Simple Shear Connection Limit States

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Understanding limit states is essential to understanding steel connection design. Here's a look at common limit states for simple shear connections.

GOOD CONNECTION DESIGN IS ALL ABOUT FOLLOWING LOAD THROUGH ALL THE ELEMENTS IN ITS PATH. Load must be able to transfer from beam web to bolts to angles to more bolts and through to the supporting web. Each of these connection elements has their own set of discrete limit states. A quick review of these limit states is a good check to make sure you are covering all your bases when designing. The following is a list of references and also some examples of the most common limit states to be checked on simple shear connections.

1 Bolt Shear

Specification Section J3.6

$R_n = F_n A_b$ (J3-1) ϕ =0.75 (LRFD)
Ω = 2.00 (ASD)

Additional References

Manual Table 7-1; *Specification* Table J3.2

Bolt shear is based upon the limit state of shear rupture of the bolt. Equation J3-1 in *Specification* Section J3.6 is general and applies to both tension and shear in bolts. The nominal strengths for use in Equation J3-1 are obtained from Table J3.2. The designer must know what bolt grade is to be used and whether he is including (N) or excluding (X) threads from the shear plane. It is conservative to design all cases assuming threads are included in the shear plane. Table 7-15 in the *Manual* gives dimensions of high-strength fasteners. Threads can be present in the "grip" area between the nut or washer and the bolt head, but cannot be fully engaged at the interface of the two plies that are being joined by the connection. A portion of one thread may be present at the shear plane and still be considered excluded, as the strength of the full bolt diameter is present at this location. Eccentricity considerations for bolted connections have been noted in the included table.

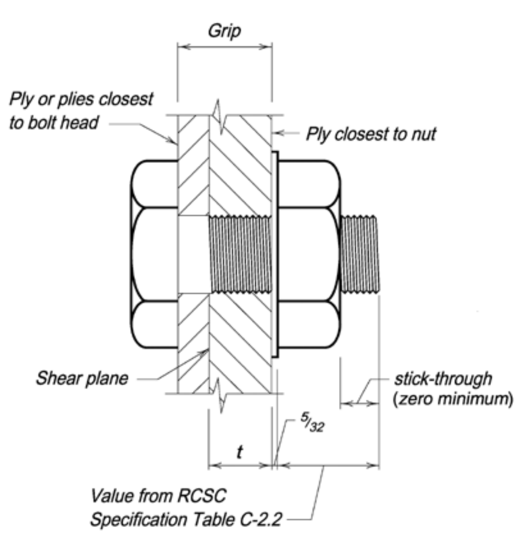

Figure 1. Bolt threads are excluded from the shear plane in this illustration.

2 Bolt Bearing

Specification Section J3.10

$R_n = 1.2 L_c t F_u \le 2.4 d t F_u$ (J3-6a)

or

$R_n = 1.5 L_c t F_u \le 3.0 d t F_u$ (J3-6b) ϕ = 0.75 (LRFD)
Ω = 2.00 (ASD)

Additional References

Bearing strength based on bolt spacing (Gr. 50 and 36): *Manual* Table 7-5

Bearing strength based on edge distance (Gr. 50 and 36): *Manual* Table 7-6

Bolts bear both on the structural member and any connection material (angles, plates, etc.). Hence, the equations in Section J3.10 must be checked for both of these situations. The nominal strength equations evaluate bearing strength based on both edge distance and the deformation of a hole edge. The lesser of these values will control your design. The edge distance value in the equation can be either the clear distance between adjacent bolt holes or between a bolt hole and the material edge.

