Wood Design

Notation:

\[ a = \text{name for width dimension} \]
\[ A = \text{name for area} \]
\[ A_{\text{req'd-adj}} = \text{area required at allowable stress when shear is adjusted to include self weight} \]
\[ b = \text{width of a rectangle} \]
\[ c = \text{largest distance from the neutral axis to the top or bottom edge of a beam} \]
\[ c_l = \text{coefficient for shear stress for a rectangular bar in torsion} \]
\[ C_C = \text{curvature factor for laminated arches} \]
\[ C_D = \text{load duration factor} \]
\[ C_{fu} = \text{size factor} \]
\[ C_H = \text{shear stress factor} \]
\[ C_i = \text{incising factor} \]
\[ C_L = \text{beam stability factor} \]
\[ C_M = \text{wet service factor} \]
\[ C_p = \text{column stability factor for wood design} \]
\[ C_r = \text{repetitive member factor for wood design} \]
\[ C_Y = \text{volume factor for glue laminated timber design} \]
\[ C_t = \text{temperature factor for wood design} \]
\[ d = \text{name for depth} \]
\[ d_{\text{min}} = \text{dimension of timber critical for buckling} \]
\[ D = \text{shorthand for dead load} \]
\[ DL = \text{shorthand for dead load} \]
\[ E = \text{modulus of elasticity} \]
\[ f = \text{stress (strength is a stress limit)} \]
\[ f_b = \text{bending stress} \]
\[ f_{\text{from table}} = \text{tabular strength (from table)} \]
\[ f_p = \text{bearing stress} \]
\[ f_y = \text{shear stress} \]
\[ f_{\text{v-max}} = \text{maximum shear stress} \]
\[ F_{\text{allow}} = \text{allowable stress} \]
\[ F_b = \text{tabular bending strength} \]
\[ F_b' = \text{allowable bending stress (adjusted)} \]
\[ F_c = \text{tabular compression strength parallel to the grain} \]
\[ F_c' = \text{allowable compressive stress (adjusted)} \]
\[ F_{c,E} = \text{theoretical allowed buckling stress} \]
\[ F_{c,L} = \text{tabular compression strength perpendicular to the grain} \]
\[ F_{\text{connector}} = \text{shear force capacity per connector} \]
\[ F_p = \text{tabular bearing strength parallel to the grain} \]
\[ F_{p,E} = \text{allowable bearing stress} \]
\[ F_t = \text{tabular tensile strength} \]
\[ F_u = \text{ultimate strength} \]
\[ F_v = \text{tabular bending strength} \]
\[ F_y = \text{yield strength} \]
\[ h = \text{height of a rectangle} \]
\[ H = \text{name for a horizontal force} \]
\[ I = \text{moment of inertia with respect to neutral axis bending} \]
\[ I_{\text{trial}} = \text{moment of inertia of trial section} \]
\[ I_{\text{req'd}} = \text{moment of inertia required at limiting deflection} \]
\[ I_y = \text{moment of inertia with respect to an y-axis} \]
\[ J = \text{polar moment of inertia} \]
\[ K = \text{effective length factor for columns} \]
\[ L_e = \text{effective length that can buckle for column design, as is } L \]
\[ L = \text{name for length or span length} \]
\[ LL = \text{shorthand for live load} \]
\[ LRFD = \text{load and resistance factor design} \]
\[ M = \text{internal bending moment} \]
\[ M_{\text{max}} = \text{maximum internal bending moment} \]
\[ M_{\text{max-adj}} = \text{maximum bending moment adjusted to include self weight} \]
\[ n = \text{number of connectors across a joint, as is } N \]
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:

\[ F' = C_D C_M C_F \cdots \times F_{from\,table} \]

Size and Use Categories

<table>
<thead>
<tr>
<th>Category</th>
<th>Thickness Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boards</td>
<td>1 to 1½ in. thick, 2 in. and wider</td>
</tr>
<tr>
<td>Dimension lumber</td>
<td>2 to 4 in. thick, 2 in. and wider</td>
</tr>
<tr>
<td>Timbers</td>
<td>5 in. and thicker, 5 in. and wider</td>
</tr>
</tbody>
</table>
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
  \[
  C_F = (12 / d)^{1/6} \leq 1.0
  \]
- $C_{fu}$: 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
- $F_{cl}$: compression stress (perpendicular to grain)
- $F_{ct}$: 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:

\[
\frac{\text{dead load}}{0.9} \quad \text{or} \quad \frac{(\text{dead load} + \text{any combination of live load})}{C_D}
\]

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:

\[ V = \sum(-w)dx \quad M = \sum(V)dx \]

\[ \frac{dV}{dx} = -w \quad \frac{dM}{dx} = V \]

**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:

The slope of the n.a. of a beam, \( \theta \), will be tangent to the radius of curvature, \( R \):

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

\[ \theta = \text{slope} = \frac{1}{EI} \int M(x)dx \]

The equation for deflection, \( y \), along a beam is:

\[ y = \frac{1}{EI} \int \theta dx = \frac{1}{EI} \int \int M(x)dx \]

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.

