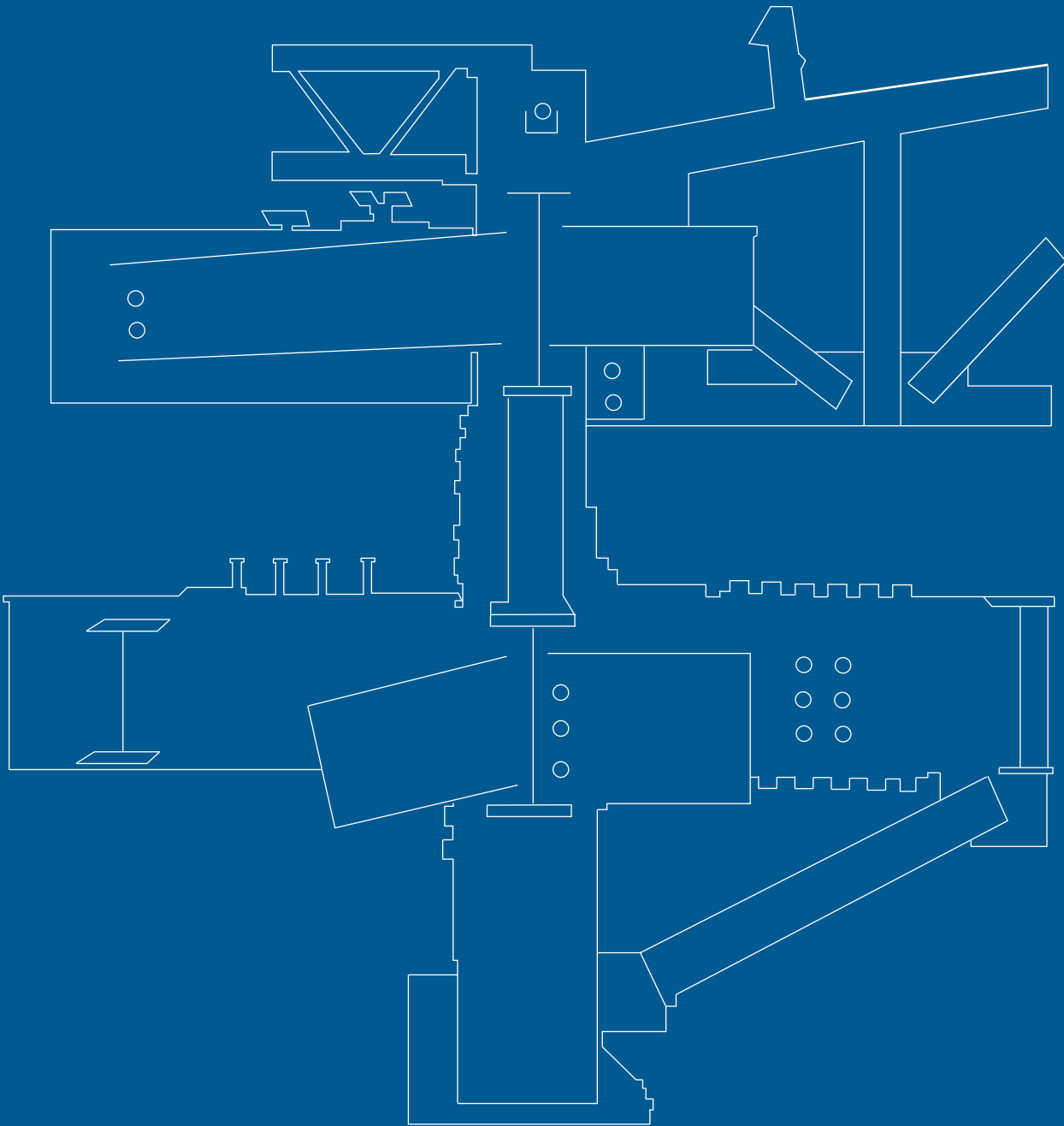# Connections Teaching Toolkit

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Patrick Veltri

A Teaching Guide for Structural Steel Connections

PREFACE

This connection design tool kit for students is based on the original steel sculpture designed by Duane S. Ellifritt, P.E., Ph.D., Professor Emeritus of Civil Engineering at the University of Florida. The tool kit includes this teaching guide, a 3D CAD file of the steel sculpture, and a shear connection calculator tool. The teaching guide contains drawings and photographs of each connection depicted on the steel sculpture, the CAD file is a 3D AutoCAD® model of the steel sculpture with complete dimensions and details, and the calculator tool is a series of MathCAD® worksheets that enables the user to perform a comprehensive check of all required limit states.

The tool kit is intended as a supplement to, not a replacement for, the information and data presented in the American Institute of Steel Construction's Manual of Steel Construction, Load & Resistance Factor Design, Third Edition, hereafter, referred to as the AISC Manual. The goal of the tool kit is to assist students and educators in both learning and teaching basic structural steel connection design by visualization tools and software application.

All information and data presented in any and all parts of the teaching tool kit are for educational purposes only. Although the steel sculpture depicts numerous connections, it is by no means all-inclusive. There are many ways to connect structural steel members together.

In teaching engineering students in an introductory course in steel design, often the topic of connections is put off until the end of the course if covered at all. Then with the crush of all the other pressures leading up to the end of the semester, even these few weeks get squeezed until connections are lucky to be addressed for two or three lectures. One reason for slighting connections in beginning steel design, other than time constraints, is that they are sometimes viewed as a "detailing problem" best left to the fabricator. Or, the mistaken view is taken that connections get standardized, especially shear connections, so there is little creativity needed in their design and engineers view it as a poor use of their time. The AISC Manual has tables and detailing information on many standard types of connections, so the process is simplified to selecting a tabulated connection that will carry the design load. Many times, the engineer will simply indicate the load to be transmitted on the design drawings and the fabricator will select an appropriate connection.

Yet connections are the glue that holds the structure together and, standardized and routine as many of them may seem, it is very important for a structural engineer to understand their behavior and design. Historically, most major structural failures have been due to some kind of connection failure. Connections are always designed as planar, two-dimensional elements, even though they have definite three-dimensional behavior. Students who have never been around construction sites to see steel being erected have a difficult time visualizing this three-dimensional character. Try explaining to a student the behavior of a shop-welded, field-bolted double-angle shear connection, where the outstanding legs are made purposely to flex under load and approximate a true pinned connection. Textbooks generally show orthogonal views of such connections, but still many students have trouble in "seeing" the real connection.

In the summer of 1985, after seeing the inability of many students to visualize even simple connections, Dr. Ellifritt began to search for a way to make connections more real for them. Field trips were one alternative, but the availability of these is intermittent and with all the problems of liability, some construction managers are not too anxious to have a group of students around the jobsite. Thought was given to building some scale models of connections and bringing them into the classroom, but these would be heavy to move around and one would have the additional problem storing them all when they were not in use.

The eventual solution was to create a steel sculpture that would be an attractive addition to the public art already on campus, something that would symbolize engineering in general, and that could also function as a teaching aid. It was completed and erected in October 1986, and is used every semester to show students real connections and real steel members in full scale.

Since that time, many other universities have requested a copy of the plans from the University of Florida and have built similar structures on their campuses.

INTRODUCTION

Connection design in an introductory steel course is often difficult to effectively communicate. Time constraints and priority of certain other topics over connection design also tend to inhibit sufficient treatment of connection design.

The Steel Connections Teaching tool kit is an attempt to effectively incorporate the fundamentals of steel connection design into a first course in steel design. The tool kit addresses three broad issues that arise when teaching students steel connection design: visualization, load paths, and limit states.

In structural analysis classes, students are shown idealized structures. Simple lines represent beams and columns, while pins, hinges, and fixed supports characterize connections. However, real structures are composed of beams, girders, and columns, all joined together through bolting or welding of plates and angles. It is no wonder that students have trouble visualizing and understanding the true three-dimensional nature of connections!

The steel sculpture provides a convenient means by which full-scale steel connections may be shown to students. The steel sculpture exhibits over 20 different connections commonly used in steel construction today. It is an exceptional teaching instrument to illustrate structural steel connections. The steel sculpture's merit is nationally recognized as more than 90 university campuses now have a steel sculpture modeled after Dr. Ellifritt's original design.

In addition to the steel sculpture, this booklet provides illustrations, and each connection has a short description associated with it.

The steel sculpture and the booklet "show" steel connections, but both are qualitative in nature. The steel sculpture's connections are simply illustrative examples. The connections on the steel sculpture were not designed to satisfy any particular strength or serviceability limit state of the AISC Specification. Also, the narratives in the guide give only cursory descriptions, with limited practical engineering information.

The main goals of this Guide are to address the issues of visualization, load paths, and limit states associated with steel connections. The guide is intended to be a teaching tool and supplement the AISC Manual of Steel Construction LRFD 3rd Edition. It is intended to demonstrate to the student the intricacies of analysis and design for steel connections.

Chapters in this guide are arranged based on the types of connections. Each connection is described discussing various issues and concerns regarding the design, erectability, and performance of the specific connection. Furthermore, every connection that is illustrated by the steel sculpture has multiple photos and a data figure. The data figure has tables of information and CAD-based illustrations and views. Each figure has two tables, the first table lists the applicable limit states for the particular connection, and the second table provides a list of notes that are informative statements or address issues about the connection. The views typically include a large isometric view that highlights the particular location of the connection relative to the steel sculpture as well as a few orthogonal elevations of the connection itself. In addition to the simple views of the connections provided in the figures, also included are fully detailed and dimensioned drawings. These views were produced from the full 3D CAD model developed from the original, manually drafted shop drawings of the steel sculpture.

The guide covers the most common types of steel connections used in practice, however more emphasis has been placed on shear connections. There are more shear connections on the steel sculpture than all other types combined. In addition to the shear connection descriptions, drawings, and photos, MathCAD® worksheets are used to present some design and analysis examples of the shear connections found on the steel sculpture.

The illustrations, photos, and particularly the detail drawings that are in the teaching guide tend to aid visualization by students. However, the 3D CAD model is the primary means by which the student can learn to properly visualize connections. The 3D model has been developed in the commonly used AutoCAD "dwg" format. The model can be loaded in AutoCAD or any Autodesk or other compatible 3D visualization application. The student can rotate, pan and zoom to a view of preference.

The issue of limit states and load paths as they apply to steel connections is addressed by the illustrations and narrative text in the guide. To facilitate a more inclusive understanding of shear connections, a series of MathCAD® worksheets has been developed to perform complete analysis for six different types of shear connections. As an analysis application, the worksheets require load and the connection properties as input. Returned as output are two tables. The first table lists potential limit states and returns either the strength of the connection based on a particular limit state or "NA" denoting the limit state is not applicable to that connection type. The second table lists connection specific and general design checks and returns the condition "OK" meaning a satisfactory value, "NA" meaning the check is not applicable to that connection type, or a phrase describing the reason for an unsatisfactory check (e.g.

"Beam web encroaches fillet of tee"). The student is encouraged to explore the programming inside these worksheets. Without such exploration, the worksheets represent "black boxes." The programming must be explored and understood for the benefits of these worksheets to be realized.

A complete user's guide for these worksheets can be found in Appendix A. Contained in the guide is one example for each type of shear connection illustrated by the steel sculpture. Each example presents a simple design problem and provides a demonstration of the use of the worksheet.

Appendix B provides a list of references that includes manuals and specifications, textbooks, and AISC engineering journal papers for students interested in further information regarding structural steel connections.

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CHAPTER 1

The Steel Sculpture

As a structure, the steel sculpture consists of 25 steel members, 43 connection elements, over 26 weld groups, and more than 144 individual bolts. As a piece of art, the steel sculpture is an innovative aesthetic composition of multiform steel members, united by an assortment of steel elements demonstrating popular attachment methods.

At first glance, the arrangement of members and connections on the steel sculpture may seem complex and unorganized. However, upon closer inspection it becomes apparent that the position of the members and connections were methodically designed to illustrate several specific framing and connection issues. The drawings, photos, and illustrations best describe the position of the members and connections on the steel sculpture on subsequent pages. The drawings are based on a 3D model of the sculpture. There are four complete elevations of the sculpture followed by thirteen layout drawings showing each connection on the sculpture. Each member and component is fully detailed and dimensioned. A bill of material is included for each layout drawing.

In general terms, the steel sculpture is a tree-like structure in both the physical and hierarchical sense. A central column, roughly 13 ft tall is comprised of two shafts spliced together 7 ft -6in. from the base. Both shafts are W12-series cross-sections. The upper, lighter section is a W12×106 and the lower, heavier section is a W12×170. Each shaft of the column has four faces (two flanges and two sides of the web) and each face is labeled according to its orientation (North, South, East, or West). A major connection is made to each face of the upper and lower shafts. Seven of the eight faces have a girder-to-column connection while the eighth face supports a truss (partial). Two short beams frame to the web of each girder near their cantilevered end. Thus, the steel sculpture does indeed resemble a tree "branching" out to lighter and shorter members.

The upper shaft girder-to-column connections and all of the beam-to-girder connections are simple shear connections. The simply supported girder-to-column connections on the upper shaft are all propped cantilevers of some form. The east-end upper girder, (Girder B8)* is supported by the pipe column that acts as a compression strut, transferring load to the lower girder (Girder B4). A tension rod and clevis support the upper west girder (Girder B6). The channel shaped brace (Beam B5A) spans diagonally across two girders (Girder B5 and Girder B8). This channel is supported by the south girder (Girder B5) and also provides support for the east girder (Girder B8).

The enclosed CD contains 18 CAD drawings of the steel connections sculpture which may serve as a useful graphical teaching aid.

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* The identification/labeling scheme for beams, columns, and girders with respect to the drawings included in this document is as follows:

- Columns have two character labels. The first character is a "C" and the second character is a number.

- Girders have two character labels. The first character is a "B" and the second character is a number.

- Beams have three character labels. Like girders, the first character is a "B" and the second character is a number. Since two beams frame into the web of each girder, the third character is either an "A" or "B" identifying that the beam frames into either the "A" or "B" side of the girder.

- Plates have two character labels that are both are lower-case letters. The first character is a "p".

- Angles have two character labels that are both lower-case letters. The first character is an "a".

GENERAL NOTES (U.N.0.)

1. W Shapes – ASTM A992.
   Steel Pipe – ASTM A53.
   Other Shapes – ASTM A36.

2. Structural Bolts – $7/8$"$\varnothing$ ASTM A325.
   Structural Anchor Rods – ASTM F1554 grade 36.

3. Drill or punch all bolt holes as STD holes.

4. Shop coat prime to 2 mils dry thickness after fabrication, except do not
   paint the area (indicated by shade) where field weld will be applied.

5. Top coat paint color to be selected by owner and applied in the field.

ABBREVIATIONS

$\varnothing$	DIAMETER
AR	ANCHOR ROD
AISC	AMERICAN INSTITUTE OF STEEL CONSTRUCTION
BM	BEAM
B.S.	BOTH SIDES
BT	BOLT
CL	CLEARANCE
COL	COLUMN
EL; ELEV.	ELEVATION
F.S.	FAR SIDE
GALV.	GALVANIZED
H.S.	HIGH STRENGTH
N.S.	NEAR SIDE
N.T.S.	NOT TO SCALE
PL	PLATE
SJI	STEEL JOIST INSTITUTE
STD	STANDARD
T & B	TOP & BOTTOM
T.B.D.	TO BE DETERMINED
U.N.O.	UNLESS NOTED OTHERWISE

NORTH ELEVATION

B9

C2

B7

1⅜" DIA.
ROD

10"

¼

B8A

B8B

¼

C2

C3

C1

B4A

B4

B3

B2

B2B

B9

½" ø
BOLT

1⅜" ø ROD

1¾" ø PIN

TOP ELEV. SAME

B5

B6

B5A

B8

C3

TOP ELEV. SAME

⅜

⅜
⅜

B2A

B2

B1

B4

B4B

⅜

SOUTH ELEVATION

B9

CUT B9A IN
FIELD TO FIT

B9B   B9A

C2

B8

B5

B7

¼

BOTTOM
ELEV. SAME

TOP ELEV.
SAME

B1A

B1

⅜

B3

B3A

⅜   ⅜

B5

⅜

C1

EAST ELEVATION

B9

2 – ½
DIA. BOLTS

1⅜″ DIA.
ROD

C2

B9A

B9B
(B.S.)

3/16
3/16 TYP.

B7

B6

B5

SHIM

B3B

B3

B2

3/4″ ∅ ERECTION
BOLT

B1B

B1

C1

WEST ELEVATION

CHAPTER 2

Limit States

Structural design is based on the concept that all structural members are designed for an appropriate level of strength and stiffness. Strength relates to safety and is essentially the capacity of a structure or member to carry a service or ultimate design load. Stiffness is typically associated with serviceability. Serviceability is concerned with various performance criteria of a structure or member during service loading and unloading.

For acceptable safety and satisfactory performance of the structure, the load and resistance factor design philosophy uses statistically based load and resistance factors to modify nominal resistance and service loads. Load factors increase the nominal loads, and resistance factors (also known as ϕ factors) reduce the nominal resistance of a member. The load factors account for the possibility of higher than anticipated loads during service. The resistance factors account for the possibility of lower than anticipate strength. Design loads and design strengths are obtained when the service loads and nominal resistance values are multiplied by the appropriate load and resistance factors.

Structural members must be proportioned with sufficient design strength to resist the applicable design loads. In addition to strength, an appropriate stiffness level must be provided to satisfy applicable serviceability requirements. When loads exceed the design strength or serviceability requirements, a limit state has been reached. A limit state is the condition where the structure or member is functionally inadequate. Structural elements tend to have several limit states, some based on strength and others based on serviceability.

A single connection might include a large number of structural members and several fastener groups. However, the basic components of connections are the fastening system and the attached plies of material. Thus, strength-based limit states for connections can be based on either the material (members) or the fasteners. Connection strength limit states of both the fasteners and the plies of material result from tension, shear, or flexural forces.

Each strength limit state has a particular failure path across, through or along the element or member. The failure path is the line along which the material yields or ruptures. Serviceability limit states typically involve providing an appropriate amount of stiffness or ductility in a structural element. The serviceability requirements depend on the

intended function of the member or element under consideration.

A connection may have many or only a few limit states. The controlling limit state can be either strength related or based on serviceability criteria. The controlling strength limit state is the specific condition that has the lowest resistance to the given design load. Initially, most designers tend to proportion elements based on strength requirements then check that the particular design meets applicable serviceability limit states, refining if necessary. The inverse design procedure is also acceptable: design for serviceability and then check strength. Regardless of the methodology the controlling limit state dictates the optimal design.

The following pages have descriptions and figures that explain the general applicability of the more common connection limit states. The applicability of any given limit state is dependent upon the specific connection geometry and loading. These figures are only a guide and are not meant to represent any and all possible combinations of limit states.

(BSR) **BLOCK SHEAR RUPTURE**

Block shear rupture is a limit state in which the failure path includes an area subject to shear and an area subject to tension. This limit state is so named because the associated failure path tears out a "block" of material. Block shear can

Figure 2-1.   Block Shear Rupture Limit State
(Photo by J.A. Swanson and R. Leon, courtesy of Georgia Institute of Technology)

occur in plies that are bolted or in plies that are welded. The only difference between the treatments of either the bolted or welded block shear limit state is that in the absence of bolt holes, the gross areas are equal to the net areas. Figure 2-1 shows the condition of the gusset plate well after the block shear rupture limit state has occurred.

(BB)   BOLT BEARING

Bolt bearing is concerned with the deformation of material at the loaded edge of the bolt holes. Bearing capacity of the connection is influenced by the proximity of the bolt to the loaded edge. Bolt bearing is applicable to each bolted ply of a connection. The AISC specification contains two design equations, one equation is based on strength (when deformation around bolt holes is not a consideration) and the other is based on serviceability (when deformation around the bolt holes is a design consideration).

(BS)   BOLT SHEAR

Bolt shear is applicable to each bolted ply of a connection that is subjected to shear. The shear strength of a bolt is directly proportional to the number of interfaces (shear planes) between the plies within the grip of the bolt that a single shear force is transmitted through. Single shear occurs when the individual shear force is transmitted through bolts that have two plies within the grip of the bolt. Additional plies further distribute the shear force. Three plies of material represent two shear planes, thus the bolt or bolt group is in double shear and has effectively twice the strength as single shear. It is important to realize that double shear, triple shear, etc. requires an individual shear force vector evenly distributed across the plies. There may be a

condition where there are indeed two or more shear planes, but the forces are not evenly distributed. (e.g. double-sided connections)

If the load path does not pass through the center of gravity of a bolt group, then the load is considered eccentric. Eccentrically loaded bolt groups are subject to a moment force that induces either additional shear (for in-plane loads) or combined shear and tension (for out-of-plane loads). In reality most connections possess some degree of eccentricity, however some of these eccentricities are small and are commonly neglected.

(BT)   BOLT TENSION FRACTURE

If bolts are subject to loading along their length then the bolt is loaded in tension. Bolts that fail in tension will do so within the threaded portion of the bolt, through one of the
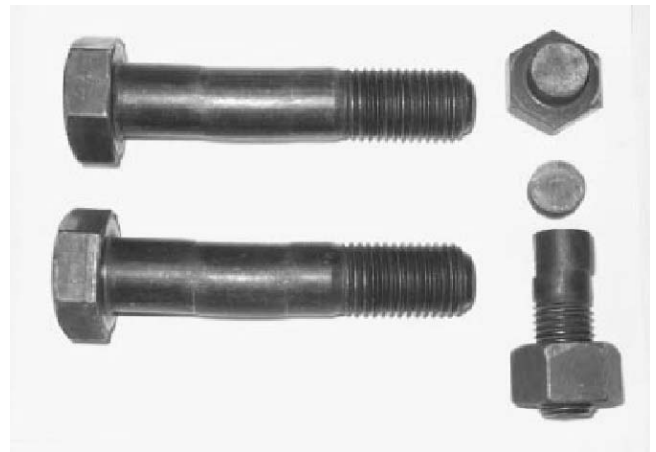

*Figure 2-3.   Bolt Shear Limit State
(Photo by P.S. Green)*

Figure 2-2.   *Bolt Bearing Limit State
(Photo by J.A. Swanson and R. Leon,
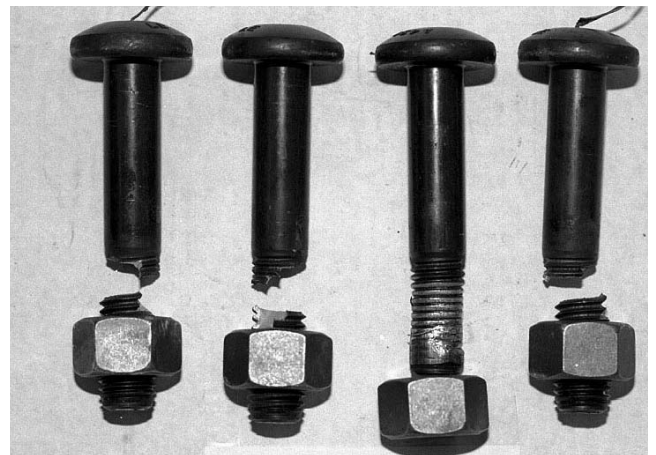courtesy of Georgia Institute of Technology)*

Figure 2-4. *Bolt Tension Fracture Limit State
(Photo by J.A. Swanson and R. Leon,
courtesy of Georgia Institute of Technology)*

roots of the threads. This coincides with the least cross-sectional area.

