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
c_1 = coefficient for shear stress for a rectangular bar in torsion
C_C = curvature factor for laminated arches
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
$f_{from\ table}$ = 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
F_y = yield strength
h = height of a rectangle
H = name for a horizontal force
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 with respect to an y-axis
J = polar moment of inertia
K = effective length factor for columns
L_e = effective length that can buckle for column design, as is ℓ_e
L = name for length or span length
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
 = interior radius of a laminated arch
R = radius of curvature of a deformed beam
 = radius of curvature of a laminated arch
 = name for a reaction force
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
$w_{self\ wt}$ = name for distributed load from self weight of member
W = shorthand for wind load
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
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).

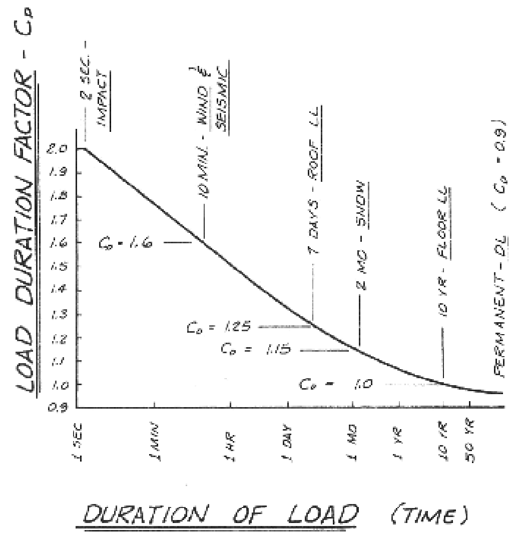
Tabulated wood strength values are used as the base allowable strength and modified by appropriate **adjustment** factors:

$$F' = C_D C_M C_F \ldots \times F_{from\,table}$$

<u>Size and Use Categories</u>

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)
- C_C curvature factor for laminated arches

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 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:

$$F'_b \geq f_b = \frac{Mc}{I}$$

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:

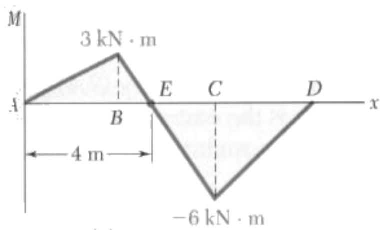
$$\theta = slope = \frac{1}{EI}\int M(x)dx$$

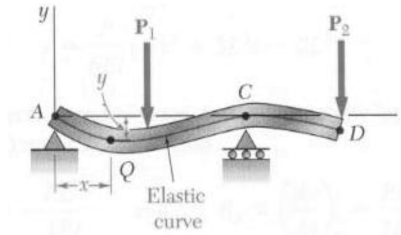
The equation for deflection, y, along a beam is:

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

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

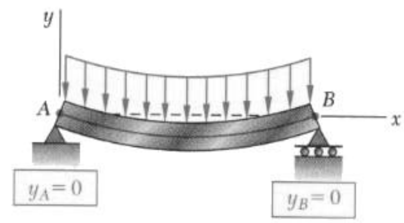
Elastic curve equations can be **superpositioned** ONLY if the stresses are in the elastic range. *The deflected shape is roughly the same shape flipped as the bending moment diagram but is constrained by supports and geometry.*



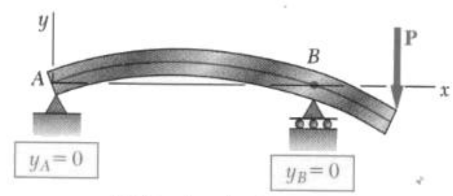


Boundary Conditions

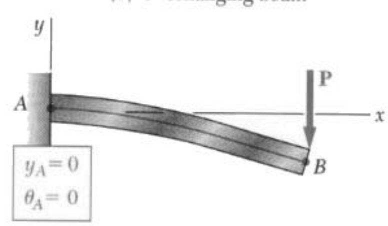
The boundary conditions are geometrical values that we know – slope or deflection – which may be restrained by supports or symmetry.



