The Virtual Steel Sculpture –Limit State Analyses and Applications of Steel Connections

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Abstract

The integrity of a structural system depends on the strength of materials, shape of the individual member and the elements used to hold the members together. In most undergraduate civil engineering curricula, a structural steel and/or reinforced concrete design course is required. Usually, the main focus of these courses is on member selection (size and shape) for a given material. Moreover, the time available for connection design is usually limited. To provide a 24/7 access to students who are interested in learning more about steel connection design, with the support from the National Science Foundation, a virtual steel sculpture was developed. The virtual steel sculpture not only provides a 360-degree view of different type of connections, but also provides information on their limit states, field examples, and finite element models showing stress distributions. In this paper, the discussion is focused solely on the content of the sample design calculation for each connection shown in the virtual steel sculpture. These sample calculations were created as additional examples that could supplement class lecture to enhance students' learning.

Introduction

Through a grant from the National Science Foundation, a three dimensional virtual steel connection sculpture was developed based on the physical sculpture that is located on the campus of Minnesota State University, Mankato. [1]. The virtual steel sculpture (see figure 1) is discussed in detail in an article by Moaveni and Chou [1]. This easily maneuverable tool allows the user to zoom, pan, rotate, and spin the sculpture to view various connections from different directions. Moreover, one can retrieve additional information on a specific connection by moving the mouse cursor over it, holding the ctrl key and left clicking the mouse button. This action will open a 2d file as shown in figure 2. By hovering the mouse cursor over the tabs shown beneath the sculpture (see figure 2) one can then preview the blueprint that was used to fabricate the connection; close-up views of the connection, limit states calculations; field example photos, and finite element analysis of stress distribution within a connection. In this paper, the content of "sample calculations" file, which shows the limit states of a connection, is discussed. These sample calculations are intended for students who have little to no experience in steel design to learn about connection assembly and capacity analysis. The sample calculations are similar to those found in a typical introductory level steel design text book. They do not represent a specific connection within an actual structural system, but instead show the steps that must be followed using presumed values.

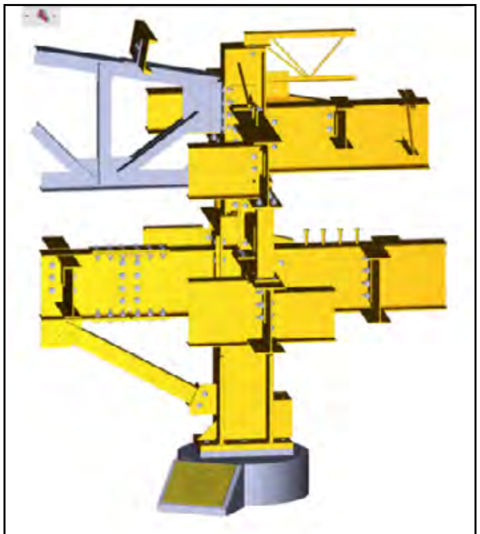

Figure 1. Virtual steel sculpture

The Virtual Steel Sculpture

The virtual steel sculpture consists of 48 different connections that are commonly found in construction practice.Connections are typically grouped as axial, shear, moment, or miscellaneous. The purpose of a connection is to transfer the force(s) from one member onto another member(s) safely. Efficiency and constructability during assembly also are considered when designing connections. The description of each connection type is given below.

An *axial connection* is the most basic connection type. As such, it is often the first studied by students when learning about steel connections. An axial connection is a system of bolts/welds that are used to support the load of a member with a force acting in a purely axial manner as shown in figure 3. An example of this connection type is the "frictionless" pin-connections of a truss.

A *shear connection* (see figure 4) is another common connection type. Like an axial connection, bolts and welds are used. However, unlike an axial connection, the forces in these connections do not act parallel to the axis of the beam. This non-parallel force creates shear in the connections. These connections are often referred to as hinged, because they allow the members to rotate relatively at the connection. This connection is commonly used to connect beams to columns or beams to beams.

While visually, a *moment connection* may not look too

Group	Description of connection group	Description of connection type	Connection numbers
I	axial	Bracing	16, 28, 40, 41
		Truss	45, 46, 47
II	shear	simple shear – beam to beam	1, 2, 8, 19, 20, 21, 22, 29, 36
		simple shear – beam to beam with flush flanges	7, 13, 14, 32
		simple shear – beam to column	4, 27, 35
		unstiffen seat	10, 44
		miscellaneous	5-6, 11-12
III	moment	column anchorage	17, 39
		end plate	15
IV	axial and shear	column post	3, 9
V	shear and moment	beam splices, column splices, beam-column (joint of moment frame)	23-24, 30-31, 25-26, 33-34, 42-43
VI	miscellaneous	joist supports	37, 38
		purlin	48
		shear stud	18

Table 1. Summary of connection types and individual connections in the virtual steel sculpture

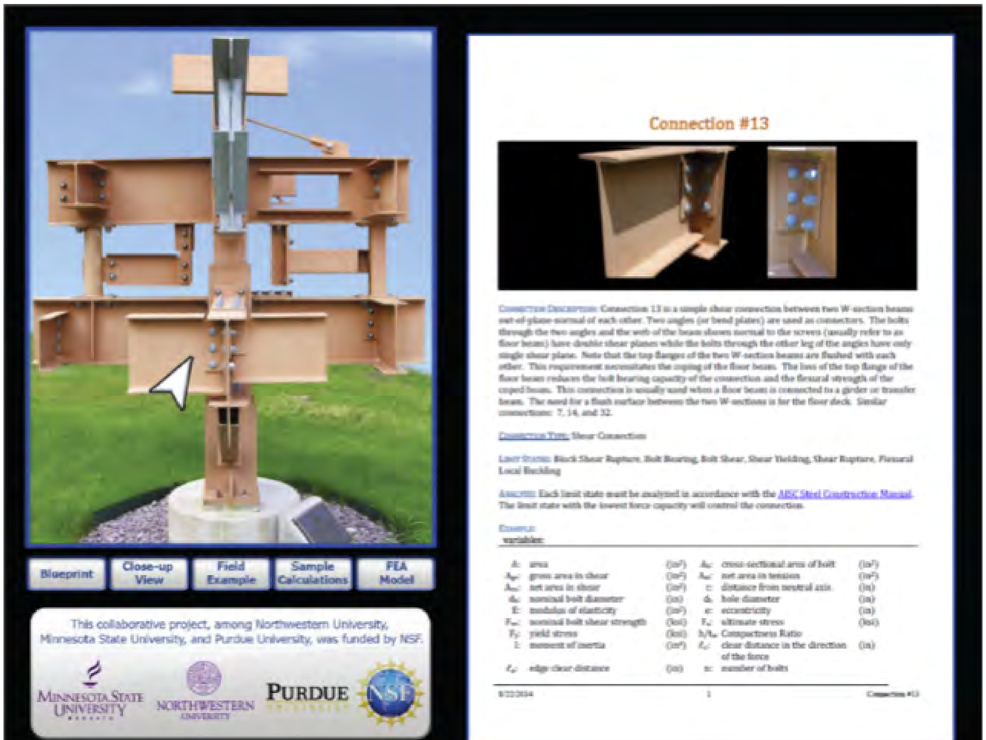

Figure 2. 2-Dimensional layout of the connection information.

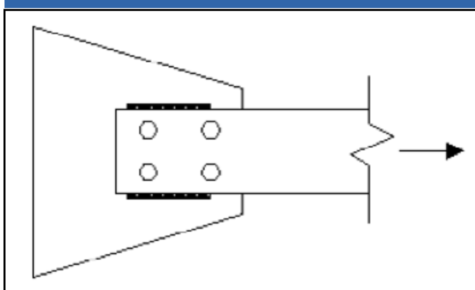

Figure 3. Axial Connection

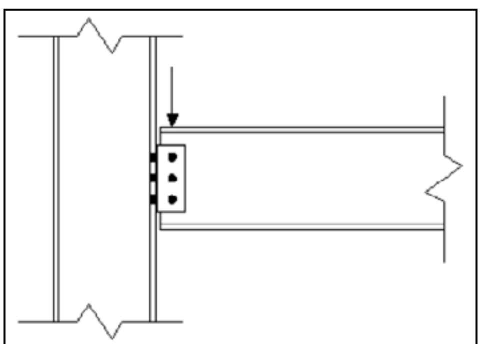

Figure 4. Shear Connection

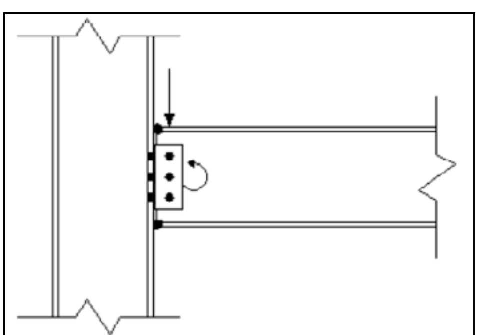

Figure 5. Shear-Moment Connection

different from a shear connection, it is used to resist an internal moment. A moment connection is usually used in conjunction with a shear connection as shown in figure 5. Often, in construction, it is required to have a system that can handle moments induced by the loading. These moment connections are often called rigid connections because they do not allow relative rotation at the connection. Note that the top and bottom flanges of the beam in the connection shown in figure 5, are either welded or bolted.

