Case Study in Timber
adapted from Simplified Design of Wood Structures, James Ambrose, 5th ed.

Building description

The building is a one-story building intended for commercial occupancy. Figure 16.1 presents a building plan, partial elevation, section and elevation of the perimeter shear walls. Light wood framing (assuming the fire resistance requirements have been met) will be used.

Loads

*Live Loads:*
  - Roof: 20 lb/ft² (0.96 kPa)

*Wind:* critical at 20 lb/ft² (0.96 kPa) on vertical exterior surfaces.

*Dead Loads:*
  - Roofing & deck: 7.5 lb/ft² (0.36 kPa)
  - Ceiling joists, ceiling & fixtures:
    - 6.5 lb/ft² (0.31 kPa)
  - Total: 14 lb/ft² (0.67 kPa)

Materials

Wood framing of Douglas fir-larch, structural grades No. 1 & 2 having a density of 32 lb/ft³, and AITC glulam timber.

Structural Elements/Plan

If the interior partition walls are arranged as in Figure 16.3a, there are options on the arrangement of the roof structure. We will analyze case 16.3b consisting of roof deck and rafters, stud walls, continuous (two span) beams, and columns.

Figure 16.1 Building One, general form.

Figure 16.3 Developed plan for interior partitioning and alternatives for the roof framing.
Decking & Rafters:

The standard size of plywood or structural deck panel is 4 ft x 8 ft. The typical orientation is with the long direction with the face grain perpendicular to the rafters or floor joists. (See cross hatching in Figure 16.3.) Typical joist and rafter spacings are 12 in., 16 in., and 24 in. on center. If we use 16 in. on center, the total distributed roof loads (with allowable stress design) with an assumed self weight of 4 lb/ft is:

$$w = (20 \text{ lb/ft}^2 + 14 \text{ lb/ft}^2) \cdot \frac{16}{12} \text{ in/ft} + 4 \text{ lb/ft} = 49.3 \text{ lb/ft}$$

$$M_{\text{max}} = \frac{wL^2}{8} = \frac{49.3}{8} \left( \frac{21}{12} \right)^2 = 2718 \text{ ftlb}$$

Tabular allowable stresses for No. 2 Douglas fir-larch, 2”-4” thick and 2” to 4” wide are:

- $F_{b\text{-single}} = 875 \text{ psi}$
- $F_v = 95 \text{ psi}$
- $F_{c\perp} = 625 \text{ psi}$
- $F_c = 1300 \text{ psi}$
- $E = 1,600,000 \text{ psi}$

The load duration for roof loads, $C_D = 1.25$. The repetitive member factor, $C_r = 1.15$, applies and the adjusted allowed stress for a fully braced 2x is:

$$F_b' = C_D C_r F_b = (1.25)(1.15)(875 \text{ psi}) = 1258 \text{ psi}$$

The required section modulus is

$$S_{\text{req'd}} \geq \frac{M}{F_b'} = \frac{2718 \text{ ftlb}}{1258 \text{ psi}} = 25.9 \text{ in}^3$$

A 2x12 will work if the deflection is limited to allowable for the building code. (This tends to govern for floors. Shear stress should also be checked).
Continuous Beams:

The distributed load, including an estimated self weight of 11 lb/ft (about a 6 in x 12 in section) of a glulam beam can be found from:

rafter distributed load:
\[
\gamma \cdot A \cdot \text{trib. width} = \frac{(32/\mu)(16.88 n^2)(21/2 + 8/2)}{16in} \cdot \frac{(1ft/12in)^2}{12in/ft} = 40.8 \text{ lb/ft}
\]

roof load:
\[
(20 \text{ lb/ft}^2 + 14 \text{ lb/ft}^2)(21ft/2 + 8ft/2) = 493 \text{ lb/ft}
\]

total distributed load:
\[
w = 40.8 \text{ lb/ft} + 493 \text{ lb/ft} + 11 \text{ lb/ft} = 545 \text{ lb/ft}
\]

The maximum positive moment is \(0.07wL^2\) and the maximum negative moment over the support is \(0.125wL^2\), where \(L\) is the length of one span. \(V_{\text{max}} = 0.625wL\). (These values come from a beam diagram.)

