

## Steel Design

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### Notation:

$a$	= name for width dimension	$D$	= shorthand for dead load
$A$	= name for area	$DL$	= shorthand for dead load
$A_b$	= area of a bolt	$e$	= eccentricity
$A_e$	= effective net area found from the product of the net area $A_n$ by the shear lag factor $U$	$E$	= shorthand for earthquake load = modulus of elasticity
$A_g$	= gross area, equal to the total area ignoring any holes	$f_c$	= axial compressive stress
$A_{gv}$	= gross area subjected to shear for block shear rupture	$f_b$	= bending stress
$A_n$	= net area, equal to the gross area subtracting any holes, as is $A_{net}$	$f_p$	= bearing stress
$A_{nt}$	= net area subjected to tension for block shear rupture	$f_v$	= shear stress
$A_{nv}$	= net area subjected to shear for block shear rupture	$f_{v-max}$	= maximum shear stress
$A_w$	= area of the web of a wide flange section	$f_y$	= yield stress
$AISC$	= American Institute of Steel Construction	$F$	= shorthand for fluid load
$ASD$	= allowable stress design	$F_a$	= allowable axial (compressive) stress
$b$	= name for a (base) width = total width of material at a horizontal section = name for height dimension	$F_b$	= allowable bending stress
$b_f$	= width of the flange of a steel beam cross section	$F_c$	= critical unfactored compressive stress for buckling in LRFD
$B_1$	= factor for determining $M_u$ for combined bending and compression	$F_{cr}$	= flexural buckling stress
$c$	= largest distance from the neutral axis to the top or bottom edge of a beam	$F_e$	= elastic critical buckling stress
$c_1$	= coefficient for shear stress for a rectangular bar in torsion	$F_{EXX}$	= yield strength of weld material
$C_b$	= modification factor for moment in ASD & LRFD steel beam design	$F_n$	= nominal strength in LRFD = nominal tension or shear strength of a bolt
$C_c$	= column slenderness classification constant for steel column design	$F_p$	= allowable bearing stress
$C_m$	= modification factor accounting for combined stress in steel design	$F_t$	= allowable tensile stress
$C_v$	= web shear coefficient	$F_u$	= ultimate stress prior to failure
$d$	= calculus symbol for differentiation = depth of a wide flange section = nominal bolt diameter	$F_v$	= allowable shear stress
$d_b$	= nominal bolt diameter	$F_y$	= yield strength
		$F_{yw}$	= yield strength of web material
		$F.S.$	= factor of safety
		$g$	= gage spacing of staggered bolt holes
		$h$	= name for a height
		$h_c$	= height of the web of a wide flange steel section
		$H$	= shorthand for lateral pressure load
		$I$	= moment of inertia with respect to neutral axis bending
		$I_{trial}$	= moment of inertia of trial section
		$I_{req'd}$	= moment of inertia required at limiting deflection
		$I_y$	= moment of inertia about the y axis
		$J$	= polar moment of inertia

$k$	= distance from outer face of W flange to the web toe of fillet = shape factor for plastic design of steel beams	$N$	= bearing length on a wide flange steel section = bearing type connection with threads included in shear plane
$K$	= effective length factor for columns, as is $k$	$p$	= bolt hole spacing (pitch)
$l$	= name for length	$P$	= name for load or axial force vector
$L$	= name for length or span length = shorthand for live load	$P_a$	= required axial force (ASD)
$L_b$	= unbraced length of a steel beam	$P_c$	= available axial strength
$L_c$	= clear distance between the edge of a hole and edge of next hole or edge of the connected steel plate in the direction of the load	$P_{e1}$	= Euler buckling strength
$L_e$	= effective length that can buckle for column design, as is $\ell_e$	$P_n$	= nominal column load capacity in steel design
$L_r$	= shorthand for live roof load = maximum unbraced length of a steel beam in LRFD design for inelastic lateral-torsional buckling	$P_r$	= required axial force
$L_p$	= maximum unbraced length of a steel beam in LRFD design for full plastic flexural strength	$P_u$	= factored column load calculated from load factors in LRFD steel design
$L'$	= length of an angle in a connector with staggered holes	$Q$	= first moment area about a neutral axis = generic axial load quantity for LRFD design
$LL$	= shorthand for live load	$r$	= radius of gyration
$LRFD$	= load and resistance factor design	$r_y$	= radius of gyration with respect to a y-axis
$M$	= internal bending moment	$R$	= generic load quantity (force, shear, moment, etc.) for LRFD design = shorthand for rain or ice load = radius of curvature of a deformed beam
$M_a$	= required bending moment (ASD)	$R_a$	= required strength (ASD)
$M_n$	= nominal flexure strength with the full section at the yield stress for LRFD beam design	$R_n$	= nominal value (capacity) to be multiplied by $\phi$ in LRFD and divided by the safety factor $\Omega$ in ASD
$M_{max}$	= maximum internal bending moment	$R_u$	= factored design value for LRFD design
$M_{max-adj}$	= maximum bending moment adjusted to include self weight	$s$	= longitudinal center-to-center spacing of any two consecutive holes
$M_p$	= internal bending moment when all fibers in a cross section reach the yield stress	$S$	= shorthand for snow load = section modulus = allowable strength per length of a weld for a given size
$M_u$	= maximum moment from factored loads for LRFD beam design	$S_{req'd}$	= section modulus required at allowable stress
$M_y$	= internal bending moment when the extreme fibers in a cross section reach the yield stress	$S_{req'd-adj}$	= section modulus required at allowable stress when moment is adjusted to include self weight
$n$	= number of bolts	$SC$	= slip critical bolted connection
$n.a.$	= shorthand for neutral axis	$t$	= thickness of the connected material
		$t_f$	= thickness of flange of wide flange

$t_w$	= thickness of web of wide flange	$Z$	= plastic section modulus of a steel beam
$T$	= torque (axial moment)	$Z_x$	= plastic section modulus of a steel beam with respect to the x axis
	= shorthand for thermal load	$\Delta_{actual}$	= actual beam deflection
	= throat size of a weld	$\Delta_{allowable}$	= allowable beam deflection
$U$	= shear lag factor for steel tension member design	$\Delta_{limit}$	= allowable beam deflection limit
$U_{bs}$	= reduction coefficient for block shear rupture	$\Delta_{max}$	= maximum beam deflection
$V$	= internal shear force	$\epsilon_y$	= yield strain (no units)
$V_a$	= required shear (ASD)	$\phi$	= resistance factor
$V_{max}$	= maximum internal shear force		= diameter symbol
$V_{max-adj}$	= maximum internal shear force adjusted to include self weight	$\phi_b$	= resistance factor for bending for LRFD
$V_n$	= nominal shear strength capacity for LRFD beam design	$\phi_c$	= resistance factor for compression for LRFD
$V_u$	= maximum shear from factored loads for LRFD beam design	$\phi_t$	= resistance factor for tension for LRFD
$w$	= name for distributed load	$\phi_v$	= resistance factor for shear for LRFD
$w_{adjusted}$	= adjusted distributed load for equivalent live load deflection limit	$\gamma$	= load factor in LRFD design
$w_{equivalent}$	= the equivalent distributed load derived from the maximum bending moment	$\pi$	= pi (3.1415 radians or 180°)
$w_{self\ wt}$	= name for distributed load from self weight of member	$\theta$	= slope of the beam deflection curve
$W$	= shorthand for wind load	$\rho$	= radial distance
$x$	= horizontal distance	$\sigma$	= engineering symbol for normal stress
$X$	= bearing type connection with threads excluded from the shear plane	$\Omega$	= safety factor for ASD
$y$	= vertical distance	$\int$	= symbol for integration
		$\Sigma$	= summation symbol

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## Steel Design

Structural design standards for steel are established by the *Manual of Steel Construction* published by the American Institute of Steel Construction, and uses **Allowable Stress Design** and **Load and Factor Resistance Design**. With the 13<sup>th</sup> edition, both methods are combined in one volume which provides common requirements for analyses and design and requires the application of the same set of specifications.

## Materials

American Society for Testing Materials (ASTM) is the organization responsible for material and other standards related to manufacturing. Materials meeting their standards are guaranteed to have the published strength and material properties for a designation.

A36 – carbon steel used for plates, angles

$F_y = 36 \text{ ksi}$ ,  $F_u = 58 \text{ ksi}$ ,  $E = 29,000 \text{ ksi}$

A572 – high strength low-alloy used for some beams

$F_y = 60 \text{ ksi}$ ,  $F_u = 75 \text{ ksi}$ ,  $E = 29,000 \text{ ksi}$

A992 – for building framing used for most beams

$F_y = 50 \text{ ksi}$ ,  $F_u = 65 \text{ ksi}$ ,  $E = 29,000 \text{ ksi}$

(A572 Grade 50 has the same properties as A992)

ASD 
$$R_a \leq R_n / \Omega$$

where  $R_a$  = required strength (dead or live; force, moment or stress)  
 $R_n$  = nominal strength specified for ASD  
 $\Omega$  = safety factor

Factors of Safety are applied to the limit strengths for allowable strength values:

bending (braced, $L_b < L_p$ )	$\Omega = 1.67$
bending (unbraced, $L_p < L_b$ and $L_b > L_r$ )	$\Omega = 1.67$ (nominal moment reduces)
shear (beams)	$\Omega = 1.5$ or $1.67$
shear (bolts)	$\Omega = 2.00$ (tabular nominal strength)
shear (welds)	$\Omega = 2.00$

- $L_b$  is the unbraced length between bracing points, laterally
- $L_p$  is the limiting laterally unbraced length for the limit state of yielding
- $L_r$  is the limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling

LRFD 
$$R_u \leq \phi R_n \quad \text{where } \dots R_u = \sum \gamma_i R_i$$

where  $\phi$  = resistance factor  
 $\gamma$  = load factor for the type of load  
 $R$  = load (dead or live; force, moment or stress)  
 $R_u$  = factored load (moment or stress)  
 $R_n$  = nominal load (ultimate capacity; force, moment or stress)

*Nominal strength* is defined as the

capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths (such as yield strength,  $F_y$ , or ultimate strength,  $F_u$ ) and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions

### Factored Load Combinations

The design strength,  $\phi R_n$ , of each structural element or structural assembly must equal or exceed the design strength based on the ASCE-7 (2010) combinations of factored nominal loads:

$$\begin{aligned}
 &1.4D \\
 &1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \\
 &1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W) \\
 &1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R) \\
 &1.2D + 1.0E + L + 0.2S \\
 &0.9D + 1.0W \\
 &0.9D + 1.0E
 \end{aligned}$$

### Criteria for Design of Beams

Allowable normal stress or normal stress from LRFD should not be exceeded:

$$F_b \text{ or } \phi F_n \geq f_b = \frac{Mc}{I}$$

$$(M_a \leq M_n / \Omega \text{ or } M_u \leq \phi_b M_n)$$

Knowing M and  $F_b$ , the minimum section modulus fitting the limit is:

$$S_{req'd} \geq \frac{M}{F_b}$$

### Determining Maximum Bending Moment

Drawing V and M diagrams will show us the maximum values for design. Remember:

$$\begin{aligned}
 V &= \Sigma(-w)dx & \frac{dV}{dx} &= -w & \frac{dM}{dx} &= V \\
 M &= \Sigma(V)dx
 \end{aligned}$$

### Determining Maximum Bending Stress

For a prismatic member (constant cross section), the maximum normal stress will occur at the maximum moment.

For a *non-prismatic* member, the stress varies with the cross section AND the moment.

### Deflections

If the bending moment changes,  $M(x)$  across a beam of constant material and cross section then the curvature will change:  $\frac{1}{R} = \frac{M(x)}{EI}$

The slope of the n.a. of a beam,  $\theta$ , will be tangent to the radius of curvature, R:  $\theta = slope = \frac{1}{EI} \int M(x)dx$

The equation for deflection,  $y$ , along a beam is:  $y = \frac{1}{EI} \int \theta dx = \frac{1}{EI} \iint M(x)dx$

Elastic curve equations can be found in handbooks, textbooks, design manuals, etc...Computer programs can be used as well. Elastic curve equations can be superimposed ONLY if the stresses are in the elastic range.

*The deflected shape is roughly the same shape flipped as the bending moment diagram but is constrained by supports and geometry.*

Allowable Deflection Limits

All building codes and design codes limit deflection for beam types and damage that could happen based on service condition and severity.

$$y_{max}(x) = \Delta_{actual} \leq \Delta_{allowable} = L / \text{value}$$

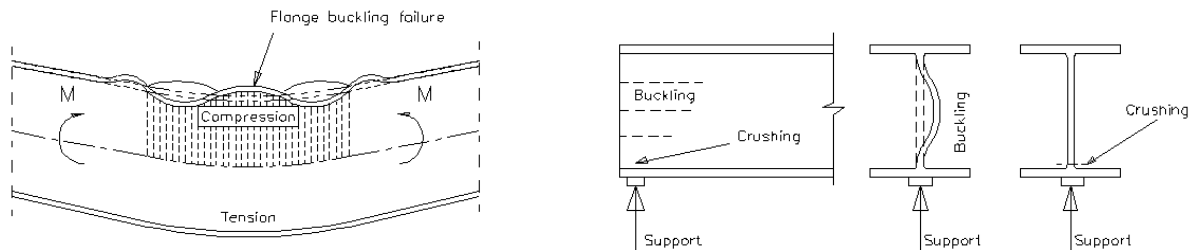
Use	LL only	DL+LL
Roof beams:		
Industrial	L/180	L/120
Commercial		
plaster ceiling	L/240	L/180
no plaster	L/360	L/240
Floor beams:		
Ordinary Usage	L/360	L/240
Roof or floor (damageable elements)		L/480

Lateral Buckling

With compression stresses in the top of a beam, a sudden “popping” or buckling can happen even at low stresses. In order to prevent it, we need to brace it along the top, or laterally brace it, or provide a bigger I<sub>y</sub>.

Local Buckling in Steel I Beams– Web Crippling or Flange Buckling

Concentrated forces on a steel beam can cause the web to buckle (called **web crippling**). Web stiffeners under the beam loads and bearing plates at the supports reduce that tendency. Web stiffeners also prevent the web from shearing in plate girders.



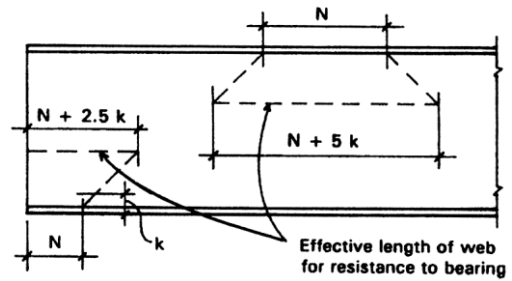
The maximum support load and interior load can be determined from:

$$P_{n(\text{max-end})} = (2.5k + N)F_{yw}t_w$$

$$P_{n(\text{interior})} = (5k + N)F_{yw}t_w$$

where  $t_w$  = thickness of the web  
 $F_{yw}$  = yield strength of the web  
 $N$  = bearing length  
 $k$  = dimension to fillet found in beam section tables

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$



Beam Loads & Load Tracing

In order to determine the loads on a beam (or girder, joist, column, frame, foundation...) we can start at the top of a structure and determine the *tributary area* that a load acts over and the beam needs to support. Loads come from material weights, people, and the environment. This area is assumed to be from half the distance to the next beam over to halfway to the next beam.

The reactions must be supported by the next lower structural element *ad infinitum*, to the ground.

*LRFD - Bending or Flexure*

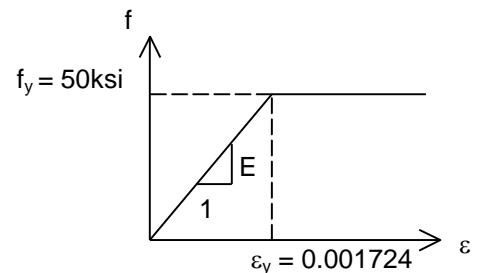
For determining the flexural design strength,  $\phi_b M_n$ , for resistance to pure bending (no axial load) in most flexural members where the following conditions exist, a single calculation will suffice:

$$\Sigma \gamma_i R_i = M_u \leq \phi_b M_n = 0.9 F_y Z$$

where  $M_u$  = maximum moment from factored loads  
 $\phi_b$  = resistance factor for bending = 0.9  
 $M_n$  = nominal moment (ultimate capacity)  
 $F_y$  = yield strength of the steel  
 $Z$  = plastic section modulus

*Plastic Section Modulus*

Plastic behavior is characterized by a yield point and an increase in strain with no increase in stress.