Table 1 shows a summary of connection types in the virtual steel sculpture. In table 1, the connection number is an identification number which was created by the authors. These numbers are used in the sculpture to name the calculation files that accompany the virtual model.

Connection Limit States

Connections serve as the "glue" that hold various members of a structure together. The integrity of a structure relies on this glue, and when a connection fails, a partial or entire structural system failure would occur. For steel structures, the glue is the steel bolts or the welds. A structural system would not be able to function or it would fail if

- A member's capacity is reached (such as yielding, buckling) under loads;
- A member is fractured anywhere, including the ends where they are attached to other member(s) through the bolts or welds or both;
- The bolts are sheared or fractured; or
- The welds are fractured or separated from the connecting members.

The limit states (failure modes) depend on the function of the connection. *The Connections Teaching Toolkit – A Teaching Guide For Structural Steel Connections [2] And Manual Of Steel Construction* [3] were used as references to identify the limit states of each connection for the virtual steel sculpture under the sample calculation tab [1].

In the following sections, the potential limit states are summarized and described. The alpha-numeric description following the limit state is the section or equation designation in the *AISC Steel Design Specifications* [3]. Definitions for limit states were largely based off *AISC'S Connections Teaching Toolkit* [2]. Many definitions were taken verbatim. Also, note that magenta color shaded areas represent the element that is being analyzed in each limit state.

Block shear rupture limit state j4.3

Block shear rupture (figure 6) is a limit state in which

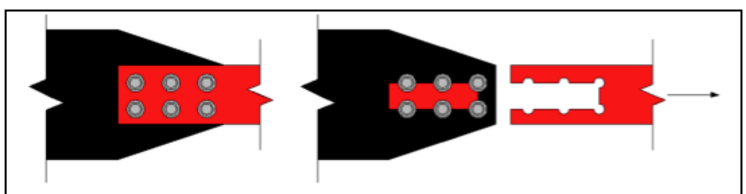

Figure 6. Block shear rupture limit state

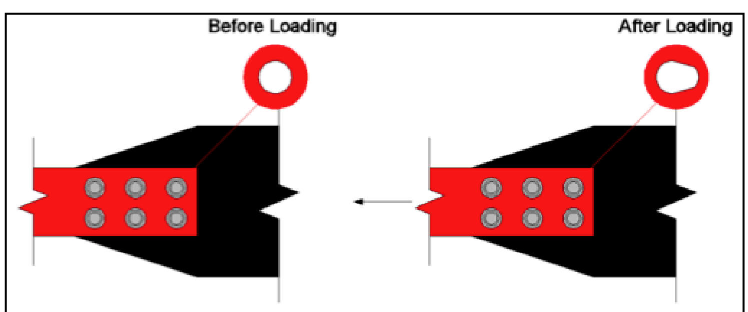

Figure 7. Bolt bearing strength limit state

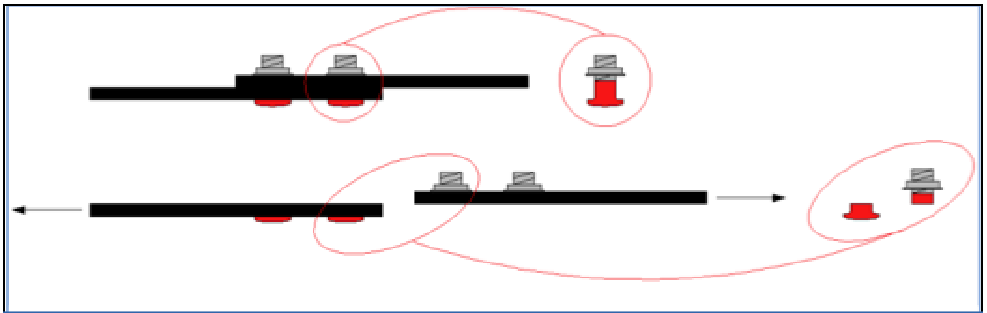

Figure 8. Bolt shear strength limit state

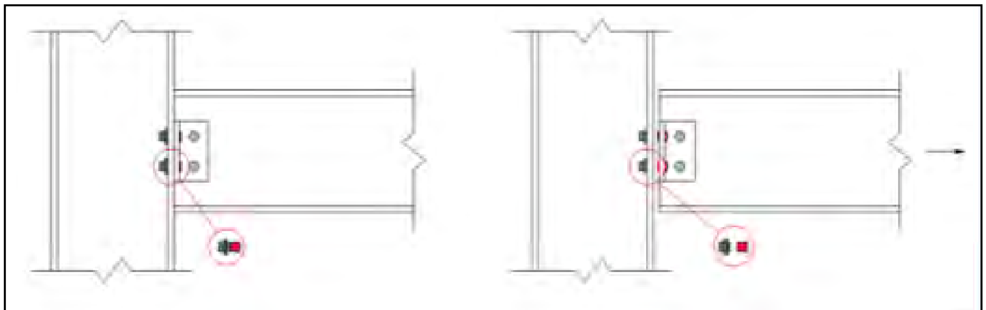

Figure 9. Bolt tension strength limit state

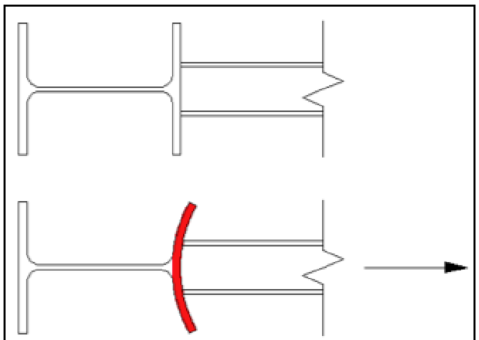

Figure 10. Flange local bending limit state (top view).

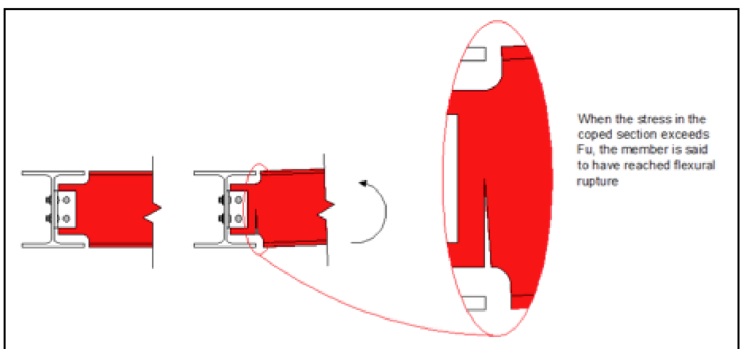

Figure 11. Flexural rupture limit state

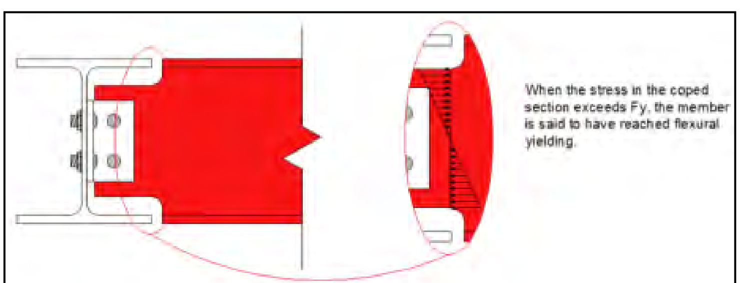

Figure 12. Flexural yielding limit state

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.

Bolt bearing strength limit state j3.10

Bolt bearing strength (figure 7) 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 or the spacing between two bolt holes, and the thickness of the connected members.

Bolt shear strength limit state j3.6

Bolt shear strength (figure 8) 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.

Bolt tension strength limit state j3.6

Bolt tension strength (figure 9) is applicable when bolts are subject to loading along their length. Bolts that fail in tension will do so within the threaded portion of the bolt, through one of the roots of the threads. This coincides with the least cross-sectional area.

Flange local bending limit state j10.1

Flange local bending (figure 10) is caused by a concentrated tensile force acting perpendicular to a beam's flange which causes increased stress in the beam flange.

Flexural rupture limit state j4.5

Flexural rupture (figure 11) occurs in moment connections where the connection must be designed to carry an applied moment. Rupture occurs when the stress caused by the applied moment is greater than or equal to the rupture strength of the material.

Flexural yielding limit state j4.5

Flexural yielding (figure 12) needs to be checked when a beam is coped. This is necessary because the reduced section modulus of the remaining beam cross section may significantly reduce the flexural strength of the member. Flexural yielding can also occur in moment connections where the connection must be designed to carry an applied moment. Yielding occurs when the stress caused by the applied moment is greater than or equal to the yield strength of the material.

