Most manufacturers are equipped with computer software that can rapidly design trusses of any configuration. In order to do the design, they must be provided with the following information:

- the type of structure
- design loads: special loading such as snow drift loads must be specified
- truss spans
- truss shape required including depth and clearance limitations
- bearing details
- overhang details

The truss plate manufacturer provides structural drawings that include such information as: the loads on which the design is based; truss spacing; points of support; species, grade and size of lumber required for the chords and webs; type, size and location and orientation of truss plates; lateral bracing requirements; bearing size; and other general notes.

When inspecting trusses *in situ*, it is important to carefully examine the truss plate connections. The plates must be properly installed and of the size specified in the drawings. A minor shift in the location of the truss plate can severely weaken the connection, so it is important to ensure that the plates are installed in the locations specified on the truss plate manufacturer's drawing.

The building designer is typically responsible for the structure that supports the trusses including the support of the truss bracing system. Bearing walls and lintels must be designed for the truss loads, including the point loads from girders. The building designer should also ensure that all connections between the trusses and support framing, such as hangers and uplift anchors are adequate. Truss drawings often call for bracing of long web members loaded in compression. It is typically the responsibility of the building designer to design the system for supporting the web bracing and to ensure that the bracing is properly installed.

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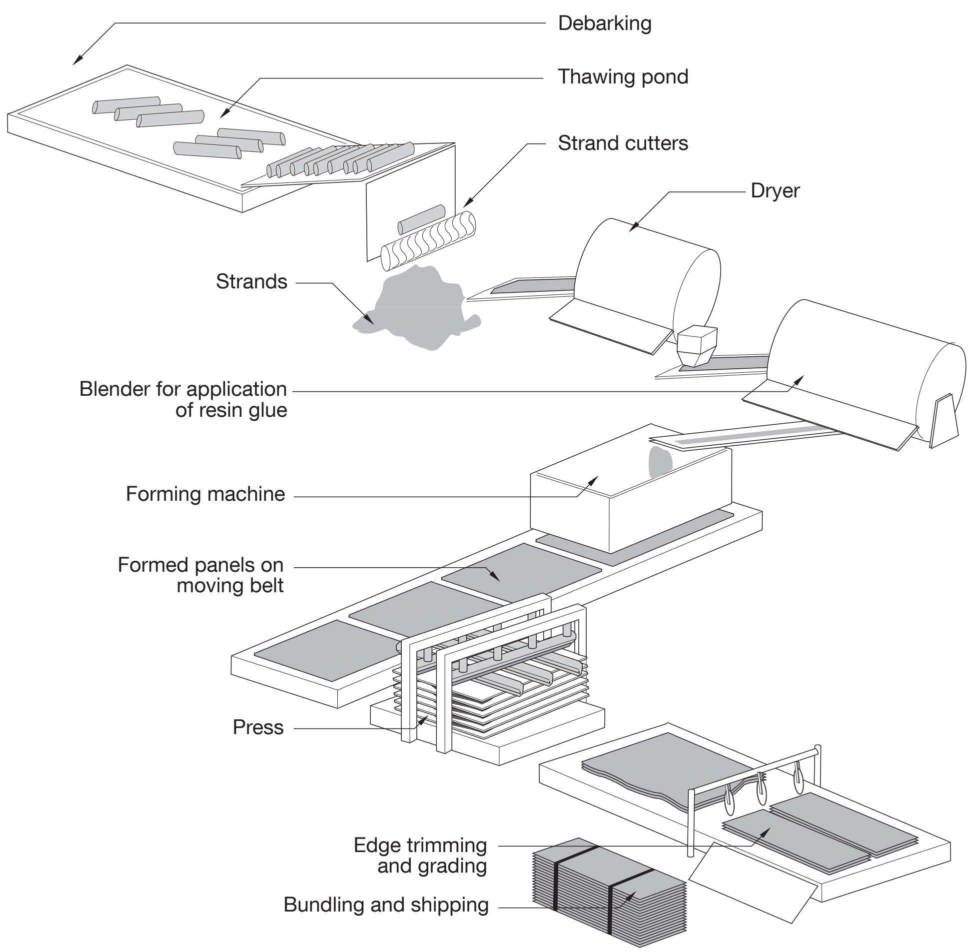
The intended end use is expressed through a letter “F”, “R” or “W”, indicating floor, roof or wall, respectively. Preceding the letter “F” or “R” is either the number 1 or 2. In the case of floors, the prefix “1” indicates that the panel is intended to be used as a single-layer floor (i.e., no separate underlayment required under carpet and pad). A number “2” indicates that an additional layer of panel-type underlayment is required. For roofs, the prefix “1” means that no additional edge support is required at maximum span while a number “2” indicates that the panel requires edge support.