CONCENTRATED FORCES

(FLB) **FLANGE LOCAL BENDING**

(WCB) **WEB COMPRESSION BUCKLING**

(WC) **WEB CRIPPLING**

(WLB) **WEB LOCAL BUCKLING**

(WLY) **WEB LOCAL YIELDING**

Sometimes forces that are transferred from one member to another create localized deformation (yielding) or buckling. The applicable limit states depend on the specific connection geometry. The limit states for concentrated forces most often occur in seated connections and moment connections. For example, when the supported beam is coped, (i.e. flange material has been removed) the remaining web may be susceptible to web local buckling.

For seated connections, the outstanding angle leg on the seat provides a bearing area for the bottom flange of the supported beam. This bearing area creates a concentrated reaction at the end of the beam. The web of the supported beam is susceptible to web crippling and web local yielding.

Since most moment connections provide continuity between the supporting and supported members, the flanges of the supported member transfer concentrated tension and
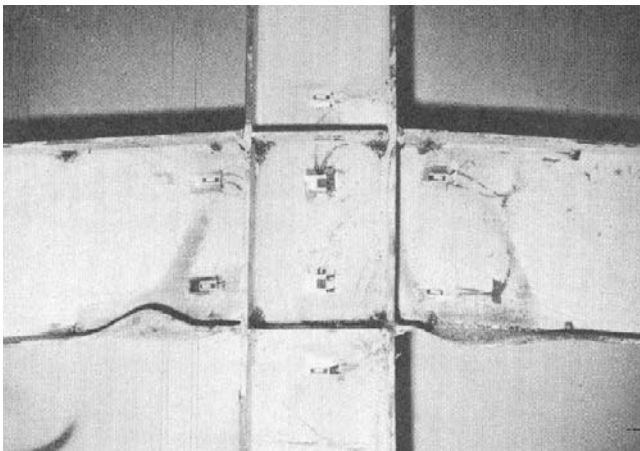

Figure 2-6. Web Crippling Limit State
(Photo by T. Murray, Virginia Tech)

Figure 2-5. Flange Local Bending Limit State
(Beedle, L.S., Christopher, R., 1964)

Figure 2-7. Web Local Buckling Limit State
(SAC Project)

compression forces to the supporting member. Flange local bending, web local yielding, web crippling and web compression buckling limit states must be investigated.

FY   FLEXURAL YIELDING

When a beam is coped, the reduced section modulus of the remaining beam cross section may significantly reduce the flexural strength of the member. Other instances of flexural yielding are flexure of the stem of a tee shape in a shear tee connection and bending of the outstanding angle leg of an unstiffened seated connection.

PA   PRYING ACTION

Prying action is a phenomenon in which additional tension forces are induced in the bolts due to deformation of the connection near the bolt. Flexibility of the connected parts within the grip of the bolts creates these additional tension forces.

SY   SHEAR YIELDING AND SHEAR RUPTURE   SR

Most connections are subjected to the shear component of loading. Even moment connections must have provisions for shear transfer. Thus, those elements in the connection that are subject to shear forces must be investigated for shear yielding and shear rupture. Both limit states will apply regardless of fastening method (bolt or weld). For welded plies, without bolt holes, shear yielding will usually control over shear rupture. (The net area of welded plies without bolt holes is equal to the gross area. If the ratio of yield strength to ultimate tensile strength is less than 1.2, then shear rupture will generally control).

*Figure 2-9 Tee Stem Deformation
(Astaneh, A., Nader, M.N., 1989)*

*Figure 2-8   Web Local Yielding Limit State
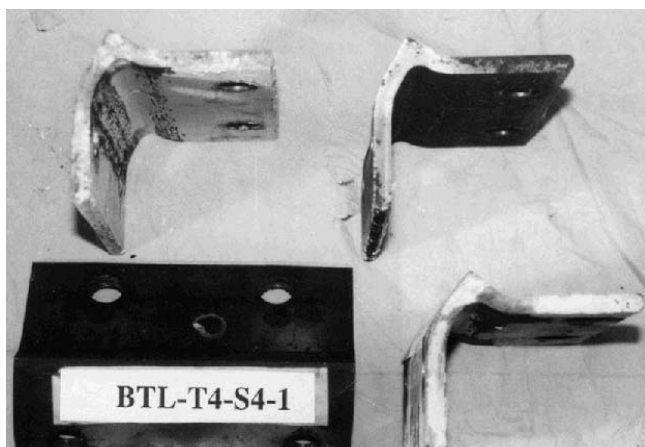(SAC Project)*

*Figure 2-10   Seat Angle Deformation
(Yang, W.H. et al., 1997)*

Shear yielding is a ductile limit state; it is a function of the gross shear area of the element. The failure path associated with shear yielding is linear in the direction of load from the top edge of the element to the bottom edge and through the thickness of the ply.

Shear rupture is an ultimate limit state; it is a function of the net shear area of the element. The failure path associated with shear rupture is also linear, in the direction of load from the top edge of the element to the bottom edge and through the thickness of the ply. If both flanges of the supported member are coped, then a potential shear failure path on the beam is present and shear yielding and shear rupture must be investigated for this member.

TENSION YIELDING AND TENSION RUPTURE

The tension yielding limit state is a function of the gross cross-sectional area of the member subjected to tension load. The tension rupture mode is a limit state that is a function of the effective net area. The net area is the reduced gross area due to bolt holes or notches. This net area is further reduced to account for the effects of shear lag. Shear lag occurs when the tension force is not evenly distributed through the cross sectional area of a member. Certain geometric areas of a section may have higher localized stresses. Shear lag often occurs in angle members when they are used as struts. The fastening (bolting or welding) is generally
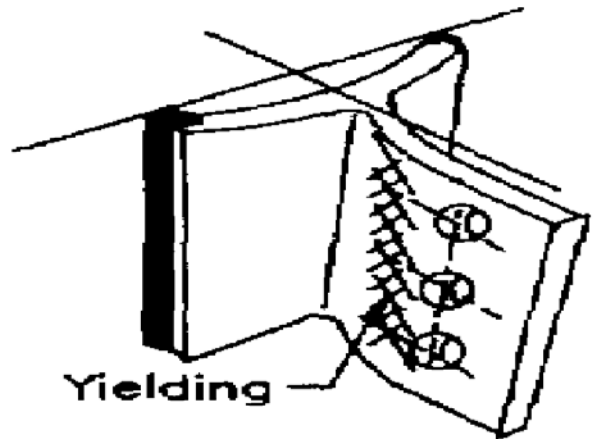

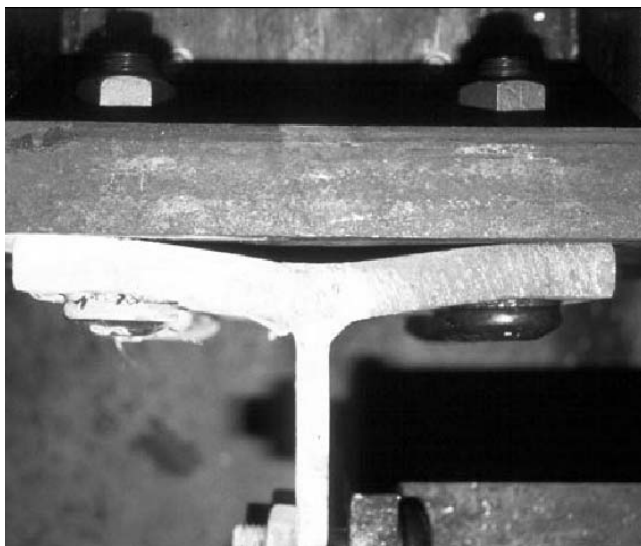Figure 2-12. *Shear Yielding Limit State (Astaneh, A. and Nader, M.N. 1989)*

Figure 2-11. *Prying Action Limit State (Photo by J.A. Swanson and R. Leon, courtesy of Georgia Institute of Technology)*
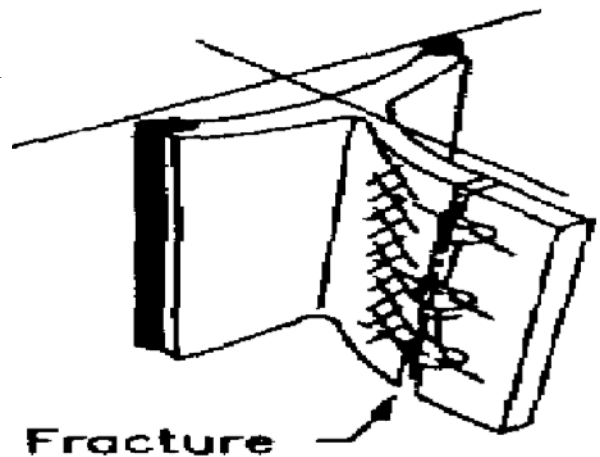

Figure 2-13. *Shear Rupture Limit State (Astaneh, A. and Nader, M.N., 1989)*

made along only one leg of the angle. When the angle is loaded in tension the leg that is fastened has a disproportionate share of the tension load. This unbalance causes a shear force to lag across the section.

(W) WELD SHEAR

Weld shear is applicable to each welded ply of a connection. The failure mode for fillet welds is always assumed to be a shear failure on the effective throat of the weld. In a similar fashion as bolt shear, if the load path does not pass through the center of gravity of a weld group, then the load is considered eccentric. Eccentrically loaded weld groups are subject to a moment that tends to induce either additional shear (for in-plane loads) or combined shear and tension (for out-of-plane loads).

(WS) WHITMORE SECTION YIELDING / BUCKLING

Whitmore section yielding or buckling is a limit state that applies to bolted and welded gusset plates and similar fittings that are much wider than the pattern of bolts or welds within them. The stress distribution through the ends of members that are attached to the gusset is complex. This limit state involves either the yielding or buckling of plate material near the ends of the attached members. The Whitmore method of analysis assumes the member force is uniformly distributed over an effective area. This effective area is determined by multiplying the gusset plate thickness by an effective length that is defined from the projection of 30-degree lines on each side of the "strut" member that is connected to the gusset plate. The projection is assumed to originate at either the first row of bolts on the plate or the origin of the weld. The projection is assumed to terminate at the plane that passes through the last row of bolts or at the end of the welds. The 30-degree projection lines form a trapezoid, and the effective length is assumed as the base dimension of this trapezoid.
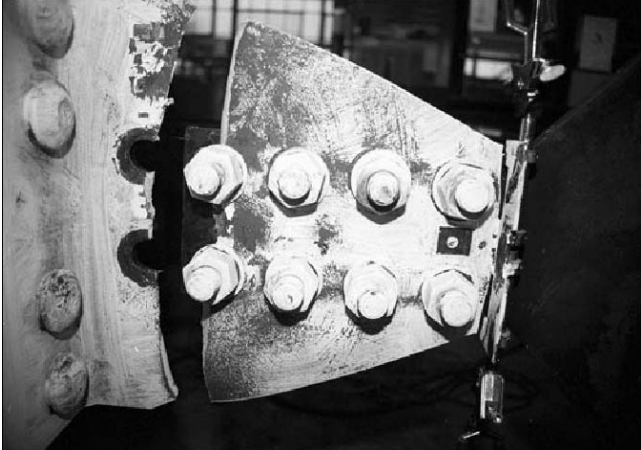
Figure 2-14. *Tension Fracture Limit State (Photo by J.A. Swanson and R. Leon, courtesy of Georgia Institute of Technology)*
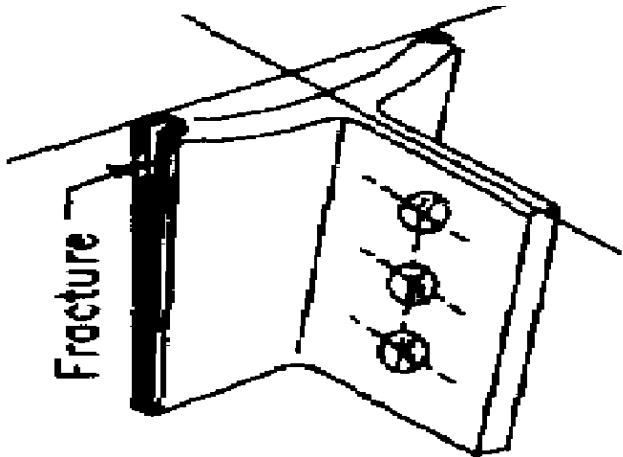
Figure 2-15. *Weld Shear Limit State (Astaneh, A. and Nader, M.N., 1989)*
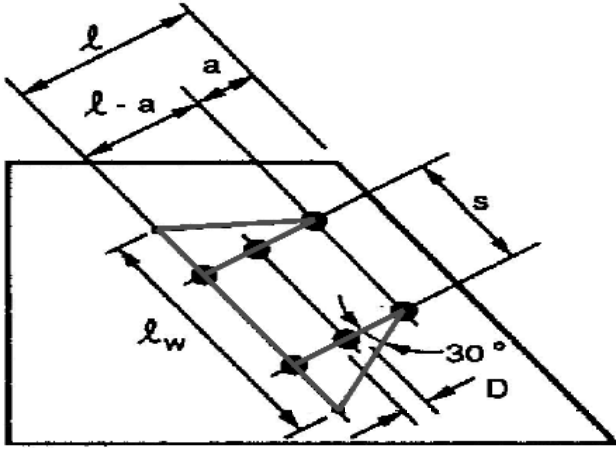
Figure 2-16. Whitmore Section Yielding/Buckling Limit State (Beedle, L.S. and Christopher, R., 1964)

Figure 2-17. Shear Connection; Potential Limit States

Figure 2-18. Moment Connection; Potential Limit States

Figure 2-19. Tension Connection (Hanger Connection Potential Limit States

Figure 2-20. *Tension Connection (Gusset Plate); Potential Limit States*

CHAPTER 3
Joining Steel Members

In current construction practice, steel members are joined by either bolting or welding. When fabricating steel for erection, most connections have the connecting material attached to one member in the fabrication shop and the other member(s) 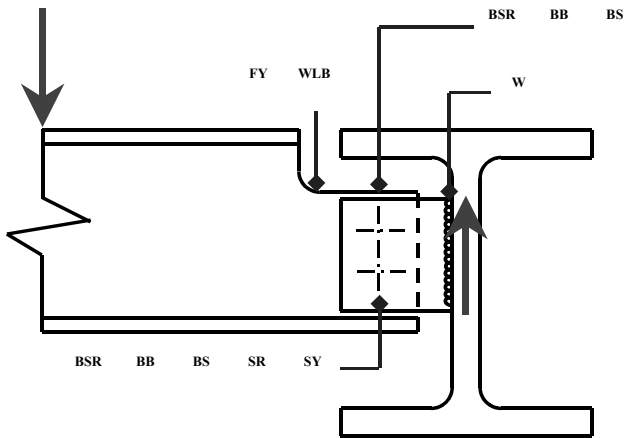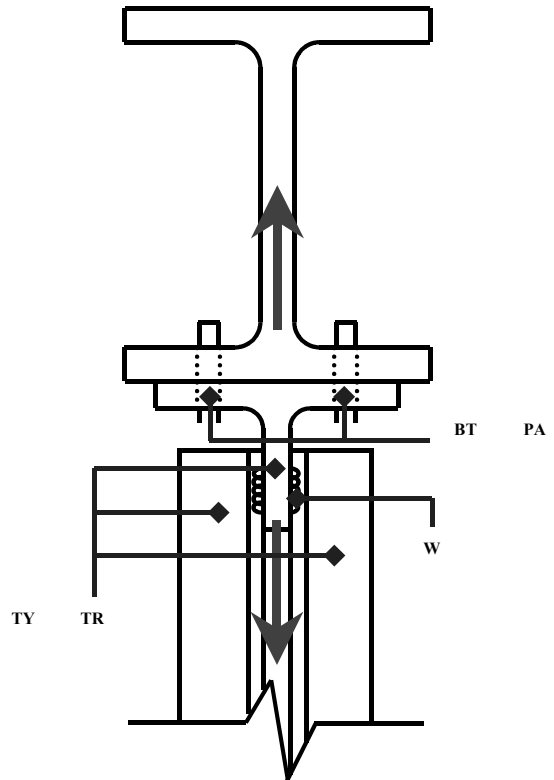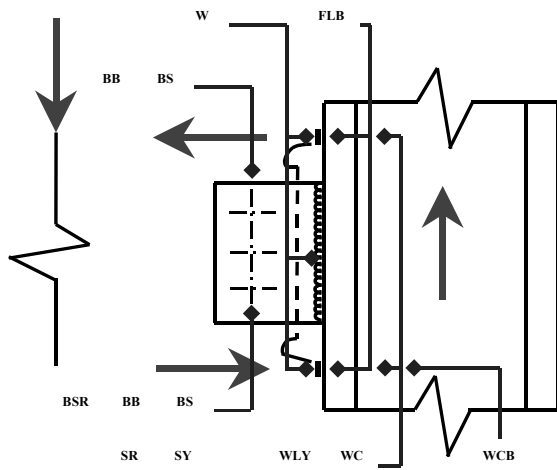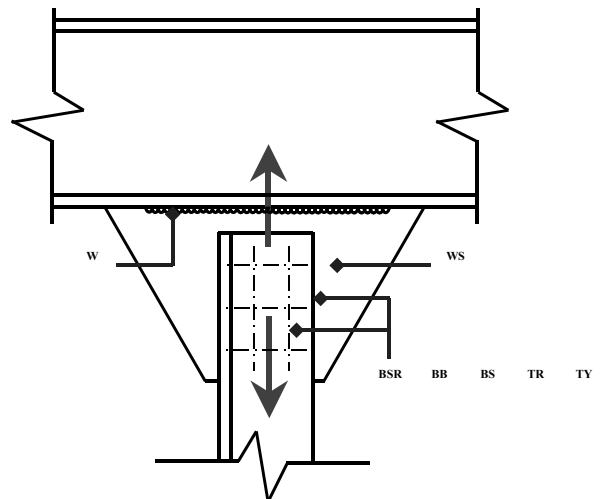attached in the field during erection. This helps simplify shipping and makes erection faster. Welding that may be required on a connection is preferably performed in the more-easily controlled environment of the fabrication shop. If a connection is bolted on one side and welded on the other, the welded side will usually be the shop connection and the bolted connection will be the field connection.

The use of either bolting or welding has certain advantages and disadvantages. Bolting requires either the punching or drilling of holes in all the plies of material that are to be joined. These holes may be a standard size, oversized, short-slotted, or long-slotted depending on the type of connection. It is not unusual to have one ply of material prepared with a standard hole while another ply of the connection is prepared with a slotted hole. This practice is common in buildings having all bolted connections since it allows for easier and faster erection of the structural framing.

Welding will eliminate the need for punching or drilling the plies of material that will make up the connection, however the labor associated with welding requires a greater level of skill than installing the bolts. Welding requires a highly skilled tradesman who is trained and qualified to make the particular welds called for in a given connection configuration. He or she needs to be trained to make the varying degrees of surface preparation required depending on the type of weld specified, the position that is needed to properly make the weld, the material thickness of the parts to be joined, the preheat temperature of the parts (if necessary), and many other variables.

STRUCTURAL BOLTING

Structural bolting was the logical engineering evolution from riveting. Riveting became obsolete as the cost of installed high-strength structural bolts became competitive with the cost associated with the four or five skilled tradesmen needed for a riveting crew. The Specification for Structural Joints Using ASTM A325 or A490 Bolts, published by the Research Council on Structural Connections (RCSC, 2000) has been incorporated by reference into the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings. Many of the bolting standards are based on work reported by in the Guide to Design Criteria for Bolted and Riveted Joints, (Kulak, Fisher and Struik, 1987).

High strength bolts can be either snug tightened or pretensioned. When bolts are installed in a snug-tightened condition the joint is said to be in bearing as the plies of joined material bear directly on the bolts. This assumes that the shank of the bolt provides load transfer from one ply to the next through direct contact. Bearing connections can be specified with either the threads included (N) or excluded (X) from the shear plane. Allowing threads to be included in the shear planes results in a shear strength about 25% less than if the threads are specified as excluded from the shear plane(s). However, appropriate care must be taken to specify bolt lengths such that the threads are excluded in the as-built condition if the bolts are indeed specified as threads excluded.

In pretensioned connections, the bolts act like clamps holding the plies of material together. The clamping force is due to the pretension in the bolts created by properly tightening of the nuts on the bolts. However, the load transfer is still in bearing like for snug-tightened joints.

The initial load transfer is achieved by friction between the faying or contact surfaces of the plies of material being joined, due to the clamping force of the bolts being normal to the direction of the load. For slip-critical joints, the bolts are pretensioned and the faying surfaces are prepared to achieve a minimum slip resistance. The reliance on friction between the plies for load transfer means that the surface condition of the parts has an impact on the initial strength of slip-critical connections. The strength of slip-critical connections is directly proportional to the mean slip coefficient. Coatings such as paint and galvanizing tend to reduce the mean slip coefficient.

The two most common grades of bolts available for structural steel connections are designated ASTM A325 and ASTM A490. The use of A307 bolts is no longer that common except for the ½-in. diameter size where they are still sometimes used in connections not requiring a pretensioned installation or for low levels of load. A307 bolts have a 60 ksi minimum tensile strength. A325 and A490 bolts are designated high-strength bolts. A325 bolts have a 120 ksi minimum tensile strength and are permitted to be galvanized, while A490 bolts have a 150 ksi minimum tensile strength, but are not permitted to be galvanized due to hydrogen embrittlement concerns. High strength bolts are available in sizes from ½- to 1½-in. diameters in ⅛ in. increments and can be ordered in lengths from 1½ to 8 inches in ¼ in. increments.

When a pretensioned installation is required, four installation methods are available: turn-of-the-nut, calibrated wrench, twist off bolt, and direct tension indicator methods. The turn-of-the-nut method involves first tightening the nut to the snug tight condition, then subsequently turning the nut a specific amount based on the size and grade of the bolt to develop the required pretension. The calibrated wrench method involves using a torque applied to the bolt to obtain the required level pretension. A torque wrench is calibrated to stall at the required tension for the bolt. Twist-off bolts have a splined end that twists off when the torque corresponding to the proper pretension is achieved. ASTM F1852 is the equivalent specification for A325 "twist-off" bolts. Currently, there is no ASTM specification equivalent for A490 tension control bolts. Direct tension indicators (DTIs) are special washers with raised divots on one face. When the bolt is installed, the divots compress to a certain level. The amount of compression must then be checked with a feeler gage.

WELDING

Welding is the process of fusing multiple pieces of metal together by heating the metal to a liquid state. Welding can often simplify an otherwise complicated joint, when compared to bolting. However, welds are subject to size and length limitations depending on the thickness of the materials and the geometry of the pieces being joined. Furthermore, welding should be preferably performed on bare metal. Paint and galvanizing should be absent from the area on the metal that is to be welded.