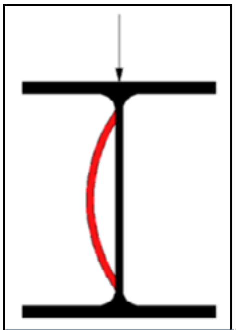

Figure 13. Local web buckling limit state

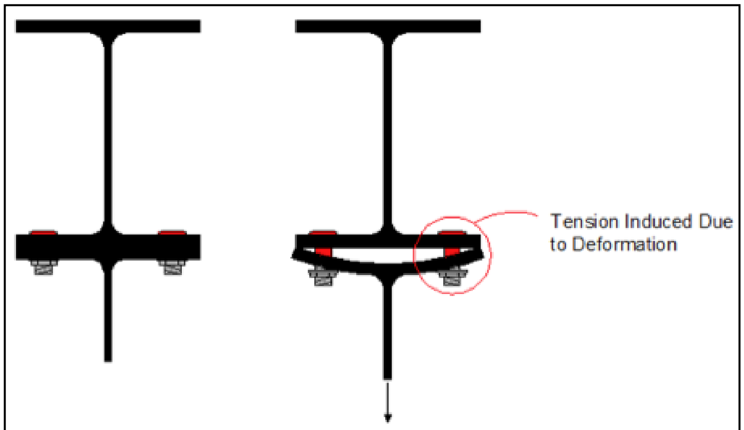

Figure 14. Prying action limit state

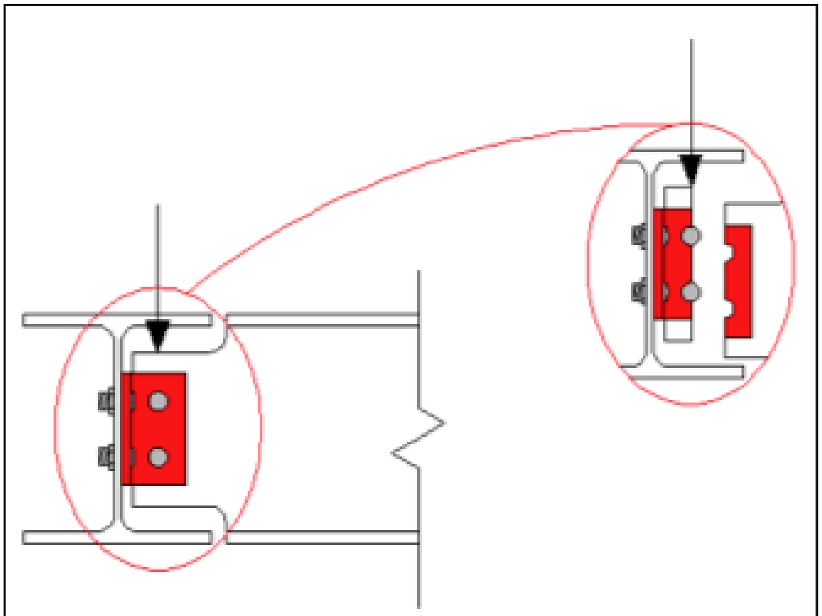

Figure 15. Shear rupture limit state

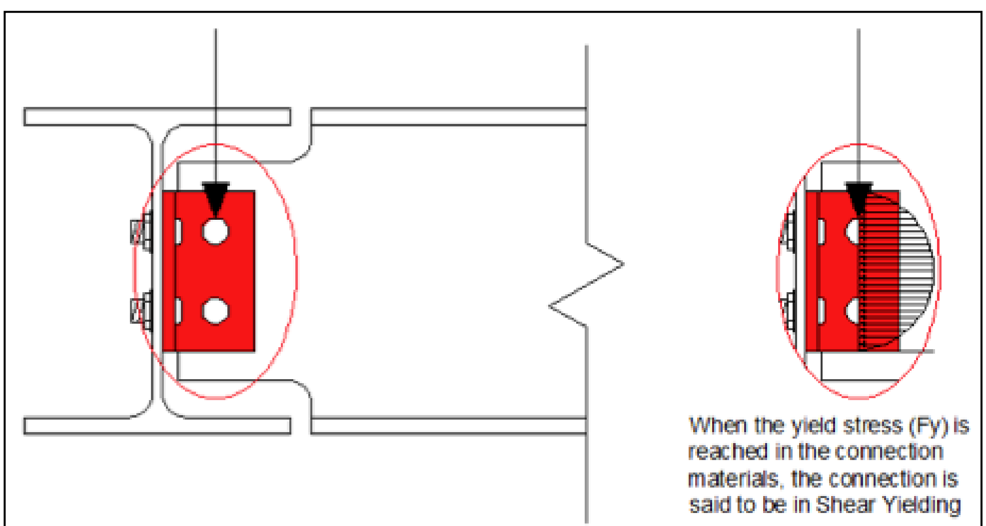

Figure 16. Shear yielding limit state

Local web buckling limit state table b4.1B

Local web buckling (figure 13) occurs when a member is considered slender. When a web is slender, it is not stable enough to properly support loading and will buckle. Because of this, the slenderness ratio of the web must be checked in accordance with table b4.1B.

Prying action limit state eq. 9-20A

Prying action (figure 14) 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.

Shear rupture limit state j4.2

Shear rupture (figure 15) is a function of the effective net area. The net area is the reduced gross area due to bolt holes or notches. When the section has a stress greater than or equal to the ultimate stress of the material it is made of, shear rupture is said to have occurred.

Shear yielding limit state j4.2

Shear yielding (figure 16) is a function of the gross cross-sectional area of the member subjected to a shear load. When the section has a stress greater than or equal to the yield stress of the material it is made of, shear yielding is said to have occurred.

Stud shear strength limit state i8.2A

Stud shear strength (figure 17) becomes a consideration when steel headed stud anchors are used to transfer shear from a concrete slab to a steel member. The shear strength is analyzed in each stud anchor to determine the permissible shear load per anchor that the system can withstand.

Tension rupture limit state j4.1

Tension rupture (figure 18) 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. When the section has a stress greater than or equal to the ultimate stress of the material it is made of, tension rupture is said to have occurred.

Tension yielding limit state j4.1

Tension yielding (figure 19) is a function of the gross cross-sectional area of the member subjected to tension load. When the section has a stress greater than or equal to the yield stress of the material it is made of, tension yielding is said to have occurred.

Web compression buckling limit state j10.5

Web compression buckling (figure 20) occurs when a concentrated force, distributed through a bearing plate to lower the applied stress, becomes too large for the web of the beam. This causes the beam to buckle similar to local web buckling.

Web crippling limit state j10.3

Web crippling (figure 21) occurs due to a concentrated compressive force acting on both flanges in line with the web. When the compressive force is large enough, the web of the beam will buckle similarly to local web buckling.

Web local yielding limit state j10.2

Web local yielding (figure 22) is caused by a compressive force acting on the beam perpendicular to the beam flange. This compressive force causes the web to develop a stress greater than or equal to the yield limit of the material and results in compressive crushing of the beam's web.

Weld strength limit state j2.4

Weld strength (figure 23) 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.

Sample calculations

The sample calculation pages associated with each connection type of the virtual steel sculpture include the following information:

- Images of the connection from the physical steel sculpture
- A description of the connection which presents the assembly, common applications, and other relevant information associated with the connection
- Connection type classification, whether it is axial, shear, moment, or miscellaneous
- An analysis section which shows the calculations of all the limit states associated with the connection. The member sizes, bolts sizes, and number of bolts, weld length, weld size, member properties are all selected a priori. It is important to note that the selection was not based on any specific design or structural system, but instead selected for illustrations of the limit state calculations only. This is similar to what an instructor might do in class with an example problem or homework assignment. The analysis section provides the

- Definitions of the symbols used in the example,
- Properties of all the components associated with the connection,
- A sketch of the connection,
- Detail analysis of the capacity for each limit state,
- Summary of capacities for all limit states, and
- Capacity of the connection with illustration of the connection and maximum design force permitted

