## Sawn Timber

$P_r$ is the lesser of:

$$P_{rd} = \phi\, F_c\, A\, K_{Zcd}\, K_{cd} \text{ or } P_{rb} = \phi\, F_c\, A\, K_{Zcb}\, K_{cb}$$

where:

$\phi F_c$ = factored compressive resistance strength (MPa) given in Table 3.6

$K_{Zc}$ = size factor

$K_{Zcd} = 6.3\,(dL_d)^{-0.13} \leq 1.3$ for buckling in direction of d

$K_{Zcb} = 6.3\,(bL_b)^{-0.13} \leq 1.3$ for buckling in direction of b

$K_C$ = slenderness factor

$K_{Cd} = \left[1.0 + \frac{F_c}{E'} K_{Zcd}\, C_{cd}^3\right]^{-1}$ for buckling in direction of d

$K_{Cb} = \left[1.0 + \frac{F_c}{E'} K_{Zcb}\, C_{cb}^3\right]^{-1}$ for buckling in direction of b

$F_c/E'$ = strength to stiffness ratio given in Table 3.7

$C_{cd} = \frac{K_e L_d}{d}$ $C_{cb} = \frac{K_e L_b}{b}$ ($C_{cd}$ or $C_{cb}$ >50 is not permitted)

$K_e$ = effective length factor, given in Figure 3.1

$L_b$, $L_d$ = unsupported length associated with d or b (mm)

d = depth of member (mm)

b = thickness of member (mm)

## Glulam

$$P_r = \phi\, F_c\, A\, K_{Zcg}\, K_C$$

where:

$\phi F_c$ = factored compressive resistance strength (MPa) given in Table 3.8

$K_{Zcg} = 0.68\,(Z)^{-0.13} \leq 1.0$, where Z=member volume in $m^3$

$K_C$ = slenderness factor

$= \left[1.0 + \frac{F_c}{E'} K_{Zcg}\, C_c^3\right]^{-1}$

$F_c/E'$ = strength to stiffness ratio given in Table 3.9

$C_c$ = the greater of $\frac{K_e L_d}{d}$ or $\frac{K_e L_b}{b}$ ($C_c$>50 is not permitted)

$K_e$ = effective length factor, given in Figure 3.1

$L_b$, $L_d$ = unsupported length associated with d or b (mm)

d = depth of member (mm)

b = thickness of member (mm)

August 2013 Errata



1. The studs must be designed to satisfy the following interaction equation

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_r}\left[\frac{1}{1-\frac{P_f}{P_E}}\right] \leq 1.0$$

where:

$P_f$ = factored axial load on stud (N)

$P_r$ = factored compressive resistance parallel to grain taken from Table 9.17 ($K_D$ = 1.0) ($K_T$ = 0.75) (N)

$M_f$ = maximum factored bending moment

$$\frac{w_f H^2}{6L}\left[L - H + \frac{2}{3}\sqrt{\frac{H^3}{3L}}\right]$$

$w_f$ = factored loading (N/mm)

= 1.5 x specified lateral soil pressure (kN/$m^2$) x stud spacing (m)

$M_r$ = factored bending moment resistance taken from Joist Selection Tables in 2.3 modified for permanent load duration ($K_D$ = 0.65) and treatment factor for preservation treated, incised lumber used in dry service conditions ($K_T$ = 0.75) (kN•m)

2. Maximum deflection under specified loads ≤ deflection criteria (actual $E_sI$ ≥ required $E_sI$)

Maximum deflection, Δ, may be calculated using the formula given below:

$$\Delta = \frac{w(L-x)}{360(E_sI)LH} K_\Delta \text{ (mm)}$$

where:

w = specified lateral soil pressure (kN/$m^2$) x stud spacing (m)

$K_\Delta$ = $10 H^3 (2L-x)x - 3H^5 + K_2$

$K_2$ = $\frac{3L}{L-x}(H-x)^5$ when $x \leq H$

$K_2$ = 0 when x > H

x = 0.45L

$E_sI$ = bending stiffness taken from Joist Selection Tables and modified by the treatment factor for preservative treated, incised lumber used in dry service conditions ($K_T$ = 0.9) (N•$mm^2$)

Generally, L/300 is used as the deflection criteria.

**January 2013 Errata**

**Notice to Purchasers of the Wood Design Manual Concerning Equivalency Between Spruce-Pine-Fir and Southern Pine Dimension Lumber**

Design values for some grades and sizes of Southern Pine visually graded lumber, published by the Southern Pine Inspection Bureau (SPIB) have been reduced. The American Lumber Standards Committee Board of Review has approved these changes to take effect June 1st 2012.

SPIB is currently reviewing design values for all sizes and grades and it is expected that there will be further changes to Southern Pine design values.

A CSA O86 Technical Committee ballot is currently being conducted to approve an amendment that would remove the equivalency between Southern Pine and Spruce-Pine-Fir in Table 5.2.1.3 of CSA O86 until the outcome of SPIB's review is completed:

**Table 5.2.1.3**
**Lumber species equivalents**

| US combination | Equivalent Canadian combination |
|---|---|
| Douglas Fir-Larch | Douglas Fir-Larch |
| Hem-Fir | Hem-Fir |
| ~~Southern Pine~~ | ~~Spruce-Pine-Fir~~ |

**Note:** *The NLGA Standard Grading Rules for Canadian Lumber incorporates the National Grading Rule for Dimension Lumber, a uniform set of grade descriptions and other requirements for softwood dimension lumber that form a required part of all softwood lumber grading rules in the United States. Thus, all dimension lumber throughout Canada and the United States is graded to uniform requirements.*

The ballot can be reviewed at http://publicreview.csa.ca/

The result of the ballot will be circulated once the ballot has been completed

Further information can be found on the SPIB website at http://www.spib.org/important-notice.shtml

**September 2012 Errata**

# Update No. 4
# O86-09
# September 2012

**Note:** *For information about the* ***Standards Update Service*** *or if you are missing any updates, go to* ***shop.csa.ca*** *or e-mail* ***techsupport@csagroup.org****.*

**Title:** *Engineering design in wood* — originally published May 2009

**Revisions issued:** Update No. 1 — September 2010
Update No. 2 — March 2011
Update No. 3 — December 2011 (applies to the French Standard only)

The following revisions have been formally approved and are marked by the symbol delta (Δ) in the margin on the attached replacement pages:

| | |
|---|---|
| **Revised** | Clause 10.11.4.2 and Tables 5.2.1.3 and 10.3.4 |
| **New** | None |
| **Deleted** | None |

- Update your copy by inserting these revised pages.
- Keep the pages you remove for reference.

Δ

**Table 5.2.1.3**
**Lumber species equivalents**

| US combination | Equivalent Canadian combination |
|---|---|
| Douglas Fir-Larch | Douglas Fir-Larch |
| Hem-Fir | Hem-Fir |

**Note:** *The NLGA's* Standard Grading Rules for Canadian Lumber *incorporates the* National Grading Rule for Dimension Lumber, *a uniform set of grade descriptions and other requirements for softwood dimension lumber that form a required part of all softwood lumber grading rules in the United States. Thus, all dimension lumber throughout Canada and the United States is graded to uniform requirements.*

## 5.2.2 Lumber grades and categories

### 5.2.2.1 Visually stress-graded lumber

Table 5.2.2.1 lists categories, limiting dimensions, and structural grades for which design data are assigned in this Standard. These grades are specified in the NLGA's *Standard Grading Rules for Canadian Lumber.*

**Table 5.2.2.1**
**Visual grades and their dimensions**

| Grade category | Smaller dimension, mm | Larger dimension, mm | Grades |
|---|---|---|---|
| Light framing | 38 to 89 | 38 to 89 | Construction, Standard |
| Stud | 38 to 89 | 38 or more | Stud |
| Structural light framing | 38 to 89 | 38 to 89 | Select Structural<br>No. 1, No. 2, No. 3 |
| Structural joists and planks | 38 to 89 | 114 or more | Select Structural<br>No. 1, No. 2, No. 3 |
| Beam and stringer | 114 or more | Exceeds smaller dimension by more than 51 | Select Structural<br>No. 1, No. 2 |
| Post and timber | 114 or more | Exceeds smaller dimension by 51 or less | Select Structural<br>No. 1, No. 2 |
| Plank decking | 38 to 89 | 140 or more | Select, Commercial |

### 5.2.2.2 Machine stress-rated (MSR) and machine evaluated lumber (MEL)

The design data specified in this Standard apply to lumber graded and grade-stamped in accordance with NLGA SPS 2 and identified by the grade stamp of a grading agency accredited for grading by mechanical means.