Section J3.10 (a) gives two equations for the nominal strength of bolts bearing against the connection material. Equation J3-6a uses a factor of

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2.4 and applies when "deformation at the bolt hole at service load is a design consideration," and equation J3-6b uses a factor of 3.0 and applies when "deformation at the bolt hole is not a consideration." How does the designer know if deformation at bolt holes is of concern? The answer to this question is linked to the development of the equations themselves. The $3.0dtF_u$ expression is the original equation that was developed when rupture limit states and deformation were first investigated. While this limit state is correct, it was found that extensive deformation will occur before it is reached. The 2.4 factor came about as a means to limit deformation when necessary. The Commentary to the *Specification* does note that hole elongation of ¼ in. or more will likely be observed when the applied force is greater than $2.4dtF_u$. It is up to the design engineer to evaluate whether this amount of hole deformation would be detrimental to the structure or connection designs.

Tables 7-5 and 7-6 in the *Manual* ("Available Bearing Strength at Bolt Holes Based on Bolt Spacing and Based on Edge Distance") are based upon equation J3-6a.

3 Shear Yield

Specification Section J4.2

$R_n = 0.60F_yA_g$ (J4-3) $\phi = 1.0$ (LRFD) $\Omega = 1.50$ (ASD)

This limit state is fairly straightforward. On a given shear plane, the shear yield strength of the gross section of the material must be greater than the applied load. This limit state applies to both bolted and welded connections. However, it is worth discussing the resistance factors and safety factors for LRFD and ASD as they apply to this limit state in the 2005 specification. For LRFD the resistance factor, ϕ, is 1.0. Previous editions of LRFD used a resistance factor of 0.9. This is one area of the 2005 specification where LRFD has been altered to conform to prior editions of ASD. One of the fundamental relationships in the 2005 specification between ASD and LRFD is that $\phi = 1.5/\Omega$ (Ω is the safety factor for ASD design). Previous editions of the ASD specification were written so that the safety factor for this limit state was 1.5. When LRFD was written, the resistance factor of 0.9 caused the equivalent safety factor of this limit state to increase to 1.67. The Commentary notes that this increase of about 10% in LRFD values was over-conservative and is supported by considerable historic evidence of the satisfactory performance of traditional ASD-designed connections.

It should also be noted that the area A_g is measured on the critical shear plane of the member or connecting element. It is not necessarily the cross-sectional area, A, of the member as located in the section properties tables in part 1 of the *Manual*. On a wide-flange section the area A_g, as it applies to shear yield, is the web area only and not the entire cross-section.

Shear yield should also be evaluated on the main supported member, particularly if its top and/or bottom flanges are coped in the connection region.

4 Shear Rupture

Specification Section J4.2

$R_n = 0.6F_uA_{nv}$ (J4-4) $\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

Shear rupture occurs on the net section, as opposed to shear yield, which occurs on the gross section. Consider the typical stress-strain curve for steel. The shear yielding limit state occurs when the material stress advances past the elastic region. Advance further along the curve, and there is a point after strain hardening where the material will rupture. This is the point where shear rupture occurs. On the gross section, the limit state of shear yield will always be reached before the limit state of shear rupture. However, connections tend to have features (such as bolt holes) that constrain yielding and cause localized stress concentrations. Because of this, rupture may occur on the net section before gross yielding can occur away from the net section.

Hang-ups in applying equation J4-4 usually come into play when calculating the net area of the cross-section, A_{nv}. The proper cross section to use in calculating this area is one cut through the element in the direction of the applied shear force. When subtracting area due to bolt holes, an extra 1⁄16 in. is added to the hole size dimension per *Specification* Section B3.13(b). This is in addition to the bolt hole being larger than the bolt diameter. For standard holes, this results in the area subtracted for bolt holes being ⅛ in. larger than the bolt diameter. See RCSC specification Table 3.1 for bolt hole sizes. This extra area is taken into account in Table 9-1, "Reduction in Area for Holes." These hole reductions have also been applied in Tables in Part 10 of the *Manual*.

5 Block Shear Rupture

Specification Section J4.3

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \quad \text{(J4-5)}$$

$\phi = 0.75$ (LRFD)
$\Omega = 2.00$(ASD)

Additional References
Manual Table 9-3

Block shear is the tearing out of a block of material at a connection as shown in Figure 2. Numerically, it is the sum of shear yield or shear rupture on a failure path parallel to the load and tension rupture perpendicular to the load. It most often applies on coped beam sections, gusset plates, and angle legs. It also is applicable to the perimeter of welded connections, such as an angle welded to a gusset plate. Calculations have been simplified in the 2005 specification.