Next, we briefly discuss the sample calculation pages

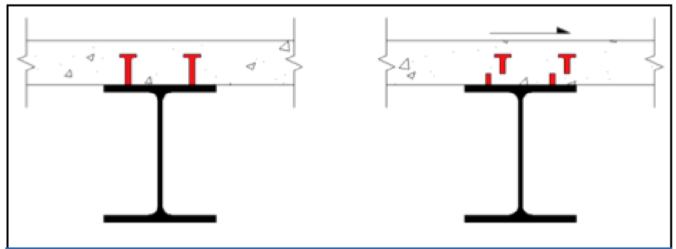

Figure 17. Stud Shear strength limit state

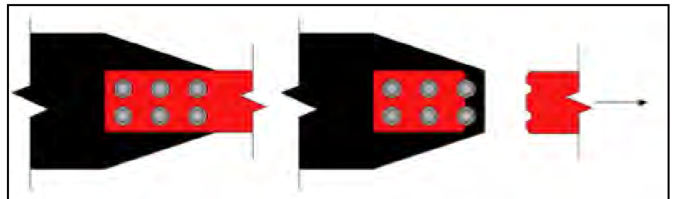

Figure 18. Tension rupture limit state

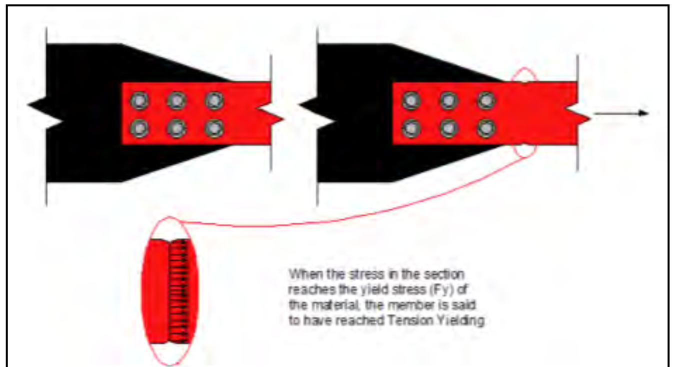

Figure 19. Tension yielding limit state

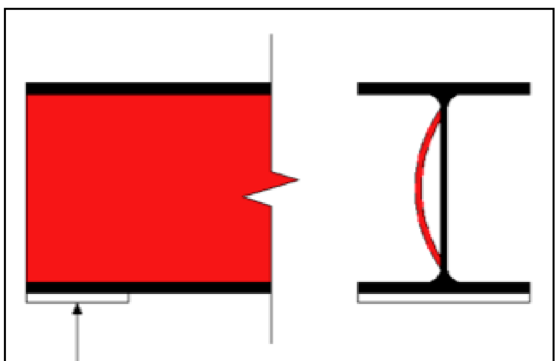

Figure 20. Web compression buckling limit state

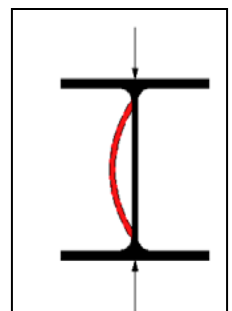

Figure 21. Web crippling limit state

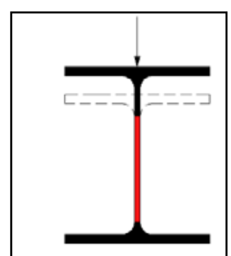

Figure 22. Web local yielding limit state

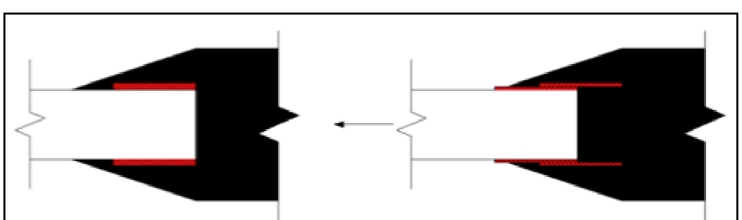

Figure 23. Weld strength limit state

of axial, shear, and moment connection types. The complete analyses of these connections are presented in the appendices.

Sample calculation 1 – connection 45 – axial connection

This connection represents a typical pinned-connection of a truss. In theory, this connection is assumed to be a frictionless pin. The gusset plate is a common element where all the axial (truss) members, associated with the connection, meet. A WT was used, in placed of gusset plate, in the physical sculpture. The function of this connection is classified as axial.

For this connection, there are five limit states to be considered: block shear rupture, bolt bearing, bolt shear, tension yielding, and tension rupture. The definition of each of the limit states can be found in the previous section.

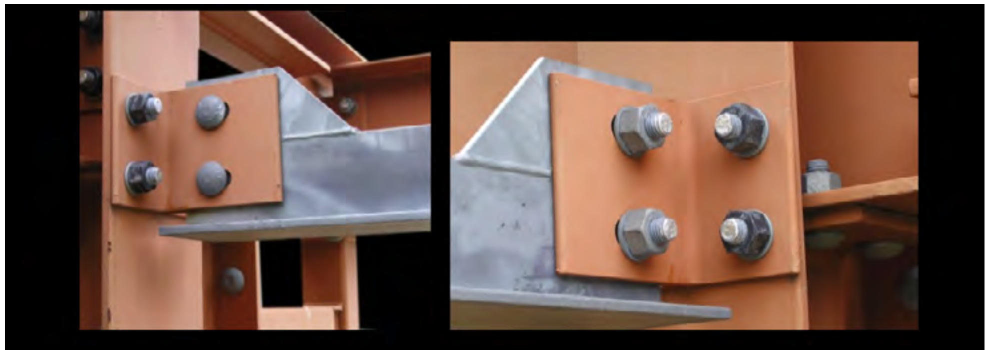

Figure for sample calculation 1 – connection 45

For the sample calculation, the bottom chord (WT 4×10.5 A992 steel) was attached to the flange of a column (W14×53 A992 steel) using ¾" diameter A325 n bolts. The connector used was 2L 3×4×1/4 A36 steel. Note that WT was used for the connector and stainless steel was used for the bottom chord in the physical sculpture. The controlling limit state was found to be bolt bearing with a maximum permissible capacity ϕP_n of 31 kips, (see appendix A for detailed analysis).

Figure for sample calculation 2 – connection 13

Sample calculation 2 – connection 13 – shear connection

This is a simple shear connection between two W-section beams out-of-plane-normal to each other. Two angles or bend plates were used as connectors. Note that the top flanges of the two W-section beams are flushed with each other. The requirement necessitates the coping of the beam normal to the paper. The loss of the top flange of the coped beam reduces the bolt bearing capacity and flexural strength of the beam.

There are six limit states to be considered: block shear rupture, bolt bearing, shear yielding, shear rupture, and flexural buckling. For the sample calculation, the support beam was assumed to be W16×57 A992 steel, the supported beam (coped beam) was assumed to be W14×53, A992 steel, the connectors were L3½×3½×3/8 A36 steel, and the bolts were of diameter A325. The controlling limit state was found to be block shear rupture with a maximum permissible capacity ϕV_n of 83 kips, (see appendix B for detailed analysis).

Figure for sample calculation 3 – connection 34

Sample calculation 3 – connection 34 – moment connection

This connection is the moment capacity portion of a shear-moment connection between a W-section beam and a W-section column at the column flange. Complete joint penetration (CPJ) grove weld at the flanges of the beam was used. The weld at the flanges must be capable of transferring flange force from the beam to the column or vice versa. The web plate and bolts, connecting the beam to the column, (shear portion of the connection) may help to resist moment, but their primary function is to transfer shear from the beam to the column.

The moment capacity for this connection is a function of the weld strength. Hence the only limit state for this connection is weld strength. For the sample calculations, the column was assumed to be W14×68 A992 steel, the beam was assumed to be W16×57 A992 steel, and the weld was E 70 electrode. The maximum permissible moment capacity ϕM_n was found to be 300 ft-kip, (see appendix C for detailed analysis).

Concluding Remarks

A fully animated three dimensional virtual steel sculpture was developed with support from the National Science Foundation. The purpose of the virtual steel sculpture is to provide a learning tool in steel connections for students and new engineering graduates. The virtual steel sculpture allows the user to zoom, pan, or rotate the sculpture to view different types of connection. Moreover, the user can isolate a connection and then learn more about it by viewing additional information. The connection information include the blue print used to fabricate the connection, a sample analysis of its limit states, field photo examples, and the finite element analysis of stress distribution within and around it. It is important to note here that the finite element models are intended only as visual tools to enhance the visualization of local deformations and stress build-up in the connections. The models are not meant for design purposes; the plasticity and the capacity bearing of connections were not considered in these models, as they were beyond the scope of this study.

In this paper, we present the sample analyses of limit states for three connection types: axial, shear, and moment. The detailed analyses of these connections (connection 45 for axial, connection 32 for shear, and connection 34 for moment) are embedded within the interactive virtual sculpture and are also presented in the appendices of this paper. The format used for these analyses is similar to those used in a typical steel design textbook or during a class lecture. The example connections were composed of members, bolts, and welds with sizes assumed by the authors for demonstration purpose only; they are not intended to represent an actual design situation. For connections associated with a specific structural system, additional design factors must be considered. For example, one must consider a member length that may affect the compression strength; eccentricity of a load that could add torsional effect to the connection and the members; or spacing of stiffeners that could influence the flexural strength of a member. These design factors could change the controlling limit state from bolts or welds to member strength.

Acknowledgment

The project was developed with the support from National Science Foundation under grant numbers: DUE-1140468 and DUE-1252371 and Program Director, Dr. Susan Finger. Student research assistants were Sara Ibarra, Lee Taylor and Thomas Gunderson. The opinions and findings expressed in this paper are the sole responsibility of the authors.

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[3] *Manual of Steel Construction* (2011) 14th Edition, American Institute of Steel Construction (AISC), Chicago, Il.

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APPENDIX A – SAMPLE CALCULATION 1

Connection #45

CONNECTION DESCRIPTION: Connection 45 represents a typical pinned-connection of a truss. In theory, this connection is assumed to be a frictionless pin. The gusset plate is a common element where all the axial (truss) members associated with the connection meet. Connection 45 shows a connection between a bottom chord member and the truss support, which in this case is a column. The connector is a WT section. Bolts were used at both legs of the angle. A gusset plate is added to the vertical leg of the bottom chord to allow sufficient depth for the two bolts. Careful observation of the enlarged connection image would reveal a small portion of a bolt hole (this is a slotted hole) at the angle leg connecting to the bottom chord of the truss. This slot is to allow for horizontal movement of the truss, which commonly idealized as a roller support in analysis, or for on-site assembly to accommodate slight imperfection on the alignment or member straightness. Similar connections: 46 and 47.