**Note:** *A list of accredited agencies can be obtained from the Canadian Lumber Standards Accreditation Board.*

### 5.2.3 Finger-joined lumber

#### 5.2.3.1
The design data specified in this Standard apply to finger-joined lumber that has been produced and grade-stamped in accordance with NLGA SPS 1.

#### 5.2.3.2
The design data specified in this Standard apply to finger-joined lumber that has been produced and grade-stamped in accordance with NLGA SPS 3 where used under the following conditions:
(a) applications where the primary loading is in compression, with only short-duration stresses in bending or tension, such as due to wind or earthquake loads; and
(b) protected from wet service conditions at all times and not used in environments where the equilibrium moisture content can be expected to exceed 19% or the temperature can be expected to exceed 50 °C for an extended period of time.

### 5.2.4 Remanufactured lumber
Dimension lumber and timbers that are resawn or otherwise remanufactured shall be regraded in accordance with Clause 5.2.1.

### 5.2.5 Mixed grades
When mixed grades are used, the specified strength shall be that of the grade having the lowest value.

## 5.3 Specified strengths

### 5.3.1 Visually stress-graded lumber

#### 5.3.1.1
The specified strengths (MPa) for visually stress-graded lumber are tabulated as follows:
(a) structural joist and plank, structural light framing, and stud grade categories of lumber in Table 5.3.1A;
(b) light framing grades in Table 5.3.1B;
(c) beam and stringer grade categories of lumber in Table 5.3.1C; and
(d) post and timber grade categories of lumber in Table 5.3.1D.

#### 5.3.1.2
The specified strengths (MPa) for plank decking shall be derived from Table 5.3.1A using the following grade equivalents:

| Decking grade | Equivalent lumber grade |
|---|---|
| Select | Select structural |
| Commercial | No. 2 |

### 5.3.2 Machine stress-rated and machine evaluated lumber
The specified strengths (MPa) for machine stress-rated lumber are given in Table 5.3.2. The specified strengths (MPa) for machine evaluated lumber are given in Table 5.3.3. Specified strengths in shear are not grade dependent and shall be taken from Table 5.3.1A for the appropriate species.

### 10.3.4 Lumber thickness

Connectors installed in lumber of a thickness less than the minimum specified in Table 10.3.4 for the connector type and use shall not be considered to provide resistance.

Δ **Table 10.3.4**
**Thickness factor for timber connector, $J_T$**

| Connector type and size | Number of faces of a piece containing connectors on a bolt | Thickness of piece, mm | $J_T$ |
|---|---|---|---|
| 2-1/2 in split ring | 1 | 38 | 1.00 |
| | | 25 | 0.85 |
| | 2 | 51 | 1.00 |
| | | 38 | 0.80 |
| 4 in split ring | 1 | 38 | 1.00 |
| | | 25 | 0.65 |
| | 2 | 76 | 1.00 |
| | | 64 | 0.95 |
| | | 51 | 0.80 |
| | | 38 | 0.65 |
| 2-5/8 in shear plate | 1 | 64 | 1.00 |
| | | 51 | 0.95 |
| | | 38 | 0.95 |
| | 2 | 64 | 1.00 |
| | | 51 | 0.95 |
| | | 38 | 0.75 |
| 4 in shear plate | 1 | 44 | 1.00 |
| | | 38 | 0.85 |
| | 2 | 89 | 1.00 |
| | | 76 | 0.95 |
| | | 64 | 0.85 |
| | | 51 | 0.75 |
| | | 44 | 0.65 |

### 10.3.5 Lag screw connector joints

When lag screws instead of bolts are used with connectors, the resistance shall vary uniformly with penetration into the member receiving the point, from the full resistance for one connector unit with bolt for standard penetration to 0.75 times the full resistance for one connector unit with bolt for minimum penetration. Penetration shall be as specified in Table 10.3.5 and shall be not less than the minimum value.

### Table 10.3.5
### Penetration factor, $J_P$, for split rings and shear plates used with lag screws

| Connector | Penetration | Penetration of lag screw into member receiving point (number of shank diameters) | | | | |
|---|---|---|---|---|---|---|
| | | Species | | | | |
| | | Douglas Fir-Larch | Hem-Fir | Spruce-Pine-Fir | Northern Species | $J_P$ |
| 2-1/2 in split ring | Standard | 8 | 10 | 10 | 11 | 1.00 |
| 4 in split ring or 4 in shear plate* | Minimum | 3.5 | 4 | 4 | 4.5 | 0.75 |
| 2-5/8 in shear plate* | Standard<br>Minimum | 5<br>3.5 | 7<br>4 | 7<br>4 | 8<br>8 | 1.00<br>0.75 |

**When steel side plates are used with shear plates, use $J_P$ = 1.0.*

**Note:** *For intermediate penetrations, linear interpolation may be used for values of $J_P$ between 0.75 and 1.00.*

## 10.3.6 Lateral resistance

The factored lateral strength resistance of a split ring or shear plate connection, $P_r$, $Q_r$, or $N_r$, determined using the equations specified in this Clause, shall be greater than or equal to the effect of the factored loads. The factored strength resistance per shear plate unit shall not exceed the values specified in Table 10.3.6C.

(a) For parallel-to-grain loading:

$$P_r = \phi P_u n_F J_F$$

(b) For perpendicular-to-grain loading:

$$Q_r = \phi Q_u n_F J_F$$

(c) For loads at angle $\theta$ to grain:

$$N_r = \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

$\phi$ = 0.6

$P_u$ = $p_u(K_D K_{SF} K_T)$

where

$p_u$ = lateral strength resistance parallel to grain, kN (Table 10.3.6A)

$J_F$ = $J_G J_C J_T J_O J_P$

where

$J_G$ = factor for groups of fastenings (Tables 10.2.2.3.4A and 10.2.2.3.4B)

$J_C$ = minimum configuration factor (Clause 10.3.3 and Tables 10.3.3A to 10.3.3C)

$J_T$ = thickness factor (Table 10.3.4)

$J_O$ = factor for connector orientation in grain

= 1.00 for side grain installation

= 0.67 for end grain and all other installations

$J_P$ = factor for lag screw penetration (Clause 10.3.5 and Table 10.3.5)

### 10.11.3.2
The factored withdrawal resistance of a single wood screw per millimetre of threaded shank penetration when driven perpendicular to the grain shall be determined in accordance with Clause 10.11.5. Wood screws installed through end grain shall not be considered to carry load in withdrawal.

## 10.11.4 Lateral resistance

### 10.11.4.1
For two- or three-member joints, the factored lateral strength resistance of a wood screw joint shall be taken as follows:

$N_r = \phi N_u n_f n_s J_A J_E$

where

$\phi$ = 0.8

$N_u$ = $n_u(K_D K_{SF} K_T)$

where

$n_u$ = unit lateral strength resistance, N (Clause 10.11.4.2)

$n_F$ = number of fasteners in the connection

$n_s$ = number of shear planes per screw

$J_A$ = toe-screwing factor

= 0.83 where screws are started at approximately one-third the screw length from the end of a piece and driven at an angle of about 30° to the grain of the member

= 1.0 in all other cases

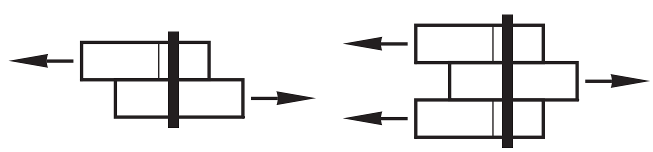
$J_E$ = factor for fastening into end grain

= 0.67 in end grain

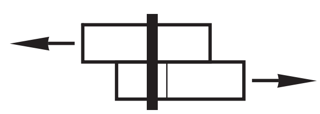
= 1.0 in all other cases

### 10.11.4.2
The unit lateral strength resistance, $n_u$ (N per shear plane), shall be taken as the smallest value calculated in accordance with Items (a) to (g). For two-member connections, only Items (a), (b), (d) to (f), and (g) are valid. For three-member connections, where screws fully penetrate all three members, only Items (a), (c), (d), and (g) are valid.