Guidelines for welded construction are published by the American Welding Society (AWS) in AWS D1.1 Structural Welding Code-Steel. These provisions have been adopted by the AISC in the Load and Resistance Factor Design Specification for Structural Steel Buildings.

Several welding processes are available for joining structural steel. The selection of a process is due largely to suitability and economic issues rather than strength. The most common weld processes are Shielded Metal Arc Welding (SMAW), Gas Metal Arc Welding (GMAW), Flux Core Arc Welding (FCAW), and Submerged Arc Welding (SAW).

SMAW uses an electrode coated with a material that vaporizes and shields the weld metal to prevent oxidation. The coated electrode is consumable and can be deposited in any position. SMAW is commonly referred to as stick welding.

GMAW and FCAW are similar weld processes that use a wire electrode that is fed by a coil to a gun-shaped electrode holder. The main difference between the processes is in the method of weld shielding. GMAW uses an externally supplied gas mixture while FCAW has a hollow electrode with flux material in the core that generates a gas shield or a flux shield when the weld is made. GMAW and FCAW can be deposited in all positions and have a relatively fast deposit rate compared to other processes.

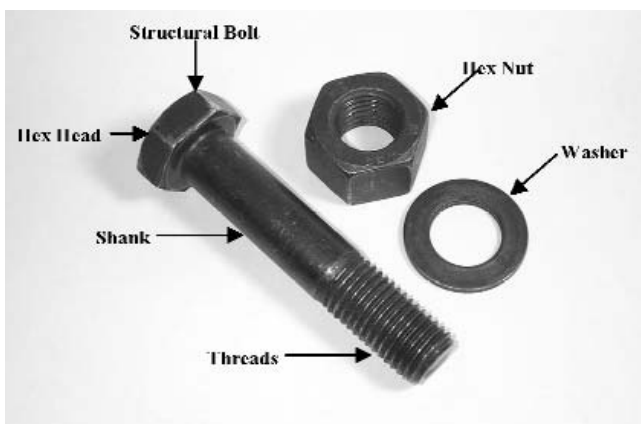*Figure 3-2. Direct Tension Indicators and Feeler Gages*

Figure 3-1. Structural Fastener - Bolt, Nut and Washer

Figure 3-3. Structural Fastener - Twist-off Bolt

In SAW welding, a consumable electrode is submerged below a blanket of granular flux. The flux protects and enhances the resulting weld. SAW tends to produce high quality welds that are strong and ductile. The major limitation of this process is that weld can only be deposited in the flat position due to the granular flux used. This process is frequently used for the web-to-flange connections of plate girders. The SAW process is most often found automated in the better-controlled conditions of shop welding operations. For engineers, it is important to realize that the effective throat dimension for the SAW process is calculated differently than for the other processes. Since the SAW process produces higher quality welds with deeper penetration, the effective throat is permitted to be equal to the weld throat size if the weld is less than $3/8$ in. For larger welds the effective throat for SAW welds is the minimum distance from the root to face of the weld plus 0.11 in. For the other processes the effective throat is taken as the minimum distance from the root to the face of the weld (for equal legs: $0.707 \times$ leg length).

There are four types of welds: fillet, groove, plug, and slot. Fillet and groove welds make up the majority of all structural welds, therefore only those types will be discussed here. There are five types of structural joints that can be made using either fillet or groove welds. These are called butt, lap, tee, corner, or edge. The welds can be placed in any of four positions depending on the configuration and location of the joint: flat, horizontal, vertical, and overhead. Whenever possible, it is beneficial to make a weld in either the horizontal or flat position for ease of workmanship and cost.

Fillet welds are by far the most common type of weld used in welded construction. In general, fillet welds are economical, easy to fabricate, and require very simple preparation of the materials being joined. Joint types are either lap or tee. Fillet weld strength is directly proportional to its length and throat dimension. The volume of weld material, and therefore the cost, is proportional to the square of the weld leg. It is more economical to use smaller and longer fillet welds with small legs rather than shorter fillet welds with large legs. There is an additional economic advantage if the weld can be made in one pass. The largest weld size that can be deposited in one pass in the horizontal or flat position is normally $5/16$ in. Multiple passes require more time and more weld metal. Due to labor costs, weld metal, by weight, is the most expensive part of a structure.

Groove welds are typically used when the plies are aligned parallel and lie in the same plane (i.e. no material overlap), as in a butt joint, or to make a tee, corner, or edge joint. Groove welds that extend through the full thickness of the materials being joined are called complete-joint-penetration groove welds. When the welds do not extend completely through the thickness, they are called partial-joint-penetration groove welds. Groove welds, particularly complete-joint-penetration groove welds, are potentially expensive. The metal pieces being joined must be prepared by shaping the edges. Additional metal is often used in the form of backing or extension bars and runoff tabs to help contain the weld metal. If the structure is loaded cyclically, the backing bars and runoff tabs must be removed and the surfaces finished smooth. This obviously adds significant labor costs to the already expensive weld. Finally, when groove welds are used for beam flange connections, small access holes (known as weld access holes) need to be cut in the web just below and above the top and bottom flanges respectively to allow access to make the weld (bottom) and to allow placement of the backing bar (top).

Weld material strength should be matched to the base metal so that the resulting weld is stronger than the pieces of metal being joined. Regardless of the process or type of welds, shear is always the controlling limit state. Weld strength is based on either the shear strength of the weld or shear strength of the base metal, through the thickness. In the AISC Manual, the shear connections have tables for standard elements and weld lengths. These tabulated strength values assume E70 electrodes and have an associated minimum thickness based on shear strength of the weld matching the shear strength of the base metal. If less than the minimum material thickness is present or the weld is not made with an E70 electrode, the tabulated strength must be reduced by the ratio of thickness provided to minimum thickness or by electrode used to E70 electrode strength (70 ksi), respectively.

CHAPTER 4

Simple Shear Connections

Simple shear connections are assumed to have little or no rotational resistance. They are assumed to carry only the shear component of the load and are idealized as pins or rollers for design. Therefore, no moment forces are assumed transmitted by the connection from the supported member to the supporting member. The attachment of a shear connection may be made to the web of the supported beam, usually with the flanges unconnected. The exception is the seated connection where the connection is made to the flanges of the supported beam.

Experimentally it has been shown that shear connections possess some amount of rotational restraint. For design purposes, ignoring this resistance produces a conservative result. The majority of the rotational flexibility of most shear connections is achieved in the deformation of the connection element (plate, angle, tee, etc.) or through slotted or oversized holes. The deformation of the fasteners, if it is a bolted connection, or the welds, if it is a welded connection usually adds little to the overall connection flexibility.

The goal for shear connections is to have both adequate strength and sufficient rotational ductility. Shear connection elements are typically designed using thin and/or mild yield strength materials (i.e. A36) to provide rotational flexibility in excess of what the supported member requires.

Many shear connection elements can be either bolted on the supported side and welded on the supporting side, or welded on the supported side and bolted on the supporting side, or all-bolted or all-welded. However, from an erectability point of view, all-welded shear connections are usually impractical. Since it is a common practice to weld shop attachments and bolt field attachments, many shear connections are bolted on one side (to either the supported member or the supporting member) and welded on the other.

When designing shear connections, the pin is assumed to be located at the most flexible part of the connection. The stiffness and strength of some connections depend on whether or not the supported member is considered "flexible" or "rigid". The terms "flexible" and "rigid" as they refer to the supporting side of a connection are subjective and somewhat open to interpretation. In general, a flexible support is one in which the supporting side of the element is attached to either a girder or column web where deformation of the web contributes to rotational flexibility (if the support is flexible rotation occurs largely in the supporting member). A rigid support is typically connected to the center of a column flange on the supporting side (if the support is rigid rotation occurs largely in the connection element).

These are qualitative definitions and it is up to the discretion of the engineer as to whether the support should be treated as flexible or rigid.

SHEAR CONNECTION EXAMPLES & MATHCAD® WORKSHEETS

Shear connections are the workhorse of structural steel framing. Today, the majority of structural steel building connections are shear connections. Even most moment connections integrate a shear connection to carry the shear component of the beam reaction. Individually, shear connections exhibit relatively complex behavior and tend to have a significant number of limit states that need to be checked.

The AISC Manual has tables that list design strengths for most shear connections. Although the tables provide a quick and simple way to design shear connections, there are a few points that must be noted concerning the use of the design tables. The first is that the tables list strengths based on assumptions of standardized connection geometry. The tables have limited effectiveness for analysis or design of connections with unusual geometry. Second, and most significant, is that the tables list only the controlling strength of the connection. The tables provide little information as to which limit state controls the design. Additionally, other applicable limit states may need to be checked. This prevents any sort of comparison of the applicable limit states for the connection.

For these reasons, MathCAD® worksheets were developed for six types of shear connections: Double-Angle, Shear End-Plate, Unstiffened Seat, Single Plate, Single-Angle and Shear Tee. The worksheets allow the user to see how a change in any particular connection parameter (i.e. bolt size, element thickness, material strength, etc.) affects the strength and/or controlling limit state. These worksheets are included on the companion CD that came with this guide.

For specific instructions and information regarding the use and installation of the MathCAD® worksheets see Appendix A. For instructions and information regarding the MathCAD® software application, refer to the MathCAD® User's Guide.
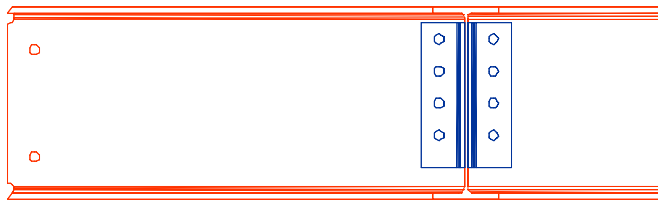
Figure 4-1 **Shear Connection: All-Bolted Double-Angle** Girder B1 / Beam B1B	Limit States
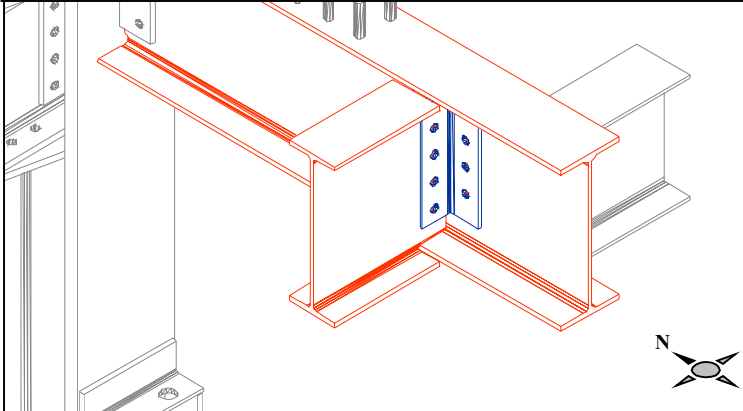
Figure 4-1
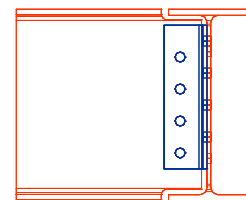**Shear Connection: All-Bolted Double-Angle**
Girder B1 / Beam B1B

Limit States

Block Shear Rupture
Bolt Bearing
Bolt Shear
Flexural Yielding
Local Web Buckling
Shear Rupture
Shear Yielding

N

West Elevation

South Elevation

Notes

- Girder B1 supports Beam B1B by an all-bolted, double-angle connection.

- These double-angles are field bolted to the supporting girder and shop bolted to the supported beam. This eliminates "knifed" erection. (Lowering the supported beam web into place between the angles).

- The offset bolt rows between the in-plane and outstanding angle legs provide better entering and tightening clearances.

- Since both of the members are the same depth, the beam is double coped to accommodate the flanges of the girder.

The steel sculpture connections were not designed for any particular loading. Therefore, in order to establish a rational and comprehensive set of shear connection examples, several basic assumptions are made:

- All wide-flange members are ASTM A992 steel (F_y = 50 ksi, F_u = 65 ksi).
- All other shapes and plates are ASTM A36 steel (F_y = 36 ksi, F_u = 58 ksi).
- All bolts are ASTM A325-N, bearing connections in standard holes.
- All welds are made using E70 electrodes and are produced by the SMAW process.
- The beams and girders have a simple span (with full lateral support) equal to 20 times their nominal depth (rounded up to the next whole foot).
- The design load for the shear connections is equal to one half the maximum design uniformly distributed load (½ UDL) based on the span to depth ratio of the supported member.
- The MathCAD® worksheets will only accept valid AISC W-shapes as supporting and supported members. A few of the supported beams on the steel sculpture are S-shapes or channels rather than W-shapes. To perform the analysis, W-shapes with section properties similar to those of S-shapes or channels will be substituted.

DOUBLE-ANGLE CONNECTION

Double-angle connections are made by attaching the in-plane pair of legs (by bolting or welding) to the web of the supported beam and the out-of-plane pair of legs (also by bolting or welding) to the web of the supporting beam, or flange or web of the supporting column.

Standard all-bolted or all-welded double-angle connections are efficient from a material standpoint. For an all-bolted double-angle connection, typically there are two lines of bolts on the supporting side (one bolt line on each outstanding leg) and one line of bolts on the supported side. The number of bolt rows is usually common to both sides. The two bolt lines on the supporting side are in single shear and the one bolt line one the supported side is in double shear. Although the supported side may have half as many bolts, they are in double shear. Often the strength based on bolt shear alone is the same on either ply of the double-angle. The same general idea is also applicable for all-welded double-angle connections. Welds on the supporting side (referred to in the AISC Manual as Weld B) are typically placed along each toe of the outstanding legs of the angle with a return at the top. Welds on the supported side (referred to in the AISC Manual as Weld A) are typically placed along the toe and/or across the top and bottom edges of both in-plane angle legs. Minimum tabulated thickness in the manual for welding assumes E70 electrodes and are based on matching shear rupture strength of the weld with the shear rupture strength of the base metal. If less than the tabulated thickness is provided the tabulated weld values must be reduced by the ratio of thickness provided to thickness required. Thus, longer yet smaller welds are better suited, as larger welds provide no strength advantage once the limiting thickness has been reached.

To facilitate erection the connection may have short slotted holes and/or a detailed length such that the overall member length is slightly shorter than the span with shims provided to fill any gap. If the angles are shop attached to the face of the supporting column, then the supported beam will be erected in a knifed manner where the bottom flange of the supported beam is coped allowing it to be lowered into position between the in-plane angle legs.
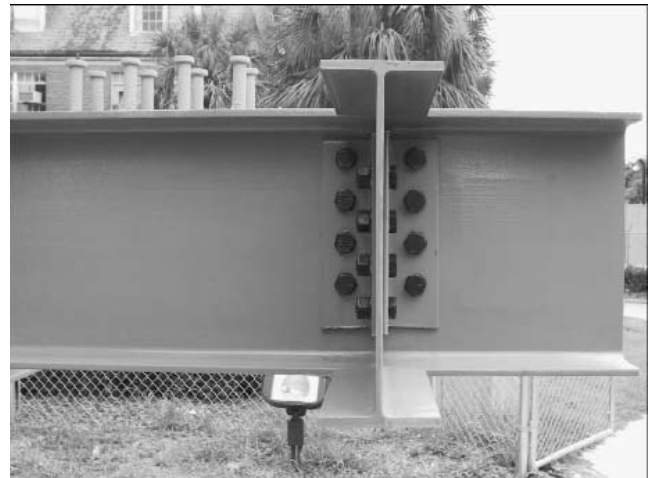

Figure 4-2. Shear Connection: All-bolted double-angle Girder B1 / Beam B1B

Figure 4-3. Shear Connection: All-bolted double angle Girder B1 / Beam B1B

The assumed location of the idealized pin for a double-angle connection is at the outstanding legs. Whether bolted or welded, it is through the deformation of the outstanding legs that the rotational flexibility is achieved. Guidance is provided in the AISC Manual for angle thickness and gage requirements to ensure adequate rotating flexibility.

The absolute position and spacing of bolt holes are controlled by either clearance or edge distance limitations, however, for some regularity, common gages have been established by usage. The usual gages are a function of the angle leg dimension and are based on design, fabrication, and erection convenience. For given angle sizes, the hole-to-hole gage is generally kept the same.

Welds on the supporting member side have ductility concerns. The welds are placed along the toe of each of the outstanding angles, thus the entire outstanding angle legs are available for deformation. The top edges of angles (as well as other shear connectors) on the supporting side should not be welded across the top, except for short weld returns. If the entire top edge of the outstanding angle leg were welded, it would inhibit its flexibility and thus the rotational flexibility of the connection.

On the steel sculpture, connections of members B1B, B3A, B3B, B8, B8A, and B8B are examples of double-angle connections. The B1B connection is an all-bolted double-angle connection with rows of bolts on the supporting member side offset from those on the supported member side. The offset pitch requires the angles to be slightly longer but provides better entering and tightening clearances. The double-angles have been shop assembled on the supported side. Since both the supporting girder and the supported beam are the same depth (W18 series) and the top flanges of both beams are aligned, the supported beam must be double coped to permit erection.

The B3A/B3B joint is an example of a back-to-back condition with two double-angle connection sharing bolts. When such connections would occur in column webs, or in girder webs directly over the top of a column (e.g. as in cantilevered roof framing), current OSHA safety standards prohibit this unless erectability is provided for with an erection seat or other means. One solution to the problem is illustrated on the steel sculpture - provide an additional row of bolts on one of the double-angles that is not shared with the other. This allows erection of the beam with the additional bolt row until the other beam can be fitted up. Other solutions involve providing one additional bolt in two opposite corners of each connection, or offsetting the beams such that they both share only one bolt line. Note that the bolts common to both connections are not in double shear. Rather, they are in single shear on two planes. The top of the supported beam has been coped to allow the flanges to be aligned vertically.

Girder B8 represents an all-welded arrangement of double-angle connections. All-welded double-angle connections are difficult to erect. At least one of the members must somehow be supported so that the double-angle can be welded. Temporary erection bolts are usually used to support and stabilize members during erection. The practice of using temporary erection bolts for support of a member to create an all-welded connection tends to nullify the reason for welding. Shop welding has been performed on the supported side of each beam. Both the outstanding angle legs and the areas they frame into have not been painted. Field welding should be performed on unpainted bare steel.
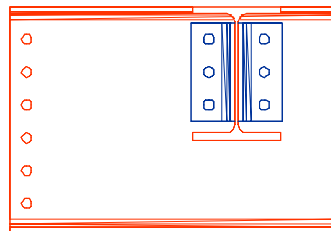
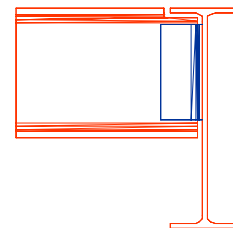| Figure 4-4 **Shear Connection: Bolted-Welded Double-Angle** Girder B3 / Beam B3A	Limit States
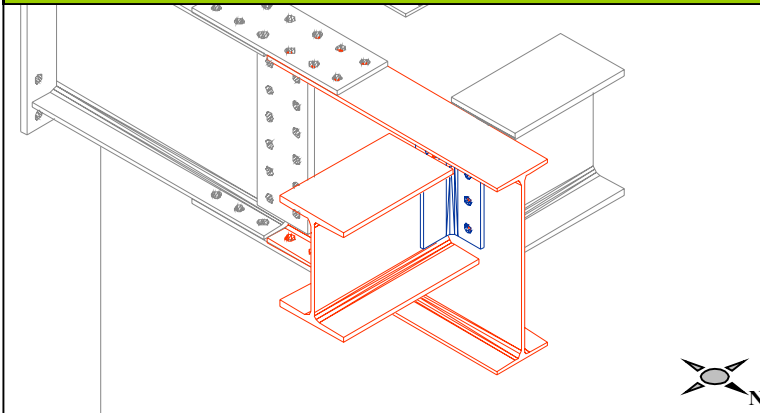
Figure 4-4

<u>**Shear Connection: Bolted-Welded Double-Angle**</u>

Girder B3 / Beam B3A

<u>**Limit States**</u>

Block Shear Rupture
Bolt Bearing
Bolt Shear
Flexural Yielding
Local Web Buckling
Shear Rupture
Shear Yielding
Weld Strength

N

East Elevation

North Elevation

<u>Notes</u>

- Girder B3 supports Beam B3A by a bolted-welded double-angle connection.

- The double-angles are shop welded to the supported beam and field bolted to the supporting girder.

- The girder web is shared between two double-angle connections. The additional row of bolts holds the east side connection temporarily in place until the west side connection can be fitted up.

- The top flanges of both the beam and the girder are at the same elevation. The top flange of the supported beam is coped to eliminate the interference of girder flange.

- Welds on the supported side are placed along the toe of each angle and optionally along the top or bottom edges of both angles.

Figure 4-5. Shear Connection: Bolted-welded double-angle Girder B3 / Beam B3A

Figure 4-8. Shear Connection: Bolted-welded double-angle Girder B3 / Beam B3B

Figure 4-6. Shear Connection: Bolted-welded double-angle Girder B3 / Beam B3A

Figure 4-9. Shear Connection: Bolted-welded double-angle Girder B3 / Beam B3B

*Figure 4-7. Shear Connection: Bolted-welded double-angle Girder B3 / Beam B3A*

Figure 4-10. Shear Connection: Bolted-welded double-angle Girder B3 / Beam B3B
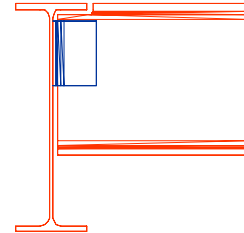
Figure 4-11 **Shear Connection: Bolted-Welded Double-Angle** Girder B3 / Beam B3B	**Limit States**

Figure 4-11
Shear Connection: Bolted-Welded Double-Angle
Girder B3 / Beam B3B

Limit States

Block Shear Rupture
Bolt Bearing
Bolt Shear
Flexural Yielding
Local Web Buckling
Shear Rupture
Shear Yielding
Weld Strength

N
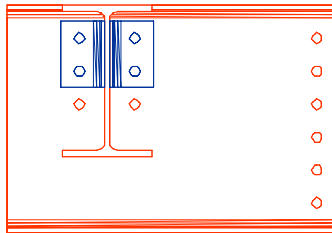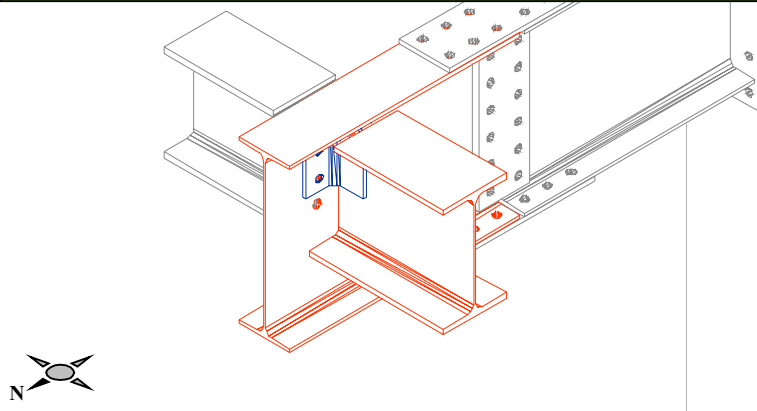
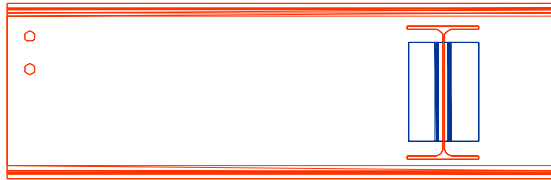**West Elevation**

North Elevation

Notes

- **Girder B3 supports Beam B3B by a bolted-welded double-angle connection.**

- **The double-angles are shop welded to the supported beam and field bolted to the supporting girder.**

- **The girder web is shared between two double-angle connections. The additional row of bolts holds the east side connection temporarily in place until the west side connection can be fitted up.**

- **The top flanges of both the beam and the girder are at the same elevation. The top flange of the supported beam is coped to eliminate the interference of girder flange.**

- **Welds on the supported side are placed along the toe of each angle and optionally along the top or bottom edges of both angles.**

Figure 4-12	**Limit States**
Shear Connection: All-Welded Double-Angle	
Girder B6 / Beam B8B	Shear Yielding
	Shear Rupture
	Weld Strength

North Elevation

West Elevation

Notes

- Girder B6 supports Beam B8B by an all-welded double-angle connection.