\[
M_{\text{max}} = 0.125(545 \text{ lb/ft})(16.67 \text{ ft})^2 = 18,931 \text{ lb-ft}
\]

\[
V_{\text{max}} = 0.625(545 \text{ lb/ft})(16.67 \text{ ft}) = 5678 \text{ lb}
\]

\[
F'_b = C_D F_b = (1.25)(2400 \text{ psi}) = 3000 \text{ psi}
\]

\[
S_{\text{req'd}} = \frac{M}{F'_b} = \frac{18931 \text{ lb-ft}}{3000 \text{ psi}} \cdot 12 \text{ in} = 75.7 \text{ in}^3
\]

From SECTION PROPERTIES/STANDARD SIZES, the \(5 \frac{3}{8}'' \times 10.5''\) is adequate, although a \(3 \frac{3}{8}'' \times 13.5''\) could be evaluated.
# TABLE DF-25

**DOUGLAS FIR - LARCH**

The American Institute of Timber Construction

## Structural Glued Laminated Timber

### ROOF BEAMS

#### CONSTRUCTION LOAD

<table>
<thead>
<tr>
<th>Simple Span Beams</th>
<th>For Preliminary Design Purposes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lamination thickness: 1,500 in.</td>
<td></td>
</tr>
</tbody>
</table>

#### BEAM CAPACITY, UNIFORM LOAD w, psf

<table>
<thead>
<tr>
<th>Beam Size</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width b, in.</td>
<td>Depth d, in.</td>
</tr>
<tr>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>3 1/8</td>
<td>3 3/4</td>
</tr>
<tr>
<td>6</td>
<td>7 1/8</td>
</tr>
<tr>
<td>8</td>
<td>8 1/8</td>
</tr>
<tr>
<td>10</td>
<td>10 1/8</td>
</tr>
<tr>
<td>12</td>
<td>12 1/8</td>
</tr>
<tr>
<td>14</td>
<td>14 1/8</td>
</tr>
<tr>
<td>16</td>
<td>16 1/8</td>
</tr>
<tr>
<td>18</td>
<td>18 1/8</td>
</tr>
<tr>
<td>20</td>
<td>20 1/8</td>
</tr>
<tr>
<td>22</td>
<td>22 1/8</td>
</tr>
<tr>
<td>24</td>
<td>24 1/8</td>
</tr>
</tbody>
</table>

#### NOTES ON TABLE DF-25

- This table applies to straight, simply supported glued laminated timber beams under dry conditions of use. Beams must be laterally supported at the top and at the top and bottom at the ends. The load carrying capacities tabulated are for total load including the weight of the member.
- BEAM WEIGHT: 35 lb per cubic foot was used to determine beam weight per linear foot shown in the table.
- DESIGN VALUE MODIFICATIONS: The allowable stress in bending, Fb, has been adjusted by the AITC volume factor, CV.
- For determination of load carrying capacities governed by shear, loads within a distance *d* (the depth of the beam) from the ends have been neglected.
- DEFLECTION LIMITS: For roof beams, deflection is limited to span/180 for total load.
- CONTROLLING VALUES: Values marked with a D are controlled by deflection, Fb is bending controlled, and S are shear controlled.
- SPAN: Span is defined as the length from centerline to centerline of bearing. This span is the length used in standard engineering equations to calculate deflection, bending and shear.
- The values have been limited to reasonable capacities. Engineering calculations may allow for greater capacities.

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**Feb-2001**

While these capacity tables have been prepared in accordance with recognized engineering principles and are based on the most accurate and reliable technical data available, these tables should not be used or relied upon for any general or specific application without competent professional examination and verification of their accuracy, suitability, and applicability by a licensed professional engineer, designer, or architect.

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The self weight should be determined to compare to the assumption. Table DF-25 indicates the self weight is 13 lb/ft, and that size at our span is controlled by deflection (I for \( \Delta = L/180 \)), but this chart is for simply supported beams and \( \Delta_{\text{max}} = \frac{5wL^4}{384EI} \).

The maximum deflection for a two span beam can be found with \( \Delta_{\text{max}} = \frac{wL^4}{185EI} \), which is only 0.415x the deflection of a simply supported span.

For sawn lumber, a 6x14 would be required from the comparison chart.

Evaluate shear strength:

\[
F'_v = C_D F_v = (1.25)240 \text{ psi} = 300 \text{ psi}
\]

\[
f_v = \frac{3V}{2A} = \frac{3(5678 \text{ lb})}{2(53.8 \text{ in}^2)} = 158 \text{ psi}
\]

which is less than the allowable of 300 psi (OK).

