Steel Design

Notation:

= name for width dimension a = name for area \boldsymbol{A} = area of a bolt A_b = effective net area found from the A_e product of the net area A_n by the shear lag factor U = gross area, equal to the total area A_{ϱ} ignoring any holes = gross area subjected to shear for A_{gv} block shear rupture = net area, equal to the gross area A_n subtracting any holes, as is A_{net} = net area subjected to tension for A_{nt} block shear rupture = net area subjected to shear for block A_{nv} shear rupture = area of the web of a wide flange A_w section AISC = American Institute of Steel Construction ASD = allowable stress design = name for a (base) width = total width of material at a horizontal section = name for height dimension = width of the flange of a steel beam b_f cross section = factor for determining M_u for B_1 combined bending and compression = largest distance from the neutral caxis to the top or bottom edge of a beam = coefficient for shear stress for a c_1 rectangular bar in torsion C_b = modification factor for moment in ASD & LRFD steel beam design = column slenderness classification C_c constant for steel column design C_m = modification factor accounting for

D = shorthand for dead load DL= shorthand for dead load = eccentricity \boldsymbol{E} = shorthand for earthquake load = modulus of elasticity = axial compressive stress f_c = bending stress f_b = bearing stress f_p = shear stress f_{ν} $f_{v-max} = \text{maximum shear stress}$ = vield stress = shorthand for fluid load F F_a = allowable axial (compressive) stress = allowable bending stress F_b = critical unfactored compressive F_c stress for buckling in LRFD F_{cr} = flexural buckling stress F_e = elastic critical buckling stress F_{EXX} = yield strength of weld material = nominal strength in LRFD F_n = nominal tension or shear strength of a bolt = allowable bearing stress F_p F_t = allowable tensile stress = ultimate stress prior to failure F_u F_{ν} = allowable shear stress $F_{\rm y}$ = yield strength = yield strength of web material F_{vw} = factor of safety F.S. = gage spacing of staggered bolt g holes h = name for a height = height of the web of a wide flange h_c steel section H= shorthand for lateral pressure load Ι = moment of inertia with respect to neutral axis bending = moment of inertia of trial section I_{trial} $I_{req'd}$ = moment of inertia required at limiting deflection I_{v} = moment of inertia about the y axis = polar moment of inertia

= web shear coefficient

 C_{v}

 d_h

combined stress in steel design

= calculus symbol for differentiation

= depth of a wide flange section

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

 $M_{max-adj}$ = 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_{el} = 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

 $S_{req'd-adj}$ = 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	Z	= plastic section modulus of a steel
T	= torque (axial moment)		beam
	= shorthand for thermal load	Z_x	= plastic section modulus of a steel
	= throat size of a weld		beam with respect to the x axis
U	= shear lag factor for steel tension		$u_{al} = actual beam deflection$
	member design		p_{wable} = allowable beam deflection
U_{bs}	= reduction coefficient for block		$_{it}$ = allowable beam deflection limit
	shear rupture	Δ max	x = maximum beam deflection
V	= internal shear force	$\boldsymbol{\mathcal{E}}_{\mathrm{y}}$	= yield strain (no units)
V_a	= required shear (ASD)	ϕ	= resistance factor
	= maximum internal shear force	Υ	= diameter symbol
V_{max-a}	adj = maximum internal shear force	4	•
	adjusted to include self weight	ϕ_b	= resistance factor for bending for
V_n	= nominal shear strength capacity for		LRFD
	LRFD beam design	ϕ_c	= resistance factor for compression
V_u	= maximum shear from factored loads		for LRFD
	for LRFD beam design	ϕ_{t}	= resistance factor for tension for
W	= name for distributed load	•	LRFD
W_{adjus}	_{ted} = adjusted distributed load for	$\phi_{_{\scriptscriptstyle \mathcal{V}}}$	= resistance factor for shear for
	equivalent live load deflection limit	$\boldsymbol{\varphi}_{v}$	
W_{equiv}	alent = the equivalent distributed load		LRFD
	derived from the maximum bending	γ	= load factor in LRFD design
	moment	π	= pi (3.1415 radians or 180°)
W_{self} w	$_{rt}$ = name for distributed load from self	θ	= slope of the beam deflection curve
	weight of member	ρ	= radial distance
W	= shorthand for wind load	σ	= engineering symbol for normal
$\boldsymbol{\mathcal{X}}$	= horizontal distance		stress
X	= bearing type connection with	Ω	= safety factor for ASD
	threads excluded from the shear		-
	plane	J	= symbol for integration
y	= vertical distance	${\it \Sigma}$	= 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.

Materials

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

ARCH 614 Note Set 16 S2014abn

A36 – carbon steel used for plates, angles $F_y = 36 \text{ ksi}, F_u = 58 \text{ ksi}, E = 29,000 \text{ ksi}$ A572 – high strength low-alloy used for some beams A992 – for building framing used for most beams $F_y = 60 \text{ ksi}, F_u = 75 \text{ ksi}, E = 29,000 \text{ ksi}$ $F_y = 50 \text{ ksi}, F_u = 65 \text{ ksi}, E = 29,000 \text{ ksi}$ (A572 Grade 50 has the same properties as A992)

$$\underline{\text{ASD}} \qquad R_a \leq \frac{R_n}{\Omega}$$

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:

 $\begin{array}{ll} \text{bending (braced, $L_b < L_p$)} & \Omega = 1.67 \\ \text{bending (unbraced, $L_p < L_b$ and $L_b > L_r$)} & \Omega = 1.67 \text{ (nominal moment reduces)} \\ \text{shear (beams)} & \Omega = 1.5 \text{ or } 1.67 \\ \text{shear (bolts)} & \Omega = 2.00 \text{ (tabular nominal strength)} \\ \text{shear (welds)} & \Omega = 2.00 \end{array}$

- 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

$$\begin{array}{ll} \underline{LRFD} & R_u \leq \phi R_n & where \cdots R_u = \Sigma \gamma_i R_i \\ & \text{where} & \phi = \text{resistance factor} \\ & \gamma = \text{load factor for the type of load} \\ & R = \text{load (dead or live; force, moment or stress)} \\ & R_u = \text{factored load (moment or stress)} \\ & R_n = \text{nominal load (ultimate capacity; force, moment or stress)} \\ \end{array}$$

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

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

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

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

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

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

Determining Maximum Bending Moment

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

$$V = \Sigma(-w)dx$$

$$M = \Sigma(V)dx$$

$$\frac{dV}{dx} = -w$$

$$\frac{dM}{dx} = V$$

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

Roof or floor (damageable elements)

All building codes and design codes limit deflection for beam types and damage that could happen based on service condition and severity. $y_{max}(x) = \Delta_{actual} \le \Delta_{allowable} = \frac{L}{value}$

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

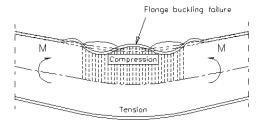
Lateral Buckling

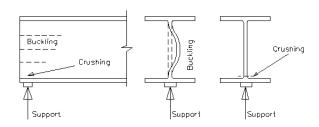
With compression stresses in the top of a beam, a sudden "popping" or buckling can happen even at low stresses. In order to prevent it, we need to brace it along the top, or laterally brace it, or provide a bigger I_{ν} .

L/480

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

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





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

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

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

where t_w = thickness of the web

 F_{vw} = yield strength of the web

N =bearing length

k = dimension to fillet found in beam section tables

$$\phi = 1.00 \, (LRFD)$$
 $\Omega = 1.50 \, (ASD)$



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 <u>tributary area</u> 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:

$$\Sigma \gamma_i R_i = M_u \le \phi_b M_n = 0.9 F_v Z$$

where

 M_u = maximum moment from factored loads

 ϕ_b = resistance factor for bending = 0.9

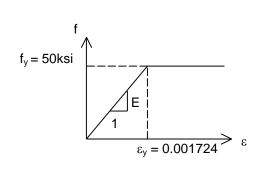
 M_n = nominal moment (ultimate capacity)

 F_v = yield strength of the steel

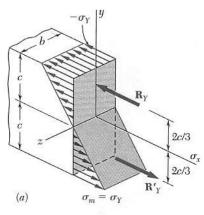
Z = plastic section modulus

Plastic Section Modulus

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



for resistance to bearing



Internal Moments and Plastic Hinges

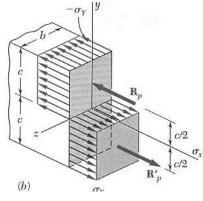
Plastic hinges can develop when all of the material in a cross section sees the yield stress. Because all the material at that section can strain without any additional load, the member segments on either side of the hinge can rotate, possibly causing instability.

For a rectangular section:

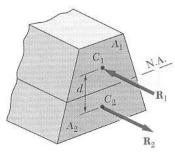
Elastic to
$$f_y$$

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

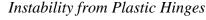
Fully Plastic:
$$M_{ult}$$
 or $M_p = bc^2 f_y = \frac{3}{2} M_y$

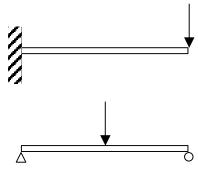


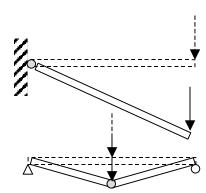
For a non-rectangular section and internal equilibrium at σ_v , the n.a. will not necessarily be at the centroid. The n.a. occurs where the $A_{tension} = A_{compression}$. The reactions occur at the centroids of the tension and compression areas.



 $A_{tension} = A_{compression}$







Shape Factor:

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

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

k = 3/2 for a rectangle

 $k \approx 1.1$ for an I beam

Plastic Section Modulus

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

Design for Shear

$$V_a \leq V_n / \Omega$$
 or $V_u \leq \phi_v V_n$

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

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

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

 V_n = nominal shear strength

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

 $A_w = t_w d = area of the web$

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

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

Design for Flexure

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

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

- 1. yielding, limited at length $L_p = 1.76r_y \sqrt{\frac{E}{F_y}}$, where r_y is the radius of gyration in y
- 2. lateral-torsional buckling (inelastic) limited at length L_{\star}
- 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 $1 < C_b \le 2.3$, the required flexural strength M_u can be reduced by dividing it by C_b . ($C_b = 1$ when the bending moment at any point within an unbraced length is larger than that at both ends of the length. C_b of 1 is conservative and permitted to be used in any case. When the free end is unbraced in a cantilever or overhang, $C_b = 1$. The full formula is provided below.)

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

Compact Sections

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

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

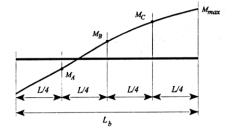
 $F_y = \text{minimum yield stress in ksi}$

 h_c = height of the web in inches

 t_w = web thickness in inches

With lateral-torsional buckling the nominal flexural strength is

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



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

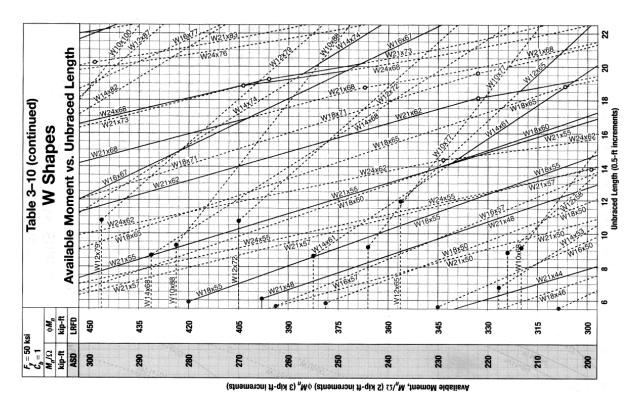
$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

 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 $M_a \leq M_n/\Omega$ or $M_u \leq \phi_b M_n$. The maximum moment that can be applied on a beam (taking self weight into account), M_a or M_u , can be plotted against the unbraced length, L_b . The limit L_p is indicated by a solid dot (\bullet), while L_r is indicated by an open dot (\bigcirc). 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 $Z_{\text{req'd}}$ when yielding is the limit state. This step is equivalent to determining if $f_b = \frac{M_{max}}{S} \le F_b$, $Z_{req'd} \ge \frac{M_{max}}{F_b} = \frac{M_{max}}{F_y}$ and $Z \ge \frac{M_u}{\phi_b F_y}$ to meet the design criteria that

$$M_a \leq M_n / \Omega$$
 or $M_u \leq \phi_b M_n$

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

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

TABLE 9.1 Load Factor Resistance Design Selection

			$F_y = 3$	86 ksi	
Designation	Z_x in. ³	L_p ft	$\frac{L_r}{\mathrm{ft}}$	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 \times 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:
$$f_{v-\text{max}} = \frac{3V}{2A} \approx \frac{V}{A_{web}} = \frac{V}{t_w d}$$

$$V_n = 0.6F_{yw}A_w \quad or \ V_n = 0.6F_{yw}A_w C_v$$
 Others:
$$f_{v-\text{max}} = \frac{VQ}{Ib}$$

- 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 $f_{v} = \frac{T\rho}{J} \text{ or } \frac{T}{c_{1}ab^{2}} \le F_{v} \text{ (circular section or rectangular)}$
- 9. Evaluate the deflection to determine if $\Delta_{maxLL} \leq \Delta_{LL-allowed}$ and/or $\Delta_{maxTotal} \leq \Delta_{Totalallowed}$

*** note: when $\Delta_{calculated} > \Delta_{limit}$, $I_{req'd}$ can be found with: $I_{req'd} \geq \frac{\Delta_{loobig}}{\Delta_{limit}}$, $I_{req'd} \geq \frac{\Delta_{loobig}}{\Delta_{limit}}$, 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 $M_{max} = \frac{W_{equivalent}L^2}{8}$ from the maximum moment equation:

If the deflection limit is less, the design live load to check against allowable must be increased, ex. $w_{adjusted} = w_{ll}$

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

Criteria for Design of Columns

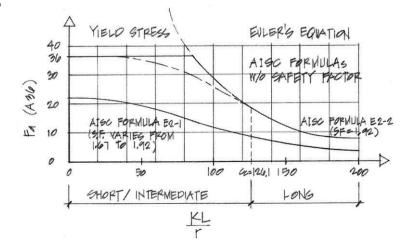
If we know the loads, we can select a section that is adequate for strength & buckling.