- The double-angles are field welded to the supporting girder and shop welded to the supported beam.

- When connection elements are field welded to members, the area of attachment must be free of any coatings (i.e. paint) or lubricants (i.e. oil or grease)

- Welds on the supporting member should be placed along the toe and optionally along the bottom edge of the angle. Properly sized weld returns should be provided at the top edge of the angle; welding across the entire top edge should be avoided since it would inhibit the flexibility of the connection.

- Welds on the supported side are placed along the toe of each angle and optionally along the top or bottom edges of both angles.
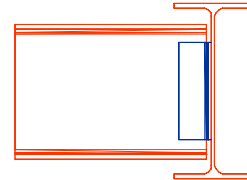
Figure 4-13 Shear Connection: Bolted-Welded Double-Angle Girder B8 / Beam B8A	Limit States Shear Rupture Shear Yielding Weld Strength

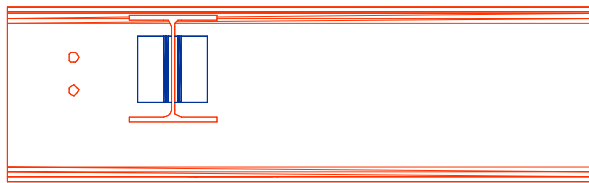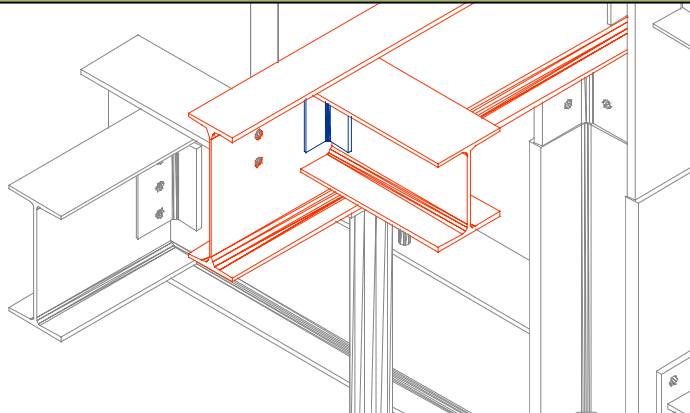
North Elevation
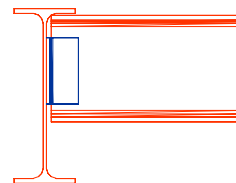
East Elevation

Notes

- B8 girder supports Beam B8A by an all-welded double-angle connection.

- The double-angles are field welded to the supporting girder and shop welded to the supported beam.

- When connection elements are field welded to members, the area of attachment must be free of any coatings (i.e. paint) or lubricants (i.e. oil or grease)

- Welds on the supporting member should be placed along the toe and optionally along the bottom edge of the angle. Properly sized weld returns should be provided at the top edge of the angle; welding across the entire top edge should be avoided since it would inhibit the flexibility of the connection.

- Welds on the supported side are placed along the toe of each angle and optionally along the top or bottom edges of both angles.

Figure 4-14 **Shear Connection: Bolted-Welded Double-Angle** **Column C2 / Girder B8**	**Limit States** **Shear Rupture** **Shear Yielding** **Weld Strength**
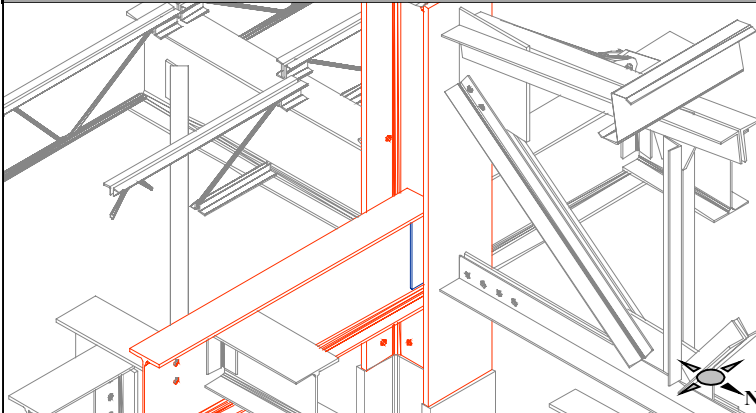
North Elevation
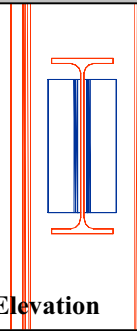
East Elevation

Notes

- Column C2 supports Girder B8 by all-welded double-angles.

- The double-angles have been field welded to the supporting column and shop welded to the supported girder.

- When connection elements are field welded to members, the area of attachment must be free of any coatings (i.e. paint) or lubricants (i.e. oil or grease)

- Welds on the supporting member should be placed along the toe and optionally along the bottom edge of the angle. Properly sized weld returns should be provided at the top edge of the angle; welding across the entire top edge should be avoided since it would inhibit the flexibility of the connection.

- Welds on the supported side are placed along the toe of each angle and optionally along the top or bottom edges of both angles.

Figure 4-15. Shear Connection: All-welded double-angle Girder B6 / Beam B8B

Figure 4-18. Shear Connection: All-welded double-angle Column C2 / Girder B8

Figure 4-16. Shear Connection: All-welded double-angle Girder B6 / Beam B8B

Figure 4-19. Shear Connection: All-welded double angle Column C2 / Girder B8

Figure 4-17. Shear Connection: All-welded double-angle Girder B8 / Beam B8A

Figure 4-20. Shear Connection: All-welded double-angle Column C2 / Girder B8

SHEAR END-PLATE CONNECTION

A shear end-plate connection involves welding a plate perpendicular to the end of the supported web and bolting or welding the plate to the supporting member. The vertical dimension of the plate should not exceed that of the supported beam web. If the supporting side is bolted, the horizontal dimension will depend on the bolt size, gage spacing, and edge distance. Shop welding is the only method of joining the supported beam web and the plate. In the AISC Manual, the minimum tabulated thickness for welding assumes E70 electrodes and is based on matching shear rupture strength of the weld with the shear rupture strength of the base metal. If less than the tabulated thickness is provided, the tabulated weld values must be reduced by the ratio of thickness provided to thickness required. Thus, longer yet smaller welds are better suited as larger welds provide no strength advantage once the limiting thickness has been reached.

Shear end-plates are generally simple to design but require good control of tolerances in fabrication since the detailed length must fit between supports. The detailed length is normally established such that a small erection gap is present. This gap can be filled with shims.

The shear plate essentially has only one ply; the assumed location of the idealized pin is at the plate itself. The rotation flexibility for a shear plate will approximate that of a double-angle connection with similar thickness, gage lines, and connection length.

Beam B2A illustrates a shear end-plate connection. The plate is shop welded to the supported beam web, and field bolted to the girder. Beam B2A is an American Standard (S) shape, characterized by tapered flanges. S shapes are not commonly used in steel framing today, but used here for illustration purposes.

UNSTIFFENED SEATED CONNECTION

The unstiffened seated connection is somewhat unique to the family of shear connections. Unlike all the other shear connections, the attachment for a seated connection is not made at the web of the supported beam. A seated connection is made from an angle that is mounted to the support such that one leg is vertical against the face of the supporting member, and the other, outstanding angle leg, provides a "seat" upon which the beam rests and supports the reaction. The seat angle also provides a location to place the supported beam during erection as the angle is shop attached to the supporting member. The seat angle may be attached to the supporting member either by bolting or welding.

In the AISC Manual, the minimum tabulated thickness for welding the seated connection assumes E70 electrodes and is based on matching the shear rupture strength of the

Figure 4-21. Shear Connection: Bolted shear end-plate Girder B2 / Beam B2A

Figure 4-22. Shear Connection: Bolted shear end-plate Girder B2 / Beam B2A

Figure 4-23
Shear Connection: Bolted Shear End-Plate
Girder B2 / Beam B2A

N

Limit States

Block Shear Rupture
Bolt Bearing
Bolt Shear
Shear Rupture
Shear Yielding
Weld Strength
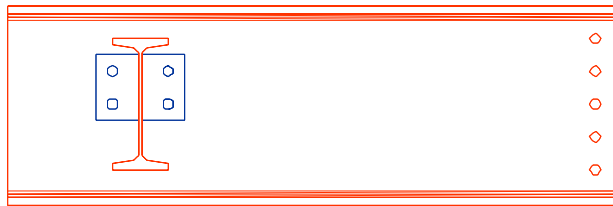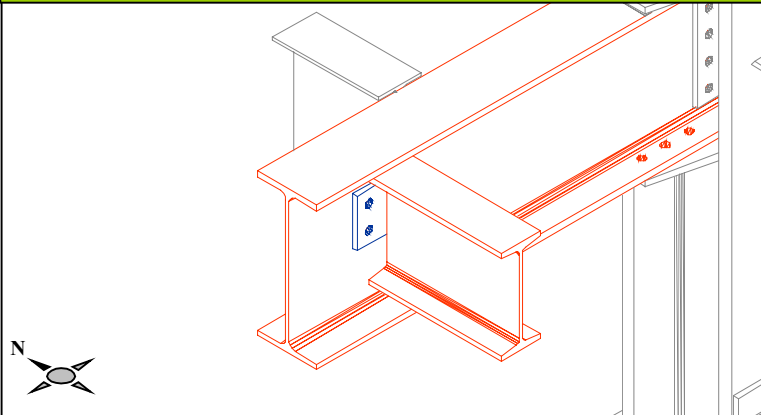
South Elevation

West Elevation

Notes

- Girder B2 supports Beam B2A by a bolted shear end-plate connection.

- Shear end-plates are always welded to the supported beam.

- The plate is shop welded to the supported beam and field bolted to the supporting girder.

- The supported beam is an American Standard (S) shape, characterized by tapered flanges.

- No geometric eccentricities are associated with this connection.

- Shear end-plate connections require tight fabrication and erection tolerances. For bolting, the beam may be specified short and shims used to fill the gap.

- Only A36 grade steel should be used for shear end-plate connections.

weld with the shear rupture strength of the base metal. If less than the tabulated thickness is provided, the tabulated weld values must be reduced by the ratio of thickness provided to thickness required. Thus, longer yet smaller welds are better suited than shorter larger welds, which provide no strength advantage once the limiting thickness has been reached.

Unlike other shear connections, bearing limit states, due to concentrated forces are applicable to seated connections. The bottom flange of the supported beam bears on the outstanding angle leg. Therefore, the additional limit states of web local crippling and web local buckling of the supported member must be investigated.

If the supported beam were to be simply placed on the seat angle, it might roll over or slide off under loading. To prevent the beam from sliding off the angle, the bottom flange of the beam must be attached to the outstanding angle leg and this is usually done by bolting with 2 A325 bolts. To prevent the beam from rolling over, an additional stabilizing angle must be attached to the top flange or along the web of the supported beam. AISC has no particular strength requirement associated with the stability angle. It should be noted that these additional attachments provide some additional stiffness to the connection. The AISC Manual of suggests using a $4 \times 4 \times \frac{1}{4}$ angle attached with the minimum size fillet weld or two bolts. However, only the seat angle is assumed to provide strength for the connection.

Rotational flexibility of the unstiffened seat connection is achieved through the deformation of the outstanding leg of the seat angle, as well as deformation in the top or side angle. The seat angle must be thick enough to carry the reaction but thin enough to provide rotational flexibility. If the seat angle is welded to the supporting member, the welds should be placed along the vertical edges of the angle. Welds for the supporting and supported side of the stability angle should be placed at each toe of the angle legs. Welding along the vertical edges of a top angle would inhibit the flexibility of the connection. The end of the beam bears on the seat angle; thus web crippling and local web yielding of the supported beam must be checked.

Beams B5 and B6 represent unstiffened seat connections. Beam B5 has the vertical seat angle leg bolted to the column flange. The bottom flange of the supported beam is welded to the seat. The top angle is attached to the top flange of the beam. Beam B6, on the other hand, has the vertical seat angle leg welded to the column web. The seat is bolted to the bottom flange of the supported beam. The B6 connection also has the top angle located in the optional side position.


Figure 4-24. Shear Connection: Bolted shear end-plate
Girder B2 / Beam B2A

Figure 4-25. Shear Connection: Bolted shear end-plate
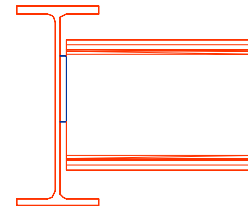Girder B2 / Beam B2A

Figure 4-26
Shear Connection: Bolted Unstiffened Seat
Column C2 / Girder B5

Limit States

Bolt Bearing
Bolt Shear
Flexural Rupture
Flexural Yielding
Local Web Yielding
Shear Rupture
Shear Yielding
Web Crippling
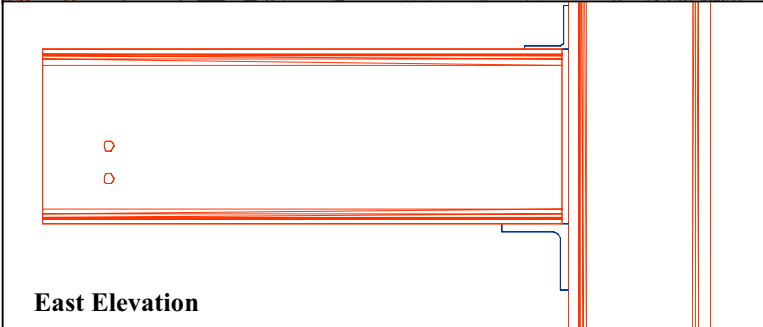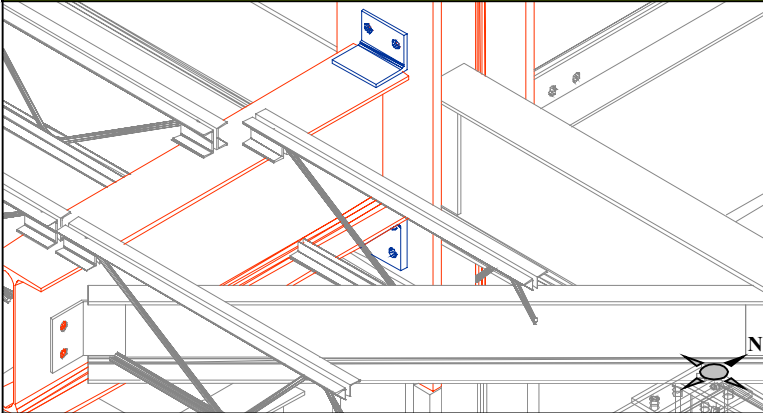
East Elevation
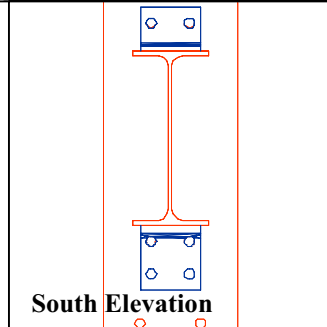
South Elevation

Notes

- **Column C2 supports Girder B5 by a bolted-welded seat connection.**

- **The seat angle is field bolted to the supporting column and shop welded to the supported girder.**

- **The top angle only provides stability to the supported beam. All shear is assumed to be carried by the seat angle.**

- **The attachment of the outstanding angle leg to the bottom flange of the girder is only to prevent the beam from slipping off the seat.**

- **The seat provides bearing for the bottom flange of the girder, thus web crippling and local web yielding limit states must also be considered.**

Figure 4-27
Shear Connection: Welded Unstiffened Seat
Column C2 / Girder B6
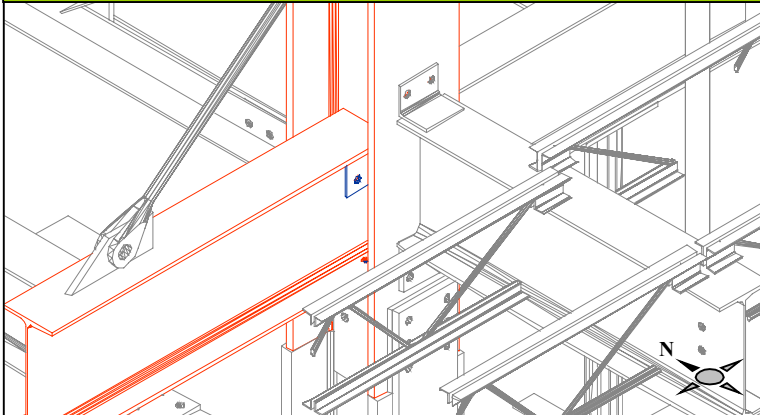
Limit States

Flexural Rupture
Flexural Yielding
Local Web yielding
Shear Rupture
Shear Yielding
Web Crippling
Weld Strength

South Elevation
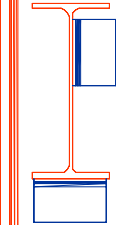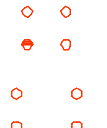
West   Elevation

Notes

- **Column C2 supports Girder B6 by a welded-bolted seat connection.**

- **The seat angle is shop welded to the supporting column and field bolted to the supported girder.**

- **The top angle only provides stability to the supported beam. All shear is assumed to be carried by the seat angle.**

- **The attachment of the outstanding angle leg to the bottom flange of the girder is only to prevent the beam from slipping off the seat.**

- **The seat provides bearing for the bottom flange of the girder, thus web crippling and local web yielding limit states must also be considered.**

SINGLE-PLATE (SHEAR TAB) CONNECTION

The single-plate (or shear tab) connection consists of a plate welded to the supporting member and bolted to the web of the supported beam. Since this connection is one sided it can be easily erected by swinging the supported beam into position from the side. The equations in the AISC Manual are based on E70 electrodes. The weld size on each side of the plate should be three-quarters the thickness of the single plate to ensure that weld strength is not the controlling element in the connection.

The orientation of the single-plate connection is in the plane of the web of the supported member. This means that the rotational flexibility and the idealized location of the pin are dependent on the relative rigidity of the plate and the support (and whether or not short slotted holes are used). If the support is flexible then the rotation is accommodated by the deformation of the supporting member. If the support is rigid, then the rotation occurs primarily within the plate connection.

Recommended upper and lower bound plate thickness have been established for this connection type. The lower bound plate thickness is to control local buckling assuming the bottom half of the plate is in compression due to flexure. The minimum thickness is a function of the length of the plate, L, material yield stress, F_y, and a plate buckling coefficient, K. The minimum thickness equals:

$$\frac{L}{234}\sqrt{\frac{F_y}{K}}$$

but not less than ¼ in. The upper bound thickness is to ensure adequate rotational ductility in the plate. The maximum plate thickness is a function of the bolt diameter used, d_b, and is equal to $d_b/2 + \frac{1}{16}$ in. but not less than the minimum plate thickness previously established.

Eccentricity must always be considered in the design of single-plate connections. The eccentricity, for calculation purposes, may be one of four possible cases depending on the rigidity of the support and whether standard or short slotted holes are used. The two equations for short slotted holes (rigid and flexible support) are nearly identical. Likewise, the two equations for the standard holes (rigid and flexible support) are also nearly identical. Based on either standard holes or short slotted holes, the only difference in the formula for eccentricity between a rigid support and a flexible support is that the flexible support equations have a lower bound value associated with them. The lower bound value for the flexible support is equal to the horizontal distance from the weld line to the bolt line.

The steel sculpture shows a single-plate connection with Beam B2B. The plate is welded to the supporting girder and bolted to the supported beam. Assuming standard holes this would correspond to a flexible support condition. The sup ported beam is top coped to provide vertical alignment of between the top flange of the girder and beam.

SINGLE-ANGLE CONNECTION

A single-angle connection is similar to a double-angle connection, except that only one angle is used. The outstanding and in-plane legs of the single-angle can be attached to either the supporting or supported member by bolting or welding. In a fashion similar to the single-plate connection, the single-angle connection is a one-sided connection, allowing the supported beam to be swung, rather than lowered, into place. Single-angles are normally shop attached to the supporting member. When field bolting, short slots in the angle can provide any needed adjustment.

Single-angle connections are simple to erect particularly when shop attached to the support. A standard all-bolted, single-angle connection has all the bolts in single shear.

