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
\( c \) = 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
\( C_D \) = 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
\( F_{b'} \) = allowable bending stress (adjusted)
\( F_c \) = tabular compression strength parallel to the grain
\( F_{c'} \) = allowable compressive stress (adjusted)
\( F_{c-E} \) = theoretical allowed buckling stress
\( F_{c-L} \) = 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
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:

\[ f = C_D C_M C_F \times f_{\text{from table}} \]

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

\[ C_F = \left( \frac{12}{d} \right)^{0.6} \leq 1.0 \]

(see Table 5.2 in text)
Cfu: flat use factor (excluding decking)
Ci: incising factor (from increasing the depth of pressure treatment)
Ct: temperature factor (at high temperatures strength decreases)
Cr: repetitive member factor
CH: shear stress factor (amount of splitting)
CV: volume factor for glued laminated timber (similar to CF)
CL: beam stability factor (for beams without full lateral support)

Tabular Design Values

Fb: bending stress
Ft: tensile stress
Fv: horizontal shear stress
Fc⊥: compression stress (perpendicular to grain)
Fc∥: compression stress (parallel to grain)
E: modulus of elasticity
Fp: 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:

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

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:

\[ F'_b \text{ or } \phi F'_u \geq f_b = \frac{Mc}{I} \]

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:

\[ V = \Sigma(-w)dx \]
\[ M = \Sigma(V)dx \]
\[ \frac{dV}{dx} = -w \]
\[ \frac{dM}{dx} = V \]

Determining Maximum Bending Stress

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

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

Deflections

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

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

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

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

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

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.

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

<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 lightweight 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 $f_{v-max} = \frac{3V}{2A} = 1.5 \frac{V}{A}$ \[ \therefore A_{req'd} \leq \frac{3V}{2F_v'} \]

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 $f_v = \frac{T_p}{J} \text{ or } \frac{T}{c_iab^2} \leq F_v'$
   (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: $I_{req'd} \geq \frac{\Delta_{too big}}{\Delta_{limit}} I_{trial}$

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 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.

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

\[ w_{adjusted} = w_{live} \left( \frac{L / 400}{L / 360} \right)^{\text{wanted table limit}} \]
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, \( \frac{L_e}{d} \leq 50 \):

\[
f_c = \frac{P}{A} \leq F'_c
\]

\[
F_c' = F_c (C_D)(C_M)(C_i)(C_F)(C_p)
\]

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
- \( c = 0.8 \) 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)
- \( C_i \) = 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 \( L_e/d_{\text{min}} \) (KL/d for each axis and chose largest)
2. Obtain \( F'_c \)
   \[
   F'_c = \frac{K_{cE} E}{(l/\ell)^2} \text{ or } \frac{0.822E'_{\text{min}}}{(l/\ell)^2} \quad \text{with } K_{cE} = 0.3 \text{ for sawn, } 0.418 \text{ for glu-lam}
   \]
3. Compute \( F^*_c \equiv F_c C_D \) 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 \( P_{\text{allowable}} = F'_c A \text{ or alternatively compute } f_{\text{actual}} = P/A \)
7. Is the design satisfactory?
   - Is \( P \leq P_{\text{allowable}}? \Rightarrow \text{yes, it is; no, it is no good} \)
   - \( \text{or Is } f_{\text{actual}} \leq F'_c ? \Rightarrow \text{yes, it is; no, it is no good} \)

Procedure for Design

1. Guess a size by picking a section
2. Calculate \( L_e/d_{\text{min}} \) (KL/d for each axis and chose largest)
3. Obtain \( F'_c \)
   \[
   F'_c = \frac{K_{cE} E}{(l/\ell)^2} \text{ or } \frac{0.822E'_{\text{min}}}{(l/\ell)^2} \quad \text{with } K_{cE} = 0.3 \text{ for sawn, } 0.418 \text{ for glu-lam}
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6. Calculate \( F'_c = F^*_c C_p \)
7. Compute \( P_{\text{allowable}} = F'_c A \text{ or alternatively compute } f_{\text{actual}} = P/A \)
8. Is the design satisfactory?
   - Is \( P \leq P_{\text{allowable}}? \Rightarrow \text{yes, it is; no, pick a bigger section and go back to step 2.} \)
   - \( \text{or Is } f_{\text{actual}} \leq F'_c ? \Rightarrow \text{yes, it is; no, pick a bigger section and go back to step 2.} \)
Columns with Bending (Beam-Columns)

The modification factors are included in the form:

\[ \left[ \frac{f_c}{F'_c} \right]^2 + \frac{f_{bx}}{F'_b \left[ 1 - \frac{f_c}{F'_{cEx}} \right]} \leq 1.0 \]

where:

\[ 1 - \frac{f_c}{F'_{cEx}} \] = magnification factor accounting for P-\( \Delta \)

\[ F'_{bx} \] = 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.