CONNECTION TYPE: Axial Connection

LIMIT STATES: Block Shear Rupture, Bolt Bearing, Bolt Shear, Tension Yielding, Tension Rupture

ANALYSIS: Each limit state must be analyzed in accordance with the AISC Steel Construction Manual. The limit state with the least allowable stress will control the section.

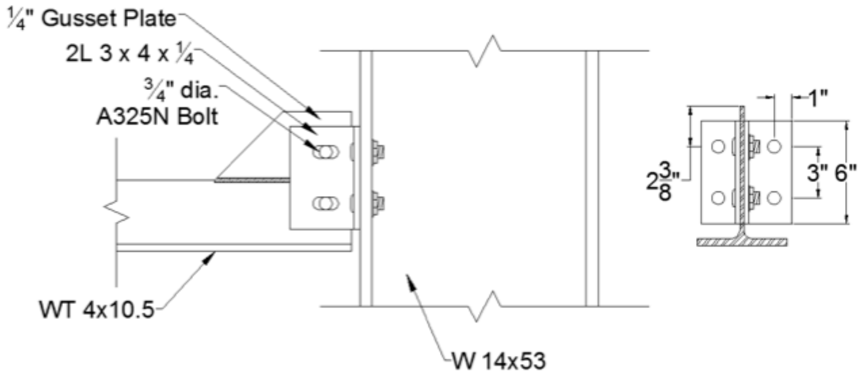
EXAMPLE:

variables

A_b:	cross-sectional area of bolt	(in^2)	A_e:	effective area	(in^2)
A_{gv}:	gross area in shear	(in^2)	A_n:	net area	(in^2)
A_{nt}:	net area in tension	(in^2)	A_{nv}:	net area in shear	(in^2)
d_b:	nominal bolt diameter	(in)	d_h:	hole diameter	(in)
F_{nv}:	nominal bolt shear strength	(ksi)	F_u:	ultimate stress	(ksi)
F_y:	yield stress (ksi)			clear distance in the direction of the force	(in)
	edge clear distance	(in)	n:	number of bolts	
s:	center to center spacing of bolts	(in)	t:	thickness	(in)
U:	shear lag factor		U_{bs}:	tension stress variation factor	

Properties

Column:	W14×53 (Properties found in Table 1–1), A992 Steel (Fy = 50 ksi and F_u = 65 ksi, from Table 2–4)
Connector:	2L 3×4×¼ (Properties found in Table 1–7), A36 Steel (F_y = 36 ksi and F_u = 58 ksi, from Table 2–4)
Plate:	¼" thick, A992 Steel (F_y = 50 ksi and F_u = 65 ksi, from Table 2–4)
Bottom Chord:	WT4×10.5 (Properties found in Table 1–8), A992 Steel (F_y = 50 ksi and F_u = 65 ksi, from Table 2–4)
Bolts:	¾" diameter A325N Steel (F_{nv} = 54 ksi and F_{nt} = 90 ksi, from Table J3–2).



Limit States

A WT4×10.5 bottom chord is supported by a W14×53 column using ¾" diameter A325N Bolts. To accommodate two bolts at the bottom chord, a ¼" thick gusset plate is welded to the web of the WT section. It is assumed that complete joint penetration (CJP) weld is used and the strength is not the controlling factor in this sample analysis. 2L 3×4×¼ are used in the connection to transfer axial force from the bottom chord to the column. Note that to keep the enlarged end section of the bottom chord the same as WT, A992 steel is used for the gusset plate. Determine the maximum axial force ΦP_n permissible at this connection.

In this connection, the connector has short slotted holes. Provisions for short slotted holes are given in AISC Specification J3.2.

Block Shear Rupture (AISC J4.3)
Block shear rupture is determined by the material strength, shear rupture or yielding and tension rupture path, and element thickness. In this connection, the WT4×10.5 connected controls

$$R_n = \min \begin{pmatrix} 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \\ 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \end{pmatrix}$$

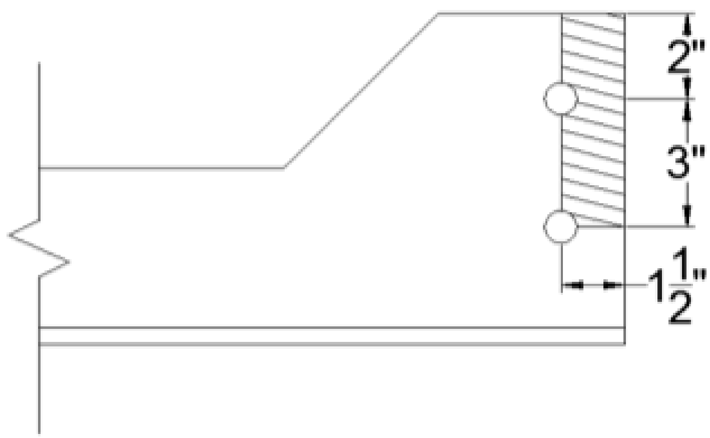
(Equation J4-5)

$F_y = 50$ ksi; $F_u = 65$ ksi; $t_w = 0.25"$

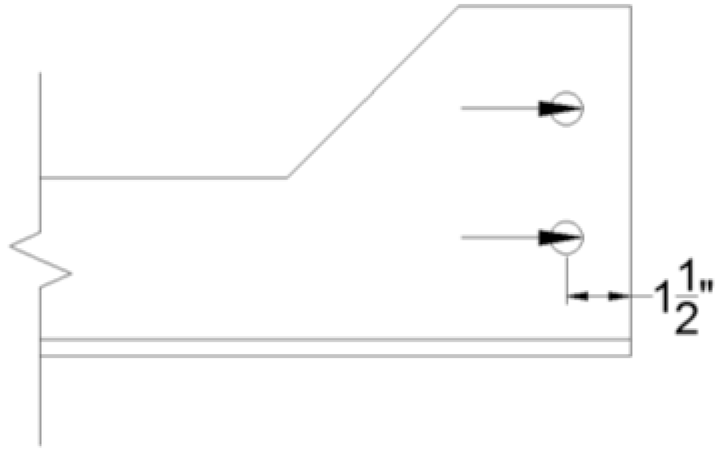
$d_h = d_b + 1/8 = 0.75" + 0.125"" = 0.875"$

$A_{gv} = 1.5"(0.25") = 0.375 in^2$

$A_{nv} = A_{gv} n_{dh} t" = 0.375 in^2 - (0.5 \text{ bolts})(0.875")(0.25") = 0.266 in^2$



$A_{nt} = (\ell\text{-}nd_h)t = [3'' + 2'' - 0.875''(1.5\text{ bolts})](0.25'') = 0.922\text{ in}^2$
Ubs = 1.0 (for uniform tension stress)



$$R_n = \min \begin{cases} 0.60(65\text{ ksi})(0.266\text{ in}^2) + 1.0(65\text{ ksi})(0.922\text{ in}^2) = 71.17\text{ kips} \\ 0.60(50\text{ ksi})(0.375\text{ in}^2) + 1.0(65\text{ ksi})(0.922\text{ in}^2) = 70.28\text{ kips} \end{cases}$$

The block shear strength of the WT4×10.5 connector is
$\Phi V_n = \Phi R_n = 0.75(70.28\text{ kips}) =$ **52.71 kips**

Bolt Bearing (AISC J3.10)
The bearing capacity is controlled by the element thickness and F_u of the element.
For this connection the WT bottom chord controls. Assume deformation at the bolt holes at service load is a design consideration.

$$R_n = \min \begin{cases} 1.2\ell_c t F_u \\ 2.4 d t F_u \end{cases} \quad \text{(Equation J3-6a)}$$

$d_h'' = d_b'' + 1''/8 = 0.75 + 0.125'' = 0.875$
$\ell_c = \ell_e - d_h/2 = 1.5'' - 0.875''/2 = 1.0625''$

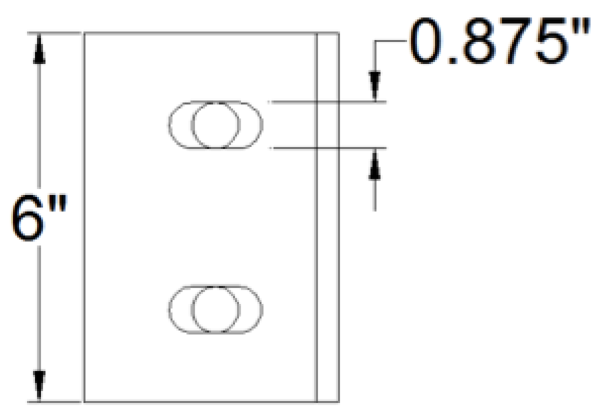
$$R_n = \min \begin{cases} 1.2(1.0625'')(0.25'')(65\text{ ksi}) = 20.72\text{ kips/bolt} \\ 2.4(0.75'')(0.25'')(65\text{ ksi}) = 29.25\text{ kips/bolt} \end{cases}$$

The bearing strength of the WT chord member is
$\Phi V_n = \Phi R_n = 0.75(2\text{ bolts})(20.72\text{ kips/bolt}) = 31.08\text{ kips}$

Bolt Shear Strength (AISC J3.6)
Bolts in this connection are in double shear. The bolt shear rupture of the connection is determined as follow:

$R_n = nF_{nv}A_b$ (Equation J3-1)

$F_{nv} = 54\text{ ksi}$
$A_b = \pi/4\, d_b^2 = \pi/4\,(0.75^2) = 0.442\text{in}^2$



The bolt shear rupture strength is
$\Phi V_n = \Phi R_n = 0.75(2\text{ bolts})(54\text{ ksi})(0.442\text{in}^2)(2\text{ shear planes}) =$ **71.60 kips**

Tension Yielding of Connecting Members (AISC J4.1)
The tension yield of the connecting elements are controlled by the element with the smallest product of A_g in tension and the yield strength of the material F_y. The double angles control as they use A36 steel vs A992 for the bottom chord and the gross cross section area of the bottom chord is 4.58 in² at the connection. The tensile area of the double angle section is defined as the Whitmore section

which is the smaller of the double angles' entire area or a projection of 30° from the beginning of the connection (first bolt or weld) to the end of the connection (last bolt or weld).