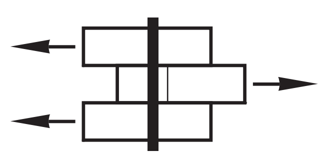
(a) $f_1 d_F t_1$



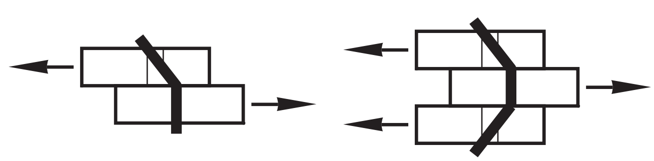
(b) $f_2 d_F t_2$



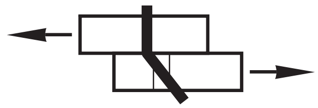
(c) $\frac{1}{2} f_2 d_F t_2$



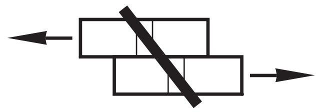
(d) $f_1 d_F^2 \left( \sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$



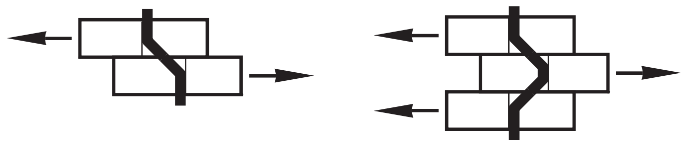
(e) $f_1 d_F^2 \left( \sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$



(f) $f_1 d_F^2 \frac{1}{5} \left( \frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$



(g) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}}$



where

$d_F$ = wood screw diameter, mm (Table 10.11.1)

$t_1$ = head-side member thickness for two-member connections, mm
= minimum side plate thickness for three-member connections, mm (Clause 10.11.2.3)

$f_2$ = embedding strength of main member where failure is wood bearing, MPa
= $50\ G\ (1 - 0.01 d_F)$

where
$G$ = mean relative density of lumber or glulam member (see Table A.10.1)

$t_2$ = length of penetration into point-side member for two-member connections, mm
= centre member thickness for three-member connections, mm (Clause 10.11.2.3)

$f_3$ = embedding strength of main member where failure is fastener yielding, MPa
= $110\ G^{1.8}\ (1 - 0.01 d_F)$

$f_y$ = wood screw yield strength, MPa (Table 10.11.1)

For lumber side plates:

$f_1$ = embedding strength, MPa
= $50\ G\ (1 - 0.01 d_F)$

For structural panel side plates:

$f_1$ = embedding strength, MPa
= $104\ G\ (1 - 0.1 d_F)$

where

Δ $G$ = 0.49 for DFP
= 0.42 for CSP and OSB

For steel side plates:

$f_1$ = embedding strength of steel side plate, MPa
= $K_{sp}\ (\phi_{steel} / \phi_{wood})\ f_u$

where

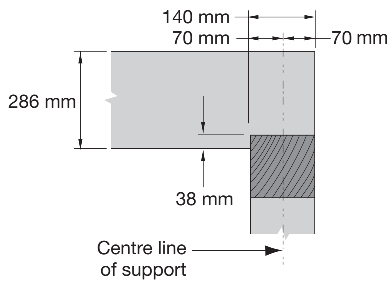
$K_{sp}$ = 3.0 for mild steel meeting the requirements of CSA G40.21 or ASTM A 36/A 36M
= 2.7 for light gauge steel

$\phi_{steel}$ = resistance factor for steel member in wood screw connection

### Example 2: Joists notched on tension side at supports

Verify that the Hem-Fir No.1/No.2 38 × 286 mm floor joists (see drawing) notched at the supports are adequate for the following conditions:

- joist spacing = 600 mm
- joist span = 4.0 m
- specified dead load = 1.2 kPa
- specified live load = 2.4 kPa
- standard load duration
- dry service conditions
- untreated
- fully laterally supported by subfloor
- Case 2 system




**Calculation**

Total factored load = (1.25 × 1.20) + (1.5 × 2.40) = 5.10 kPa
Total specified load = 1.20 + 2.40 = 3.60 kPa

$$w_f = 5.10 \times 0.6 = 3.06 \text{ kN/m}$$

$$w = 3.60 \times 0.6 = 2.16 \text{ kN/m}$$

$$w_L = 2.40 \times 0.6 = 1.44 \text{ kN/m}$$

$$M_f = \frac{w_f L^2}{8} = \frac{3.06 \times 4.0^2}{8} = 6.12 \text{ kN}\bullet\text{m}$$

$$V_f = \frac{w_f L}{2} = \frac{3.06 \times 4.0}{2} = 6.12 \text{ kN}$$

$$A_n = 9424 \text{ mm}^2$$

$$A = 10870 \text{ mm}^2$$

$$A_n/A = 0.87$$

Note: In this example, the maximum factored reaction force $Q_f$ is equal to $V_f$.

From the Serviceability Table:

$E_S I_{REQ'D} = 432 \times 10^9$ N•mm² for L/360 deflection based on live load ($w_L$)

From Joist Selection Tables:

$M_r = 7.18 > 6.12$ kN•m *Acceptable*

$V_r = 14.6 \times 0.87 = 12.70 > 6.12$ kN *Acceptable*

$E_S I = 815 \times 10^9 > 432 \times 10^9$ N•mm² *Acceptable*

The factored notch shear force resistance at the support:

$$F_r = \phi F_f A K_N$$

$$\phi = 0.9$$

$$F_f = 0.5 \times 1.4 = 0.7 \text{ MPa}$$

$$A = 38 \times 286 = 10868 \text{ mm}^2$$

$$K_N = \left[0.006\,d\left[1.6\left[\frac{1}{\alpha} - 1\right] + \eta^2\left[\frac{1}{\alpha^3} - 1\right]\right]\right]^{-0.5}$$

$$\alpha = 1 - \frac{d_n}{d} = 1 - \frac{38}{286} = 0.24$$

$$\eta = \frac{e}{d} = \frac{70}{286} = 0.87$$

$$K_N = \left[0.006 \times 286\left[1.6\left[\frac{1}{0.87} - 1\right] + 0.24^2\left[\frac{1}{0.87^3} - 1\right]\right]\right]^{-0.5} = 1.45$$

Note: As an alternative, $K_N$ may also be calculated using Table 5.5.5.4 of CSA O86.

Therefore:

$$F_r = 0.9 \times 0.7 \times 10868 \times 1.45$$
$$= 9.93 \text{ kN} > Q_f = 6.12 \text{ kN}$$ *Acceptable*

Note: Also verify acceptable bearing capacity as per Chapter 6.