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 \( y_{\text{max}} \):

\[
\theta = \frac{dy}{dx} = \text{slope} = 0
\]

**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.

\[
y_{\text{max}} (x) = \Delta_{\text{actual}} \leq \Delta_{\text{allowable}} = \frac{L}{y_{\text{value}}}
\]

<table>
<thead>
<tr>
<th>Use</th>
<th>LL only</th>
<th>DL+LL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof beams:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Industrial</td>
<td>L/180</td>
<td>L/120</td>
</tr>
<tr>
<td>Commercial</td>
<td></td>
<td></td>
</tr>
<tr>
<td>plaster ceiling</td>
<td>L/240</td>
<td>L/180</td>
</tr>
<tr>
<td>no plaster</td>
<td>L/360</td>
<td>L/240</td>
</tr>
<tr>
<td>Floor beams:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ordinary Usage</td>
<td>L/360</td>
<td>L/240</td>
</tr>
<tr>
<td>Roof or floor (damageable elements)</td>
<td>L/480</td>
<td></td>
</tr>
</tbody>
</table>
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 $M_{\text{max}}$.
3. Calculate $S_{\text{req'd}}$. This step is equivalent to determining $f_b = \frac{M_{\text{max}}}{S} \leq F'_b$.
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 $S_{\text{req'd}}$.

   $w_{\text{self wt}} = \gamma A$

   

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

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

   

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

7. Provide adequate bearing area at supports: $f_p = \frac{P}{A} \leq F'_c$ or $F'_{c\perp}$

8. Evaluate shear due to torsion

   $f_v = \frac{T_P}{J} \text{ or } \frac{T}{c_1 ab^2} \leq F'_v$

   (circular section or rectangular)

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

   

   **** note: when $\Delta_{\text{calculated}} > \Delta_{\text{limit}}$, $I_{\text{required}}$ can be found with:

   and $S_{\text{req'd}}$ will be satisfied for similar self weight ****

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

   

   

   6
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 exist 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, $L/e/d \leq 50$:

$$ f_c = \frac{P}{A} \leq F_c' $$

$$ F_c' = F_c (C_D)(C_M)(C_t)(C_p) $$

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

$${C_p} = \frac{1 + (F_{cc}/F_c^*)}{2c} - \sqrt{\frac{1 + (F_{cc}/F_c^*)^2}{2c}} \frac{F_{cc}/F_c^*}{c}$$

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
- $C_p$ = size factor
- $C_p$ = column stability factor off chart or equation:
For preliminary column design:

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

Procedure for Analysis

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

\[ \text{compute } F_{cE} = \frac{K_{cE} E}{(l/d)^2} \quad \text{with } K_{cE} = 0.3 \text{ for sawn, } = 0.418 \text{ for glu-lam} \]

3. Compute \( F^*_c \approx F_c C_D \) 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 \( P_{\text{allowable}} = F'_c A \) or alternatively compute \( f_{\text{actual}} = P/A \)
7. Is the design satisfactory?
   
   Is \( P \leq P_{\text{allowable}}? \Rightarrow \text{yes, it is; no, it is no good} \)
   
   or Is \( f_{\text{actual}} \leq F'_c ? \Rightarrow \text{yes, it is; no, it is no good} \)

Procedure for Design

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

\[ \text{compute } F_{cE} = \frac{K_{cE} E}{(l/d)^2} \quad \text{with } K_{cE} = 0.3 \text{ for sawn, } = 0.418 \text{ for glu-lam} \]

4. Compute \( F^*_c \approx F_c C_D \) 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 \( P_{\text{allowable}} = F'_c A \) or alternatively compute \( f_{\text{actual}} = P/A \)
8. Is the design satisfactory?