For example, a panel marked with 1R24 indicates roof sheathing on supports spaced 24 inches on centre and without support on the long edges of the panel. Detailed information on span ratings for other types of end uses are provided in Table 6.16.

FIGURE 6.24

Manufacture of OSB



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Beams with a slenderness ratio, C_B, of less than 10 are generally considered to be stocky. Intermediate beams have a C_B of 10 to about 20, and slender beams have a C_B between about 20 and 50.

The stability rules for sawn lumber and glulam are essentially the same. Since sawn lumber beams are usually stocky, simple check rules to guard against lateral torsional buckling are adequate (Table 7.1). Glulam beams are typically much larger and more slender, and a detailed calculation of the lateral stability factor K_L is typically required.

The lateral stability factor, K_L, is used to determine the bending moment resistance. For stocky lumber beams meeting the slenderness and support conditions shown in Table 7.1 (CSA O86 Clause 6.5.4.2.1), $K_L = 1$. For intermediate and slender beams, K_L must be calculated (Figure 7.3) and be used to modify the bending moment resistance.

Imperfections such as out-of-straightness of the beam, knots and grain irregularities can affect the lateral stability, especially for very slender beams. These factors are implicitly considered in the design values.

Beam stability is an important consideration, but is not overly complicated. The design procedure involves determining the degree of lateral support and selecting a trial section based on the assumption that no buckling occurs. If the beam is stocky ($C_B \leq 10$), the design can be finalized. If the trial section is not stocky ($C_B > 10$), the K_L factor must be calculated and used to determine the bending moment resistance. If this is less than the maximum bending moment, iterative design must be performed to select a section which meets the stability and strength criteria for the given slenderness, loading and support conditions.

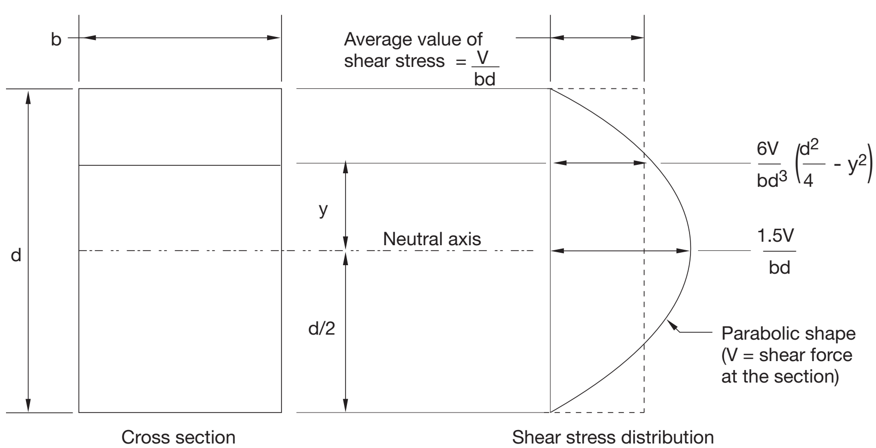
SHEAR RESISTANCE

In the calculation of maximum shear force, the loads within a distance from the support equal to the depth of the member may be neglected. This is because these loads are considered to be distributed down through the member in compression perpendicular to the grain.

For rectangular cross sections (the most common shape for solid sawn lumber), longitudinal shear stress is parabolically distributed over the depth of members. The maximum value, at mid-depth (the neutral axis) is 1.5 times the average shear stress (Figure 7.4).

FIGURE 7.4

Shear stress distribution



η = e/d

e = length of notch, mm, measured from the centre of the bearing (of the nearest support) to re-entrant corner of notch (Figure 7.10). For a member notched over an end support, the length of support may be taken as the lesser of the minimum required bearing length or the actual bearing length. For a continuous member, the length of support equals the actual bearing length.

For notches located on the tension side of the beam it is recommended that designers apply a more restrictive limit. For example, the 0.25d of notch depth could be further reduced to 0.1d.