$R_n=F_y A_g$ (Equation J4-1)
$F_y=36$ ksi
$A_g=lt=6''(0.25'')(2 \text{ angles})=3.00 \text{ in}^2$

The tension yield of the WT connector is
$\Phi V_n=\Phi R_n=0.90(3.00 \text{ in}^2)(36 \text{ ksi})=$ **97.20 kips**

Tension Rupture of Connecting Elements (AISC J4.1)

The tension rupture of the connecting elements are controlled by the element with the smallest product of A_n in tension and the yield strength of the material F_u. For the same reason as tension yield, double angles control this limit state.

$R_n=F_u A_e$ (Equation J4-2)
$F_u=58$ ksi
$U=1.0$
$A_g=lt=6''(0.25'')(2 \text{ angles})=3.00 \text{ in}^2$
$A_n=A_e U=A_g-nd_h t=3.00 \text{ in}^2-(2 \text{ bolts})(0.875'')(0.25'')(2 \text{ angles})=2.125 \text{ in}^2$

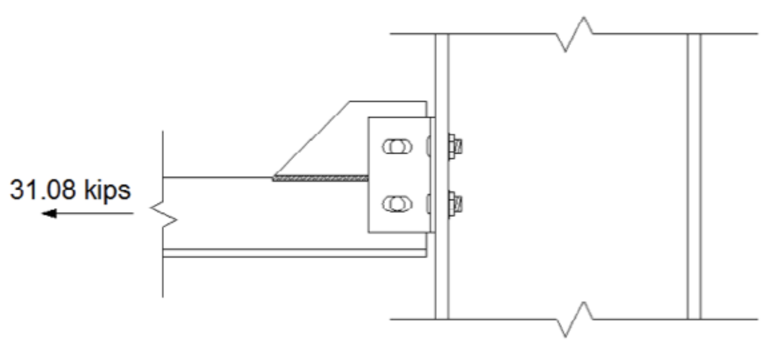
The tension rupture of the WT connector is
$\Phi V_n=\Phi R_n=0.75(2.125 \text{ in}^2)(58 \text{ ksi})=$ **92.44 kips**

Example Summary

Limit State	Max Allowable Force
Block Shear Rupture	52.71 kips
Bolt Bearing	**31.08 kips**
Bolt Shear	71.60 kips
Tension Yielding	97.20 kips
Tension Rupture	92.44 kips

The Example's Connection Capacity

The maximum ΦP_n permissible at this connection is controlled by bolt bearing of the bottom chord. It is assumed that the WT bottom chord and the W14×53 column were selected to meet P_u required.



APPENDIX B – SAMPLE CALCULATION 2

Connection #13

CONNECTION DESCRIPTION: Connection 13 is a simple shear connection between two W-section beams out-of-plane-normal of each other. Two angles (or bend plates) are used as connectors. The bolts through the two angles and the web of the beam shown normal to the screen (usually refer to as floor beam) have double shear planes while the bolts through the other leg of the angles have only single shear plane. Note that the top flanges of the two W-section beams are flushed with each other. This requirement necessitates the coping of the floor beam. The loss of the top flange of the floor beam reduces the bolt bearing capacity of the connection and the flexural strength of the coped beam. This connection is usually used when a floor beam is connected to a girder or transfer beam. The need for a flush surface between the two W-sections is for the floor deck. Similar connections: 7, 14, and 32.

CONNECTION TYPE: Shear Connection

LIMIT STATES: Block Shear Rupture, Bolt Bearing, Bolt Shear, Shear Yielding, Shear Rupture, Flexural Local Buckling

ANALYSIS: Each limit state must be analyzed in accordance with the AISC Steel Construction Manual. The limit state with the lowest force capacity will control the connection.

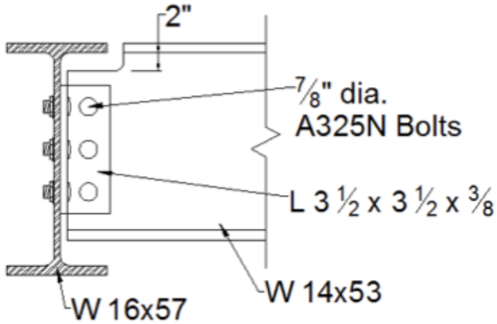
EXAMPLE:

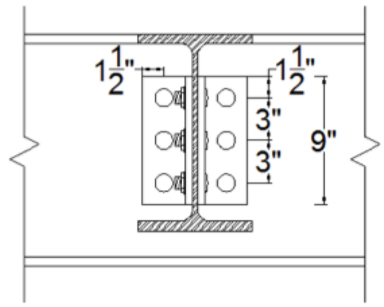
variables:

A:	area	(in^2)	A_b:	cross-sectional area of bolt	(in^2)
A_{gv}:	gross area in shear	(in^2)	A_{nt}:	net area in tension	(in^2)
A_{nv}:	net area in shear	(in^2)	c:	distance from neutral axis	(in)
d_b:	nominal bolt diameter	(in)	dh	hole diameter	(in)
E:	modulus of elasticity	(in^2)	e:	eccentricity (in)	
F_{nv}:	nominal bolt shear strength	(ksi)	Fu:	ultimate stress	(ksi)
F_y:	yield stress	(ksi)	h/t_w	Compactness Ratio	
I:	moment of inertia	(in^4)	ℓ_c:	clear distance in the direction of the force	(in)
ℓ_e:	edge clear distance	(in)	n:	number of bolts	
s:	center to center spacing of bolts	(in)	t:	thickness	(in)
U_{bs}:	tension stress variation factor				

Properties:

Support beam: W16×57 (Properties found in Table 1–1), A992 Steel (F_y = 50 ksi and F_u = 65 ksi, from Table 2–4)
Supported beam: W14×53 (Properties found in Table 1–1), A992 Steel (F_y = 50 ksi and F_u = 65 ksi, from Table 2–4)
Angle: L3½ × 3½ × 3/8, A36 Steel (F_y = 36 ksi and F_u = 58 ksi, from Table 2–4)
Bolts: 7/8" diameter A325N Steel (F_{nv} = 54 ksi and $F_n t$ = 90 ksi, from Table J3–2)





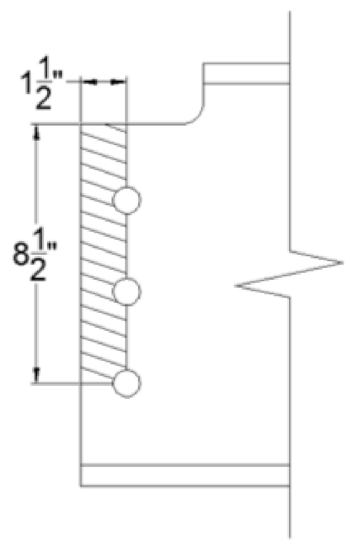
Limit States:

A W14×53 beam is connected to a W16×57 beam. This is done by using two angles that are L3½ × 3½ × 3/8. This connection is made by using three 7/8" diameter A325N bolts to bolt each leg to the web of a beam. The cope distance of the W14×53 is 4". Determine the maximum shear force ΦV_n permissible.