(*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' 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 $f_b = \frac{M_{max}}{S} \le F'_b$
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 $f_v \le F'_v$ *or* find $A_{req'd}$

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

7. Provide adequate bearing area at supports: $f_p = \frac{P}{A} \le F'_c \text{ or } F'_{c\perp}$
8. Evaluate shear due to torsion $f_v = \frac{T\rho}{J} \text{ or } \frac{T}{c_1 ab^2} \le F'_v$

   (circular section or rectangular)

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

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

FOR ANY EVALUATION:

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

Load Tables for Uniformly Loaded Joists & 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.

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, $L_e/d \leq 50$:

$$f_c = \frac{P}{A} \leq F_c' \qquad\qquad F_c' = F_c(C_D)(C_M)(C_t)(C_F)(C_p)$$

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_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:

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

Procedure for Analysis

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

   compute $F_{cE} = \frac{K_{cE} E}{\left(\frac{l_e}{d}\right)^2}$ 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, etc....
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 $P \le P_{allowable}$? $\Rightarrow$ yes, it is; no, it is no good

   or Is $f_{actual} \le F'_c$? $\Rightarrow$ 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 choose largest)
3. Obtain F'_c

   compute $F_{cE} = \frac{K_{cE} E}{\left(\frac{l_e}{d}\right)^2}$ 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 $P \le P_{allowable}$? $\Rightarrow$ yes, it is; *no, pick a bigger section and go back to step 2.*

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

Trusses

Timber trusses are commonly manufactured with continuous top or bottom chords, but the members are still design as compression and tension members (without the effect of bending.)

Stud Walls

Stud wall construction is often used in *light frame construction* together with joist and rafters. Studs are typically 2-in. nominal thickness and must be braced in the weak axis. Most wall coverings provide this function. Stud spacing is determined by the width of the panel material, and is usually 16 in. The lumber grade can be relatively low. The walls must be designed for a combination of wind load and bending, which means beam-column analysis.

Columns with Bending (Beam-Columns)

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

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.

Laminated Arches

The radius of curvature, *R*, is limited because of residual bending stresses between lams of thickness *t* to 100t for Southern pine and hardwoods and 250t for softwoods.

The allowable bending stress for combined stresses is $F'_b = F_b(C_F C_C)$

where $C_C = 1 - 2000\left(\frac{t}{r}\right)^2$

and r is the radius to the inside of the lamination.

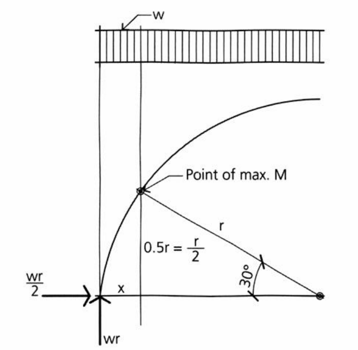


Fig. 24.6 Circular arch moment analysis

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

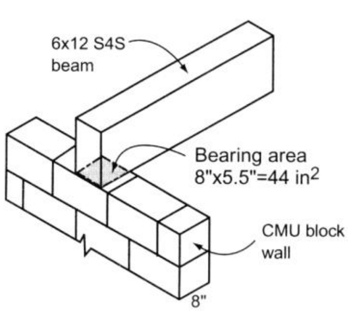
Because nails rely on shear resistance, a common problem when nailing is splitting of the wood at the end of the member, which is a shear failure. Tables list the shear force capacity per unit length of embedment per nail. Jointed members used for beams will have shear stress across the connector, and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

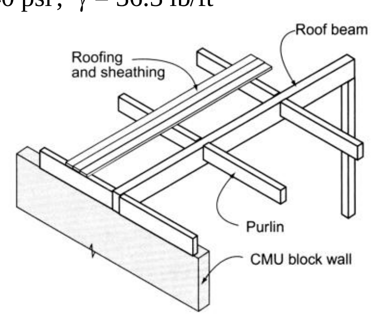
$$nF_{connector} \geq \frac{VQ_{connected\ area}}{I} \cdot p$$