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 ($F_y = 36 \text{ ksi}$) and 107.0 ($F_y = 50 \text{ ksi}$) 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 <u>load</u> type

φ is a <u>resistance factor</u>

P_n is the <u>nominal load capacity (strength)</u>

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

For compression $P_n = F_{cr} A_g$

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

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

when
$$\frac{KL}{r} \le 4.71 \sqrt{\frac{E}{F_y}}$$
 or $(F_e \ge 0.44F_y)$:
$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}} \right] F_y$$
when $\frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}}$ or $(F_e < 0.44F_y)$:
$$F_{cr} = 0.877 F_e$$

where
$$F_e$$
 is the elastic critical buckling stress:

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

Design Aids

Ē

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

m',	= 50 ksi	⋖	Ava Axial			trer essi		in Kips	(A		r
				57	W Shapes	apes				W12	
Shape	2					¥	W12×				3
W/W	#	6	96	87	2	7	62	1	72	9	65
Dec	Deelon	P_n/Ω_c	φ _c P _n	P_{μ}/Ω_{c}	$\phi_c P_n$	P,102	φ _c P _n	P_n/Ω_c	φ.P.	P_n/Ω_c	$\phi_c P_n$
S	5	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	0	844	1270	992	1150	694	1040	633	951	175	859
4	91	811	1220	735	1110	299	1000	209	913	548	824
no r	> 	287	1180	3 2	1070	646	987	28 28	884	531	798
yrati	on ⊊	772	1160	669	1050	634	952	577	867	520	782
g to	2 =	739	1110	699	1010	909	910	35.5	828	497	747
snib	7	720	1080	652	086	290	887	537	807	484	727
er i	<u>e</u> :	707	1050	634	953	573	862	225	784	470	902
see	4 to	920	1020	615 595	924	238	836	96	736	456	685
ot :	9	637	957	575	864	520	781	473	710	425	639
bec	4	614	923	554	833	201	752	455	684	409	615
resi	∞ 9	591	888	233	801	481	723	437	657	393	591
rttiv	2 8	543	852	490	736	461	694 664	401	630 603	360	541
v (M)	8	495	744	446	029	402	603	365	548	327	491
KT	24	447	672	405	909	362	544	328	493	294	442
цьб	8 8	401	602	360	541	323	486	283	380	262	393
uəj	8 8	312	469	279	420	220	376	528	340	202	303
элдэ	8	274	.412	246	369	220	331	199	299	177	267
oey	ਲ	243	365	218	327	195	293	176	265	157	236
3	% %	217	326	194	292	174	261	157	236	140	211
	3 8	176	264	157	236	<u>4</u>	212	127	191	114	171
					Properties	rties					
" (kips)		137	206	121	181	104	157	6.06	136	78.2	117
(kips/i	<u>2</u>	18.3	27.5	17.2	25.8 366	15.7	23.5	14.3	21.5	13.0	19.5
(kips)		152	228	123	185	101	152	84.0	126	68.5	103
€€		- 4	10.9	- 4	10.8	- 6	39.9	- 6	10.7	- 6	11.9
Ag (in. ²) / (in. ⁴)		83	28.2 833	25. 740	25.6	66	23.2 662	59	21.1 597 105	53	19.1
ale (3)	r, (in.) Ratio r, /r, P_, (KL ²)/10 ⁴ (K-in. ²)	83	3.09	3. 3. 1. 21200	3.07 1.75	3. 1. 18900	3.05 1.75 0	3. 1. 17100	3.04 1.75	1530	3.02
(KL ²)/1	04 (k-in.2)		Q	0069	0	6180	0	2280	8	4980	0
ASD	0	LREG	9								

Procedure for Analysis

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

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

Procedure for Design

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

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

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

Is $f_c \le F_{cr}/\Omega$ 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:

$$\frac{P_a}{P_n}$$
 ·100% or $\frac{P_u}{\phi_c P_n}$ ·100%

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

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

Columns with Bending (Beam-Columns)

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

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

Design for Combined Compression and Flexure:

The interaction of compression and bending are included in the form for two conditions based on the size of the required axial force to the available axial strength. This is notated as P_r (either P from ASD or P_u from LRFD) for the axial force being supported, and P_c (either P_n/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
$$\frac{P_{r}}{P_{c}} \ge 0.2$$
: $\frac{P}{P_{n}/\Omega} + \frac{8}{9} \left(\frac{M_{x}}{M_{nx}/\Omega} + \frac{M_{y}}{M_{ny}/\Omega} \right) \le 1.0$ $\frac{P_{u}}{\phi_{c}P_{n}} + \frac{8}{9} \left(\frac{M_{ux}}{\phi_{b}M_{nx}} + \frac{M_{uy}}{\phi_{b}M_{ny}} \right) \le 1.0$ (ASD) (LRFD)

For $\frac{P_{r}}{P_{c}} < 0.2$: $\frac{P}{2P_{n}/\Omega} + \left(\frac{M_{x}}{M_{nx}/\Omega} + \frac{M_{y}}{M_{ny}/\Omega} \right) \le 1.0$ $\frac{P_{u}}{2\phi_{c}P_{n}} + \left(\frac{M_{ux}}{\phi_{b}M_{nx}} + \frac{M_{uy}}{\phi_{b}M_{ny}} \right) \le 1.0$ (ASD) (LRFD)

where:

$$\begin{array}{ll} \text{for compression} & \underline{\varphi_c} = 0.90 \text{ (LRFD)} & \Omega = 1.67 \text{ (ASD)} \\ \text{for bending} & \underline{\varphi_b} = 0.90 \text{ (LRFD)} & \Omega = 1.67 \text{ (ASD)} \end{array}$$

For a <u>braced</u> condition, the moment magnification factor B_I is determined by $B_1 = \frac{C_m}{1 - (P_u/P_{el})} \ge 1.0$

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

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

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*.

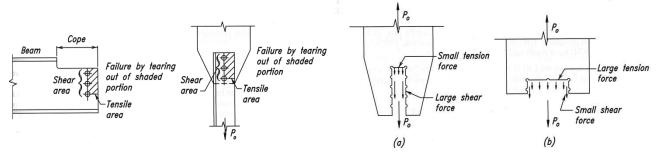


Fig. C-J4.1. Failure for block shear rupture limit state.

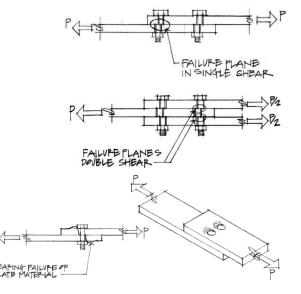
Fig. C-J4.2. Block shear rupture in tension

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 (Group A)

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

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

Slip-critical connections: bolts are torqued to a high tensile stress in the shank, resulting in a clamping force on the connected parts. (Shear resisted by clamping force). Requires inspections and is useful for structures seeing dynamic or fatigue loading. Class A indicates the *faying* (contact) surfaces are clean mill scale or adequate paint system, while Class B indicates blast cleaning or paint for $\mu = 0.50$.

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

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

Design for Bolts in Bearing, Shear and Tension

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

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

For shear OR tension (same equation) in bolts:

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

where
$$R_u = \sum \gamma_i R_i$$

• single shear (or tension)
$$R_n = F_n A_b$$

$$R_n = F_n 2A_b$$

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

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

For bearing of plate material at bolt holes:

$$R_a \le R_n / \Omega$$
 or $R_u \le \phi R_n$
where $R_u = \sum \gamma_i R_i$

• deformation at bolt hole is a concern

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

deformation at bolt hole is not a concern

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

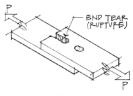


Figure 10.11 End tear-out.

• long slotted holes with the slot perpendicular to the load

$$R_{y} = 1.0 L_{o} t F_{y} \le 2.0 dt F_{y}$$

where

 R_n = the nominal bearing strength

 F_u = specified minimum tensile strength

 L_c = clear distance between the edges of the hole and the next hole or edge in the direction of the load

d = nominal bolt diameter

t = thickness of connected material

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

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 $1\frac{1}{4}$ in..

Tension Member Design

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

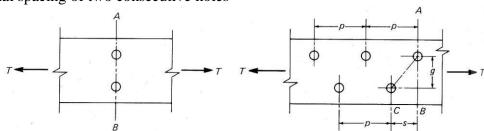
g refers to the row spacing or gage

- p refers to the bolt spacing or pitch
- s refers to the longitudinal spacing of two consecutive holes

Effective Net Area:

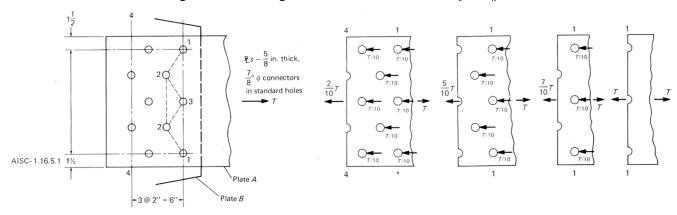
The smallest effective are must be determined by subtracting the bolt hole areas. With

staggered holes, the shortest length must be evaluated.



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

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



For tension elements:

$$R_a \le R_n / \Omega$$
 or $R_u \le \phi R_n$
where $R_u = \sum \gamma_i R_i$

$$R_n = F_y A_g$$

$$\phi = 0.90 \text{ (LRFD)}$$

$$\Omega = 1.67 \text{ (ASD)}$$

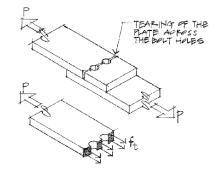
2. rupture

$$R_n = F_u A_e$$

$$\phi = 0.75 \text{ (LRFD)}$$

$$\Omega = 2.00 \text{ (ASD)}$$

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



 F_y = the yield strength of the steel

 F_n = 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: $A_n = bt - \Sigma ht - \Sigma \left(\frac{s^2}{4g}\right)t$

where b is the plate width

t is the plate thickness

h is the standard hole diameter of each hole

s is the staggered hole spacing

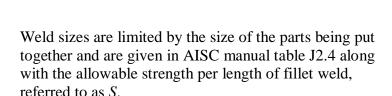
g is the gage spacing between rows

Welded Connections

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

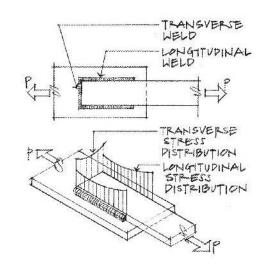
The throat size, T, of a fillet weld is determined trigonometry by: $T = 0.707 \times \text{weld size}^*$

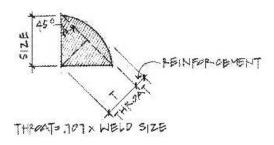
* When the submerged arc weld process is used, welds over 3/8" will have a throat thickness of 0.11 in. larger than the formula.



The *maximum* size of a fillet weld:

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





The *minimum length* of a fillet weld is 4 times the nominal size. If it is not, then the weld size used for design is ½ 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 Over ¼ to ½	1/8 3/16 1/4
Over ½ to ¾ Over ¾	9/4 5/16

	C.11 .	1 1	
For	tillet	welds:	

$R_a \leq R_n$	Ω	or R_u	$\leq \phi R_n$
where	R_{u}	$=\Sigma\gamma_i$	R_i

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

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

where:

T is throat thickness l is length of the weld

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

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

BEAM

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

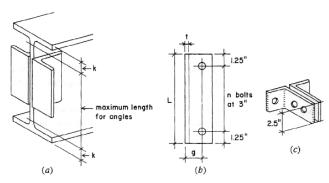
Framed Beam Connections

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

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

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

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



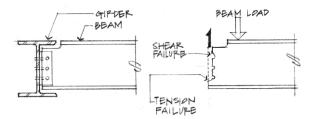
Sample AISC Table for Bolt and Angle Available Strength in All-Bolted Double-Angle Connections

