Wood Design

Notation:

a = name for width dimension
A = name for area
$A_{req'd-adj}$ = area required at allowable stress when shear is adjusted to include self weight
b = width of a rectangle
= name for height dimension
c = largest distance from the neutral axis to the top or bottom edge of a beam
= constant in C_p expression
c_1 = coefficient for shear stress for a rectangular bar in torsion
C_D = load duration factor
C_{fu} = flat use factor for other than decks
C_F = size factor
C_H = shear stress factor
C_i = incising factor
C_L = beam stability factor
C_M = wet service factor
C_p = column stability factor for wood design
C_r = repetitive member factor for wood design
C_V = volume factor for glue laminated timber design
C_t = temperature factor for wood design
d = name for depth
= calculus symbol for differentiation
d_{min} = dimension of timber critical for buckling
D = shorthand for dead load
= name for diameter
DL = shorthand for dead load
E = modulus of elasticity
f = stress (strength is a stress limit)
f_b = bending stress
f_c = compressive stress
$f_{from\ table}$ = tabular strength (from table)
f_p = bearing stress
f_v = shear stress
f_{v-max} = maximum shear stress
F_{allow} = allowable stress
F_b = tabular bending strength
= allowable bending stress
F_b' = allowable bending stress (adjusted)
F_c = tabular compression strength parallel to the grain
F_c' = allowable compressive stress (adjusted)
F^*_c intermediate compressive stress for dependant on load duration
F_{cE} = theoretical allowed buckling stress
$F_{c\perp}$ = tabular compression strength perpendicular to the grain
$F_{connector}$ = shear force capacity per connector
F_p = tabular bearing strength parallel to the grain
= allowable bearing stress
F_t = tabular tensile strength
F_u = ultimate strength
F_v = tabular bending strength
= allowable shear stress
h = height of a rectangle
I = moment of inertia with respect to neutral axis bending
I_{trial} = moment of inertia of trial section
$I_{req'd}$ = moment of inertia required at limiting deflection
I_y = moment of inertia about the y axis
J = polar moment of inertia
K = effective length factor for columns
K_{cE} = material factor for wood column design
L = name for length or span length
L_e = effective length that can buckle for column design, as is ℓ_e
LL = shorthand for live load
$LRFD$ = load and resistance factor design
M = internal bending moment
M_{max} = maximum internal bending moment
$M_{max-adj}$ = maximum bending moment adjusted to include self weight
n = number of connectors across a joint, as is N
p = pitch of connector spacing
= safe connector load parallel to the grain

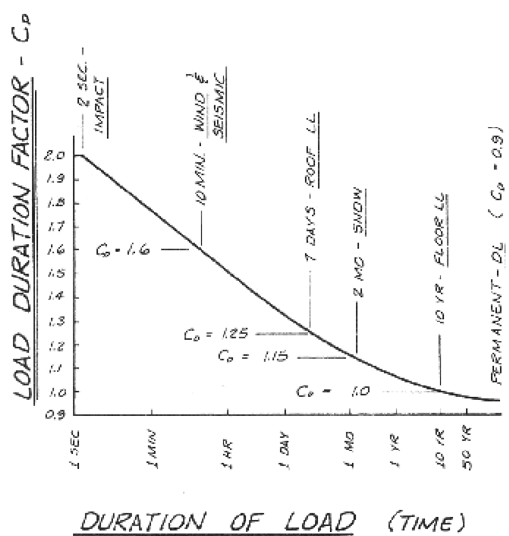
P = name for axial force vector
$P_{allowable}$ = allowable axial force
q = safe connector load perpendicular to the grain
$Q_{connected}$ = first moment area about a neutral axis for the connected part
r = radius of gyration
R = radius of curvature of a deformed beam
S = section modulus
$S_{req'd}$ = section modulus required at allowable stress
$S_{req'd\text{-}adj}$ = section modulus required at allowable stress when moment is adjusted to include self weight
T = torque (axial moment)
V = internal shear force
V_{max} = maximum internal shear force
$V_{max\text{-}adj}$ = maximum internal shear force adjusted to include self weight
w = name for distributed load
$w_{equivalent}$ = equivalent distributed load to produce the maximum moment
$w_{self\ wt}$ = name for distributed load from self weight of member
x = horizontal distance
y = vertical distance
Z = force capacity of a connector
Δ_{actual} = actual beam deflection
$\Delta_{allowable}$ = allowable beam deflection
Δ_{limit} = allowable beam deflection limit
Δ_{max} = maximum beam deflection
ϕ = resistance factor for LRFD
γ = density or unit weight
θ = slope of the beam deflection curve
ρ = radial distance
$\int$ = symbol for integration
Σ = summation symbol

Wood or Timber Design

Structural design standards for wood are established by the *National Design Specification (NDS)* published by the National Forest Products Association. There is a combined specification (from 2005) for **Allowable** Stress Design and limit state design (LRFD).

Tabulated wood strength values are used as the base allowable strength and modified by appropriate **adjustment** factors:

$$f = C_D C_M C_F \ldots \times f_{from\,table}$$



(also see Table 5.2 in text)

Size and Use Categories

Boards:	1 to 1½ in. thick	2 in. and wider
Dimension lumber	2 to 4 in. thick	2 in. and wider
Timbers	5 in. and thicker	5 in. and wider

Adjustment Factors
(partial list)

C_D load duration factor
C_M wet service factor
(1.0 dry < 16% moisture content)
C_F size factor for visually graded sawn lumber and round timber > 12" depth

$$C_F = (12/d)^{1/9} \leq 1.0$$

C_{fu} flat use factor (excluding decking)
C_i incising factor (from increasing the depth of pressure treatment)
C_t temperature factor (at high temperatures **strength** decreases)
C_r repetitive member factor
C_H shear stress factor (amount of splitting)
C_V volume factor for glued laminated timber (similar to C_F)
C_L beam stability factor (for beams without full lateral support)

Tabular Design Values

F_b: bending stress
F_t: tensile stress
F_v: horizontal shear stress
$F_{c\perp}$: compression stress (perpendicular to grain)
F_c: compression stress (parallel to grain)
E: modulus of elasticity
F_p: bearing stress (parallel to grain)

Wood is significantly weakest in **shear** and strongest along the direction of the grain (tension and compression).

Load Combinations and Deflection

The critical load combination (ASD) is determined by the largest of either:

$$\frac{dead\ load}{0.9}\ or\ \frac{(dead\ load + any\ combination\ of\ live\ load\ )}{C_D}$$

The deflection limits may be increased for less stiffness with total load: LL + 0.5(DL)

Criteria for Design of Beams

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

$$F_b'\ or\ \phi F_u \geq f_b = \frac{Mc}{I}$$

Knowing M and F'$_b$, the minimum section modulus fitting the limit is:

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

Besides strength, we also need to be concerned about *serviceability*. This involves things like limiting deflections & cracking, controlling noise and vibrations, preventing excessive settlements of foundations and durability. When we know about a beam section and its material, we can determine beam deformations.