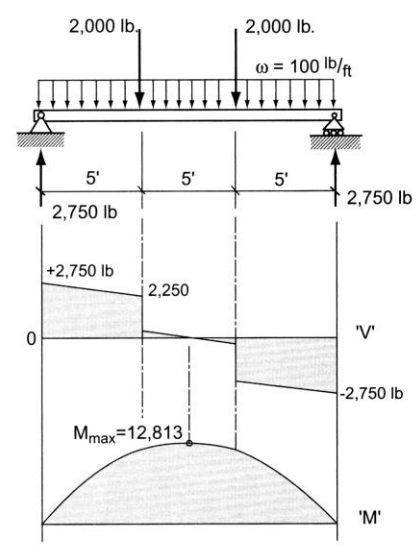
Example 1 (pg 328)

Example Problem 9.15 (Figures 9.73 to 9.75)

Design a Southern pine No. 1 beam to carry the loads shown (roof beam, no plaster). Assume the beam is supported at each end by an 8" block wall. F_b = 1550 psi; F_v = 110 psi; $E = 1.6 \times 10^6$ psi. $F_{c\perp}$ = 440 psi , γ = 36.3 lb/ft^3







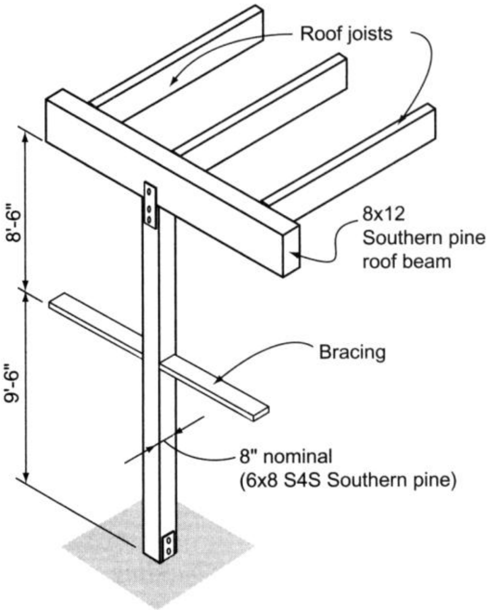
Example 1 (continued)

<u>Example 2 (pg 379)</u>

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.

$$F_c = 975 \text{ psi}, \; E = 1.6 \times 10^6 \text{ psi}$$



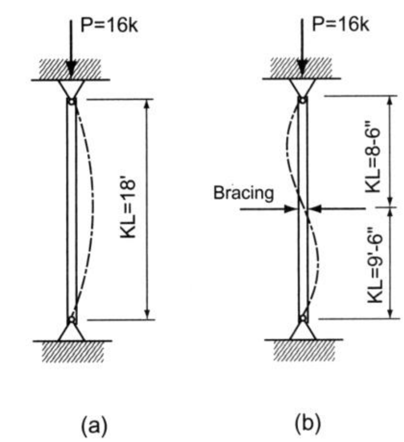


Figure 10.61 (a) Strong axis. (b) Weak axis.

Example 3 (pg 381)

Example Problem 10.20:
Design of Wood Columns(Figure 10.66)

A 22'-tall glu-lam column is required to support a roof load (including snow) of 40 kips. Assuming $8\frac{3}{4}$" in one dimension (to match the beam width above), determine the minimum column size if the top and bottom are pin supported.

F_c = 1650 psi, E = 1.8 x 10^6 psi

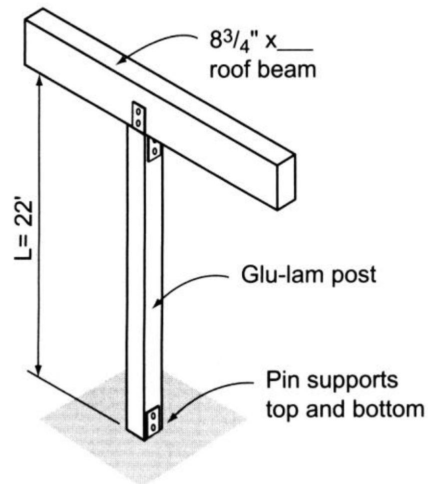
Also verify with allowable load tables

Select from the following sizes:

$8\frac{3}{4}" \times 9"$ ($A = 78.75$ in.^{2})

$8\frac{3}{4}" \times 10\frac{1}{2}"$ ($A = 91.88$ in.^{2})