June 2012 Errata



# Column Selection Tables

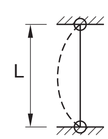
## Glulam

**365 mm**

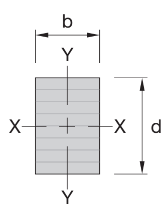
| L | Spruce-Pine 12c-E | | | | | | | | D.Fir-L 16c-E | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| d (mm) | 342 | | 380 | | 418 | | 456 | | 342 | | 380 | | 418 | | 456 | |
| m | $P_{rx}$ kN | $P_{ry}$ kN | $P_{rx}$ kN | $P_{ry}$ kN | $P_{rx}$ kN | $P_{ry}$ kN | $P_{rx}$ kN | $P_{ry}$ kN | $P_{rx}$ kN | $P_{ry}$ kN | $P_{rx}$ kN | $P_{ry}$ kN | $P_{rx}$ kN | $P_{ry}$ kN | $P_{rx}$ kN | $P_{ry}$ kN |
| 2.0 | 2020 | 2030 | 2220 | 2220 | 2420 | 2410 | 2620 | 2600 | 2420 | 2430 | 2670 | 2660 | 2900 | 2890 | 3140 | 3120 |
| 2.5 | 1940 | 1950 | 2140 | 2140 | 2340 | 2320 | 2530 | 2500 | 2330 | 2340 | 2570 | 2560 | 2800 | 2790 | 3030 | 3010 |
| 3.0 | 1860 | 1880 | 2060 | 2060 | 2260 | 2240 | 2450 | 2410 | 2240 | 2250 | 2480 | 2470 | 2710 | 2680 | 2940 | 2900 |
| 3.5 | 1780 | 1800 | 1990 | 1980 | 2190 | 2150 | 2380 | 2320 | 2140 | 2170 | 2390 | 2380 | 2630 | 2590 | 2860 | 2790 |
| 4.0 | 1700 | 1730 | 1910 | 1900 | 2120 | 2060 | 2310 | 2230 | 2050 | 2080 | 2300 | 2280 | 2540 | 2480 | 2780 | 2680 |
| 4.5 | 1610 | 1650 | 1830 | 1810 | 2040 | 1970 | 2240 | 2130 | 1950 | 1990 | 2210 | 2190 | 2460 | 2380 | 2690 | 2570 |
| 5.0 | 1530 | 1570 | 1750 | 1720 | 1960 | 1880 | 2170 | 2030 | 1850 | 1900 | 2110 | 2080 | 2370 | 2270 | 2610 | 2450 |
| 5.5 | 1430 | 1490 | 1670 | 1630 | 1880 | 1780 | 2090 | 1920 | 1740 | 1800 | 2020 | 1980 | 2280 | 2150 | 2520 | 2330 |
| 6.0 | 1340 | 1400 | 1580 | 1540 | 1800 | 1680 | 2020 | 1810 | 1630 | 1700 | 1910 | 1870 | 2180 | 2040 | 2430 | 2200 |
| 6.5 | 1250 | 1320 | 1490 | 1450 | 1720 | 1580 | 1940 | 1710 | 1520 | 1600 | 1810 | 1760 | 2080 | 1920 | 2340 | 2080 |
| 7.0 | 1160 | 1230 | 1400 | 1350 | 1630 | 1480 | 1860 | 1600 | 1420 | 1500 | 1710 | 1650 | 1980 | 1800 | 2250 | 1950 |
| 7.5 | 1070 | 1140 | 1310 | 1260 | 1550 | 1380 | 1770 | 1490 | 1310 | 1400 | 1600 | 1540 | 1880 | 1680 | 2150 | 1820 |
| 8.0 | 982 | 1060 | 1220 | 1170 | 1460 | 1280 | 1690 | 1380 | 1210 | 1300 | 1500 | 1440 | 1780 | 1570 | 2050 | 1700 |
| 8.5 | 901 | 983 | 1140 | 1080 | 1370 | 1180 | 1600 | 1280 | 1110 | 1210 | 1400 | 1330 | 1680 | 1460 | 1960 | 1580 |
| 9.0 | 826 | 908 | 1060 | 1000 | 1290 | 1090 | 1520 | 1190 | 1020 | 1120 | 1300 | 1240 | 1580 | 1350 | 1860 | 1460 |
| 9.5 | 755 | 837 | 979 | 923 | 1210 | 1010 | 1440 | 1090 | 937 | 1040 | 1210 | 1140 | 1480 | 1250 | 1760 | 1350 |
| 10.0 | 690 | 770 | 905 | 850 | 1130 | 930 | 1360 | 1010 | 858 | 955 | 1120 | 1050 | 1390 | 1150 | 1670 | 1250 |
| 10.5 | 630 | 708 | 836 | 782 | 1050 | 856 | 1280 | 929 | 786 | 880 | 1040 | 972 | 1300 | 1060 | 1570 | 1150 |
| 11.0 | 576 | 651 | 772 | 719 | 983 | 787 | 1200 | 855 | 719 | 810 | 959 | 895 | 1220 | 980 | 1480 | 1060 |
| 11.5 | 526 | 598 | 712 | 661 | 916 | 724 | 1130 | 786 | 658 | 746 | 887 | 825 | 1130 | 903 | 1390 | 980 |
| 12.0 | 481 | 549 | 657 | 607 | 852 | 665 | 1060 | 723 | 603 | 687 | 819 | 759 | 1060 | 831 | 1310 | 903 |
| 12.5 | 440 | 505 | 606 | 559 | 793 | 612 | 994 | 665 | 552 | 632 | 757 | 699 | 986 | 766 | 1230 | 833 |
| 13.0 | 403 | 464 | 559 | 514 | 737 | 563 | 931 | 612 | 506 | 582 | 700 | 644 | 918 | 706 | 1160 | 767 |
| 13.5 | 369 | 427 | 516 | 473 | 685 | 518 | 872 | 564 | 464 | 537 | 647 | 594 | 855 | 651 | 1080 | 708 |
| 14.0 | 339 | 393 | 476 | 436 | 637 | 478 | 816 | 520 | 427 | 495 | 598 | 548 | 796 | 601 | 1020 | 653 |
| 14.5 | 311 | 363 | 440 | 402 | 592 | 441 | 764 | 479 | 392 | 457 | 554 | 506 | 742 | 555 | 953 | 603 |
| 15.0 | 286 | 335 | 407 | 371 | 551 | 407 | 715 | 443 | 361 | 422 | 513 | 467 | 691 | 513 | 893 | 558 |

Notes:

1. $P_{rx}$ is the factored resistance to buckling about the x-x (strong) axis.
   $P_{ry}$ is the factored resistance to buckling about the y-y (weak) axis.
2. For $L \leq 2.0$ m, use $P_r$ for L = 2.0 m.
   Where $P_r$ values are not given, the slenderness ratio exceeds 50 (maximum permitted).
3. Tabulated values are valid for the following conditions:
   - standard term load (dead plus snow or occupancy loads)
   - dry service conditions
   - no fire-retardant treatment
   - $K_e = 1.0$
   - concentrically loaded
4. L = unsupported length




$K_e = 1.0$



June 2012 Errata



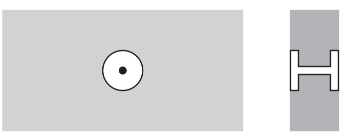
# Tension Member Selection Tables

## Sawn Timber

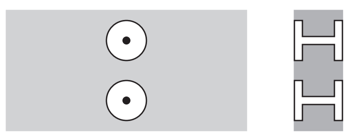
**140 mm**

| Species | Size (b × d) mm | Select Structural $T_r$ kN | Select Structural $T_{rN}$ for use with shear plates or split rings (kN) 2-5/8” SH PL 1 row | 2-5/8” SH PL 2 rows | 4” SH PL 1 row | 4” SH PL 2 rows | No.1 $T_r$ kN | No.1 $T_{rN}$ for use with shear plates or split rings (kN) 2-5/8” SH PL 1 row | 2-5/8” SH PL 2 rows | 4” SH PL 1 row | 4” SH PL 2 rows |
|---|---|---|---|---|---|---|---|---|---|---|---|
| D.Fir-L | 140 × 140 | 245 | 196 | | | | 186 | 148 | | | |
| | 140 × 191 | 309 | 264 | | 245 | | 234 | 199 | | 185 | |
| | 140 × 241 | 334 | 295 | 256 | 279 | | 234 | 207 | 179 | 195 | |
| | 140 × 292 | 368 | 332 | 297 | 318 | | 258 | 233 | 208 | 223 | |
| | 140 × 343 | 389 | 357 | 325 | 344 | 299 | 272 | 250 | 228 | 241 | 209 |
| | 140 × 394 | 397 | 369 | 340 | 357 | 317 | 278 | 258 | 238 | 250 | 222 |
| Hem-Fir | 140 × 140 | 181 | 145 | | | | 138 | 110 | | | |
| | 140 × 191 | 228 | 195 | | 181 | | 173 | 148 | | 137 | |
| | 140 × 241 | 247 | 218 | 189 | 207 | | 174 | 153 | 133 | 145 | |
| | 140 × 292 | 272 | 246 | 220 | 235 | | 191 | 173 | 154 | 165 | |
| | 140 × 343 | 288 | 264 | 241 | 255 | 221 | 202 | 186 | 169 | 179 | 156 |
| | 140 × 394 | 294 | 273 | 252 | 264 | 235 | 207 | 192 | 177 | 186 | 165 |
| S-P-F | 140 × 140 | 170 | 136 | | | | 128 | 103 | | | |
| | 140 × 191 | 214 | 182 | | 169 | | 162 | 138 | | 128 | |
| | 140 × 241 | 234 | 207 | 179 | 195 | | 164 | 145 | 125 | 137 | |
| Northern | 140 × 140 | 161 | 128 | | | | 122 | 97.1 | | | |
| | 140 × 191 | 202 | 172 | | 160 | | 153 | 131 | | 121 | |
| | 140 × 241 | 217 | 192 | 166 | 181 | | 154 | 136 | 118 | 128 | |

Notes:
1. $T_r$ = Factored tensile resistance based on gross area
   $T_{rN}$ = Factored tensile resistance based on net area with one or two shear plates or split ring on each side of member at a given cross section:



One row



Two rows

2. SH PL = Shear Plate, SR = Split Ring
3. Where $T_r$ values are not shown, area removed exceeds 25% of gross area.
4. For 4” diameter shear plates, values are based on the use of 3/4” diameter bolts.