*Internal Moments and Plastic Hinges*

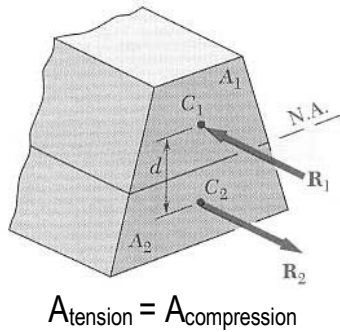
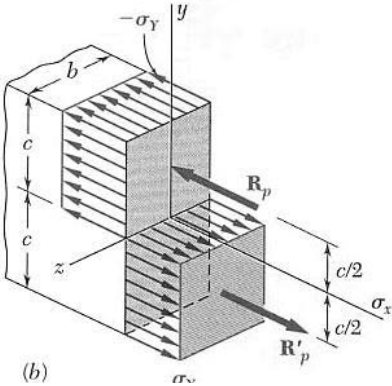
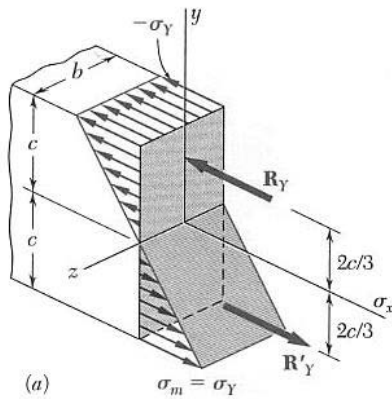
Plastic hinges can develop when all of the material in a cross section sees the yield stress. Because all the material at that section can strain without any additional load, the member segments on either side of the hinge can rotate, possibly causing instability.

For a rectangular section:

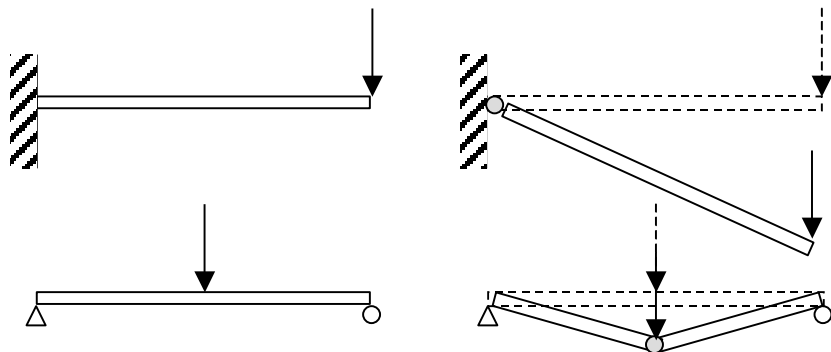
Elastic to  $f_y$ : 
$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y = \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

Fully Plastic: 
$$M_{ult} \text{ or } M_p = bc^2 f_y = \frac{3}{2} M_y$$

For a non-rectangular section and internal equilibrium at  $\sigma_y$ , the n.a. will not necessarily be at the centroid. The n.a. occurs where the  $A_{tension} = A_{compression}$ . The reactions occur at the centroids of the tension and compression areas.



*Instability from Plastic Hinges*



*Shape Factor:*

The ratio of the plastic moment to the elastic moment at yield:

$$k = \frac{M_p}{M_y}$$

$k = 3/2$  for a rectangle  
 $k \approx 1.1$  for an I beam

*Plastic Section Modulus*

$$Z = \frac{M_p}{f_y} \quad \text{and} \quad k = \frac{Z}{S}$$



*Design for Shear*

$$V_a \leq V_n / \Omega \text{ or } V_u \leq \phi_v V_n$$

The nominal shear strength is dependent on the cross section shape. Case 1: With a thick or stiff web, the shear stress is resisted by the web of a wide flange shape (with the exception of a handful of W's). Case 2: When the web is not stiff for doubly symmetric shapes, singly symmetric shapes (like channels) (excluding round high strength steel shapes), inelastic web buckling occurs. When the web is very slender, elastic web buckling occurs, reducing the capacity even more:

$$\text{Case 1) For } h/t_w \leq 2.24 \sqrt{\frac{E}{F_y}} \quad V_n = 0.6 F_{yw} A_w \quad \phi_v = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

where  $h$  equals the clear distance between flanges less the fillet or corner radius for rolled shapes

$V_n$  = nominal shear strength

$F_{yw}$  = yield strength of the steel in the web

$A_w = t_w d$  = area of the web

$$\text{Case 2) For } h/t_w > 2.24 \sqrt{\frac{E}{F_y}} \quad V_n = 0.6 F_{yw} A_w C_v \quad \phi_v = 0.9 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where  $C_v$  is a reduction factor (1.0 or less by equation)

*Design for Flexure*

$$M_a \leq M_n / \Omega \text{ or } M_u \leq \phi_b M_n \quad \phi_b = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

The nominal flexural strength  $M_n$  is the *lowest* value obtained according to the limit states of

1. yielding, limited at length  $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}$ , where  $r_y$  is the radius of gyration in  $y$
2. lateral-torsional buckling (inelastic) limited at length  $L_r$
3. flange local buckling
4. web local buckling

Beam design charts show available moment,  $M_n/\Omega$  and  $\phi_b M_n$ , for unbraced length,  $L_b$ , of the compression flange in one-foot increments from 1 to 50 ft. for values of the bending coefficient  $C_b = 1$ . For values of  $1 < C_b \leq 2.3$ , the required flexural strength  $M_u$  can be reduced by dividing it by  $C_b$ . ( $C_b = 1$  when the bending moment at any point within an unbraced length is larger than that at both ends of the length.  $C_b$  of 1 is conservative and permitted to be used in any case. When the free end is unbraced in a cantilever or overhang,  $C_b = 1$ . The full formula is provided below.)

**NOTE:** the self weight is not included in determination of  $M_n/\Omega$  or  $\phi_b M_n$

### Compact Sections

For a laterally braced *compact* section (one for which the plastic moment can be reached before local buckling) only the limit state of yielding is applicable. For unbraced compact beams and non-compact tees and double angles, only the limit states of yielding and lateral-torsional buckling are applicable.

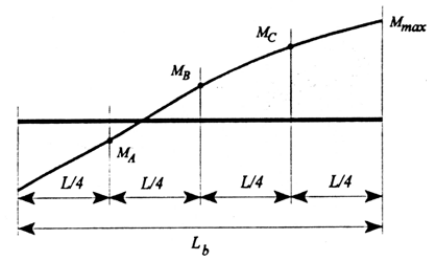
Compact sections meet the following criteria:  $\frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}}$  and  $\frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}}$

where:

- $b_f$  = flange width in inches
- $t_f$  = flange thickness in inches
- $E$  = modulus of elasticity in ksi
- $F_y$  = minimum yield stress in ksi
- $h_c$  = height of the web in inches
- $t_w$  = web thickness in inches

With lateral-torsional buckling the nominal flexural strength is

$$M_n = C_b \left[ M_p - (M_p - 0.7F_y S_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$



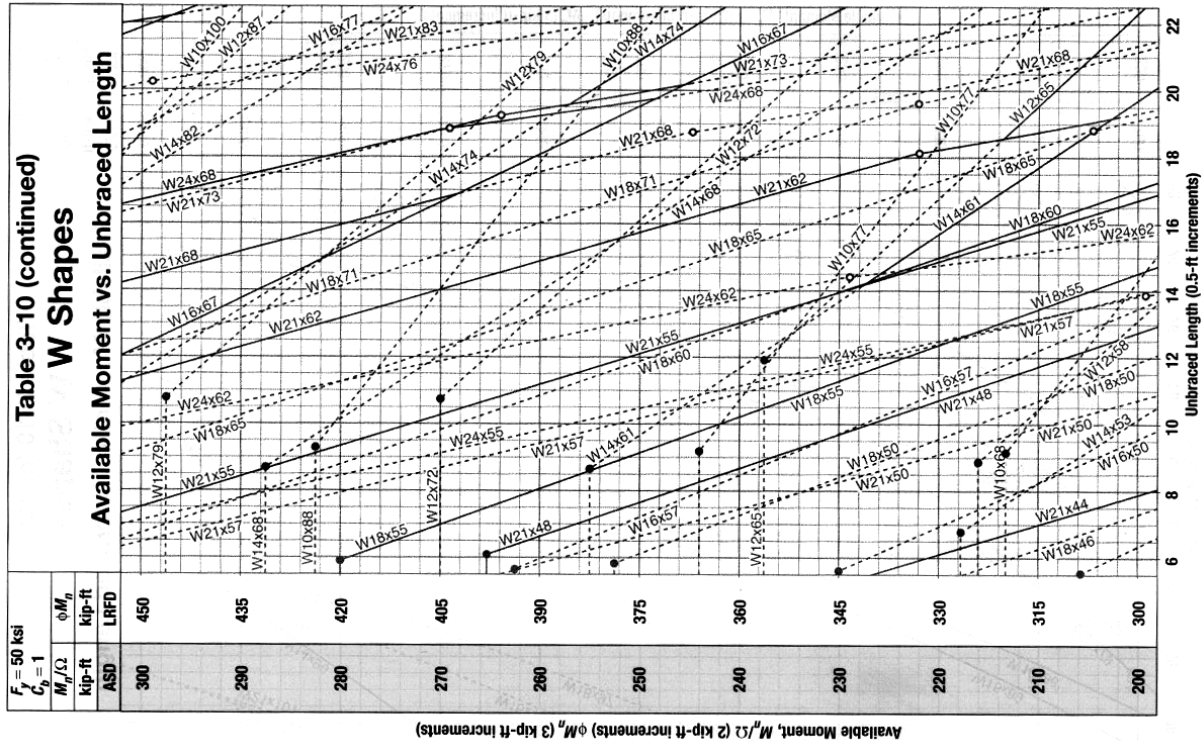
where  $C_b$  is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

- $M_{max}$  = absolute value of the maximum moment in the unbraced beam segment
- $M_A$  = absolute value of the moment at the quarter point of the unbraced beam segment
- $M_B$  = absolute value of the moment at the center point of the unbraced beam segment
- $M_C$  = absolute value of the moment at the three quarter point of the unbraced beam segment length.

### Available Flexural Strength Plots

Plots of the available moment for the unbraced length for wide flange sections are useful to find sections to satisfy the design criteria of  $M_a \leq M_n / \Omega$  or  $M_u \leq \phi_b M_n$ . The maximum moment that can be applied on a beam (taking self weight into account),  $M_a$  or  $M_u$ , can be plotted against the unbraced length,  $L_b$ . The limit  $L_p$  is indicated by a solid dot ( $\bullet$ ), while  $L_r$  is indicated by an open dot ( $\circ$ ). Solid lines indicate the most economical, while dashed lines indicate there is a lighter section that could be used.  $C_b$ , which is a modification factor for non-zero moments at the ends, is 1 for simply supported beams (0 moments at the ends). (see figure)



Design Procedure

The intent is to find the most light weight member (which is economical) satisfying the section modulus size.

1. Determine the unbraced length to choose the limit state (yielding, lateral torsional buckling or more extreme) and the factor of safety and limiting moments. Determine the material.
2. Draw V & M, finding  $V_{max}$  and  $M_{max}$ . for unfactored loads (ASD,  $V_a$  &  $M_a$ ) or from factored loads (LRFD,  $V_u$  &  $M_u$ )
3. Calculate  $Z_{req'd}$  when yielding is the limit state. This step is equivalent to determining if

$$f_b = \frac{M_{max}}{S} \leq F_b, Z_{req'd} \geq \frac{M_{max}}{F_b} = \frac{M_{max}}{F_y/\Omega} \text{ and } Z \geq \frac{M_u}{\phi_b F_y}$$

$$M_a \leq M_n / \Omega \text{ or } M_u \leq \phi_b M_n$$

If the limit state is something other than yielding, determine the nominal moment,  $M_n$ , or use plots of available moment to unbraced length,  $L_b$ .

4. For steel: use the section charts to find a trial  $Z$  and remember that the beam self weight (the second number in the section designation) will increase  $Z_{req'd}$ . The design charts show the lightest section within a grouping of similar  $Z$ 's.

\*\*\*\* Determine the "updated"  $V_{max}$  and  $M_{max}$  including the beam self weight, and verify that the updated  $Z_{req'd}$  has been met. \*\*\*\*

**TABLE 9.1 Load Factor Resistance Design Selection**

Designation	$Z_x$ in. <sup>3</sup>	$F_y = 36$ ksi			
		$L_p$ ft	$L_r$ ft	$M_p$ kip-ft	$M_r$ kip-ft
W 33 × 141	514	10.1	30.1	1,542	971
W 30 × 148	500	9.50	30.6	1,500	945
W 24 × 162	468	12.7	45.2	1,404	897
W 24 × 146	418	12.5	42.0	1,254	804
W 33 × 118	415	9.67	27.8	1,245	778
W 30 × 124	408	9.29	28.2	1,224	769
W 21 × 147	373	12.3	46.4	1,119	713
W 24 × 131	370	12.4	39.3	1,110	713
W 18 × 158	356	11.4	56.5	1,068	672

5. Consider lateral stability.
6. Evaluate horizontal shear using  $V_{\max}$ . This step is equivalent to determining if  $f_v \leq F_v$  is satisfied to meet the design criteria that  $V_a \leq V_n / \Omega$  or  $V_u \leq \phi V_n$

$$\text{For I beams: } f_{v-\max} = \frac{3V}{2A} \approx \frac{V}{A_{web}} = \frac{V}{t_w d} \quad V_n = 0.6F_{yw}A_w \quad \text{or } V_n = 0.6F_{yw}A_w C_v$$

$$\text{Others: } f_{v-\max} = \frac{VQ}{Ib}$$

7. Provide adequate bearing area at supports. This step is equivalent to determining if  $f_p = \frac{P}{A} \leq F_p$  is satisfied to meet the design criteria that  $P_a \leq P_n / \Omega$  or  $P_u \leq \phi P_n$

8. Evaluate shear due to torsion  $f_v = \frac{T\rho}{J}$  or  $\frac{T}{c_1 ab^2} \leq F_v$  (circular section or rectangular)

9. Evaluate the deflection to determine if  $\Delta_{\max LL} \leq \Delta_{LL-\text{allowed}}$  and/or  $\Delta_{\max Total} \leq \Delta_{Total\text{ allowed}}$

\*\*\*\* note: when  $\Delta_{\text{calculated}} > \Delta_{\text{limit}}$ ,  $I_{\text{req'd}}$  can be found with:  $I_{\text{req'd}} \geq \frac{\Delta_{\text{too big}}}{\Delta_{\text{limit}}} I_{\text{trial}}$   
and  $Z_{\text{req'd}}$  will be satisfied for similar self weight \*\*\*\*\*

FOR ANY EVALUATION:

Redesign (with a new section) at any point that a stress or serviceability criteria is NOT satisfied and re-evaluate each condition until it is satisfactory.

### Load Tables for Uniformly Loaded Joists & Beams

Tables exist for the common loading situation of uniformly distributed load. The tables either provide the safe distributed load based on bending and deflection limits, they give the allowable span for specific live and dead loads including live load deflection limits.

If the load is *not uniform*, an *equivalent uniform load* can be calculated from the maximum moment equation:  $M_{\max} = \frac{w_{\text{equivalent}} L^2}{8}$

If the deflection limit is less, the design live load to check against allowable must be increased, ex.

$$w_{\text{adjusted}} = w_{\text{ll-have}} \left( \frac{L/360}{L/400} \right) \begin{array}{l} \text{table limit} \\ \text{wanted} \end{array}$$

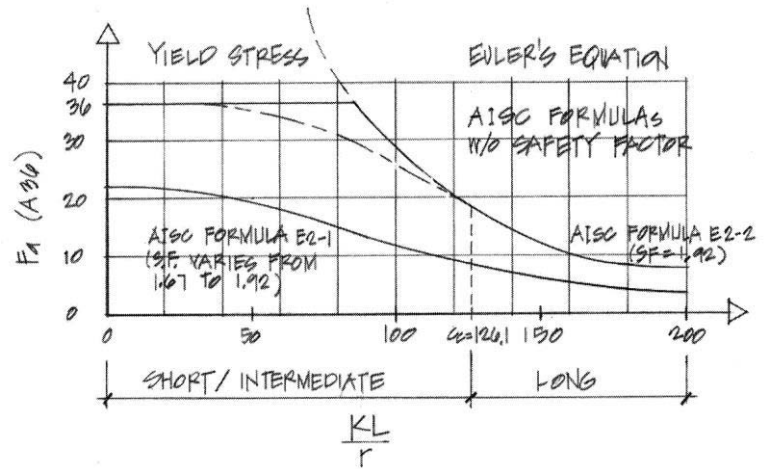
### Criteria for Design of Columns

If we know the loads, we can select a section that is adequate for strength & buckling.

If we know the length, we can find the limiting load satisfying strength & buckling.

*Allowable Stress Design*

The allowable stress design provisions prior to the combined design of the 13<sup>th</sup> edition of the AISC Steel Construction Manual had relationships for short and intermediate length columns (crushing and the transition to inelastic buckling), and long columns (buckling) as shown in the figure. The transition slenderness ratio is based on the yield strength and modulus of elasticity and are 126.1 ( $F_y = 36$  ksi) and 107.0 ( $F_y = 50$  ksi) with a limiting slenderness ratio of 200.