8	$F_U = 65 \text{ ksi}$	(3)	₹	-Bolted Double-Ai	S	b	20	碞	All-Bolted Double-Angle	V	<u>e</u>	16 70 21	ار 4	3/4-in
algr rr ₂	, = 36 ksi				ŏ	Ē	Connections	Ę	ns	•			Bolts	ts .
À آع	, = 58 ksi	053	mgm.	अपह अप	8	alt and	Angle	Availab	Bolt and Angle Available Strength, kips	ngth, ki	sd	30 30 30		3 P.
4	4 Rows	Bolt		Thread	Ĭ	Hole	o lo	3	An	Angle Thickness, in.	kness	Ë	ROWS	
No.	21 10 16	Group	8	Cond.	-	Jype a	-05	1,4	6	5/16	3,	3/8	100	1/2
478	W24, 21, 10, 10	0.83		1884	848	325	ASD	ASD LRFD	ASD	ASD LRFD	ASD		ASD	
				z >	s v	E E	67.1	5 5	83.9	126	95.5	143	95.5	143
					3 6	OT.	200	75.0		75.0	909	75.0	202	
		Group	S	သွ	, 6	ONS	43.1	64.5		64.5	43.1	64.5	43.1	
4	88	A	Clar	Class A	8	SSLT	50.6	75.9	.HPR2	75.9	50.6	75.9	50.6	-1.
1:	6-6	2	0	5	S	STD	67.1	101	17753	126	84.4	127	84.4	
	Ør .		Clas	Class B	۰ <i>و</i>	SNO	65.3	97.9	71.9	108	71.9	108	71.9	108
1			-		N O	STD	67.1	101	83.0	126	1 0	151	120	
7 6			_	: ×	o io	STD	67.1	101	83.9	126	5	151	134	201
- coc	~		"	5	S	STD	63.3	94.9		94.9	63.3	94.9	63.3	94.9
10-7		Group	Clas	Class A	68	SNO	53.9	80.7		80.7	53.9	80.7	53.9	80.7
		n			8 0	STO	67.1	101	03.3	126	101	151	105	
			S	သ	, 6	SNO	65.3	97.9	On land	122	89.9	134	89.9	
			Clas	Class B	SS	SSLT	65.8	98.7		123	98.7	148	105	
		Be	am We	b Avail	able St	rength	per In	ch Thic	Beam Web Available Strength per Inch Thickness, kips/in.	kips/ir				
	22.0			STD	و		L	6	OVS			SSLT	5	
	HOIE 13PE			=				Leh	Leh*, in.					
		-	11/2	12	7	13/4	-	11/2	7	13/4	=	11/2	-	13/4
		OBA.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
		11/4	167	250	175	262	156	234	164	246	164	245	172	257
č	14.4	17/8	1 69	254	//	907	200	238	167	250	99 9	249	174	197
3 6	Coped at 10p	15/0	174	261	195	273	163	245	5 5	257	20 1	256	1/1	269
	din a		1 2	222	180	284	3 5	256	- 2	268	- 42	267	186	279
		۰ ۳	201	305	209	313	190	285	198	297	86	296	206	309
		1/4	156	234	156	234	146	219	146	219	156	234	156	234
		13/8	191	241	161	241	151	227	151	227	161	241	161	241
8	Coped at Both	11/2	166	249	166	249	156	234	156	234	166	249	166	249
ᄄ	- Hanges	15/8	171	256	171	256	161	241	161	241	171	526	171	256
		7	181	272	185	278	171	256	176	263	178	267	185	278
	ASA 1888	3	201	301	209	313	190	285	198	297	198	596	206	309
	Nucoped	37.5	234	321	234	351	234	351	234	351	234	321	234	351
<u>B</u> ~ <u>≡</u>	Support Available Strength per Inch Thickness, kips/in.	e "	Notes: STD = 0VS = SSLT =	votes: STD = Standard holes OVS = Oversized holes SSLT = Short-slotted ho to direction of lo	Standard holes Oversized holes Short-slotted holes to direction of load	Standard holes Oversized holes Short-slotted holes transverse to direction of load	sverse		N = Th N = Th SC = Sig	N = Threads included X = Threads excluded SC = Slip critical	papnic			
Type Type	ASD	LRFD	* Tabuk	ated valu	les inclu	de 1/4-in	r. reducti	ion in en	Tabulated values include ¹ /4-in, reduction in end distance, L _{eh} , to account for possible	3e, Len, to	o accour	nt for pos	sible	Hate Sugar
STD/ OVS/ SSLT	ded or both	702	Note: Si been ad	under un in bean rengun. Note: Slip-critical bolt values assume no mo been added to distribute loads in the fillers.	al bolt valistribute	gui. alues as: 3 loads ii	sume no	more the	uncustrial in beam rengal. With soft was Selected as selected or botts have with a been provided or botts have been added to its simple loads in the fillers.	iller has	peen pri	ovided or	bolts h	ave

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



AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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Block Shear Strength (or Rupture):

$$R_a \le R_n / \Omega$$
 or $R_u \le \phi R_n$
where $R_u = \sum \gamma_i R_i$

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

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

where:

 A_{nv} is the net area subjected to shear A_{nt} is the net area subjected to tension

 A_{gy} 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)

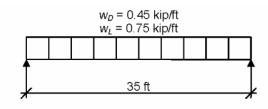
Example 2. A simple beam consisting of a W 21×57 is subjected to bending. Find the limiting moments (a) based on elastic stress conditions and a limiting stress of $F_y = 36$ ksi, and (b) based on full development of the plastic moment.

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.

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

 $F_v = 50 \text{ ksi}$

 $F_u = 65 \text{ ksi}$

- 1. The unbraced length is 0 because it says it is fully braced.
- 2. Find the maximum shear and moment from unfactored loads:

 $w_a = 0.450 \text{ k/ft} + 0.750 \text{ k/ft} = 1.20 \text{ k/ft}$

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

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

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

3. Find Zreg'd:

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

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

W18 x 40 has a plastic section modulus of 78.4 in³ and is the most light weight (as indicated by the bold text) in Table 9.1

Include the self weight in the maximum values:

$$w^*_{a-adjusted} = 1.20 \text{ k/ft} + 0.04 \text{ k/ft}$$

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

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

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

Evaluate shear (is V_a ≤ V₀/Ω): A_w = dt_w so look up section properties for W18 x 40: d = 17.90 in and t_w = 0.315 in

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

9. Evaluate the deflection with respect to the limit stated of L/360 for the live load. (If we knew the **total** load limit we would check that as well). The moment of inertia for the W18 x 40 is needed. I_x = 612 in⁴

 Δ live load limit = 35 ft(12 in/ft)/360 = 1.17 in

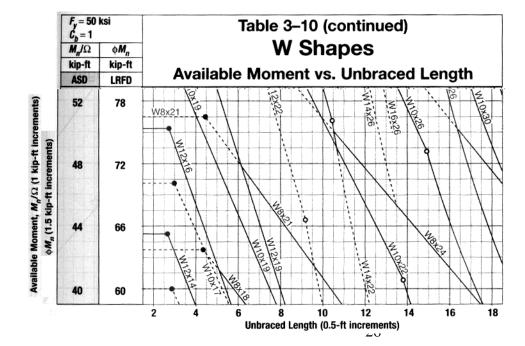
 Δ = 5wL⁴/384EI = 5(0.75 k/ft)(35 ft)⁴(12 in/ft)³/384(29 x 10³ ksi)(612 in⁴) = 1.42 in! This is TOO BIG (not less than the limit. Find the moment of inertia needed:

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

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 in⁴)

Choose a W18 x 50

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.



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):

$$M_{u} = \frac{wl^{2}}{2} + Pb = \frac{3.07 \frac{k}{ft} (20 ft)^{2}}{2} + 4.8k(10 ft) = 662^{k-ft} \quad V_{u} = wl + P = 3.07 \frac{k}{ft} (20 ft) + 4.8k = 66.2k$$

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

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

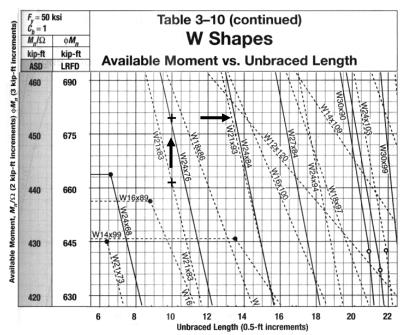
Replot 680.2 k-ft vs. 10ft, which lands *above* the capacity of the W21x83. We can't look up because the chart ends, but we can look for that capacity with a longer unbraced length. This leads us to a **W24** x **84** as the most economical. (With the additional self weight of 84 - 76 lb/ft = 8 lb/ft,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

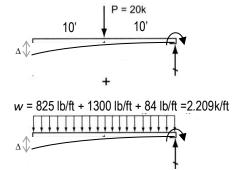
Evaluate the shear capacity:

$$\phi_v V_n = \phi_v 0.6 F_{vw} A_w = 1.0(0.6) 50 ksi(24.10 in) 0.47 in = 338.4 k$$
 so yes, 68 k \le 338.4 k

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.

$$\Delta_{total} = \frac{Pb^2(3l-b)}{6EI} + \frac{wL^4}{24EI} = \frac{3k(10ft)^{12}(3\cdot20-10ft)(12\frac{i\eta_{ft}}{10})^3}{6(30x10^3ksi)2370in^3} + \frac{(2.209\frac{k_{ft}}{10})(20ft)^4(12\frac{i\eta_{ft}}{10})^3}{24(30x10^3ksi)2370in^3} = 0.06 + 0.36 = 0.42in$$





P = 1.6(3k) = 4.8k

w = 1.2(825 lb/ft) + 1.6(1300 lb/ft) = 3.07 k/ft

So, Δ LL \leq Δ LL-limit and Δ total \leq Δ total-limit:

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

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

∴ FINAL SELECTION IS W24x84

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/ft² and a live load of 100 lb/ft². 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 Co	mputat	ion of	Truss Jo	int Loads							
		<u>area</u> W _{dead}	<u>loads</u> w	/live	tributary Node- to- Node Spacing	Truss- to- Truss Spacing	Floor Area per Node A	P _{dead} (=W _{dead} ⋅ A)	P _{live} (=w _{live} ⋅ A)	Factored Dead Load 1.2 · P _{dead}	Factored Live Load 1.6 • P _{live}	Factored Total Load 1.2 [•] P _{dead} + 1.6 [•] P _{live}
Truss	(#/ft ²)	(K/ft²)	(#/ft²)	(K/ft ²)	(ft)	(ft)	(ft²)	(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 $1.2P_{\text{dead}} = 1.2w_{\text{dead}} \cdot tributary \ width = 0.14 \ \text{K}$

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

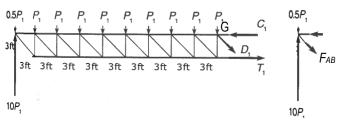
NOTE - end panels only have half the tributary width of interior panels 0.5P₁, P₁, P₁, P₁, P₁, P₂, P₃ 20 bays \times 3 ft = 60 ft 10P,

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

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

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

FBD 1 for 3 ft deep truss



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

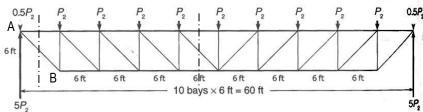
$$\Sigma M_G = 9.5P_1(30^{\hat{n}}) - P_1(27^{\hat{n}}) - P_1(24^{\hat{n}}) - P_1(21^{\hat{n}}) - P_1(18^{\hat{n}}) - P_1(15^{\hat{n}}) - P_1(12^{\hat{n}}) - P_1(9^{\hat{n}}) - P_1(6^{\hat{n}}) - P_1(3^{\hat{n}}) - T_1(3^{\hat{n}}) = 0$$

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

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

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

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

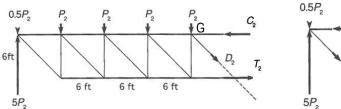


FBD 6: Maximum web force will be in the end diagonal

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

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

FBD 4 for 6 ft deep truss



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

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

 $T_2 = P_2(75^{ft})/6^{ft} = (7 \text{ k})(12.5) = 87.5 \text{ k}$
 $\Sigma F_V = 5P_2 - 4.5P_1 - D_s \cdot \sin 45^\circ = 0$

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

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

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

 $C_2 = 92.4 k$

Example 7 (pg 339)

(from unfactored loads)

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

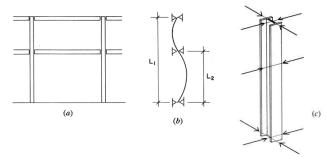
are 2 ft [0.61 m] on center, and deflection is limited to 1/240 of the span under total load and 1/360 of the span under live load only. Determine the lightest possible joist and the lightest joist of least depth possible.

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



Example 9 (pg 361)

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

Example 10

Investigate the accepatiblity of a W16 x 67 used as a beam-column under the unfactored loading shown in the figure. It is A992 steel ($F_v = 50 \text{ ksi}$). 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-ft) + 1.6(0.75)(60 k-ft) = 90 k-ft

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

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

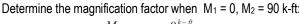
$$P_c = \phi_c P_n = \phi_c F_{cr} A_g = (30.5 ksi) 19.7 in^2 = 600.85 k$$

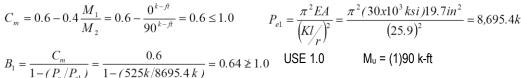
$$\frac{P_r}{P_c} = \frac{525k}{600.85k} = 0.87 > 0.2 \quad \text{so use} \quad \frac{P_u}{\phi_c P_n} + \frac{8}{9} \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \le 1.0$$

There is no bending about the v 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_0 at L_b =15 ft is 422 k-ft.

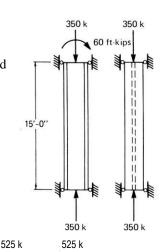




Finally, determine the interaction value:

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

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



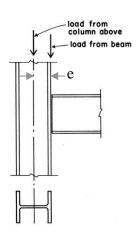
525 k

90 k-ft

15'-0"

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.

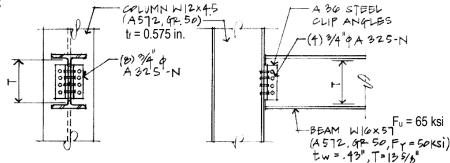


Figure 10.28 Typical beam-column connection.

10.2 The butt splice shown in Figure 10.22 uses two 8 x $\frac{3}{8}$ " plates to "sandwich" in the $8 \times \frac{1}{2}$ " plates being joined. Four \%"\phi 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.

CENTER PLATE

SOLUTION:

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

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

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

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

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

$$\phi R_0 = 91.4 \text{ k/bolt/in. x } 0.5 \text{ in. x } 4 \text{ bolts} = 182.8 \text{ k}$$

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

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

$$A_g = 8 \text{ in. } x \frac{1}{2} \text{ in. } = 4 \text{ in}^2$$

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

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

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

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

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

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

$$\phi R_n = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \le \phi(0.6F_v A_{qv} + U_{bs} F_u A_{nt})$$

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.