Determining Maximum Bending Moment

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

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

Determining Maximum Bending Stress

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

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

Deflections

If the bending moment changes, M(x) across a beam of constant material and cross section then the curvature will change:

$$\frac{1}{R} = \frac{M(x)}{EI}$$

The slope of the n.a. of a beam, θ, will be tangent to the radius of curvature, R:

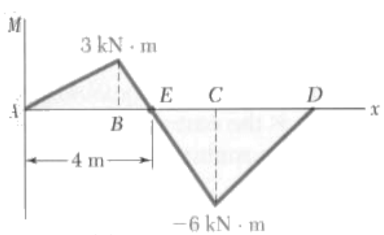
$$\theta = slope = \frac{1}{EI}\int M(x)dx$$

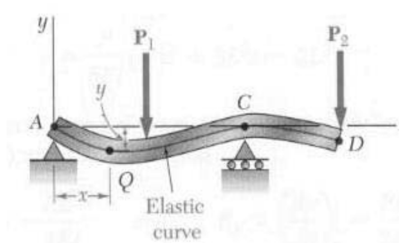
The equation for deflection, y, along a beam is:

$$y = \frac{1}{EI}\int \theta dx = \frac{1}{EI}\iint M(x)dx$$

Elastic curve equations can be found in handbooks, textbooks, design manuals, etc...Computer programs can be used as well (like *Multiframe*).

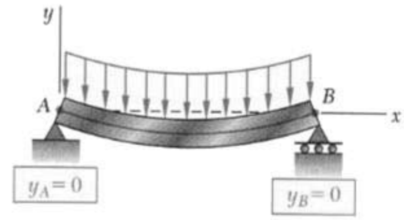
Elastic curve equations can be **superpositioned** ONLY if the stresses are in the elastic range. *The deflected shape is roughly the same shape flipped as the bending moment diagram but is constrained by supports and geometry.*



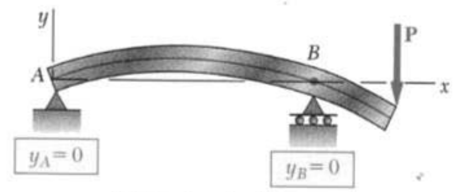


Boundary Conditions

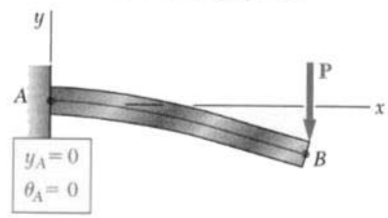
The boundary conditions are geometrical values that we know – slope or deflection – which may be restrained by supports or symmetry.



(*a*) Simply supported beam



(*b*) Overhanging beam



(*c*) Cantilever beam

At Pins, Rollers, Fixed Supports: $y = 0$

At Fixed Supports: $\theta = 0$

At Inflection Points From Symmetry: $\theta = 0$

The Slope Is Zero At The Maximum Deflection y_{max}:

$$\theta = \frac{dy}{dx} = slope = 0$$

Allowable Deflection Limits

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

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

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

Lateral Buckling

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

Beam Loads & Load Tracing

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

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

Design Procedure

The intent is to find the most light weight member satisfying the section modulus size.

1. Know F_{all} (F'_b) for the material or F_U for LRFD.
2. Draw V & M, finding M_{max}.
3. Calculate $S_{req'd}$. This step is equivalent to determining $f_b = \frac{M_{max}}{S} \leq F'_b$
4. For rectangular beams $S = \frac{bh^2}{6}$

   - For timber: use the section charts to find S that will work *and remember that the beam self weight will increase $S_{req'd.}$* $w_{self\ wt} = \gamma A$

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

5. Consider lateral stability.
6. Evaluate horizontal shear stresses using V_{max} to determine if $f_v \leq F'_v$ *or* find $A_{req'd}$

   For rectangular beams $f_{v-max} = \frac{3V}{2A} = 1.5\frac{V}{A}$ $\quad \therefore A_{req'd} \leq \frac{3V}{2F'_v}$

7. Provide adequate bearing area at supports: $f_p = \frac{P}{A} \leq F'_p$ (*from* F_c *or* $F_{c\perp}$)
8. Evaluate shear due to torsion $f_v = \frac{T\rho}{J}$ *or* $\frac{T}{c_1 ab^2} \leq F'_v$

   (circular section or rectangular)

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

**** *note: when $\Delta_{calculated} > \Delta_{limit}$, $I_{required}$ can be found with: and $S_{req'd}$ will be satisfied for similar self weight ******

$$I_{req'd} \geq \frac{\Delta_{too\ big}}{\Delta_{limit}} I_{trial}$$

FOR ANY EVALUATION:

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

Load Tables for Uniformly Loaded Joists & Rafters

Tables exists for the common loading situation for joists and rafters – that 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. If the load is *not uniform*, an *equivalent distributed load* can be calculated from the maximum moment equation.

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

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

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

Decking

Flat panels or planks 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 or vertical (if the panels are used in a shear wall) unit tying the sheathing to the joists or studs that resists forces parallel to the surface of the diaphragm.

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.

Any slenderness ratio, $L_e/d \le 50$:

$$f_c = \frac{P}{A} \le F'_c \qquad\qquad F'_c = F_c\left(C_D\right)\left(C_M\right)\left(C_t\right)\left(C_F\right)\left(C_p\right)$$

The allowable stress equation uses factors to replicate the combination crushing-buckling curve:

where:

F'_c = allowable compressive stress parallel to the grain
F_c = compressive strength parallel to the grain
c = 0.8 for sawn lumber, 0.85 for poles, 0.9 for glulam timber
C_D = load duration factor
C_M = wet service factor (1.0 for dry)
C_t = temperature factor
C_F = size factor
C_p = column stability factor off chart or equation:

$$C_p = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+F_{cE}/F_c^*}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

For preliminary column design:

$$F'_c = F_c^* C_p = \left(F_c C_D\right)C_p$$

Procedure for Analysis

1. Calculate L_e/d_{min} (KL/d for each axis and chose largest)
2. Obtain F'_c

   compute $F_{cE} = \dfrac{K_{cE}E}{\left(\frac{l_e}{d}\right)^2}$ *or* $\dfrac{0.822E'_{\min}}{\left(\frac{L_e}{d}\right)^2}$ with K_{cE} =0.3 for sawn, = 0.418 for glu-lam

3. Compute $F_c^* \cong F_c C_D$ with C_D = 1, normal, C_D =1.25 for 7 day roof...
4. Calculate F_{cE}/F_c^* and get C_p from table or calculation
5. Calculate $F'_c = F_c^* C_p$
6. Compute $P_{allowable} = F'_c \cdot A$ *or alternatively* compute $f_{actual} = P/A$
7. Is the design satisfactory?