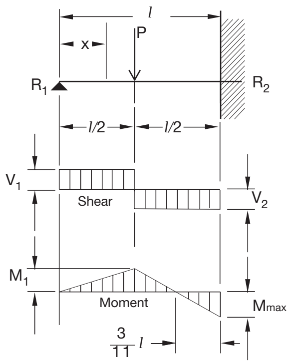
June 2012 Errata



# Beam Diagrams and Formulae*

## Beam Fixed at one end, Supported at Other

19.
Concentrated load at centre




$$R_1 = V_1 \dots = \frac{5P}{16}$$

$$R_2 = V_2 \text{ max.} \dots = \frac{11P}{16}$$

$$M \text{ max. (at fixed end)} \dots = \frac{3Pl}{16}$$

$$M_1 \text{ (at point of load)} \dots = \frac{5Pl}{32}$$

$$M_x \left(\text{when } x < \frac{l}{2}\right) \dots = \frac{5Px}{16}$$

$$M_x \left(\text{when } x > \frac{l}{2}\right) \dots = P\left(\frac{l}{2} - \frac{11x}{16}\right)$$

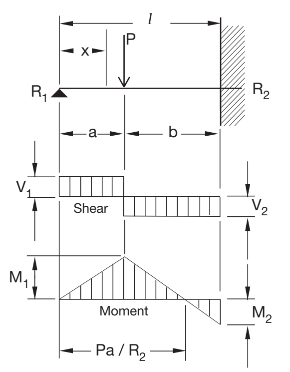
$$\Delta \text{ max. (at } x = l\sqrt{\frac{1}{5}} = 0.4472l\text{)} \dots = \frac{Pl^3}{48EI\sqrt{5}} = 0.009317\frac{Pl^3}{EI}$$

$$\Delta_x \text{ (at point of load)} \dots = \frac{7Pl^3}{768EI}$$

$$\Delta_x \left(\text{when } x < \frac{l}{2}\right) \dots = \frac{Px}{96EI}(3l^2 - 5x^2)$$

$$\Delta_x \left(\text{when } x > \frac{l}{2}\right) \dots = \frac{P}{96EI}(x - l)^2(11x - 2l)$$

20.
Concentrated load at any point




$$R_1 = V_1 \dots = \frac{Pb^2}{2l^3}(a + 2l)$$

$$R_2 = V_2 \dots = \frac{Pa}{2l^3}(3l^2 - a^2)$$

$$M_1 \text{ (at point of load)} \dots = R_1 a$$

$$M_2 \text{ (at fixed end)} \dots = \frac{Pab}{2l^2}(a + l)$$

$$M_x \text{ (when } x < a\text{)} \dots = R_1 x$$

$$M_x \text{ (when } x > a\text{)} \dots = R_1 x - P(x - a)$$

$$\Delta \text{ max. (} a < 0.414l \text{ at } x = l\frac{l^2 + a^2}{3l^2 - a^2}\text{)} = \frac{Pa}{3EI}\frac{(l^2 - a^2)^3}{(3l^2 - a^2)^2}$$

$$\Delta \text{ max. (} a > 0.414l \text{ at } x = l\sqrt{\frac{a}{2l + a}}\text{)} = \frac{Pab^2}{6EI}\sqrt{\frac{a}{2l + a}}$$

$$\Delta_a \text{ (at point of load)} \dots = \frac{Pa^2b^3}{12EIl^3}(3l + a)$$

$$\Delta_x \text{ (when } x < a\text{)} \dots = \frac{Pb^2x}{12EIl^3}(3al^2 - 2lx^2 - ax^2)$$

$$\Delta_x \text{ (when } x > a\text{)} \dots = \frac{Pa}{12EIl^3}(l - x)^2(3l^2x - a^2x - 2a^2l)$$



Note:
* By kind permission of the American Institute of Steel Construction.

# Decking Selection Tables

## Select Grade

Sel

### $W_{FR}$ Maximum factored uniform load $W_{FR}$ (kPa)*

| Span m | D.Fir-L Thickness, mm 38[1] | 64 | 89 | Hem-Fir Thickness, mm 38[1] | 64 | 89 | S-P-F Thickness, mm 38[1] | 64 | 89 | Northern Thickness, mm 38[1] | 64 | 89 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 39.5 | | | 38.3 | | | 39.5 | | | 25.4 | | |
| 1.2 | 27.4 | | | 26.6 | | | 27.4 | | | 17.6 | | |
| 1.4 | 20.2 | | | 19.6 | | | 20.2 | | | 13.0 | | |
| 1.6 | 15.4 | | | 15.0 | | | 15.4 | | | 9.92 | 31.3 | |
| 1.8 | 12.2 | | | 11.8 | | | 12.2 | 38.5 | | 7.84 | 24.8 | |
| 2.0 | 9.88 | 31.2 | | 9.58 | 30.3 | | 9.88 | 31.2 | | 6.35 | 20.1 | |
| 2.2 | 8.16 | 25.8 | | 7.92 | 25.0 | | 8.16 | 25.8 | | 5.25 | 16.6 | |
| 2.4 | 6.86 | 21.7 | | 6.65 | 21.0 | | 6.86 | 21.7 | | 4.41 | 13.9 | 28.9 |
| 2.6 | 5.85 | 18.5 | | 5.67 | 17.9 | | 5.85 | 18.5 | 38.3 | | 11.9 | 24.6 |
| 2.8 | 5.04 | 15.9 | 33.0 | 4.89 | 15.4 | 32.0 | 5.04 | 15.9 | 33.0 | | 10.2 | 21.2 |
| 3.0 | 4.39 | 13.9 | 28.8 | | 13.5 | 27.9 | | 13.9 | 28.8 | | 8.92 | 18.5 |
| 3.2 | | 12.2 | 25.3 | | 11.8 | 24.5 | | 12.2 | 25.3 | | 7.84 | 16.2 |
| 3.4 | | 10.8 | 22.4 | | 10.5 | 21.7 | | 10.8 | 22.4 | | 6.94 | 14.4 |
| 3.6 | | 9.64 | 20.0 | | 9.34 | 19.4 | | 9.64 | 20.0 | | 6.19 | 12.8 |
| 3.8 | | 8.65 | 17.9 | | 8.39 | 17.4 | | 8.65 | 17.9 | | 5.56 | 11.5 |
| 4.0 | | 7.81 | 16.2 | | 7.57 | 15.7 | | | 16.2 | | | 10.4 |
| 4.2 | | | 14.7 | | | 14.2 | | | 14.7 | | | 9.42 |
| 4.4 | | | 13.4 | | | 13.0 | | | 13.4 | | | 8.59 |
| 4.6 | | | 12.2 | | | 11.9 | | | 12.2 | | | 7.86 |
| 4.8 | | | 11.2 | | | 10.9 | | | 11.2 | | | 7.22 |
| 5.0 | | | 10.4 | | | 10.0 | | | 10.4 | | | 6.65 |
| 5.2 | | | 9.57 | | | 9.28 | | | 9.57 | | | 6.15 |
| 5.4 | | | 8.87 | | | 8.61 | | | 8.87 | | | 5.70 |
| 5.6 | | | 8.25 | | | 8.00 | | | 8.25 | | | 5.30 |
| 5.8 | | | 7.69 | | | 7.46 | | | 7.69 | | | 4.94 |

### $W_{\Delta R}$ Maximum specified uniform load for L/240 deflection $W_{\Delta R}$ (kPa)

| Span m | D.Fir-L 38[1] | 64 | 89 | Hem-Fir 38[1] | 64 | 89 | S-P-F 38[1] | 64 | 89 | Northern 38[1] | 64 | 89 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 20.2 | | | 19.4 | | | 17.0 | | | 12.1 | | |
| 1.2 | 11.7 | | | 11.2 | | | 9.81 | | | 7.01 | | |
| 1.4 | 7.35 | | | 7.06 | | | 6.18 | | | 4.41 | | |
| 1.6 | 4.93 | | | 4.73 | | | 4.14 | | | 2.96 | 16.6 | |
| 1.8 | 3.46 | | | 3.32 | | | 2.91 | 16.3 | | 2.08 | 11.7 | |
| 2.0 | 2.52 | 14.2 | | 2.42 | 13.6 | | 2.12 | 11.9 | | 1.51 | 8.50 | |
| 2.2 | 1.90 | 10.6 | | 1.82 | 10.2 | | 1.59 | 8.94 | | 1.14 | 6.39 | |
| 2.4 | 1.46 | 8.20 | | 1.40 | 7.87 | | 1.23 | 6.89 | | 0.88 | 4.92 | 13.2 |
| 2.6 | 1.15 | 6.45 | | 1.10 | 6.19 | | 0.96 | 5.42 | 14.6 | | 3.87 | 10.4 |
| 2.8 | 0.92 | 5.16 | 13.9 | 0.88 | 4.96 | 13.3 | 0.77 | 4.34 | 11.7 | | 3.10 | 8.33 |
| 3.0 | 0.75 | 4.20 | 11.3 | | 4.03 | 10.8 | | 3.53 | 9.49 | | 2.52 | 6.78 |
| 3.2 | | 3.46 | 9.30 | | 3.32 | 8.93 | | 2.91 | 7.82 | | 2.08 | 5.58 |
| 3.4 | | 2.88 | 7.76 | | 2.77 | 7.45 | | 2.42 | 6.52 | | 1.73 | 4.65 |
| 3.6 | | 2.43 | 6.54 | | 2.33 | 6.27 | | 2.04 | 5.49 | | 1.46 | 3.92 |
| 3.8 | | 2.07 | 5.56 | | 1.98 | 5.33 | | 1.74 | 4.67 | | 1.24 | 3.33 |
| 4.0 | | 1.77 | 4.76 | | 1.70 | 4.57 | | | 4.00 | | | 2.86 |
| 4.2 | | | 4.12 | | | 3.95 | | | 3.46 | | | 2.47 |
| 4.4 | | | 3.58 | | | 3.44 | | | 3.01 | | | 2.15 |
| 4.6 | | | 3.13 | | | 3.01 | | | 2.63 | | | 1.88 |
| 4.8 | | | 2.76 | | | 2.65 | | | 2.32 | | | 1.65 |
| 5.0 | | | 2.44 | | | 2.34 | | | 2.05 | | | 1.46 |
| 5.2 | | | 2.17 | | | 2.08 | | | 1.82 | | | 1.30 |
| 5.4 | | | 1.94 | | | 1.86 | | | 1.63 | | | 1.16 |
| 5.6 | | | 1.74 | | | 1.67 | | | 1.46 | | | 1.04 |
| 5.8 | | | 1.56 | | | 1.50 | | | 1.31 | | | 0.94 |