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

$$A_{nv} = A_{gv} - 1 \frac{1}{2}$$
 holes area = 3 in² - 1.5 x 1 in. x $\frac{1}{2}$ in. = 2.25 in²

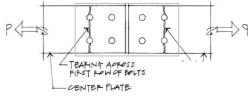
$$A_{nt}$$
 = 3.5 in. x t – 1 holes = 3.5 in. x $\frac{1}{2}$ in – 1 x 1 in. x $\frac{1}{2}$ in. = 1.25 in²

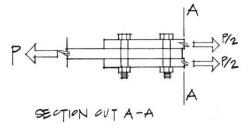
$$\phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) = 0.75 \text{ x } (0.6 \text{ x } 58 \text{ ksi x } 2.25 \text{ in}^2 + 1 \text{ x } 58 \text{ ksi x } 1.25 \text{ in}^2) = 113.1 \text{ k}$$

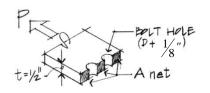
$$\phi(0.6F_vA_{gv} + U_{bs}F_vA_{nt}) = 0.75 \text{ x } (0.6 \text{ x } 36 \text{ ksi x } 3 \text{ in}^2 + 1 \text{ x } 58 \text{ ksi x } 1.25 \text{ in}^2) = 103.0 \text{ k}$$

The maximum connection capacity is governed by block shear rupture.

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

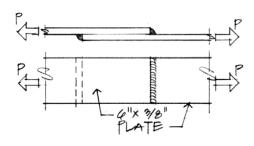






4 in. 2 in.

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



Example 15

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

Solution:

Capacity of weld:

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

Weld length = 22"

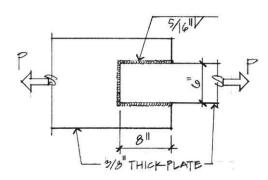
Weld capacity = $22'' \times 6.96$ k/in = 153.1 k

Capacity of plate: $0.9 \times 36 \text{ k/in}^2 \times 3/8'' \times 6'' = 72.9 \text{ k}$

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

Plate capacity = $0.9 \times 36 \text{ k/in}^2 \times 3/8'' \times 6'' = 72.9 \text{ k}$

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



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

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

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

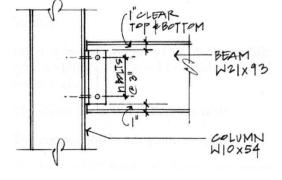
From Available Strength table, use 3/16'' weld $(\phi S = 4.18 \text{ k/in.})$

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

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 =
$$21.62 \text{ in} - 2(0.93 \text{ in}) - 2(1 \text{ in}) = 17.76 \text{ in}$$
.

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

Available length
$$\geq$$
 1.25 in. + 1.25 in. + 3 in. x (number of bolts – 1)
number of bolts \leq (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 $\frac{3}{4}$ ", $\frac{7}{8}$ ", and 1" bolt diameters and list angle thicknesses of $\frac{1}{4}$ ", $\frac{5}{16}$ ", $\frac{3}{8}$ ", and $\frac{1}{2}$ ". Increasing the angle thickness is likely to increase the angle strength, although the limit states include shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Beam	<i>F_y</i> = 50 ksi <i>F_u</i> = 65 ksi	010	Ta All-B e	ible 10 olted	•			-	jle	: 0 ::02	⁷ /8	-in.
Angle	$F_y = 36 \text{ ksi}$ $F_u = 58 \text{ ksi}$		17.00	Con						n 9	Bol	ts
⋖	1 u = 00 Koi	1.50	Olympia V.	Bolt and	Angle .	Availab	le Stre	ngth, k	ips			1,500
	6 Rows	Bolt	Thread	Hole			An	gle Thi	ckness	, in.	entit i	
W40	0, 36, 33, 30, 27,	Group	Cond.	Туре	1	/4	5	16	3	/8	,88 .6 1	/2
	24, 21	Стопр	oonu.	.,,,,	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	5 v 1 m .		N	STD	98.6	148	123	185	148	222	195	292
			X	STD	98.6	148	123	185	148	222	197	296
			SC	STD	98.6	148	106	159	106	159	106	159
	Varies	Group	Class A	OVS	90.1	135	90.1	135	90.1	135	90.1	135
F		A	Class A	SSLT	97.3	146	106	159	106	159	106	159
	81 - 800		SC	STD	98.6	148	123	185	148	222	176	264
			Class B	OVS	93.5	140	117	175	140	210	150	225
	3 max.			SSLT	97.3	146	122	182	146	219	176	264
31	Т"	337	N	STD	98.6	148	123	185	148	222	197	296
1	1 188		X	STD	98.6	148	123	185	148	222	197	296
563 = 15	1	881	99	STD	98.6	148	123	185	133	199	133	199
8		Group	SC Class A	OVS	93.5	140	113	169	113	169	113	169
19	-	В	Class A	SSLT	97.3	146	122	182	133	199	133	199
			SC	STD	98.6	148	123	185	148	222	197	296
			Class B	OVS	93.5	140	117	175	140	210	187	281
		12	Class D	SSLT	97.3	146	122	182	146	219	195	292

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

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

 $\phi R_n = 367.8$ kips for double shear of 7/8" bolts $\phi R_n = 296$ 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.

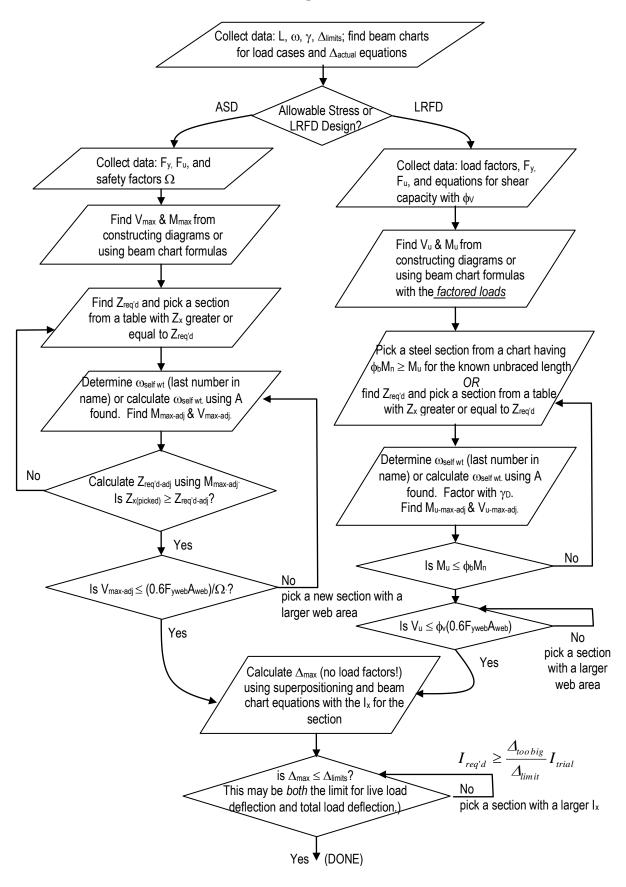
$$\phi R_n = 6 \text{ bolts} \cdot (102 \text{ k/bolt/inch}) \cdot (0.58 \text{ in}) = 355.0 \text{ kips}$$

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

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

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

Beam Design Flow Chart



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

$Z_x - US$	$I_x - US$	l	$I_x - SI$	7 (1	7 110	$I_x - US$		$I_x - SI$	$Z_x - SI$
$(in.^3)$	$\lim_{x \to 0.5} (in.^4)$	Section	(10^6mm.^4)	$\frac{Z_{x} - SI}{(10^{3} \text{mm.3})}$	$Z_x - US$ (in. ³)	(in. ⁴)	Section	(10^6mm.^4)	(10^3mm.3)
514	7450	W33X141	3100	8420	289	3100	W24X104	1290	4740
511	5680	W24X176	2360	8370	287	1900	W14X159	791	4700
509	7800	W36X135	3250	8340	283	3610	W30X90	1500	4640
500	6680	W30X148	2780	8190	280	3000	W24X103	1250	4590
490	4330	W18X211	1800	8030	279	2670	W21X111	1110	4570
487	3400	W14X257	1420	7980	278	3270	W27X94	1360	4560
481	3110	W12X279	1290	7880	275	1650	W12X170	687	4510
476	4730	W21X182	1970	7800	262	2190	W18X119	912	4290
468	5170	W24X162	2150	7670	260	1710	W14X145	712	4260
467	6710	W33X130	2790	7650	254	2700	W24X94	1120	4160
464	5660	W27X146	2360	7600	253	2420	W21X101	1010	4150
442	3870	W18X192	1610	7240	244	2850	W27X84	1190	4000
437	5770	W30X132	2400	7160	243	1430	W12X152	595	3980
436	3010	W14X233	1250	7140	234	1530	W14X132	637	3830
432	4280	W21X166	1780	7080	230	1910	W18X106	795	3770
428	2720	W12X252	1130	7010	224	2370	W24X84	986	3670
418	4580	W24X146	1910	6850	221	2070	W21X93	862	3620
415	5900	W33X118	2460	6800	214	1240	W12X136	516	3510
408	5360	W30X124	2230	6690	212	1380	W14X120	574	3470
398	3450	W18X175	1440	6520	211	1750	W18X97	728	3460
395	4760	W27X129	1980	6470	200	2100	W24X76	874	3280
390	2660	W14X211	1110	6390	198	1490	W16X100	620	3240
386	2420	W12X230	1010	6330	196	1830	W21X83	762	3210
378	4930	W30X116	2050	6190	192	1240	W14X109	516	3150
373	3630	W21X147	1510	6110	186	1530	W18X86	637	3050
370	4020	W24X131	1670	6060	186	1070	W12X120	445	3050
356	3060	W18X158	1270	5830	177	1830	W24X68	762	2900
355	2400	W14X193	999	5820	175	1300	W16X89	541	2870
348	2140	W12X210	891	5700	173	1110	W14X99	462	2830
346	4470	W30X108	1860	5670	172	1600	W21X73	666	2820
343	4080	W27X114	1700	5620	164	933	W12X106	388	2690
333	3220	W21X132	1340	5460	163	1330	W18X76	554	2670
327	3540	W24X117	1470	5360	160	1480	W21X68	616	2620
322	2750	W18X143	1140	5280	157	999	W14X90	416	2570
320	2140	W14X176	891	5240	153	1550	W24X62	645	2510
312	3990	W30X99	1660	5110	150	1110	W16X77	462	2460
311	1890	W12X190	787	5100	147	833	W12X96	347	2410
307	2960	W21X122	1230	5030	147	716	W10X112	298	2410
305	3620	W27X102	1510	5000	146	1170	W18X71	487	2390
290	2460	W18X130	1020	4750				((continued)

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

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

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

KL/r	$\phi_c F_{cr}$								
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	$\phi_c F_{cr}$								
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

No	minal Bolt	Diamete	er, <i>d</i> , in.		5	/8	3,	/4	7	/8	1	1
	Nominal B	olt Area	, in.²	P 196	0.3	307	0.4	142	0.6	601	0.	785
ASTM	Thread	F_{nv}/Ω (ksi)	φ <i>F_{nv}</i> (ksi)	Load-	r_n/Ω	φr _n	r _n /Ω	φ r _n	r _n /Ω	φ r _n	r _n /Ω	φrn
Desig.	Cond.	ASD	LRFD	ing	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group	ve N ()	27.0	40.5	S D	8.29 16.6	12.4 24.9	11.9 23.9	17.9 35.8	16.2 32.5	24.3 48.7	21.2 42.4	31.8 63.6
A .E.) - X qq	34.0	51.0	S D	10.4 20.9	15.7 31.3	15.0 30.1	22.5 45.1	20.4 40.9	30.7 61.3	26.7 53.4	40.0 80.1
Group	N N	34.0	51.0	S D	10.4 20.9	15.7 31.3	15.0 30.1	22.5 45.1	20.4 40.9	30.7 61.3	26.7 53.4	40.0 80.1
рш В) ш	X X	42.0	63.0	S D	12.9 25.8	19.3 38.7	18.6 37.1	27.8 55.7	25.2 50.5	37.9 75.7	33.0 65.9	49.5 98.9
A307	gr _Gam	13.5	20.3	S D	4.14 8.29	6.23 12.5	5.97 11.9	8.97 17.9	8.11 16.2	12.2 24.4	10.6 21.2	15.9 31.9
nd No	minal Bolt	Diamete	er, <i>d</i> , in.	ons to	01	1/8 200	mmT ₁	y a gmia	" (4	3/8	.a. n	1/2
	Nominal B	olt Area	, in.²	01 140E IQ	0.9	994	i staday	23	1.	48	1. 100	.77
ASTM	Thread	F _{nv} /Ω (ksi)	φ <i>F_{nv}</i> (ksi)	Load-	r_n/Ω	φr _n	r _n /Ω	φ r _n	r_n/Ω	φ r _n	r _n /Ω	φ r _n
Desig.	Cond.	ASD	LRFD	ing	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group	N	27.0	40.5	S D	26.8 53.7	40.3 80.5	33.2 66.4	49.8 99.6	40.0 79.9	59.9 120	47.8 95.6	71.7 143
A	x	34.0	51.0	S D	33.8 67.6	50.7 101	41.8 83.6	62.7 125	50.3 101	75.5 151	60.2 120	90.3 181
Group	N	34.0	51.0	S D	33.8 67.6	50.7 101	41.8 83.6	62.7 125	50.3 101	75.5 151	60.2 120	90.3 181
В	. x	42.0	63.0	S D	41.7 83.5	62.6 125	51.7 103	77.5 155	62.2 124	93.2 186	74.3 149	112 223
A307	-	13.5	20.3	S D	13.4 26.8	20.2 40.4	16.6 33.2	25.0 49.9	20.0 40.0	30.0 60.1	23.9 47.8	35.9 71.9
ASD	LRFD	For end	loaded co	onnections	greater t	han 38 in	., see AISO	Specification Sp	ation Tabl	e J3.2 foo	otnote b.	. 4
2 = 2.00	o = 0.75											