Figure 4-28. Shear Connection: Welded unstiffened seat Column C2 / Girder B6

Figure 4-29. Shear Connection: Welded unstiffened seat Column C2 / Girder B6

Figure 4-30 **Shear Connection: Single-Plate** **Girder B2 / Beam B2B**	**Limit States** **Block Shear Rupture** **Bolt Bearing** **Bolt Shear** **Flexural Yielding** **Local Web Yielding** **Shear Rupture** **Shear Yielding** **Weld Strength**
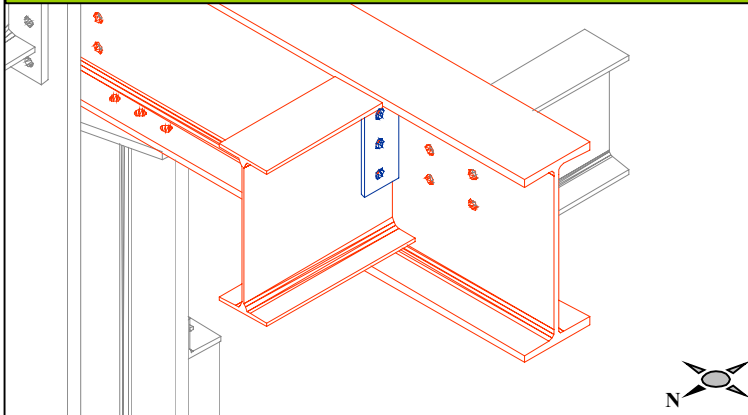
N

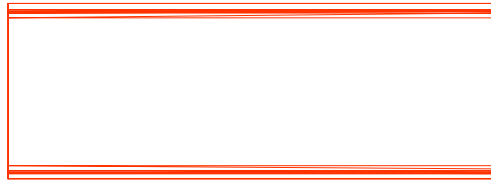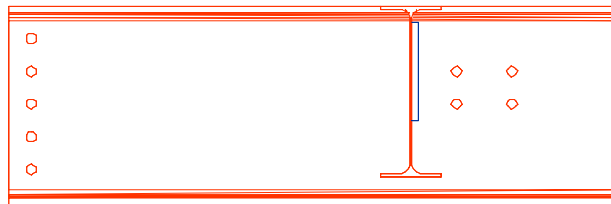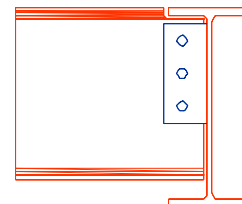| North Elevation	West Elevation

Notes

- **Girder B2 supports Beam B2B by a single-plate connection.**

- **The plate is shop welded to the supporting girder and field bolted to the supported beam.**

- **This is a one sided connection. Erection is simplified as the beam can be swung into place.**

- **The top flanges of both the beam and the girder are at the same elevation. The top flange of the supported beam is coped to eliminate the interference of girder flange.**

- **The weld size should be limited to three-quarters the thickness of the single plate to ensure that weld strength is not the controlling element in the connection.**

- **Only A36 grade steel should be used for single-plate connections.**

*Figure 4-31. Shear Connection: Single plate
Girder B2 / Beam B2B*

*Figure 4-32. Shear Connection: Single plate
Girder B2 / Beam B2B*

Thus, the strength based on bolt shear alone is the same on either ply of the single-angle. For a standard all-bolted single-angle connection, there is typically one bolt line on both plies with all the bolts in single shear. Thus, like a double-angle, the strength based on bolt shear alone is the same on either ply of the single-angle.
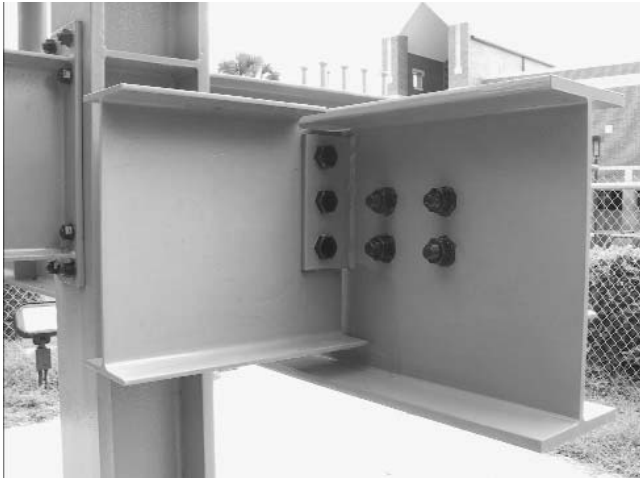
Eccentricity should always be considered on the outstanding angle leg. Eccentricity should also be considered on a welded angle leg. If two or more lines of bolts are used on the web of the supported beam, then eccentricity should be considered on the in-plane angle leg.

Like a double-angle connection, the rotational flexibility of the single-angle connection is achieved primarily through the deformation of the outstanding angle leg. Welds on the supporting side should be placed along the toe and bottom edge of the angle. Properly sized weld returns should be provided at the top edge of the angle; welding across the entire top edge on the supported side should be avoided since it would inhibit the flexibility of the connection.

Beams B4A and B4B demonstrate the use of single-angle connections on the steel sculpture. The supported members in each of these connections are channel shapes, which represent typically lightly loaded members used for infill steel framing.

The single-angle for Beam B4A is shop welded to the girder and field bolted to the beam, while Beam B4B has the single-angle field bolted to the girder and shop welded to the channel.

TEE SHEAR CONNECTION

The tee shear connection is fabricated from a WT section with the stem connected to the web of the supported member and the flange attached to the supporting member. Both the stem and the flange may be either bolted or welded to the appropriate members.

The required beam setback for a shear tee connection is greater than that of other connections. The tee has a rolling fillet on each side at the junction of the flange and the stem, which requires a setback distance sufficient enough so as not to interfere with the rolling fillet. A limited amount of rolling fillet encroachment is permitted depending on the size of the rolling fillet of the WT section.

The theoretical location of the pin (assumed most flexible part of the connection) will depend on the support and tee chosen. The AISC Manual gives guidance for proper design in this regard.

Tees with thick flanges may provide for rotational flexibility through stem behavior similar to that of a shear tab. Tees with thicker flanges may provide for rotational flexibility through flange behavior similar to that of an end plate or double angle connection. The AISC Manual provides suitable guidance for all cases.

Beam B1A shows a tee shear connection. The tee has been shop welded to the girder and field bolted to the beam.

Figure 4-33 **Shear Connection: Welded-Bolted Single-Angle** Girder B4 / Beam B4A	**Limit States** **Bolt Shear** **Bolt Bearing** **Block Shear Rupture** **Shear Yielding** **Shear Rupture** **Weld Strength**
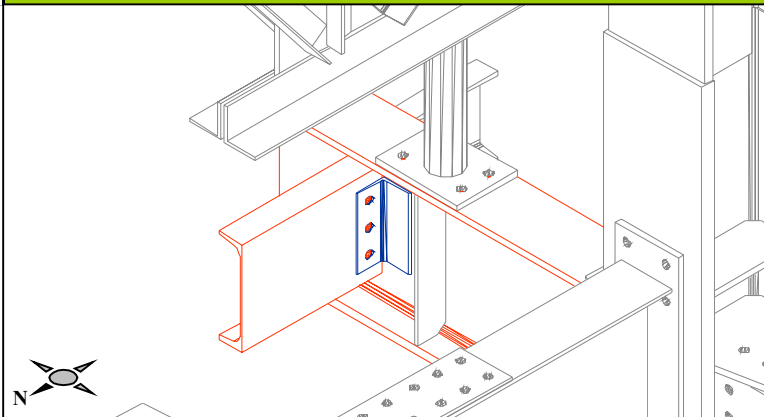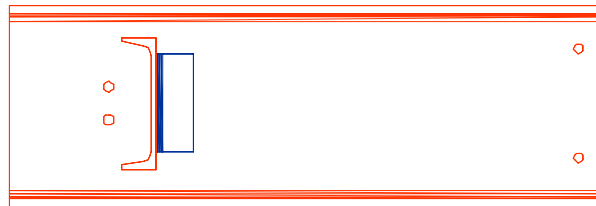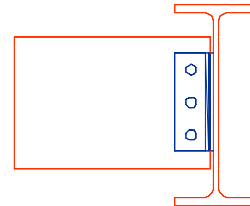
North Elevation	**West Elevation**

Notes

- **Girder B4 supports Beam B4A by a welded-bolted single-angle connection.**

- **This single-angle is field bolted to the supported beam and shop welded to the supporting girder.**

- **This is a one sided connection. Erection is simplified as the beam can be swung into place.**

- **The supported beam is a channel (C) shape. Channels have tapered flanges similar to American Standard (S) shapes.**

- **Single-angle connections tend to have lower load capacities then double-angle connections.**

- **In-plane and out-of-plane eccentricity should be considered.**

- **Welds on the supporting member should be placed along the toe and optionally along the bottom edge of the angle. Properly sized weld returns should be provided at the top edge of the angle; welding across the entire top edge should be avoided since it would inhibit the flexibility of the connection.**

Figure 4-34 **Shear Connection: Bolted-Welded Single-Angle** **Girder B4 / Beam B4B**	**Limit States**
	Block Shear Rupture **Bolt Bearing** **Bolt Shear** **Shear Rupture** **Shear Yielding** **Weld Strength**

N
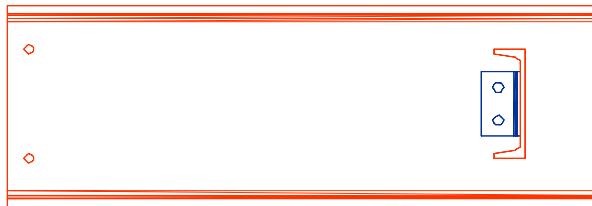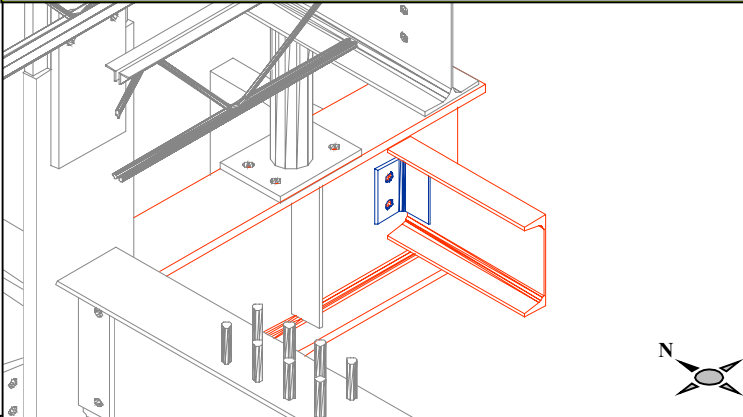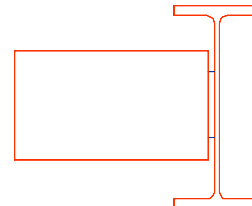
South Elevation	East Elevation

Notes

- **Girder B4 supports Beam B4B by a bolted-welded single-angle connection.**

- **The single-angle is shop welded to the supported beam and field bolted to the supporting girder.**

- **This is a one sided connection. Erection is simplified as the beam can be swung into place.**

- **The supported beam is a channel (C) shape. Channels have tapered flanges similar to American Standard (S) shapes.**

- **Single-angle connections tend to have lower load capacities then double-angle connections.**

- **In-plane and out-of-plane eccentricity should be considered.**

- **Welds on the supported side are placed along the toe of each angle and optionally along the top or bottom edges of both angles.**

*Figure 4-35. Shear Connection: Welded-bolted single-angle
Girder B4 / Beam B4A*

*Figure 4-38. Shear Connection: Bolted-welded shingle-angle
Girder B4 / Beam B4B*

*Figure 4-36. Shear Connection: Welded-bolted single angle
Girder B4 / Beam B4A*
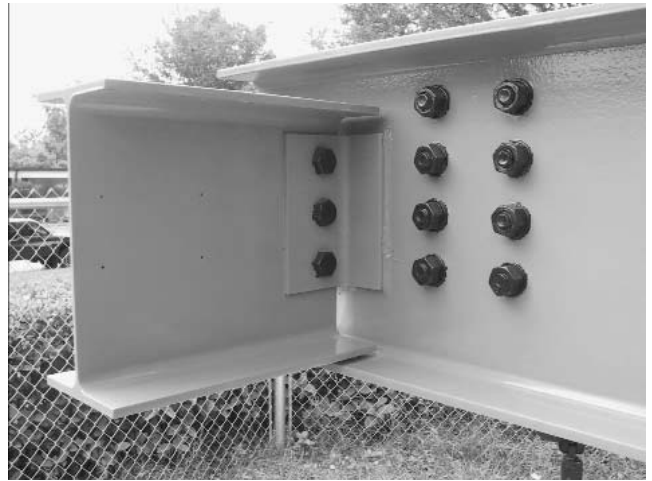
*Figure 4-39. Shear Connection: Welded-bolted tee
Girder B1 / Beam B1A*

*Figure 4-37. Shear connection: Bolted-welded single-angle
Girder B4 / Beam B4B*

*Figure 4-40. Shear Connection: Welded-bolted tee
Girder B1 / Beam B1A*

| *Figure 4-41* **Shear Connection: Welded-Bolted Tee** Girder B1 / Beam B1A | **Limit States** **Block Shear Rupture** **Bolt Bearing** **Bolt Shear** **Flexural Rupture** **Shear Rupture** **Shear Yielding** **Weld Strength** |

N

| West Elevation | North Elevation |
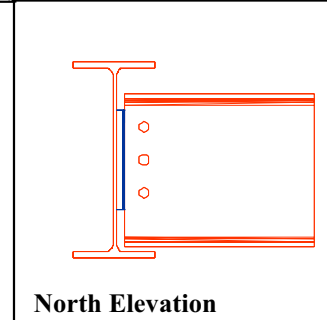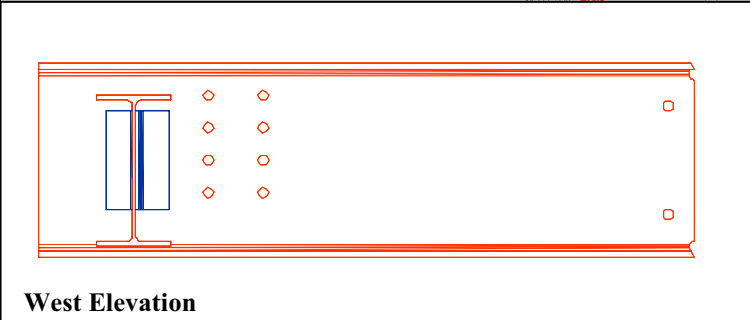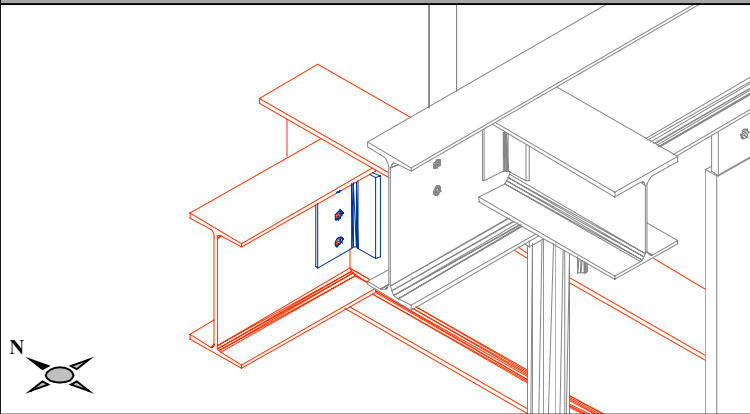
Notes

- **Girder B1 supports Beam B1A by a welded-bolted shear tee connection.**

- **The tee is shop welded to the supporting girder and field bolted to the supported beam.**

- **This is considered a flexible support condition since the support of this connection is the web of the girder.**

- **This is a one-sided connection. Erection is simplified as the beam can be swung into place.**

- **Welds on the supporting member should be placed along the toe and optionally along the bottom edge of the angle. Properly sized weld returns should be provided at the top edge of the angle; welding across the entire top edge should be avoided since it would inhibit the flexibility of the connection.**

- **To ensure adequate connection flexibility the welds connecting the tee flange to the supporting member and the thickness of the tee stem are subject to specific size limitations.**

- **Setback of the supported beam must extend beyond the k distance of the tee flange.**

- **Due to the extended setback, eccentricity and tee stem flexure must be considered.**

CHAPTER 5

Moment Connections

Moment connections transfer the moment carried by the flanges of the supported beam to the supporting member. Moment connections (or continuous or rigid-frame connections) are assumed to have little or no relative rotation between the supporting member and the supported members. A Fully Restrained (FR) connection assumes the measured angles between intersecting members are maintained (i.e. no relative rotation) and there is full transfer of the moments. Partially Restrained (PR) connections assume that there will be some relative rotational movement that occurs between intersecting members, though there will still be transfer of the moments.

Infinite rigidity can never be realistically attained; therefore, even fully restrained moment connections do possess some minimal amount of rotational flexibility, which is usually neglected. FR connections are idealized as having full fixity between members.

To transfer the tension and compression forces carried by the flanges, continuity between the supported beam flanges and the supporting member must be realized. Hence, the flanges of the supported member are attached to either a connection element or directly to the supporting member.

Most moment connections are made from the supported beam to either the flange or the web of column members (called beam-to-column connections). Beam splices to transfer moments are also common. Moment connections also normally include a simple shear connection at the web of the supported member to carry the shear component of the beam reaction. Any load eccentricity considerations in the shear connection as part of a moment connection may be ignored as it is carried by the moment connection. If a moment connection is made to the web of a column, the connection elements may be extended so that field welds and/or bolts can be located outside of the column flanges for easier erectability.

Columns in beam-to-column connections are subjected to concentrated forces from the flanges of the supported member. In some instances the column section may have insufficient local strength at the location of these concentrated forces. In such circumstances, a heavier column section or a higher strength column may be substituted, or transverse column stiffeners and/or web doubler plates may be installed.

Transverse stiffeners are plates fabricated to fit between the flanges of the column at the point(s) of concentrated loading (tension or compression). Web doubler plates are steel plates that are fabricated to increase the overall thickness of the web of a section. Both types of components (transverse stiffeners and web doubler plates) are welded to the section to enhance the stiffness. The use of these components will increase fabrication costs, so it may be more economical to select a heavier column section or one with higher yield strength.

FLANGE-PLATED CONNECTIONS
(BEAM-TO-COLUMN)

Flange-plated connections are made with top and bottom flange-plates that connect the flanges of the supported beam to the supporting column. There are several types of flange-plated connections. The flange-plates are fillet or groove welded to the supports. The flanges of the supported member may be either bolted or welded to the plates. Flange-plates are usually shop attached to the column and field attached to the flanges of the supported member.

Flange-plates are usually positioned slightly wider apart than the depth of the supporting member if they are to be bolted to the flanges of the supported member. This arrangement makes it easier to erect, providing adequate clearance. Shims are provided to fill any of the resulting gaps. If the flange-plates are welded to the flanges of the supporting member, then the plates are designed and detailed such that flange plate welding can be performed in the flat position. This can be accomplished by using a flange-plate that is slightly narrower than the beam flange on top and a flange-plate that is slightly wider than the beam flange on the bottom. This arrangement permits flat-position (down-hand) welding.

If the beam frames into the web of a column, the plates should be sized wide enough to fill the space within the column flanges, but narrowing outside the column to allow downhand welding. In this case the top plate will likely be wider than the top flange of the supported beam. Thus, to accommodate the flat welding position of flange plates attached to column webs, the top flange plate should be blocked to make an easier joint for welding to the supported member. Regardless of the bolting or welding arrangements, if the connection is made to the web of a column, the supported side attachment should provide enough space to accommodate bolting or welding access (i.e. extending plates past the column flanges). See the AISC Manual for discussion of corner clips and plate configurations when attaching to column webs.

Girders B2 and B4 are bolted and welded flange plate connections to the web of Column C1, respectively. The flange-plates for Girder B2 are bolted to the top and bottom flanges of the beam. As mentioned above, the flange-plates

Figure 5-1	**Limit States**
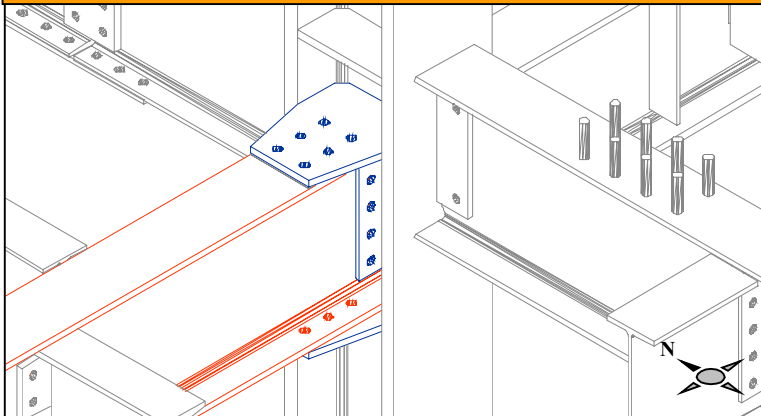
Moment Connection: Bolted Flange-Plates
Column C1 / Girder B2

Limit States

Block Shear Rupture
Bolt Bearing
Bolt Shear
Plate Buckling
Tension Rupture
Tension Yielding
Weld Strength

South Elevation
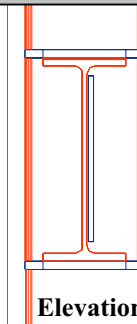
West | Elevation

Notes

- Column C2 supports Girder B2 by bolted flange-plates.

- These flange-plates are shop welded to the supporting column and field bolted to the supported girder.

- The moment connection is made to the web of Column C1. The flange-plates are cut to fill the space between the column flanges.

- The corners of the flange-plate are clipped to eliminate the possibility of creating a stress concentration at the re-entrant corner of the web-flange junction. No weld is provided at these locations.

- The plates attached to the flanges of the girder are for transfer of the moment forces.

- The plate attached to the web of the girder is for transfer of the shear force.

- The effect of eccentricity in the shear connection is neglected.

- For all FR and PR column connections, column stiffening must be investigated to ensure that the connection flange forces do not exceed applicable limit states.

Figure 5-2. Moment Connection: Bolted flange plates
Column C1 / Girder B2

Figure 5-4. Moment Connection: Welded flange plates
Column C1 / Girder B4

Figure 5-3. Moment Connection: Bolted flange plates
Column C1 / Girder B2

Figure 5-5. Moment Connection: Welded flange plates
Column C1 / Girder B4

Figure 5-6 **Moment Connection: Welded Flange-Plates** **Column C1 / Girder B4**	**Limit States** **Bolt Bearing** **Bolt Shear** **Plate Buckling** **Tension Rupture** **Tension Yielding** **Weld Strength**

N

South Elevation | **East   Elevation**

Notes

- **Column C1 supports Girder B4 by welded flange-plates.**

- **The flange-plates are shop welded to the supporting column and field welded to the supported girder.**

- **The moment connection is made to the web of Column C1. The flange-plates are cut to fill the space between the column flanges.**

- **When connection elements are field welded to members, the area of attachment must be free of any coatings (i.e. paint) or lubricants (i.e. oil or grease)**

- **The plates attached to the flanges of the girder are for transfer of the moment forces.**

- **The plate attached to the web of the girder is for transfer of the shear force.**

- **The effect of eccentricity in the shear connection is neglected.**

- **For all FR and PR column connections, column stiffening must be investigated to ensure that the connection flange forces do not exceed applicable limit states.**

are positioned to be slightly wider apart than the depth of Girder B2 and a shim has been provided to fill the gap. Both connections are attached to the web of the column. The plates are shaped such that they fill the entire space between the flanges. The corners of the plates have been clipped to accommodate the rolling fillets of the supporting column and separate the welds. Although the shear connection (shear tab) has not been extended (bolting of this element takes place inside the flanges of the column), in practice this might prove to be a difficult connection to make in the field. It is advised to extend the shear tab. Both girders would normally be specified short and the connection elements extended so that all welds and bolts were located outside the column flanges for easier erectability.

DIRECTLY WELDED FLANGE CONNECTIONS

Directly welded moment connections are typically made with complete-joint-penetration groove welds that directly connect the top and bottom flanges of the supported beam to a supporting column. Fillet welds, or partial-joint-penetration groove welds may be used if suitable for the required force transfer.

Groove welds for directly welded flange connections require significant joint preparation. Weld access holes are cut in the web at the intersection of the flanges of the supported beam. Backing bars and weld runoff tabs are added to the flanges. The groove welds connecting the beam to the column flanges can then be made in the flat welding position. Once the joint is completed, the runoff tabs are removed; however it is sometimes permissible to leave the backing bars in place.

