Steel Design

Notation:

a = name for width dimension
A = name for area
A_b = area of a bolt
A_e = effective net area found from the product of the net area A_n by the shear lag factor U
A_g = gross area, equal to the total area ignoring any holes
A_{gv} = gross area subjected to shear for block shear rupture
A_n = net area, equal to the gross area subtracting any holes, as is A_{net}
A_{nt} = net area subjected to tension for block shear rupture
A_{nv} = net area subjected to shear for block shear rupture
A_w = area of the web of a wide flange section
$AISC$= American Institute of Steel Construction
ASD = allowable stress design
b = name for a (base) width
= total width of material at a horizontal section
= name for height dimension
b_f = width of the flange of a steel beam cross section
B_1 = factor for determining M_u for combined bending and compression
c = largest distance from the neutral axis to the top or bottom edge of a beam
c_1 = coefficient for shear stress for a rectangular bar in torsion
C_b = modification factor for moment in ASD & LRFD steel beam design
C_c = column slenderness classification constant for steel column design
C_m = modification factor accounting for combined stress in steel design
C_v = web shear coefficient
d = calculus symbol for differentiation
= depth of a wide flange section
= nominal bolt diameter
d_b = nominal bolt diameter
D = shorthand for dead load
DL = shorthand for dead load
e = eccentricity
E = shorthand for earthquake load
= modulus of elasticity
f_c = axial compressive stress
f_b = bending stress
f_p = bearing stress
f_v = shear stress
$f_{v\text{-}max}$ = maximum shear stress
f_y = yield stress
F = shorthand for fluid load
F_a = allowable axial (compressive) stress
F_b = allowable bending stress
F_c = critical unfactored compressive stress for buckling in LRFD
F_{cr} = flexural buckling stress
F_e = elastic critical buckling stress
F_{EXX} = yield strength of weld material
F_n = nominal strength in LRFD
= nominal tension or shear strength of a bolt
F_p = allowable bearing stress
F_t = allowable tensile stress
F_u = ultimate stress prior to failure
F_v = allowable shear stress
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
K = effective length factor for columns, as is k
l = name for length
L = name for length or span length
= shorthand for live load
L_b = unbraced length of a steel beam
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
L_e = effective length that can buckle for column design, as is ℓ_e
L_r = shorthand for live roof load
= maximum unbraced length of a steel beam in LRFD design for inelastic lateral-torsional buckling
L_p = maximum unbraced length of a steel beam in LRFD design for full plastic flexural strength
L' = length of an angle in a connector with staggered holes
LL = shorthand for live load
$LRFD$ = load and resistance factor design
M = internal bending moment
M_a = required bending moment (ASD)
M_n = nominal flexure strength with the full section at the yield stress for LRFD beam design
M_{max} = maximum internal bending moment
$M_{max\text{-}adj}$ = maximum bending moment adjusted to include self weight
M_p = internal bending moment when all fibers in a cross section reach the yield stress
M_u = maximum moment from factored loads for LRFD beam design
M_y = internal bending moment when the extreme fibers in a cross section reach the yield stress
n = number of bolts
$n.a.$ = shorthand for neutral axis
N = bearing length on a wide flange steel section
= bearing type connection with threads included in shear plane
p = bolt hole spacing (pitch)
P = name for load or axial force vector
P_a = required axial force (ASD)
P_c = available axial strength
P_{e1} = Euler buckling strength
P_n = nominal column load capacity in steel design
P_r = required axial force
P_u = factored column load calculated from load factors in LRFD steel design
Q = first moment area about a neutral axis
= generic axial load quantity for LRFD design
r = radius of gyration
r_y = radius of gyration with respect to a y-axis
R = generic load quantity (force, shear, moment, etc.) for LRFD design
= shorthand for rain or ice load
= radius of curvature of a deformed beam
R_a = required strength (ASD)
R_n = nominal value (capacity) to be multiplied by ϕ in LRFD and divided by the safety factor Ω in ASD
R_u = factored design value for LRFD design
s = longitudinal center-to-center spacing of any two consecutive holes
S = shorthand for snow load
= section modulus
= allowable strength per length of a weld for a given size
$S_{req'd}$ = section modulus required at allowable stress
$S_{req'd\text{-}adj}$ = section modulus required at allowable stress when moment is adjusted to include self weight
SC = slip critical bolted connection
t = thickness of the connected material
t_f = thickness of flange of wide flange

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

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 13th 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.

<u>Materials</u>

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 ksi, F_u = 58 ksi, E = 29,000 ksi
A572 – high strength low-alloy used for some beams F_y = 60 ksi, F_u = 75 ksi, E = 30,000 ksi
A992 – for building framing used for most beams F_y = 50 ksi, F_u = 65 ksi, E = 30,000 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
Ω = safety factor

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

bending (braced, $L_b < L_p$)	Ω = 1.67
bending (unbraced, $L_p < L_b$ and $L_b > L_r$)	Ω = 1.67 (nominal moment reduces)
shear (beams)	Ω = 1.5 or 1.67
shear (bolts)	Ω = 2.00 (tabular nominal strength)
shear (welds)	Ω = 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 $R_u \leq \phi R_n$ $where \cdots R_u = \Sigma \gamma_i R_i$

where ϕ = resistance factor
γ = 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, ϕR_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:

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

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:

$$V = \Sigma(-w)dx \qquad \frac{dV}{dx} = -w \qquad \frac{dM}{dx} = V$$
$$M = \Sigma(V)dx$$

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, θ, 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.

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

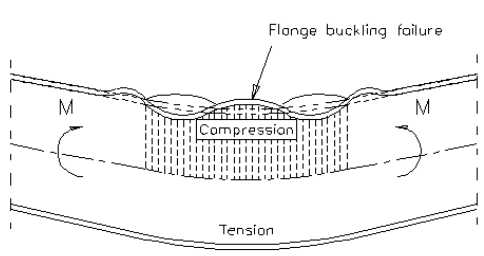
Use	LL only	DL+LL
Roof beams:		
Industrial	L/180	L/120
Commercial		
plaster ceiling	L/240	L/180
no plaster	L/360	L/240
Floor beams:		
Ordinary Usage	L/360	L/240
Roof or floor (damageable elements)		L/480

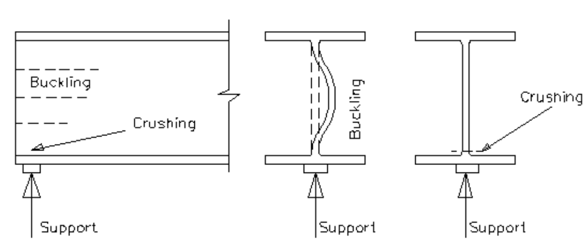
Lateral Buckling

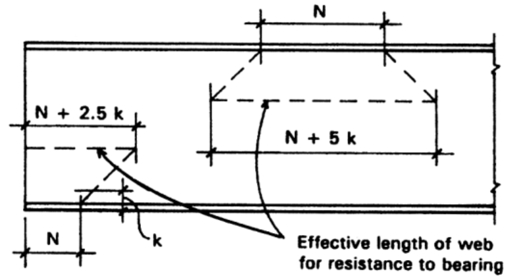
With compression stresses in the top of a beam, a sudden "popping" or buckling can happen even at low stresses. In order to prevent it, we need to brace it along the top, or laterally brace it, or provide a bigger I_y.

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:

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

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

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

ϕ = 1.00 (LRFD) Ω = 1.50 (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

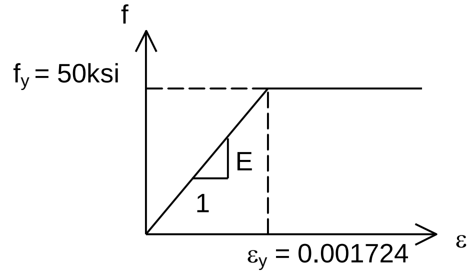
For determining the flexural design strength, $\phi_b M_n$, for resistance to pure bending (no axial load) in most flexural members where the following conditions exist, a single calculation will suffice:

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

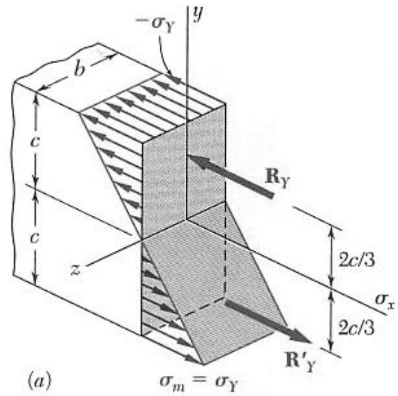
where M_u = maximum moment from factored loads
ϕ_b = resistance factor for bending = 0.9
M_n = nominal moment (ultimate capacity)
F_y = yield strength of the steel
Z = plastic section modulus

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

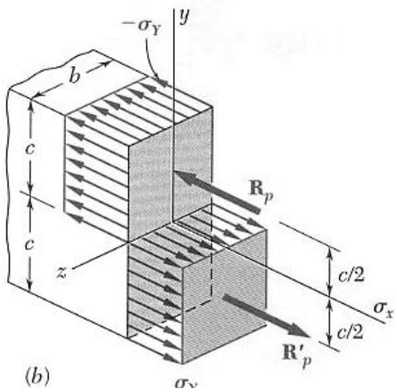


Plastic hinges can develop when all of the material in a cross section sees the yield stress. Because all the material at that section can strain without any additional load, the member segments on either side of the hinge can rotate, possibly causing instability.