   Is \( P \leq P_{\text{allowable}}? \Rightarrow \text{yes, it is; no, pick a bigger section and go back to step 2.} \)
   
   or Is \( f_{\text{actual}} \leq F'_c ? \Rightarrow \text{yes, it is; no, pick a bigger section and go back to step 2.} \)

Trusses

Timber trusses are commonly manufactured with continuous top or bottom chords, but the members are still designed 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 \(S (=l/c_{max})\)

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 100\(t\) for Southern pine and hardwoods and 250\(t\) 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.

**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:

\[
nF_{\text{connector}} \geq \frac{VQ_{\text{connected area}}}{I} \cdot P
\]

**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. \( F_b = 1550 \text{ psi} \); \( F_e = 110 \text{ psi} \); \( E = 1.6 \times 10^6 \text{ psi} \). \( F_{\perp} = 440 \text{ psi}, \gamma = 36.3 \text{ lb/ft}^2 \)
Example 1 (continued)
Example 2 (pg 379)

Example Problem 10.18 (Figures 10.60 and 10.61)

An 18' tall 6x8 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.

\[ F_c = 975 \, \text{psi}, \quad E = 1.6 \times 10^6 \, \text{psi} \]

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 $8^{3/4}"$ in one dimension (to match the beam width above), determine the minimum column size if the top and bottom are pin supported.

Select from the following sizes:

- $8^{3/4}" \times 9"$ ($A = 78.75 \text{ in.}^2$)
- $8^{3/4}" \times 10^{1/2}"$ ($A = 91.88 \text{ in.}^2$)
- $8^{3/4}" \times 12"$ ($A = 105.00 \text{ in.}^2$)

$F_c = 1650 \text{ psi}$, $E = 1.8 \times 10^6 \text{ psi}$

Also verify with allowable load tables.
**Example 4**

**EXAMPLE 7.16** Combined Bending and Compression in a Stud Wall

Check the $2 \times 6$ stud in the first-floor bearing wall in the building shown in Fig. 7.20a. 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

**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.):

\[
\left( \frac{L}{d} \right)_y = 0 \quad \text{because of sheathing}
\]

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

$E = 1,600,000$ psi

For visually graded sawn lumber:

\[
F_{ck} = 0.3
\]

\[
c = 0.8
\]

\[
F_{ck} = \frac{K_{ck}E'}{(I/d)^2} = \frac{0.3(1,600,000)}{(22.9)^2} = 915 \text{ psi}
\]

\[
F'_c = F_c(C_D) \quad C_D = 1.6 \text{ from wind loading}
\]

\[
= 1350(1.6) = 2376 \text{ psi}
\]

\[
F'_c = F_c(C_D) \quad C_D = 0.35
\]

\[
= 2376(0.35) = 832 \text{ psi}
\]

**Load Case 2: Gravity Loads + Lateral Forces**

**BENDING:**

Wind governs over seismic. Force to one stud:

\[
\text{Wind} = 27.8 \text{ psf}
\]

\[
\omega = 27.8 \text{ psf} \times \frac{16 \text{ in.}}{12 \text{ in./ft}} = 37.0 \text{ lb/ft}
\]

\[
M = \frac{\omega L^2}{8} = \frac{37.0(10.5)^2}{8} = 510 \text{ ft-lb} = 6115 \text{ in.-lb}
\]

**AXIAL:**

\[
f_b = \frac{M}{S} = \frac{6115}{7.56} = 809 \text{ psi} \quad F'_b = 2152 \text{ psi}
\]

\[
D + W: \quad f_e = \frac{P}{A} = \frac{378}{8.25} = 46 \text{ psi}
\]

**COMBINED STRESS:**

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

\[
\left( \frac{f_e}{F'_c} \right)^2 + \frac{f_{ax}}{F'_{cx}(1 - f_e/F_{cx})} \leq 1.0
\]

\[
F_{cx} = F_{cx} = 915 \text{ psi}
\]

\[
\frac{46}{832} + \frac{1}{1 - 46/832} \approx 0.399 < 1.0
\]

\[
2 \times 6 \text{ No. 2 DF-L exterior bearing wall OK}
\]
Example 5

Example 5. 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.)