   Is $P \leq P_{allowable}$? $\Rightarrow$ yes, it is; no, it is no good

or Is $f_{actual} \leq F'_c$? $\Rightarrow$ yes, it is; no, it is no good

Procedure for Design

1. Guess a size by picking a section
2. Calculate L_e/d_{min} (KL/d for each axis and chose largest)
3. Obtain F'_c

   compute $F_{cE} = \dfrac{K_{cE}E}{\left(\frac{l_e}{d}\right)^2}$ *or* $\dfrac{0.822E'_{\min}}{\left(\frac{L_e}{d}\right)^2}$ with K_{cE} =0.3 for sawn, = 0.418 for glu-lam

4. Compute $F_c^* \cong F_c C_D$ with C_D = 1, normal, C_D =1.25 for 7 day roof...
5. Calculate F_{cE}/F_c^* and get C_p from table or calculation
6. Calculate $F'_c = F_c^* C_p$
7. Compute $P_{allowable} = F'_c \cdot A$ *or alternatively* compute $f_{actual} = P/A$
8. Is the design satisfactory?

   Is $P \leq P_{allowable}$? $\Rightarrow$ yes, it is; *no, pick a bigger section and go back to step 2.*

or Is $f_{actual} \leq F'_c$? $\Rightarrow$ yes, it is; *no, pick a bigger section and go back to step 2.*

Columns with Bending (Beam-Columns)

The modification factors are included in the form: $$\left[\frac{f_c}{F'_c}\right]^2 + \frac{f_{bx}}{F'_{bx}\left[1-\frac{f_c}{F_{cEx}}\right]} \le 1.0$$
where:

$1-\frac{f_c}{F_{cEx}}$ = magnification factor accounting for P-Δ

F'_{bx} = allowable bending stress

f_{bx} = working stress from bending about x-x axis

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 S (=I/c_{max})

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 stress test?
   - If not, do you *increase* r or A or S?
   - If so, is the difference really big so that you could *decrease* r or A or S 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.

Criteria for Design of Connections

Connections for wood are typically mechanical fasteners. Shear plates and split ring connectors are common in trusses. Bolts of metal bear on holes in wood, and nails rely on shear resistance transverse and parallel to the nail shaft. Timber rivets with steel side plates are allowed with glue laminated timber.

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

Bolted Joints

Stress must be evaluated in the member being connected using the load being transferred and the reduced cross section area called *net area*. Bolt capacities are usually provided in tables and take into account the allowable shearing stress across the diameter for *single* and *double shear*, and the allowable bearing stress of the connected material based on the direction of the load with respect to the grain. Problems, such as ripping of the bolt hole at the end of the member, are avoided by following code guidelines on minimum edge distances and spacing.

Nailed Joints

Because nails rely on shear resistance, a common problem when nailing is splitting of the wood at the end of the member, which is a shear failure. Tables list the shear force capacity per unit length of embedment per nail. Jointed members used for beams will have shear stress across the connector, and the pitch spacing, *p*, can be determined from the shear stress equation when the capacity, *F*, is known:

$$nF_{connector} \geq \frac{VQ_{connected\,area}}{I} \cdot p$$

Example 1 (pg 204)

Example 2. A simple beam has a span of 16 ft [4.88 m] and supports a total uniformly distributed load, including its own weight, of 6500 lb [28.9 kN]. Using Douglas fir-larch, select structural grade, determine the size of the beam with the least cross-sectional area on the basis of limiting bending stress. Density of douglas fir-larch is 32 lb/ft^3

Example 2 (pg 207)

Example 3. A 6 × 10 beam of Douglas fir-larch, No. 2 grade, has a total horizontally distributed load of 6000 lb [26.7 kN]. Investigate for shear stress.

Example 3 (pg 209)

Example 6. A two-span 3 × 12 beam of Douglas fir-larch, No. 1 grade, bears on a 3 × 14 beam at its center support. If the reaction force is 4200 lb [18.7 kN], is this safe for bearing?

Example 4 (pg 212)

Example 7. An 8 × 12 wood beam with E = 1,600,000 psi is used to carry a total uniformly distributed load of 10 kips on a simple span of 16 ft. Find the maximum deflection of the beam.

Example 5 (pg 223)

Example 13. Using Table 5.10 select joists to carry a live load of 40 psf and a dead load of 10 psf on a span of 15 ft 6 in. if the spacing is 16 in. on center.

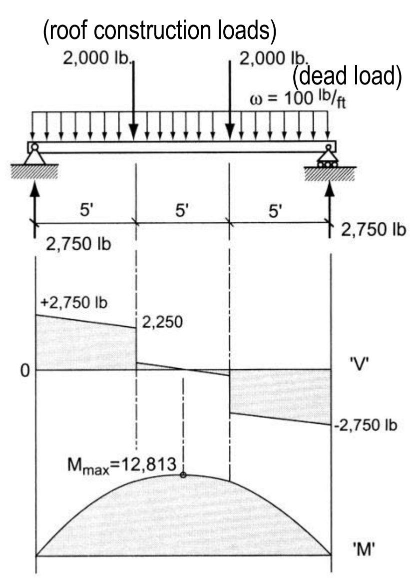
TABLE 5.10 Maximum Spans for Floor Joists (ft-in.)[a]

	Joist Size			
Spacing (in.)	2 × 6	2 × 8	2 × 10	2 × 12
Live load = 40 psf, Dead load = 10 psf, Maximum live-load deflection = L/360				
12	10-9	14-2	17-9	20-7
16	9-9	12-7	15-5	17-10
19.2	9-1	11-6	14-1	16-3
24	8-1	10-3	12-7	14-7
Live load = 40 psf, Dead load = 20 psf, Maximum live-load deflection = L/360				
12	10-6	13-3	16-3	18-10
16	9-1	11-6	14-1	16-3
19.2	8-3	10-6	12-10	14-10
24	7-5	9-50	11-6	13-4

Source: Compiled from data in the *International Building Code* (Ref. 4), with permission of the publisher, International Code Council.
[a] Joists are Douglas fir–larch, No. 2 grade. Assumed maximum available length of single piece is 26 ft.

Example 6

Design a Southern pine No. 1 beam to carry the loads shown (roof beam, no plaster). Assume the beam is supported at each end of by an 8" block wall. F_b = 1500 psi; F_v = 110 psi; $F_{c\perp}$ = 440 psi; E = 1.6 x 10^6 psi; γ = 36.3 lb/ft^3.