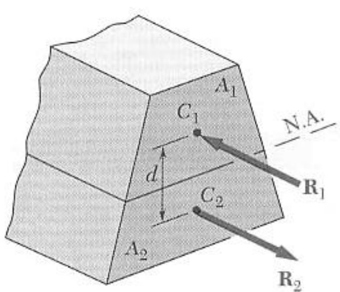
For a rectangular section:

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: $$M_{ult} \text{ or } M_p = bc^2 f_y = {}^3\!/_2 M_y$$

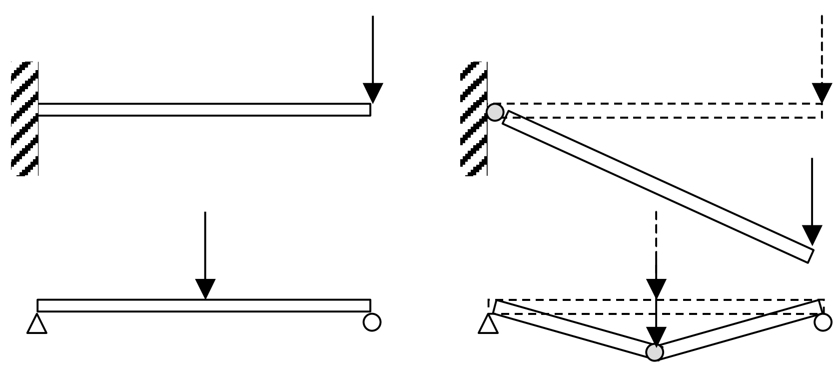


For a non-rectangular section and internal equilibrium at σ_y, the n.a. *will not necessarily be at the centroid.* The n.a. occurs where the $A_{tension} = A_{compression}$. The reactions occur at the centroids of the tension and compression areas.



$A_{tension} = A_{compression}$

Instability from Plastic Hinges



Shape Factor:

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

$$k = {}^{M_p}\!/_{M_y}$$

k = 3/2 for a rectangle
k ≈ 1.1 for an I beam

Plastic Section Modulus

$$Z = \frac{M_p}{f_y} \qquad \textit{and} \qquad k = {}^{Z}\!/_{S}$$

Design for Shear

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

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:

Case 1) For $h/t_w \le 2.24\sqrt{\frac{E}{F_y}}$ $V_n = 0.6F_{yw}A_w$ ϕ_v = 1.00 (LRFD) Ω = 1.50 (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

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

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

Design for Flexure

$M_a \le M_n / \Omega$ or $M_u \le \phi_b M_n$ ϕ_b = 0.90 (LRFD) Ω = 1.67 (ASD)

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

1. yielding, limited at length $L_p = 1.76r_y\sqrt{\frac{E}{F_y}}$, 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/Ω 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 $1<C_b\le2.3$, 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/Ω or $\phi_b M_n$

<u>Compact Sections</u>

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.

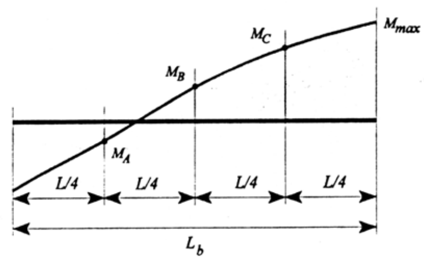
Compact sections meet the following criteria: $\frac{b_f}{2t_f} \le 0.38\sqrt{\frac{E}{F_y}}$ and $\frac{h_c}{t_w} \le 3.76\sqrt{\frac{E}{F_y}}$

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

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



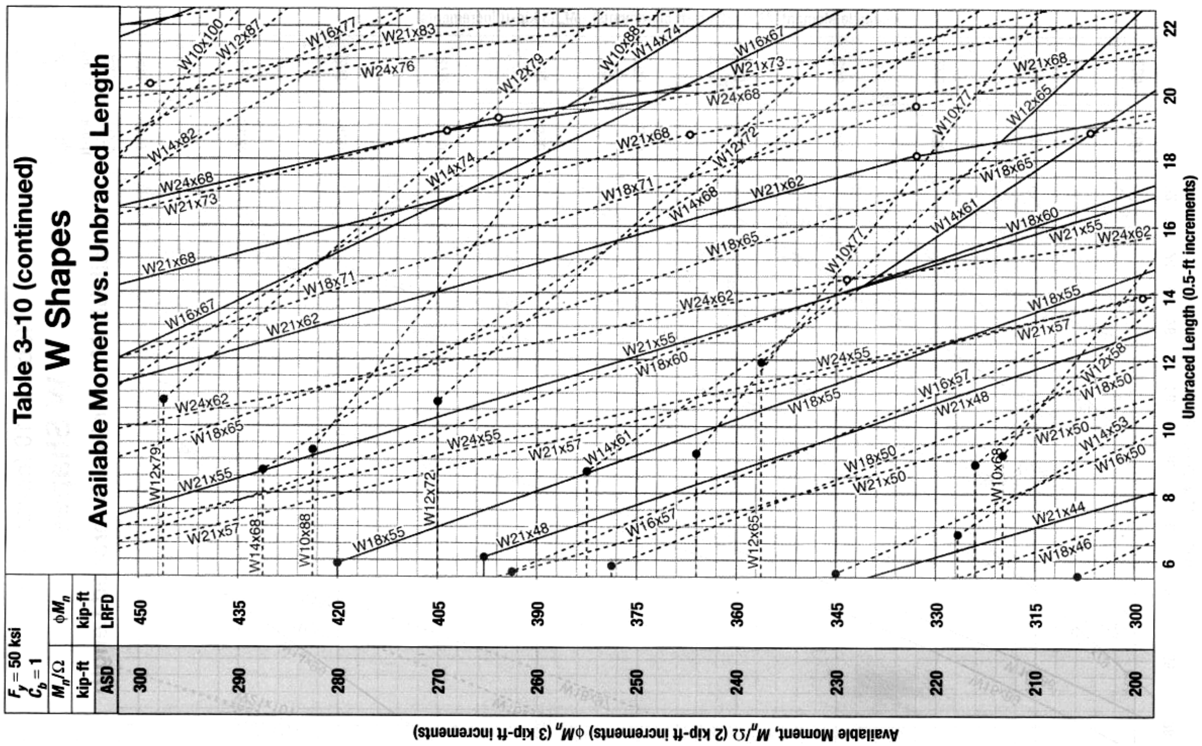
where C_b is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

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

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 $M_a \le M_n/\Omega$ *or* $M_u \le \phi_b M_n$. 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 (•), while L_r is indicated by an open dot (○). 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 $f_b = \frac{M_{max}}{S} \le F_b$, $Z_{req'd} \ge \frac{M_{max}}{F_b} = \frac{M_{max}}{F_y/\Omega}$ and $Z \ge \frac{M_u}{\phi_b F_y}$ to meet the design criteria that

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

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.

TABLE 9.1 Load Factor Resistance Design Selection

Designation	Z_x in.^{3}	L_p ft (F_y = 36 ksi)	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

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

5. Consider lateral stability.

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

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

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

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

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

9. Evaluate the deflection to determine if $\Delta_{maxLL} \le \Delta_{LL-allowed}$ and/or $\Delta_{maxTotal} \le \Delta_{Total\,allowed}$

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

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:

$$M_{max} = \frac{w_{equivalent} L^2}{8}$$

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

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

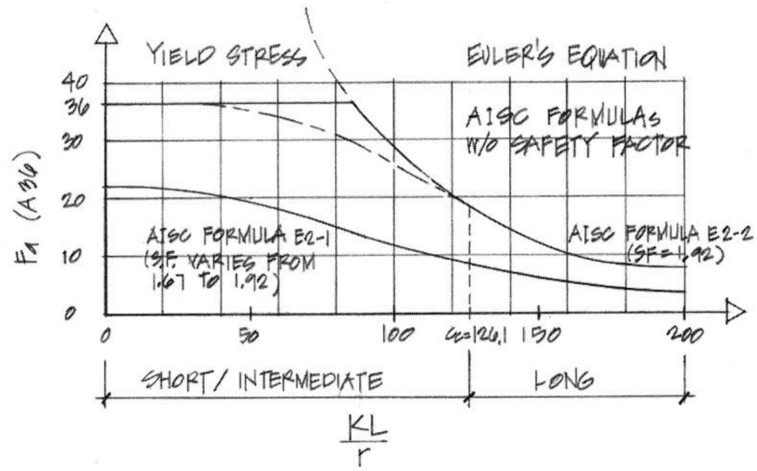
Criteria for Design of Columns

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 13th 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 14th ed:

$P_a \le P_n / \Omega$ or $P_u \le \phi_c V_n$ where

$P_u = \Sigma \gamma_i P_i$

γ is a load factor
P is a load type
ϕ is a resistance factor
P_n is the nominal load capacity (strength)

ϕ = 0.90 (LRFD) Ω = 1.67 (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 $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ or ($F_e \ge 0.44 F_y$):

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

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

$$F_{cr} = 0.877 F_e$$

where F_e is the elastic critical buckling stress: $F_e = \frac{\pi^2 E}{\left(KL/r\right)^2}$

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 with the fraction r_x/r_y.

Sample AISC Table for Available Strength in Axial Compression

F_y = 50 ksi

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W12

Shape	W12×									
Wt/ft	96		87		79		72		65	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y										
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
Properties										
P_{wo} (kips)	137	206	121	181	104	157	90.9	136	78.2	117
P_{wi} (kips/in.)	18.3	27.5	17.2	25.8	15.7	23.5	14.3	21.5	13.0	19.5
P_{wb} (kips)	296	445	243	366	185	278	142	213	106	159
P_{fb} (kips)	152	228	123	185	101	152	84.0	126	68.5	103
L_p (ft)	10.9		10.8		10.8		10.7		11.9	
L_r (ft)	46.6		43.0		39.9		37.4		35.1	
A_g (in.^{2})	28.2		25.6		23.2		21.1		19.1	
I_x (in.^{4})	833		740		662		597		533	
I_y (in.^{4})	270		241		216		195		174	
r_y (in.)	3.09		3.07		3.05		3.04		3.02	
Ratio r_x/r_y	1.76		1.75		1.75		1.75		1.75	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	23800		21200		18900		17100		15300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	7730		6900		6180		5580		4980	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

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<u>Procedure for Analysis</u>

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 $P_a \leq P_n/\Omega$ or $P_u \leq \phi_c P_n$? $\Rightarrow$ yes, it is; no, it is no good

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

<u>Procedure for Design</u>

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.*

<u>Columns with Bending (Beam-Columns)</u>

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 <u>r</u> or <u>A</u> or <u>Z</u>

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/Ω for ASD or $\phi_c P_n$ for LRFD). The increased bending moment due to the P-Δ effect must be determined and used as the moment to resist.