*Design for Compression*

American Institute of Steel Construction (AISC) Manual 14<sup>th</sup> ed:

$$P_a \leq P_n / \Omega \text{ or } P_u \leq \phi_c V_n \quad \text{where}$$

$$P_u = \sum \gamma_i P_i$$

- $\gamma$  is a load factor
- P is a load type
- $\phi$  is a resistance factor
- $P_n$  is the nominal load capacity (strength)

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

For compression  $P_n = F_{cr} A_g$

where :  $A_g$  is the cross section area and  $F_{cr}$  is the flexural buckling stress

The flexural buckling stress,  $F_{cr}$ , is determined as follows:

when  $\frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}}$  or ( $F_e \geq 0.44F_y$ ):

$$F_{cr} = \left[ 0.658^{\frac{F_y}{F_e}} \right] F_y$$

when  $\frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}}$  or ( $F_e < 0.44F_y$ ):

$$F_{cr} = 0.877F_e$$

where  $F_e$  is the elastic critical buckling stress:

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Design Aids

Tables exist for the value of the flexural buckling stress based on slenderness ratio. In addition, tables are provided in the AISC Manual for Available Strength in Axial Compression based on the effective length with respect to least radius of gyration,  $r_y$ . If the critical effective length is about the largest radius of gyration,  $r_x$ , it can be turned into an effective length about the y axis by dividing by the fraction  $r_x/r_y$ .

Sample AISC Table for Available Strength in Axial Compression

Shape		W12 <sub>x</sub>											
		96		87		79		72		65			
Design	Wt/ft	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$		
		0	844	1270	766	1150	694	1040	633	951	571	859	
6	811	1220	735	1110	667	1000	607	913	548	824			
7	800	1200	725	1090	657	987	598	899	540	811			
8	787	1180	713	1070	646	971	588	884	531	798			
9	772	1160	699	1050	634	952	577	867	520	782			
10	756	1140	685	1030	620	932	565	849	509	765			
11	739	1110	669	1010	606	910	551	828	497	747			
12	720	1080	652	980	590	887	537	807	484	727			
13	701	1050	634	953	573	862	522	784	470	706			
14	680	1020	615	924	556	836	506	761	456	685			
15	659	990	595	895	538	809	490	736	441	662			
16	637	957	575	864	520	781	473	710	425	639			
17	614	923	554	833	501	752	455	684	409	615			
18	591	888	533	801	481	723	437	657	393	591			
19	567	852	511	769	461	694	419	630	377	566			
20	543	816	490	736	442	664	401	603	360	541			
22	495	744	446	670	402	603	365	548	327	491			
24	447	672	402	605	362	544	328	493	294	442			
26	401	602	360	541	323	486	293	440	262	393			
28	356	534	319	479	286	430	259	389	231	347			
30	312	469	279	420	250	376	226	340	202	303			
32	274	412	246	369	220	331	199	299	177	267			
34	243	365	218	327	195	293	176	265	157	236			
36	217	326	194	292	174	261	157	236	140	211			
38	195	292	174	262	156	234	141	212	126	189			
40	176	264	157	236	141	212	127	191	114	171			
Effective length $KL$ (ft) with respect to least radius of gyration $r_y$		Properties											
		$P_u$ (kips)	137	206	121	181	104	157	90.9	136	78.2	117	
$P_w$ (kips/in.)	18.3	27.5	17.2	25.8	15.7	23.5	14.3	21.5	13.0	19.5			
$P_u/\Omega_c$ (kips)	296	445	243	366	185	278	142	213	106	159			
$\phi_c P_u$ (kips)	152	228	123	185	101	152	84.0	126	68.5	103			
$L_p$ (ft)	10.9	10.8	10.8	10.8	10.8	10.7	10.7	10.7	11.9	11.9			
$L_r$ (ft)	46.6	43.0	43.0	43.0	39.9	37.4	37.4	37.4	35.1	35.1			
$A_g$ (in. <sup>2</sup> )	28.2	28.2	28.2	25.6	23.2	21.1	21.1	21.1	19.1	19.1			
$I_y$ (in. <sup>4</sup> )	833	740	740	740	662	597	597	597	533	533			
$I_x$ (in. <sup>4</sup> )	270	241	241	241	216	195	195	195	174	174			
$r_y$ (in.)	3.09	3.07	3.07	3.07	3.05	3.04	3.04	3.04	3.02	3.02			
Ratio $r_x/r_y$	1.76	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75			
$P_u/(KL)^2/10^4$ (k-in. <sup>2</sup> )	23800	21200	21200	18900	17100	15300	15300	15300	15300	15300			
$\phi_c P_u/(KL)^2/10^4$ (k-in. <sup>2</sup> )	7730	6900	6900	6180	5580	4980	4980	4980	4980	4980			
ASD		LRFD											
$\Omega_c = 1.67$		$\phi_c = 0.90$											

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Procedure for Analysis

1. Calculate  $KL/r$  for each axis (if necessary). The largest will govern the buckling load.
2. Find  $F_{cr}$  as a function of  $KL/r$  from the appropriate equation (above) or table.
3. Compute  $P_n = F_{cr} \cdot A_g$  or alternatively compute  $f_c = P/A$  or  $P_u/A$
4. Is the design satisfactory?

Is  $P_a \leq P_n/\Omega_c$  or  $P_u \leq \phi_c P_n$ ?  $\Rightarrow$  yes, it is; no, it is no good  
 or Is  $f_c \leq F_{cr}/\Omega_c$  or  $\phi_c F_{cr}$ ?  $\Rightarrow$  yes, it is; no, it is no good

Procedure for Design

1. Guess a size by picking a section.
2. Calculate  $KL/r$  for each axis (if necessary). The largest will govern the buckling load.

3. Find  $F_{cr}$  as a function of  $KL/r$  from appropriate equation (above) or table.
4. Compute  $P_n = F_{cr} \cdot A_g$  or alternatively compute  $f_c = P/A$  or  $P_u/A$
5. Is the design satisfactory?

Is  $P_a \leq P_n/\Omega$  or  $P_u \leq \phi_c P_n$ ? yes, it is; no, pick a bigger section and go back to step 2.

Is  $f_c \leq F_{cr}/\Omega$  or  $\phi_c F_{cr}$ ?  $\Rightarrow$  yes, it is; no, pick a bigger section and go back to step 2.

6. Check design efficiency by calculating percentage of capacity used:

$$\frac{P_a}{P_n/\Omega} \cdot 100\% \text{ or } \frac{P_u}{\phi_c P_n} \cdot 100\%$$

If value is between 90-100%, it is efficient.

If values is less than 90%, pick a smaller section and go back to step 2.

### Columns with Bending (Beam-Columns)

In order to *design* an adequate section for allowable stress, we have to start somewhere:

1. Make assumptions about the limiting stress from:
  - buckling
  - axial stress
  - combined stress
2. See if we can find values for  $r$  or  $A$  or  $Z$
3. Pick a trial section based on if we think  $r$  or  $A$  is going to govern the section size.
4. Analyze the stresses and compare to allowable using the allowable stress method or interaction formula for eccentric columns.
5. Did the section pass the capacity adequacy test?
  - If not, do you *increase*  $r$  or  $A$  or  $Z$ ?
  - If so, is the difference really big so that you could *decrease*  $r$  or  $A$  or  $Z$  to make it more efficient (economical)?
6. Change the section choice and go back to step 4. Repeat until the section meets the stress criteria.

### *Design for Combined Compression and Flexure:*

The interaction of compression and bending are included in the form for two conditions based on the size of the required axial force to the available axial strength. This is notated as  $P_r$  (either  $P$  from ASD or  $P_u$  from LRFD) for the axial force being supported, and  $P_c$  (either  $P_n/\Omega$  for ASD or  $\phi_c P_n$  for LRFD). The increased bending moment due to the P- $\Delta$  effect must be determined and used as the moment to resist.

For  $\frac{P_r}{P_c} \geq 0.2$ :

$$\frac{P}{P_n/\Omega} + \frac{8}{9} \left( \frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega} \right) \leq 1.0 \quad \text{(ASD)}$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \quad \text{(LRFD)}$$

For  $\frac{P_r}{P_c} < 0.2$ :

$$\frac{P}{2P_n/\Omega} + \left( \frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega} \right) \leq 1.0 \quad \text{(ASD)}$$

$$\frac{P_u}{2\phi_c P_n} + \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \quad \text{(LRFD)}$$

where:

for compression  $\phi_c = 0.90$  (LRFD)  $\Omega = 1.67$  (ASD)  
 for bending  $\phi_b = 0.90$  (LRFD)  $\Omega = 1.67$  (ASD)

For a braced condition, the moment magnification factor  $B_1$  is determined by  $B_1 = \frac{C_m}{1 - (P_u/P_{e1})} \geq 1.0$

where  $C_m$  is a modification factor accounting for end conditions

When not subject to transverse loading between supports in plane of bending:

=  $0.6 - 0.4 (M_1/M_2)$  where  $M_1$  and  $M_2$  are the end moments and  $M_1 < M_2$ .  $M_1/M_2$  is positive when the member is bent in reverse curvature (same direction), negative when bent in single curvature.

When there is transverse loading between the two ends of a member:

= 0.85, members with restrained (fixed) ends  
 = 1.00, members with unrestrained ends

$P_{e1}$  = Euler buckling strength

$$P_{e1} = \frac{\pi^2 EA}{(Kl/r)^2}$$

### Criteria for Design of Connections

Connections must be able to transfer any axial force, shear, or moment from member to member or from beam to column.

Connections for steel are typically high strength bolts and electric arc welds. Recommended practice for ease of construction is to specified *shop welding* and *field bolting*.

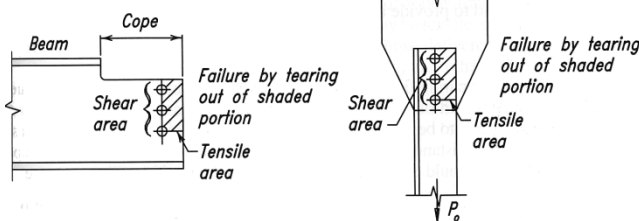


Fig. C-J4.1. Failure for block shear rupture limit state.

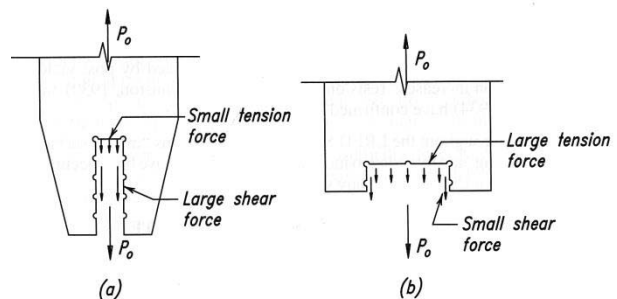


Fig. C-J4.2. Block shear rupture in tension.

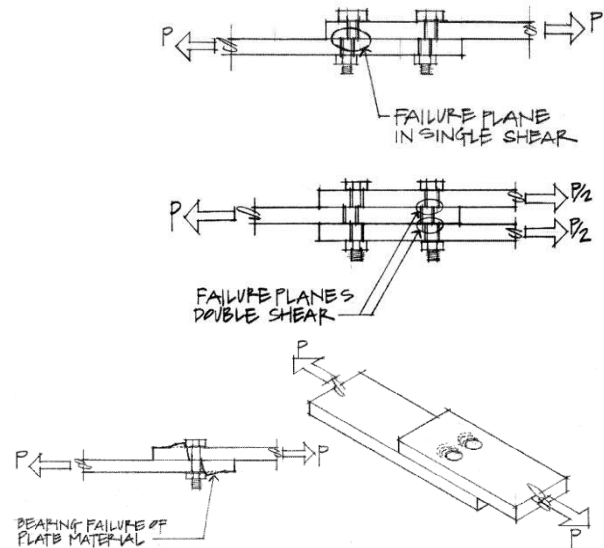


## Bolted and Welded Connections

The limit state for connections depends on the loads:

1. tension yielding
2. shear yielding
3. bearing yielding
4. bending yielding due to eccentric loads
5. rupture

Welds must resist tension AND shear stress. The design strengths depend on the weld materials.



## Bolted Connection Design

Bolt designations signify material and type of connection where

SC: slip critical

N: bearing-type connection with bolt threads *included* in shear plane

X: bearing-type connection with bolt threads *excluded* from shear plane

A307: similar in strength to A36 steel (also known as ordinary, common or unfinished bolts)

A325: high strength bolts (Group A)

A490: high strength bolts (higher than A325) (Group B)

Bearing-type connection: no frictional resistance in the contact surfaces is assumed and slip between members occurs as the load is applied. (Load transfer through bolt only).

Slip-critical connections: bolts are torqued to a high tensile stress in the shank, resulting in a clamping force on the connected parts. (Shear resisted by clamping force).

Requires inspections and is useful for structures seeing dynamic or fatigue loading.

Class A indicates the *faying* (contact) surfaces are clean mill scale or adequate paint system, while Class B indicates blast cleaning or paint for  $\mu = 0.50$ .

Bolts rarely fail in **bearing**. The material with the hole will more likely yield first.

For the determination of the net area of a bolt hole the width is taken as  $1/16$ " greater than the nominal dimension of the hole. Standard diameters for bolt holes are  $1/16$ " larger than the bolt diameter. (This means the net width will be  $1/8$ " larger than the bolt.)

## *Design for Bolts in Bearing, Shear and Tension*

Available shear values are given by bolt type, diameter, and loading (Single or Double shear) in AISC manual tables. Available shear value for slip-critical connections are given for limit states of serviceability or strength by bolt type, hole type (standard, short-slotted, long-slotted or oversized), diameter, and loading. Available tension values are given by bolt type and diameter in AISC manual tables.

Allowable bearing force values are given by bolt diameter, ultimate tensile strength,  $F_u$ , of the connected part, and thickness of the connected part in AISC manual tables.

For shear OR tension (same equation) in bolts:  $R_u \leq R_n / \Omega$  or  $R_u \leq \phi R_n$   
 where  $R_u = \sum \gamma_i R_i$

- single shear (or tension)  $R_n = F_n A_b$
- double shear  $R_n = F_n 2A_b$

where  $\phi$  = the resistance factor  
 $F_n$  = the nominal tension or shear strength of the bolt  
 $A_b$  = the cross section area of the bolt

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

For bearing of plate material at bolt holes:

$$R_u \leq R_n / \Omega \text{ or } R_u \leq \phi R_n$$

where  $R_u = \sum \gamma_i R_i$

- deformation at bolt hole is a concern

$$R_n = 1.2L_c t F_u \leq 2.4dt F_u$$

- deformation at bolt hole is not a concern

$$R_n = 1.5L_c t F_u \leq 3.0dt F_u$$

- long slotted holes with the slot perpendicular to the load

$$R_n = 1.0L_c t F_u \leq 2.0dt F_u$$

where  $R_n$  = the nominal bearing strength  
 $F_u$  = specified minimum tensile strength  
 $L_c$  = clear distance between the edges of the hole and the next hole or edge in the direction of the load  
 $d$  = nominal bolt diameter  
 $t$  = thickness of connected material

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

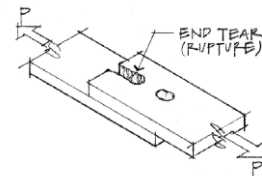


Figure 10.11 End tear-out.

The *minimum* edge distance from the center of the outer most bolt to the edge of a member is generally  $1\frac{3}{4}$  times the bolt diameter for the sheared edge and  $1\frac{1}{4}$  times the bolt diameter for the rolled or gas cut edges.

The *maximum* edge distance should not exceed 12 times the thickness of thinner member or 6 in.

Standard bolt hole spacing is 3 in. with the minimum spacing of  $2\frac{2}{3}$  times the diameter of the bolt,  $d_b$ . Common edge distance from the center of last hole to the edge is  $1\frac{1}{4}$  in..

### Tension Member Design

In steel tension members, there may be bolt holes which reduce the size of the cross section.

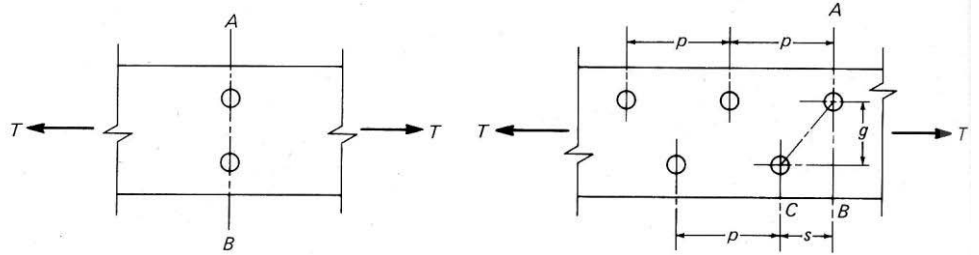
$g$  refers to the row spacing or *gage*

$p$  refers to the bolt spacing or *pitch*

$s$  refers to the longitudinal spacing of two consecutive holes

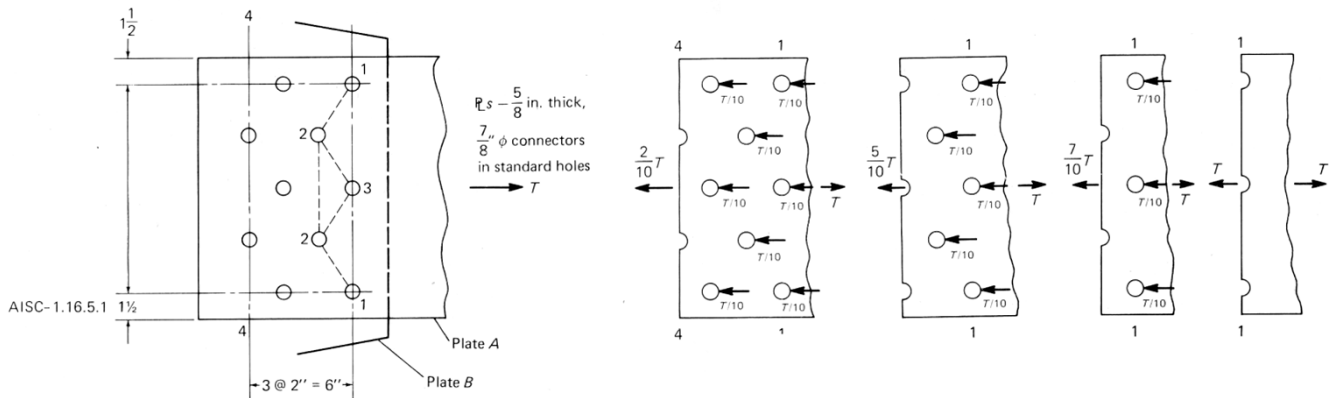
**Effective Net Area:**

The smallest effective are must be determined by subtracting the bolt hole areas. With staggered holes, the shortest length must be evaluated.



