity decreases with the increase of the a-distance due to the increase of the applied moment components on the connection but the rotational behavior stays similar.
21. The connection orientation has no effect on the rotational ductility of the connection but barely reducing the moment exerted on the column due to separation to two moment components and plate twist.

1.21 Future Work

As this research was aiming to clarify the rotational behavior of extended shear tab connections attached to column webs, future tests on other types of flexible supports such as girder webs could be studied. Investigating the introduced torsion in the girder insures proper analysis in the design process of extended shear tabs attached to girders.

Further, to have a valid design procedure to frames with composite members, the influence of concrete slab on the rotational ductility and shear resistance of the connections may be studied.
APPENDIX A. SNAPSHOTS OF CONNECTIONS DEFORMED SHAPES

While all connections have similar form, listing all pictures along with the observations analysis and results description in previous chapters is thought to overburden the thesis body; causing confusion to the reader instead of ease of understanding. For that, only pictures of components with observed critical behavior were added. However, to serve the comprehensiveness of the study and, more importantly, to allow readers to make their own observations and conclusions, the pictures for each component of every tested connection is attached in this appendix. The pictures are taken of deformed shapes with plastic strain and von Mises stress contours. The listing of the appendix sub-titles follows the same sequence of the previous chapters to allow fast passing between content.
A.1  Extended Shear Tab Connection with Rigid Supports (Metzger)

A.1.1  Metzger Test No. 5
A.1.2 Metzger Test No. 7
A.1.3 Metzger Test No. 8
A.2 Extended Shear Tab Connection with Flexible Supports

A.2.1 Three Bolted Connections
A.2.2 Five Bolted Connections
A.2.3 Eight Bolted Connections
A.3 Conventional Shear Tab Connection with Flexible Supports

A.3.1 Three Bolted Connections
A.3.2 Five Bolted Connections
A.3.3 Eight Bolted Connections
A.4 Extended Shear Tab Connection with Flexible Supports with Multiple Bolt Columns

A.4.1 Three Bolted Connections with Double Bolts Columns
A.4.2 Three Bolted Connections with Triple Bolts Columns
A.4.3 Five Bolted Connections with Double Bolts Columns
A.4.4  Five Bolted Connections with Triple Bolts Columns
A.4.5 Eight Bolted Connections with Double Bolts Columns
A.4.6  Eight Bolted Connections with Triple Bolts Columns
APPENDIX B. SHEAR VS. ROTATION CURVES

B.1 Extended Shear Tab Connection with Flexible Supports

B.1.1 Three Bolted Connections

B.1.2 Five Bolted Connections
B.1.3  Eight Bolted Connections

B.2  Conventional Shear Tab Connection with Flexible Supports

B.2.1  Three Bolted Connections
B.2.2 Five Bolted Connections

B.2.3 Eight Bolted Connections
B.3 Extended Shear Tab Connection with Flexible Supports with Multiple Bolt Columns

B.3.1 Three Bolted Connections

B.3.2 Five Bolted Connections
B.3.3 Eight Bolted Connections

B.4 Parameters Effects on Extended Shear Tab Connections with Flexible Supports

B.4.1 Three Bolted Connections

B.4.1.1 \(a\)-distance
B.4.1.2 Plate Thickness

![Diagram showing plate thickness variations with angle and rotation.]

B.4.1.3 Skewed Angle

![Diagram showing skewed angle variations with rotation.]

95
B.4.2 Five Bolted Connections

B.4.2.1 a-distance

B.4.2.2 Plate Thickness
B.4.2.3 Skewed Angle

B.4.3 Eight Bolted Connections

B.4.3.1 $a$-distance
B.4.3.2 Plate Thickness

B.4.3.3 Skewed Angle
REFERENCES


VITA

EDUCATION

<table>
<thead>
<tr>
<th>Degree</th>
<th>Institution</th>
<th>Field</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>MS</td>
<td>University of Illinois at Chicago, Civil and Materials Engineering</td>
<td></td>
<td>July 2018</td>
</tr>
<tr>
<td>BS</td>
<td>Middle East Technical University, Civil Engineering</td>
<td></td>
<td>July 2016</td>
</tr>
</tbody>
</table>

SCHOLARSHIPS AND GRANTS

<table>
<thead>
<tr>
<th>Scholarship</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>TUBITAK Undergraduate Research Scholarship - 2205</td>
<td>2015-2016</td>
</tr>
<tr>
<td>King Abdullah Scholarship</td>
<td>2012-2016</td>
</tr>
</tbody>
</table>

PUBLICATIONS

Journal Publications


Conference Papers

H.F. Özel, A. Sarıtaş, T. Tasbahji; "Finite element model with semi-rigid connections for vibration assessment of steel moment resisting framed structures", The International Colloquium on Stability and Ductility of Steel Structures, Timisoara, Romania, 30 May – 01 June 2016


H.F. Özel, T. Tasbahji, A. Saritas; "Vibration characteristics of various wide flange steel beams and columns", 6th International Conference on Structural Engineering and Construction Management, Kandy, Sri Lanka, 11-13 December 2015

H.F. Özel, A. Sarıtaş, T. Tasbahji; "Yarı-rijit bağlantılı çelik çerçeve yapıların titreşim özellikleriinin modellenmesi", 3. Türkiye Deprem Mühendisliği ve Sismoloji Konferansı, İzmir, 14-16 Ekim 2015

T. Tasbahji, M. Mahamid, M. Al Hijaj; " Rotational Ductility of Unstiffened Extended Shear Tab Connections Welded to Column Webs", SEI Structure Congress 2019, Orlando, Florida, 25-27 April 2019