[1] Thinner decking may result from remanufacturing. Loads are based on 36 mm thick decking and may be increased when the actual decking thickness is 38 mm.

* Where decking is used to support roof loads, the maximum spans for Northern Species decking may be limited by the *NBCC* roof point load requirements. The maximum calculated span, based on NBC roof point load requirements, for 36 mm Northern Species decking is 2.2 m. The span may be increased to 2.4 m if 38 mm thick decking is used.

November 2011 Errata



# Decking Selection Tables

## Com Commercial Grade

### $W_{FR}$ Maximum factored uniform load $W_{FR}$ (kPa)*

| Span m | D.Fir-L Thickness, mm 38[1] | 64 | 89 | Hem-Fir Thickness, mm 38[1] | 64 | 89 | S-P-F Thickness, mm 38[1] | 64 | 89 | Northern Thickness, mm 38[1] | 64 | 89 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 24.0 | | | 26.3 | | | 28.3 | | | 18.2 | | |
| 1.2 | 16.6 | | | 18.3 | | | 19.6 | | | 12.6 | | |
| 1.4 | 12.2 | | | 13.4 | | | 14.4 | | | 9.29 | | |
| 1.6 | 9.36 | | | 10.3 | | | 11.0 | | | 7.11 | 22.5 | |
| 1.8 | 7.39 | | | 8.13 | | | 8.72 | 27.6 | | 5.62 | 17.8 | |
| 2.0 | 5.99 | 18.9 | | 6.59 | 20.8 | | 7.07 | 22.3 | | 4.55 | 14.4 | |
| 2.2 | 4.95 | 15.6 | | 5.44 | 17.2 | | 5.84 | 18.5 | | 3.76 | 11.9 | |
| 2.4 | 4.16 | 13.1 | | 4.57 | 14.5 | | 4.91 | 15.5 | | 3.16 | 9.99 | 20.7 |
| 2.6 | 3.54 | 11.2 | | 3.90 | 12.3 | | 4.18 | 13.2 | 27.4 | | 8.51 | 17.6 |
| 2.8 | 3.05 | 9.65 | 20.0 | 3.36 | 10.6 | 22.0 | | 11.4 | 23.6 | | 7.34 | 15.2 |
| 3.0 | | 8.41 | 17.4 | | 9.25 | 19.2 | | 9.92 | 20.6 | | 6.39 | 13.2 |
| 3.2 | | 7.39 | 15.3 | | 8.13 | 16.8 | | 8.72 | 18.1 | | 5.62 | 11.6 |
| 3.4 | | 6.55 | 13.6 | | 7.20 | 14.9 | | 7.73 | 16.0 | | 4.98 | 10.3 |
| 3.6 | | 5.84 | 12.1 | | 6.42 | 13.3 | | 6.89 | 14.3 | | 4.44 | 9.20 |
| 3.8 | | 5.24 | 10.9 | | 5.77 | 11.9 | | 6.19 | 12.8 | | 3.98 | 8.25 |
| 4.0 | | 4.73 | 9.80 | | 5.20 | 10.8 | | 5.58 | 11.6 | | 3.60 | 7.45 |
| 4.2 | | | 8.89 | | | 9.78 | | | 10.5 | | | 6.76 |
| 4.4 | | | 8.10 | | | 8.91 | | | 9.56 | | | 6.16 |
| 4.6 | | | 7.41 | | | 8.15 | | | 8.75 | | | 5.63 |
| 4.8 | | | 6.81 | | | 7.49 | | | 8.03 | | | 5.17 |
| 5.0 | | | 6.27 | | | 6.90 | | | 7.40 | | | 4.77 |
| 5.2 | | | 5.80 | | | 6.38 | | | 6.84 | | | 4.41 |
| 5.4 | | | 5.38 | | | 5.92 | | | 6.35 | | | 4.09 |
| 5.6 | | | 5.00 | | | 5.50 | | | 5.90 | | | 3.80 |
| 5.8 | | | 4.66 | | | 5.13 | | | 5.50 | | | 3.54 |

### $W_{\Delta R}$ Maximum specified uniform load for L/240 deflection $W_{\Delta R}$ (kPa)

| Span m | D.Fir-L 38[1] | 64 | 89 | Hem-Fir 38[1] | 64 | 89 | S-P-F 38[1] | 64 | 89 | Northern 38[1] | 64 | 89 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 17.8 | | | 17.8 | | | 15.3 | | | 11.3 | | |
| 1.2 | 10.3 | | | 10.3 | | | 8.87 | | | 6.54 | | |
| 1.4 | 6.47 | | | 6.47 | | | 5.59 | | | 4.12 | | |
| 1.6 | 4.34 | | | 4.34 | | | 3.74 | | | 2.76 | 15.5 | |
| 1.8 | 3.04 | | | 3.04 | | | 2.63 | 14.8 | | 1.94 | 10.9 | |
| 2.0 | 2.22 | 12.5 | | 2.22 | 12.5 | | 1.92 | 10.8 | | 1.41 | 7.94 | |
| 2.2 | 1.67 | 9.37 | | 1.67 | 9.37 | | 1.44 | 8.09 | | 1.06 | 5.96 | |
| 2.4 | 1.28 | 7.22 | | 1.28 | 7.22 | | 1.11 | 6.23 | | 0.82 | 4.59 | 12.4 |
| 2.6 | 1.01 | 5.68 | | 1.01 | 5.68 | | 0.87 | 4.90 | 13.2 | | 3.61 | 9.71 |
| 2.8 | 0.81 | 4.55 | 12.2 | 0.81 | 4.55 | 12.2 | | 3.93 | 10.6 | | 2.89 | 7.78 |
| 3.0 | | 3.70 | 9.94 | | 3.70 | 9.94 | | 3.19 | 8.58 | | 2.35 | 6.32 |
| 3.2 | | 3.04 | 8.19 | | 3.04 | 8.19 | | 2.63 | 7.07 | | 1.94 | 5.21 |
| 3.4 | | 2.54 | 6.83 | | 2.54 | 6.83 | | 2.19 | 5.90 | | 1.62 | 4.34 |
| 3.6 | | 2.14 | 5.75 | | 2.14 | 5.75 | | 1.85 | 4.97 | | 1.36 | 3.66 |
| 3.8 | | 1.82 | 4.89 | | 1.82 | 4.89 | | 1.57 | 4.22 | | 1.16 | 3.11 |
| 4.0 | | 1.56 | 4.19 | | 1.56 | 4.19 | | 1.35 | 3.62 | | 0.99 | 2.67 |
| 4.2 | | | 3.62 | | | 3.62 | | | 3.13 | | | 2.30 |
| 4.4 | | | 3.15 | | | 3.15 | | | 2.72 | | | 2.00 |
| 4.6 | | | 2.76 | | | 2.76 | | | 2.38 | | | 1.75 |
| 4.8 | | | 2.43 | | | 2.43 | | | 2.10 | | | 1.54 |
| 5.0 | | | 2.15 | | | 2.15 | | | 1.85 | | | 1.37 |
| 5.2 | | | 1.91 | | | 1.91 | | | 1.65 | | | 1.21 |
| 5.4 | | | 1.70 | | | 1.70 | | | 1.47 | | | 1.08 |
| 5.6 | | | 1.53 | | | 1.53 | | | 1.32 | | | 0.97 |
| 5.8 | | | 1.38 | | | 1.38 | | | 1.19 | | | 0.88 |

[1] Thinner decking may result from remanufacturing. Loads are based on 36 mm thick decking and may be increased when the actual decking thickness is 38 mm.