A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.

The effective net area,  $A_e$ , is determined from the net area,  $A_n$ , multiplied by a shear lag factor,  $U$ , which depends on the element type and connection configuration. If a portion of a connected member is not fully connected (like the leg of an angle), the unconnected part is not subject to the full stress and the shear lag factor can range from 0.6 to 1.0:  $A_e = A_n U$



For tension elements:

$$R_a \leq R_n / \Omega \text{ or } R_u \leq \phi R_n$$

$$\text{where } R_u = \sum \gamma_i R_i$$

1. yielding

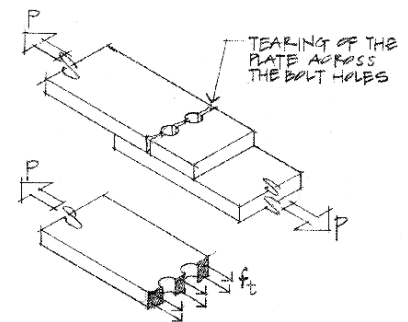
$$R_n = F_y A_g$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

2. rupture

$$R_n = F_u A_e$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

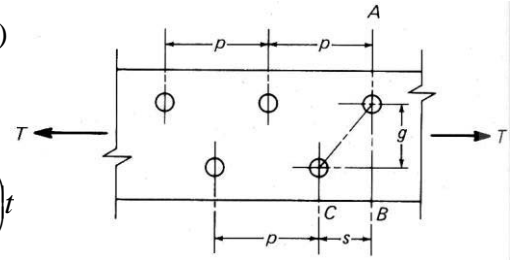


where  $A_g$  = the gross area of the member (excluding holes)  
 $A_e$  = the effective net area (with holes, etc.)

$F_y$  = the yield strength of the steel  
 $F_u$  = the tensile strength of the steel (ultimate)

When holes are staggered in a chain of holes (zigzagging) at diagonals, the length of each path from hole edge to edge is taken as the net area less each bolt hold area and the addition of

$$s^2/4g \text{ for each gage space in the chain: } A_n = bt - \sum ht - \sum \left( \frac{s^2}{4g} \right) t$$



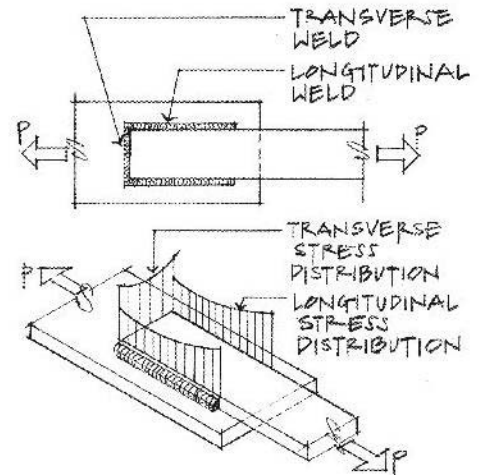
- where  $b$  is the plate width
- $t$  is the plate thickness
- $h$  is the standard hole diameter of each hole
- $s$  is the staggered hole spacing
- $g$  is the gage spacing between rows

Welded Connections

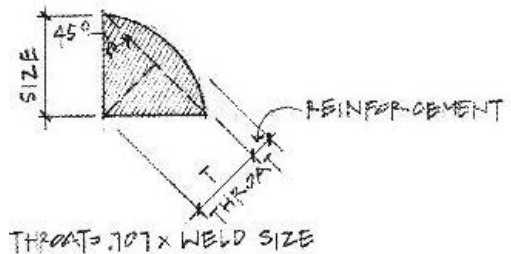
Weld designations include the strength in the name, i.e. E70XX has  $F_y = 70$  ksi. Welds are weakest in shear and are assumed to always fail in the shear mode.

The throat size,  $T$ , of a fillet weld is determined trigonometry by:  $T = 0.707 \times \text{weld size}^*$

\* When the submerged arc weld process is used, welds over 3/8" will have a throat thickness of 0.11 in. larger than the formula.



Weld sizes are limited by the size of the parts being put together and are given in AISC manual table J2.4 along with the allowable strength per length of fillet weld, referred to as  $S$ .



The *maximum* size of a fillet weld:

- a) can't be greater than the material thickness if it is 1/4" or less
- b) is permitted to be 1/16" less than the thickness of the material if it is over 1/4"

The *minimum length* of a fillet weld is 4 times the nominal size. If it is not, then the weld size used for design is 1/4 the length.

Intermittent fillet welds cannot be less than four times the weld size, not to be less than 1 1/2".

**TABLE J2.4**  
**Minimum Size of Fillet Welds**

Material Thickness of Thicker Part Joined (in.)	Minimum Size of Fillet Weld <sup>a</sup> (in.)
To 1/4 inclusive	1/8
Over 1/4 to 1/2	3/16
Over 1/2 to 3/4	1/4
Over 3/4	5/16

<sup>a</sup>Leg dimension of fillet welds. Single-pass welds must be used.

For fillet welds:

$$R_a \leq R_n / \Omega \text{ or } R_u \leq \phi R_n$$

$$\text{where } R_u = \sum \gamma_i R_i$$

for the weld metal:  $R_n = 0.6 F_{EXX} Tl = Sl$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where:

$T$  is throat thickness  
 $l$  is length of the weld

Available Strength of Fillet Welds per inch of weld ( $\phi S$ )		
Weld Size (in.)	E60XX (k/in.)	E70XX (k/in.)
3/16	3.58	4.18
1/4	4.77	5.57
5/16	5.97	6.96
3/8	7.16	8.35
7/16	8.35	9.74
1/2	9.55	11.14
5/8	11.93	13.92
3/4	14.32	16.70

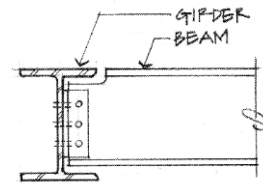
(not considering increase in throat with submerged arc weld process)

For a connected part, the other limit states for the base metal, such as tension yield, tension rupture, shear yield, or shear rupture **must** be considered.

Framed Beam Connections

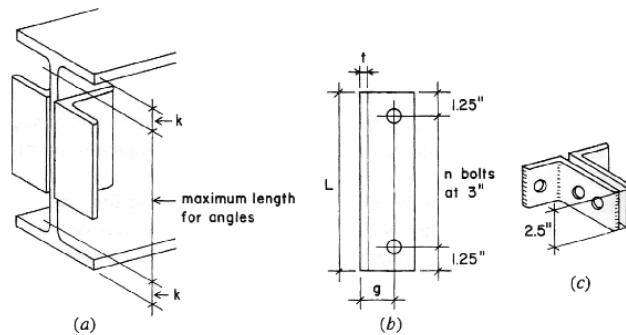
*Coping* is the term for cutting away part of the flange to connect a beam to another beam using welded or bolted angles.

AISC provides tables that give bolt and angle available strength knowing number of bolts, bolt type, bolt diameter, angle leg thickness, hole type and coping, and the wide flange beam being connected.



Group A bolts include A325, while Group B includes A490.

There are also tables for bolted/welded double-angle connections and all-welded double-angle connections.



**Sample AISC Table for Bolt and Angle Available Strength in All-Bolted Double-Angle Connections**

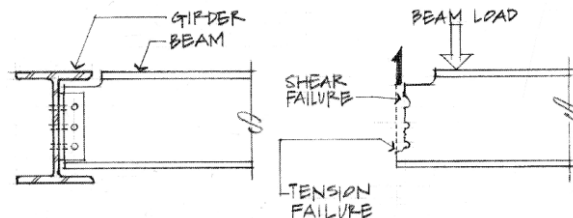
Beam		Angle		Table 10-1 (continued) All-Bolted Double-Angle Connections 3/4-in. Bolts															
F <sub>y</sub> = 50 ksi F <sub>u</sub> = 65 ksi		F <sub>y</sub> = 36 ksi F <sub>u</sub> = 58 ksi		Bolt and Angle Available Strength, kips															
4 Rows W24, 21, 18, 16		Bolt Group		Thread Cond.		Hole Type		Angle Thickness, in.											
								1/4		5/16		3/8		1/2					
								ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD				
Coped at Top Flange Only	Group A	N	STD	N	X	STD	STD	67.1	101	83.9	126	95.5	143	95.5	143				
								67.1	101	83.9	126	101	151	120	180				
		SC	Class A	STD	OVS	SSLT	STD	STD	50.6	75.9	50.6	75.9	50.6	75.9	50.6	75.9			
									43.1	64.5	43.1	64.5	43.1	64.5	43.1	64.5			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	84.4	127	84.4	127			
									65.3	97.9	71.9	108	71.9	108	71.9	108			
	Group B	N	STD	X	STD	STD	STD	67.1	101	83.9	126	101	151	120	180				
								67.1	101	83.9	126	101	151	134	201				
		SC	Class A	STD	OVS	SSLT	STD	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9			
									53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	101	151	105	158			
									65.3	97.9	81.6	122	89.9	134	89.9	134			
Coped at Both Flanges	Group A	N	STD	N	X	STD	STD	67.1	101	83.9	126	101	151	120	180				
								67.1	101	83.9	126	101	151	134	201				
		SC	Class A	STD	OVS	SSLT	STD	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9			
									53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	101	151	105	158			
									65.3	97.9	81.6	122	89.9	134	89.9	134			
	Group B	N	STD	X	STD	STD	STD	67.1	101	83.9	126	101	151	120	180				
								67.1	101	83.9	126	101	151	134	201				
		SC	Class A	STD	OVS	SSLT	STD	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9			
									53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	101	151	105	158			
									65.3	97.9	81.6	122	89.9	134	89.9	134			
Uncoped	Group A	N	STD	N	X	STD	STD	67.1	101	83.9	126	101	151	120	180				
								67.1	101	83.9	126	101	151	134	201				
		SC	Class A	STD	OVS	SSLT	STD	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9			
									53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	101	151	105	158			
									65.3	97.9	81.6	122	89.9	134	89.9	134			
	Group B	N	STD	X	STD	STD	STD	67.1	101	83.9	126	101	151	120	180				
								67.1	101	83.9	126	101	151	134	201				
		SC	Class A	STD	OVS	SSLT	STD	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9			
									53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	101	151	105	158			
									65.3	97.9	81.6	122	89.9	134	89.9	134			
Support Available Strength per Inch Thickness, kips/in.	Group A	N	STD	N	X	STD	STD	67.1	101	83.9	126	101	151	120	180				
								67.1	101	83.9	126	101	151	134	201				
		SC	Class A	STD	OVS	SSLT	STD	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9			
									53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	101	151	105	158			
									65.3	97.9	81.6	122	89.9	134	89.9	134			
	Group B	N	STD	X	STD	STD	STD	67.1	101	83.9	126	101	151	120	180				
								67.1	101	83.9	126	101	151	134	201				
		SC	Class A	STD	OVS	SSLT	STD	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9			
									53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7			
		SC	Class B	STD	OVS	SSLT	STD	STD	67.1	101	83.9	126	101	151	105	158			
									65.3	97.9	81.6	122	89.9	134	89.9	134			
Notes:	STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load N = Threads included X = Threads excluded SC = Slip critical * Tabulated values include 1/4-in. reduction in end distance, L <sub>eh</sub> , to account for possible underun in beam length. Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.																		
	Hole Type	ASD	LRFD																
		468	702																

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

*Limiting Strength or Stability States*

In addition to resisting shear and tension in bolts and shear in welds, the connected materials may be subjected to shear, bearing, tension, flexure and even prying action. Coping can significantly reduce design strengths and may require web reinforcement. All the following must be considered:

- shear yielding
- shear rupture
- block shear rupture - failure of a block at a beam as a result of shear and tension
- tension yielding
- tension rupture
- local web buckling
- lateral torsional buckling



*Block Shear Strength (or Rupture):*

$$R_a \leq R_n / \Omega \text{ or } R_u \leq \phi R_n$$

$$\text{where } R_u = \sum \gamma_i R_i$$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where:

$A_{nv}$  is the net area subjected to shear

$A_{nt}$  is the net area subjected to tension

$A_{gv}$  is the gross area subjected to shear

$U_{bs} = 1.0$  when the tensile stress is uniform (most cases)

$= 0.5$  when the tensile stress is non-uniform

### Gusset Plates

Gusset plates are used for truss member connections where the geometry prevents the members from coming together at the joint “point”. Members being joined are typically double angles.

### Decking

Shaped, thin sheet-steel panels that span several joists or evenly spaced support behave as continuous beams. Design tables consider a “1 unit” wide strip across the supports and determine maximum bending moment and deflections in order to provide allowable loads depending on the depth of the material.

The other structural use of decking is to construct what is called a *diaphragm*, which is a horizontal unit tying the decking to the joists that resists forces parallel to the surface of the diaphragm.

When decking supports a concrete topping or floor, the steel-concrete construction is called *composite*.

Example 1 (pg 290)

**Example 2.** A simple beam consisting of a W 21 × 57 is subjected to bending. Find the limiting moments (a) based on elastic stress conditions and a limiting stress of  $F_y = 36$  ksi, and (b) based on full development of the plastic moment.

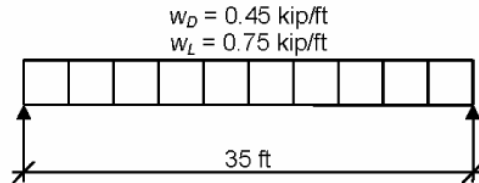
Example 2 (pg 300)

**Example 7.** Design a simply supported floor beam to carry a superimposed load of 2 kips per ft [29.2 kN/m] over a span of 24 ft [7.3 m]. (The term *superimposed load* is used to denote any load other than the weight of a structural member itself.) The superimposed load is 25 percent dead load and 75 percent live load. The yield stress is 36 ksi [250 MPa]. The floor beam is continuously supported along its length against lateral buckling.



Example 3**Given:**

Select an ASTM A992 W-shape beam with a simple span of 35 feet. Limit the member to a maximum nominal depth of 18 in. Limit the live load deflection to  $L/360$ . The nominal loads are a uniform dead load of 0.45 kip/ft and a uniform live load of 0.75 kip/ft. Assume the beam is continuously braced. Use ASD of the Unified Design method.



*Beam Loading & Bracing Diagram  
(full lateral support)*

**Solution:****Material Properties:**

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

1. The unbraced length is 0 because it says it is fully braced.

2. Find the maximum shear and moment from unfactored loads:  $w_a = 0.450 \text{ k/ft} + 0.750 \text{ k/ft} = 1.20 \text{ k/ft}$

$$V_a = 1.20 \text{ k/ft}(35 \text{ ft})/2 = 21 \text{ k}$$

$$M_a = 1.20 \text{ k/ft}(35 \text{ ft})^2/8 = 184 \text{ k-ft}$$

If  $M_a \leq M_n/\Omega$ , the maximum moment for design is  $M_a\Omega$ :  $M_{\max} = 184 \text{ k-ft}$

3. Find  $Z_{\text{req'd}}$ :

$$Z_{\text{req'd}} \geq M_{\max}/F_b = M_{\max}(\Omega)/F_y = 184 \text{ k-ft}(1.67)(12 \text{ in/ft})/50 \text{ ksi} = 73.75 \text{ in}^3 \quad (F_y \text{ is the limit stress when fully braced})$$

4. Choose a trial section, and also limit the depth to 18 in as instructed:

W18 x 40 has a plastic section modulus of  $78.4 \text{ in}^3$  and is the most light weight (as indicated by the bold text) in Table 9.1

Include the self weight in the maximum values:  $w_{a\text{-adjusted}}^* = 1.20 \text{ k/ft} + 0.04 \text{ k/ft}$

$$V_{a\text{-adjusted}}^* = 1.24 \text{ k/ft}(35 \text{ ft})/2 = 21.7 \text{ k}$$

$$M_{a\text{-adjusted}}^* = 1.24 \text{ k/ft}(35 \text{ ft})^2/8 = 189.9 \text{ k}$$

$Z_{\text{req'd}} \geq 189.9 \text{ k-ft}(1.67)(12 \text{ in/ft})/50 \text{ ksi} = 76.11 \text{ in}^3$  And the Z we have (78.4) is larger than the Z we need (76.11), so OK.