The shear force may be transferred by either the addition of a standard shear connection (i.e. shear tab, single-angle, etc.) or by directly welding the supported beam web to the column flange. Direct welding of the web requires very

close accommodation of mill, fabrication, and erection tolerances and is not often used.

On the steel sculpture, Girder B1 is directly welded to the flange of Column C1. The runoff tabs have been removed, but the backing bars have been left in place after welding. Also note that the end of the beam was left unpainted to accommodate the welding of the joint. A shear tab transfers the shear load to the column.

EXTENDED END-PLATE CONNECTIONS

Extended end-plates are similar in appearance and orientation to shear end-plates. The primary physical difference is that the plate is longer than the depth of the supported beam as it must be attached to both the web and the flanges of the supported beam. The plate is usually fillet welded to the flanges and web of the supported beam, however complete- or partial-joint-penetration welds may be used if the fillet size is excessively large. The plate is then bolted with high-strength bolts to the supporting member.

End-plate connections are classified based on the number of bolts used at the tension flange, such as four-bolt unstiffened and eight-bolt stiffened. The bolts in tension should be arranged in a symmetrical pattern with half above and half below the tension flange. At least two bolts should be used at the compression flange; these bolts serve primarily to carry shear forces. Furthermore, the bolts at the compression flange should be placed between the flanges of the supported beam whenever possible to reduce the required plate length. Extra bolts may be placed in the plate, near the neutral axis of the beam to ensure proper fit-up with the column and assist the compression flange bolts in shear transfer. Like their shear counterparts, extended end-plate connections require close accommodation of mill, fabrication, and erection tolerances. The beam may be fabricated short to
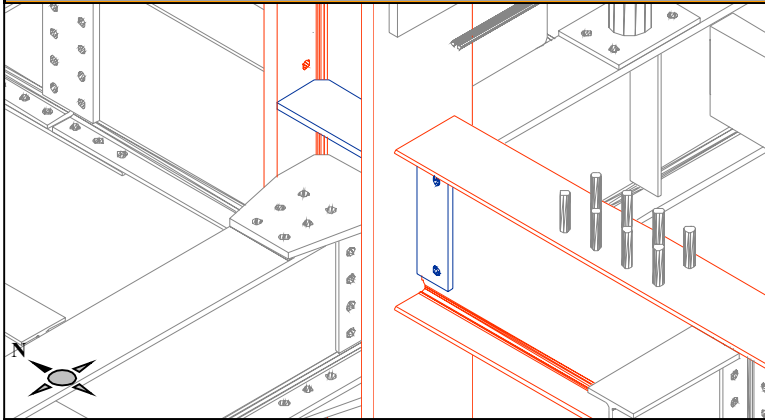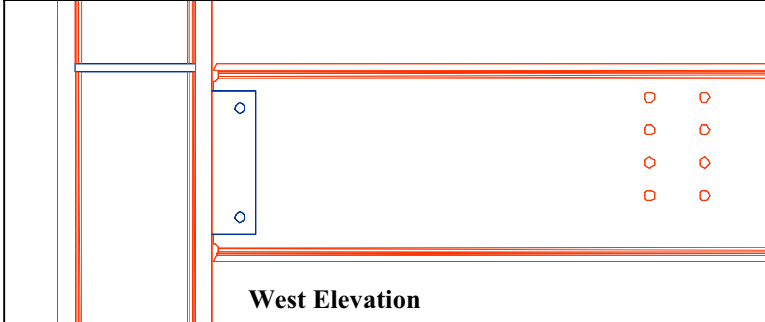

Figure 5-7. Moment Connection: Directly welded flanges. Column C1 / Girder B1

Figure 5-8. Moment Connection: Directly welded flanges. Column C1 / Girder B1

Figure 5-9
Moment Connection: Directly Welded Flanges
Column C1 / Girder B1

Limit States

Bolt Bearing
Bolt Shear
Compression Buckling of Web
Local Flange Bending
Local Web yielding
Weld Strength
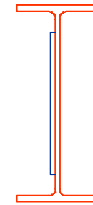
West Elevation

South Elevation

Notes

- Column C1 supports Girder B1 by directly welded flanges.

- A transverse stiffener is attached between the flanges of the support column. The plate is aligned to receive the concentrated force (tension or compression) from the girder flange. For all FR and PR Column Connections, column stiffening should be investigated to ensure that the connection flange forces do not exceed applicable limit states.

- Weld access holes are cut in the supported girder to accommodate welding in the flat position and to relieve thermal stresses.

- The plate attached to the web of the girder is designed for shear transfer.

- The effects of eccentricity in the shear connection are neglected.

- When connection elements are field welded to members, the area of attachment must be free of any coatings (i.e. paint) or lubricants (i.e. oil or grease)

- For all FR and PR Column Connections, column stiffening must be investigated.

accommodate field tolerances with shims furnished to fill any resulting gaps.

Girder B3 has an extended end-plate connection to Column C1 of the steel sculpture. The connection is a four-bolt unstiffened connection with four bolts in two rows at the top flange. The transverse stiffener is also part of the connection. The plate provides continuity transfer of the moment force from Girder B3 on the north side, through the column section and into Girder B1 on the south side. The plate is welded between the flanges of the column section and aligned vertically with the top flanges of Girders B3 and B4. There is only one such continuity plate on the steel sculpture. This arrangement is for illustration purposes only; in practice transverse stiffeners would be provided on both sides of the column web.

MOMENT SPLICE CONNECTIONS

A moment splice connection is designed to transfer flange forces across two beams or two girders that are connected end to end to make up one longer member. Like other moment connections, a shear connection (shear splice) is typically provided at the web to handle the shear force component. Moment splices can be fashioned in a similar manner to any of the three beam-to-column moment connections previously discussed. Extended end-plates can be used in a back-to-back arrangement at the ends of the beams to form a moment splice. Flange plates can be bolted or welded on the top or bottom of both flanges on both members to transfer flange forces. Flanges of the two members can be directly welded to one another by a complete-joint-penetration or partial-joint-penetration groove (butt) weld. If the flanges are directly welded, weld access holes and backing bars may be required. Backing bars and runoff tabs, subject to fatigue loading, will need to be removed. Girder B3 is in fact two, girders of identical cross-sections, spliced together with a flange-plated connection. Plates are bolted to the top of the top flanges and to the bottom of the

Figure 5-10. Moment Connection: Extended end-plate. Column C1 / Girder B3.

Figure 5-12. Moment Connection: All-bolted moment splice. Girder B3 / Girder B3

Figure 5-11. Moment Connection: Extended end-plate. Column C1 / Girder B3.

Figure 5-13. Moment Connection: All-bolted moment splice. Girder B3 / Girder B3

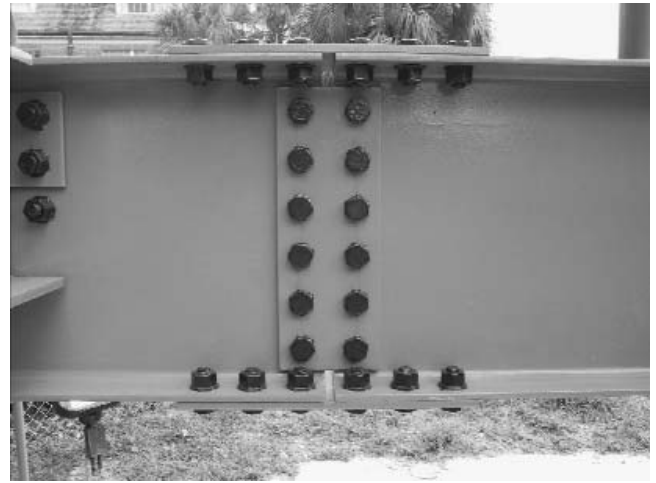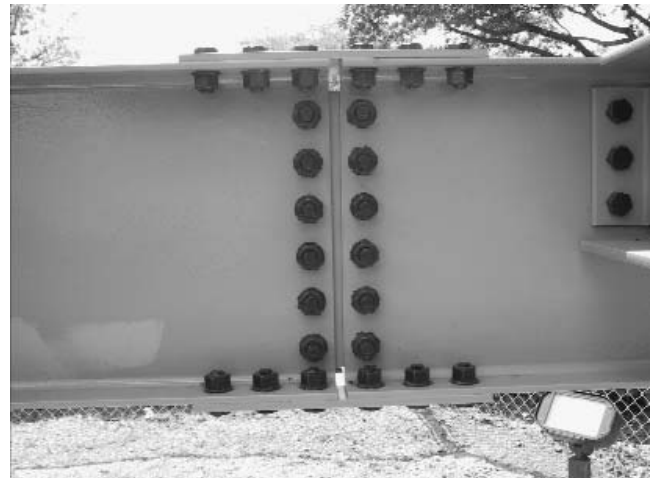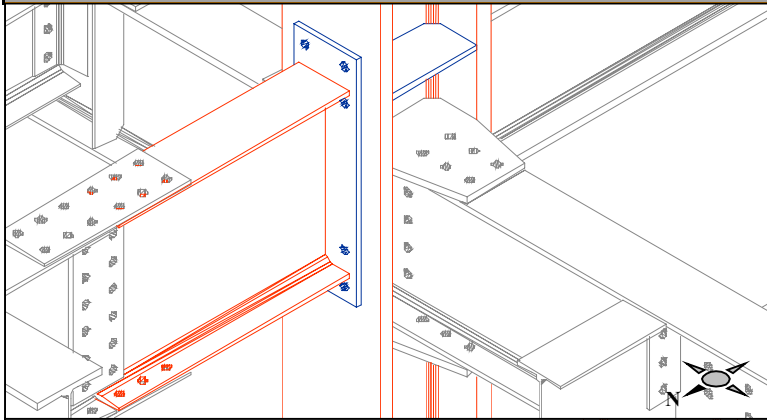| **Figure 5-14**	**Limit States**
Moment Connection: Extended end-plate	
Column C1 / Girder B3	

Figure 5-14
Moment Connection: Extended end-plate
Column C1 / Girder B3

Limit States

Bolt Shear
Bolt Tension
Compression Buckling of Web
Local Flange Bending
Local Web yielding
Shear Rupture
Shear Yielding
Weld Strength
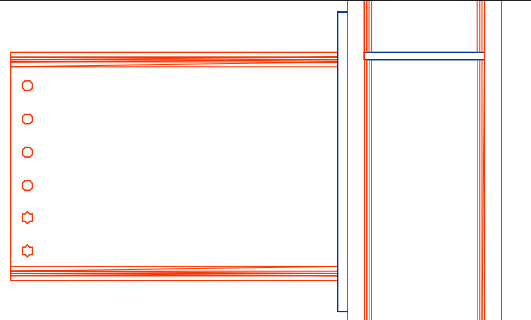
West Elevation

North Elevation
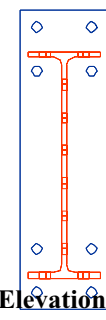
Notes

- **Column C1 supports Girder B3 by a four-bolt unstiffened extended end-plate.**

- **A transverse stiffener is attached between the flanges of the support column. The plate is aligned to receive the concentrated force (tension or compression) from the girder flange. For all FR and PR Column Connections, column stiffening must be investigated to ensure that the connection flange forces do not exceed applicable limit states.**

- **Extended end-plate connections require tight fabrication and erection tolerances.**

- **Extended end-plates are classified based on the number of bolts at the tension flange and may be used with or without stiffeners.**

- **Based on current research, only A36 grade steel should be used for the extended end-plate as outlined in the Manual of Steel Construction.**

- **If bolting is based on bearing, the bolts must be designed for shear-tension interaction. If bolting is based on slip-critical conditions, the bolts may be designed for shear only.**

Figure 5-15
Moment Connection: All-Bolted Moment Splice
Girder B3 / Girder B3
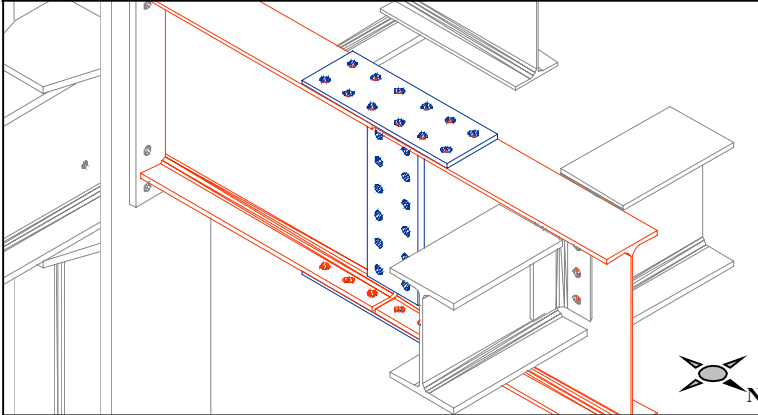
Limit States

Block Shear Rupture
Bolt Bearing
Bolt Shear
Flexural Rupture
Flexural Yielding
Shear Rupture
Shear Yielding
Tension Rupture
Tension Yielding
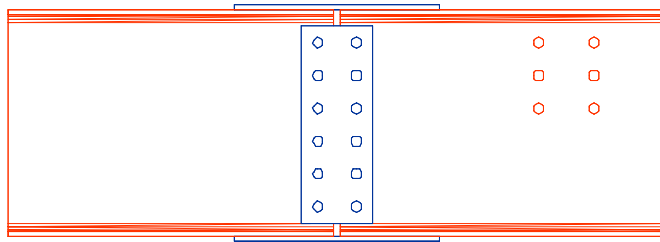
East Elevation
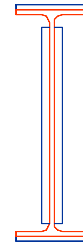
North Elevation

Notes

- Girder B3 is spliced at the midspan by an all-bolted moment splice.

- The plates attached to the flanges of the girders are designed for moment transfer.

- The plates attached to the webs of the girders are designed for shear transfer.

- The web plates are subjected to a moment equal to the shear component force times the distance between the centroids of the fasteners.

bottom flanges. If plates are used on both sides of each flange, then the bolts are in double shear and a more compact moment splice may result. The web plate is a shear splice. The plate is bolted on both sides and transfers the shear force between webs of the girders.

CHAPTER 6

Column Connections

Columns are primarily loaded in compression, but may also be subject to axial tension, shear, and moment. Therefore, the purpose of column connections is to transfer the loads to either a supporting member or to the foundation of the structure.

When a column is used to transfer loads between a supported member and a supporting member, each of these members needs to be checked locally. Stiffeners may be required to prevent local yielding or buckling from the compression forces being transferred. When the column connection is to a base plate, it is necessary that a suitable bearing area be provided to prevent crushing of the concrete foundation.

COLUMN SPLICE

Column splices are used when it is either economical to change column sizes or the height of the structure exceeds the available column length. Column splices at perimeter locations should preferably be located four feet above a finished floor to accommodate attachment of safety cables, which may be required at floor edges or openings and to ease erection. The upper and lower shafts of the column do not necessarily need to be in full bearing contact with one another. A gap up to $1/16$ in. is permitted without the need for repair or shimming. If the gap is between $1/16$ and $1/4$ in. then non-tapered steel shims are required. Engineering evaluation should be performed on gaps larger than $1/4$ in.

In most column splices, the bearing area between the columns will be sufficient to transfer the compression load. In addition, lateral (shear) forces are typically distributed among several columns. When the force on any single column is small, the friction on the contact bearing area and/or the flange plates may be sufficient to resist these forces. If the shear force is large, the column splice connection must be designed to resist these forces and hold all parts securely in place. Column splices must also be designed to resist the tension forces that may develop due to uplift loads. Regardless of the controlling load condition, the splice should be sufficient to hold all parts securely in place. Flange plates, butt plates, or directly welding of flanges may effectively splice columns.

Flange-plated splices involve attaching plates (by bolting or welding) to the flanges of the upper and lower shafts. When two columns being spliced are of different sizes but of the same nominal depth, which is a common occurrence, then shims or filler plates must be used to fill any gaps.

Butt plates are used between the ends of the upper and lower shafts of the column splice. Butt plates are convenient when the nominal depths of the upper and lower shafts are significantly different.

Columns may also be spliced by directly welding the flanges of the upper and lower shafts. The webs of the columns should be attached by welding or by installing plates. The plates may be either bolted or welded to the upper or lower shaft.

The steel sculpture uses one joint to illustrate two different column splices. Column C1 (the lower shaft) is spliced to Column C2 (the upper shaft) by direct welding the flanges on the north side and using a welded-bolted flange plate on the south side. This would never be done in prac-

Figure 6-1. Column Connection: Column splice Column C1 / Column C2
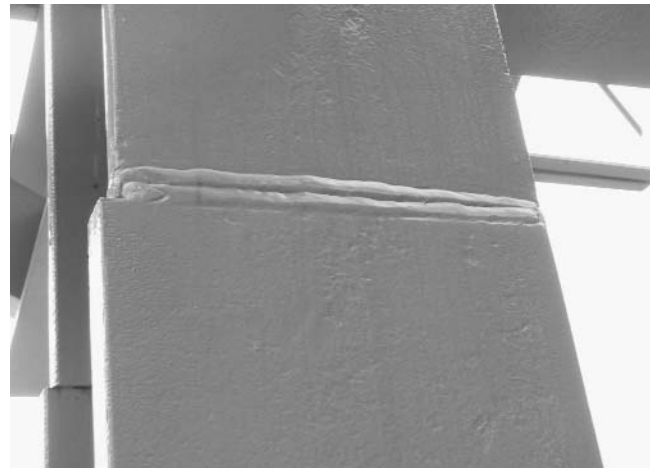
Figure 6-2. Column Connection: Column splice Column C1 / Column C2

| **Figure 6-3** **Column Connection: Column Splice** Column C1 / Column C2 | **Limit States** Bolt Bearing Bolt Shear Shear Rupture Shear Yielding Tension Rupture Tension Yielding Weld Strength |

| **South Elevation** | **West Elevation** | **North Elevation** |
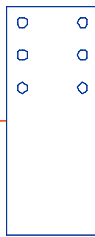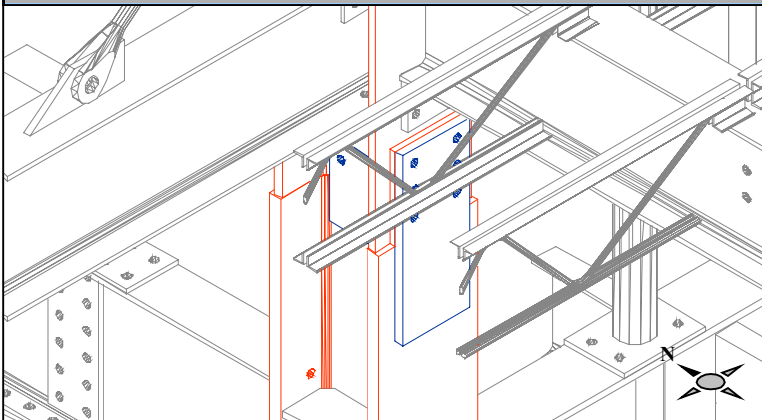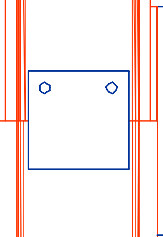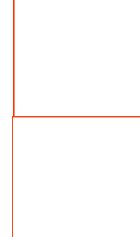
Notes

- The column splice attaches the lighter upper column, Column C2, to the lower heavier column, C1.

- The splice should be designed for any shear (i.e. lateral) and/or tension (i.e. uplift) loads.

- The column splice on the steel sculpture has two different arrangements on one connection. This particular layout is for illustration only. In practice, one form of the splice or the other would be utilized on both flanges of the column.

    NORTH FACE:
    The north side is a directly welded flange splice. This design uses groove welds to join the ends of the flanges of both columns. The columns are field welded together. Therefore, the area of attachment should be free of any coatings (i.e. paint). No strength is associated with the interior web plate connection; it is for erection stability only.

    SOUTH FACE:
    The south side is a flange-plated splice. The flange plate is shop welded to the lower shaft and field bolted to the upper shaft. Filler plates used as required to achieve suitable bolt bearing strength at the splice. Shim plates are used as required to fill gaps between the flange plate and the upper column flange.

tice; the splice would either have entirely welded flanges or use flange plates on both sides.

The end of the flange on the north side of the upper shaft (side of the directly welded flanges) has been beveled to accommodate a bevel groove weld. In addition, a plate welded to the lower shaft and bolted to the upper shaft has been included for fit-up and erection.

On the south side, a flange plate is shop welded to the lower shaft and field bolted to the upper shaft. Both shafts of the column are of the same nominal depth (W12 series), but the bottom shaft is a W12×170 while the upper shaft is a W12×106. Typically column shapes of the same nominal depth have equal distances between the inner faces, while the flange and web thickness vary with respect to the nominal weight per foot of the section. Since the outer faces of the shafts are not equal, the field bolted part of the upper splice is furnished with shims to fill the resulting gap.

BASE PLATES

Column base plates are used to provide a sufficient bearing area on the material below in order that the forces in a column are properly transferred to the foundation. Base plates usually anchor columns to a concrete foundation by anchor rods. The base plates of bracing struts may be bolted or welded to other steel members.

The base plate can be attached to the column either by direct welding of the column to the plate, or additional elements (i.e. angles) can be connected to the column that facilitate attachment of the base plate

The AISC-LRFD Specification for Structural Steel Buildings does not specify a particular method for the design of base plates. However, the AISC Manual does provide the designer with procedures for base plate design.

When the column is in compression, it bears directly on the material below. The compression load determines the size of the base plate. The actual connection of the base plate is effectively passive when there is only an axial compression load. If, however, the column is loaded in tension and/or shear, then the base plate connection becomes active. The base plate size is a function of the compression load and the connection to the base plate is a function of the shear and/or tension loads.

The base plate for the main column of the steel sculpture is 1ft 4 in. long, 11in. wide and 1½ in. thick. It is secured to the concrete foundation with six 1¼-in. diameter anchor rods with leveling nuts. The column is attached to the plate with a ½ in. fillet weld all around the column section.

The north side of the base plate has a ¾ in. thick, 4¼-in. long (each leg) triangular stiffener. The legs of the triangular stiffener have been welded to the column flange and to the base plate using a ³⁄₈-in. fillet weld. This triangular stiffener serves stiffen the base plate in cases where moment must be transferred.

The south side of the base plate has a series of three, 4-in. by 8-in. by ¾-in. thick vertical plate stiffeners. These plates are attached to the column flange using ³⁄₈-in. fillet welds on each side of the plates. A gap is provided between the top of the base plate and the bottom edge of the stiffeners. These gaps avoid having the stiffeners bear directly on plate, thus reducing the possibility of buckling of the plate stiffeners. Additionally a horizontal plate is attached across the top edges of the plate stiffeners by a ³⁄₈-inch fillet weld. This upper plate has two bolt holes aligned with the anchor rod holes in the base plate. The anchor rods extend up through both the base plate and upper plate. There is one each anchor rod between each pair of vertical plates. The three vertical plates should be placed as close together as possible and washers under the nuts of the anchor rods are used to minimize any bending of the upper plate.

The base plate attachment on the steel sculpture shows very different connection geometry between the north and the south sides. In practice only one of the methods would be used on both sides stiffen and attach the column to the base plate. The steel sculpture connection is for illustrative purposes.