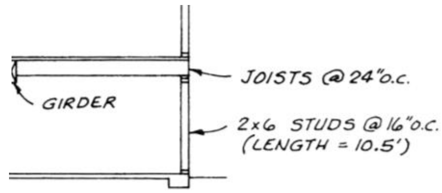
$8\frac{3}{4}" \times 12"$ ($A = 105.00$ in.^{2})



Example 4

EXAMPLE 7.16 Combined Bending and Compression in a Stud Wall

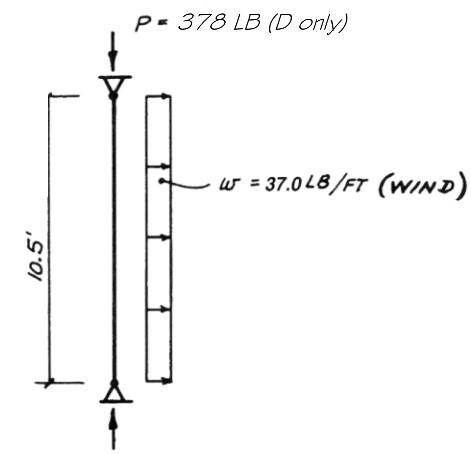
Check the 2 × 6 stud in the first-floor bearing wall in the building shown in Fig. 7.20*a*. Consider the given vertical loads and lateral forces. Lumber is No. 2 DF-L. MC ≤ 19 percent and normal temperatures apply. Allowable stresses are to be in accordance with the NDS. F'_b = 2152 psi F_c = 1350 psi



$A = 8.25\ in^2$
$S_x^* = 7.56\ in^3$

COLUMN CAPACITY:

Sheathing provides lateral support about the weak axis of the stud. Therefore, check column buckling about the x axis only (L = 10.5 ft and d_x = 5.5 in.):



$$\left(\frac{l_e}{d}\right)_y = 0 \quad \text{because of sheathing}$$

$$\left(\frac{l_e}{d}\right)_{max} = \left(\frac{l_e}{d}\right)_x = \frac{10.5 \text{ ft} \times 12 \text{ in./ft}}{5.5 \text{ in.}} = 22.9$$

$$E = 1{,}600{,}000 \text{ psi}$$

For visually graded sawn lumber:

$$K_{cE} = 0.3$$

$$c = 0.8$$

$$F_{cE} = \frac{K_{cE}E'}{(l_e/d)^2} = \frac{0.3(1{,}600{,}000)}{(22.9)^2} = 915 \text{ psi}$$

$$F_c^* = F_c(C_D) \qquad C_D = 1.6 \text{ from wind loading}$$

$$= 1350(1.6) = 2376 \text{ psi}$$

$$\frac{F_{cE}}{F_c^*} = \frac{915}{2376} = 0.385 \quad C_P = 0.35$$

$$F'_c = F_c(C_D)(C_P) = 2376(0.35) = 832 \text{ psi}$$

Load Case 2: Gravity Loads + Lateral Forces

BENDING:

Wind governs over seismic. Force to one stud:

$$\text{Wind} = 27.8 \text{ psf}$$

$$w = 27.8 \text{ psf} \times \frac{16in}{12^{in/ft}} = 37.0 \text{ lb/ft}$$

$$M = \frac{wL^2}{8} = \frac{37.0(10.5)^2}{8} = 510 \text{ ft-lb} = 6115 \text{ in.-lb}$$

$$f_b = \frac{M}{S} = \frac{6115}{7.56} = 809 \text{ psi} \qquad F'_b = 2152 \text{ psi}$$

AXIAL:

$$\text{D + W:} \quad f_c = \frac{P}{A} = \frac{378}{8.25} = 46 \text{ psi}$$

COMBINED STRESS:

The simplified interaction formula from Example 7.13 (Sec. 7.12) applies:

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{bx}}{F'_{bx}(1 - f_c/F_{cEx})} \le 1.0$$

$$F_{cEx} = F_{cE} = 915 \text{ psi}$$

D + W:

In this load combination, D produces the axial stress f_c and W results in the bending stress f_{bx}.

$$\left(\frac{f_c}{F'_c}\right)^2 + \left(\frac{1}{1 - f_c/F_{cEx}}\right)\frac{f_{bx}}{F'_{bx}} =$$

$$\left(\frac{46}{832}\right)^2 + \left(\frac{1}{1 - 46/915}\right)\frac{809}{2152} = 0.399 < 1.0$$

2 × 6 No. 2 DF-L exterior bearing wall *OK*

Example 5

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).