6. Evaluate shear (is  $V_a \leq V_n/\Omega$ ):  $A_w = dt_w$  so look up section properties for W18 x 40:  $d = 17.90 \text{ in}$  and  $t_w = 0.315 \text{ in}$

$$V_n/\Omega = 0.6F_y A_w/\Omega = 0.6(50 \text{ ksi})(17.90 \text{ in})(0.315 \text{ in})/1.5 = 112.8 \text{ k} \text{ which is much larger than } 21.7 \text{ k, so OK.}$$

9. Evaluate the deflection with respect to the limit stated of  $L/360$  for the live load. (If we knew the **total** load limit we would check that as well). The moment of inertia for the W18 x 40 is needed.  $I_x = 612 \text{ in}^4$

$$\Delta_{\text{live load limit}} = 35 \text{ ft}(12 \text{ in/ft})/360 = 1.17 \text{ in}$$

$$\Delta = 5wL^4/384EI = 5(0.75 \text{ k/ft})(35 \text{ ft})^4(12 \text{ in/ft})^3/384(29 \times 10^3 \text{ ksi})(612 \text{ in}^4) = 1.42 \text{ in! This is TOO BIG (not less than the limit.)}$$

Find the moment of inertia needed:

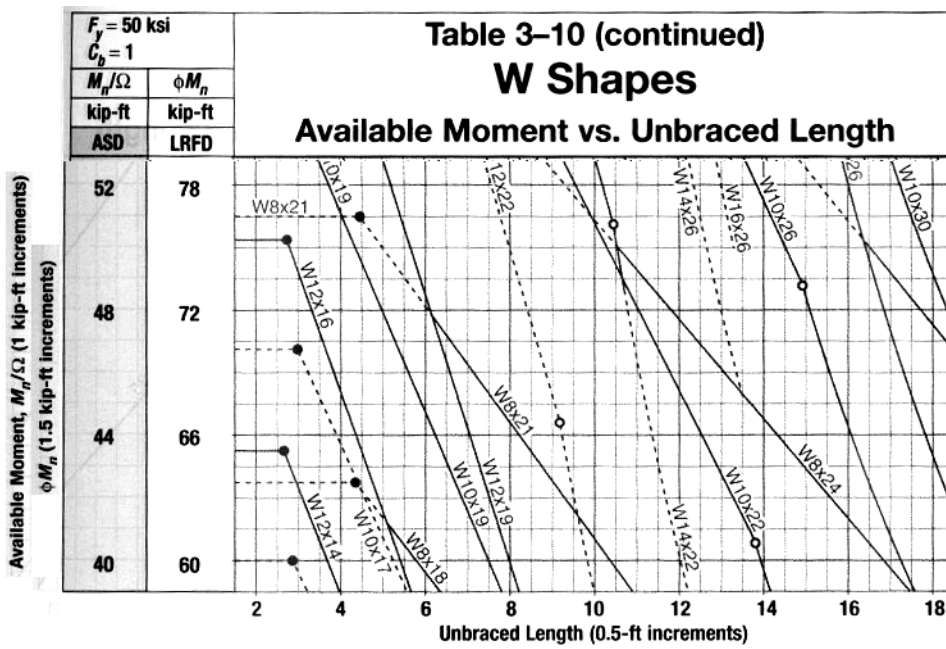
$$I_{\text{req'd}} \geq \Delta_{\text{too big}}(I_{\text{trial}})/\Delta_{\text{limit}} = 1.42 \text{ in}(612 \text{ in}^4)/(1.17 \text{ in}) = 742.8 \text{ in}^4$$

From Table 9.1, a W16 x 45 is larger (by Z), but not the most light weight (efficient), as is W10 x 68, W14 x 53, W18 x 46, (W21 x 44 is too deep) and W18 x 50 is bolded (efficient). (Now look up  $I_x$ 's). (In order:  $I_x = 586, 394, 541, 712$  and  $800 \text{ in}^4$ )

Choose a W18 x 50

**Example 4**

A steel beam with a 20 ft span is designed to be simply supported at the ends on columns and to carry a floor system made with open-web steel joists at 4 ft on center. The joists span 28 feet and frame into the beam *from one side only* and have a self weight of 8.5 lb/ft. Use A992 (grade 50) steel and select the most economical wide-flange section for the beam. Floor loads are 50 psf LL and 14.5 psf DL.



**Example 5**

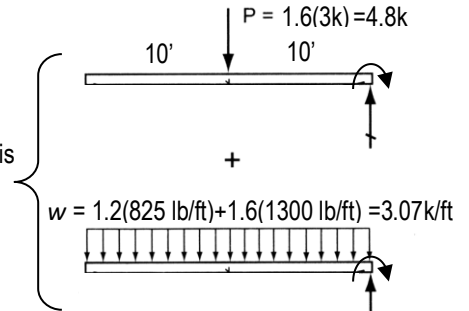
Select a A992 W shape flexural member ( $F_y = 50$  ksi,  $F_u = 65$  ksi) for a beam with distributed loads of 825 lb/ft (dead) and 1300 lb/ft (live) and a live point load at midspan of 3 k using the Available Moment tables. The beam is simply supported, 20 feet long, and braced at the ends and midpoint only ( $L_b = 10$  ft.) The beam is a roof beam for an institution without plaster ceilings. (LRFD)

**SOLUTION:**

To use the Available Moment tables, the maximum moment required is plotted against the unbraced length. The first solid line with capacity or unbraced length *above* what is needed is the most economical.

DESIGN LOADS (load factors applied on figure):

$$M_u = \frac{wl^2}{2} + Pb = \frac{3.07 \text{ k/ft} (20 \text{ ft})^2}{2} + 4.8k(10 \text{ ft}) = 662 \text{ k-ft} \quad V_u = wl + P = 3.07 \text{ k/ft} (20 \text{ ft}) + 4.8k = 66.2k$$



Plotting 662 k-ft vs. 10 ft lands just on the capacity of the W21x83, but it is dashed (and not the most economical) AND we need to consider the contribution of self weight to the total moment. Choose a *trial* section of W24 x 76. Include the new dead load:

$$M_{u-adj}^* = 662 \text{ k-ft} + \frac{1.2(76 \text{ lb/ft})(20 \text{ ft})^2}{2(1000 \text{ lb/k})} = 680.2 \text{ k-ft} \quad V_{u-adj}^* = 66.2k + 1.2(0.076 \text{ k/ft})(20 \text{ ft}) = 68.0k$$

Replot 680.2 k-ft vs. 10ft, which lands *above* the capacity of the W21x83. We can't look up because the chart ends, but we can look for that capacity with a longer unbraced length. This leads us to a **W24 x 84** as the most economical. (With the additional self weight of 84 - 76 lb/ft = 8 lb/ft, the increase in the *factored* moment is only 1.92 k-ft; therefore, it is still OK.)

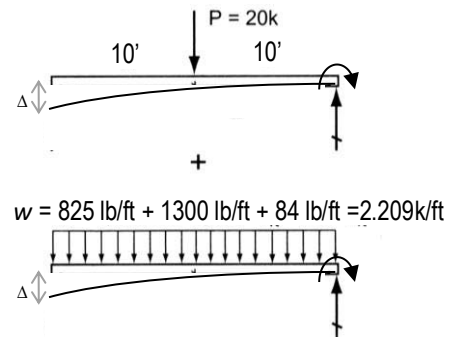
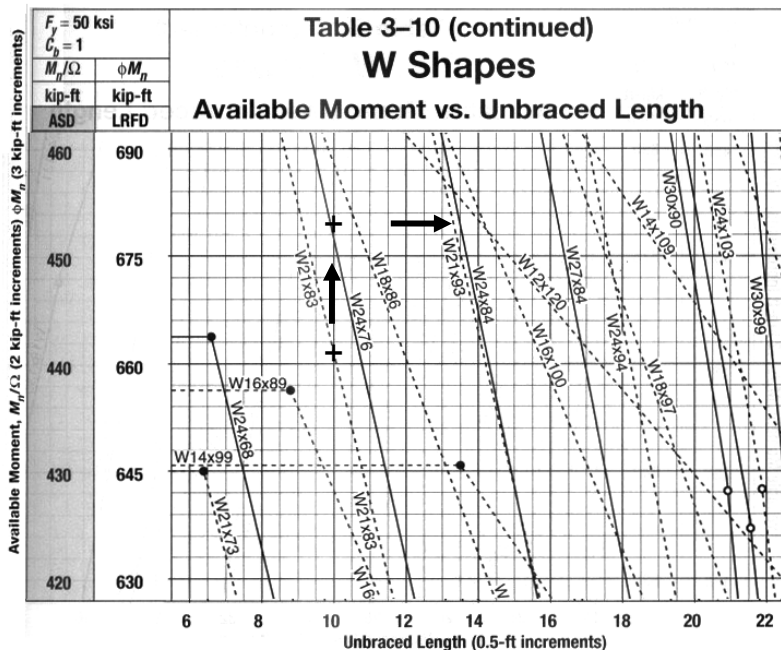
Evaluate the shear capacity:

$$\phi_v V_n = \phi_v 0.6 F_y A_w = 1.0(0.6)50 \text{ ksi}(24.10 \text{ in})0.47 \text{ in} = 338.4k \quad \text{so yes, } 68 \text{ k} \leq 338.4k \quad \text{OK}$$

Evaluate the deflection with respect to the limits of L/240 for live (*unfactored*) load and L/180 for total (*unfactored*) load:

$$L/240 = 1 \text{ in. and } L/180 = 1.33 \text{ in.}$$

$$\Delta_{total} = \frac{Pb^2(3l-b)}{6EI} + \frac{wl^4}{24EI} = \frac{3k(10 \text{ ft})^2(3 \cdot 20 - 10 \text{ ft})(12 \text{ in/ft})^3}{6(30 \times 10^3 \text{ ksi})2370 \text{ in}^3} + \frac{(2.209 \text{ k/ft})(20 \text{ ft})^4(12 \text{ in/ft})^3}{24(30 \times 10^3 \text{ ksi})2370 \text{ in}^3} = 0.06 + 0.36 = 0.42 \text{ in}$$



So,  $\Delta_{LL} \leq \Delta_{LL-limit}$  and  $\Delta_{total} \leq \Delta_{total-limit}$ .

$$0.06 \text{ in.} \leq 1 \text{ in. and } 0.42 \text{ in.} \leq 1.33 \text{ in.}$$

(This section is so big to accommodate the large bending moment at the cantilever support that it deflects very little.)

**∴ FINAL SELECTION IS W24x84**

**Example 6**

A floor is to be supported by trusses spaced at 5 ft. on center and spanning 60 ft. having a dead load of 53 lb/ft<sup>2</sup> and a live load of 100 lb/ft<sup>2</sup>. With 3 ft.-long panel points, the depth is assumed to be 3 ft with a span-to-depth ratio of 20. With 6 ft.-long panel points, the depth is assumed to be 6 ft with a span-to-depth ratio of 10. Determine the maximum force in a horizontal chord and the maximum force in a web member. Use factored loads. Assume a self weight of 40 lb/ft.

**Table 7.2 Computation of Truss Joint Loads**

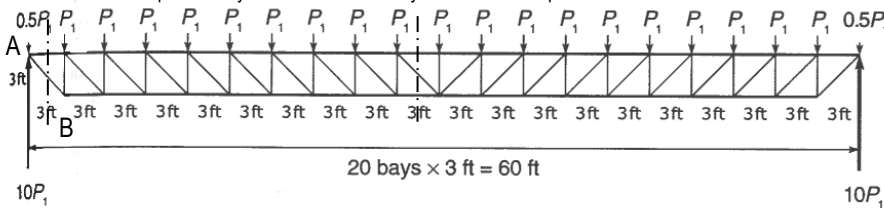
Truss	area loads				tributary widths		Floor Area per Node A (ft <sup>2</sup> )	P <sub>dead</sub> (=W <sub>dead</sub> · A) (K)	P <sub>live</sub> (=W <sub>live</sub> · A) (K)	Factored Dead Load (K)	Factored Live Load (K)	Factored Total Load (K)
	W <sub>dead</sub>		W <sub>live</sub>		Node-to-Node Spacing (ft)	Truss-to-Truss Spacing (ft)						
	(#/ft <sup>2</sup> )	(K/ft <sup>2</sup> )	(#/ft <sup>2</sup> )	(K/ft <sup>2</sup> )								
3 ft deep	53	0.053	100	0.100	3	5	15	0.795	1.50	0.954	2.40	3.35 + 0.14 = 3.49
6 ft deep	53	0.053	100	0.100	6	5	30	1.59	3.00	1.908	4.80	6.71 + 0.29 = 7.00

self weight 0.04 k/ft (distributed)  
 3  
 6

$$1.2P_{dead} = 1.2W_{dead} \cdot \text{tributary width} = 0.14 \text{ K}$$

$$1.2P_{dead} = 1.2W_{dead} \cdot \text{tributary width} = 0.29 \text{ K}$$

NOTE – end panels only have half the tributary width of interior panels

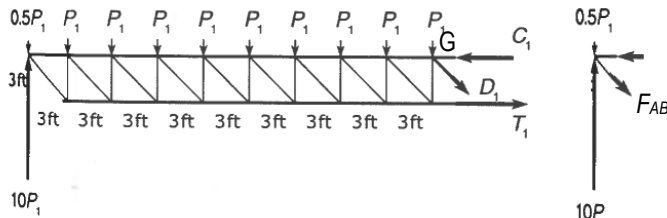


**FBD 3:** Maximum web force will be in the end diagonal (just like maximum shear in a beam)

$$\Sigma F_y = 10P_1 - 0.5P_1 - F_{AB} \sin 45^\circ = 0$$

$$F_{AB} = 9.5P_1 / \sin 45^\circ = 9.5(3.49 \text{ k}) / 0.707 = \underline{46.9 \text{ k}}$$

**FBD 1** for 3 ft deep truss



**FBD 2:** Maximum chord force (top or bottom) will be at midspan

$$\Sigma M_G = 9.5P_1(30^{\text{ft}}) - P_1(27^{\text{ft}}) - P_1(24^{\text{ft}}) - P_1(21^{\text{ft}}) - P_1(18^{\text{ft}}) - P_1(15^{\text{ft}}) - P_1(12^{\text{ft}}) - P_1(9^{\text{ft}}) - P_1(6^{\text{ft}}) - P_1(3^{\text{ft}}) - T_1(3^{\text{ft}}) = 0$$

$$T_1 = P_1(150^{\text{ft}}) / 3^{\text{ft}} = (3.49 \text{ k})(50) = 174.5 \text{ k}$$

$$\Sigma F_y = 10P_1 - 9.5P_1 - D_1 \sin 45^\circ = 0$$

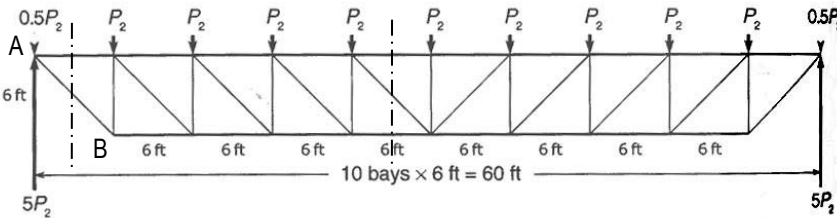
$$D_1 = 0.5(3.49 \text{ k}) / 0.707 = 2.5 \text{ k (minimum near midspan)}$$

**FBD 2** of cut just to the left of midspan

**FBD 3** of cut just to right of left support

$$\Sigma F_x = -C_1 + T_1 + D_1 \cos 45^\circ = 0$$

$$C_1 = \underline{176.2 \text{ k}}$$

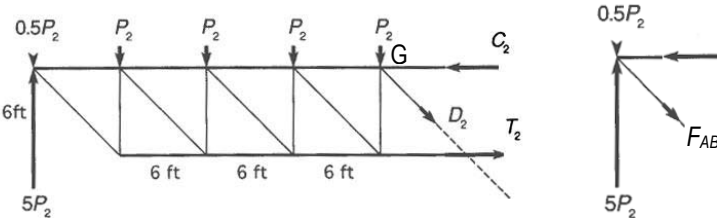


**FBD 4** for 6 ft deep truss

**FBD 6:** Maximum web force will be in the end diagonal

$$\Sigma F_y = 5P_2 - 0.5P_2 - F_{AB} \sin 45^\circ = 0$$

$$F_{AB} = 4.5P_2 / \sin 45^\circ = 4.5(7 \text{ k}) / 0.707 = \underline{44.5 \text{ k}}$$