The steel sculpture also illustrates a base plate connection for a bracing strut. The pipe column, Column C3, is supported by Girder B4 and supports Girder B8 above. Plates have been welded to each end of the pipe column. The plates are bolted to the top and bottom flanges of Girders B4 and B8, respectively.

Two different web stiffeners have been welded on each side of the web of Girder B4. The stiffeners improve the web buckling performance of the girder. Each of the web stiffeners is essentially the same, the difference lies in that south side web stiffener extends completely from the top flange to the bottom flange of Girder B4. The north side stiffener is cut short and a gap is provided between the bottom edge of the stiffener and the bottom flange of the girder. The gap avoids having the stiffener bear directly on the bottom flange and allows for less restrictive fabrication tolerances. This arrangement is for illustration only. Again, in practice only one stiffener configuration would be used.

Figure 6-4. Column Connection: Base plate
Foundation / Column C1

Figure 6-7. Column Connection: Base plate
Foundation / Column C1
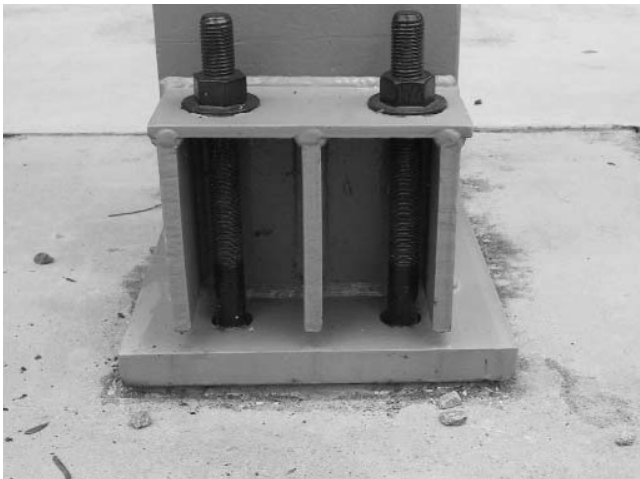
Figure 6-5. Column Connection: Base plate
Foundation / Column C1

Figure 6-8. Column Connection: Base plate (bracing column)
Girder B4 / Column C3 / Girder B8

Figure 6-6. Column Connection: Base Plate
Foundation / Column C1

Figure 6-9. Column Connection: Base Plate
(bracing column)
Girder B4 / Column C3 / Girder B8

Figure 6-10
Column Connection: Base Plate
Foundation / Column C1

N

Limit States

Bolt Bearing
Bolt Shear
Bolt Tension
Flexural Yielding
Weld Strength
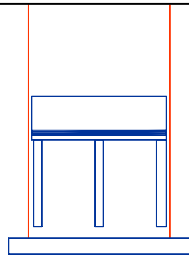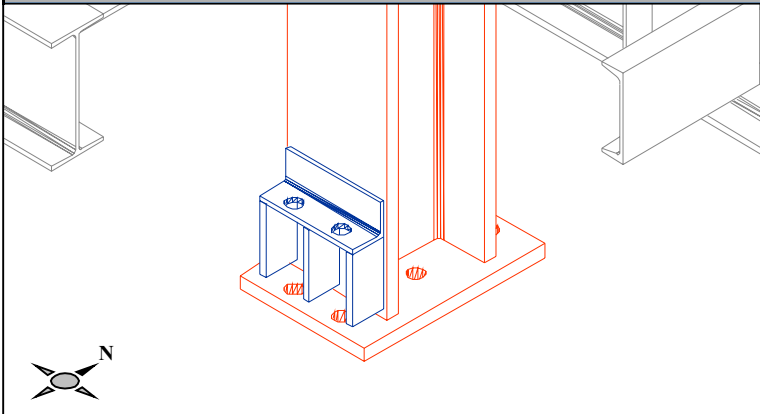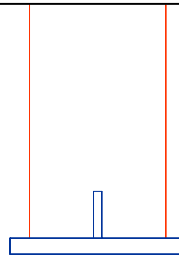
South Elevation

North Elevation
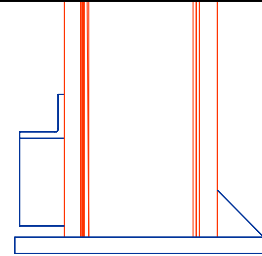
East Elevation

Notes

- The column base plate provides suitable bearing area to prevent crushing of the concrete foundation.

- The base plate on the steel sculpture has two different arrangements on one connection. This particular layout is for illustration only. In practice, one form of stiffening or the other would be utilized on both flanges of the column.

    NORTH FACE:
    The north side attachment of the base plate is a moment connection. This design uses vertical plate stiffeners with a welded top angle. There is a small gap between the bottom edge of the stiffeners and the base plate. This gap avoids having to fit the stiffener to bear directly against the base-plate. The top angle provides a bearing surface for the anchor rods and a means to level the column.

    SOUTH FACE:
    The south side attachment of the base plate is a moment connection. This design uses a single triangular plate stiffener welded between the top of the base plate and the southern face of the column flange. Although the triangular stiffener tends to reduce bending, the primary purpose is to provide additional material to increase the weld length and thus the shear strength of the base plate.

Figure 6-11	Limit States
Column Connection: Base Plate (Bracing Column)	
Girder B4 / Column C3 / Girder B8	

Column Connection: Base Plate (Bracing Column)
Girder B4 / Column C3 / Girder B8

Limit States

Bolt Bearing
Bolt Shear
Bolt Tension
Prying Action
Shear Rupture
Shear Yielding
Weld Strength

N
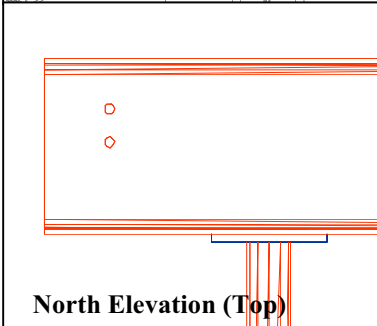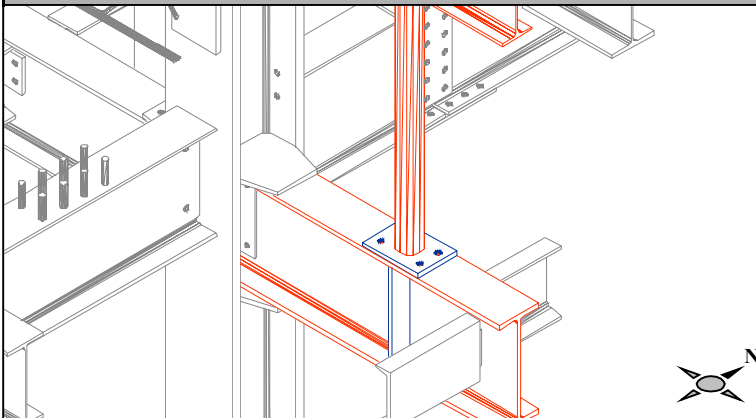
North Elevation (Top)	North Elevation (Bottom)	East Elevation (Bottom)
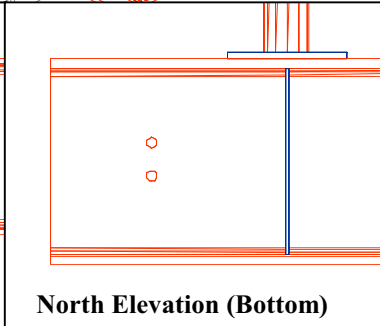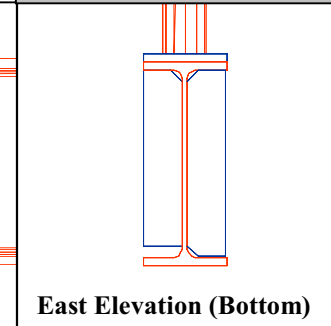
Notes

- The bracing column is bolted to the top flange of Girder B4 and the bottom flange of Girder B8.

- The plate is shop welded to the supporting pipe column and bolted to the supported girder.

- The pipe column acts as a compression/tension brace for Girder B8. Girder B8 is cantilevered and simply supported at the column. Girder B6 would be unstable without the pipe column to transfer the load to rigidly supported Girder B4 below.

- The brace acts primarily in compression, however the connections of the pipe column should be designed for any shear (i.e. lateral) and/or tension (i.e. uplift) loads.

- Acting in compression, the pipe column delivers a concentrated load to the web of the lower girder (Girder B4). To manage the localized load from the pipe column, two different web stiffeners have been added to Girder B4. Each of the web stiffeners is essentially the same, the difference lies in that south side web stiffener extends completely from the top flange to the bottom flange of Girder B4. The north side stiffener is cut short and a gap is provided between the bottom edge of the stiffener and the bottom flange of the girder. The gap avoids having the stiffener bear directly on the bottom flange and allows for less restrictive fabrication tolerances. This arrangement is for illustration only; in practice only one stiffener configuration would be used.

CHAPTER 7

Miscellaneous Connections

Miscellaneous connections are attachments that cannot be characterized by one of the connection categories previously discussed. These connections might be specified for specific types of members such as roof joists or truss members. They may also include connections with unusual framing geometry such as skewed or canted connections.

CLEVISES

Bracing members are typically used to add stiffness and/or stability to a structure. Most braces are axially loaded compression or tension members. Rarely will a bracing member need to transfer shear or flexure. Substantial material

savings in both the brace member and the connection may be possible if the bracing is designed as a tension only member. The advantage is that the brace member may be slender (such as a cable or rod). Stiffeners may be required to handle the concentrated forces often associated with axially loaded members.

Clevises are mechanical fixtures that are designed to transfer load from a threaded rod to a transverse pin. The clevis transfers tension from the threaded rod into double shear on the pin. The pin is secured through a hole in a plate that is joined to the appropriate steel member.

There are many sizes of clevises available to take an assortment of different rod and pin sizes. The clevises are

Figure 7-1. Misc. Connection: Clevis, plate & rod
Column C2 / Girder B6

Figure 7-3. Misc. Connection: Clevis, plate & rod.
Column C2 / Girder B6

Figure 7-2. Misc. Connection: Clevis, plate & rod.
Column C2 / Girder B6.

Figure 7-4. Misc. Connection: Clevis, plate & rod.
Column C2 / Girder B6.

Figure 7-5

Misc. Connection: Clevis, Plate & Rod

Column C2 / Girder B6

Limit States

Pin Bearing
Pin Shear
Tension Rupture
Tension Yielding
Weld Strength
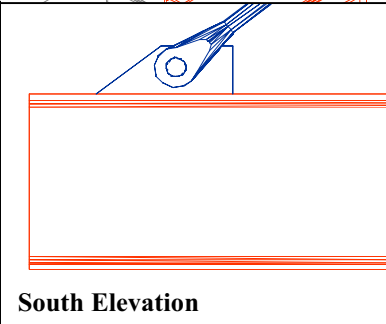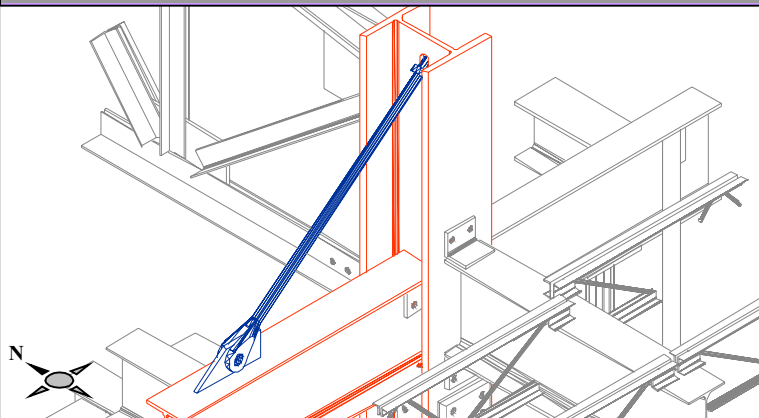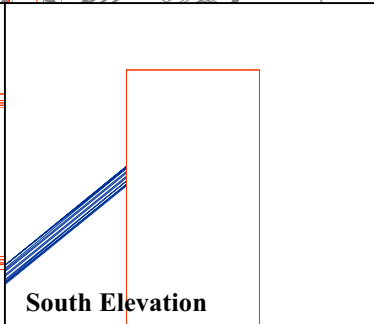
South Elevation

South Elevation
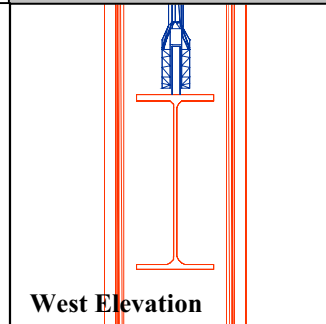
West Elevation

Notes

- Column C2 supports Girder B6 with a #3 clevis and $1^3/_8$ inch diameter rod.

- The rod acts as a support for the girder. The girder is simply supported at the column. Without the tension rod attached to the end, Girder B6 would be unstable.

- The pin that attaches the clevis to the plate is in double shear.

- A tapered washer and is used in conjunction with a nut to attach the rod to the column web.

- Strengths (and dimensions and weights) for different size clevises are given in the Manual of Steel Construction.

classified based on a clevis number that represents the outer diameter of the eye. The design strength of a clevis is based on the size of the clevis.

The tension rod that supports propped cantilever Girder B6 is probably the most obvious bracing member on the steel sculpture. Girder B6 requires a tension brace because the connection that attaches the girder to the column is a seat connection. The seat connection is assumed to provide no rotational restraint, thus without the tension brace, the cantilevered girder would collapse.

The tension brace supports the girder with a #3 clevis that is screwed on the 4 in. long threaded end at the lower end of the rod. The clevis uses a 1¾-in. pin that is secured through a plate that is welded to the top flange of the girder. The opposite end of the rod passes through a slotted hole in the column web and is anchored with a tapered (or hillside) washer and a nut.

SKEWED CONNECTION (BENT-PLATE)

Skewed connections result from members that do not frame together in an orthogonal fashion. Skewed connections may be made by a bent plate, or if more strength is required, a pair of plates may be bent. The design of a bent plate or skewed double-plate is analogous to similar shear connections. The plate or double-plates may be bolted or welded on the supporting side or the supported side. Extended gages may be necessary for suitable entering and tightening clearances. Any eccentricities that result from skew should be duly considered.

A single bent plate at each end attaches the channel brace that obliquely spans between the ends of Girders B4 and B5. The skew angle is about 45 degrees. The plates are field bolted to the girder and shop welded to the channel.

Figure 7-6. Misc. Connection: Bolted-welded bent-plate.
Girder B5 / Beam B5A

Figure 7-8. Misc. Connection: Welded joist
Girder B5 / Joists B9A & B9B

Figure 7-7. Misc. Connection: Bolted-welded bent-plate.
Girder B5 / Beam B5A

Figure 7-9. Misc. Connection: Welded joist.
Girder B5 / Joists B9A & B9B

Figure 7-10
Misc. Connection: Bolted-Welded Bent-Plate
Girder B5 / Beam B5A

Limit States

Block Shear Rupture
Bolt Bearing
Bolt Shear
Shear Rupture
Shear Yielding
Weld Strength

N
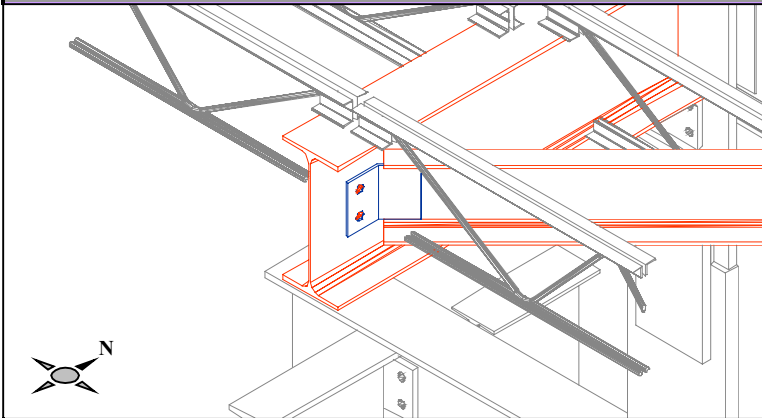
East Elevation

South Elevation
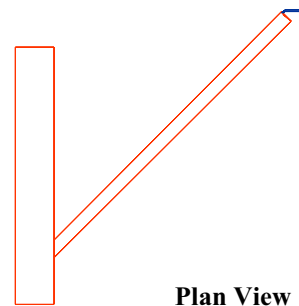
Plan View

Notes

- **Beam B5A supports/braces Girder B5 by a bolted-welded bent plate connection.**

- **The plate is field bolted to the girder and shop welded to the beam/brace.**

- **Girder B5 is simply supported at the column. Without the brace attached to the end, Girder B5 would be unstable.**

- **Skew connections may be furnished with bent double-angles (for small skew angles) or bent plate(s) (for large skew angles).**

- **Larger gages may be required to accommodate entering and tightening clearances.**

- **Due to skew, in-plane and out-of-plane eccentricities should be considered.**

Figure 7-11
Misc. Connection: Welded Joist
Girder B5 / Joists B9A & B9B

Limit States

Weld Strength

N
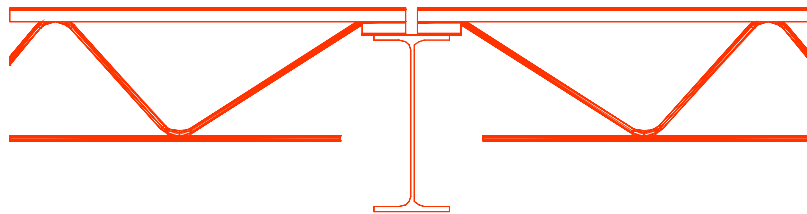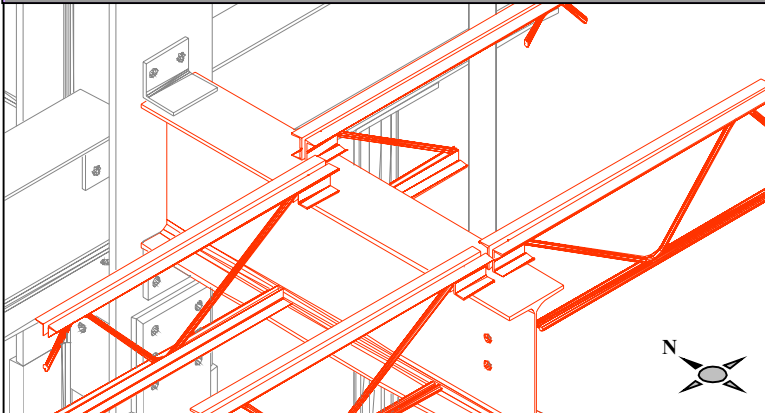
South Elevation

Notes

- **Girder B5 supports joists B9A and B9B.**

- **The joists are welded to the top flange of Girder B5**

- **Consult joist manufacturer for connection requirements and specifications.**

OPEN WEB STEEL JOIST

Open web steel joists are pre-manufactured standardized structural members. The Steel Joist Institute (SJI) and joist manufacturers provide connection guidelines for these members.

Four open web steel joists welded to Girder B5 illustrate the attachment of these members. When used for roof framing, the joist connection is typically subject to bearing and tension (uplift) loadings.

COLD-FORMED ROOF PURLIN

In metal building systems, roof purlins are often cold-formed steel members. Cutting, rolling, and bending thin sheets of steel form cold-formed steel members. Connection guidelines for cold-formed members can be found in the American Iron and Steel Institute (AISI) "Cold-Formed Steel Design Manual."

The Z shaped cold-formed steel roof purlin B9 is attached to Truss B7 on the steel sculpture. The purlin has been bolted to the top chord of the roof truss. Z purlins have point symmetry and have a tendency to rollover on inclined slopes (such as this roof truss). To prevent the Z purlins from rolling over, the upper flange should be installed facing up slope.

SHEAR STUD CONNECTORS

Shear stud connectors are used in composite construction to transfer horizontal shear forces between a steel beam and a concrete slab. The studs are resistance welded to the top flange of the beam through the metal deck (note: no metal decking is included in the Steel Sculpture). When a concrete slab is poured over the metal deck and beam, the studs become embedded in the concrete. A group of eight shear studs have been attached to Girder B1 on the steel sculpture.

TRUSS CONNECTIONS

Trusses are typically used as a roof framing system or in bridge applications. Members are loaded either in tension, compression, or bending and may be welded together, or bolted or welded together using gusset plates. Gusset plates provide the extra area necessary for welding or bolting. If the connections are bolted and the truss is for a bridge application, then the moving load causes stress reversal in the members and slip-critical joints must be used. When several truss members frame together, the line of action for the force should preferably meet at a common work point. Gusset plates must be sized for buckling under compression, tension yielding, tension rupture, block shear rupture and Whitmore section buckling and yielding.

On the steel sculpture Truss B7 is framed into the north face of Column C2 using gusset plates.