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

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

(ASD)

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

(LRFD)

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

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

(ASD)

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

(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

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

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.*

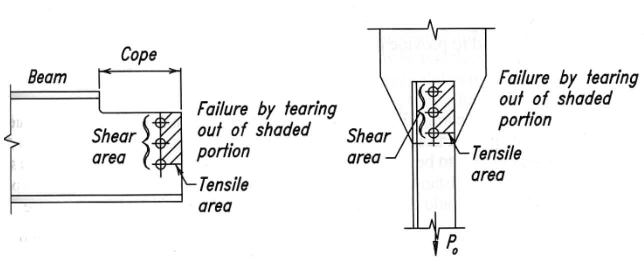


Fig. C-J4.1. Failure for block shear rupture limit state.

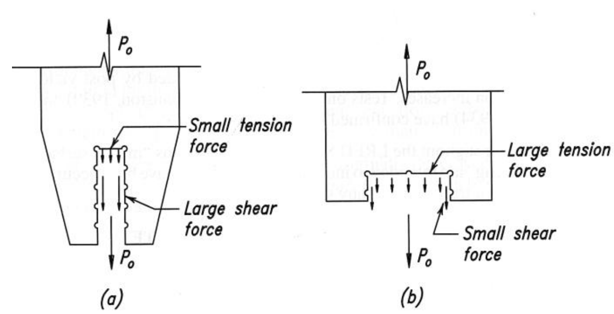
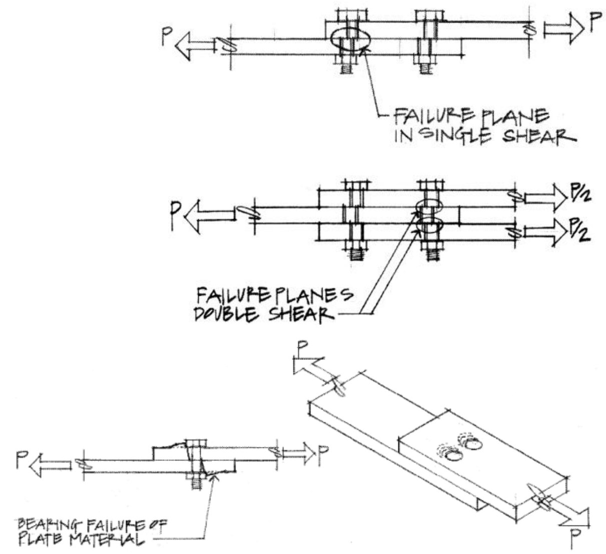


Fig. C-J4.2. Block shear rupture in tension.

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
A490: high strength bolts (higher than A325)
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.

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.

<u>For shear OR tension (same equation) in bolts:</u> $R_a \leq R_n / \Omega$ or $R_u \leq \phi R_n$

where $R_u = \Sigma \gamma_i R_i$

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

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

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

<u>For bearing of plate material at bolt holes</u>: $R_a \leq R_n / \Omega$ or $R_u \leq \phi R_n$

where $R_u = \Sigma \gamma_i R_i$

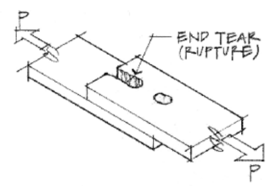


Figure 10.11 End tear-out.

- deformation at bolt hole is a concern

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

- deformation at bolt hole is not a concern

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

- long slotted holes with the slot perpendicular to the load

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

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

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

The *minimum* edge desistance from the center of the outer most bolt to the edge of a member is generally 1¾ times the bolt diameter for the sheared edge and 1¼ 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.

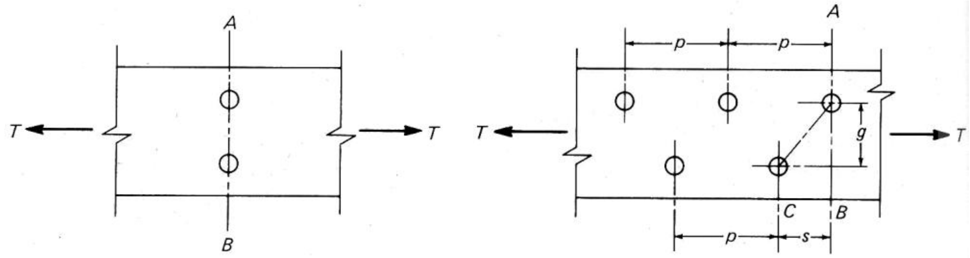
Standard bolt hole spacing is 3 in. with the minimum spacing of $2\frac{2}{3}$ times the diameter of the bolt, d_b. Common edge distance from the center of last hole to the edge is 1¼ in..

<u>Tension Member Design</u>

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

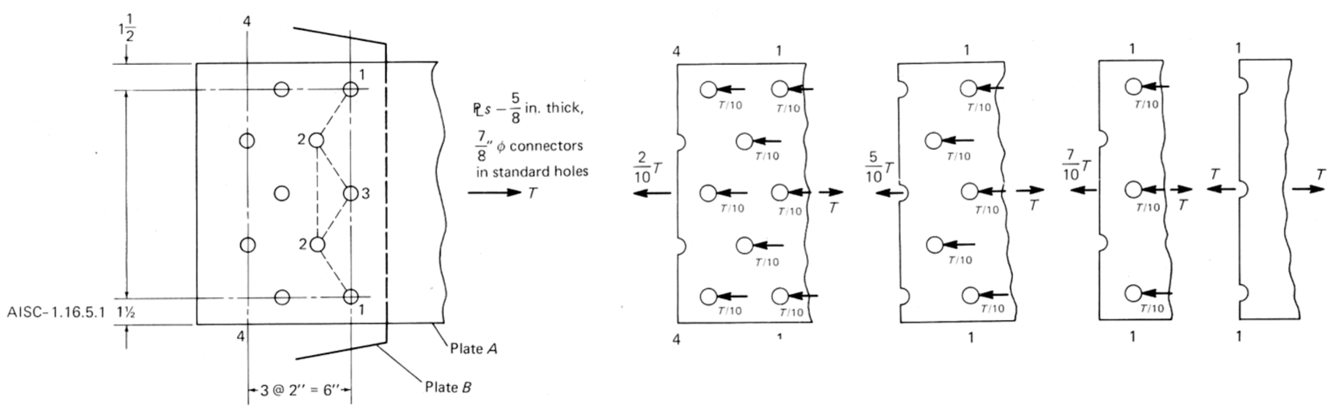
Effective Net Area:



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

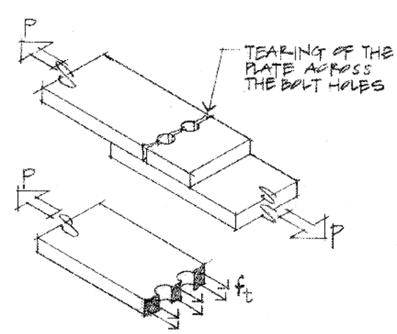
A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.

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: $A_e = A_n U$



<u>*For tension elements:*</u>

$$R_a \le R_n / \Omega \text{ or } R_u \le \phi R_n$$
$$\text{where } R_u = \Sigma \gamma_i R_i$$



1. yielding $R_n = F_y A_g$

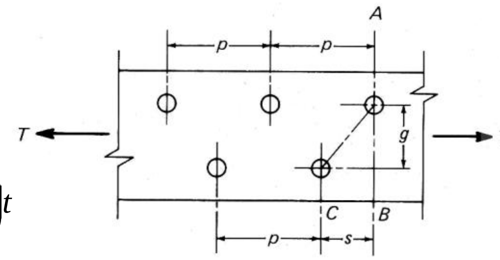
   ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

2. rupture $R_n = F_u A_e$

   ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

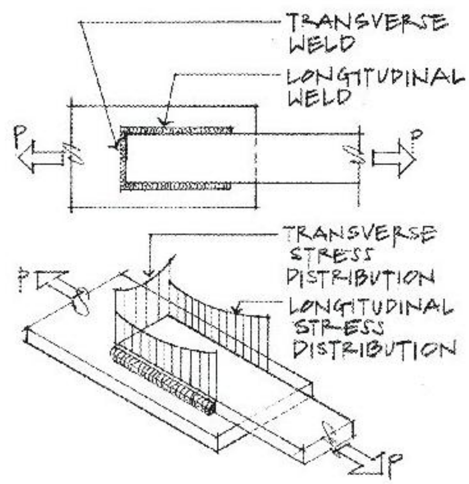
where A_g = the gross area of the member (excluding holes)
A_e = the effective net area (with holes, etc.)
F_y = the yield strength of the steel
F_u = the tensile strength of the steel (ultimate)

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



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

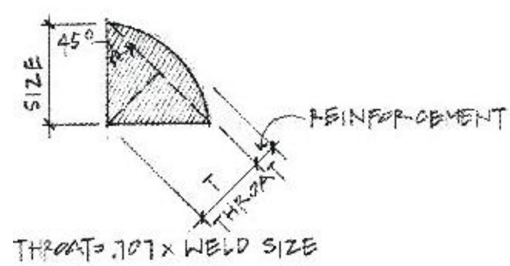


Weld designations include the strength in the name, i.e. E70XX has Fy = 70 ksi. Welds are weakest in shear and are assumed to always fail in the shear mode.

The throat size, T, of a fillet weld is determined trigonometry by: T = 0.707 ×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 ¼" or less
b) is permitted to be 1/16" less than the thickness of the material if it is over ¼"

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 ¼ the length.

Intermittent fillet welds cannot be less than four times the weld size, not to be less than 1 ½".

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Size of Fillet Weld[a] (in.)
To ¼ inclusive	⅛
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾	5/16

[a]Leg dimension of fillet welds. Single-pass welds must be used.

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For fillet welds:

$$R_a \leq R_n / \Omega \text{ or } R_u \leq \phi R_n$$
$$\text{where } R_u = \Sigma \gamma_i R_i$$

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

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

where:

T is throat thickness
l is length of the weld

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.