**FBD 5** of cut just to the left of midspan

**FBD 6** of cut just to right of left support

**FBD 5:** Maximum chord (top or bottom) force will be at midspan

$$\Sigma M_G = 4.5P_2(30^{\text{ft}}) - P_2(24^{\text{ft}}) - P_2(18^{\text{ft}}) - P_2(12^{\text{ft}}) - P_2(6^{\text{ft}}) - T_2(6^{\text{ft}}) = 0$$

$$T_2 = P_2(75^{\text{ft}}) / 6^{\text{ft}} = (7 \text{ k})(12.5) = 87.5 \text{ k}$$

$$\Sigma F_y = 5P_2 - 4.5P_2 - D_2 \sin 45^\circ = 0$$

$$D_2 = 0.5(7 \text{ k}) / 0.707 = 4.9 \text{ k (minimum near midspan)}$$

$$\Sigma F_x = -C_2 + T_2 + D_2 \cos 45^\circ = 0$$

$$C_2 = \underline{92.4 \text{ k}}$$

Example 7 (pg 339)

**Example 14.** Open web steel joists are to be used for a floor with a unit live load of 75 psf [3.59 kN/m<sup>2</sup> and a unit dead load of 40 psf [1.91 kN/m<sup>2</sup> (not including the joist weight) on a span of 30 ft [9.15 m]. Joists

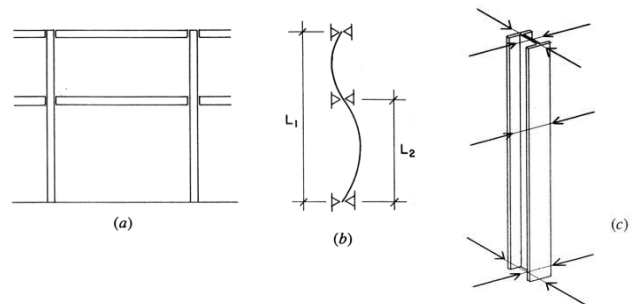
are 2 ft [0.61 m] on center, and deflection is limited to  $\frac{1}{240}$  of the span under total load and  $\frac{1}{360}$  of the span under live load only. Determine the lightest possible joist and the lightest joist of least depth possible. <sup>(from unfactored loads)</sup>

**TABLE 9.5 (Continued)**

Joist Designation:	18K3	18K5	18K7	20K3	20K5	20K7	22K4	22K6	22K9
Weight (lb/ft):	6.6	7.7	9.0	6.7	8.2	9.3	8.0	8.8	11.3
Span (ft)									
28	347 (151)	472 (199)	571 (239)	387 (189)	527 (248)	638 (298)	516 (270)	634 (328)	816 (413)
30	301 (123)	409 (161)	497 (194)	337 (153)	457 (201)	555 (242)	448 (219)	550 (266)	738 (349)
32	264 (101)	359 (132)	436 (159)	295 (126)	402 (165)	487 (199)	393 (180)	484 (219)	647 (287)

Example 8 (pg 353)

**Example 3.** Figure 10.5a shows an elevation of the steel framing at the location of an exterior wall. The column is laterally restrained but rotationally free at the top and bottom in both directions. (The end condition is as shown for Case (d) in Figure 10.3.) With respect to the *x*-axis of the section, the column is laterally unbraced for its full height. However, the existence of the horizontal framing in the wall plane provides lateral bracing with respect to the *y*-axis of the section; thus, the buckling of the column in this direction takes the form shown in Figure 10.5b. If the column is a W 12 × 53 of A36 steel,  $L_1$  is 30 ft [9.15 m], and  $L_2$  is 18 ft [5.49 m], what is the maximum factored compression load?



Example 9 (pg 361)

**Example 6.** Using Table 10.4, select a standard weight steel pipe to carry a dead load of 15 kips [67 kN] and a live load of 26 kips [116 kN] if the unbraced height is 12 ft [3.66 m].

Example 10

Investigate the acceptability of a W16 x 67 used as a beam-column under the unfactored loading shown in the figure. It is A992 steel ( $F_y = 50$  ksi). Assume 25% of the load is dead load with 75% live load.

SOLUTION:

DESIGN LOADS (shown on figure):

$$\text{Axial load} = 1.2(0.25)(350k) + 1.6(0.75)(350k) = 525k$$

$$\text{Moment at joint} = 1.2(0.25)(60 \text{ k-ft}) + 1.6(0.75)(60 \text{ k-ft}) = 90 \text{ k-ft}$$

Determine column capacity and fraction to choose the appropriate interaction equation:

$$\frac{kL}{r_x} = \frac{15 \text{ ft}(12 \text{ in/ft})}{6.96 \text{ in}} = 25.9 \quad \text{and} \quad \frac{kL}{r_y} = \frac{15 \text{ ft}(12 \text{ in/ft})}{2.46 \text{ in}} = 73 \quad (\text{governs})$$

$$P_c = \phi_c P_n = \phi_c F_{cr} A_g = (30.5 \text{ ksi})19.7 \text{ in}^2 = 600.85k$$

$$\frac{P_r}{P_c} = \frac{525k}{600.85k} = 0.87 > 0.2 \quad \text{so use} \quad \frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0$$

There is no bending about the y axis, so that term will not have any values.

Determine the bending moment capacity in the x direction:

The unbraced length to use the full plastic moment ( $L_p$ ) is listed as 8.69 ft, and we are over that so of we don't want to determine it from formula, we can find the beam in the Available Moment vs. Unbraced Length tables. The value of  $\phi M_n$  at  $L_b = 15$  ft is 422 k-ft.

Determine the magnification factor when  $M_1 = 0, M_2 = 90$  k-ft:

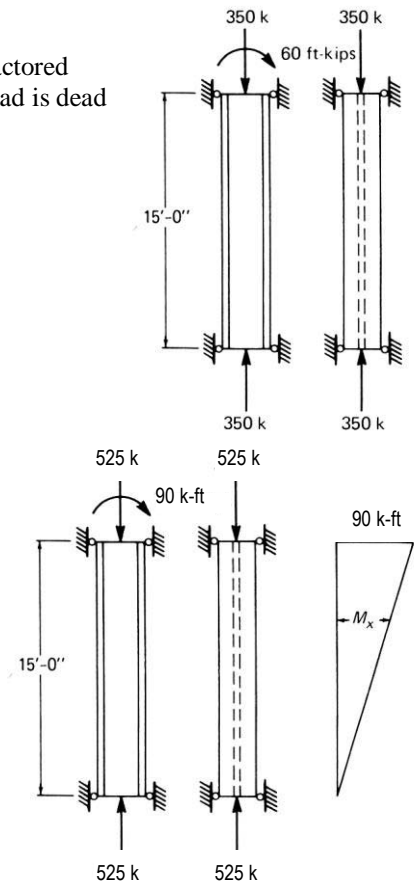
$$C_m = 0.6 - 0.4 \frac{M_1}{M_2} = 0.6 - \frac{0^{k-ft}}{90^{k-ft}} = 0.6 \leq 1.0 \quad P_{e1} = \frac{\pi^2 EA}{(Kl/r)^2} = \frac{\pi^2 (30 \times 10^3 \text{ ksi})19.7 \text{ in}^2}{(25.9)^2} = 8,695.4k$$

$$B_1 = \frac{C_m}{1 - (P_u/P_{e1})} = \frac{0.6}{1 - (525k/8695.4k)} = 0.64 \geq 1.0 \quad \text{USE 1.0} \quad M_u = (1)90 \text{ k-ft}$$

Finally, determine the interaction value:

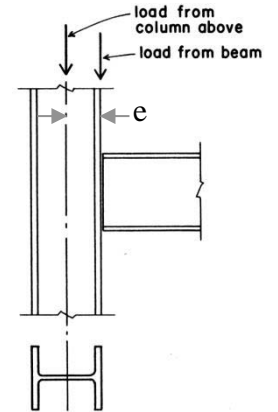
$$\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) = 0.87 + \frac{8}{9} \left( \frac{90^{k-ft}}{422^{k-ft}} \right) = 1.06 \leq 1.0$$

This is **NOT OK.** (and outside error tolerance). The section should be larger.



Example 11 (pg 371)

**Example 7.** It is desired to use a 10-in. W shape for a column in a situation such as that shown in Figure 10.7. The factored axial load from above on the column is 175 kips [778 kN], and the factored beam load at the column face is 35 kips [156 kN]. The column has an unbraced height of 16 ft [4.88 m] and a  $K$  factor of 1.0. ~~Select a trial section for the column.~~ Evaluate the trial W10x45 chosen in the text of A36 steel with  $d = 10.1$  in and  $\phi_b M_n = 133.4$  k-ft (16 ft unbraced length).



Example 12

**10.5** Using the AISC framed beam connection bolt shear in Table 7-1, determine the shear adequacy of the connection shown in Figure 10.28. What thickness and angle length are required? Also determine the bearing capacity of the wide flange sections.

Factored end beam reaction = 90 k.

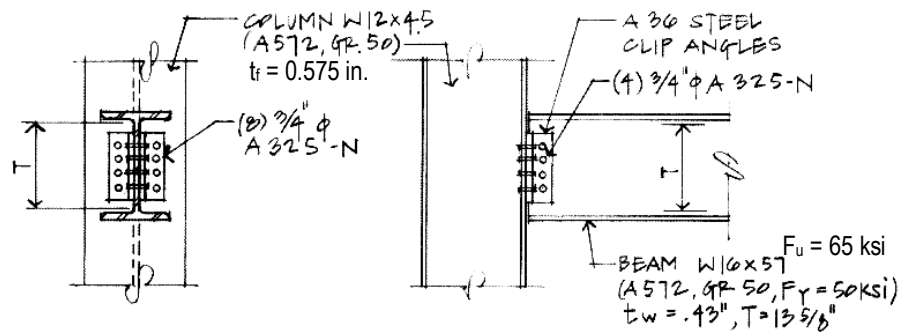
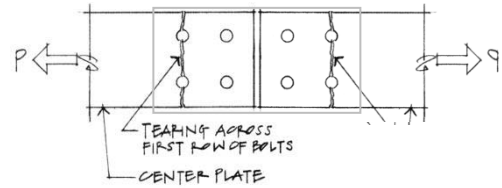


Figure 10.28 Typical beam-column connection.

**Example 13**

**10.2** The butt splice shown in Figure 10.22 uses two 8 × 3/8" plates to "sandwich" in the 8 × 1/2" plates being joined. Four 7/8"φ A325-SC bolts are used on both sides of the splice. Assuming A36 steel and standard round holes, determine the allowable capacity of the connection.



**SOLUTION:**

Shear, bearing and net tension will be checked to determine the critical conditions that governs the capacity of the connection. (The edge distance to the holes is presumed to be adequate.)

*Shear:* Using the AISC available shear in Table 7-3 (Group A):

$$\phi R_n = 26.4 \text{ k/bolt} \times 4 \text{ bolts} = 105.6 \text{ k}$$

*Bearing:* Using the AISC available bearing in Table 7-4:

There are 4 bolts bearing on the center (1/2") plate, while there are 4 bolts bearing on a total width of two sandwich plates (3/4" total). The thinner bearing width will govern. Assume 3 in. spacing (center to center) of bolts. For A36 steel,  $F_u = 58 \text{ ksi}$ .

$$\phi R_n = 91.4 \text{ k/bolt/in.} \times 0.5 \text{ in.} \times 4 \text{ bolts} = 182.8 \text{ k}$$

*Tension:* The center plate is critical, again, because its thickness is less than the combined thicknesses of the two outer plates. We must consider tension yielding and tension rupture:

$$\phi R_n = \phi F_y A_g \quad \text{and} \quad \phi R_n = \phi F_u A_e \quad \text{where} \quad A_e = A_{net} U$$

$$A_g = 8 \text{ in.} \times 1/2 \text{ in.} = 4 \text{ in}^2$$

The holes are considered 1/8 in. larger than the nominal bolt diameter =  $7/8 + 1/8 = 1 \text{ in.}$

$$A_n = (8 \text{ in.} - 2 \text{ holes} \times 1 \text{ in.}) \times 1/2 \text{ in.} = 3 \text{ in}^2$$

The whole cross section sees tension, so the shear lag factor  $U = 1$

$$\phi F_y A_g = 0.9 \times 36 \text{ ksi} \times 4 \text{ in}^2 = 129.6 \text{ k}$$

$$\phi F_u A_e = 0.75 \times 58 \text{ ksi} \times (1) \times 3 \text{ in}^2 = 130.5 \text{ k}$$

*Block Shear Rupture:* It is possible for the center plate to rip away from the sandwich plates leaving the block (shown hatched) behind:

$$\phi R_n = \phi(0.6 F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt})$$

where  $A_{nv}$  is the area resisting shear,  $A_{nt}$  is the area resisting tension,  $A_{gv}$  is the gross area resisting shear, and  $U_{bs} = 1$  when the tensile stress is uniform.

$$A_{gv} = (4 + 2 \text{ in.}) \times 1/2 \text{ in.} = 3 \text{ in}^2$$

$$A_{nv} = A_{gv} - 1 \text{ } 1/2 \text{ holes area} = 3 \text{ in}^2 - 1.5 \times 1 \text{ in.} \times 1/2 \text{ in.} = 2.25 \text{ in}^2$$

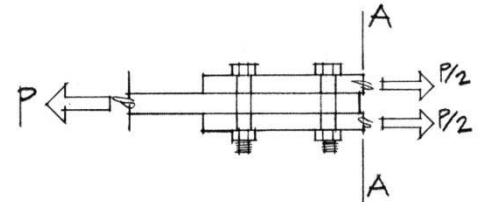
$$A_{nt} = 3.5 \text{ in.} \times t - 1 \text{ holes} = 3.5 \text{ in.} \times 1/2 \text{ in.} - 1 \times 1 \text{ in.} \times 1/2 \text{ in.} = 1.25 \text{ in}^2$$

$$\phi(0.6 F_u A_{nv} + U_{bs} F_u A_{nt}) = 0.75 \times (0.6 \times 58 \text{ ksi} \times 2.25 \text{ in}^2 + 1 \times 58 \text{ ksi} \times 1.25 \text{ in}^2) = 113.1 \text{ k}$$

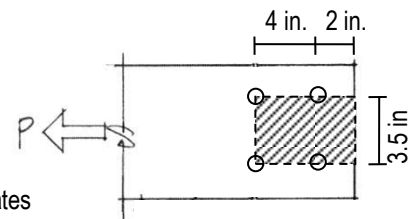
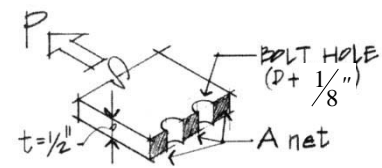
$$\phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt}) = 0.75 \times (0.6 \times 36 \text{ ksi} \times 3 \text{ in}^2 + 1 \times 58 \text{ ksi} \times 1.25 \text{ in}^2) = 103.0 \text{ k}$$

The maximum connection capacity is governed by block shear rupture.

$$\phi R_n = 103.0 \text{ k}$$



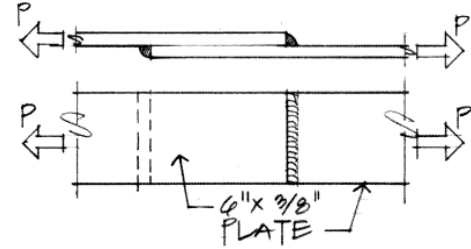
SECTION OUT A-A





Example 14

10.9 Determine the maximum load carrying capacity of this lap joint., assuming A36 steel with E60XX electrodes.

Example 15

10.7 Determine the capacity of the connection in Figure 10.44 assuming A36 steel with E70XX electrodes.

Solution:

Capacity of weld:

For a  $\frac{5}{16}$ " fillet weld,  $\phi S = 6.96$  k/in

Weld length = 22"

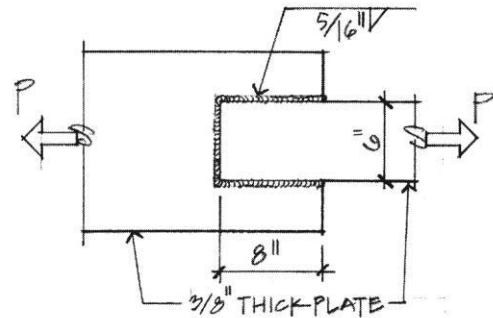
Weld capacity =  $22" \times 6.96$  k/in = 153.1 k

Capacity of plate:  $0.9 \times 36$  k/in<sup>2</sup>  $\times$   $\frac{3}{8}" \times 6"$  = 72.9 k

$$\phi P_n = \phi F_y A_g \quad \phi = 0.9$$

Plate capacity =  $0.9 \times 36$  k/in<sup>2</sup>  $\times$   $\frac{3}{8}" \times 6"$  = 72.9 k

$\therefore$  Plate capacity governs,  $P_{\text{allow}} = 72.9$  k



The weld size used is obviously too strong. What size, then, can the weld be reduced to so that the weld strength is more compatible to the plate capacity? To make the weld capacity  $\approx$  plate capacity:

$$22" \times (\text{weld capacity per in.}) = 72.9 \text{ k}$$

$$\text{Weld capacity per inch} = \frac{72.9 \text{ k}}{22 \text{ in.}} = 3.31 \text{ k/in.}$$

From Available Strength table, use  $\frac{3}{16}"$  weld

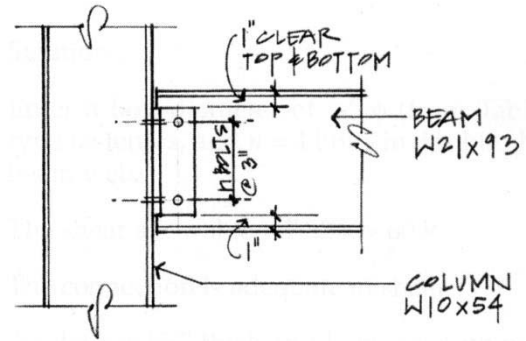
( $\phi S = 4.18$  k/in.)