SOLUTION:

Because this beam appears to support other beams at the locations of the roof construction loads, we have to assume that this beam is not closely spaced to others and the repetitive use adjustment factor doesn't apply. The load duration factor, C_D, is 1.25 for roof construction loads. The other conditions (like temperature and moisture) must be assumed to be normal (and have values of 1.0). The allowable stresses can be determined from:

$F'_b = C_D F_b$ = (1.25)(1500) = 1875 psi $F'_v = C_D F_v$ = (1.25)(110) = 137.5 psi

$F'_{c\perp} = C_D F_{c\perp}$ = (1.25)(440) = 550 psi

Bending:

$$S_{req'd} \geq \frac{M}{F'_b} = \frac{12{,}813^{lb-ft}(12^{in}/_{ft})}{1875\,psi} = 82.0 in^3$$

Shear:

$$A_{req'd} \geq \frac{3V}{2F'_v} = \frac{3(2{,}750lb)}{2(137.5psi)} = 30.0in^2$$

Try a 3 x 16. This satisfies both requirements with the least amount of area. (See the 4 x 14, 6 x 10, and 8 x 10.)

(A = 38.13 in^2, S = 96.90 in^3, I_x = 738.9 in^4)

$$w_{self\ wt.} = \gamma A = \frac{36.3\,^{lb}\!/\!_{ft^3}(38.13in^2)}{(12\,^{in}\!/\!_{ft})^2} = 9.61\,^{lb}\!/\!_{ft}$$ which is _additional_ dead load!

Because the maximum moment from the additional distributed load is at the same location as the maximum moment from the diagram, we can add them:

$$M_{adjusted} = 12{,}813^{lb-ft} + \frac{9.61\,^{lb}\!/\!_{ft}(15ft)^2}{8} = 13{,}083.3^{lb-ft} \quad and \quad S^*_{req'd} \geq \frac{13{,}083.3^{lb-ft}(12\,^{in}\!/\!_{ft})}{1875psi} = 83.7in^3$$

The same holds true for the contribution to the shear:

$$V_{adjusted} = 2{,}750lb + \frac{9.61\,^{lb}\!/\!_{ft}(15ft)}{2} = 2822.1lb \qquad and \quad A^*_{req'd} \geq \frac{3(2822.1lb)}{2(137.5psi)} = 30.79in^2$$

<u>Check that the section chosen satisfies the new required section modulus and area:</u>

Is $S_{\text{that I have}} \geq S_{\text{that I need}}$? Is 96.90 in^3 ≥ 83.7 in^3? Yes, OK.

Is $A_{\text{that I have}} \geq A_{\text{that I need}}$? Is 38.13 in^2 ≥ 30.79 in^2? Yes, OK.

NOTE: If the area or section that I have is not adequate, I need to choose one that is. This will have a **larger** self weight that must be determined and included in the maximum moment (with the initial maximum). It will make $S^*_{req'd}$ and $A^*_{req'd}$ bigger as well, and the new section properties must be evaluated with respect to these new values.

Deflection:

The total deflection due to dead and live loads must not exceed a limit specified by the building code adopted (for example, the *International Building Code*) or recommended by construction manuals. For a commercial roof beam with no plaster, the usual limits are L/360 for *live load only* and L/240 for *live and dead load*.

$$\Delta_{LL-limit} = \frac{15ft(12\,^{in}\!/\!_{ft})}{360} = 0.5in \qquad \Delta_{total-limit} = \frac{15ft(12\,^{in}\!/\!_{ft})}{240} = 0.75in$$

<u>*Superpositioning*</u> *(combining or superimposing)* of several load conditions can be performed, but **care must be taken** that the deflections calculated for the separate cases to obtain the maximum ***must be deflections at the same location*** in order to be added together:

two symmetrically placed equal point loads (live load): (a is the distance from the supports to the loads)

$$\Delta_{max}(at\ center) = \frac{Pa}{24EI}(3l^2 - 4a^2) = \frac{2000lb(5ft)}{24(1.6x10^6\,psi)(738.9in^4)}(3(15ft)^2 - 4(5ft)^2)(12\,^{in}\!/\!_{ft})^3 = 0.35in$$

distributed load (dead load)

$$\Delta_{max}(at\ center) = \frac{5wl^4}{384EI} = \frac{5(100 + 9.61\,^{lb}\!/\!_{ft})(15ft)^4(12\,^{in}\!/\!_{ft})^3}{384(1.6x10^6\,psi)(738.9in^4)} = 0.11in$$

Is $\Delta_{\text{live that I have}} \leq \Delta_{\text{live-limit}}$? Is 0.35 in ≤ 0.5 in? Yes, OK.

Is $\Delta_{\text{total that I have}} \leq \Delta_{\text{total-limit}}$? Is (0.35 in + 0.11 in) = 0.46 in ≤ 0.75 in? Yes, OK.

Bearing:

Determine if the bearing stress between the beam and the block wall support less than the allowable. If it is not, the beam width must be increased:

$$f_p = \frac{P}{A} = \frac{2822.1lb}{(2.5in)(8in)} = 141.1psi \leq F'_{c\perp} = 550psi$$ so, yes the beam width (2.5 in) is adequate.

USE a 3 x 16.

Example 7 (pg 239)

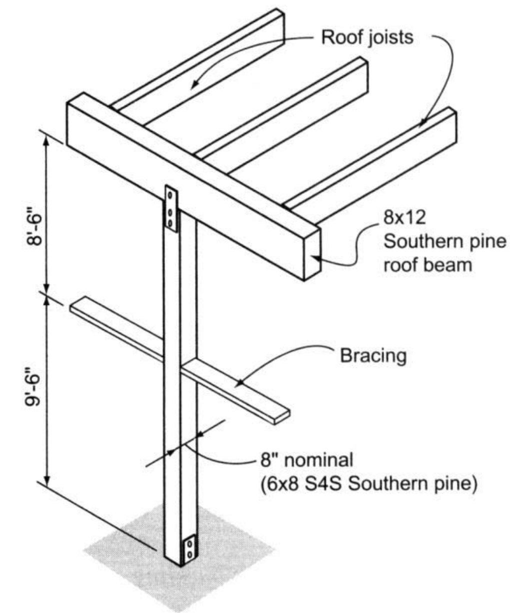
Example 1. A wood column consists of a 6 × 6 of Douglas fir-larch, No. 1 grade. Find the safe axial compression load for unbraced lengths of: (1) 2 ft, (2) 8 ft, (3) 16 ft. using the ASD method.

Example 8

Example Problem 10.18 (Figures 10.60 and 10.61)

An 18' tall 6×8 Southern pine column supports a roof load (dead load plus a 7-day live load) equal to 16 kips. The weak axis of buckling is braced at a point 9'6" from the bottom support. Determine the adequacy of the column.

$F_c = 975$ psi, $E = 1.6 \times 10^6$ psi



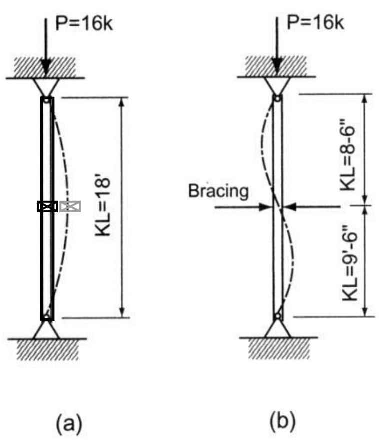


Figure 10.61 (a) Strong axis. (b) Weak axis.