* Where decking is used to support roof loads, the maximum spans for decking may be limited by the NBCC roof point load requirements. Maximum calculated spans for 36 mm decking, based on NBCC roof point load requirements, are 2.0 m for D.Fir-L, 2.2 m for Hem-Fir, 2.4 m for S-P-F and 1.6 m for Northern Species. Maximum calculated spans for 38 mm thick decking are 2.2 m for D.Fir-L, 2.4 m for Hem-Fir, 2.6 m for S-P-F and 1.8 m for Northern Species.

Where the column is short, or where sheathing prevents buckling about the weak axis of the plies, $P_r$ may calculated as the combined factored resistance of the individual pieces taken as independent members. This will provide a larger factored resistance than calculated from the formula given above for these cases. The resistance of an individual piece may be calculated from the design formula given in Section 3.1.

**Example 1: Built-up Columns**

Design a built-up column for the following conditions:

- specified dead load = 1.0 kPa
- specified snow load = 2.4 kPa
- tributary area = 9 $m^2$
- unsupported length = 3 m

- dry service conditions
- untreated
- column effectively pinned at both ends

Use Stud grade S-P-F.

**Calculation**

Total factored load:

$$P_f = (1.25 \times 1.0 + 1.5 \times 2.4) \times 9$$
$$= 43.6 \text{ kN}$$

From Built-up Column Selection Tables select 4-ply 38 × 140 mm:

$P_r$ = 75.2 kN > 43.6 kN *Acceptable*

**Use 4-ply 38 × 140 mm Stud grade S-P-F built-up columns nailed together as shown in Figure 3.2.**

where

$P_f$ = factored axial load

e = load eccentricity

= distance between the centre of the column and the centroid of the applied load

Midway between the location where the load is applied and the column support (typically mid-height of the column) the interaction equation takes the following form when the column is designed with pinned end conditions ($K_e$=1.0).

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{1}{2}\frac{P_f e}{M_r}\left[\frac{1}{1 - \frac{P_f}{P_E}}\right] \leq 1$$



### Lateral Restraint

In the calculation of moment resistance for combined bending and compression, the lateral stability factor $K_L$ may be taken as unity where lateral support is provided at points of bearing and the depth to width ratio of the member does not exceed the following values:

| | |
|---|---|
| 4:1 | if no additional intermediate support is provided |
| 5:1 | if the member is held in line by purlins or tie rods |
| 6.5:1 | if the compressive edge is held in line by direct connection of decking or joists spaced not more than 610 mm apart |
| 7.5:1 | if the compressive edge is held in line by direct connection of decking or joists spaced not more than 610 mm apart and adequate bridging or blocking is installed at intervals not exceeding 8 times the depth of the member |
| 9:1 | if both edges are held in line |

These rules apply to rectangular glulam members subjected to combined bending and compression as well as sawn lumber and timber. If the above limits are exceeded, $K_L$ may be calculated in accordance with CSA O86 Clause 6.5.6.4.

November 2011 Errata



**Net Area**

Members that have been drilled or bored must be designed using the net cross-sectional area. The net area $A_N$ is calculated as follows:

$$A_N = A_G - A_R$$

where:

$A_G$ = gross cross-sectional area

$A_R$ = area removed due to drilling, boring, grooving or other means; $A_R$ must not exceed 0.25 $A_G$

Only the following fasteners are considered to cause an area reduction:

- split rings
- shear plates
- bolts
- lag screws
- drift pins

The area reduction due to bolt, lag screw or drift pin holes is equal to the diameter of the hole multiplied by the thickness of the member. Table 7.2 contains area reduction information for shear plates and split rings.

Where staggered rows of fasteners are used, adjacent fasteners must be considered to occur at the same cross-sectional plane when the centre-to-centre spacing along the grain is less than the following values:

2 × fastener diameter for split rings and shear plates

8 × fastener diameter for bolts, lag screws or drift pins

Table 7.2
**Area removed for shear plate and split ring joints**
**$A_R$ (mm$^2$ × $10^3$)**

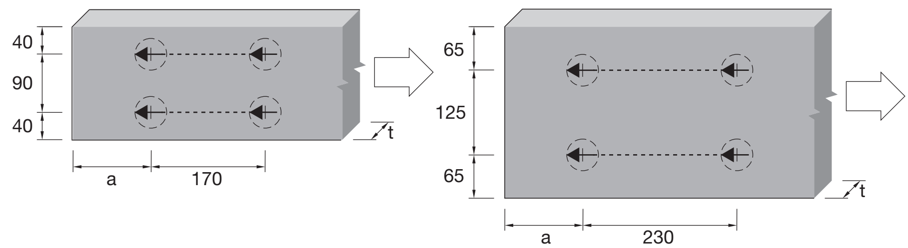
| Diameter in. Connector | Bolt | Number of faces with connector | Member thickness mm 38 | 64 | 80 | 89 | 130 | 140 | 175 | 191 | 215 | 241 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Split rings** | | | | | | | | | | | | |
| 2-1/2 | 1/2 | 1 | 1.11 | 1.48 | 1.71 | 1.84 | 2.43 | 2.57 | 3.07 | 3.30 | 3.65 | 4.02 |
| | | 2 | 1.68 | 2.05 | 2.28 | 2.41 | 3.00 | 3.14 | 3.64 | 3.87 | 4.22 | 4.58 |
| 4 | 3/4 | 1 | 1.97 | 2.51 | 2.84 | 3.02 | 3.87 | 4.07 | 4.79 | 5.12 | 5.61 | 6.15 |
| | | 2 | 3.16 | 3.70 | 4.02 | 4.21 | 5.06 | 5.26 | 5.98 | 6.31 | 6.80 | 7.34 |
| **Shear plates** | | | | | | | | | | | | |
| 2-5/8 | 3/4 | 1 | 1.31 | 1.85 | 2.18 | 2.36 | 3.20 | 3.41 | 4.13 | 4.46 | 4.95 | 5.49 |
| | | 2 | 1.84 | 2.37 | 2.70 | 2.89 | 3.73 | 3.94 | 4.66 | 4.99 | 5.48 | 6.02 |
| 4 | 3/4 | 1 | 2.12 | 2.65 | 2.98 | 3.17 | 4.01 | 4.22 | 4.94 | 5.27 | 5.76 | 6.30 |
| | | 2 | - | 3.99 | 4.32 | 4.50 | 5.34 | 5.55 | 6.27 | 6.60 | 7.09 | 7.63 |
| | 7/8 | 1 | 2.19 | 2.80 | 3.18 | 3.40 | 4.38 | 4.61 | 5.45 | 5.83 | 6.40 | 7.02 |
| | | 2 | - | 4.09 | 4.47 | 4.68 | 5.66 | 5.89 | 6.73 | 7.11 | 7.68 | 8.30 |

Note:
Net area for split rings and shear plates has been included in the Tension Member Selection Tables.

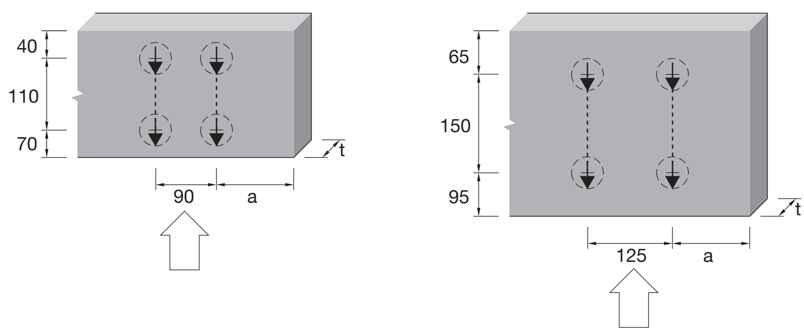
Figure 7.16
**Minimum configuration (for $J_C = 1.0$) and minimum thickness (for $J_T = 1.0$) for split rings and shear plates**

| **2-1/2" split rings**<br>**2-5/8" shear plates** | **4" split rings**<br>**4" shear plates** |
|---|---|

Load parallel to grain




Load perpendicular to grain




t = minimum thickness (mm)
a = end distance for tension members (mm)

| | |
|---|---|
| a = 210<br>when t < 130 | a = 270<br>when t < 130 |
| a = 140<br>when t ≥ 130 | a = 180<br>when t ≥ 130 |
| t = 51 for split rings on two faces<br>38 for split rings on one face<br>64 for shear plates on one or two faces | t = 76 for split rings on two faces<br>38 for split rings on one face<br>89 for shear plates on two faces<br>44 for shear plates on one face |

Notes:
1. All dimensions are in mm.
2. Reduced end, edge and spacing dimensions are possible; see Tables 7.22, 7.23, 7.24 and 7.25.