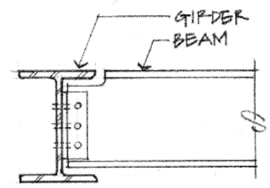
Available Strength of Fillet Welds per inch of weld (ϕS)

Weld Size (in.)	E60XX (k/in.)	E70XX (k/in.)
3/16	3.58	4.18
¼	4.77	5.57
5/16	5.97	6.96
3/8	7.16	8.35
7/16	8.35	9.74
½	9.55	11.14
5/8	11.93	13.92
¾	14.32	16.70

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

Framed Beam Connections

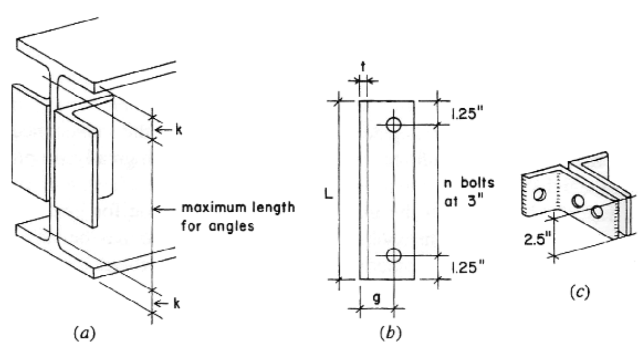
Coping is the term for cutting away part of the flange to connect a beam to another beam using welded or bolted angles.



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.



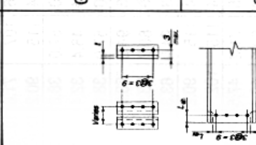
Sample AISC Table for Bolt and Angle Available Strength in All-Bolted Double-Angle Connections

Beam F_y = 50 ksi, F_u = 65 ksi
Angle F_y = 36 ksi, F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections
3/4-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows W24, 21, 18, 16	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD	67.1	101	83.9	126	95.5	143	95.5	143
		X	STD	67.1	101	83.9	126	101	151	120	180
		SC Class A	STD	50.6	75.9	50.6	75.9	50.6	75.9	50.6	75.9
			OVS	43.1	64.5	43.1	64.5	43.1	64.5	43.1	64.5
			SSLT	50.6	75.9	50.6	75.9	50.6	75.9	50.6	75.9
		SC Class B	STD	67.1	101	83.9	126	84.4	127	84.4	127
			OVS	65.3	97.9	71.9	108	71.9	108	71.9	108
			SSLT	65.8	98.7	82.2	123	84.4	127	84.4	127
	Group B	N	STD	67.1	101	83.9	126	101	151	120	180
		X	STD	67.1	101	83.9	126	101	151	134	201
		SC Class A	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9
			OVS	53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7
			SSLT	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9
		SC Class B	STD	67.1	101	83.9	126	101	151	105	158
			OVS	65.3	97.9	81.6	122	89.9	134	89.9	134
			SSLT	65.8	98.7	82.2	123	98.7	148	105	158



Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
L_{eh}*, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
	L_{ev}, in.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	167	250	175	262	156	234	164	246	164	245	172	257
	1 3/8	169	254	177	266	158	238	167	250	166	249	174	261
	1 1/2	171	257	180	269	161	241	169	254	168	253	177	265
	1 5/8	174	261	182	273	163	245	171	257	171	256	179	268
	2	181	272	189	284	171	256	179	268	178	267	186	279
	3	201	301	209	313	190	285	198	297	198	296	206	309
Coped at Both Flanges	1 1/4	156	234	156	234	146	219	146	219	156	234	156	234
	1 3/8	161	241	161	241	151	227	151	227	161	241	161	241
	1 1/2	166	249	166	249	156	234	156	234	166	249	166	249
	1 5/8	171	256	171	256	161	241	161	241	171	256	171	256
	2	181	272	185	278	171	256	176	263	178	267	185	278
	3	201	301	209	313	190	285	198	297	198	296	206	309
Uncoped		234	351	234	351	234	351	234	351	234	351	234	351

Support Available Strength per Inch Thickness, kips/in.		
Hole Type	ASD	LRFD
STD/OVS/SSLT	468	702

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_{eh}, to account for possible underrun in beam length.

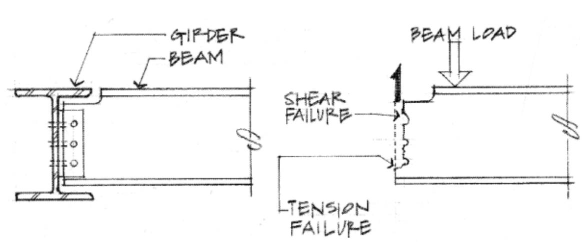
Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.

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): $R_a \leq R_n / \Omega$ or $R_u \leq \phi R_n$

where $R_u = \Sigma \gamma_i R_i$

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

ϕ = 0.75 (LRFD) Ω = 2.00 (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.

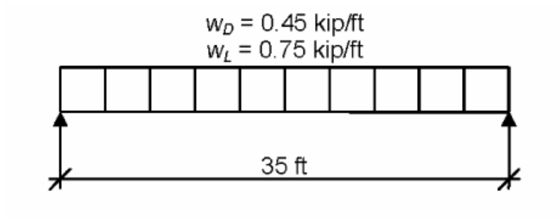
Example 2 (pg 300)

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:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ 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 k/ft + 0.750 k/ft = 1.20 k/ft

   V_a = 1.20 k/ft(35 ft)/2 = 21 k

   M_a = 1.20 k/ft(35 ft)^2/8 = 184 k-ft

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

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

   $Z_{req'd} \geq M_{max}/F_b = M_{max}(\Omega)/F_y$ = 184 k-ft(1.67)(12 in/ft)/50 ksi = 73.75 in^3 (F_y 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 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 k/ft + 0.04 k/ft

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

   $M^*_{a\text{-}adjusted}$ = 1.24 k/ft(35 ft)^3/8 = 189.9 k

   $Z_{req'd} \geq$ 189.9 k-ft(1.67)(12 in/ft)/50 ksi = 76.11 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 in and t_w = 0.315 in

   $V_n/\Omega = 0.6F_{yw}A_w/\Omega$ = 0.6(50 ksi)(17.90 in)(0.315 in)/1.5 = 112.8 k which is much larger than 21.7 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 in^4

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

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

   Find the moment of inertia needed:

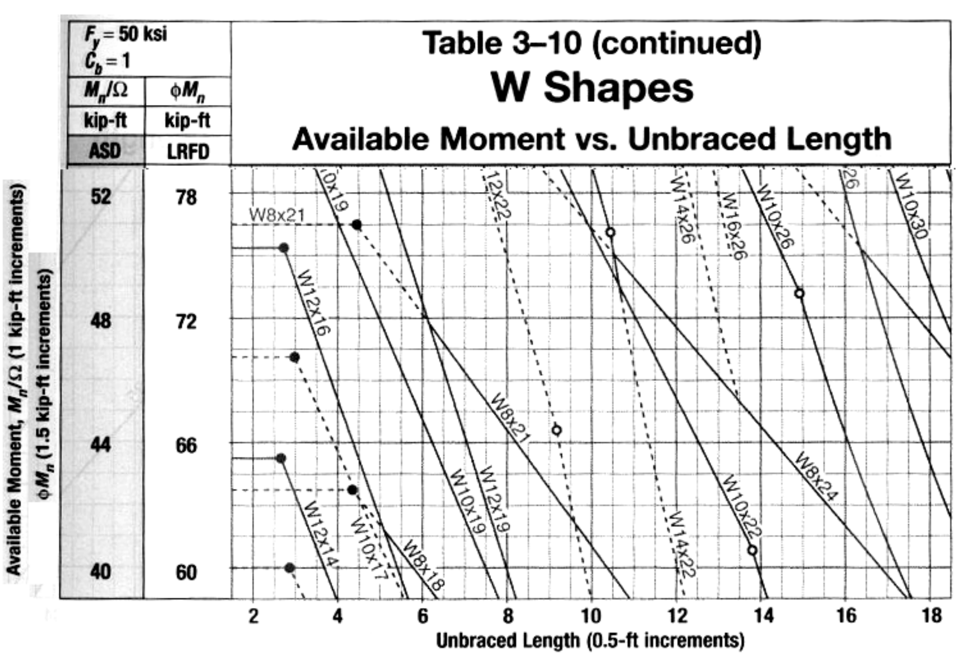
   $I_{req'd} \geq \Delta_{too\ big}(I_{trial})/\Delta_{limit}$ = 1.42 in(612 in^4)/(1.17 in) = 742.8 in^4

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's). (In order: I_x = 586, 394, 541, 712 and 800 in^4)

Choose a W18 x 50

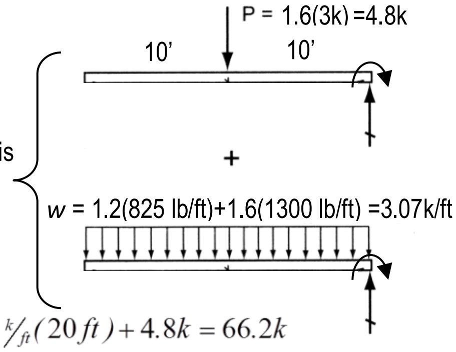
Example 4

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):

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

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:

$$M^*_{u-adjusted} = 662^{k-ft} + \frac{1.2(76\,^{lb}\!/\!_{ft})(20ft)^2}{2(1000\,^{lb}\!/\!_{k})} 680.2^{k-ft} \qquad V^*_{u-adjusted} = 66.2k + 1.2(0.076\,^k\!/\!_{ft})(20ft) = 68.0k$$