Block Shear Rupture: (AISC J4.3)

Block shear rupture is determined by the material strength, shear rupture or yielding and tension rupture path, and element thickness. In this connection, the coped W-section will control because the web thickness of W14×53, t_w = 0.370", is significantly lower than the angle thickness of 0.75" total. Even though the material strength of A992 is stronger than A36, it is insufficient to compensate the low t_w.

$$R_n = \min \begin{Bmatrix} (0.60F_u A_{nv} + U_{bs} F_u A_{nt} \\ 0.60F_y A_{gv} + U_{bs} F_u A_{nt}) \end{Bmatrix} \quad \text{(Equation J4-5)}$$



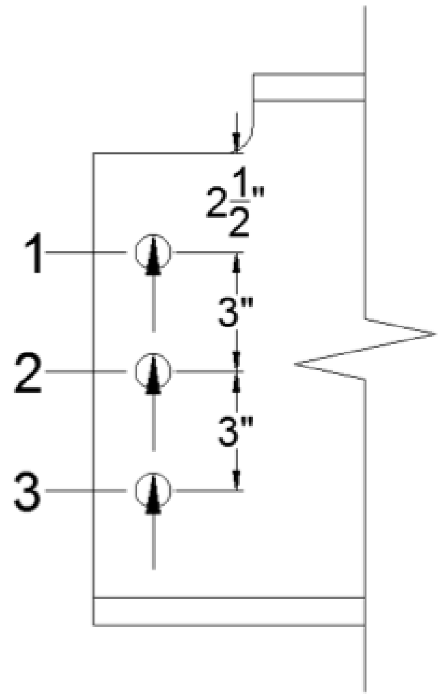
$d_h = d_b" + 1/8 = 0.875" + 0.125\backslash = 1.0"$
U_{bs} = 1.0 (for uniform tension stress)
F_y = 50 ksi; F_u = 65 ksi
$A_{gv} = (\ell_e + 2s)\, t_w = [2.5" + 2(3")]0.37" = 3.15\ in^2$
$A_{nv} = A_{gv} - nd_h t" = 3.15\ in^2 - (2.5"\ holes)\ (1.0")(0.370") = 2.23\ in^2$
$A_{nt} = (\ell - nd_h)t = [1.5" - (0.5\ holes)(1.0\ in^2)](0.370") = 0.370\ in^2$

$$R_n = \min \begin{Bmatrix} (0.60(65\ ksi)(2.23\ in^2) + 1.0(65\ ksi)(0.37\ in^2) = 111.02\ kips \\ 0.60(50\ ksi)(3.15\ in^2) + 1.0(65\ ksi)(0.37\ in^2) = 118.55\ kips) \end{Bmatrix}$$

The block shear strength of the coped W-section is
$\Phi V_n = \Phi R_n$ "=0.75(111.02 "kips")=**83.27 "kips**

Bolt Bearing (AISC J3.10)
The web of the supported beam (W14×53) controls bolt bearing strength for this connection because the web thickness ($t_w = 0.37''$) is thinner than the thickness of the two angles combined (t= 0.75"). The higher strength of A992 steel would not compensate for the extra thickness of the angles. Note that the support beam (W16×57) does not control. The reason is that each angle transfers only half the force to the support beam. Hence, the beam web only bears half of the force at each location. Assume deformation at the bolt holes at service load is a design consideration.

R_n= min $\begin{Bmatrix} (1.2\ell_c tF_u \\ 2.4dtF_u) \end{Bmatrix}$ (Equation J3-6a)



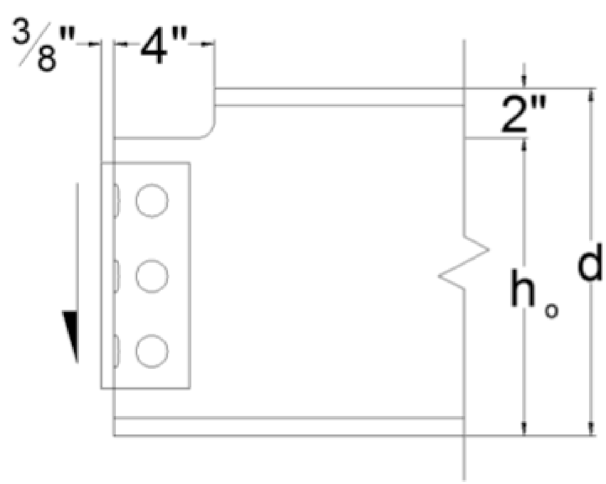
Since the edge distance and bolt spacing are different, R_n for each bolt needs to be analyzed separately.
$d_h = d_b + 1/8'' = 0.875'' + 0.125'' = 1.0''$
$t_w = 0.37''; F_u = 65$ ksi

Bolt 1
$\ell_c'' = \ell_e - d_h/2 = 2.5'' - 1/2\,(1'') = 2.0''$
R_n=min $\begin{Bmatrix} (1.2(2.0'')(0.37'')(65 \text{ ksi})=57.72\text{kips/bolt} \\ 2.4(0.875'')(0.37'')(65 \text{ ksi})=50.51 \text{ kips/bolt})\} \end{Bmatrix}$

Bolts 2 and 3
$\ell_c = s - d_h'' = 3'' - (1'') = 2.0''$
R_n=min (1.2(2.0")(0.37")(65 ksi)=57.72kips/bolt
2.4(0.875")(0.37")(65 ksi)=50.51 kips/bolt)}

All bolts have the same bearing strength of 50.51 kips/bolt. Hence the bearing strength of the connection is
$\Phi V_n = \Phi R_n$"=0.75(3 "bolts)(50.51 kips/bolt)"=**115.90 kips**

Bolt Shear (AISC J3.6)
Bolts are in double shear so a multiplier of two is needed.



$R_n = nF_{nv}A_b$ (Equation J3-1)
$F_{nv} = 54$ ksi
$A_b'' = \pi/4\, d^2/b = \pi/4\,(0.875^2) = 0.601$ in^2

The bolt shear rupture strength is
$\Phi V_n'' = \Phi R_n$ =0.75(3 bolts)(54 ksi)(0.601 in^2)(2 shear planes)
=**146.04 kips**

Flexural Local Buckling: (AISC J4.5)

When a W-section is coped, flexural local buckling of the section is reduced. The beam web was supported by the flanges, when the section is coped, one of the flanges is removed and the web behaves like a cantilever element. From the flexure's perspective, ΦM_n of a coped beam is severely compromised when a beam has all or part of its flange removed. This is due to the fact that flanges are responsible for the majority of bending moment. The AISC manual considers the primary failure mechanism of a coped beam being the local buckling. Because of this, flexural strength of a coped section is determined using serviceability limit state instead of strength. Elastic modulus is used here as opposed to the plastic modulus used in typical flexural design.

$M_n=F_{cr} S_{net}$ (Equation 9-6)

$S_{net}= 14.20\ in^3$ (From Table "9-2")
$F_{cr}=26,210(t_w/h_o)^2 fk\le F_y$ (Equation 9-7)
$d = 13.9$ in.
$d_c = 2$ in.
$c = 4$ in.
$h_o=d-d_c=13.9''-2''=11.9''$
$f=^{2c}/d=^{2(4^2}/_{13.9}=0.576$ (Equation "9-8")
$k=2.2(^{h_o}/_c)^{1.65}=2.2(^{11.9''}/_{4''})^{1.65}=13.29$ (Equation "9-10")
$F_{cr}=26,210(^{0.370''}/_{11.9''})^2 (0.576)(13.29)=193.97\ ksi\ge 50\ ksi$

The moment strength of the coped W14×53 becomes
$\Phi M_n=0.90(50\ ksi)(14.20\ in^3)(1'/12'') =53.25$ "kip-ft"

Failure in flexure is caused by product of the moment arm between shear force acting at the interface of the angle and the beam perpendicular to the coped face ($e = c + t_{angle} = 4'' + 0.375''$).

$\Phi V_n=\Phi R_n= ^{\Phi Mn}/e =(53.25\ \text{kip-ft})/4.375\ (12''/1')$ =**146.06 kips**

Shear Rupture of Connecting Elements (AISC J4-2)

Shear rupture of connecting element is determined by the ultimate strength of the material and the thickness of the element.

$R_n=0.60F_u A_{nv}$ (Equation J4-4)
$d_h''=d_b +1''/8=0.875''+0.125''=1.0''$

Angle

$F_u = 58$ ksi
$A_{gv}''=9''(0.375'')=3.375\ in^2$/angle"
$A_{nv}''=A_{gv}-nd_h t''=3.375\ in^2-(3\ bolts)(1'')(0.375'')=2.25$ "in" ^2"/angle"

The shear rupture strength of the double angles is
$\Phi V_n = \Phi R_n =0.75(0.60)(58\ ksi)(2.25\ in^2/angle)(2\ angles)$ =**117.45 kips ◄— controls shear rupture**

W14×53
$F_u = 65$ ksi
$A_{gv}=A_g-t_f b_f-(d_c-t_f) t_w=15.6\ in^2- 0.660''(8.06'')-(2''-0.660'')(0.370'')= 9.78\ in^2$
$A_{nv}=A_{gv}-nd_h t_w=9.78\ in^2-3(1'')(0.370)=8.67\ in^2$

The shear rupture strength of the coped W14×53 is
$\Phi V_n = \Phi R_n = 0.75(0.60)(65\text{"ksi})(8.67\ in^2) =$ **253.60 kips**

Shear Yielding of Connecting Elements (AISC J4-2)
From shear rupture, it is seen that the angles would control shear yield

$R_n = 0.60 F_y A_{gv}$ (Equation J4-3)
$F_y = 36$ ksi
$A_{gv} = 9''(0.375'') = 3.375\ in^2$/angle