<table>
<thead>
<tr>
<th>Side Member Thickness, ( t ) (in.)</th>
<th>Nail Length, ( L ) (in.)</th>
<th>Nail Diameter, ( D ) (in.)</th>
<th>Nail Pennyweight</th>
<th>Load per Nail, ( Z ) (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 2 )</td>
<td>0.113</td>
<td>6d</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>( \frac{3}{16} )</td>
<td>2.12</td>
<td>6d</td>
<td>63</td>
<td></td>
</tr>
<tr>
<td>( \frac{3}{8} )</td>
<td>0.148</td>
<td>10d</td>
<td>76</td>
<td></td>
</tr>
<tr>
<td>( \frac{1}{2} )</td>
<td>0.131</td>
<td>8d</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>( \frac{3}{4} )</td>
<td>0.148</td>
<td>10d</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>( 1 )</td>
<td>0.162</td>
<td>16d</td>
<td>92</td>
<td></td>
</tr>
</tbody>
</table>

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 inch bolts will be required to safely carry a load of 1500 lb? Use the chart provided.

Table 234-11—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.

<table>
<thead>
<tr>
<th>LENGTH OF BOLT IN MAIN WOOD MEMBER(^4) (inches)</th>
<th>DIAMETER OF BOLT (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \frac{3}{8} )</td>
</tr>
<tr>
<td></td>
<td>( \times 25.4 ) mm</td>
</tr>
<tr>
<td>2( \frac{1}{2} ) Single ( p )</td>
<td>630</td>
</tr>
<tr>
<td>2( \frac{1}{2} ) Double ( p )</td>
<td>710</td>
</tr>
<tr>
<td>2( \frac{1}{2} ) Single ( q )</td>
<td>620</td>
</tr>
<tr>
<td>2( \frac{1}{2} ) Double ( q )</td>
<td>710</td>
</tr>
</tbody>
</table>

\(^1\) 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 x 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 x 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.

\[ \Sigma M_0 = 0 \]

\[ 3H - 209(10) = 0 \]

\[ H = 697 \text{ lb} \]

\[ B = \sqrt{2H} = \sqrt{2(697)} \]

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

\[ = \text{force on nailed connection} \]

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

\[ Z = 154 \text{ lb/nail} \]

**END SIDE ELEVATIONS**
Example 7 (continued)

**Adjustment Factors**

**Penetration**

Required penetration to use the full value of $Z$

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

:. 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 $C_p = 1.6$. 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:

$$C_t = 1.0 \quad \text{because normal temperature range is assumed}$$

$$C_{es} = 1.0 \quad \text{because nails are driven into side grain of holding member}$$

$$C_{da} = 1.0 \quad \text{because connection is not part of nailing for diaphragm or shearwall}$$

$$C_{tn} = 1.0 \quad \text{because nails are not toenailed}$$

Allowable load for 16d common nail in Southern Pine:

$$Z' = Z(C_t C_{sh} C_d C_{es} C_{da} C_{tn})$$

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

Required number of nails:

$$N = \frac{B}{Z'} = \frac{985}{172} = 5.73$$

*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

Collect data: L, \( \varepsilon \), \( \gamma \), \( \Delta_{\text{lim}} \); find beam charts for load cases and \( \Delta_{\text{actual}} \) equations.

Collect data: \( F_b \) & \( F_v \), and all adjustment factors (C\( D \), etc.).

Find \( V_{\text{max}} \) & \( M_{\text{max}} \) from constructing diagrams or using beam chart formulas.

Find \( S_{\text{req'd}} \) and pick a section from a table with \( S_x \) greater or equal to \( S_{\text{req'd}} \).

Calculate \( \varepsilon_{\text{self-wt}} \) using \( A \) found and \( \gamma \). Find \( M_{\text{max-adj}} \) & \( V_{\text{max-adj}} \).

Calculate \( S_{\text{req'd-adj}} \) using \( M_{\text{max-adj}} \).

Is \( S_{\text{req'd}} \geq S_{\text{req'd-adj}} \)?

(OR calculate \( f_b \). Is \( f_b \leq F'_b \)?)

Yes

Calculate \( A_{\text{req'd-adj}} \) using \( V_{\text{max-adj}} \).

Is \( A_{\text{req'd}} \geq A_{\text{req'd-adj}} \)?

(OR calculate \( f_v \). Is \( f_v \leq F'_v \)?)

No

pick a new section with a larger area

Calculate \( \Delta_{\text{max}} \) using superpositioning and beam chart equations with the \( I_x \) for the section.

\( I_{\text{req'd}} \geq \frac{\Delta_{\text{no big}}}{\Delta_{\text{lim}}} I_{\text{trial}} \)

Is \( \Delta_{\text{max}} \leq \Delta_{\text{lim}} \)?

This may be both the limit for live load deflection and total load deflection.

No

pick a section with a larger \( I_x \)

Yes (DONE)