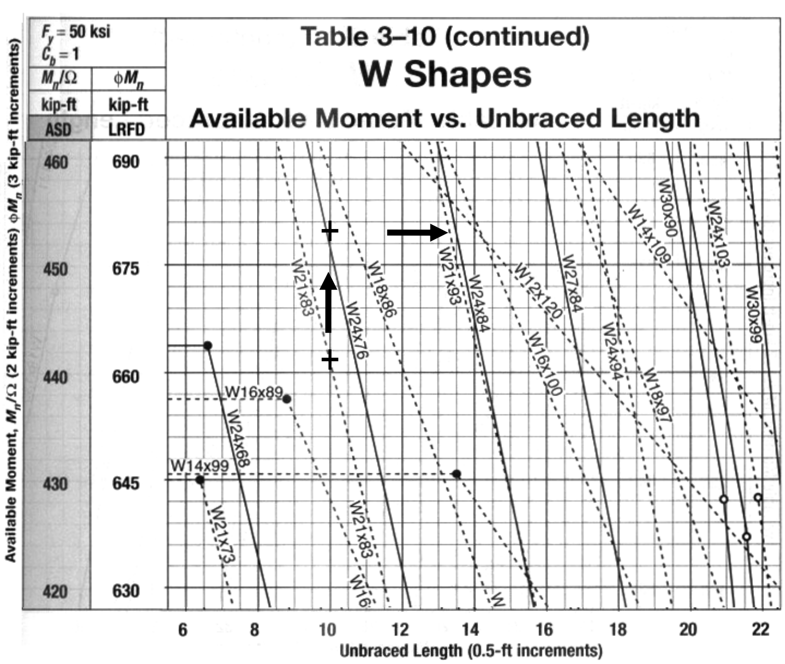
Replot 680.2 k-ft vs. 10ft, which lands *above* the capacity of the W21x83. We can't look up because the chart ends, but we can look for that capacity with a longer unbraced length. This leads us to a **W24 x 84** as the most economical. (With the additional self weight of 84 - 76 lb/ft = 8 lb/ft, the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

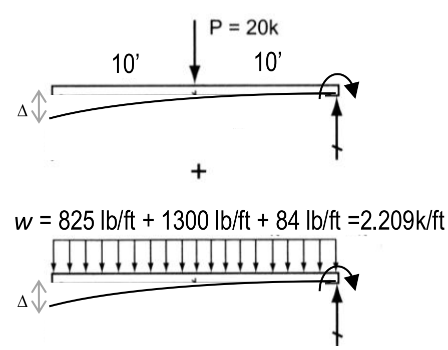
Evaluate the shear capacity:

$$\phi_v V_n = \phi_v 0.6 F_{yw} A_w = 1.0(0.6)50ksi(24.10in)0.47in = 338.4k$$ so yes, 68 k ≤ 338.4k 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 in. and L/180 = 1.33 in.

$$\Delta_{total} = \frac{Pb^2(3l-b)}{6EI} + \frac{wL^4}{24EI} = \frac{3k(10ft)^2(3\cdot 20 - 10ft)(12\,^{in}\!/\!_{ft})^3}{6(30x10^3ksi)2370in^3} + \frac{(2.209\,^k\!/\!_{ft})(20ft)^4(12\,^{in}\!/\!_{ft})^3}{24(30x10^3ksi)2370in^3} = 0.06 + 0.36 = 0.42in$$





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

0.06 in. ≤ 1 in. and 0.42 in. ≤ 1.33 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^2 and a live load of 100 lb/ft^2. 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.*

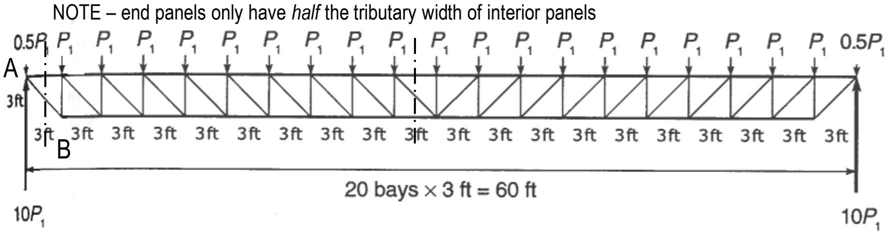
Table 7.2 Computation of Truss Joint Loads

	area loads				*tributary widths*							
	w_{dead}		w_{live}		Node-to-Node Spacing	Truss-to-Truss Spacing	Floor Area per Node A	P_{dead} (=$w_{dead}\cdot$ A)	P_{live} (=$w_{live}\cdot$ A)	Factored Dead Load $1.2\cdot P_{dead}$	Factored Live Load $1.6\cdot P_{live}$	Factored Total Load $1.2\cdot P_{dead}$ + $1.6\cdot P_{live}$
Truss	(#/ft^2)	(K/ft^2)	(#/ft^2)	(K/ft^2)	(ft)	(ft)	(ft^2)	(K)	(K)	(K)	(K)	(K)
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 $1.2P_{dead} = 1.2w_{dead}\cdot$ *tributary width* = 0.14 K

6 $1.2P_{dead} = 1.2w_{dead}\cdot$ *tributary width* = 0.29 K

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

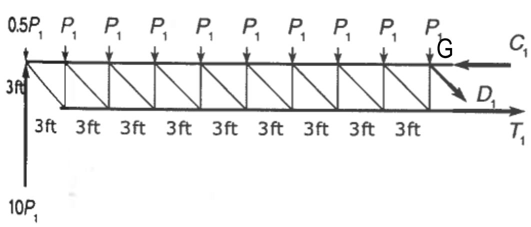


***FBD 1** for 3 ft deep truss*

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

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

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



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

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

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

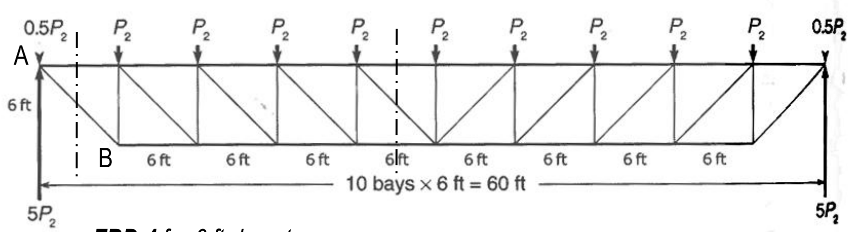
$\Sigma M_G = 9.5P_1(30^{ft}) - P_1(27^{ft}) - P_1(24^{ft}) - P_1(21^{ft}) - P_1(18^{ft})$
$- P_1(15^{ft}) - P_1(12^{ft}) - P_1(9^{ft}) - P_1(6^{ft}) - P_1(3^{ft}) - T_1(3^{ft}) = 0$

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

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

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

$\Sigma F_x = -C_1 + T_1 + D_1\cdot\cos 45^\circ = 0$ $\underline{C_1 = 176.2\text{ k}}$

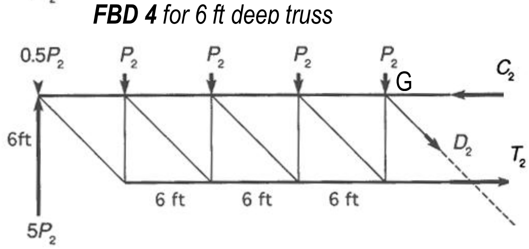


***FBD 4** for 6 ft deep truss*

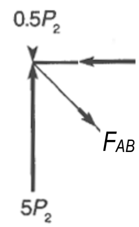
FBD 6: Maximum web force will be in the end diagonal

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

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



***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

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

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

$\Sigma F_y = 5P_2 - 4.5P_1 - D_s\cdot\sin 45^\circ = 0$

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

$\Sigma F_x = -C_2 + T_2 + D_2\cdot\cos 45^\circ = 0$ $\underline{C_2 = 92.4\text{ k}}$

Example 7 (pg 339)

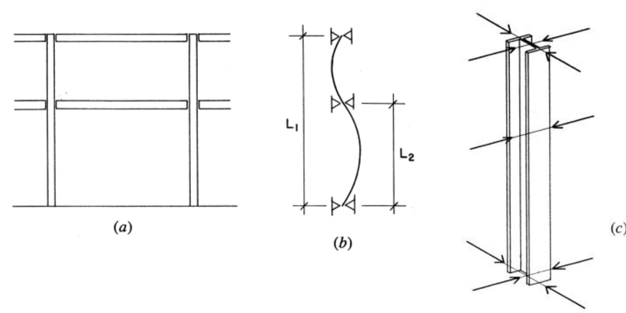
Example 14. Open web steel joists are to be used for a floor with a unit live load of 75 psf [3.59 kN/m^2 and a unit dead load of 40 psf [1.91 kN/m^2 (not including the joist weight) on a span of 30 ft [9.15 m]. Joists are 2 ft [0.61 m] on center, and deflection *(from unfactored loads)* is limited to 1/240 of the span under total load and 1/360 of the span under live load only. Determine the lightest possible joist and the lightest joist of least depth possible.

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	472	571	387	527	638	516	634	816
	(151)	(199)	(239)	(189)	(248)	(298)	(270)	(328)	(413)
30	301	409	497	337	457	555	448	550	738
	(123)	(161)	(194)	(153)	(201)	(242)	(219)	(266)	(349)
32	264	359	436	295	402	487	393	484	647
	(101)	(132)	(159)	(126)	(165)	(199)	(180)	(219)	(287)

Example 8 (pg 353)

Example 3. Figure 10.5*a* 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 10.5*b*. 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?

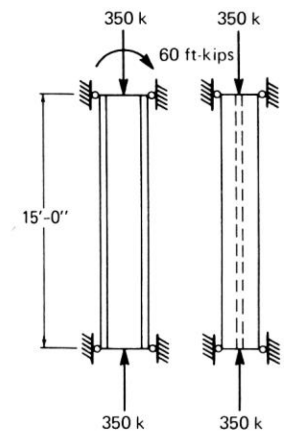


Example 9 (pg 361)

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 accepatbility 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):

Axial load = 1.2(0.25)(350k)+1.6(0.75)(350k)=525k

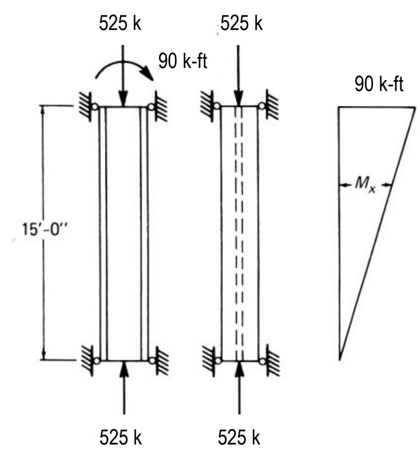
Moment at joint = 1.2(0.25)(60 $^{k\text{-}ft}$) + 1.6(0.75)(60 $^{k\text{-}ft}$) = 90 $^{k\text{-}ft}$

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

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

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

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



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 ϕM_n at L_b =15 ft is 422 k-ft.