Table 7-2 Available Tensile Strength of Bolts, kips

Nominal Bo	olt Diameter,	d, in.	5,	8	3	/4	7	/8		1	
Nomina	l Bolt Area, in	.2	0.3	07	0.	442	0.0	501	0.3	785	
ASTM Desig	F_{nt}/Ω (ksi)	φ <i>F_{nt}</i> (ksi)	r_n/Ω	φ r _n	r_n/Ω	φ r _n	r_n/Ω	φ r _n	r_n/Ω	φ r _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 B	olt Diameter,	d, in.	8. 8.4	/8	2 1	1/4	781 1	3/8	145-1	1/2	
Nomina	l Bolt Area, ir	1.2	0.9	0.994 1.23		1.23		48	- 1.	.77	
ASTM Desig	F_{nt}/Ω (ksi)	φ <i>F_{nt}</i> (ksi)	r _n /Ω	φ r _n	r_n/Ω	φ r _n	r_n/Ω	φ r _n	r _n /Ω	φr _n	
200000000000000000000000000000000000000	ASD	ASD LRFD ASD LRFD ASD	ASD LRFD	SD 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	95(803)31°3	Hoa H. qu	Auto man	That we want to	Denis Denis		paibsou	99	Hole Ty	
$\Omega = 2.00$	$\phi = 0.75$	28									

	S	Table 7-3 (continued) Slip-Critical Connections	ble 7-	ο <u>π</u>	Table 7-3 (continued) Critical Connec	ctio	ns	Group B Bolts	roup B Bolts
	4 <u>D</u>	Available Shear Strength, kips (Class A Faying Surface, μ = 0.30)	le Sh Fayin	ear S	trengt face,	:h, kip μ = 0.		A490, A490M F2280 A354 Grade BD	90M
-			5	Group B Bolts	Its			Boll	l _t e
		To Alle		Non	Nominal Bolt Diameter, d, in.	Diameter,	d, in.		
8		9	9/9	3	3/4	2 13	8/2	1	_
				Minimum	Minimum Group B Bolt Pretension, kips	3olt Preter	ısion, kiş	8	
Hole Iype	Loading	24	**		35	4	49		64
		Ω/"J	or _n	Ω/uJ	orn	Ω/uJ	φŁ	Ω/n	orn
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	sα	5.42	8.14	15.8	11.9	11.1	16.6	14.5	21.7
OVS/SSLP	s a	4,62	6.92	6.74	10.1	9.44	14.1	12.3	18.4
TST	s a	3.80	5.70	5.54	8.31	7.76	11.6	10.1	15.2
HT.				Non	Nominal Bolt Diameter, d, in.	Diameter,	d, in.	N SPACE	100
		100	11/8		11/4	Ī	13/8	10	11/2
10 - 11 13		9		Minimum	Minimum Group B Bolt Pretension, kips	3olt Preter	ısion, kiş	SI	
Hole Type	Loading	80	0		102		121	Ī	148
		Ω/η_J	φŁ	Ω/uJ	φľa	Ω/″J	ofn	Ω/n	φŁ
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
TISS/UTS	s	18.1	27.1	23.1	34.6	27.3	41.0	33.4	50.2
	0	36.2	54.2	46.1	69.2	54.7	82.0	6.99	100
0VS/SSLP	s a	15.4	23.1	19.6	29.4	23.3	34.9	28.5	42.6
-	s	12.7	19.0	16.2	24.2	19.2	28.7	23.4	35.1
r r	0	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 h	= standard hole = oversized hole = short-slotted hole transverse to the line of force = short-slotted hole parallel to the line of force	sverse to th	e line of force	orce		S = single shear D = double shear	shear e shear	2 To 1	
LSL = long-slc	= long-slotted hole transverse or parallel to the line of force	verse or pa	allel to th	e line of fo	rce				
Hole Type	ASD	LRFD	Note: Slip	-critical bol	t values assu	ime no mor	e than one	Note: Slip-critical bolt values assume no more than one filler has been provided	n provid
STD and SSLT	$\Omega = 1.50$	φ = 1.00	See AISC	Specification	n Sections	3.8 and J5	for provisic	or botts nave been about to distribute hous in the inters. See AISC Specification Sections J3.8 and J5 for provisions when fillers	22
OVS and SSLP	$\Omega = 1.76$	$\phi = 0.85$	are present For Class B	int. B faving su	rfaces, multi	olv the tabu	lated avail	are present. For Class B faving surfaces, multiply the fabulated available strength by 1.67,	by 1.67.
15	0-214	Δ- 0 Z0	5	Sinfri	and framework	and find	The point		

Group A Bolts		Table 7-3 Slip-Critical Connections	ritic.	Table 7-3	-3 onne	ctio	ns		
A325, A325M F1858 A354 Grade BC	. 0	Available Shear Strength, kips (Class A Faying Surface, μ = 0.30)	ole Sh Fayir	ear S ng Sur	trengi face,	th, kip	30)		
A449	Water State of		5	Group A Bolts	olts				
É	804	2	0.00	Non	Nominal Bolt Diameter, d, in.	Diameter,	ď, in.		
15		10	8/9		3/4		8/2		-
Hele Time	Looding			Minimum	Group A	3olt Prete	Minimum Group A Bolt Pretension, kips	_	
noie iype	Luaumy	5	19		28	5/2	39	43	21
		r_n/Ω	φŁ	r _n /Ω	φŁ	Ω/nJ	φr	Ω/uJ	or,
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	s c	4.29	6.44	6.33	9.49	8.81	13.2	11.5	17.3
Overee B	s	3.66	5.47	5.39	8.07	7.51	11.2	9.82	14.7
OVS/SSLF	٥	7.32	10.9	10.8	16.1	15.0	22.5	19.6	29.4
TST	s a	3.01	9.02	4.44	6.64	6.18	9.25	8.08	12.1
		- Files	Fil.	Non	Nominal Bolt Diameter, d, in.	Diameter,	d, in.		9
		-	11/8	-	11/4	+	13/8	-	11/2
				Minimum	Group A E	3oft Preter	Minimum Group A Bolt Pretension, kips		
adkı alou	Loading	2	99	7	11		85	=	103
		r_n/Ω	φŁu	r_a/Ω	φŁ	Ω / u_1	φľn	Ω/"J	orn
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	sα	12.7	19.0	16.0	24.1	19.2	28.8	23.3	34.9
OVS/SSLP	s o	10.8	16.1	13.7	20.5	16.4	24.5	19.8	29.7
TST	s o	8.87	13.3	11.2	16.8	13.5	20.2	16.3	24.4
STD = standard hole OVS = oversized hole SSLT = short-slotted h	= standard hole = oversized hole = short-slotted hole transverse to the line of force	sverse to th	e line of fo	e).		S = single shear D = double shear	shear s shear		8 5
SSLP = short-sl	= short-slotted hole parallel to the line of force = long-slotted hole transverse or parallel to the line of force	illel to the li	ne of force rallel to the	e line of for	ce				
Hole Type	ASD	LRFD	Note: Slip	Note: Slip-critical bolt values assume no more than one filler has been provided	values assu	me no more	than one fil	ller has been	provided
STD and SSLT	$\Omega = 1.50$	$\phi = 1.00$	See AISC	or bons have been added to distribute loads in the fillers. See AISC Specification Sections J3.8 and J5 for provisions when fillers	n Sections J	3.8 and J5 f	or provisions	s when fillers	
OVS and SSLP	$\Omega = 1.76$	$\phi = 0.85$	are present.	are present. For Class B faving surfaces, multiply the tabulated available strength by 1.67.	faces, multir	olv the tabul	ated availab	le strenath t	v 1.67.
TST	$\Omega = 2.14$	$\phi = 0.70$,				, , , , , , , , , , , , , , , , , , ,	