The specification can be read as:

R_n = Shear Rupture + Tension Rupture
≤ Shear Yield + Tension Rupture

U_{bs} in equation J4-5 is either 1 or 0.5.

Cases where 0.5 is applicable are illustrated in the Commentary. *Manual* Tables 9-3a, b, and c list reduced nominal load capacities for tension rupture, shear yield, and shear rupture components of the equation respectively. They are shown for ASD and LRFD and grade 36 or 50 steel. It is easier than ever to evaluate block shear!

6 Fillet Welds in Shear

Specification Section J2.4

Weld Metal: $R_n = F_wA_w$ (J2-3)

ϕ=0.75(LRFD) Ω=2.00(ASD)

Additional References:
Weld strength Table J2.5
Minimum fillet weld sizes Table J 2.4

Weld strength is determined using the strength level of the electrode and the length, orientation, and effective throat of the weld. Electrodes with F_{exx} = 70ksi are the most common. Eccentrically loaded welds can be analyzed using the Instantaneous Center of Rotation Method or the Elastic Method. See Manual Part 8 for how to apply these methods.

Welds are only permitted to share load with bolts in shear connections when the bolt holes are standard or short-slotted

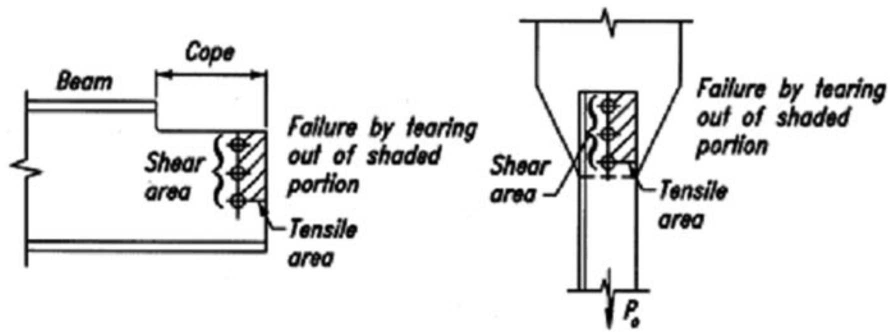

Figure 2. Typical block shear failure paths from AISC *Specification* Commentary.

LIMIT STATES FOR SEATED CONNECTIONS

	Unstiffened Seated Connection (Welded)	Unstiffened Seated Connection (Bolted)	Stiffened Seat (Welded)	Stiffened Seat (Bolted)
Table in *Manual*	10-6	10-5	10-8	10-7
BOLTS				
shear rupture (slip for SC) eccentricity not considered unless noted		X		X
CONNECTION MATERIAL (ANGLES OR PLATES)				
bolt bearing		X		X
shear yielding	X, 8	X, 8	X, 9	
shear rupture			X, 10	
flexural yielding	X, 8	X, 8		
end bearing			X	X
WELDS				
shear strength	X, 4		X, 4	
BEAM WEB				
web local yield	X	X	X	X
web local cripling	X	X	X	X
SUPPORTING ELEMENT				
bolt bearing		X		X
shear rupture at weld, 5	X,4		X	
punching shear			X	

Notes for tables located on next page.

and the slots are transverse to the direction of the load. The strength of the bolts in the connection is then limited to 50% of the available bearing strength of the bolted connection. See *Specification* Section J1.8 for more information.

Minimum sizes of fillet welds and partial-joint penetration welds are given in Tables J2.3 and 2.4 in the Specification and are based on the thickness of the thinner part joined. This is a change from previous editions of the *Specification*, in which minimum weld sizes were based off the thicker part joined.