Minimum size fillet =  $\frac{3}{16}"$  based on a  $\frac{3}{8}"$  thick plate.

**Example 16**

The steel used in the connection and beams is A992 with  $F_y = 50$  ksi, and  $F_u = 65$  ksi. Using A490-N bolt material, determine the maximum capacity of the connection based on shear in the bolts, bearing in all materials and pick the number of bolts and angle length (not staggered). Use A36 steel for the angles.

W21x93:  $d = 21.62$  in,  $t_w = 0.58$  in,  $t_f = 0.93$  in  
 W10x54:  $t_f = 0.615$  in



**SOLUTION:**

The maximum length the angles can depend on how it fits between the top and bottom flange with some clearance allowed for the fillet to the flange, and getting an air wrench in to tighten the bolts. This example uses 1" of clearance:

$$\begin{aligned} \text{Available length} &= \text{beam depth} - \text{both flange thicknesses} - 1" \text{ clearance at top} \& \text{ 1" at bottom} \\ &= 21.62 \text{ in} - 2(0.93 \text{ in}) - 2(1 \text{ in}) = 17.76 \text{ in.} \end{aligned}$$

With the spaced at 3 in. and 1 1/4 in. end lengths (each end), the maximum number of bolts can be determined:

$$\begin{aligned} \text{Available length} &\geq 1.25 \text{ in.} + 1.25 \text{ in.} + 3 \text{ in.} \times (\text{number of bolts} - 1) \\ \text{number of bolts} &\leq (17.76 \text{ in} - 2.5 \text{ in.} - (-3 \text{ in.}))/3 \text{ in.} = 6.1, \text{ so 6 bolts.} \end{aligned}$$

It is helpful to have the All-bolted Double-Angle Connection Tables 10-1. They are available for 3/4", 7/8", and 1" bolt diameters and list angle thicknesses of 1/4", 5/16", 3/8", and 1/2". Increasing the angle thickness is likely to increase the angle strength, although the limit states include shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Beam	Angle	Table 10-1 (continued) All-Bolted Double-Angle Connections										7/8-in. Bolts	
		Bolt and Angle Available Strength, kips											
6 Rows W40, 36, 33, 30, 27, 24, 21		Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in.								
					1/4		5/16		3/8		1/2		
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD		
	Group A	N	STD	98.6	148	123	185	148	222	195	292		
		X	STD	98.6	148	123	185	148	222	197	296		
		SC Class A	STD	98.6	148	106	159	106	159	106	159	106	159
			OVS	90.1	135	90.1	135	90.1	135	90.1	135	90.1	135
		SC Class B	STD	98.6	148	123	185	148	222	176	264	264	
			OVS	93.5	140	117	175	140	210	150	225	225	
	Group B	N	STD	98.6	148	123	185	148	222	197	296		
		X	STD	98.6	148	123	185	148	222	197	296		
		SC Class A	STD	98.6	148	123	185	133	199	133	199	199	
			OVS	93.5	140	113	169	113	169	113	169	169	
		SC Class B	STD	98.6	148	123	185	148	222	197	296	296	
			OVS	93.5	140	117	175	140	210	187	281	281	
		SSLT	97.3	146	122	182	146	219	195	292			

For these diameters, the available shear (double) from Table 7-1 for 6 bolts is (6)45.1 k/bolt = 270.6 kips, (6)61.3 k/bolt = 367.8 kips, and (6)80.1 k/bolt = 480.6 kips.

Tables 10-1 (not all provided here) list a bolt and angle available strength of 271 kips for the  $\frac{3}{4}$ " bolts, 296 kips for the  $\frac{7}{8}$ " bolts, and 281 kips for the 1" bolts. It appears that increasing the bolt diameter to 1" will not gain additional load. Use  $\frac{7}{8}$ " bolts.

$$\phi R_n = 367.8 \text{ kips for double shear of } \frac{7}{8} \text{ bolts} \qquad \phi R_n = 296 \text{ kips for limit state in angles}$$

We also need to evaluate **bearing** of bolts on the beam web, and column flange where there are bolt holes. Table 7-4 provides available bearing strength for the material type, bolt diameter, hole type, and spacing per inch of material thicknesses.

- a) Bearing for beam web: There are 6 bolt holes through the beam web. This is typically the critical bearing limit value because there are two angle legs that resist bolt bearing and twice as many bolt holes to the column. The material is A992 ( $F_u = 65$  ksi), 0.58" thick, with  $\frac{7}{8}$ " bolt diameters at 3 in. spacing.

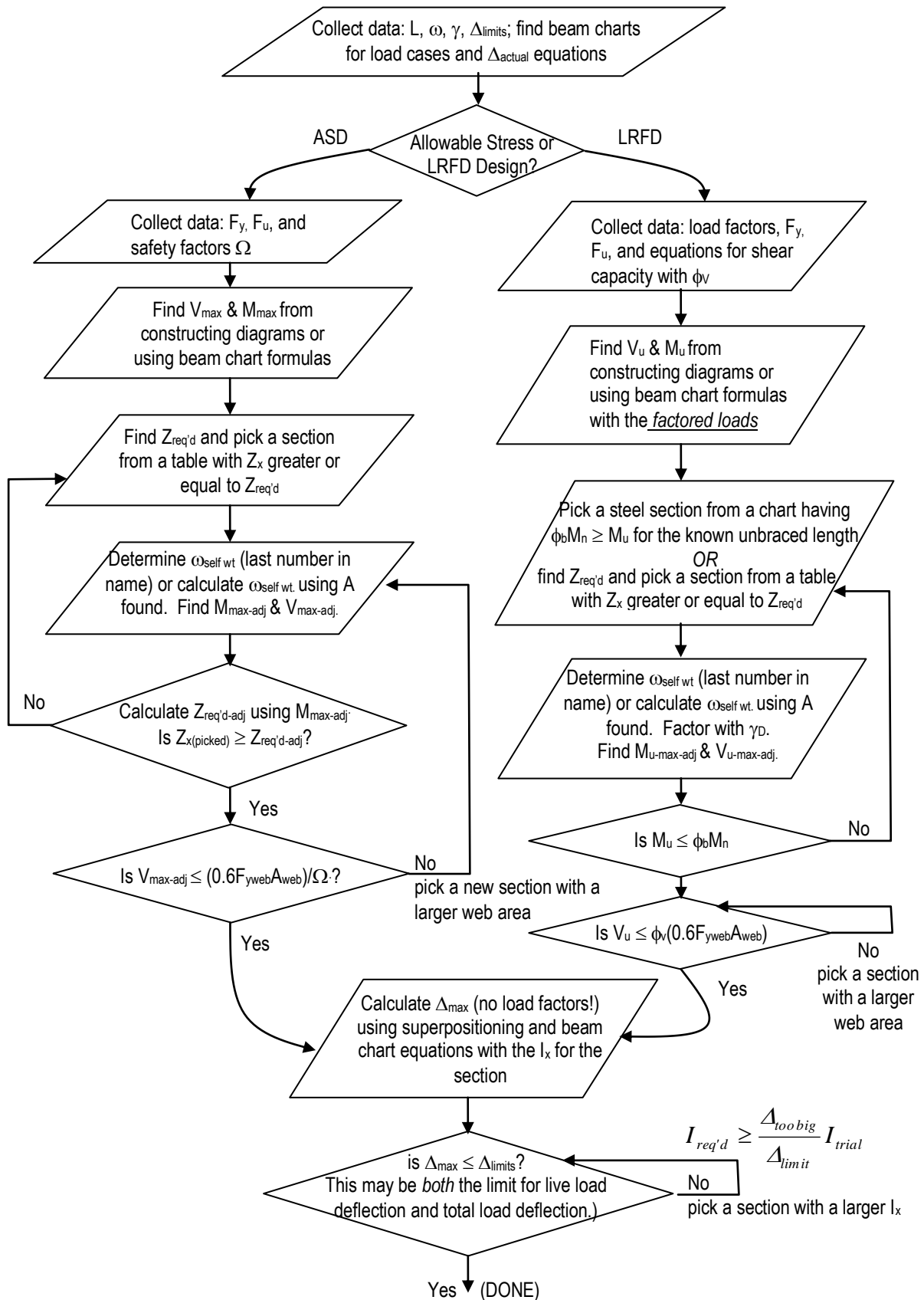
$$\phi R_n = 6 \text{ bolts} \cdot (102 \text{ k/bolt/inch}) \cdot (0.58 \text{ in}) = 355.0 \text{ kips}$$

- b) Bearing for column flange: There are 12 bolt holes through the column. The material is A992 ( $F_u = 65$  ksi), 0.615" thick, with 1" bolt diameters.

$$\phi R_n = 12 \text{ bolts} \cdot (102 \text{ k/bolt/inch}) \cdot (0.615 \text{ in}) = 752.8 \text{ kips}$$

Although, the bearing in the beam web is the smallest at 355 kips, with the shear on the bolts even smaller at 324.6 kips, the maximum capacity for the simple-shear connector is 296 kips limited by the critical capacity of the angles.

### Beam Design Flow Chart



Listing of W Shapes in Descending order of  $Z_x$  for Beam Design

$Z_x - US$ (in. <sup>3</sup> )	$I_x - US$ (in. <sup>4</sup> )	Section	$I_x - SI$ (10 <sup>6</sup> mm. <sup>4</sup> )	$Z_x - SI$ (10 <sup>3</sup> mm. <sup>3</sup> )	$Z_x - US$ (in. <sup>3</sup> )	$I_x - US$ (in. <sup>4</sup> )	Section	$I_x - SI$ (10 <sup>6</sup> mm. <sup>4</sup> )	$Z_x - SI$ (10 <sup>3</sup> mm. <sup>3</sup> )
<b>514</b>	<b>7450</b>	<b>W33X141</b>	<b>3100</b>	<b>8420</b>	289	3100	W24X104	1290	4740
511	5680	W24X176	2360	8370	287	1900	W14X159	791	4700
<b>509</b>	<b>7800</b>	<b>W36X135</b>	<b>3250</b>	<b>8340</b>	<b>283</b>	<b>3610</b>	<b>W30X90</b>	<b>1500</b>	<b>4640</b>
500	6680	W30X148	2780	8190	280	3000	W24X103	1250	4590
490	4330	W18X211	1800	8030	279	2670	W21X111	1110	4570
487	3400	W14X257	1420	7980	278	3270	W27X94	1360	4560
481	3110	W12X279	1290	7880	275	1650	W12X170	687	4510
476	4730	W21X182	1970	7800	262	2190	W18X119	912	4290
468	5170	W24X162	2150	7670	260	1710	W14X145	712	4260
<b>467</b>	<b>6710</b>	<b>W33X130</b>	<b>2790</b>	<b>7650</b>	254	2700	W24X94	1120	4160
464	5660	W27X146	2360	7600	253	2420	W21X101	1010	4150
442	3870	W18X192	1610	7240	<b>244</b>	<b>2850</b>	<b>W27X84</b>	<b>1190</b>	<b>4000</b>
437	5770	W30X132	2400	7160	243	1430	W12X152	595	3980
436	3010	W14X233	1250	7140	234	1530	W14X132	637	3830
432	4280	W21X166	1780	7080	230	1910	W18X106	795	3770
428	2720	W12X252	1130	7010	<b>224</b>	<b>2370</b>	<b>W24X84</b>	<b>986</b>	<b>3670</b>
418	4580	W24X146	1910	6850	221	2070	W21X93	862	3620
<b>415</b>	<b>5900</b>	<b>W33X118</b>	<b>2460</b>	<b>6800</b>	214	1240	W12X136	516	3510
408	5360	W30X124	2230	6690	212	1380	W14X120	574	3470
398	3450	W18X175	1440	6520	211	1750	W18X97	728	3460
395	4760	W27X129	1980	6470	<b>200</b>	<b>2100</b>	<b>W24X76</b>	<b>874</b>	<b>3280</b>
390	2660	W14X211	1110	6390	198	1490	W16X100	620	3240
386	2420	W12X230	1010	6330	196	1830	W21X83	762	3210
<b>378</b>	<b>4930</b>	<b>W30X116</b>	<b>2050</b>	<b>6190</b>	192	1240	W14X109	516	3150
373	3630	W21X147	1510	6110	186	1530	W18X86	637	3050
370	4020	W24X131	1670	6060	186	1070	W12X120	445	3050
356	3060	W18X158	1270	5830	<b>177</b>	<b>1830</b>	<b>W24X68</b>	<b>762</b>	<b>2900</b>
355	2400	W14X193	999	5820	175	1300	W16X89	541	2870
348	2140	W12X210	891	5700	173	1110	W14X99	462	2830
<b>346</b>	<b>4470</b>	<b>W30X108</b>	<b>1860</b>	<b>5670</b>	172	1600	W21X73	666	2820
343	4080	W27X114	1700	5620	164	933	W12X106	388	2690
333	3220	W21X132	1340	5460	163	1330	W18X76	554	2670
327	3540	W24X117	1470	5360	<b>160</b>	<b>1480</b>	<b>W21X68</b>	<b>616</b>	<b>2620</b>
322	2750	W18X143	1140	5280	157	999	W14X90	416	2570
320	2140	W14X176	891	5240	<b>153</b>	<b>1550</b>	<b>W24X62</b>	<b>645</b>	<b>2510</b>
<b>312</b>	<b>3990</b>	<b>W30X99</b>	<b>1660</b>	<b>5110</b>	150	1110	W16X77	462	2460
311	1890	W12X190	787	5100	147	833	W12X96	347	2410
307	2960	W21X122	1230	5030	147	716	W10X112	298	2410
305	3620	W27X102	1510	5000	146	1170	W18X71	487	2390
290	2460	W18X130	1020	4750					

(continued)

Listing of W Shapes in Descending order of  $Z_x$  for Beam Design (Continued)

$Z_x$ – US (in. <sup>3</sup> )	$I_x$ – US (in. <sup>4</sup> )	Section	$I_x$ – SI (10 <sup>6</sup> mm. <sup>4</sup> )	$Z_x$ – SI (10 <sup>3</sup> mm.3)	$Z_x$ – US (in. <sup>3</sup> )	$I_x$ – US (in. <sup>4</sup> )	Section	$I_x$ – SI (10 <sup>6</sup> mm. <sup>4</sup> )	$Z_x$ – SI (10 <sup>3</sup> mm.3)
<b>144</b>	<b>1330</b>	<b>W21X62</b>	<b>554</b>	<b>2360</b>	<b>66.5</b>	<b>510</b>	<b>W18X35</b>	<b>212</b>	<b>1090</b>
139	881	W14X82	367	2280	64.2	348	W12X45	145	1050
<b>134</b>	<b>1350</b>	<b>W24X55</b>	<b>562</b>	<b>2200</b>	64.0	448	W16X36	186	1050
133	1070	W18X65	445	2180	61.5	385	W14X38	160	1010
132	740	W12X87	308	2160	60.4	272	W10X49	113	990
130	954	W16X67	397	2130	59.8	228	W8X58	94.9	980
130	623	W10X100	259	2130	57.0	307	W12X40	128	934
129	1170	W21X57	487	2110	54.9	248	W10X45	103	900
<b>126</b>	<b>1140</b>	<b>W21X55</b>	<b>475</b>	<b>2060</b>	<b>54.6</b>	<b>340</b>	<b>W14X34</b>	<b>142</b>	<b>895</b>
126	795	W14X74	331	2060	<b>54.0</b>	<b>375</b>	<b>W16X31</b>	<b>156</b>	<b>885</b>
123	984	W18X60	410	2020	51.2	285	W12X35	119	839
119	662	W12X79	276	1950	49.0	184	W8X48	76.6	803
115	722	W14X68	301	1880	<b>47.3</b>	<b>291</b>	<b>W14X30</b>	<b>121</b>	<b>775</b>
113	534	W10X88	222	1850	46.8	209	W10X39	87.0	767
<b>112</b>	<b>890</b>	<b>W18X55</b>	<b>370</b>	<b>1840</b>	<b>44.2</b>	<b>301</b>	<b>W16X26</b>	<b>125</b>	<b>724</b>
<b>110</b>	<b>984</b>	<b>W21X50</b>	<b>410</b>	<b>1800</b>	43.1	238	W12X30	99.1	706
108	597	W12X72	248	1770	<b>40.2</b>	<b>245</b>	<b>W14X26</b>	<b>102</b>	<b>659</b>
<b>107</b>	<b>959</b>	<b>W21X48</b>	<b>399</b>	<b>1750</b>	39.8	146	W8X40	60.8	652
105	758	W16X57	316	1720	38.8	171	W10X33	71.2	636
102	640	W14X61	266	1670	<b>37.2</b>	<b>204</b>	<b>W12X26</b>	<b>84.9</b>	<b>610</b>
101	800	W18X50	333	1660	36.6	170	W10X30	70.8	600
97.6	455	W10X77	189	1600	34.7	127	W8X35	52.9	569
96.8	533	W12X65	222	1590	<b>33.2</b>	<b>199</b>	<b>W14X22</b>	<b>82.8</b>	<b>544</b>
<b>95.4</b>	<b>843</b>	<b>W21X44</b>	<b>351</b>	<b>1560</b>	31.3	144	W10X26	59.9	513
92.0	659	W16X50	274	1510	30.4	110	W8X31	45.8	498
90.7	712	W18X46	296	1490	<b>29.3</b>	<b>156</b>	<b>W12X22</b>	<b>64.9</b>	<b>480</b>
87.1	541	W14X53	225	1430	27.2	98.0	W8X28	40.8	446
86.4	475	W12X58	198	1420	<b>26.0</b>	<b>118</b>	<b>W10X22</b>	<b>49.1</b>	<b>426</b>
85.3	394	W10X68	164	1400	<b>24.7</b>	<b>130</b>	<b>W12X19</b>	<b>54.1</b>	<b>405</b>
82.3	586	W16X45	244	1350	23.1	82.7	W8X24	34.4	379
<b>78.4</b>	<b>612</b>	<b>W18X40</b>	<b>255</b>	<b>1280</b>	<b>21.6</b>	<b>96.3</b>	<b>W10X19</b>	<b>40.1</b>	<b>354</b>
78.4	484	W14X48	201	1280	20.4	75.3	W8X21	31.3	334
77.9	425	W12X53	177	1280	<b>20.1</b>	<b>103</b>	<b>W12x16</b>	<b>42.9</b>	<b>329</b>
74.6	341	W10X60	142	1220	18.7	81.9	W10X17	34.1	306
<b>73.0</b>	<b>518</b>	<b>W16X40</b>	<b>216</b>	<b>1200</b>	<b>17.4</b>	<b>88.6</b>	<b>W12X14</b>	<b>36.9</b>	<b>285</b>
71.9	391	W12X50	163	1180	17.0	61.9	W8X18	25.8	279
70.1	272	W8X67	113	1150	16.0	68.9	W10X15	28.7	262
69.6	428	W14X43	178	1140	13.6	48.0	W8X15	20.0	223
66.6	303	W10X54	126	1090	<b>12.6</b>	<b>53.8</b>	<b>W10X12</b>	<b>22.4</b>	<b>206</b>
					11.4	39.6	W8X13	16.5	187
					<b>8.87</b>	<b>30.8</b>	<b>W8X10</b>	<b>12.8</b>	<b>145</b>