Type Spacing, f_{ab} ksi $f_$	Both straing, spaning, spaning, from Kall From Kall from ASD 17/s ortn n1/s	Specing, F _n ksi	\sqrt{e}		28.8	ALC: N	W. Director	Nomi	Nominal Bolt Diameter, d, in.	iameter,	d, in.		
Spaning, f., K.S. Γ_{μ}/Ω $\phi_{\tau n}$ Γ_{μ}/Ω $\phi_{\tau n}/\Omega$ $\rho_{\tau n}/\Omega$	Spacing, spin. Fur Kall (AL) ϕ_{1} </th <th>Specing, $r_{\rm b}$ KS $r_{\rm b}$ ($r_{\rm b}$ C $r_{\rm b$</th> <th></th> <th>Bolt</th> <th>-</th> <th></th> <th>11/8</th> <th></th> <th>11/4</th> <th></th> <th>13/8</th> <th></th> <th>11/2</th>	Specing, $r_{\rm b}$ KS $r_{\rm b}$ ($r_{\rm b}$ C $r_{\rm b$		Bolt	-		11/8		11/4		13/8		11/2
22_{0} d_{0} 58 68.1 94.6 70.3 LRFD ASD ASD ASD BSD	22/3 db 655 63.1 94.6 70.3 105 77.6 116 84.8 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	22/3 db 656 63.1 94.6 70.3 105 77.6 116 84.8 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	Hole Type	Spacing, s. in.	F _{th} KSI	1	of,	r _n /Ω	φŁ	Ω/"	φŁ	ιηΩ	φŁ
22/3 db 58 63.1 94.6 70.3 105 77.6 116 94.8 3 ln. 58 63.1 94.6 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — —	$2P_{3}$ d_{b} $5B$ $6B.1$ 94.6 70.3 105 77.6 116 94.8 3 in. $5B$ $6B.1$ 94.6 70.7 106 70.7 106 70.7 106 70.7 106 70.7 106 70.7 106 70.7 100 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0 74.0	22/3 db 655 70.7 106 78.8 118 86.9 130 95.1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1		ii 6	8	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3 in. 56 63.1 94.6 — — — — — — $2^2/3$ d_b 56 58 52.2 78.3 — — — — — $3^2/3$ d_b 56 58.5 58.5 78.3 — — — — — $2^2/3$ d_b 56 58.5 58.5 7.83 — — — — — $2^2/3$ d_b 56 58.4 81.6 61.6 92.4 68.9 103 76.1 $2^2/3$ d_b 56 60.9 91.4 69.1 104 77.2 116 8.3 $2^2/3$ d_b 66 7.31 11.0 8.13 12.2 8.94 13.4 9.7 $2^2/3$ d_b 66 7.31 11.0 8.13 12.2 8.94 13.4 9.7 3 in. 66 58.9 88.4 — — — — — — 3 in. 68 52.6 78.3 17.7 18.4 104 104 $8 > 5$ s_{tot} 58 65.3 97.9 72.5 146 107 179.2 3 s_{tot} 66 58.9 <	3 in. 56 63.1 94.6 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — <	22/3 db 65 58.5 89.2 66.7 100 74.0 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	eTD	22/3 db	58	63.1	94.6 106	70.3	105	77.6	116	84.8 95.1	127
22/3 db 58 52.2 78.3 59.5 89.2 66.7 100 74.0 3 in. 56 58.5 87.8 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — —	22/3 db 65 58.5	22/3 db 58 52.2 78.3 59.5 99.9 74.0 112 82.9 112 82.9 112 82.9 112 82.9 112 82.9 112 82.9 112 82.9 112 82.9 112 82.9 112 112 82.9 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 112 <td>SSLT</td> <td>3 in.</td> <td>52</td> <td>63.1</td> <td>94.6 106</td> <td>1.1</td> <td>FI</td> <td>11</td> <td>1.1</td> <td>1.1</td> <td></td>	SSLT	3 in.	52	63.1	94.6 106	1.1	FI	11	1.1	1.1	
3 in. 56 58.5 78.3 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — —	3in. 56 52.2 78.3 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — <t< td=""><td>22/3 d_b 65 58.5</td><td></td><td>22/3 db</td><td>58</td><td>52.2</td><td>78.3 87.8</td><td>59.5</td><td>89.2 99.9</td><td>66.7</td><td>100</td><td>74.0</td><td>111</td></t<>	22/3 d _b 65 58.5		22/3 db	58	52.2	78.3 87.8	59.5	89.2 99.9	66.7	100	74.0	111
$22/3$ db 56 54.4 81.6 61.6 92.4 68.9 103 76.1 3 in. 56 56.9 91.4 69.1 104 77.2 116 85.3 $2^2/3$ db 66 53.4 81.6 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — —	22_{13} db 56 54.4 81.6 61.6 92.4 68.9 103 76.1 3 in. 56 60.9 91.4 — — — — — — 2^{2} db 66 60.9 91.4 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — —	$2^2/3$ db 58 54.4 81.6 61.6 92.4 68.9 103 76.1 11 $31n$. 66 60.9 91.4 69.1 104 77.2 116 85.3 11 $2^2/3$ db 66 60.9 91.4 $ -$ <td>SSLP</td> <td>3 in.</td> <td>82 99</td> <td>52.2</td> <td>78.3</td> <td>1.1</td> <td>1.1</td> <td>1.1</td> <td>11</td> <td>11</td> <td></td>	SSLP	3 in.	82 99	52.2	78.3	1.1	1.1	1.1	11	11	
3 in. 58 54,4 81.6 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — <	3 in. 58 54.4 81.6 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — <	3 in. 66 60.9 91.4 — — — — — — — — — — — — — — — — — — —	9	22/3 db	88 88	54.4	81.6 91.4	61.6	92.4	68.9	103	76.1 85.3	114
$22/3$ sb 56 6.53 9.79 7.25 10.9 7.98 12.0 8.7 3 in. 58 6.53 9.79 7.25 10.9 7.98 12.0 9.7 $2^2/3$ db 66 7.31 11.0 8.13 12.2 8.94 13.4 9.7 $2^2/3$ db 58 6.5 7.81 58.6 8.79 64.6 97.0 70.7 $2^2/3$ db 58 52.6 78.8 65.7 98.5 72.4 109 79.2 3 in. 66 58.9 88.4 — — — — — — s > shull 66 87.8 117 87.0 131 95.7 144 104 s $\geq s$ shull 66 87.8 132 97.5 146 107 161 117 s $\leq s$ shull 65 73.1 110 81.3 122 89.4 134 97.6 strength SSL 31/4<	$ 22_{13} s_{0b} \qquad \begin{array}{ccccccccccccccccccccccccccccccccccc$	22/3 db 665 7.31 11.0 8.13 12.2 8.94 13.4 9.75 8 13.1 13.0 8.13 12.2 8.94 13.4 9.75 8 13.1 13.0 8.13 12.2 8.94 13.4 9.75 8 13.1 13.0 8.13 12.2 8.94 13.4 9.75 13.1 13.0 8.13 12.2 8.94 13.4 9.75 13.1 13.0 8.13 12.2 8.94 13.4 9.75 13.1 13.0 8.13 12.2 8.94 13.4 9.75 13.1 13.0 8.13 13.2 8.94 13.4 10.9 79.2 13.1 13.1 13.1 13.1 13.1 13.1 13.1 13	SAO	3 in.	82 83	54.4	91.6	11	1-1	1.1	H	11	
3 in. 58 6.53 9.79 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — <	3 in. 58 6.53 9.79 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — <	3 in. 22/3 db 3 in. s ≥ stuff for full strength ', in. pacing® = 22/ gard hole 1-slotted hol		22/3 db	58	6.53	9.79	7.25	10.9	7.98	12.0	8.70 9.75	13.1
$2^2/3$ db 58 52.6 78.8 58.6 87.9 64.6 97.0 70.7 3 in. 56 58.9 88.4 65.7 98.5 72.4 109 79.2 3 in. 65 58.9 88.4 65.7 98.5 72.4 109 79.2 $8 \ge S_{hull}$ 65 78.3 117 87.0 131 95.7 144 104 $S \ge S_{hull}$ 65 87.8 175 175 146 107 161 117 $S \ge S_{hull}$ 65 73.1 110 81.3 122 89.4 134 97.5 fortuli LSLT 37/16 313/16 45/16 45/16 97.6 strength OVS 31/16 41/16 47/16 45/16 97.6 LSLP 51/16 55/16 55/16 65/16 65/16 65/16	22/3 db 65 58.9 78.8 58.6 87.9 64.6 97.0 70.7 31.0 58 58.9 88.4 65.7 98.5 72.4 109 79.2 31n. 65 58.9 88.4 65.7 98.5 72.4 109 79.2 31n. 65 58.9 88.4 — — — — — — — — — — — — — — — — — — —	3 in. 3 in. \$ ≥ \$full \$ ≥ \$full \$ ≥ \$full \$ +	LSL	3 in.	58 65	6.53	9.79	11	1.1	11	1-1	11	E I
3 in. 58 52.6 78.8 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — <	3 in. 58 52.6 78.8 — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — — —	3 in. \$ ≥ \$full for full strength for full strength lin. pacing* = 2²/ dard hole 1-slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole -slotted hole	10.	22/3 db	58	52.6 58.9	78.8	58.6	87.9 98.5	64.6	97.0	70.7	106
$s \ge s_{hull}$ 58 78.3 117 87.0 131 95.7 144 104 $s \ge s_{hull}$ 65 87.8 132 97.5 146 107 161 117 section 65 87.8 110 81.3 122 89.8 120 87.6 fortuli strength in. SSLI, strength in. 37/16 41/16 47/16 47/16 47/16 pacing* = 22/3d, in. 3 35/16 55/16 55/16 55/16 33/17	\$\sigma\$ \sigma\$ \si	s ≥ s _{full} for full strength i.in. pacing* = 2²/ ard hole t-slotted hole t-siotted hole sized hole t-slotted hole sized hole size	3	3 in.	92 92	52.6 58.9	78.8	11	11	11	11	11	4 100
SLT \$\sigma\$ & \$\text{66.3}\$ \$97.9 \$72.5 \$109 \$79.8 \$120 \$87.4 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$134 \$13	SLT $s \ge s_{full} $ $\frac{58}{65}$ $\frac{65.3}{73.1} $ $\frac{97.9}{110}$ $\frac{72.5}{81.3} $ $\frac{79.8}{122} $ $\frac{120}{97.5} $ $\frac{87.0}{97.5} $ $\frac{87.0}{122} $ $\frac{87.0}{97.5} $ $\frac{87.0}{122} $ 87.0	pacing for full saring strength string. Stuff's, in. mum Spacing* = 2²/ = standard hole = short-slotted hole = short-slotted hole = noversized hole = noversized hole = long-slotted ho	STD, SSLT, SSLP, OVS, LSLP	S	828	78.3 87.8	117	87.0 97.5	131 146	95.7 107	144	104	157
SSLT, 3 ⁷ /16 3 ¹³ /16 4 ³ /16 4 ³ /16 SSLT, SSLT, 3 ¹ /16 4 ¹ /16	pacing for full SSLT, 37/16 313/16 43/16 43/16 SSLT, SSLT, 31/16 41/16 47/16 47/16 OVS 311/16 41/16 41/16 47/16 47/16 1SLT 33/4 41/16 47/16 63/16 ILSLP 51/16 55/8 63/16 63/16 = short-slotted hole oriented transverse to the line of force = oversized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = long-slotted hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized hole oriented parallel to the line of force = noresized h	pacing for full stun, in. "num Spacing = 2²/ = standard hole = short-slotted hole = short-slotted hole = long-slotted hole	LSLT	\Al	92	65.3	97.9	72.5	109	79.8	120	87.0 97.5	131
saring strength OVS 311/16 41/16 47/16 47/16 Studina, in. SSLP 33/4 41/6 41/2 41/2 Inum Spacinga = 2/3df, in. 31/16 35/16 35/16 31/16	Stull*, in. SSLP $3^{1}/_{16}$ $4^{1}/_{16}$ $4^{7}/_{16}$ Stull*, in. SSLP $3^{3}/_{4}$ $4^{1}/_{8}$ $4^{7}/_{16}$ mum Spacing* = $2^{1}/_{3}/_{4}$, in. 3 $3^{5}/_{16}$ $5^{5}/_{16}$ $6^{3}/_{16}$ mum Spacing* = $2^{1}/_{3}/_{4}$, in. 3 $3^{5}/_{16}$ $6^{3}/_{16}$ $6^{3}/_{16}$ = standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = oversized hole = conversized hole = conversized hole = conversized hole = long-slotted hole oriented transverse to the line of force = conversized hole = conversized hole	mum Spacinga = 2²/ mum Spacinga = 2²/ = standard hole = short-slotted hole = short-slotted hole = long-slotted hole	Spacing	for full	STD, SSLT, LSLT	37	/16	31	3/16	43	/16	49	16
$\begin{array}{c ccccccccccccccccccccccccccccccccccc$	Stulf*, III. SSLP 33_{4} 41_{8} 41_{2} num Spacing* = 2^{3} 3d, in. 3 3^{5} /16 6^{3} /16 6^{3} /16 = standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = oversized hole = a oversized hole = long-slotted hole oriented parallel to the line of force = long-slotted hole oriented transverse to the line of force	mum Spacing ^a = 2 ² / ₁ = standard hole = short-slotted hole = short-slotted hole = oversized hole = long-slotted hole = long-slotted hole = long-slotted hole long-slotted hole	Dearing	strength	SAO	31	1/16	41	/16	47	/16	41	3/16
mum Spacing ^a = $2^2/_3 d$, in. 3 $3^5/_{16}$ $6^5/_{16}$ $6^3/_{16}$	num Spacing* = $2^3/36$, in. $5^1/16$ $5^5/16$ $6^3/16$ = standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole = short-slotted hole = sversized hole = a oversized hole = inor-soluted hole oriented parallel to the line of force = inor-soluted hole oriented parallel to the line of force	mum Spacing ^a = 2²/ = standard hole = short-slotted hole = short-slotted hole = coversized hole = long-slotted hole	Stull	H	SSLP	33	/4	41	8/	41	/2	47	8
mum Spacing ^a = $2^2/3^4$, in. 3 $3^5/16$ $3^{11}/16$	### Spacing* = 2 ² / ₃ d, in. 3 3 ⁵ / ₁₆ 3 ¹ / ₁₆ = standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = oversized hole = long-slotted hole oriented parallel to the line of force = long-slotted hole oriented transverse to the line of force	mum Spacing* = 27, = standard hole = short-slotted hole = short-slotted hole = noversized hole = long-slotted hole = long-slotted hole = long-slotted hole or			LSLP	51	/16	52	8/	63	/16	63	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 LSLP = long-slotted hole oriented parallel to the line of force	= standard hole = short-slotted hole = short-slotted hole = oversized hole = long-slotted hole = long-slotted hole = long-slotted hole or	Minimum S	pacing ^a = 2	2/3d, in.	က		35	/16	31	1/16	4	

		0.20	¥	Kips/in.	unickness	ness		5		
	100				Nom	nal Bolt [Nominal Bolt Diameter, d, in.	d, in.		
Hole Type	Spacing.	F _{th} ksi	5970	8/8	VE.	3/4	- 1	8/2		_
1	s, in.	8 8	ν/υ	φŁ	ν,/Ω	φĽ	r,/12	Φζ0	Ω/uJ	orn
		- 81	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD	22/3 db	92 92	34.1	51.1	41.3	62.0	48.6	72.9	55.8	83.7
SSLT	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	67.4	101
	22/3 db	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1 52.8	70.7
SSL	3 in.	58 65	43.5	65.3	52.2 58.5	78.3	60.9	91.4	58.7	98.7
one	22/3 db	58 65	29.7	44.6 50.0	37.0	55.5 62.2	44.2	66.3	49.3	74.0
SAN T	3 in.	65 65	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
3	22/3 db	28	3.62	5.44 6.09	4.35	6.53	5.08	7.61	5.80	8.70
Laci	3 in.	58 65	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
F	22/3 db	58 65	28.4	42.6	34.4	51.7	40.5	68.0	46.5 52.1	69.8
191	3 in.	58 65	36.3	54.4	43.5	65.3	50.8	76.1	56.2	84.3
STD, SSLT, SSLP, OVS, LSLP	S ≥ Sfull	58	43.5	65.3	52.2 58.5	78.3 87.8	60.9	91.4	69.6	104
LSLT	S ≥ Stull	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
Spacing for full	for full	STD, SSLT, LSLT	ely:	115/16	25	25/16	211	211/16	31	31/16
bearing strength	strength	SAO	21	21/16	27	27/16	213	213/16	3	31/4
Stull ^a , III.	i .	SSLP	2	21/8	2	21/2	27	27/8	35	35/16
		LSLP	213	213/16	3	33/8	318	315/16	4	41/2
Minimum Spacing ^a =	$pacing^a = 2^2$	2 ² /3d, in.	+	111/16		2	25	25/16	21.	211/16
STD = stan SSLT = shor SSLP = shor OVS = over LSLP = long	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 SSLP = long-slotted hole oriented parallel to the line of force LSLP = long-slotted hole oriented transverse to the line of force	s oriented to soriented parameters oriented parameters	ransverse parallel to arallel to ansverse	to the line of the line of the line of to the line of to the line in the line to the line the	e of force force force of force					(明華) (新
ASD	LRFD	Note: Spac	ing indicate	ed is from the	ne center of	the hole or	slot to the	center of th	e adjacent h	iole of
000	0.75	see AISC S	line of Total	3. Hole gero 7 Section J3	rmation is c. .10.	onsidereo.	slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.	eformation	IS NOT COUS	Qeich,