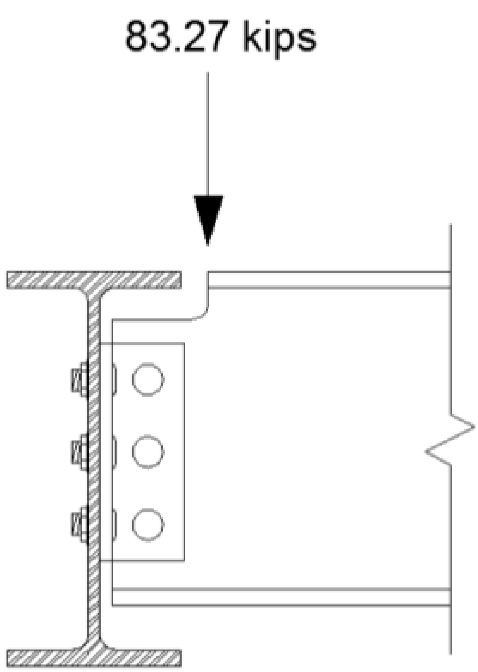
The shear yielding of the double angles is
$\Phi V_n = \Phi R_n = 1.0(0.60)(36\ ksi)(3.375\ in^2\text{"/angle})(2\ angles) =$ **145.80 kips**

Example Summary:

Limit State	Max Factored Load
Block Shear Rupture	83.27 kips
Bolt Bearing	115.90 kips
Bolt Shear	146.04 kips
Flexural Local Buckling	146.06 kips
Shear Rupture	117.45 kips
Shear Yielding	145.80 kips

The Example's Connection Capacity:

The shear capacity ΦV_n of this connection is controlled by block shear rupture. Note that there are other limit states one may need to consider during design. These are web compression buckling, web local crippling, and web local yielding associated with the beam. It is assumed that the beam was selected to meet V_u required.



APPENDIX C – SAMPLE CALCULATION 3

Connection #34

CONNECTION DESCRIPTION: Connection 34 is the moment capacity portion of the shear-moment connection between a W-section beam and a W-section column at the column flange. Complete joint penetration (CJP) grove weld at the flanges of the beam are used. Note that there is a metal strip at the under-side of the beam flange to hold the electrode in place during the welding process. This strip of metal was shop-welded and can also serve as a seat for the beam. There is also a small cut-out on the beam web just below the top flange. This opening allows the plate strip to go from one end of the flange to the other. It also allows the stick weld to go through the web. See Connection 33 for additional views of this detail.

The weld at the flanges must be capable of transferring flange force from the beam to the column or vice versa. The web plate and bolts (see Connection 33) may help to resist moment, but their primary function is to transfer shear from the beam to the column. Companion connection: 33. Similar moment connections: 15, 26 and 43. Similar shear-moment connection groups: 25 and 26, 42 and 43.

CONNECTION TYPE: Moment Connection

LIMIT STATES: Weld Strength

ANALYSIS: Each limit state must be analyzed in accordance with the AISC Steel Construction Manual. The limit state with the least allowable stress will control the section.

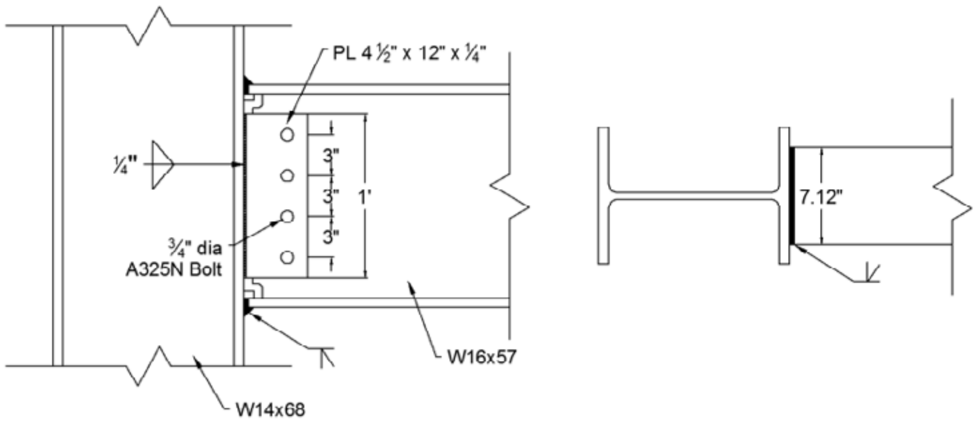
EXAMPLE:

variables

A_{we}:	effective area of the weld	(in^2)	B_f:	flange width of W section	(in)
d:	depth of W-section	(in)	F_{EXX}:	filler metal classification (electrode) strength	(ksi)
Fnw:	nominal shearing stress of weld	(ksi)	F_u:	ultimate stress	(ksi)
Fy:	yield stress	(ksi)	L:	length of weld	(in)
t:	thickness	(in)	t_f:	thickness of flange	(in)
w:	weld size	(in)			

Properties

Beam: W16×57 (Properties found in Table 1–1), A992 Steel ($F_y = 50$ ksi and $F_u = 65$ ksi, from Table 2–4)
Column: W14×68 (Properties found in Table 1–1), A992 Steel ($F_y = 50$ ksi and $F_u = 65$ ksi, from Table 2–4)
Plate: 4.5″×12″×¼″, A36 Steel ($F_y = 36$ ksi and $F_u = 58$ ksi, from Table 2–4)
Bolts: ¾″ diameter A325N Steel ($F_{nv} = 54$ ksi and $F_{nt} = 90$ ksi, from Table J3–2)
Welds: Electrode with $F_{EXX} = 65$ ksi/0.60 = 108 ksi for tension or E70 with $F_{EXX} = 70$ ksi for shear



Limit States

A 4.5″×12″×¼″ gusset plate is welded to the flange of a W14×68 column using ¼″-E70 fillet weld on both sides of the plate. A W16×57 beam is connected to the column by four-¾″ diameter A325N bolts via the gusset plate (yields simple connection). The bolts are used to transfer shear from the beam to the column and will not be considered here. The complete joint penetration (CJP) groove weld is used at both beam flanges for moment resistance. Determine the maximum moment capacity ΦM_n permissible. Connection 33 shows the calculations for the shear capacity of this connection.

Weld Strength (AISC J2.2)

For CJP groove weld, the strength of the weld joint is controlled by base metal as long the as the strength of the weld metal (filler) is higher than the strength of the base metal. One can always use weld metal such that this condition occurs.

W16×57: $F_y = 50$ ksi; $F_u = 65$ ksi; $t_f = 0.715''$; $b_f = 7.12''$; $d = 16.4''$

Tension Yield of Beam Flange

$R_n = A_g F_y$ "Equation (J4-1)"

$A_g = b_f t_f = 7.12''(0.715'') = 5.091 \text{ in}^2$

The tensile yield strength of one beam flange is

$\Phi T_n = \Phi R_n = 0.9(50 \text{ ksi})(5.091 \text{ in}^2) =$ **229.09 kips** ◄— **control weld strength**

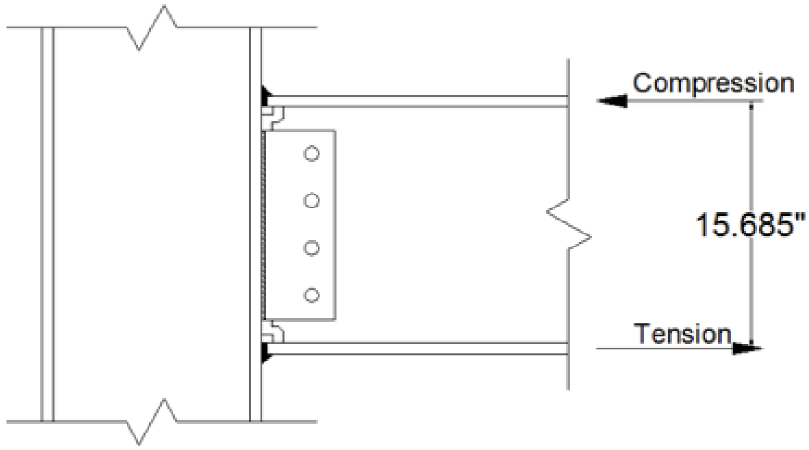
Tension Rupture of Beam Flange

$R_n = A_e F_u$ "Equation (J4-2)"

For welded element, $A_n = A_g$ and $U = 1.0$. Therefore, $A_e = A_g$. The tension rupture strength of one beam flange is
$\Phi T_n = \Phi R_n = 0.75(65\ ksi)(5.091\ in^2) = 248.9$"kips"

To determine the moment capacity ΦM_n, the forces from the beam flanges is multiplied by the distance between the center of the beam flanges thickness which, in this case, is the depth of the W16×57 minus the flange thickness.

$\Phi M_n = \Phi T_n\ (d - t_f)$
$= 229.09$ kips (16.4"-0.715")
$= 3593.28$"kips-=299.4 **kip-ft**



Example Summary

Limit State	Max Allowable Force
Moment from base metal	299.4 kip-ft

The Example's Connection Capacity

The moment capacity ΦM_n of this connection is controlled by the based metal of the beam flange. Note that there are other limit states one must consider during design. These are flexure rupture, flexure yielding of elements (beam and column) associated with the connection, web compression buckling, web local crippling, and web local yielding associated with the beam. The moment capacity shown here is from the weld strength alone at the flanges of the beam. To see the shear capacity, please refer to Connection #33. Also, it is important to keep in mind that in connection design it is assumed that all members (in this case the beam and column) were selected to meet the M_u required.