7 Base Metal at Welds

Specification Section J2.4

Base Metal: $R_n = F_{BM}A_{BM}$ (J2-4)

$\phi = 0.75$ (LRFD)

$\Omega = 2.00$ (ASD)

Additional References

Weld strength, Table J2.5

Minimum fillet weld sizes, Table J2.4

The nominal strength of a welded connection is the lower value between the strength of the base metal at the weld and the weld itself. The base metal must be checked for the limit states of shear yield and/or shear rupture. Table J2.4 gives minimum thicknesses for base metal at welds. This assures that the shear rupture strength of the base metal will match the shear rupture strength of the weld.

In Conclusion

There are limit states outside the scope of this article, such as coped beam limits states, prying action, and local buckling. These are addressed in detail in Parts 7 (Bolts), 8 (Welds), and 9 (Connection Elements) in the *Manual*.

As a summary, I have included two tables detailing limit states to be checked for various connection configurations. These tables follow the connections outlined in Part 10 of the *Manual*. I hope you find this helpful in your design!

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LIMIT STATES FOR CONVENTIONAL SHEAR CONNECTIONS

	Double Angle (All-Bolted)	Single Angle (All-Bolted)	Double Angle (Bolted-Welded) welded to beam	Double Angle (Bolted-Welded) welded to support	Single Angle (Bolted-Welded) welded to support	Double Angle (All-Welded)	Shear End Plate (Welded-Bolted)	Conventional Single Plate
Table in *Manual*	10-1	10-10	10-2 10-1	10-2 10-1	10-11 10-10	10-3	10-4	10-9
BOLTS								
shear rupture (slip for SC) eccentricity not considered unless noted	X	X, 11	X	X	X, 11		X	X, 13
CONNECTION MATERIAL (ANGLES OR PLATES)								
bolt bearing	X	X	X	X	X		X	X
shear yielding	X	X	X, 6	X, 6	X, 6	X, 6	X	X
shear rupture	X	X	X, 6	X, 6	X, 6	X, 6	X	X
block shear rupture	X	X	X	X	X		X	X
flexural yielding		X, 11			X, 11			
flexural rupture		X, 11						
WELDS								
shear strength			X, 3	X, 4	X, 3	X, 3	X	X, 7
BEAM WEB								
bolt bearing	X	X		X	X			X
block yield rupture	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2		X, 1, 2
shear yielding	X, 2	X, 2	X, 2	X, 2	X, 2	X, 2		X, 2
shear rupture	X, 2	X, 2		X, 2	X, 2			X, 2
flexural yielding	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2, 12	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2
local buckling	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2	X, 1, 2
shear rupture at weld, 5			X			X	X	
SUPPORTING ELEMENT								
bolt bearing	X	X	X				X	
shear rupture at weld, 5				X	X	X		X

NOTES FOR BOTH TABLES

1. Required with top cope only.
2. Required with top and bottom cope.
3. Instantaneous center of rotation method to account for eccentricity.
4. Elastic method to account for eccentricity.
5. See "Connecting Element Rupture Strength at Welds" in Part 9 of the *Steel Construction Manual* (usually indicated by a minimum thickness required in the tables).
6. Minimum thickness of angle is required to handle these requirements, based on weld thickness. See *Manual* Part 10.
7. Minimum weld size, $^{5}/_{8}t_p$, required for weld to match plate strength.
8. On outstanding leg of angle.
9. Limit state addressed by having a minimum thickness of the stiffener equal to the beam web thickness multiplied by the ratio of the F_y of the beam material to the F_y of the stiffener material.
10. Limit state addressed by having a minimum thickness of the stiffener equal to $2w$ for stiffener material with F_y = 36 ksi or $1.5w$ for stiffener material with F_y = 50 ksi.
11. Eccentricity of the load to the bolt group is always considered in the angle leg attached to the support. Eccentricity should be considered in the case of a double vertical row of bolts through the web of the supported beam or if the eccentricity exceeds 3 in.
12. Required with bottom cope only.
13. The requirement for eccentricity is based on the number of bolts in the row.