$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	Edge Los for Large 11/s 11/s 11/s 13/s 17/s 17/s 17/s 17/s 17/s 17/s 17/s 17/s 17/s 6fm LRFD 6fm ASD 6fm LRFD 6fm ASD 6fm A	Type Distance Edge Tyle				<u> </u>	kips/in. thickness	thick	rness	ICKNESS Nominal Bolt Diameter, d, in.	ď, ir.		
The control of the	174 158 r_0/Ω r_0/Ω	179e Distance To NSI		Edge	3		11/8		1/4		13/8		11/2
SLT 1/4 58 22.8 34.3 20.7 31.0 18.5 27.7 16.3 1.1/4 58 22.8 34.3 20.7 31.0 18.5 27.7 16.3 2 58 48.9 73.4 46.8 70.1 44.6 66.9 42.4 2 58 43.5 65.3 17.1 25.6 14.6 21.9 12.2 2 65 48.8 73.1 46.3 62.0 39.1 19.6 10.9 2 65 43.5 65.3 41.3 62.0 39.2 65.8 41.4 2 65 40.8 73.1 46.3 69.5 43.9 65.8 41.4 2 65 40.8 73.1 18.3 27.4 15.8 23.8 13.4 11/4 65 20.7 31.1 18.3 27.4 15.8 23.8 13.4 2 65 20.7 31.0 15.2 22.8 9.79 14.7 4.3 2 65 23.2 34.7 17.1 25.6 11.0 16.5 4.8 11/4 65 20.7 31.0 15.2 22.8 9.79 14.7 4.3 2 65 23.2 34.7 17.1 25.6 11.0 16.5 4.8 2 65 23.2 34.7 17.1 25.6 11.0 16.5 4.8 2 65 20.3 31.0 15.2 22.8 9.79 14.7 4.3 2 65 20.3 31.0 15.2 22.8 9.79 14.7 4.3 2 65 20.3 31.0 28.5 17.2 25.8 17.3 25.9 15.2 3 11/4 65 45.7 68.6 43.7 66.5 41.6 62.5 39.6 3 10/8, $L_e \ge L_e tutt$ 66 87.8 11.7 87.0 131 95.7 144 104 3 10/8, $L_e \ge L_e tutt$ 66 87.8 13.2 39.16 35/16 3 10/8, $L_e \ge L_e tuttt$ 67.8 31.7 35.3 3 10/8, $L_e \ge L_e tutttt$ 67.8 31.7 31.8 3 10/8, $L_e \ge L_e tutttt$ 67.8 31.7 31.8 3 10/8, $L_e \ge L_e tutttt$ 67.8 31.7 31.8 4 10/8 31.8 32.0 33/16 33/16 33/16 5 10/8 31.4 31.4 31.4 31.4 5 10/8 31.8 31.4 31.4 31.4 5 10/8 31.8 31.2 35/16 35/16 5 10/8 31.4 31.4 31.4 5 10/8 31.4 31.4 5 10/8 31.4 31.4 5 10/8 31.4 31.4 5 10/8 31.4 31.4 5 10/8 31.4 31.4 5 10/8 31.5 31.6 5 10/8 31.8 31.7 5 10/8 31.8 31.5 5 10/8 31.8 31.5 5 10/8 31.8 31.5 5 10/8 31.8 31.4 5 10/8 31.8 5 10/8 31.8 5 10/8 31.8 5 10/8 31.8 5 10/8 31.8 5	11/4 58 22.8 34.3 20.7 31.0 18.5 27.7 16.3 17	11/4 58 22.8 34.3 20.7 31.0 18.5 27.7 16.3 11/4 58 22.8 34.3 20.7 31.0 18.5 27.7 16.3 12 2 58 48.8 62.3 24.7 20.7 31.1 18.3 12 58 48.8 62.3 22.4 48.6 50.0 75.0 47.5 11/4 58 17.4 26.1 15.2 22.8 13.1 19.6 10.9 11/4 58 17.4 26.1 15.2 22.8 13.1 19.6 10.9 11/4 58 18.5 27.7 16.3 24.5 14.1 21.2 12.0 11/4 58 20.7 31.1 18.3 27.4 15.8 23.8 13.4 11/4 58 20.7 31.1 18.3 27.4 15.8 27.8 13.4 11/4 58 20.7 31.1 18.3 27.4 15.8 27.8 13.4 11/4 58 20.7 31.1 18.3 27.4 15.8 23.8 13.4 11/4 58 20.7 31.1 17.1 25.6 14.6 27.8 13.4 11/4 58 20.7 31.1 17.1 25.8 17.3 40.2 67.4 38.1 11/4 58 20.7 31.7 17.1 25.8 17.3 25.9 15.2 11/4 58 20.7 31.7 17.1 25.8 17.3 25.9 15.2 11/4 58 20.7 31.7 17.1 25.8 17.3 25.9 15.2 11/4 58 20.7 31.7 32.0 37.3 25.9 15.2 11/4 58 20.7 31.7 32.0 37.3 25.9 15.2 11/4 58 20.7 31.7 32.0 37.3 25.9 15.2 11/4 58 20.7 31.7 32.0 37.4 35.8 11/4 58 20.7 31.7 32.0 37.3 37.3 35.3 20.7 20.8 20.7 31.7 32.0 37.3 37.3 37.3 20.8 20.7 31.7 32.0 37.4 33.9 20.8 20.7 31.7 32.0 37.6 37.6 37.6 20.8 20.7 31.7 32.0 37.6 37.6 37.6 20.8 20.7 31.7 32.6 37.6 37.6 37.6 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7 31.7 20.8 20.7 31.7 31.7	Hole Iype	Le, in.	Les KS	r _n /Ω	φŁ	r _n /Ω	φŁ	Ω/uJ	φľπ	Ω/us	φŁ
SLT 2 56 38.4 23.2 34.7 20.7 31.0 18.5 27.7 16.3 SLT 2.5 56 38.4 23.2 34.7 20.7 31.1 18.3 SLT 2.5 56 5.8 38.4 23.2 34.7 20.7 31.1 18.3 SLT 2.5 56 5.4 8.8 2.3 52.4 78.6 50.0 75.0 75.0 475.5 56.5 14.1 20.1 19.6 10.9 10.9 10.9 10.9 10.9 10.9 10.9 10.9	11/4 56 22.8 34.4 23.2 34.7 18.5 27.7 16.3 12 2 56 54.8 22.3 34.7 20.7 31.1 18.3 13 2 56 54.8 22.3 34.7 20.7 31.1 18.3 14 56 54.8 72.4 46.8 70.1 44.6 66.9 42.4 2 56 54.8 77.4 26.1 15.2 22.8 13.1 19.6 10.9 17 66 19.5 29.3 17.1 25.6 14.6 21.9 12.2 17 66 19.5 27.7 16.3 27.4 41.8 27.7 46.3 69.5 43.9 65.8 37.0 17 66 20.7 31.1 18.3 27.4 15.8 23.8 13.4 17 66 20.7 31.1 18.3 27.4 15.8 23.8 13.4 17 66 20.7 31.1 18.3 27.4 15.8 23.8 13.4 17 66 20.7 31.1 18.3 27.4 15.8 23.8 13.4 17 66 20.7 31.0 15.2 22.8 9.79 14.7 4.35 17 66 21.3 31.0 15.2 22.8 9.79 14.7 4.35 18 2 66 21.3 31.7 25.6 11.0 16.5 4.88 19 2 66 21.3 31.7 25.6 11.0 16.5 4.88 10 2 66 21.3 31.7 25.6 11.0 16.5 4.88 10 2 66 21.3 17.1 25.6 11.0 16.5 4.88 10 2 66 21.3 11.7 87.0 131 96.7 144 104 10 2 66 67.8 37.5 109 79.8 120 87.0 11 $t_e \ge L_e t_{total}$	11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 2 2 2 2 2 2 2 2 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 2 2 2 2 2 2 2 2 2 2 2 2 2				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
SLT 2 56 48.9 7.3.4 46.8 70.1 44.6 66.9 4.24 7.5 50.0 75.0 47.5 50.0 17.4 46.8 82.3 52.4 78.6 50.0 75.0 47.5 50.0 17.4 66.5 19.5 29.3 17.1 25.6 14.6 21.9 12.2 12.0 17.4 65 19.5 29.3 17.1 25.6 14.6 21.9 12.2 12.0 17.4 65 20.7 31.1 18.3 27.4 15.8 23.8 13.4 11.0 17.4 65 20.7 31.1 18.3 27.4 15.8 23.8 13.4 13.6 20.0 39.2 58.7 37.0 17.1 18.3 27.4 15.8 23.8 13.4 13.6 20.0 19.3 20.7 11.3 45.1 67.6 4.2 13.0 13.6 20.0 19.3 20.0 17.3 45.1 67.6 4.2 13.0 15.2 22.8 9.79 14.7 4.3 25.0 17.1 25.6 11.0 16.5 4.8 13.0 19.3 22.8 17.2 25.8 17.3 25.9 17.3 25.9 15.2 22.8 9.79 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 25.9 17.3 2	17 2 56 54.8 82.3 52.4 78.6 50.0 75.0 47.5 17 2 56 54.8 82.3 52.4 78.6 50.0 75.0 47.5 17 2 56 54.8 82.3 52.4 78.6 50.0 75.0 47.5 18 2 56 54.8 82.3 17.1 25.6 14.6 51.9 10.9 17 4 56 19.5 22.3 17.1 25.6 14.6 21.2 12.2 17 56 48.8 73.1 18.3 27.7 16.3 24.5 14.1 21.2 12.0 17 4 56 50.0 75.0 47.5 71.3 45.1 67.6 42.7 18 2 58 20.7 31.1 18.3 27.4 15.8 23.8 13.4 17 65 20.7 31.1 18.3 27.4 15.8 23.8 13.4 17 65 20.7 31.1 18.3 27.4 15.8 23.8 13.4 18 2 56 50.0 75.0 47.5 71.3 45.1 67.6 42.7 19 2 56 50.0 75.0 19.3 28.9 17.3 25.9 15.2 10 2 56 23.2 34.7 17.1 25.6 11.0 16.5 4.88 10 2 56 23.2 34.7 17.1 25.6 11.0 16.5 4.88 10 2 56 23.2 34.7 17.1 25.6 11.0 16.5 4.88 10 2 56 20.7 31.1 31.2 25.8 17.3 25.9 15.2 10 2 56 23.2 34.7 17.1 25.6 11.0 16.5 4.88 10 2 56 20.7 31.1 31.2 25.8 17.3 25.9 15.2 10 2 56 20.3 37.9 25.5 17.3 25.9 15.2 10 2 56 31.3 37.6 33.7 10 3 33.7 33.7 33.7 33.7 11 4 56 23.3 37.9 37.8 37.8 37.8 10 2 56 33.8 37.9 33.7 11 4 56 20.7 31.1 31.2 33.7 12 5 5 5 5 30.8 10 10 10 10 10 10 10	174 2 2 2 3 3 3 3 3 3 3	OT.	11/4	58	22.8	34.3	20.7	31.0	18.5	27.7	16.3	24.5
SLP 11/4 56 177, 4 26.1 16.2 22.8 13.1 19.6 10.9 10.9 10.9 10.9 10.9 10.9 10.9 10.9	114 58 174 26.1 15.2 22.8 13.1 19.6 10.9 174 665 48.8 73.1 15.2 22.8 13.1 19.6 10.9 174 665 48.8 73.1 46.3 69.5 43.9 65.8 41.4 174 665 48.8 73.1 46.3 69.5 43.9 65.8 41.4 174 665 20.7 31.1 18.3 27.4 15.8 23.8 13.4 174 665 20.7 31.1 18.3 27.4 15.8 23.8 13.4 174 665 20.7 31.0 15.2 22.8 9.79 14.7 42.7 174 665 20.7 31.0 15.2 22.8 9.79 14.7 4.35 174 665 23.2 34.7 17.1 25.6 11.0 16.5 4.88 185 20.7 31.0 15.2 22.8 9.79 14.7 4.35 196 23.2 34.7 17.1 25.6 11.0 16.5 39.6 197 198 32.0 19.2 28.9 17.3 25.9 15.2 198 20.7 31.0 32.0 19.3 28.9 17.3 25.9 15.2 199 28.5 17.2 25.8 17.3 25.9 15.2 190 28.5 17.2 29.8 17.3 25.9 15.2 190 28.5 31.7 31.7 35.6 31.7 190 28.5 31.7 31.7 31.7 190 28.5 31.7 31.7 31.7 190 28.5 31.7 31.7 31.7 190 28.5 31.7 31.7 190 28.5 31.7 31.7 190 31.7 31.7 31.7 190 31.3 190 31.7 31.7 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 31.7 190 31.3 190 31.3 190 31.4 31.4 190 31.4 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 190 31.4 19	11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 2 2 2 2 2 2 2 2 2 2 2 2 2	SSLT	2	28	48.9	73.4	46.8	70.1	44.6	6.99	42.4	63.6
SLP 11/4 65 19.5 29.3 17.1 25.6 14.6 21.9 122 2 65 48.8 73.1 46.3 62.0 39.2 58.7 37.0 11/4 56 20.7 31.1 18.3 27.4 15.8 23.8 13.4 SLP 2 58 44.6 66.9 42.4 63.8 63.8 13.4 11/4 65 20.7 31.0 15.2 22.8 9.79 14.7 4.3 SLT 2 58 20.7 31.0 15.2 22.8 9.79 14.7 4.3 11/4 56 21.3 32.0 17.1 25.6 11.0 16.5 4.8 SLI 1 65 21.3 32.0 17.1 25.6 11.0 16.5 4.8 SLI 2 58 40.8 61.2 39.0 58.5 17.3 25.9 15.2 SLI $L_e \ge L_e tull$ 56 65.3 97.9 72.5 109 79.8 120 87.0 11.1 10.0 18.1 17.1 11.0 81.3 12.2 89.4 13.4 97.5 10.4 10.4 10.4 10.4 10.4 10.4 10.4 10.4	11/4 65 19.5 29.3 17.1 25.6 14.6 21.9 12.2 2 65 48.8 73.1 46.3 69.5 43.9 65.8 41.4 2 65 48.8 73.1 46.3 69.5 43.9 65.8 41.4 2 65 20.7 31.1 18.3 24.5 14.1 21.2 12.0 2 65 50.0 75.0 42.4 63.6 40.2 60.4 38.1 2 65 50.0 75.0 42.4 63.6 40.2 60.4 38.1 2 65 50.0 75.0 42.4 63.6 40.2 60.4 38.1 3 11/4 65	11/4 2 2 2 2 2 2 2 2 2		Œ la	8 8	17.4	26.1	15.2	72.8	13.1	19.6	10.9	16.3
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NS $\frac{11}{4}$ $\frac{58}{65}$ $\frac{18.5}{20.7}$ $\frac{27.7}{31.1}$ $\frac{16.3}{18.3}$ $\frac{24.5}{27.4}$ $\frac{14.1}{15.8}$ $\frac{21.2}{23.8}$ $\frac{13.4}{13.4}$ $\frac{13.4}{45.1}$ $\frac{58}{65}$ $\frac{44.6}{50.0}$ $\frac{66.9}{75.0}$ $\frac{42.4}{47.5}$ $\frac{63.6}{42.7}$ $\frac{40.2}{42.1}$ $\frac{60.4}{65}$ $\frac{38.1}{42.1}$ $\frac{2}{23.2}$ $\frac{20.7}{34.7}$ $\frac{17.3}{17.1}$ $\frac{45.1}{25.6}$ $\frac{60.4}{42.7}$ $\frac{42.1}{42.3}$ $\frac{42.1}{42.1}$ $\frac{65}{65}$ $$	11/4 58 18.5 27.7 16.3 24.5 14.1 21.2 12.0 2 58 44.6 66.9 47.4 63.6 44.7 67.6 42.7 11/4 65 1 1/4 65 1 1/4 65 1 1/4 65 1 1/4 65 23.2 34.7 17.1 25.6 11.0 16.5 4.88 1 1/4 65 23.2 34.7 17.1 25.6 11.0 16.5 4.88 1 1/4 65 23.2 34.7 17.1 25.6 11.0 16.5 4.88 1 1/4 65 23.2 34.7 17.1 25.6 11.0 16.5 4.88 1 1 2 2 58 19.0 28.5 17.2 25.8 15.4 23.1 13.6 1 1 2 2 58 40.8 61.2 39.0 58.5 37.2 55.7 35.3 1 2 5 5 40.8 61.2 39.0 58.5 37.2 55.7 39.6 1 1 1 2 2 2 3 3 3 3 1 2 2 2 3 3 3 3 1 3 3 3 3 3 1 4 5 5 5 3 3 3 1 4 5 5 5 5 3 3 1 5 5 5 5 5 3 1 7 8 8 8 3 3 3 1 8 8 8 8 3 3 3 1 8 8 8 8 3 3 1 8 8 8 8 3 3 1 8 8 8 8 3 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 3 1 8 8 8 8 8 1 8 8 8 8 8 1 8 8 8 8 8 1 8 8 8 8 8 1 8 8 8 8 8 1 8 8 8 8 8 1 8 8 8 8 1 9 8 8 8 1 1 1 1 1 1 1 1 1	VS 2 2 2 1/4 1/4 1/4 1/14 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2	995	2	65 65	43.5	65.3	41.3	62.0 69.5	39.2	58.7	37.0	55.5 62.2
$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	11/4 58 44.6 66.9 42.4 63.6 40.2 60.4 38.1 11/4 58 17 4 58 20.7 31.0 15.2 22.8 9.79 14.7 4.35 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 32.0 19.3 28.9 17.3 25.9 15.2 11/4 65 21.3 37.0 37.5 19.0 79.8 12.0 87.0 11/4 65 73.1 110 81.3 122 89.4 134 97.5 11/4 65 73.1 110 81.3 122 89.4 134 97.5 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/4 11/	11/4 2 2 2 2 2 2 2 2 2	9	11/4	85 58	18.5	31.1	16.3	24.5	14.1	21.2	12.0	17.9
SLP 11/4 65 — — — — — — — — — — — — — — — — — —	11/4 58	11/4 2 2 2 2 2 2 2 2 2	SAO	2	88 88	44.6	66.9 75.0	42.4	63.6	40.2	60.4	38.1	57.1
SLT $L_e \ge L_e \ null \ bearing the result of the state $	1	SSLT, $L_e \geq L_e \ tuni$ SIT $L_e \geq L_e \ tu$		11/4	85 85	1.1	11	1.1	11	11	11	11	
$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	11/4 58 19.0 28.5 17.2 25.8 15.4 23.1 13.6 2 5 5 40.8 61.2 39.0 58.5 37.2 55.7 35.3 2 5 5 40.8 61.2 39.0 58.5 37.2 55.7 35.3 3 5 5 5 5 5 5 5 5 5	SSLT, 2 SSLT, $L_e \geq L_e nur$ Lip (10VS, $L_e \geq L_e nur$ Lip (20VS, $L_e \geq L_e nur$ SIT $L_e \geq L_e nu$	ואר	2	82 93	20.7	34.7	15.2	22.8	9.79	14.7	4.35	
SLI, 2	SSLI, les Le Inii	SSLT, 10 VS, 10 VS, 10 VS, 11 Le > Le tutt 11 Le > Le tutt 12 VI	1	11/4	82 93	19.0	28.5	17.2	25.8	15.4	23.1	13.6	20.4
SLT, $L_e \ge L_e \ null \ $ 58 $78.3 \ 117$ $87.0 \ 131$ 95.7 144 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104 104	SSLT, Le \ge Le $=$ Le	SSLT, Le > Le num ILP ILP ILP ILP ILP ILP ILP Le > Le num Gge distance or full bearing strength e > Le num² in. e > In tungento hole = short-slotted hole = short-slotted hole = long-slotted hole	ראר	2	8 8	40.8	61.2	39.0	58.5	37.2	55.7	35.3	53.0
SLT $L_e \ge L_e tntll$ 58 65.3 97.9 72.5 109 79.8 120 87.0 87.0 SLT, SLD, SLT, SLD, SLL, SLD, SLD, SLD, SLD, SLD, SLD	the distance strength strength strength strength strength e short-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of force e long-slotted hole oriented transverse to the line of long-slotted hole oriented transverse to the line o	dge distance or full bearing strength e > Le nul ² , in. = standard hole = short-slotted hole = short-slotted hole = noresized hole = long-slotted hole	STD, SSLT, SSLP, OVS, LSLP	$L_{\theta} \ge L_{\theta}$ full	88 88	78.3	117	87.0 97.5	131	95.7	144 161	104	157
Edge distance STD, SSLT, LSLT $2^7/6$ 3 $^{3}/16$ $3^3/16$ 3 $^{5}/16$ $3^1/2$ 3 $^{5}/16$ strength Le \geq Le tull, in. SSLP 3 $3^5/16$ $3^5/6$ $3^5/6$ Le \geq Le tull, in. SSLP 3 $3^5/16$ $3^5/6$ $3^5/6$	dge distance STD, SSLT, strail bearing $27/6$ SSLT, USIT $27/6$ 33/16 $33/16$ 35/16 $31/2$ 35/16 strangth e ≥ Le null*, in. SSLP 3 $35/16$ 31/16 $35/16$ 41/16 $35/16$ 41/16 e standard hole = short-slotted hole oriented transverse to the line of force = oversized hole = oversized hole = oversized hole oriented parallel to the line of force = long-slotted hole oriented transverse to the line of force = long-slotted hole oriented transverse to the line of force = long-slotted hole oriented transverse to the line of force	dge distance strength e ≥ Le tull bearing astrength e > Strength e > S	LSLT	Le ≥ Le full	88 88	65.3	97.9	72.5	109	79.8	120	87.0	131
strength OVS 3 35_{16} 35_{8} $L_{\theta} \ge L_{\theta} \rho u r^{\theta}$, in. SSLP 3 35_{16} 35_{8} LSLP $3^{1}/_{16}$ $4^{1}/_{16}$ $4^{1}/_{2}$	strength OVS 3 $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/16$ $3^5/1$	strength e ≥ Le nul², in. = standard hole = short-slotted hole = oversized hole = long-slotted hole	Edge di	stance	SSLT, LSLT	27	8/		13/16	31,		15	3/16
$L_0 \ge L_0 h u l^2$, in. SSLP 3 $3^5 f_{16}$ $3^5 f_8$ $3^5 f_8$ $4^1 f_8$ $4^1 f_8$	$e \ge \textit{Le}_{\textit{full}^2}, \text{in.}$ SSLP 3 $3^5/16$ $3^5/16$ $3^5/16$ $4^1/2$ = standard hole = short-slotted hole oriented transverse to the line of force = orient-slotted hole oriented parallel to the line of force = orientsized hole oriented parallel to the line of force = long-slotted hole oriented transverse to the line of force = long-slotted hole oriented transverse to the line of force = long-slotted hole oriented transverse to the line of force	e ≥ Le nul², in. = standard hole = short-slotted hole = oversized hole = long-slotted hole	stren	, uth	SAO	က		69	15/16	35	8,	31	5/16
LSLP 311/16 41/16 41/2	= standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = organizated hole oriented parallel to the line of force = long-slotted hole oriented parallel to the line of force = long-slotted hole oriented transverse to the line of force	= standard hole = short-slotted hole = short-slotted hole = oversized hole = long-slotted hole = long-slotted hole = long-slotted hole = long-slotted hole	$L_{\theta} \geq L_{\theta}$	tulr ^a , in.	SSLP	3		69	15/16	35	8,	31	5/16
	= standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = oversized hole = long-slotted hole oriented parallel to the line of force = long-slotted hole oriented transverse to the line of force	= standard hole = short-slotted hole = short-slotted hole = oversized hole = long-slotted hole = long-slotted hole = long-slotted hole = long-slotted hole		1	LSLP	31	1/16	4	11/16	41,	2	47	8