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

$$C_m = 0.6 - 0.4\frac{M_1}{M_2} = 0.6 - \frac{0^{k-ft}}{90^{k-ft}} = 0.6 \le 1.0 \qquad P_{e1} = \frac{\pi^2 EA}{\left(Kl/r\right)^2} = \frac{\pi^2(30x10^3\,ksi)19.7in^2}{(25.9)^2} = 8{,}695.4k$$

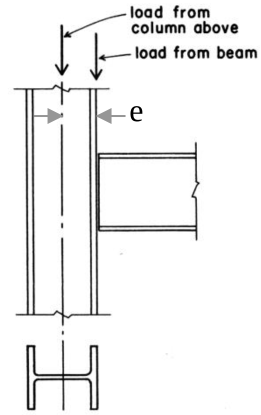
$$B_1 = \frac{C_m}{1-(P_u/P_{e1})} = \frac{0.6}{1-(525k/8695.4\,k)} = 0.64 \not\ge 1.0 \quad \text{USE 1.0} \qquad M_u = (1)90 \text{ k-ft}$$

Finally, determine the interaction value:

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = 0.87 + \frac{8}{9}\left(\frac{90^{k-ft}}{422^{k-ft}}\right) = 1.06 \not\le 1.0$$

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.

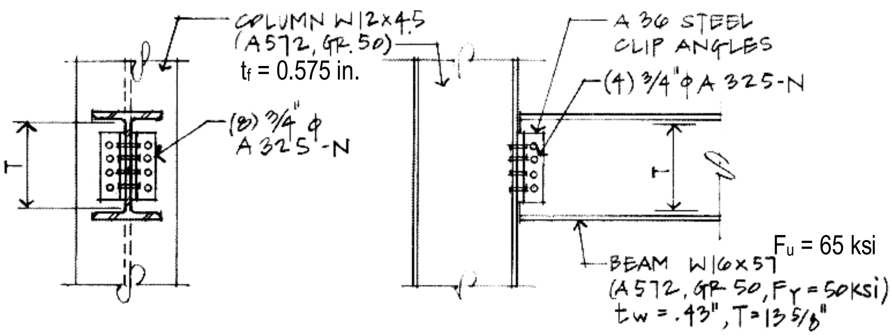
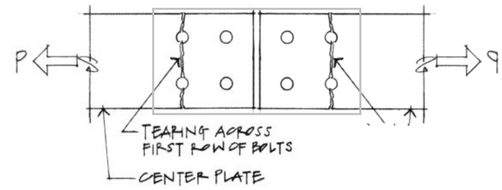


Figure 10.28 Typical beam–column connection.

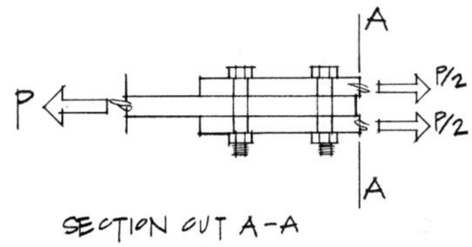
Example 13

10.2 The butt splice shown in Figure 10.22 uses two 8 × ⅜" plates to "sandwich" in the 8 × ½" plates being joined. Four ⅞"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.)



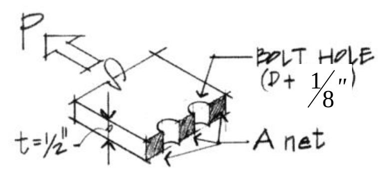
Shear: Using the AISC available shear in Table 7-3 (Group A):

ϕR_n = 26.4 k/bolt x 4 bolts = 105.6 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 ksi.

ϕR_n = 91.4 k/bolt/in. x 0.5 in. x 4 bolts = 182.8 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:

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

A_g = 8 in. x ½ in. = 4 in^2

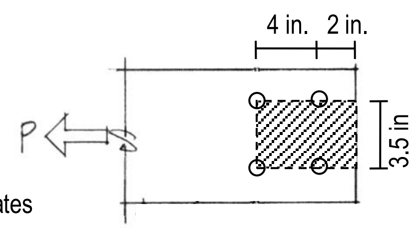
The holes are considered 1/8 in. larger than the nominal bolt diameter = 7/8 + 1/8 = 1 in.

A_n = (8 in. – 2 holes x 1 in.) x ½ in. = 3 in^2

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

$\phi F_y A_g$ = 0.9 x 36 ksi x 4 in^2 = 129.6 k

$\phi F_u A_e$ = 0.75 x 58 ksi x (1) x 3 in^2 = 130.5 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 in.) x ½ in. = 3 in^2

$A_{nv} = A_{gv}$ – 1 ½ holes area = 3 in^2 – 1.5 x 1 in. x ½ in. = 2.25 in^2

A_{nt} = 3.5 in. x t – 1 holes = 3.5 in. x ½ in – 1 x 1 in. x ½ in. = 1.25 in^2

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

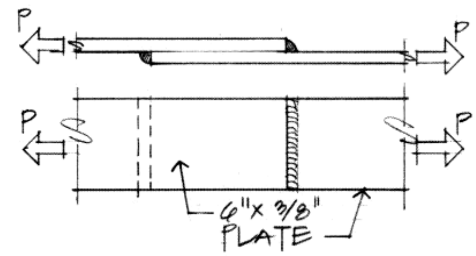
$\phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt})$ = 0.75 x (0.6 x 36 ksi x 3 in^2 + 1 x 58 ksi x 1.25 in^2) = 103.0 k

The maximum connection capacity is governed by block shear rupture.

ϕR_n = 103.0 k

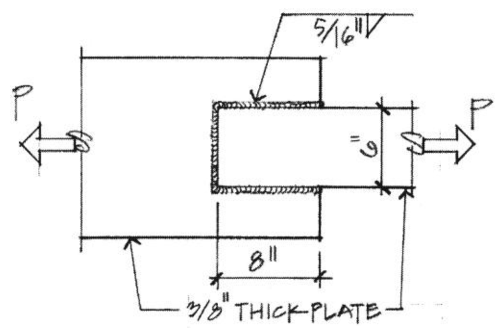
Example 14

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 5⁄16″ fillet weld, ϕS = 6.96 k/in

Weld length = 22″

Weld capacity = 22″ × 6.96 k/in = 153.1 k

Capacity of plate: 0.9 x 36 k/in^2 x 3/8″ x 6″= 72.9 k

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

Plate capacity = 0.9 x 36 k/in^2 x 3/8″ x 6″= 72.9 k

∴ Plate capacity governs, P_{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 ≈ plate capacity:

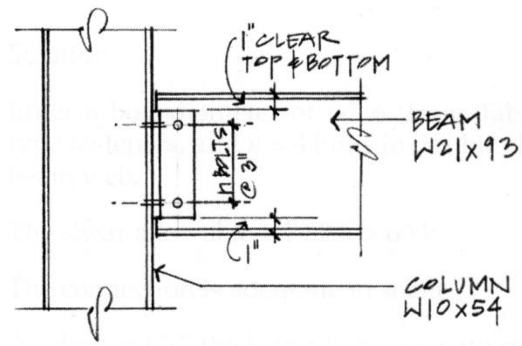
22″ × (weld capacity per in.) = 72.9 k

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

From Available Strength table, use 3/16″ weld (ϕS = 4.18 k/in.)

Minimum size fillet = 3⁄16″ based on a 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 be depends 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:

Available length = beam depth – both flange thicknesses – 1" clearance at top & 1" at bottom

= 21.62 in – 2(0.93 in) – 2(1 in) = 17.76 in.

With the spaced at 3 in. and 1 ¼ in. end lengths (each end), the maximum number of bolts can be determined:

Available length ≥ 1.25 in. + 1.25 in. + 3 in. x (number of bolts – 1)

number of bolts ≤ (17.76 in – 2.5 in. - (-3 in.))/3 in. = 6.1, so 6 bolts.

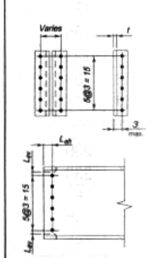
It is helpful to have the All-bolted Double-Angle Connection Tables 10-1. They are available for ¾", 7/8", and 1" bolt diameters and list angle thicknesses of ¼", 5/16", 3/8", and ½". Increasing the angle thickness is likely to increase the angle strength, although the limit states include shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Table 10-1 (continued)
All-Bolted Double-Angle Connections — **7/8-in. Bolts**

Beam: F_y = 50 ksi, F_u = 65 ksi
Angle: F_y = 36 ksi, F_u = 58 ksi

Bolt and Angle Available Strength, kips

6 Rows — W40, 36, 33, 30, 27, 24, 21



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 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
		OVS	90.1	135	90.1	135	90.1	135	90.1	135
		SSLT	97.3	146	106	159	106	159	106	159
	SC Class B	STD	98.6	148	123	185	148	222	176	264
		OVS	93.5	140	117	175	140	210	150	225
		SSLT	97.3	146	122	182	146	219	176	264
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
		OVS	93.5	140	113	169	113	169	113	169
		SSLT	97.3	146	122	182	133	199	133	199
	SC Class B	STD	98.6	148	123	185	148	222	197	296
		OVS	93.5	140	117	175	140	210	187	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 ¾" bolts, 296 kips for the 7/8" bolts, and 281 kips for the 1" bolts. It appears that increasing the bolt diameter to 1" will not gain additional load. <u>Use 7/8" bolts.</u>

ϕR_n = 367.8 kips for double shear of 7/8" bolts ϕR_n = 296 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 7/8" bolt diameters at 3 in. spacing.