For glulam members notched on the compression side, the factored shear resistance, V_r, is

$$V_r = \phi F_v \frac{2A_n}{3} \quad \text{for } e_c > d$$

$$V_r = \phi F_v \frac{2A_n}{3}\left(1 - \frac{d_n e_c}{d(d - d_n)}\right) \quad \text{for } e_c < d$$

where

ϕ = 0.9

F_v = $f_v (K_D K_H K_{Sv} K_T)$

f_v = specified strength in shear (CSA O86 Table 7.3), MPa

A_n = net cross-sectional area of member, mm^2

d = depth of beam, mm

d_n = notch depth, mm, which must not exceed 0.25d

e_c = length of notch, mm, from inner edge of closest support to farthest edge of notch (Figure 7.10)

Deflection

CSA O86 requires that elastic deflection under the specified loads be no greater than the span divided by 180. There are some additional requirements for certain cases to prevent plastic (non-recoverable) deformation. Under continuous loads of long duration, or severe ponding conditions, some non-recoverable defelction can occur. This is more likely where wood is wet and cannot dry out since wet wood is more susceptible to deformation.

Limits on deflection may be imposed to prevent either elastic or plastic deformation. They may also be intended to control unacceptable performance. A deflection limit of span divided by 360 under live loading is commonly used to prevent cracking in ceiling finishes such as gypsum board and plaster. A deflection limit may also be appropriate in some cases to prevent vibration in floors.

For glulam, it is possible to build in some camber to offset the deflections due to applied loads. Camber is curvature opposite to the anticipated deflection in service. This is not usually done for stock beams which are premanufactured for stocking in a distribution depot and which may be cut to length on order. However, most glulam beams and arches are made to order and may be cambered to suit the particular design conditions by gluing the laminations to the slightly curved shape required for camber.

For glulam beams of approximately 6 m in length or more, it is usual practice in the Canadian laminating industry to incorporate camber (Table 7.5). The camber used may range from approximately 24 mm for a simple beam spanning 6 m to 80 mm for a simple beam on a span of 20 m. Generally, designers should specify camber to counteract at least the deflection due to dead load. CSA O86 relaxes the elastic deflection limit to span/180 due to specified live, snow and wind loads where the camber provided is equal to calculated dead load deflection. Glulam fabricators may recommend camber based on their experiences.

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Check Deflection

By observation, the deflection at the end of the cantilever will govern cantilever live load deflection.

Limit = span/180

$$E_S I = 12400 \times 1 \times 1 \times \left(\frac{265 \times 1444^3}{12}\right)$$

$$= 8.24 \times 10^{14} \text{ N•mm}^2$$

$$\Delta_{overhang} = \frac{w_L a}{24EI}(4a^2 l - l^3 + 3a^3) + \frac{Pa}{6EI}(2al + 2a^2)$$

$$= \frac{27 \times 3000}{24 \times 8.24 \times 10^{14}} \times (4 \times 3000^2 \times 10000 - 10000^3 + 3 \times 3000^3) + \frac{200000 \times 3000}{6 \times 8.24 \times 10^{14}} \times (2 \times 3000 \times 10000 + 2 \times 3000^2)$$

= 7.18 mm

= span/418 *(Acceptable)*

Use a 265 × 1444 mm 20f-EX Douglas Fir-Larch glulam beam.

Note: In some cases engineering may choose an alternate approach to determine unsupported length based on the greater of:

- The length of the cantilever, or
- Where the member is subject to negative bending,
  - The distance between the support and zero moment, or
  - The distance between zero moments within a span.

See Example 13.3.

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$t = 191 - 15 = 176$ mm

$n_c = 1$

$a_{cr\,i} = 57$ mm

$PR_{r\,T} = 0.7 \times (1.2 \times 1.5 \times 0.65 \times 0.67 \times 1.0 \times 0.65 \times 176 \times 1 \times 57) \times 2$

$= 7.16$ kN

Group Tear-Out Resistance

$$PG_{ri} = \phi_w \left[\frac{PR_{i1} + PR_{i2}}{2} + f_t (K_D K_{Sv} K_T) A_{PGi} \right]$$

where

$\phi_w = 0.7$

$PR_{i1} = 1.2\, f_v(K_D\, K_{Sv}\, K_T) K_{ls}\, t\, n_c\, a_{cr\,1}$

$= 1.2 \times 1.5 \times (0.65 \times 0.67 \times 1.0) \times 0.65 \times 176 \times 1 \times 57$