Example 8 (fully worked)

Example Problem 10.18 (Figures 10.60 and 10.61)

An 18' tall 6×8 Southern pine column supports a roof load (dead load plus a 7-day live load) equal to 16 kips. The weak axis of buckling is braced at a point 9'6" from the bottom support. Determine the adequacy of the column.

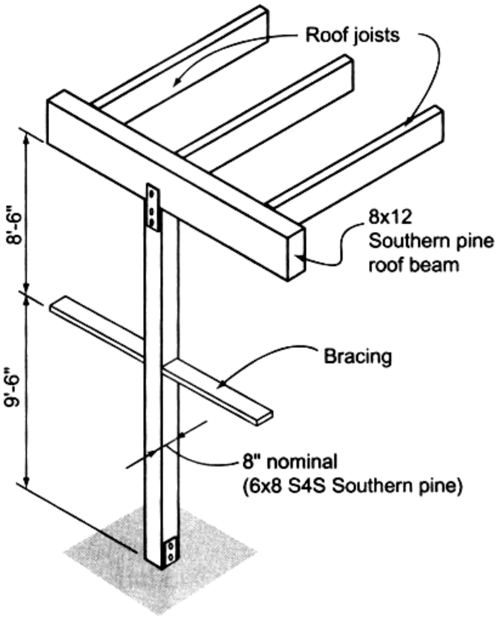


Figure 10.60 Wood column with intermediate bracing.

Solution:

6×8 S4S Southern pine post: (A = 41.25 in.², F_c = 975 psi, $E = 1.6 \times 10^6$ psi)

Check the slenderness ratio about the weak axis:

$$\frac{L_e}{d} = \frac{(9.5' \times 12 \text{ in./ft.})}{5.5"} = 20.7$$

The slenderness ratio about the strong axis is:

$$\frac{L_e}{d} = \frac{(18' \times 12 \text{ in./ft.})}{7.5"} = 28.8 \leftarrow \text{governs}$$

$$F_{cE} = \frac{0.3E}{(L_e/d)^2} = \frac{0.3\,(1.6 \times 10^6 \text{ lb./in.}^2)}{(28.8)^2} = 579 \text{ psi}$$

$$F_c^* \cong F_c C_D = (975 \text{ lb./in.}^2)(1.25) = 1220 \text{ psi}$$

$$F_{cE} \big/ F_c^* = \frac{579\,psi}{1220\,psi} = 0.475$$

where: $C_D = 1.25$ for 7-day-duration load

From Appendix Table 14: $C_p = 0.412$

$$\therefore\ F'_c = F_c^* C_p = 1220 \text{ lb./in.}^2 \times 0.412 = 503 \text{ psi}$$

$$P_a = F'_c \times A = (503 \text{ lb./in.}^2) \times (41.25 \text{ in.}^2)$$

$$= 20{,}700 \text{ lb.}$$

$$P_a = 20.7 \text{ k} > P_{\text{actual}} = 16 \text{ k}$$

The column is adequate.

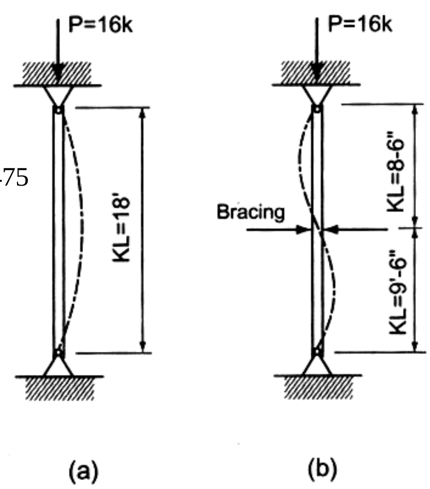
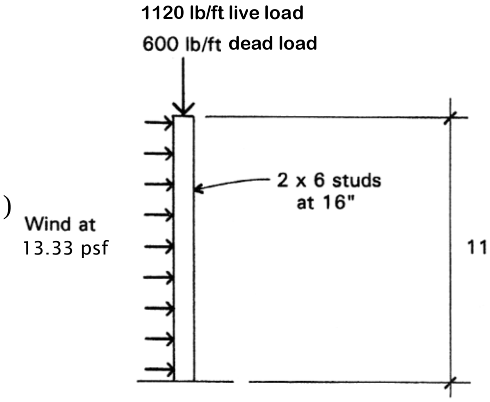


Figure 10.61 (a) Strong axis. (b) Weak axis.



Example 9 (pg 251)

***Example** 4.* An exterior wall stud of Douglas fir-larch, stud grade, is loaded as shown in Figure 6.5*a.* Investigate the stud for the combined loading. (*Note:* This is the wall stud from the building example in Chapter 18.) (*Wind load duration does apply, as well as load combinations.*)

Example 10 (pg 264)

Example 2. The truss heel joint shown in Figure 7.5 is made with 2-in. nominal thickness lumber and gusset plates of ½-in.-thick plywood. Nails are 6d common wire with the nail layout shown occurring in both sides of the joint. Find the tension load capacity for the bottom chord member (load 3 in the figure).

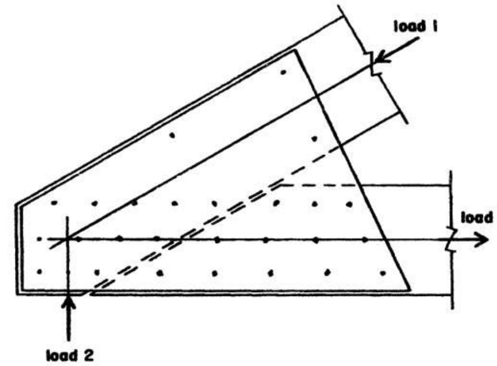


TABLE 7.1 Reference Lateral Load Values for Common Wire Nails (lb/in.)

Side Member Thickness, t_s (in.)	Nail Length, L (in.)	Nail Diameter, D (in.)	Nail Pennyweight	Load per Nail, Z (lb)
Part 1 — With Wood Structural Panel Side Members[a] (G = 0.42)				
	2	0.113	6d	48
3/8	2½	0.131	8d	63
	3	0.148	10d	76
	2	0.113	6d	50
15/32	2½	0.131	8d	65
	3	0.148	10d	78
	3½	0.162	16d	92

Example 11

A nominal 4 x 6 in. redwood beam is to be supported by two 2 x 6 in. members acting as a spaced column. The minimum spacing and edge distances for the ½ inch bolts are shown. How many ½ in. bolts will be required to safely carry a load of 1500 lb? Use the chart provided.