Figure 7-12. Misc. Connection: Z Purlin. Truss B7 / Purlin B9.
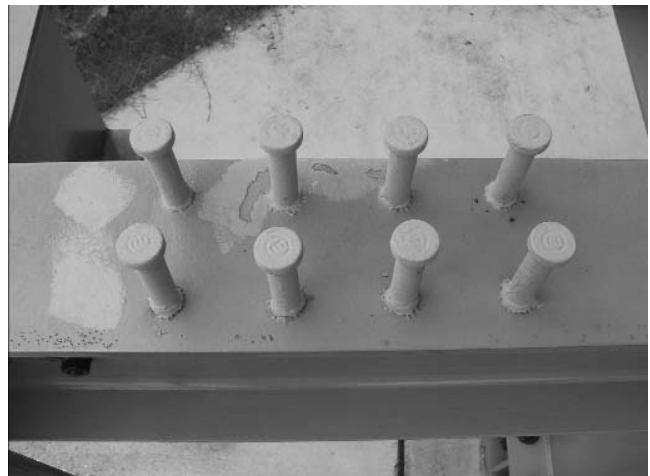
Figure 7-13. Misc Connections: Shear studs. Girder B1 / Shear Studs

Figure 7-14. Misc Connections: Shear studs. Girder B1 / Shear Studs

Figure 7-15
Misc. Connection: Cold Formed Z Purlin
Truss B7 / Purlin B9

Limit States

Bolt Bearing
Bolt Shear
Bolt Tension
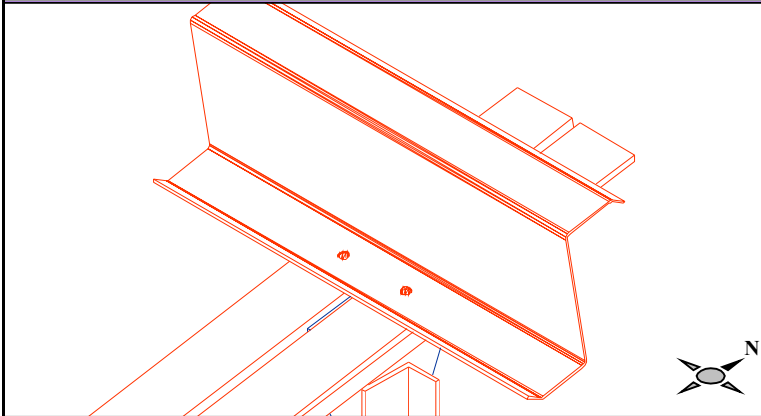
Notes

- **Truss B7 supports cold formed Z roof purlin B9**

- **Z purlins have point symmetry and have a tendency to rollover on inclined slopes (such as those attached to a roof truss). To prevent the Z purlins from rolling over, the upper flange should be installed in facing up slope.**

- **Consult American Iron and Steel Institute (AISI) "Cold Formed Steel Design Manual" for connection design requirements and specifications.**

Figure 7-16
Misc. Connection: Shear Studs
Girder B1 / Shear Studs
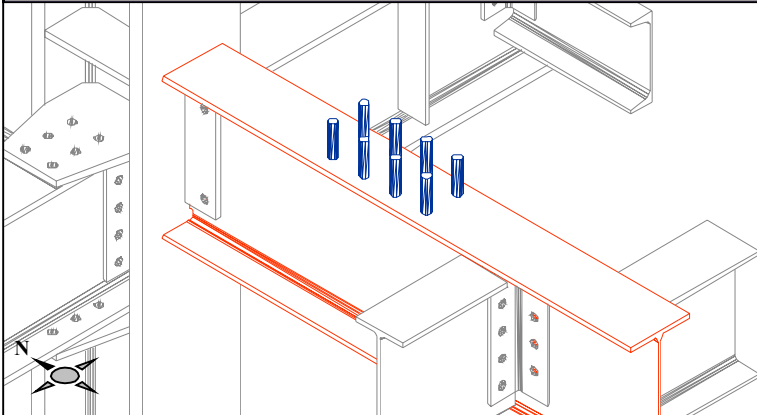
Limit States

Stud Shear
Weld Strength

Notes

- **Girder B1 has several headed shear studs welded to the top flange.**

- **The studs transfer horizontal shear force between a concrete slab and a steel beam. With the shear studs embedded in the concrete, the slab and the steel girder act as a composite member.**

- **Consult section I5 of the AISC Specification for shear stud design requirements and specifications.**

Figure 7-17. Misc. Connection: Truss Column C2 / Truss B7

Figure 7-18. Misc Connection: Truss Column C2 / Truss B7

Figure 7-19
Misc. Connection: Truss
Column C2 / Truss B7

Limit States

Bolt Bearing
Bolt Shear
Plate Buckling
Tension Yielding
Weld Strength
Whitemore
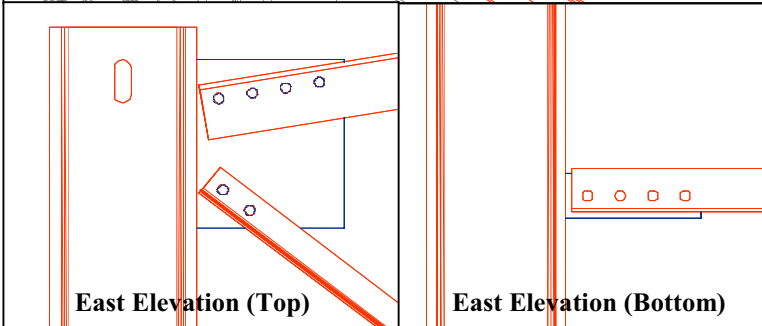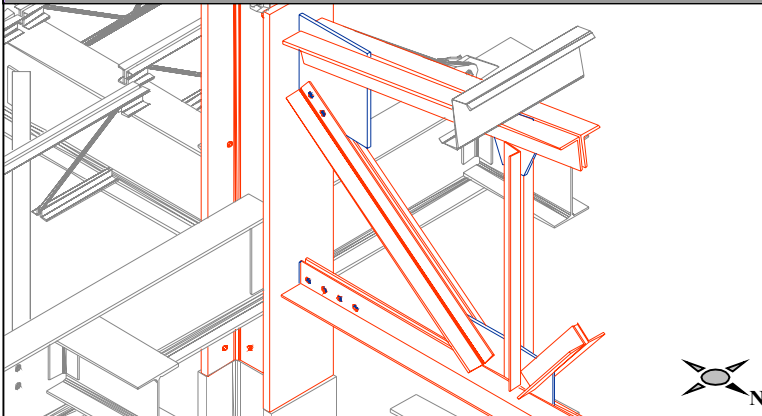
East Elevation (Top)   East Elevation (Bottom)

Notes

- Column C2 supports Truss B7 by gusset plates welded to the column and bolted to the truss. All-welded gusset plate connections join the interior members of the truss.

- The support gussets have been shop welded to the supporting column and field bolted to the supported truss.

- Trusses assembled with unequal leg double-angle struts should have the long legs back to back.

CHAPTER 8

Closing Remarks

The importance of proper connection design cannot be overstated. Connection design requires satisfying several, sometimes conflicting, design requirements thereby making them less tolerant to changes in design parameters. Slight adjustments in the design parameters of connections may drastically affect the strength and/or performance of the connections. In general, connections have many limit states, many of which are based on relatively complex behavior, which need to be considered.

APPENDIX A

MathCAD Worksheets User's Guide

A series of MathCAD® shear connection analysis worksheets have been developed as part of the Steel sculpture Teaching tool kit. Currently the worksheets are only for the analysis of six simple shear connections. The have been designed as teaching tools to give students a sense of the level of intricacy involved in designing steel shear connections. The worksheets follow the design and analysis provisions outlined in the AISC Manual. The software is for educational purposes only, and while every effort has been made to ensure correctness, there is no guarantee against either computational or procedural errors.

FEATURES:

- Simple to use.
- Complies with AISC-LRFD Manual of Steel Construction, 2nd Edition
- Over 25 limit states (including bolt and weld group eccentricity) are checked.
- Six shear connection modules (Double-angle, Shear End-plate, Extended End-plate, Single-angle, Tee, and Unstiffened Seated Connection) are included.
- Capable of analyzing a virtually unlimited number of connection geometries.
- Design/Feasibility/Serviceability checks are performed.
- Ply independent analysis (i.e. the software can handle any combination of bolts and weld sizes and strengths between the supporting and supported plies).
- The software is customizable (i.e. users can define and write custom functions).
- Other features of the software (i.e. it is capable of handling copes, vertical alignments, and column or beam support).
- Bolted connections
  - ASTM A325 or A490 bolts
  - Supports all bolt sizes permitted by the AISC-LRFD Manual of Steel Construction, 2nd Edition
  - Supports all thread conditions (iNcluded, eXcluded, Slip-Critical)
  - Supports all bolt hole types (STD, OVS, SSLP, SSLT, LSLP, LSLT)
- Welded connections
  - Any matched, under-matched, or over-matched weld can be specified
  - Supports four weld processes SMAW, GMAW, FCAW, and SAW

SYSTEM REQUIREMENTS
(RECOMMENDED MINIMUMS)

HARDWARE:

- Pentium II 300 mHz w/64 MB
- 10 MB hard disk space

SOFTWARE:

- MathCAD® 2000 or later
- Windows 95, Windows NT 4.0, or later

INSTALLATION

These worksheets can be installed anywhere on the hard drive as long as the relative directory structure remains constant. The required relative file and directory structure is as follows:

```
\ Connection Modules
\ Double-Angle Connection \Double-Angle.mcd
\ Shear End-Plate Connection \ Shear End-Plate.mcd
\ Single-Angle Connection \ Single-Angle.mcd
\ Single-plate Connection \ Single-plate.mcd
\ Tee connection \ Tee.mcd
\ Unstiffened Seat Connection \ Unstiffened Seat.mcd
\ Libraries
\ Definitions \ Definitions.mcd
\ Design Checks \ Design Checks.mcd
\ Miscellaneous \ Miscellaneous.mcd
\ Phi Factors \ Phi Factors.mcd
\ Section Properties \ Section Properties.mcd
\ User \ User.mcd
HUB.mcd
```

To install the worksheets simply decompress the zip file in the desired directory. The above directory structure will automatically be created.

USING THE WORKSHEETS

- Launch MathCAD® 2000 or later.
- Units are not labeled in the worksheets; however, the fundamental unit values are the in. and kip.

CONNECTION ANALYSIS

- Open the preferred connection module.
- Input values for all input parameters (MathCAD® does not handle null values).
- Output is registered in the two tables (Summary 1 and Summary 2 at the end of the worksheet.
- Expand the collapsed yellow areas for additional, more detailed connection data.

COMPARATIVE STUDIES

Change any of the input values and the worksheets will automatically update the output values.

FUNCTION INVESTIGATION

- To see how particular functions are defined, open the appropriate library

**NOTE: There is no active worksheet protection. Use caution when modifying and saving worksheets.

HOW IT WORKS

Although extensive knowledge of MathCAD® is not required, most users will find it helpful to understand the basic structure of how the software application works. There are essentially two main types of worksheets: libraries and connection modules. Connection modules are the front-side, input-output part of the software. Libraries are lists of functions and/or definitions that are referenced by the connection modules.

Connection Modules

These are the MathCAD® worksheets that the user will work with to perform a particular connection analysis. There is one connection module for each type of shear connection (Double-angle, Single-angle, Tee, etc.) The user will enter the load, the section and connection geometries, the joint type and other properties, in the connection module. At the end of the worksheet are two output tables summarizing the analysis results. The first output table (Summary 1) lists the limit states and their applicability or strength. The second table (Summary 2) lists serviceability and/or other design checks and whether the required design criteria have been met.

Each connection module is organized in a similar fashion. The various input and output sections are arranged in the same manner from module to module. In addition, the connection modules take advantage of the organizational features of MathCAD® including colors, fonts, and areas. Most descriptive text and headings are in a black Arial font. Functions, definitions, variables, etc. are in a blue Times New Roman font. Whenever a particular property of the

connection does not apply, the heading will be subdued with a gray font and "**Not applicable to this connection**" will be written below.

Areas are a MathCAD® feature that allows sections of a worksheet to be collapsed and/or locked. Areas can be recognized as either a gray or yellow colored stripe with an arrow and heading at the left margin. Grey areas indicate regions of internal calculations that are either the extensive connection property calculations or sorting/organizational in nature. These regions are typically unnecessary for expansion and viewing. (The gray areas have been locked by default. No password is required to unlock these areas.) The yellow areas are either output, or calculations that may be of interest to the user. To expand or collapse an area, double click on the arrow near the left margin. (See MathCAD® documentation for more information on areas.)

The placement of the headings within the worksheets has some significance. Each ply and/or member of a particular connection is organized into a "column" in the connection modules. The columns are arranged left to right for a right-hand connection. For example the first column represents the supporting member; the middle column represents the connection element and the last column represents the supported member. The idea of independent plies is extended even further by making use of an extensive subscripting convention. Most variables have subscript text describing which ply or connection side they are associated.

Libraries

The second type of worksheet is the library. Libraries are simply worksheets that contain nothing but globally defined functions or definitions. The above mentioned connection module worksheets reference all the libraries so that the module can have access (call upon) a particular function. There are several libraries available to each module. These libraries are characterized by the purpose of the functions that they contain.

Currently, there are seven library worksheets. Each library is referenced by the HUB worksheet. The HUB worksheet is in turn referenced by each connection module worksheet. Thus, each connection module has access to each library. A description of each library and an example function or definition for most libraries follows:

- Definitions.mcd
  This library lists the definitions used in the connection modules. Text-based input parameters are assigned integer values so they will work properly inside of functions and programs. All text based input parameters are written in all capital letters. The origin is defined as one (ORIGIN=1) in all worksheets and for all functions.

Example: Simple integer values are assigned to variables for thread condition definition.

- Design Checks.mcd
  The design checks library contains functions that determine if general design criteria have been met. These functions also make sure that there is no geometric interference between the connection elements given their size and position.

  Example: Minimum factored load function. This function requires only one argument, Pu, and does a simple check to see that the argument is greater than 10 kips. The function returns the caption "Minimum factored load is 10 kips if the criteria is not met, or "OK" otherwise.

- Limit States.mcd
  The functions in the limit states library are from the AISC-LRFD Manual of Steel Construction. These are the main functions to calculate the strength for a given limit state.

  Example: Shear yielding limit state function. This function requires two arguments, Ag and F_y, and calculates the nominal shear yielding strength.

- Miscellaneous.mcd
  The miscellaneous library contains those functions that do not fall neatly within one of the other libraries. Many of these functions reproduce chart and table values found in the AISC-LRFD Manual of Steel Construction.

  Example: Function to return the effective throat of the weld, based on the size of the weld and the process used. This function requires two arguments: Process and w.

- Phi Factors.mcd
  The phi factors library defines the resistance factors for different categories of limit states from the AISC-LRFD Manual of Steel Construction.

  Example: Definition that assigns the value 0.90 to the phi factor based on shear yielding.

- Section Properties.mcd
  The section properties library contains only two functions. The first returns section properties for a W shape, the second returns section properties for a WT shape.

- User.mcd
  The user library is a blank/empty worksheet library for that is available for user-defined functions.

SUBSCRIPTING

The worksheet variables use an extensive subscripting convention to indicate which element or ply the variable is applicable. For purposes of this manual the subscript position in variable definitions and explanations is indicated by "…" where "…" represents the actual subscript

Subscript:	Associated Element or Ply:
Supporting	Element: Supporting member.
Supported	Element: Supported member.
Beam_Web	Ply: Supported member web.
T	Element: Shear tee.
Tee_Stem	Ply: Shear tee stem.
Tee_Flange	Ply: Shear tee flange.
A	Element: Angle or double-angle.
Leg1, Legs1	Ply: Out-of-plane angle leg(s).
Leg2, Legs2	Ply: In-plane angle leg(s).
Outstanding_Leg,	Ply: Outstanding angle leg (seat connection).
Vertical_Leg	Ply: Vertical angle leg (seat connection).
PL	Element: Plate.

FUNCTIONS, INPUT, OUTPUT, LIMITATIONS

Functions

Most calculations in the worksheets are done by functions. Functions are essentially subroutines that have arguments passed to them, perform calculations on the arguments and then return results. The advantage of functions is that they can be used repeatedly to perform the same operation using different argument values. Functions can be identified in the worksheets by the prime " ' " notation that precedes the name of every function. Most functions are defined in one of the previously mentioned libraries.

Input

The input region should be evident in the worksheets. All input parameters are essentially the same from module to module. Values are required for each variable under the Input Parameters heading. This is true regardless of the bolting and welding combination. MathCAD® cannot have null values for variables.

Input Parameter:	**Explanation:**
Pu	Connection shear loading in kips. Only gravity loads are permitted. Minimum load (based on AISC-LRFD Manual of Steel Construction) is 10 kips.
Support	Support condition. This variable takes a text-based input value. Determines whether the girder web (WEBG), column web (WEBC), or column flange (FLANGE) supports the connection.
m.	Mean slip coefficient. Value of slip coefficient between faying surfaces.
Setback	Beam setback in inches Shortest distance from face of support to the web of supported beam $F_{y\_}$… Yield strength of particular member in ksi. $F_{u\_}$… Ultimate strength of particular member in ksi.
D_...	Member designation, nominal depth in inches, for supporting and supported member. AISC wide flange members only (WT members available as connection element for the Shear tee connection module).
W_…	Member designation, nominal weight per linear foot in pounds, for supporting and supported member. AISC wide flange members only (WT members available as connection element for the Shear tee connection module).
Leg1_…	Outstanding leg (or vertical leg for seated connection) length in inches
Leg2_…	In-plane leg (or horizontal leg for seated connection) length in inches
W_…	Width of plate in inches
L_…	Length of plate in inches
t_…	Thickness of connection element in inches
Joint_…	Joint condition. This variable takes a text-based input value. Determines whether the joint is bolted (BOLTED) or welded (WELDED).
y_…	Vertical alignment in inches Alignment is measured from the mid-depth of the connection element to the mid-depth of the beam or girder.
dct	Depth of top cope in inches Enter 0 for no top cope. Copes must be deeper than the flange thickness.
dcb	Depth of bottom cope in inches Enter 0 for no bottom cope. Copes must be deeper than the flange thickness.
c	Length of cope(s) in inches Both top and bottom copes must be the same length.
FEXX_…	Weld metal strength in ksi.
Process_…	Welding process. This variable takes a text-based input value. Determines whether the weld is made by the shielded metal arc welding (SMAW), flux core arc welding (FCAW), gas metal arc welding (GMAW), or submerged arc welding (SAW) process.
weld_…	Weld size in inches
dweld_…	Length (Height) of vertical weld in inches
bweld_…	Length (Width) of horizontal weld segments in inches (Both top and bottom horizontal weld segments must be the same length).
db_…	Bolt diameter in inches.

ASTM_...	ASTM Bolt designation. Determines whether the bolts are A325 (325) or A490 (490).

Threads_...	Bolt group thread condition. This variable takes a text-based input value. Determines whether the bolt threads are iNcluded (N), eXcluded (X), or Slip-critical (SC).

s_...	Bolt spacing in inches (spacing is measured parallel to direction of load).

g_...	Bolt gage in inches (gage is measured normal to direction of load).

Rows_...	Number of bolt rows on a given side of the connection (rows are normal to direction of load).

Columns_...	Number of bolt lines on a given side of the connection (columns are parallel to direction of load).

Columns_Supporting_HF

Number of bolt lines per flange of a double-angle or shear tee connection.

Hole_...	Hole type for a given ply of material: Standard (STD), Oversized (OVS), Short-slotted/load transverse (SSLT), Short-slotted/load parallel (SSLP), Long-slotted/load transverse (LSLT), or Long-slotted/load parallel (LSLP)

Lev_...	Vertical edge distance, in inches, for a given ply of connection element.

Leh_...	Horizontal edge distance, in inches, for a given ply of connection element.

Output

Primary results (limit state strengths and design checks) are returned in two tables at the end of the connection modules. The first table (Summary 1) returns the connection loading and strengths of applicable limit states. No flags or warnings are issued if the strength of a limit state is less than the required strength. The output table only provides a listing of the applicable limit states and their respective strengths. If a limit is not applicable for given the connection parameters, "NA" is returned.

Limit State:		**Explanation:**

Slip		Slip resistance based on factored load. If the connection is bearing, then resistance to slip is 0 kips.

Shear Yielding	Shear yielding for each applicable ply of material. This limit state applies to the connection element and the supported member if it is double coped.

Shear Rupture	Shear rupture for each applicable ply of material. This limit state applies to the connection element and the supported member if it is double coped.

Block Shear Rupture

Block shear rupture for each applicable ply of material. This limit state applies to the connection element if it is bolted and to the supported member if it is bolted and top coped.

Bolt Shear	Bolt shear limit state for sides of the connection that are bolted.

Bolt Bearing	Bolt bearing limit state for sides of the connection that are bolted.

Weld		Weld strength for sides of the connection that are welded based on the strength of the weld or the base material.

Flexural Yielding of Beam

Flexural yielding limit state for supported beams that are coped.

Local Web Buckling of Beam

        Local web buckling limit state for supported beams that are top or double coped.

Local Web Yielding of Beam

        Local web yielding limit state for unstiffened seat connections.

Web Crippling of Beam

        Web crippling limit state for unstiffened seat connections.

Prying Action   Prying action limit state for shear tee connections with a bolted flange.

The second table (Summary 2) returns common design checks. "Design check" is a general term that relates to the serviceability, erectability, or connection performance issues. The design checks return either an "OK" meaning all required criteria for a particular design check has been met, or a short caption describing the issue for non-compliance.

There are two kinds of design checks. Some checks are specific to a particular connection type (e.g. single plate connection). Other checks are common to each shear connection (e.g. minimum weld size). If the design check is specific to a particular connection, its function is included inside the connection module worksheet. Only general design check functions are written in and thus called from the design checks worksheet library. Also, the general design checks for the different connection modules are not all-inclusive; the checks included tend to represent the most common concerns and issues.

Design Check: **Explanation:**

Minimum Factored Load

        Checks whether or not the user has input a design load of at least 10 kips.

Minimum Size of Cope(s)

        Checks that the cope(s) extend past the flange thickness of the supported beam.

Homogeneous Fastener Use

        Checks that the bolts are common to both sides of the connection for an all-bolted connection (i.e. both sides have same grade, same diameter, etc.)

Girder Alignment

        Checks that there is no geometric interference between the girder and the connection element based on the vertical alignment of the members.

Beam Alignment

        Checks that there is no geometric interference between the beam and the connection element based on the vertical alignment of the members.

Cope Length   Checks that the cope is long enough to accommodate the girder flange.

Interference   Checks that there is no geometric interference between the beam and the girder based on the vertical alignment of the members.

Beam vs. Girder

        Checks that the depth of the supported beam does not exceed the "T" dimension of the girder.

Beam vs. Column Web

        Checks that the width of the supported beam does not exceed the "T" dimension of the column.

Element vs. Beam

        Checks that the length of the connection element is compatible with the beam.

Element vs. Column

        Checks that the width of the connection element is compatible with the column.

Minimum Fillet Weld

> Checks that the fillet weld size satisfies the minimum size requirements.

Maximum Fillet Weld

> Checks that the fillet weld size satisfies the maximum size requirements.

Bolt Hole Check

> Checks the proper use of oversized, and slotted holes. Checks that oversized holes are used only in conjunction with slip-critical connections. Checks that the lengths of slotted holes are normal to direction of load in bearing connections. Checks that long slotted holes are used in only one ply of material.

Bolt Hole Spacing

> Checks for adequate bolt hole spacing.

Weld Length Check

> Checks that the element being welded has a sufficient edge length for the specified length of weld.

Additional, more detailed output and/or calculations can been viewed by expanding any or all of the yellow areas. The gray areas contain sorting functions or extensive internal calculations that would typically not need to be seen by the user.

Limitations

Although the worksheets are versatile, some limitations do exist:

- Only vertical weld lines are permitted on the supporting side of a connection. If horizontal welds are to be used on the supported side, they must be used at both the top and the bottom, and they must be the same length. (i.e. No L shaped weld groups are permitted).

- The unstiffened seat connections module is somewhat unique. It is the only shear connection with no joint at the web of the supported beam. No provisions exist in the worksheet for interference design checks or beam copes.

- Only wide flange shapes for beams and girders are presently allowed.

- Some of the design checks that analyze the compatibility of certain connection member lengths and widths (i.e. Beam vs. Girder, Beam vs. Column, Element vs. Beam) check the web depth of flange width of the beam against the T dimension of the girder or column. In some circumstances these checks are conservative. With careful geometry and alignment it may be possible to encroach on the rolling fillet areas of girders, beams, and columns.

- MathCAD® does handle null values for variables (there must be a value for every variable in the worksheet). To facilitate proper logic in the flow of the programs, several variables are defined that act as switches and tell the programs what variables and functions to calculate and which ones to ignore. The best example of this is the BOLTED/WELDED switch. Even if a connection is all-bolted, there must be values in the variables for welded properties. (e.g. enter 70 ksi welds, ¼ in. weld size, etc.) The text value definition BOLTED input at the JOINT variable effectively tells the worksheet to ignore any welded values and functions.

APPENDIX B

Sources for Additional Steel Connection Information

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