**Stud Walls & Columns:**

Building codes dictate the maximum height for slenderness (10 ft typical), and the spacing of wall studs depending on what they support (roof, roof and one floor, roof and two floors). Structural design focuses on shear wall behavior.

The interior column load is:

\[
P = 1.25wL = 1.25(545 \text{ lb/ft} + 2 \text{ lb/ft of extra beam self weight})(16.67 \text{ ft}) = 11.4 \text{ kips}
\]

For a 10 ft braced column height, choose a 6 x 6.
Wind Design:

Diaphragms are categorized as flexible or rigid and must resist lateral forces in both transverse and longitudinal directions. A diaphragm is made up of a shear-resisting element (sheathing) and boundary members called chords and collectors (struts or drag struts). The chords are designed to carry the moment in the diaphragm. The collectors are designed to transmit the horizontal reactions to the shear walls. The structural behavior is often compared to that of a steel I section on its side (Figure 15.6).

Tables in building codes for combinations of plywood grade, common nail size, plywood thickness, how the panels are arrayed and if blocking is used provide allowable shear in pounds per foot.

Consideration of lateral wind loads will be presented, but uplift on the roof must be accounted for with anchorage if the live load exceeds the downward gravity loads.

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**Selected Tables from the Uniform Building Code, 1997 Edition**

**Table 23-II-H—Allowable Shear in Pounds Per Foot for Horizontal Wood Structural Panel Diaphragms with Framing of Douglas Fir-Larch or Southern Pine**

<table>
<thead>
<tr>
<th>PANEL GRADE</th>
<th>COMMON NAIL SIZE</th>
<th>MINIMUM NAIL PENETRATION IN FRAMING (inches)</th>
<th>MINIMUM DIAPHRAGM WIDTH OF FRAMING MEMBER (inches)</th>
<th>NAILS SPACED 6 in (152 mm) max. at supported edges</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d Structural</td>
<td>11/16 2 11/16 3</td>
<td>1.85 2.10 3.50 4.20 3.75 4.75 4.20 4.75</td>
<td>1.65 1.85 3.00 2.40 2.00 2.60 2.40 2.60</td>
<td>1.25 1.40</td>
</tr>
<tr>
<td>8d Structural</td>
<td>11/16 2 11/16 3</td>
<td>2.70 3.00 3.50 4.00 3.00 4.00 3.00 4.00</td>
<td>2.40 2.60 3.60 2.40 2.60 3.60 2.40 2.60</td>
<td>1.90 2.00</td>
</tr>
<tr>
<td>10d Structural</td>
<td>11/16 2 11/16 3</td>
<td>3.20 3.60 4.80 6.40 7.20 8.30 7.20 8.30</td>
<td>2.85 3.20 4.80 2.85 3.20 4.80 2.85 3.20</td>
<td>2.15 2.40</td>
</tr>
</tbody>
</table>

1These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading. Space nails 12 inches (305 mm) on center along intermediate framing members.

2These values for nails in framing members of other species set forth in Division III, Part III, shall be calculated for all other species by multiplying the shear capacities for nails in Structural 1 by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49; 0.65 for species with a specific gravity less than 0.42.

3Framing at adjacent panel edges shall be 3-inch (76 mm) nominal or wider and nails shall be staggered where nails are spaced 2 inches (51 mm) or 2½ inches (64 mm) on center.

4Framing at adjacent panel edges shall be 3-inch (76 mm) nominal or wider and nails shall be staggered where nails having penetration into framing of more than 1 inch (25 mm) are spaced 3 inches (76 mm) or less on center.
North-South

The tributary height for the wall and parapet is $17.5 \div 2 + 2.5 = 11.25$ ft

The distributed lateral wind load = $(20 \ lb/ft^2)(11.25\text{ft}) = 225\ lb/ft$

The total lateral wind load = $(225\ lb/ft)(100\ ft) = 22,500\ lb$

The end reactions to the lateral load = $22,500\ lb / 2 = 11,250\ lb$

The unit shear (or distributed shear) in the diaphragm = $11,250\ lb / (50\ ft) = 225\ lb/ft$;

so a roof deck can be chosen that has an allowable shear > 225 lb/ft.

Knowing that $\frac{1}{2}$ in. decking is the minimum for a membrane-type roof, we use table 23-II-H to select $\frac{3}{8}$ in. sheathing with 2 x framing and 8d nails at 6 in. at all panel edges and a blocked diaphragm having an allowable shear in pounds per foot of 270 lb/ft.

The moment of the “deep beam” is used to determine the force in the top and bottom chords as show in Figure 16.6 which is 5.62 kips.