$= 5.11$ kN

$PR_{i2} = 1.2\, f_v(K_D\, K_{Sv}\, K_T)\, K_{ls}\, t\, n_c\, a_{cr\,2}$

$= 1.2 \times 1.5 \times (0.65 \times 0.67 \times 1.0) \times 0.65 \times 176 \times 1 \times 77$

$= 6.90$ kN

$f_t = 3.8$ MPa (CSA O86 Table 6.3.1D)

$A_{pGi} = (77 - 19) \times 191$

$= 11100\ mm^2$

$PG_{r\,T} = 0.7 \times \left[\frac{(5110 + 6900)}{2} + [3.8 \times (0.65 \times 0.67 \times 1.0) \times 11100] \right]$

$= 17.1$ kN

Net Tension Resistance

$T_r = \phi F_t A_n K_{Zt}$

where

$\phi_w = 0.9$

$F_t = f_t(K_D\, K_H\, K_{St}\, K_T)$

$f_t = 3.8$ MPa (CSA O86 Table 6.3.1D)

$A_n = (191 - 19 \times 2) \times 191$

$= 29.2 \times 10^3\ mm^2$

$A_n \geq 0.75 A_g$ *(Acceptable)*

$K_{Zt} = 1.2$ (CSA O86 Table 6.4.5)

$T_r = 0.9 \times (3.8 \times 0.65 \times 1.0 \times 1.0 \times 1.0) \times 29.2 \times 10^3 \times 1.2$

$T_{Nrt} = 78.0$ kN

Splitting Resistance

$QS_{r\,i} = \phi_w QS_i (K_D\, K_{SF} K_T)$

where

$\phi_w = 0.7$

$$QS_i = 14t \sqrt{\frac{d_e}{1 - \frac{d_e}{d}}}$$

where

$t = 191$ mm

$d_e = d - e_p$

13.3 Glulam Roof Beam – Cantilever and Suspended

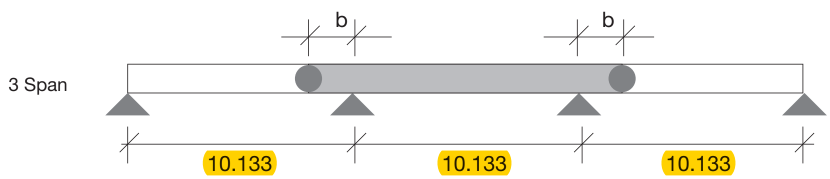
GIVEN VALUES AND ASSUMPTIONS

Design glulam roof beam B1 using cantilever and suspended span construction as an alternative to simple span construction in Section 13.2.

- Beam spacing = 12.2 m
- Beam span = 10.133 m
- Flat roof with no ceiling
- Flat roof trusses provide full lateral support on the upper surface of the beam
- Use 20f-EX D.Fir-L glulam

FIGURE 13.1

Glulam roof beam B1 using cantilever and suspended span construction



CALCULATION

1. Select configuration with hinges in outside bays (WDM Figure 2.10)

2. Determine factored load ratio, n

   For roofs: $n = \dfrac{D + 0.5S}{D + S} = \dfrac{(1.25 \times 0.85) + (0.5 \times 1.5 \times 1.6)}{(1.25 \times 0.85) + (1.5 \times 1.6)} = 0.65$

3. Determine hinge points (WDM Figure 2.10)

   $b = 0.152 \times \text{span} = 0.152 \times 10.133 = 1.540 \text{ m}$

4. Determine maximum factored moments and shears on the suspended beam

   $M_f \quad = \dfrac{w_f L^2}{8} = \dfrac{42.2 \text{ x } (10.133 - 1.540)^2}{8} = 390 \text{ kN•m}$

   $V_f \quad = \dfrac{w_f L}{2} = \dfrac{42.2 \text{ x } (10.333 - 1.540)}{2} = 181 \text{ kN}$

5. Determine maximum factored moments and shears on the cantilevered beam (concentrated loads are end shears from suspended beams). See Figure 13.2.
   M_f- for Case 1 should be equal to M_f+ for Case 2 since optimum hinge points are used

   M_f- = 329 kN•m (Case 1)

   M_f+ = 327 kN•m (Case 2)

   M_f = 329 kN•m (Case 1)

   V_f = 246 kN (Case 1)