F _B Ksi F _B F _B	Edge L _e , in. ASD LRFD ASD A 44.0 27.2 40.8 25.0 20.7 20.7 20.7 20.7 20.7 20.7 20.7 20			of the market	Y See	Kips/In. Inickness Nominal Bolt	Nom	ness inal Bolt [ICKNESS Nominal Bolt Diameter, d, in.	ď, in.		
Le, in. ASD LRFD A	174 58 31.5 47.0 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ 67_0 $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$ $17/0$	Hole Tues	Edge	3		8/9		3/4		1/8		-
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$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	11/4 58 26.3 39.4 24.5 36.7 22.7 34.0 20.8 2 56 29.5 44.2 27.4 41.1 25.4 38.1 23.4 2 56 36.3 54.4 43.5 65.3 44.4 66.6 42.6 3 5 5 40.6 60.9 48.8 73.1 49.8 74.6 47.7 3 5 5 5 5 5 5 5 3 5 5 5 5 5 3 5 5 5 5 5 3 5 5 5 5 3 5 5 5 5 3 5 5 5 3 5 5 5 3 5 5 5 3 5 5 5 3 5 5 5 3 5 5 5 3 5 5 5 4 5 5 5 5 5 5 5 5 5	di Si	2	58	42.4	63.6	37.0	55.5	31.5	47.3	26.1	39.2
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	SSLI, $L_0 \ge L_0 \text{full}$ 58 36.3 54.4 43.5 65.3 44.4 66.6 42.6 55.1 55.1 49.8 73.1 49.8 74.6 47.7 55.1 5.0 4.8 73.1 49.8 74.6 47.7 74.0 42.6 60.9 48.8 73.1 58.3 102 78.0 11.0 48.8 73.1 58.3 50.8 102 78.0 11.0 48.8 73.1 58.3 50.8 76.1 58.0 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.0 5.1 11.		11/4	82 59	26.3	39.4	24.5	36.7	22.7	34.0	20.8	31.3
$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	SSL, 102 Le Le IIII $L_e \ge L_e$ IIII $L_e \ge L_$	LSLT	2	83 83	36.3	54.4	43.5	65.3	44.4	66.6	42.6	63.9
$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	dge distance SSL, strength strength SSL, strength strength $11^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$ $2^5/16$	STD, SSLT, SSLP, OVS, LSLP	Lo ≥ Le tull		43.5	65.3	52.2 58.5	78.3	60.9	91.4	69.6	104
SSLT, 15/8 115/16 21/4 LSLT OVS 111/16 2 2 25/16	dge distance STD, sSLT, strull bearing 15/8 $1^{15}/16$ $2^{1}/4$ strength strength OVS $1^{11}/16$ 2 $2^{5}/16$ $e \ge L_e tull^h$, in. SSLP $1^{11}/16$ 2 $2^{5}/16$ = standard hole LSLP $2^{1}/16$ $2^{7}/16$ $2^{7}/16$ $2^{7}/16$ = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = oversized hole = oversized hole = one string hand oriented parallel to the line of force = oversized hole	LSLT	Le ≥ Le full	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
0VS 1 ¹¹ / ₁₁₆ 2 2 ⁵ / ₁₆	strength over the first strength of the line of force a postsized hole oriented parallel to the line of force and short-slotted hole oriented parallel to the line of force and short-slotted hole oriented parallel to the line of force and the line of force and the line of the line of force and the oriented parallel to the line of force and the line of the line	Edge di	stance	STD, SSLT, LSLT	15,			15/16	21/		29	91
	$e \ge L_{e} t_{uul}^{\mu}$, in. SSLP $1^{1}/_{16}$ 2 $2^{5}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ $2^{7}/_{16}$ 2^{7	strer	ngth	OVS	1-	1/16	2		25	16	25	.00
SSLP 11/16 2 23/16	= standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = oversized hole = non-slotted hole oriented parallel to the line of force	Le > Le	tulta, in.	SSLP	-	1/16	2		25/	1,16	21	91/
LSLP 21/16 27/16 27/8	= standard hole = short-slotted hole oriented transverse to the line of force = short-slotted hole oriented parallel to the line of force = oversized hole = long-slotted hole oriented parallel to the line of force			LSLP	21	16	2	1/16	27/	. 80	31	4
		Ω = 2.00	φ = 0.75	Note: Spac slot in the see AISC S	Note: Spacing indicated is from the c slot in the line of force. Hole deforma see AISC Specification Section J3.10.	Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole of Notes. The line of force. Hole deformation is considered. When hole deformation is not considered, see AIQS careful Section 33:10.	ne center of rmation is c	the hole or considered.	slot to the When hole of	center of the deformation	adjacent l is not cons	nole of idered,