The unit shear in the two shear walls of 21 ft each = $11,250\ lb / (2 \cdot 21\ ft) = 268\ lb/ft$;

so a stud wall can be chosen that has an allowable shear > 268 lb/ft.

Using table 23-II-I-1, $\frac{3}{8}$ in. plywood sheathing with 6d nails at 4 in. at all panel edges directly applied to framing (not over gypsum sheathing) has an allowable shear in pounds per foot of 300 lb/ft.
Wall overturning must be considered from the shear and compared to the resisting moment from gravity loads and proper anchorage must be provided to keep the wall from sliding off the foundation. Referring to Figure 16.7:

\[
V = \frac{11.25 \text{ k}}{2} = 5.625 \text{ k}
\]

Roof dead load is determined from a tributary area of half a rafter spacing width, one rafter, and the wall length

\[
roof \text{ DL} = (14 \text{ lb/ft}^2 \cdot 16 \text{ in}/12 \text{ in}/\text{ft}/2 + 4 \text{ lb/ft})21 \text{ ft} = 280 \text{ lb}
\]

Wall dead load can be determined with the material weights for stud walls, sheathing, gypsum board and wood shingles:

\[
wall \text{ DL} = (2 \text{ lb/ft}^2 + 3 \text{ lb/ft}^2 + 5 \text{ lb/ft}^2 + 2 \text{ lb/ft}^2) \cdot (21 \text{ ft})(17 \text{ ft}) = 4.3 \text{ k}
\]

\[
\text{overturning moment} = (5.625\text{k})(17 \text{ ft}) = 95.6 \text{ k-ft}
\]

\[
\text{resisting moment} = (4.6 \text{ k})(21 \text{ ft}/2) = 48.4 \text{ k-ft}
\]
The resisting moment is not enough to compensate for the overturning moment. We like the factor of safety for overturning to be 1.5, and there is no safety in this case, which means we must provide a tie down in tension (T). The L shape of the corner will help some resisting overturning, as well as the glulam beam reaction.

For equilibrium of moments (positive = negative)

\[ SF = \frac{M_{\text{resist}}}{M_{\text{overturning}}} \geq 1.5 \]

\[ T(21 \text{ ft}) + 48.4 \text{ k-ft} = (95.6 \text{ k-ft})1.5; \quad T_{\text{req'd}} = 4.52 \text{ k} \]

The shear must be resisted, and the code minimum bolting usually consists of \(\frac{1}{2}\) in. diameter bolts at 1 ft from the wall ends and at a maximum of 6 ft on center for the remainder of the wall length. If design for wind loading allows us to increase the allowable stress by 1/3, the number of bolts from single shear in a 2” sill plate parallel to the grain will be:

\[ (1.33)(480 \text{ lb/bolt})(n) \geq 5,625 \text{ lb} \]

\[ n \geq 8.8 \text{ bolts} \]

Use 9 bolts, spaced at 2.375 ft

(see next page for description of design value symbols)
\[ Z_v = \text{nominal lateral design value for single bolt in connection with all wood members loaded parallel to grain} \]

\[ Z_{s\perp} = \text{nominal lateral design value for single bolt in wood-to-wood connection with main member loaded parallel to grain and side member loaded perpendicular to grain} \]

\[ Z_{m\perp} = \text{nominal lateral design value for single bolt in wood-to-wood connection with main member loaded parallel to grain and side member loaded perpendicular to grain and side member loaded parallel to grain} \]

**East-West**

The tributary height for the wall and parapet and the distributed lateral wind load are the same as in the North-South direction.

The total lateral wind load = \((225 \text{ lb/ft})(50 \text{ ft}) = 11,250 \text{ lb}\)

The end reactions to the lateral load = \(11,250 \text{ lb}/2 = 5,625 \text{ lb}\)

The *unit shear* (or distributed shear) in the **diaphragm** = \(5,625 \text{ lb}/(100 \text{ ft}) = 56.25 \text{ lb/ft}.\)

It is convenient to use the diaphragm structural panel construction chosen in the North-South direction with a capacity of 270 lb/ft.

The *unit shear* (or distributed shear) in the five **shear walls** of 10.67 ft each:

\[ = 5,625 \text{ lb}/(5\cdot10.67 \text{ ft}) = 105 \text{ lb/ft}. \]

It is convenient to use the shear wall structural panel construction chosen in the North-South direction with a capacity of 300 lb/ft.