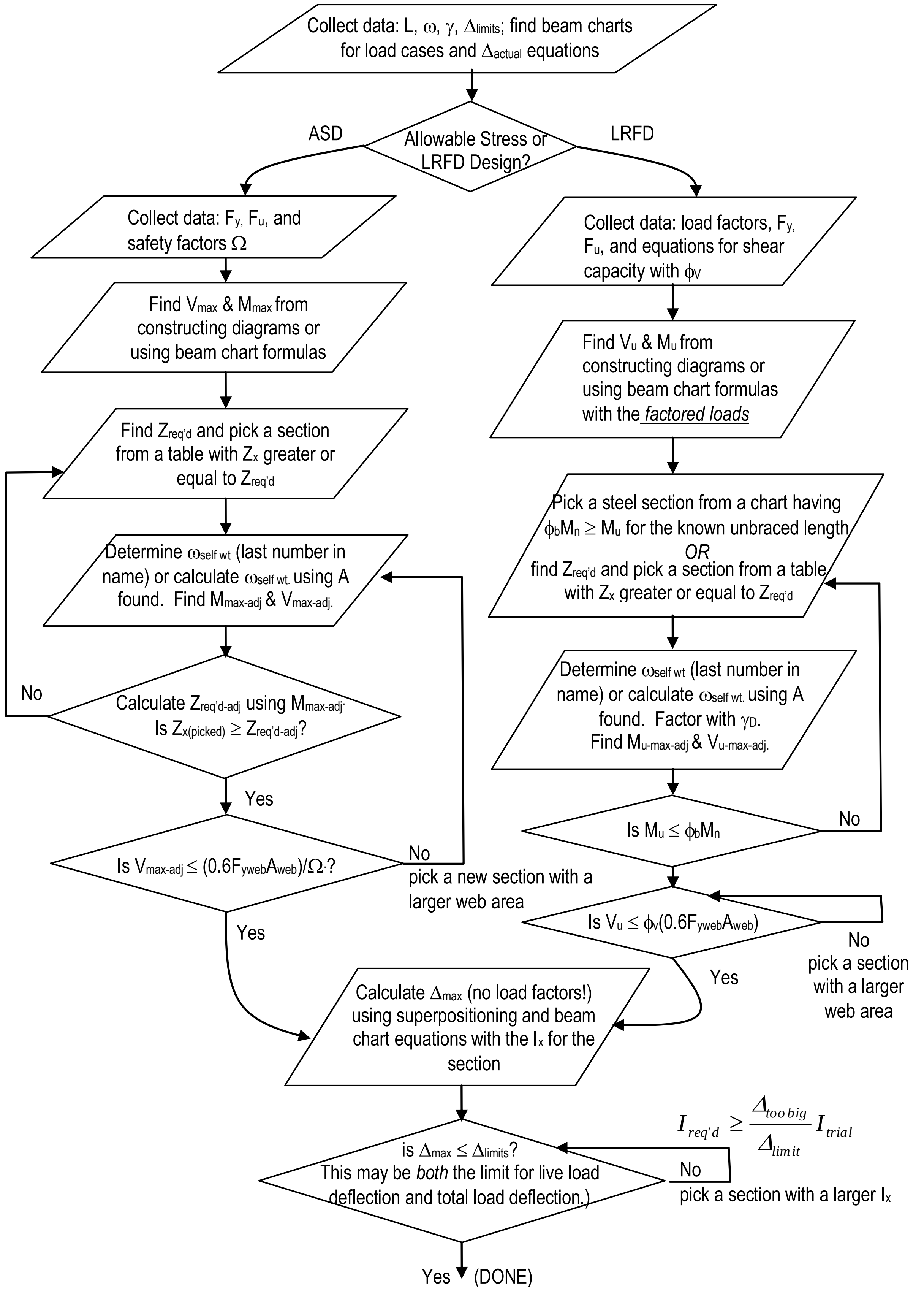
$$\phi R_n = 6 \text{ bolts}\cdot(102 \text{ k/bolt/inch})\cdot(0.58 \text{ in}) = 355.0 \text{ kips}$$

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



Available Critical Stress, $\phi_c F_{cr}$, for Compression Members, ksi (F_y = 36 ksi and ϕ_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 ϕ_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, d, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.²					0.307		0.442		0.601		0.785	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		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
	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
	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, d, in.					1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.²					0.994		1.23		1.48		1.77	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		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
	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
	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.										
$\Omega = 2.00$	$\phi = 0.75$											

Table 7-2
Available Tensile Strength of Bolts, kips

Nominal Bolt Diameter, d, in.			5/8		3/4		7/8		1	
Nominal Bolt Area, in.²			0.307		0.442		0.601		0.785	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	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, d, in.			1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.²			0.994		1.23		1.48		1.77	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	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									
$\Omega = 2.00$	$\phi = 0.75$									

Group A Bolts

A325, A325M
F1858
A354 Grade BC
A449

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

Group A Bolts

Hole Type	Loading	5/8 r_n/Ω ASD	5/8 ϕr_n LRFD	3/4 r_n/Ω ASD	3/4 ϕr_n LRFD	7/8 r_n/Ω ASD	7/8 ϕr_n LRFD	1 r_n/Ω ASD	1 ϕr_n LRFD
Min. Group A Bolt Pretension, kips		19		28		39		51	
STD/SSLT	S	4.29	6.44	6.33	9.49	8.81	13.2	11.5	17.3
	D	8.59	12.9	12.7	19.0	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
	D	7.32	10.9	10.8	16.1	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
	D	6.02	9.02	8.87	13.3	12.4	18.5	16.2	24.2

Hole Type	Loading	1 1/8 r_n/Ω ASD	1 1/8 ϕr_n LRFD	1 1/4 r_n/Ω ASD	1 1/4 ϕr_n LRFD	1 3/8 r_n/Ω ASD	1 3/8 ϕr_n LRFD	1 1/2 r_n/Ω ASD	1 1/2 ϕr_n LRFD
Min. Group A Bolt Pretension, kips		56		71		85		103	
STD/SSLT	S	12.7	19.0	16.0	24.1	19.2	28.8	23.3	34.9
	D	25.3	38.0	32.1	48.1	38.4	57.6	46.6	69.8
OVS/SSLP	S	10.8	16.1	13.7	20.5	16.4	24.5	19.8	29.7
	D	21.6	32.3	27.4	40.9	32.7	49.0	39.7	59.4
LSL	S	8.87	13.3	11.2	16.8	13.5	20.2	16.3	24.4
	D	17.7	26.6	22.5	33.7	26.9	40.3	32.6	48.9

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

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	$\Omega = 1.50$	$\phi = 1.00$
OVS and SSLP	$\Omega = 1.76$	$\phi = 0.85$
LSL	$\Omega = 2.14$	$\phi = 0.70$

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 AISC *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.

Group B Bolts

A490, A490M
F2280
A354 Grade BD

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

Group B Bolts

Hole Type	Loading	5/8 r_n/Ω ASD	5/8 ϕr_n LRFD	3/4 r_n/Ω ASD	3/4 ϕr_n LRFD	7/8 r_n/Ω ASD	7/8 ϕr_n LRFD	1 r_n/Ω ASD	1 ϕr_n LRFD
Min. Group B Bolt Pretension, kips		24		35		49		64	
STD/SSLT	S	5.42	8.14	7.91	11.9	11.1	16.6	14.5	21.7
	D	10.8	16.3	15.8	23.7	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
	D	9.25	13.8	13.5	20.2	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
	D	7.60	11.4	11.1	16.6	15.5	23.3	20.3	30.4

Hole Type	Loading	1 1/8 r_n/Ω ASD	1 1/8 ϕr_n LRFD	1 1/4 r_n/Ω ASD	1 1/4 ϕr_n LRFD	1 3/8 r_n/Ω ASD	1 3/8 ϕr_n LRFD	1 1/2 r_n/Ω ASD	1 1/2 ϕr_n LRFD
Min. Group B Bolt Pretension, kips		80		102		121		148	
STD/SSLT	S	18.1	27.1	23.1	34.6	27.3	41.0	33.4	50.2
	D	36.2	54.2	46.1	69.2	54.7	82.0	66.9	100
OVS/SSLP	S	15.4	23.1	19.6	29.4	23.3	34.9	28.5	42.6
	D	30.8	46.1	39.3	58.8	46.6	69.7	57.0	85.3
LSL	S	12.7	19.0	16.2	24.2	19.2	28.7	23.4	35.1
	D	25.3	38.0	32.3	48.4	38.3	57.4	46.9	70.2

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

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	$\Omega = 1.50$	$\phi = 1.00$
OVS and SSLP	$\Omega = 1.76$	$\phi = 0.85$
LSL	$\Omega = 2.14$	$\phi = 0.70$

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 AISC *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 7-4
Available Bearing Strength at Bolt Holes Based on Bolt Spacing
kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	5/8 r_n/Ω ASD	5/8 ϕr_n LRFD	3/4 r_n/Ω ASD	3/4 ϕr_n LRFD	7/8 r_n/Ω ASD	7/8 ϕr_n LRFD	1 r_n/Ω ASD	1 ϕr_n LRFD
STD SSLT	$2^2/_3 d_b$	58	34.1	51.1	41.3	62.0	48.6	72.9	55.8	83.7
		65	38.2	57.3	46.3	69.5	54.4	81.7	62.6	93.8
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	67.4	101
		65	48.8	73.1	58.5	87.8	68.3	102	75.6	113
SSLP	$2^2/_3 d_b$	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1	70.7
		65	30.9	46.3	39.0	58.5	47.1	70.7	52.8	79.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	58.7	88.1
		65	48.8	73.1	58.5	87.8	68.3	102	65.8	98.7
OVS	$2^2/_3 d_b$	58	29.7	44.6	37.0	55.5	44.2	66.3	49.3	74.0
		65	33.3	50.0	41.4	62.2	49.6	74.3	55.3	82.9
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
		65	48.8	73.1	58.5	87.8	68.3	102	68.3	102
LSLP	$2^2/_3 d_b$	58	3.62	5.44	4.35	6.53	5.08	7.61	5.80	8.70
		65	4.06	6.09	4.88	7.31	5.69	8.53	6.50	9.75
	3 in.	58	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
		65	48.8	73.1	43.9	65.8	31.7	47.5	19.5	29.3
LSLT	$2^2/_3 d_b$	58	28.4	42.6	34.4	51.7	40.5	60.7	46.5	69.8
		65	31.8	47.7	38.6	57.9	45.4	68.0	52.1	78.2
	3 in.	58	36.3	54.4	43.5	65.3	50.8	76.1	56.2	84.3
		65	40.6	60.9	48.8	73.1	56.9	85.3	63.0	94.5
STD, SSLT, SSLP, OVS, LSLP	$s \ge s_{full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$s \ge s_{full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Spacing for full bearing strength s_{full}[a], in.		STD, SSLT, LSLT	$1^{15}/_{16}$		$2^5/_{16}$		$2^{11}/_{16}$		$3^1/_{16}$	
		OVS	$2^1/_{16}$		$2^7/_{16}$		$2^{13}/_{16}$		$3^1/_4$	
		SSLP	$2^1/_8$		$2^1/_2$		$2^7/_8$		$3^5/_{16}$	
		LSLP	$2^{13}/_{16}$		$3^3/_8$		$3^{15}/_{16}$		$4^1/_2$	
Minimum Spacing[a] = $2^2/_3 d$, in.			$1^{11}/_{16}$		2		$2^5/_{16}$		$2^{11}/_{16}$	