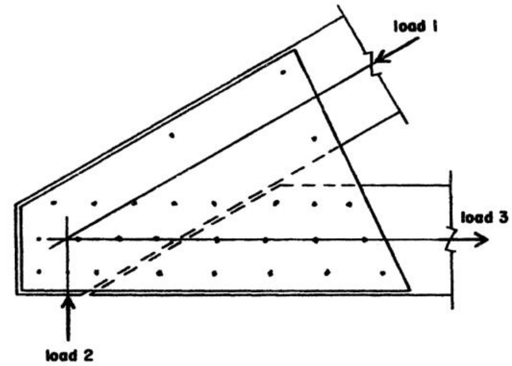


TABLE 7.1 Reference Lateral Load Values for Common Wire Nails (lb/in.)

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 6

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.

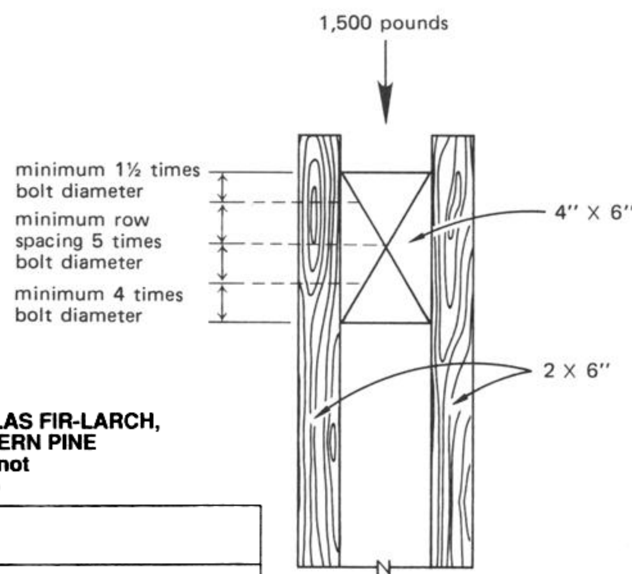


TABLE 23-I-F—HOLDING POWER OF BOLTS[1,2,3] FOR DOUGLAS FIR-LARCH, CALIFORNIA REDWOOD (CLOSE GRAIN) AND SOUTHERN PINE (See U.B.C. Standard 23-17 where members are not of equal size and for values in other species.)

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.

Example 7

EXAMPLE 12.8 Knee Brace Connection

The carport shown in Fig. 12.13a uses 2 × 6 knee braces to resist the longitudinal seismic force. Determine the number of 16d common nails required for the connection of the brace to the 4 × 4 post. Material is Southern Pine lumber that is dry at the time of construction. Normal temperatures apply.

Force to one row of braces:

$$R = \frac{wL}{2} = 76\left(\frac{22}{2}\right) = 836 \text{ lb}$$

Assume the force is shared equally by all braces.