Available Critical Stress,  $\phi_c F_{cr}$ , for Compression Members, ksi ( $F_y = 36$  ksi and  $\phi_c = 0.90$ )

$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$
1	32.4	41	29.7	81	22.9	121	15.0	161	8.72
2	32.4	42	29.5	82	22.7	122	14.8	162	8.61
3	32.4	43	29.4	83	22.5	123	14.6	163	8.50
4	32.4	44	29.3	84	22.3	124	14.4	164	8.40
5	32.4	45	29.1	85	22.1	125	14.2	165	8.30
6	32.3	46	29.0	86	22.0	126	14.0	166	8.20
7	32.3	47	28.8	87	21.8	127	13.9	167	8.10
8	32.3	48	28.7	88	21.6	128	13.7	168	8.00
9	32.3	49	28.6	89	21.4	129	13.5	169	7.91
10	32.2	50	28.4	90	21.2	130	13.3	170	7.82
11	32.2	51	28.3	91	21.0	131	13.1	171	7.73
12	32.2	52	28.1	92	20.8	132	12.9	172	7.64
13	32.1	53	27.9	93	20.5	133	12.8	173	7.55
14	32.1	54	27.8	94	20.3	134	12.6	174	7.46
15	32.0	55	27.6	95	20.1	135	12.4	175	7.38
16	32.0	56	27.5	96	19.9	136	12.2	176	7.29
17	31.9	57	27.3	97	19.7	137	12.0	177	7.21
18	31.9	58	27.1	98	19.5	138	11.9	178	7.13
19	31.8	59	27.0	99	19.3	139	11.7	179	7.05
20	31.7	60	26.8	100	19.1	140	11.5	180	6.97
21	31.7	61	26.6	101	18.9	141	11.4	181	6.90
22	31.6	62	26.5	102	18.7	142	11.2	182	6.82
23	31.5	63	26.3	103	18.5	143	11.0	183	6.75
24	31.4	64	26.1	104	18.3	144	10.9	184	6.67
25	31.4	65	25.9	105	18.1	145	10.7	185	6.60
26	31.3	66	25.8	106	17.9	146	10.6	186	6.53
27	31.2	67	25.6	107	17.7	147	10.5	187	6.46
28	31.1	68	25.4	108	17.5	148	10.3	188	6.39
29	31.0	69	25.2	109	17.3	149	10.2	189	6.32
30	30.9	70	25.0	110	17.1	150	10.0	190	6.26
31	30.8	71	24.8	111	16.9	151	9.91	191	6.19
32	30.7	72	24.7	112	16.7	152	9.78	192	6.13
33	30.6	73	24.5	113	16.5	153	9.65	193	6.06
34	30.5	74	24.3	114	16.3	154	9.53	194	6.00
35	30.4	75	24.1	115	16.2	155	9.40	195	5.94
36	30.3	76	23.9	116	16.0	156	9.28	196	5.88
37	30.1	77	23.7	117	15.8	157	9.17	197	5.82
38	30.0	78	23.5	118	15.6	158	9.05	198	5.76
39	29.9	79	23.3	119	15.4	159	8.94	199	5.70
40	29.8	80	23.1	120	15.2	160	8.82	200	5.65

Available Critical Stress,  $\phi_c F_{cr}$ , for Compression Members, ksi ( $F_y = 50$  ksi and  $\phi_c = 0.90$ )

$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$	$KL/r$	$\phi_c F_{cr}$
1	45.0	41	39.8	81	27.9	121	15.4	161	8.72
2	45.0	42	39.6	82	27.5	122	15.2	162	8.61
3	45.0	43	39.3	83	27.2	123	14.9	163	8.50
4	44.9	44	39.1	84	26.9	124	14.7	164	8.40
5	44.9	45	38.8	85	26.5	125	14.5	165	8.30
6	44.9	46	38.5	86	26.2	126	14.2	166	8.20
7	44.8	47	38.3	87	25.9	127	14.0	167	8.10
8	44.8	48	38.0	88	25.5	128	13.8	168	8.00
9	44.7	49	37.8	89	25.2	129	13.6	169	7.91
10	44.7	50	37.5	90	24.9	130	13.4	170	7.82
11	44.6	51	37.2	91	24.6	131	13.2	171	7.73
12	44.5	52	36.9	92	24.2	132	13.0	172	7.64
13	44.4	53	36.6	93	23.9	133	12.8	173	7.55
14	44.4	54	36.4	94	23.6	134	12.6	174	7.46
15	44.3	55	36.1	95	23.3	135	12.4	175	7.38
16	44.2	56	35.8	96	22.9	136	12.2	176	7.29
17	44.1	57	35.5	97	22.6	137	12.0	177	7.21
18	43.9	58	35.2	98	22.3	138	11.9	178	7.13
19	43.8	59	34.9	99	22.0	139	11.7	179	7.05
20	43.7	60	34.6	100	21.7	140	11.5	180	6.97
21	43.6	61	34.3	101	21.3	141	11.4	181	6.90
22	43.4	62	34.0	102	21.0	142	11.2	182	6.82
23	43.3	63	33.7	103	20.7	143	11.0	183	6.75
24	43.1	64	33.4	104	20.4	144	10.9	184	6.67
25	43.0	65	33.0	105	20.1	145	10.7	185	6.60
26	42.8	66	32.7	106	19.8	146	10.6	186	6.53
27	42.7	67	32.4	107	19.5	147	10.5	187	6.46
28	42.5	68	32.1	108	19.2	148	10.3	188	6.39
29	42.3	69	31.8	109	18.9	149	10.2	189	6.32
30	42.1	70	31.4	110	18.6	150	10.0	190	6.26
31	41.9	71	31.1	111	18.3	151	9.91	191	6.19
32	41.8	72	30.8	112	18.0	152	9.78	192	6.13
33	41.6	73	30.5	113	17.7	153	9.65	193	6.06
34	41.4	74	30.2	114	17.4	154	9.53	194	6.00
35	41.1	75	29.8	115	17.1	155	9.40	195	5.94
36	40.9	76	29.5	116	16.8	156	9.28	196	5.88
37	40.7	77	29.2	117	16.5	157	9.17	197	5.82
38	40.5	78	28.8	118	16.2	158	9.05	198	5.76
39	40.3	79	28.5	119	16.0	159	8.94	199	5.70
40	40.0	80	28.2	120	15.7	160	8.82	200	5.65



### Bolt Strength Tables

**Table 7-1  
Available Shear  
Strength of Bolts, kips**

Nominal Bolt Diameter, <i>d</i> , in.		5/8		3/4		7/8		1				
Nominal Bolt Area, in. <sup>2</sup>		0.307		0.442		0.601		0.785				
ASTM Desig.	Thread Cond.	<i>F<sub>nv</sub></i> /Ω (ksi)	φ <i>F<sub>nv</sub></i> (ksi)	Load-ing	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>		
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD		
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	32.5	48.7	42.4	63.6
Group A	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
Group B	N	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
Group B	X	42.0	63.0	S	12.9	19.3	18.6	27.8	25.2	37.9	33.0	49.5
				D	25.8	38.7	37.1	55.7	50.5	75.7	65.9	98.9
A307	-	13.5	20.3	S	4.14	6.23	5.97	8.97	8.11	12.2	10.6	15.9
				D	8.29	12.5	11.9	17.9	16.2	24.4	21.2	31.9
Nominal Bolt Diameter, <i>d</i> , in.		1 1/8		1 1/4		1 3/8		1 1/2				
Nominal Bolt Area, in. <sup>2</sup>		0.994		1.23		1.48		1.77				
ASTM Desig.	Thread Cond.	<i>F<sub>nv</sub></i> /Ω (ksi)	φ <i>F<sub>nv</sub></i> (ksi)	Load-ing	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>		
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD		
Group A	N	27.0	40.5	S	26.8	40.3	33.2	49.8	40.0	59.9	47.8	71.7
				D	53.7	80.5	66.4	99.6	79.9	120	95.6	143
Group A	X	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
Group B	N	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
Group B	X	42.0	63.0	S	41.7	62.6	51.7	77.5	62.2	93.2	74.3	112
				D	83.5	125	103	155	124	186	149	223
A307	-	13.5	20.3	S	13.4	20.2	16.6	25.0	20.0	30.0	23.9	35.9
				D	26.8	40.4	33.2	49.9	40.0	60.1	47.8	71.9
ASD	LRFD	For end loaded connections greater than 38 in., see AISC Specification Table J3.2 footnote b.										
Ω = 2.00	φ = 0.75											

**Table 7-2  
Available Tensile  
Strength of Bolts, kips**

Nominal Bolt Diameter, <i>d</i> , in.		5/8		3/4		7/8		1		
Nominal Bolt Area, in. <sup>2</sup>		0.307		0.442		0.601		0.785		
ASTM Desig.	<i>F<sub>nt</sub></i> /Ω (ksi)	φ <i>F<sub>nt</sub></i> (ksi)	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	45.0	67.5	13.8	20.7	19.9	29.8	27.1	40.6	35.3	53.0
Group B	56.5	84.8	17.3	26.0	25.0	37.4	34.0	51.0	44.4	66.6
A307	22.5	33.8	6.90	10.4	9.94	14.9	13.5	20.3	17.7	26.5
Nominal Bolt Diameter, <i>d</i> , in.		1 1/8		1 1/4		1 3/8		1 1/2		
Nominal Bolt Area, in. <sup>2</sup>		0.994		1.23		1.48		1.77		
ASTM Desig.	<i>F<sub>nt</sub></i> /Ω (ksi)	φ <i>F<sub>nt</sub></i> (ksi)	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>	<i>r<sub>n</sub></i> /Ω	φ <i>r<sub>n</sub></i>
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	45.0	67.5	44.7	67.1	55.2	82.8	66.8	100	79.5	119
Group B	56.5	84.8	56.2	84.2	69.3	104	83.9	126	99.8	150
A307	22.5	33.8	22.4	33.5	27.6	41.4	33.4	50.1	39.8	59.6
ASD	LRFD									
Ω = 2.00	φ = 0.75									

**Table 7-3 (continued)**  
**Slip-Critical Connections**  
 Available Shear Strength, kips  
 (Class A Faying Surface,  $\mu = 0.30$ )

**Group B Bolts**  
 A490, A490M  
 F2280  
 A354 Grade BD

Hole Type		Group B Bolts											
		Nominal Bolt Diameter, $d$ , in.											
		5/8		3/4		7/8		1		1 1/2		1 3/4	
Loading		Minimum Group B Bolt Pretension, kips											
		24		35		49		64		80		102	
Hole Type	Loading	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$
		STD/SSLT	S	5.42	8.14	7.91	11.9	11.1	16.6	14.5	21.7	11.1	16.6
D	10.8		16.3	15.8	23.7	22.1	33.2	28.9	43.4	22.1	33.2	28.9	43.4
OVS/SSLP	S	4.62	6.92	6.74	10.1	9.44	14.1	12.3	18.4	9.44	14.1	12.3	18.4
	D	9.25	13.8	13.5	20.2	18.9	28.2	24.7	36.9	18.9	28.2	24.7	36.9
LSL	S	3.80	5.70	5.54	8.31	7.76	11.6	10.1	15.2	7.76	11.6	10.1	15.2
	D	7.60	11.4	11.1	16.6	15.5	23.3	20.3	30.4	15.5	23.3	20.3	30.4

STD = standard hole  
 OVS = oversized hole  
 SSLT = short-slotted hole transverse to the line of force  
 SSLP = short-slotted hole parallel to the line of force  
 LSL = long-slotted hole transverse or parallel to the line of force

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers. See ASC Specification Sections J3.8 and J5 for provisions when fillers are present. For Class B faying surfaces, multiply the tabulated available strength by 1.67.

**Table 7-3**  
**Slip-Critical Connections**  
 Available Shear Strength, kips  
 (Class A Faying Surface,  $\mu = 0.30$ )

**Group A Bolts**  
 A325, A325M  
 F1858  
 A354 Grade BC

Hole Type		Group A Bolts											
		Nominal Bolt Diameter, $d$ , in.											
		5/8		3/4		7/8		1		1 1/2		1 3/8	
Loading		Minimum Group A Bolt Pretension, kips											
		19		28		39		51		65		85	
Hole Type	Loading	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$	$r_n/\Omega$	$\phi r_n$
		STD/SSLT	S	4.29	6.44	6.33	9.49	8.81	13.2	11.5	17.3	8.81	13.2
D	8.59		12.9	12.7	19.0	17.6	26.4	23.1	34.6	17.6	26.4	23.1	34.6
OVS/SSLP	S	3.66	5.47	5.39	8.07	7.51	11.2	9.82	14.7	7.51	11.2	9.82	14.7
	D	7.32	10.9	10.8	16.1	15.0	22.5	19.6	29.4	15.0	22.5	19.6	29.4
LSL	S	3.01	4.51	4.44	6.64	6.18	9.25	8.08	12.1	6.18	9.25	8.08	12.1
	D	6.02	9.02	8.87	13.3	12.4	18.5	16.2	24.2	12.4	18.5	16.2	24.2

STD = standard hole  
 OVS = oversized hole  
 SSLT = short-slotted hole transverse to the line of force  
 SSLP = short-slotted hole parallel to the line of force  
 LSL = long-slotted hole transverse or parallel to the line of force

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers. See ASC Specification Sections J3.8 and J5 for provisions when fillers are present. For Class B faying surfaces, multiply the tabulated available strength by 1.67.



Table 7-5 (continued) Available Bearing Strength at Bolt Holes Based on Edge Distance kips/in. thickness

Table with columns for Hole Type, Edge Distance, Fb ksi, and Nominal Bolt Diameter d, in. (1 1/8, 1 1/4, 1 3/8, 1 1/2). Rows include STD, SSLT, SSLP, OVS, LSLP, and LSLT. Values are provided for ASD and LRFD design methods.

STD = standard hole
SSLT = short-slotted hole oriented transverse to the line of force
SSLP = short-slotted hole oriented parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented parallel to the line of force
LSLT = long-slotted hole oriented transverse to the line of force
ASD LRFD
Omega = 2.00 phi = 0.75
Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.
a Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7-5 Available Bearing Strength at Bolt Holes Based on Edge Distance kips/in. thickness

Table with columns for Hole Type, Edge Distance, Fb ksi, and Nominal Bolt Diameter d, in. (5/8, 3/4, 7/8, 1). Rows include STD, SSLT, SSLP, OVS, LSLP, and LSLT. Values are provided for ASD and LRFD design methods.

STD = standard hole
SSLT = short-slotted hole oriented transverse to the line of force
SSLP = short-slotted hole oriented parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented parallel to the line of force
LSLT = long-slotted hole oriented transverse to the line of force
ASD LRFD
Omega = 2.00 phi = 0.75
Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.
a Decimal value has been rounded to the nearest sixteenth of an inch.