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 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 $6 \times 10$ 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 $3 \times 12$ beam of Douglas fir-larch, No. 1 grade, bears on a $3 \times 14$ 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 $8 \times 12$ 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>
<thead>
<tr>
<th>TABLE 5.10 Maximum Spans for Floor Joists (ft-in.)^p</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing (in.)</td>
</tr>
<tr>
<td>-----------------</td>
</tr>
<tr>
<td>Live load = 40 psf, Dead load = 10 psf, Maximum live-load deflection = $L/360$</td>
</tr>
<tr>
<td>12</td>
</tr>
<tr>
<td>16</td>
</tr>
<tr>
<td>19.2</td>
</tr>
<tr>
<td>24</td>
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</tr>
<tr>
<td>24</td>
</tr>
</tbody>
</table>

Source: Compiled from data in the International Building Code (Ref. 4), with permission of the publisher, International Code Council.

^p Joists are Douglas fir–larch, No. 2 grade. Assumed maximum available length of single piece is 20 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; $E = 1.6 \times 10^6$ psi; $\gamma = 36.3$ 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:

\[ F'_b = C_d F_b = (1.25)(1500) = 1875 \text{ psi} \]
\[ F'_v = C_d F_v = (1.25)(110) = 137.5 \text{ psi} \]
\[ F'_{c,\perp} = C_d F_{c,\perp} = (1.25)(440) = 550 \text{ psi} \]

Bending:

\[ S_{req'd} = \frac{M}{F'_b} = \frac{12,813 \omega - p (12 w/\beta)}{1875 \omega} = 82.0 \text{ in}^3 \]
Shear:

\[ A_{req,d} \geq \frac{3W}{2F_u^*} = \frac{3 \times 2.750lb}{2 \times 137.5 \text{ psi}} = 30.0 \text{ in}^2 \]

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.)

\[ (A = 38.13 \text{ in}^2, S = 96.90 \text{ in}^3, L_e = 738.9 \text{ in}^4) \]

\[ w_{self} \text{ wt} = \frac{36.3 \text{ lb/ft} \times (38.13 \text{ in}^2)}{12 \times \frac{96.90}{\text{in}^3}} = 9.61 \text{ lb/ft} \]

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:

\[ M_{adjusted} = 12,813 \frac{\text{lb} \cdot \text{ft}}{15 \text{ ft}^2} + \frac{9.61 \text{ lb/ft} \times (15 \text{ ft})^2}{8} = 13,083.3 \frac{\text{lb} \cdot \text{ft}}{1875 \text{ psi}} \]

\[ S_{req,d} \geq \frac{13,083.3 \frac{\text{lb} \cdot \text{ft}}{1875 \text{ psi}} \text{ft} \cdot \text{lb}}{(12 \frac{\text{lb}}{\text{ft}})^3} = 83.7 \text{ in}^3 \]

The same holds true for the contribution to the shear:

\[ V_{adjusted} = 2.750 \text{ lb} + \frac{9.61 \text{ lb/ft} \times (15 \text{ ft})}{2} = 2822.1 \text{ lb} \]

\[ A_{req,d} = \frac{3(2822.1 \text{ lb})}{2 \times 137.5 \text{ psi}} = 30.79 \text{ in}^2 \]

Check that the section chosen satisfies the new required section modulus and area:

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

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

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.

\[ \Delta_{LL-limit} = \frac{15 \text{ ft})(12 \frac{\text{lb}}{\text{ft}})}{360} = 0.5 \text{ in} \]

\[ \Delta_{total-limit} = \frac{15 \text{ ft})(12 \frac{\text{lb}}{\text{ft}})}{240} = 0.75 \text{ in} \]

Superpositioning (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): \((a \text{ is the distance from the supports to the loads})\)

\[ \Delta_{\text{max (at center)}} = \frac{Pa}{24EI} \left( 3a^2 - a^2 \right) = \frac{2000 \text{ lb} \times (5 \text{ ft})}{24(1.6 \times 10^6 \text{ psi})(738.9 \text{ in}^4)} \left( 3 \times 15 \text{ ft}^2 - 4a \text{ ft}^2 \right) (12 \frac{\text{lb}}{\text{ft}})^3 = 0.35 \text{ in} \]

distributed load (dead load)

\[ \Delta_{\text{max (at center)}} = \frac{5wl^4}{384EI} = \frac{5(100 + 9.61 \frac{\text{lb}}{\text{ft}} \times (15 \text{ ft})^3)(12 \frac{\text{lb}}{\text{ft}})^3}{384(1.6 \times 10^6 \text{ psi})(738.9 \text{ in}^4)} = 0.1 \text{ in} \]

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

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

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:

\[ f_{p} = \frac{P}{A} = \frac{2822.1 \text{ lb}}{2.5 \text{ in} \times 16 \text{ in}} = 141.1 \text{ psi} \leq F_{p} = 550 \text{ psi} \]

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.

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

\[ P_{16} = 16 \text{ kips} \]

\[ K_{L} = 9'6" \]

\[ K_{L} = 18' \]

\[ (a) \quad \text{Strong axis.} \quad (b) \quad \text{Weak axis.} \]
Example 8 (fully worked)

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.