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 7-4 (continued)
Available Bearing Strength at Bolt Holes Based on Bolt Spacing
kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	1 1/8 r_n/Ω ASD	1 1/8 ϕr_n LRFD	1 1/4 r_n/Ω ASD	1 1/4 ϕr_n LRFD	1 3/8 r_n/Ω ASD	1 3/8 ϕr_n LRFD	1 1/2 r_n/Ω ASD	1 1/2 ϕr_n LRFD
STD SSLT	$2^2/_3 d_b$	58	63.1	94.6	70.3	105	77.6	116	84.8	127
		65	70.7	106	78.8	118	86.9	130	95.1	143
	3 in.	58	63.1	94.6	—	—	—	—	—	—
		65	70.7	106	—	—	—	—	—	—
SSLP	$2^2/_3 d_b$	58	52.2	78.3	59.5	89.2	66.7	100	74.0	111
		65	58.5	87.8	66.6	99.9	74.8	112	82.9	124
	3 in.	58	52.2	78.3	—	—	—	—	—	—
		65	58.5	87.8	—	—	—	—	—	—
OVS	$2^2/_3 d_b$	58	54.4	81.6	61.6	92.4	68.9	103	76.1	114
		65	60.9	91.4	69.1	104	77.2	116	85.3	128
	3 in.	58	54.4	81.6	—	—	—	—	—	—
		65	60.9	91.4	—	—	—	—	—	—
LSLP	$2^2/_3 d_b$	58	6.53	9.79	7.25	10.9	7.98	12.0	8.70	13.1
		65	7.31	11.0	8.13	12.2	8.94	13.4	9.75	14.6
	3 in.	58	6.53	9.79	—	—	—	—	—	—
		65	7.31	11.0	—	—	—	—	—	—
LSLT	$2^2/_3 d_b$	58	52.6	78.8	58.6	87.9	64.6	97.0	70.7	106
		65	58.9	88.4	65.7	98.5	72.4	109	79.2	119
	3 in.	58	52.6	78.8	—	—	—	—	—	—
		65	58.9	88.4	—	—	—	—	—	—
STD, SSLT, SSLP, OVS, LSLP	$s \ge s_{full}$	58	78.3	117	87.0	131	95.7	144	104	157
		65	87.8	132	97.5	146	107	161	117	176
LSLT	$s \ge s_{full}$	58	65.3	97.9	72.5	109	79.8	120	87.0	131
		65	73.1	110	81.3	122	89.4	134	97.5	146
Spacing for full bearing strength s_{full}[a], in.		STD, SSLT, LSLT	$3^7/_{16}$		$3^{13}/_{16}$		$4^3/_{16}$		$4^9/_{16}$	
		OVS	$3^{11}/_{16}$		$4^1/_{16}$		$4^7/_{16}$		$4^{13}/_{16}$	
		SSLP	$3^3/_4$		$4^1/_8$		$4^1/_2$		$4^7/_8$	
		LSLP	$5^1/_{16}$		$5^5/_8$		$6^3/_{16}$		$6^3/_4$	
Minimum Spacing[a] = $2^2/_3 d$, in.			3		$3^5/_{16}$		$3^{11}/_{16}$		4	

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	
$\Omega = 2.00$	$\phi = 0.75$	— indicates spacing less than minimum spacing required per AISC *Specification* Section J3.3. 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

Hole Type	Edge Distance L_e, in.	F_u, ksi	5/8 r_n/Ω ASD	5/8 ϕr_n LRFD	3/4 r_n/Ω ASD	3/4 ϕr_n LRFD	7/8 r_n/Ω ASD	7/8 ϕr_n LRFD	1 r_n/Ω ASD	1 ϕr_n LRFD
STD SSLT	1 1/4	58	31.5	47.3	29.4	44.0	27.2	40.8	25.0	37.5
		65	35.3	53.0	32.9	49.4	30.5	45.7	28.0	42.0
	2	58	43.5	65.3	52.2	78.3	53.3	79.9	51.1	76.7
		65	48.8	73.1	58.5	87.8	59.7	89.6	57.3	85.9
SSLP	1 1/4	58	28.3	42.4	26.1	39.2	23.9	35.9	20.7	31.0
		65	31.7	47.5	29.3	43.9	26.8	40.2	23.2	34.7
	2	58	43.5	65.3	52.2	78.3	50.0	75.0	46.8	70.1
		65	48.8	73.1	58.5	87.8	56.1	84.1	52.4	78.6
OVS	1 1/4	58	29.4	44.0	27.2	40.8	25.0	37.5	21.8	32.6
		65	32.9	49.4	30.5	45.7	28.0	42.0	24.4	36.6
	2	58	43.5	65.3	52.2	78.3	51.1	76.7	47.9	71.8
		65	48.8	73.1	58.5	87.8	57.3	85.9	53.6	80.4
LSLP	1 1/4	58	16.3	24.5	10.9	16.3	5.44	8.16	—	—
		65	18.3	27.4	12.2	18.3	6.09	9.14	—	—
	2	58	42.4	63.6	37.0	55.5	31.5	47.3	26.1	39.2
		65	47.5	71.3	41.4	62.2	35.3	53.0	29.3	43.9
LSLT	1 1/4	58	26.3	39.4	24.5	36.7	22.7	34.0	20.8	31.3
		65	29.5	44.2	27.4	41.1	25.4	38.1	23.4	35.0
	2	58	36.3	54.4	43.5	65.3	44.4	66.6	42.6	63.9
		65	40.6	60.9	48.8	73.1	49.8	74.6	47.7	71.6
STD, SSLT, SSLP, OVS, LSLP	$L_e \geq L_{e\,full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$L_e \geq L_{e\,full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Edge distance for full bearing strength $L_e \geq L_{e\,full}$[a], in.		STD, SSLT, LSLT	1 5/8		1 15/16		2 1/4		2 9/16	
		OVS	1 11/16		2		2 5/16		2 5/8	
		SSLP	1 11/16		2		2 5/16		2 11/16	
		LSLP	2 1/16		2 7/16		2 7/8		3 1/4	

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	
$\Omega = 2.00$	$\phi = 0.75$	— indicates spacing less than minimum spacing required per AISC *Specification* Section J3.3. 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 (continued)
Available Bearing Strength at Bolt Holes Based on Edge Distance
kips/in. thickness

Hole Type	Edge Distance L_e, in.	F_u, ksi	1 1/8 r_n/Ω ASD	1 1/8 ϕr_n LRFD	1 1/4 r_n/Ω ASD	1 1/4 ϕr_n LRFD	1 3/8 r_n/Ω ASD	1 3/8 ϕr_n LRFD	1 1/2 r_n/Ω ASD	1 1/2 ϕr_n LRFD
STD SSLT	1 1/4	58	22.8	34.3	20.7	31.0	18.5	27.7	16.3	24.5
		65	25.6	38.4	23.2	34.7	20.7	31.1	18.3	27.4
	2	58	48.9	73.4	46.8	70.1	44.6	66.9	42.4	63.6
		65	54.8	82.3	52.4	78.6	50.0	75.0	47.5	71.3
SSLP	1 1/4	58	17.4	26.1	15.2	22.8	13.1	19.6	10.9	16.3
		65	19.5	29.3	17.1	25.6	14.6	21.9	12.2	18.3
	2	58	43.5	65.3	41.3	62.0	39.2	58.7	37.0	55.5
		65	48.8	73.1	46.3	69.5	43.9	65.8	41.4	62.2
OVS	1 1/4	58	18.5	27.7	16.3	24.5	14.1	21.2	12.0	17.9
		65	20.7	31.1	18.3	27.4	15.8	23.8	13.4	20.1
	2	58	44.6	66.9	42.4	63.6	40.2	60.4	38.1	57.1
		65	50.0	75.0	47.5	71.3	45.1	67.6	42.7	64.0
LSLP	1 1/4	58	—	—	—	—	—	—	—	—
		65	—	—	—	—	—	—	—	—
	2	58	20.7	31.0	15.2	22.8	9.79	14.7	4.35	6.53
		65	23.2	34.7	17.1	25.6	11.0	16.5	4.88	7.31
LSLT	1 1/4	58	19.0	28.5	17.2	25.8	15.4	23.1	13.6	20.4
		65	21.3	32.0	19.3	28.9	17.3	25.9	15.2	22.9
	2	58	40.8	61.2	39.0	58.5	37.2	55.7	35.3	53.0
		65	45.7	68.6	43.7	65.5	41.6	62.5	39.6	59.4
STD, SSLT, SSLP, OVS, LSLP	$L_e \geq L_{e\,full}$	58	78.3	117	87.0	131	95.7	144	104	157
		65	87.8	132	97.5	146	107	161	117	176
LSLT	$L_e \geq L_{e\,full}$	58	65.3	97.9	72.5	109	79.8	120	87.0	131
		65	73.1	110	81.3	122	89.4	134	97.5	146
Edge distance for full bearing strength $L_e \geq L_{e\,full}$[a], in.		STD, SSLT, LSLT	2 7/8		3 3/16		3 1/2		3 13/16	
		OVS	3		3 5/16		3 5/8		3 15/16	
		SSLP	3		3 5/16		3 5/8		3 15/16	
		LSLP	3 11/16		4 1/16		4 1/2		4 7/8	

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	
$\Omega = 2.00$	$\phi = 0.75$	— indicates spacing less than minimum spacing required per AISC *Specification* Section J3.3. 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.