$$\Sigma M_0 = 0$$

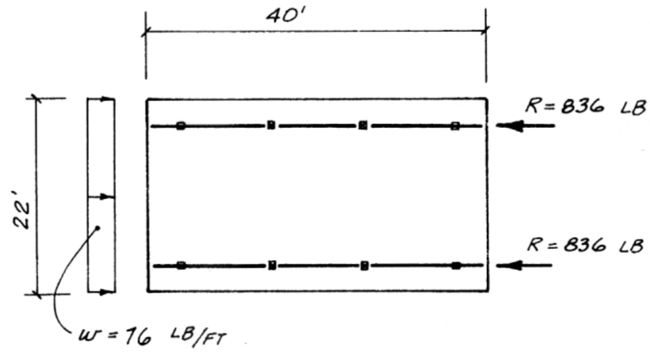
$$3H - 209(10) = 0$$

$$H = 697 \text{ lb}$$

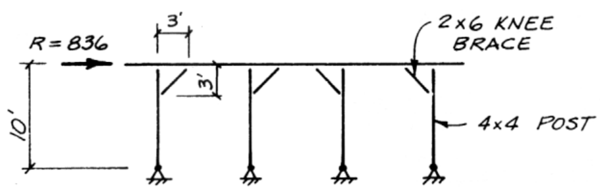
$$B = \sqrt{2}H = \sqrt{2}(697)$$

$$= 985 \text{ lb axial force in knee brace}$$

$$= \text{force on nailed connection}$$



PLAN

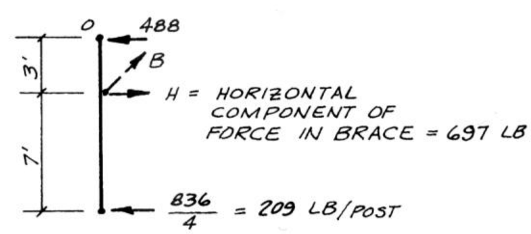


ELEVATION

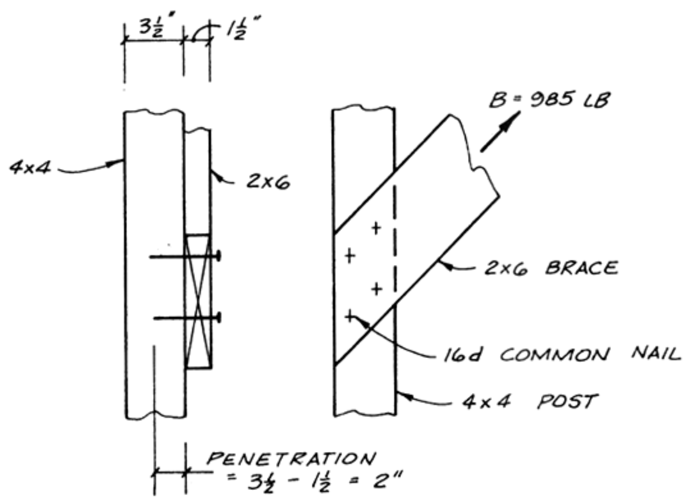
The nominal design value for a 16d common nail in Southern Pine can be evaluated using the yield equations (Sec. 12.4), or it can be obtained from NDS Table 12.3B.

Nominal design value from NDS Table 12.3B

$$Z = 154 \text{ lb/nail}$$



FBD OF COLUMN



END SIDE

ELEVATIONS

Example 7 (continued)

Adjustment Factors

Penetration

Required penetration to use the full value of Z

$$12D = 12(0.162) = 1.94 \text{ in.} < 2.0$$

$\therefore$ Penetration depth factor is

$$C_d = 1.0$$

Moisture content

Because the building is "unenclosed," the brace connection may be exposed to the weather, and the severity of this exposure must be judged by the designer. Assume that a reduction for high moisture content is deemed appropriate, and the wet service factor C_M is obtained from NDS Table 7.3.3.

$$C_M = 0.7$$

Load duration

The load duration factor recommended in the NDS for seismic forces is $C_D = 1.6$. The designer is cautioned to verify local code acceptance before using this value in practice.

Other adjustment factors

All other adjustment factors for allowable nail capacity do not apply to the given problem, and each can be set equal to unity:

$C_t = 1.0$	because normal temperature range is assumed
$C_{eg} = 1.0$	because nails are driven into side grain of holding member
$C_{di} = 1.0$	because connection is not part of nailing for diaphragm or shearwall
$C_{tn} = 1.0$	because nails are not toenailed

Allowable load for 16d common nail in Southern Pine:

$$Z' = Z(C_D C_M C_t C_d C_{eg} C_{di} C_{tn})$$

$$= 154(1.6)(0.7)(1.0)(1.0)(1.0)(1.0)(1.0) = 172 \text{ lb/nail}$$

Required number of nails: $N = \dfrac{B}{Z'} = \dfrac{985}{172} = 5.73$

Use	six 16d common nails each end of knee brace for high-moisture conditions.*

If the reduction for wet service is not required, $C_M = 1.0$. The revised connection is

$$Z' = 154(1.6)(1.0) = 246 \text{ lb/nail}$$

$$N = \frac{985}{246} = 4.00$$

Use	four 16d common nails each end of knee brace if moisture is not a concern.

ASD Beam Design Flow Chart