Solution:

6x8 S4S Southern pine post: \( A = 41.25 \text{ in.}^2, F_c = 975 \text{ psi, } E = 1.6 \times 10^6 \text{ psi} \)

Check the slenderness ratio about the weak axis:

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

The slenderness ratio about the strong axis is:

\[
\frac{L_s}{d} = \frac{(18' \times 12 \text{ in.}/\text{ft.})}{7.5''} = 28.8 \text{ governs}
\]

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

\[
F_c^* = F_cC_D = (975 \text{ lb.}/\text{in.}^2)(1.25) = 1220 \text{ psi}
\]

where: \( C_D = 1.25 \text{ for 7-day-duration load} \)

\[
\frac{F_{cE}}{F_c^*} = \frac{579 \text{ psi}}{1220 \text{ psi}} = 0.475
\]

From Appendix Table 14: \( C_p = 0.412 \)

\[.\] \( F_c' = F_c^*C_p = 1220 \text{ lb.}/\text{in.}^2 \times 0.412 = 503 \text{ psi} \)

\[
P_a = F_c' \times A = (503 \text{ lb.}/\text{in.}^2) \times (41.25 \text{ in.}^2)
\]

\[
= 20,700 \text{ lb.}
\]

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

The column is adequate.
Example 9 (pg 251)

Example 4. An exterior wall stud of Douglas fir-larch, stud grade, is loaded as shown in Figure 6.5a. 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 #6 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).

---

**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 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 Pennweight</th>
<th>Load per Nail Z (lb)</th>
</tr>
</thead>
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<tr>
<td>Part 1 - With Wood Structural Panel Side Members* (G = 0.42)</td>
<td>2</td>
<td>0.113</td>
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<td>48</td>
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<tr>
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<td>2 1/2</td>
<td>0.131</td>
<td>8d</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.148</td>
<td>10d</td>
<td>78</td>
</tr>
<tr>
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<td>3</td>
<td>0.148</td>
<td>10d</td>
<td>78</td>
</tr>
</tbody>
</table>

Minimum 1 1/2 times bolt diameter
Minimum row spacing 5 times bolt diameter
Minimum 4 times bolt diameter

---

**TABLE 23-4-F—HOLDING POWER OF BOLTS1,2,3 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.)**

\[ p = \text{safe loads parallel to grain, in pounds.} \]
\[ q = \text{safe loads perpendicular to grain, in pounds.} \]

\[ \times 4.45 \text{ for } N \]

\[ \times 25.4 \text{ for } \text{mm} \]

<table>
<thead>
<tr>
<th>LENGTH OF BOLT IN MAIN WOOD MEMBER* (INCHES)</th>
<th>DIAMETER OF BOLT (INCHES)</th>
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<th>5/8</th>
<th>3/4</th>
<th>7/8</th>
<th>1</th>
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<tr>
<td>2 1/2</td>
<td>Single p</td>
<td>630</td>
<td>910</td>
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<td>495</td>
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<td></td>
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<td></td>
<td>Double p</td>
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1Tabulated 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.

2Double 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.

3See Division III for wood-to-metal bolted joints.

4The 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.

---

16
ASD Beam Design Flow Chart

Collect data: L, εo, ξ, Δlimits; find beam charts for load cases and Δactual equations

Collect data: F_b & F_v

Find V_max & M_max from constructing diagrams or using beam chart formulas

Find S_{req} and pick a section from a table with S_x greater or equal to S_{req}

Calculate ω_{self wt.} using A found and ϑ. Find M_{max-adj} & V_{max-adj}.

Calculate S_{req(adj)} using M_{max-adj}
Is S_{(picked)} ≥ S_{req(adj)}?
(OR calculate f_{b} Is f_{b} ≤ F_{b}?)

Yes

Calculate A_{req(adj)} using V_{max-adj}
Is A_{(picked)} ≥ A_{req(adj)}?
(OR calculate f_{v} Is f_{v} ≤ F_{v}?)

No

pick a new section with a larger area

Calculate Δ_{max} using superpositioning and beam chart equations with the I_x for the section

I_{req} ≥ \frac{Δ_{no big}}{Δ_{limit}} I_{trial}

Is Δ_{max} ≤ Δ_{limits}?
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)
### SECTION PROPERTIES / STANDARD SIZES

To the extent that other considerations will permit, the finished sizes of structural glued laminated timber as given in Table 8 constitute normal industry practice. Industry standards do, however, permit the use of any depth or width of glued laminated timber. Dimension lumber of 1/8 in. net thickness is normally used for laminating straight members.

The modified section modulus includes size factor ($C_p$), and no further reduction of bending stress for size is needed.

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<th>DEPTH, d in.</th>
<th>AREA, A in.²</th>
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**Note Set 13.2 S2014abn**

18
Table 14  Column Stability Factor \( C_f \)

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