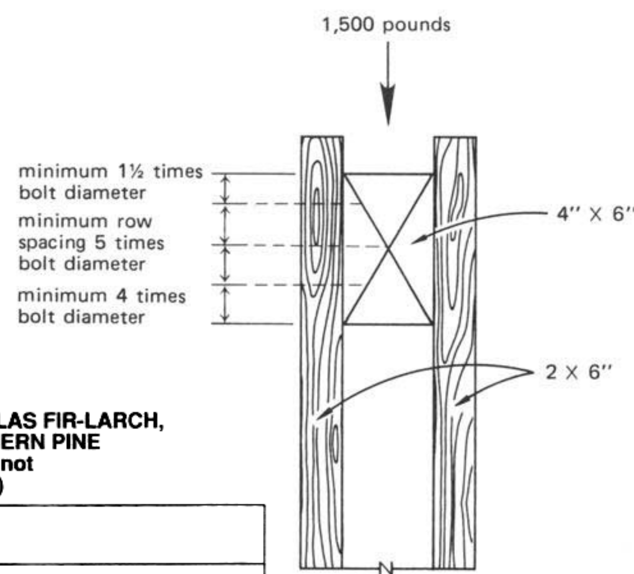


TABLE 23-I-F—HOLDING POWER OF BOLTS[1,2,3] FOR DOUGLAS FIR-LARCH, CALIFORNIA REDWOOD (CLOSE GRAIN) AND SOUTHERN PINE (See U.B.C. Standard 23-17 where members are not of equal size and for values in other species.)

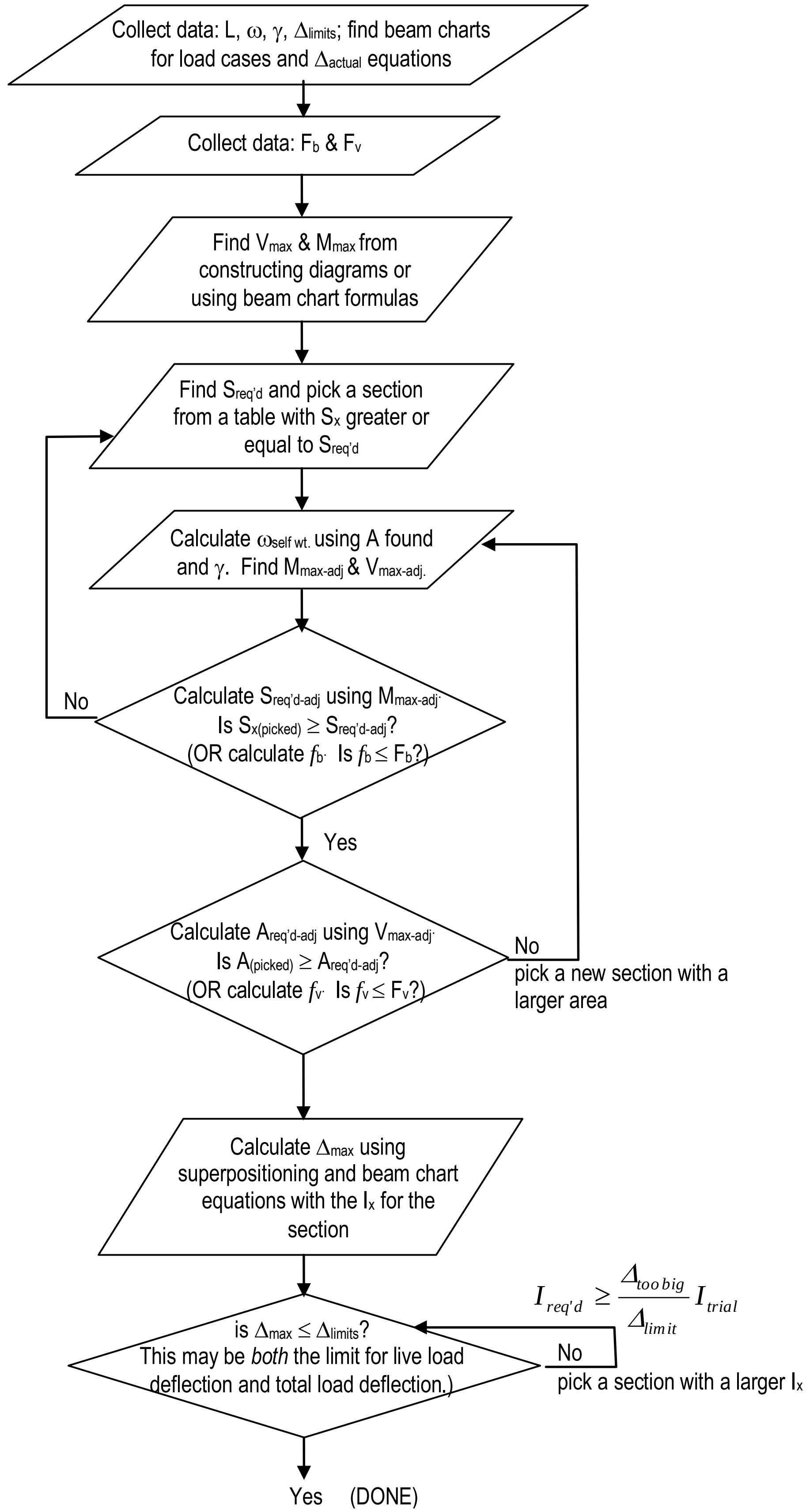
p = safe loads parallel to grain, in pounds.
q = safe loads perpendicular to grain, in pounds.

× 4.45 for N

LENGTH OF BOLT IN MAIN WOOD MEMBER[4] (inches)		DIAMETER OF BOLT (inches) 3/8	1/2	5/8	3/4	7/8	1
× 25.4 for mm							
2½	Single p		630	910	1,155	1,370	1,575
	Shear q		360	405	450	495	540
	Double p	710	1,260	1,820	2,310	2,740	3,150
	Shear q	620	720	810	900	990	1,080
3½	Single p			990	1,400	1,790	2,135
	Shear q			565	630	695	760
	Double p	710	1,270	1,980	2,800	3,580	4,270
	Shear q	640	980	1,130	1,260	1,390	1,520

[1]Tabulated values are on a normal load-duration basis and apply to joints made of seasoned lumber used in dry locations. See Division III for other service conditions.
[2]Double shear values are for joints consisting of three wood members in which the side members are one half the thickness of the main member. Single shear values are for joints consisting of two wood members having a minimum thickness not less than that specified.
[3]See Division III for wood-to-metal bolted joints.
[4]The length specified is the length of the bolt in the main member of double shear joints or the length of the bolt in the thinner member of single shear joints.

ASD Beam Design Flow Chart



SECTION PROPERTIES / STANDARD SIZES

To the extent that other considerations will permit, the finished sizes of structural glued laminated timber as given in Table B constitute normal industry practice. Industry standards do, however, permit the use of any depth or width of glued laminated timber. Dimension lumber of 1½ in. net thickness is normally used for laminating straight members.
The modified section modulus includes size factor (C_F), and no further reduction of bending stress for size is needed.

DEPTH, d in.	AREA, A in.^{2}	MODIFIED SECTION MODULUS, SC_F in.^{3}	MOMENT OF INERTIA, I in.^{4}
3⅛" WIDTH			
6.0	18.8	18.8	56
7.5	23.4	29.3	110
9.0	28.1	42.2	190
10.5	32.8	57.4	302
12.0	37.5	75.0	450
13.5	42.2	93.7	641
15.0	46.9	114.3	879
16.5	51.6	136.9	1,170
18.0	56.3	161.3	1,519
19.5	60.9	187.6	1,931
21.0	65.6	215.8	2,412
22.5	70.3	245.9	2,966
24.0	75.0	277.8	3,600
5⅛" WIDTH			
7.5	38.4	48.0	180
9.0	46.1	69.2	311
10.5	53.8	94.2	494
12.0	61.5	123.0	738
13.5	69.2	153.6	1,051
15.0	76.9	187.5	1,441
16.5	84.6	224.5	1,919
18.0	92.3	264.6	2,491
19.5	99.9	307.7	3,167
21.0	107.6	354.0	3,955
22.5	115.3	403.2	4,865
24.0	123.0	455.5	5,904
25.5	130.7	510.8	7,082
27.0	138.4	569.0	8,406
28.5	146.1	630.2	9,887
30.0	153.8	694.3	11,531
31.5	161.4	761.4	13,349
33.0	169.1	831.3	15,348
34.5	176.8	904.1	17,538
36.0	184.5	979.8	19,926
6¾" WIDTH			
12.0	81.0	162.0	972
13.5	91.1	202.4	1,384
15.0	101.3	246.9	1,898
16.5	111.4	295.6	2,527
18.0	121.5	348.4	3,280
19.5	131.6	405.3	4,171
21.0	141.8	466.2	5,209
22.5	151.9	531.1	6,407
24.0	162.0	600.0	7,776
25.5	172.1	672.8	9,327
27.0	182.3	749.5	11,072
28.5	192.4	830.0	13,021
30.0	202.5	914.5	15,188
31.5	212.6	1,002.8	17,581
33.0	222.8	1,094.9	20,215
34.5	232.9	1,190.8	23,098
36.0	243.0	1,290.5	26,244
37.5	253.1	1,393.9	29,663
39.0	263.3	1,501.1	33,367
40.5	273.4	1,612.0	37,367
42.0	283.5	1,726.6	41,674
43.5	293.6	1,845.0	46,301
45.0	303.8	1,967.0	51,258
46.5	313.9	2,092.6	56,556
48.0	324.0	2,222.0	62,208
8¾" WIDTH			
12.0	105.0	210.0	1,260
13.5	118.1	262.3	1,794
15.0	131.3	320.1	2,461
16.5	144.4	383.2	3,276
18.0	157.5	451.7	4,252
19.5	170.6	525.4	5,407
21.0	183.8	604.4	6,753
22.5	196.9	688.5	8,306
24.0	210.0	777.7	10,080
25.5	223.1	872.1	12,091
27.0	236.3	971.5	14,352
28.5	249.4	1,076.0	16,880
30.0	262.5	1,185.5	19,688
31.5	275.6	1,299.9	22,791
33.0	288.8	1,419.3	26,204
34.5	301.9	1,543.6	29,942
36.0	315.0	1,672.8	34,020
37.5	328.1	1,806.9	38,452
39.0	341.3	1,945.9	43,253
40.5	354.4	2,089.6	48,439
42.0	367.5	2,238.2	54,022
43.5	380.6	2,391.6	60,020
45.0	393.8	2,549.8	66,445
46.5	406.9	2,712.7	73,314
48.0	420.0	2,880.3	80,640
49.5	433.1	3,052.7	88,439
51.0	446.3	3,229.8	96,725
52.5	459.4	3,411.6	105,513
54.0	472.5	3,598.0	114,818
55.5	485.6	3,789.1	124,654
57.0	498.8	3,984.9	135,037
58.5	511.9	4,185.3	145,980
60.0	525.0	4,390.3	157,500
10¾" WIDTH			
15.0	161.3	393.3	3,023
16.5	177.4	470.8	4,024
18.0	193.5	554.9	5,224
19.5	209.6	645.5	6,642
21.0	225.8	742.5	8,296
22.5	241.9	845.8	10,204
24.0	258.0	955.5	12,384
25.5	274.1	1,071.4	14,854
27.0	290.3	1,193.6	17,633
28.5	306.4	1,321.9	20,738
30.0	322.5	1,456.4	24,188
31.5	338.6	1,597.0	28,000
33.0	354.8	1,743.7	32,194
34.5	370.9	1,896.4	36,786
36.0	387.0	2,055.2	41,796
37.5	403.1	2,219.9	47,241
39.0	419.3	2,390.6	53,140
40.5	435.4	2,567.3	59,510
42.0	451.5	2,749.8	66,370
43.5	467.6	2,938.3	73,739
45.0	483.8	3,132.6	81,633
46.5	499.9	3,332.7	90,071
48.0	516.0	3,538.7	99,072
49.5	532.1	3,750.5	108,653
51.0	548.3	3,968.0	118,833
52.5	564.4	4,191.4	129,630
54.0	580.5	4,420.4	141,062
55.5	596.6	4,655.2	153,146
57.0	612.8	4,895.7	165,902
58.5	628.9	5,141.9	179,347
60.0	645.0	5,398.8	193,500
61.5	661.1	5,651.4	208,379
63.0	677.3	5,914.5	224,000
64.5	693.4	6,183.3	240,384
66.0	709.5	6,457.8	257,548
67.5	725.6	6,737.8	275,511
69.0	741.8	7,023.4	294,289
70.5	757.9	7,314.6	313,902
72.0	774.0	7,611.3	334,368
73.5	790.1	7,913.6	355,704

Column Stability Factors

from Statics and Strength of Materials: Foundations for Structural Design, Onouye

Table 14 Column Stability Factor C_p.

"C_p" $F_c' = C_p \cdot F_c^{*}$ $F_{CE} = \frac{.30\,E}{(l/d)^2}$ for sawed posts $F_{CE} = \frac{.418\,E}{(l/d)^2}$ for glu-lam posts

F_{CE}/F_C^*	Sawed C_p	Glu-Lam C_p
0.00	0.000	0.000
0.01	0.010	0.010
0.02	0.020	0.020
0.03	0.030	0.030
0.04	0.040	0.040
0.05	0.049	0.050
0.06	0.059	0.060
0.07	0.069	0.069
0.08	0.079	0.079
0.09	0.088	0.089
0.10	0.098	0.099
0.11	0.107	0.109
0.12	0.117	0.118
0.13	0.126	0.128
0.14	0.136	0.138
0.15	0.145	0.147
0.16	0.154	0.157
0.17	0.164	0.167
0.18	0.173	0.176
0.19	0.182	0.186
0.20	0.191	0.195
0.21	0.200	0.205
0.22	0.209	0.214
0.23	0.218	0.224
0.24	0.227	0.233
0.25	0.235	0.242
0.26	0.244	0.252
0.27	0.253	0.261
0.28	0.261	0.270
0.29	0.270	0.279
0.30	0.278	0.288
0.31	0.287	0.297
0.32	0.295	0.306
0.33	0.304	0.315
0.34	0.312	0.324
0.35	0.320	0.333
0.36	0.328	0.342
0.37	0.336	0.351
0.38	0.344	0.360
0.39	0.352	0.368

F_{CE}/F_C^*	Sawed C_p	Glu-Lam C_p
0.40	0.360	0.377
0.41	0.367	0.386
0.42	0.375	0.394
0.43	0.383	0.403
0.44	0.390	0.411
0.45	0.398	0.420
0.46	0.405	0.428
0.47	0.412	0.436
0.48	0.419	0.444
0.49	0.427	0.453
0.50	0.434	0.461
0.51	0.441	0.469
0.52	0.448	0.477
0.53	0.454	0.484
0.54	0.461	0.492
0.55	0.468	0.500
0.56	0.474	0.508
0.57	0.481	0.515
0.58	0.487	0.523
0.59	0.494	0.530
0.60	0.500	0.538
0.61	0.506	0.545
0.62	0.512	0.552
0.63	0.518	0.559
0.64	0.524	0.566
0.65	0.530	0.573
0.66	0.536	0.580
0.67	0.542	0.587
0.68	0.548	0.593
0.69	0.553	0.600
0.70	0.559	0.607
0.71	0.564	0.613
0.72	0.569	0.619
0.73	0.575	0.626
0.74	0.580	0.632
0.75	0.585	0.638
0.76	0.590	0.644
0.77	0.595	0.650
0.78	0.600	0.655
0.79	0.605	0.661

F_{CE}/F_C^*	Sawed C_p	Glu-Lam C_p
0.80	0.610	0.667
0.81	0.614	0.672
0.82	0.619	0.678
0.83	0.623	0.683
0.84	0.628	0.688
0.85	0.632	0.693
0.86	0.637	0.698
0.87	0.641	0.703
0.88	0.645	0.708
0.89	0.649	0.713
0.90	0.653	0.718
0.91	0.658	0.722
0.92	0.661	0.727
0.93	0.665	0.731
0.94	0.669	0.735
0.95	0.673	0.740
0.96	0.677	0.744
0.97	0.680	0.748
0.98	0.684	0.752
0.99	0.688	0.756
1.00	0.691	0.760
1.01	0.694	0.764
1.02	0.698	0.767
1.03	0.701	0.771
1.04	0.704	0.774
1.05	0.708	0.778
1.06	0.711	0.781
1.07	0.714	0.784
1.08	0.717	0.788
1.09	0.720	0.791
1.10	0.723	0.794
1.11	0.726	0.797
1.12	0.729	0.800
1.13	0.731	0.803
1.14	0.734	0.806
1.15	0.737	0.809
1.16	0.740	0.811
1.17	0.742	0.814
1.18	0.745	0.817
1.19	0.747	0.819

F_{CE}/F_C^*	Sawed C_p	Glu-Lam C_p
1.20	0.750	0.822
1.22	0.755	0.826
1.24	0.760	0.831
1.26	0.764	0.836
1.28	0.769	0.840
1.30	0.773	0.844
1.32	0.777	0.848
1.34	0.781	0.852
1.36	0.785	0.855
1.38	0.789	0.859
1.40	0.793	0.862
1.42	0.796	0.865
1.44	0.800	0.868
1.46	0.803	0.871
1.48	0.807	0.874
1.50	0.810	0.877
1.52	0.813	0.879
1.54	0.816	0.882
1.56	0.819	0.884
1.58	0.822	0.887
1.60	0.825	0.889
1.62	0.827	0.891
1.64	0.830	0.893
1.66	0.832	0.895
1.68	0.835	0.897
1.70	0.837	0.899
1.72	0.840	0.901
1.74	0.842	0.903
1.76	0.844	0.904
1.78	0.846	0.906
1.80	0.849	0.908
1.82	0.851	0.909
1.84	0.853	0.911
1.86	0.855	0.912
1.88	0.857	0.914
1.90	0.858	0.915
1.92	0.860	0.916
1.94	0.862	0.918
1.96	0.864	0.919
1.98	0.868	0.920

F_{CE}/F_C^*	Sawed C_p	Glu-Lam C_p
2.00	0.867	0.921
2.02	0.869	0.922
2.04	0.870	0.924
2.06	0.872	0.925
2.08	0.874	0.926
2.10	0.875	0.927
2.12	0.876	0.928
2.14	0.878	0.929
2.16	0.879	0.930
2.18	0.881	0.931
2.20	0.882	0.932
2.22	0.883	0.932
2.24	0.885	0.933
2.26	0.886	0.934
2.28	0.887	0.935
2.30	0.888	0.936
2.32	0.889	0.937
2.34	0.891	0.937
2.36	0.892	0.938
2.38	0.893	0.939
2.40	0.894	0.940
2.45	0.897	0.941
2.50	0.899	0.943
2.55	0.901	0.944
2.60	0.904	0.946
2.65	0.906	0.947
2.70	0.908	0.949
2.75	0.910	0.950
2.80	0.912	0.951
2.85	0.914	0.952
2.90	0.916	0.953
2.95	0.917	0.954
3.00	0.919	0.955
3.05	0.920	0.956
3.10	0.922	0.957
3.15	0.923	0.958
3.20	0.925	0.959
3.25	0.926	0.960
3.30	0.927	0.961
3.35	0.929	0.961

F_{CE}/F_C^*	Sawed C_p	Glu-Lam C_p
3.40	0.930	0.962
3.45	0.931	0.963
3.50	0.932	0.963
3.55	0.933	0.964
3.60	0.934	0.965
3.65	0.936	0.965
3.70	0.937	0.966
3.75	0.938	0.966
3.80	0.938	0.967
3.85	0.939	0.968
3.90	0.940	0.968
3.95	0.941	0.969
4.00	0.942	0.969
4.05	0.943	0.969
4.10	0.944	0.970
4.15	0.944	0.970
4.20	0.945	0.971
4.25	0.946	0.971
4.30	0.947	0.972
4.35	0.947	0.972
4.40	0.948	0.972
4.45	0.949	0.973
4.50	0.949	0.973
4.55	0.950	0.974
4.60	0.950	0.974
4.65	0.951	0.974
4.70	0.952	0.975
4.75	0.952	0.975
4.80	0.953	0.975
4.85	0.953	0.975
4.90	0.954	0.976
5.00	0.955	0.976
6.00	0.963	0.981
8.00	0.973	0.986
10.0	0.979	0.989
20.0	0.990	0.995
40.0	0.995	0.997
60.0	0.997	0.998
100.0	0.998	0.999
200.0	0.999	0.999

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