## Sawn Timber

$P_r$ is the lesser of:

$$P_{rd} = \phi F_c A K_{Zcd} K_{cd} \text{ or } P_{rb} = \phi F_c A K_{Zcb} K_{cb}$$

where:

$\phi F_c$ = factored compressive resistance strength (MPa) given in Table 3.6

$K_{Zc}$ = size factor

$K_{Zcd} = 6.3 (dL_d)^{-0.13} \leq 1.3$ for buckling in direction of d

$K_{Zcb} = 6.3 (bL_b)^{-0.13} \leq 1.3$ for buckling in direction of b

$K_C$ = slenderness factor

$K_{Cd} = \left[1.0 + \frac{F_C}{E'} K_{Zcd} C_{cd}^3\right]^{-1}$ for buckling in direction of d

$K_{Cb} = \left[1.0 + \frac{F_C}{E'} K_{Zcd} C_{cd}^3\right]^{-1}$ for buckling in direction of b

$F_c/E'$ = strength to stiffness ratio given in Table 3.7

$C_{cd} = \frac{K_e L_d}{d}$ $C_{cb} = \frac{K_e L_b}{b}$ ($C_{cd}$ or $C_{cd}$ >50 is not permitted)

$K_e$ = effective length factor, given in Figure 3.1

$L_b$, $L_d$ = unsupported length associated with d or b (mm)

d = depth of member (mm)

b = thickness of member (mm)

## Glulam

$$P_r = \phi F_c A K_{Zcg} K_C$$

where:

$\phi F_c$ = factored compressive resistance strength (MPa) given in Table 3.8

$K_{Zcg} = 0.68 (Z)^{-0.13} \leq 1.0$, where Z=member volume in $m^3$

$K_C$ = slenderness factor

$= \left[1.0 + \frac{F_c}{E'} K_{Zcg} C_c^3\right]^{-1}$

$F_c/E'$ = strength to stiffness ratio given in Table 3.9

$C_c$ = the greater of $\frac{K_e L_d}{d}$ or $\frac{K_e L_b}{b}$ ($C_c$>50 is not permitted)

$K_e$ = effective length factor, given in Figure 3.1

$L_b$, $L_d$ = unsupported length associated with d or b (mm)

d = depth of member (mm)

b = thickness of member (mm)

**July 2011 Errata**



Table 3.8
**Factored compressive strength for glulam**
**$\phi F_c$ (MPa)**

| Service conditions | Dry service | | | Wet service | | |
|---|---|---|---|---|---|---|
| Load duration[2] | Std. | Perm. | Short term | Std. | Perm. | Short term |
| Compression grades | | | | | | |
| D.Fir-L 16c-E | 24.2 | 15.7 | 27.8 | 18.1 | 11.8 | 20.8 |
| Spruce-Pine 12c-E | 20.2 | 13.1 | 23.2 | 15.1 | 9.83 | 17.4 |

Notes:
1. For fire-retardant treated material, multiply by a treatment factor $K_T$. See the Commentary for further guidance.
2. Standard term loading = dead loads plus snow or occupancy loads
   Permanent loading = dead loads alone
   Short term loading = dead plus wind loads
3. $\phi F_c = \phi f_c K_D K_H K_{Sc} K_T$ (refer to Clause 6.5.8 of CSA O86).

Table 3.9
**Modified strength to stiffness ratio for glulam**
**$F_c / E'$ ($\times 10^{-6}$)**

| Service conditions | Dry service | | | Wet service | | |
|---|---|---|---|---|---|---|
| Load duration[1] | Std. | Perm. | Short term | Std. | Perm. | Short term |
| Compression grades | | | | | | |
| D.Fir-L 16c-E | 80.0 | 52.0 | 92.0 | 66.7 | 43.3 | 76.7 |
| Spruce-Pine 12c-E | 85.3 | 55.5 | 98.1 | 71.1 | 46.2 | 81.8 |

Notes:
1. Standard term loading = dead loads plus snow or occupancy loads
   Permanent loading = dead loads alone
   Short term loading = dead plus wind loads
2. $F_c / E' = (f_c K_D K_H K_{Sc} K_T) / (35 E_{05} K_{SE} K_T)$ (refer to Clause 6.5.8 of CSA O86).

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## Modification Factors

The composite modification factors $K'$ and $J_F$ adjust the basic factored lateral resistance for the actual conditions of use and are calculated as follows:

$$K' = K_D K_{SF} K_T$$

$$J_F = J_E J_A J_B J_D$$

where:

$K_D$ = duration of load factor

$K_{SF}$ = service condition factor

$K_T$ = fire-retardant treatment factor

$J_E$ = factor for nailing into end grain

$J_A$ = factor for toe nailing

$J_B$ = factor for nail clinching

$J_D$ = factor for diaphragm construction

In many cases $K'$ and $J_F$ will equal 1.0. Review the following checklist to determine if each factor is equal to 1.0. Where a factor does not equal 1.0 determine the appropriate value and calculate $K'$ and $J_F$.

## Nail Penetration

Minimum required penetration for single shear connections are:

- For the side member with the nail head
  - 3 nail diameters for wood members
  - 0.9mm for steel members
- 5 nail diameters for the side member with the nail point.

Minimum required nail penetration for double shear connections are

- 3 nail diameters for the member with the nail head
- 8 nail diameters for the centre member
- 5 nail diameters for the member with the point

**Calculation**

Try 2-1/2” common wire nails driven from alternate sides of the main member. For double shear connections penetration into the point side member must be 5 nail diameters (15mm) or greater. The connection cannot be considered double shear.

From the Nail Selection Tables:

Minimum penetration into the main member is 16 mm and corresponding $N'_r\, n_s$ is 0.395 kN

Maximum penetration into the main member is 51 mm and corresponding $N'_r\, n_s$ is 0.544 kN

Actual penetration is 38 mm

$N'_r\, n_s$ based on actual penetration of 38 mm can be calculated by interpolation.

For a 38 mm penetration in the main member $N'_r\, n_s$ is equal to 0.489 kN.

$N_r = N'_r\, n_s\, n_F\, K'\, J_F$

$N_r = 0.489 \times n_F \times 1 \times 1 = 0.489\, n_F$ kN

$n_F$ required = 8/0.489 = 16.4

Therefore, use seventeen 2-1/2” nails per side.

Determine minimum spacing, end distance and edge distance from Figure 7.4:

minimum spacing perpendicular to grain c = 26 mm

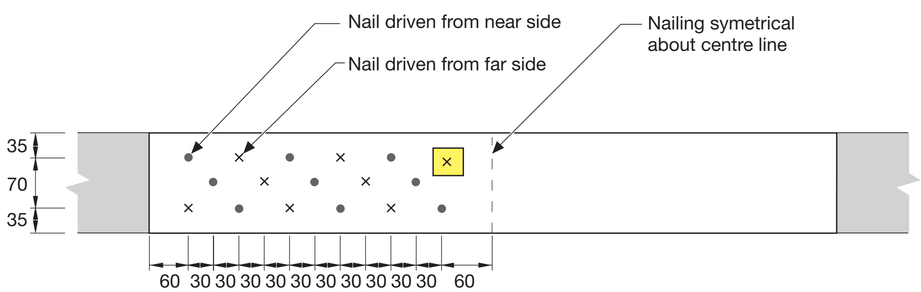
minimum edge distance d = 13 mm

Therefore, use two rows of nails spaced at 70 mm, with 35 mm edge distance:

minimum spacing parallel to grain a = 52 mm, use 60 mm

minimum end distance b = 39 mm, use 60 mm

The final connection geometry is shown below (dimensions in mm). Note that the tensile resistance of the plywood and lumber members should also be checked.




**Use seventeen 2-1/2” common nails.**

## Modification Factors

The composite modification factors K′ and $J_F$ adjust the basic factored lateral resistance for the actual condition of use and are calculated as follows:

$$K' = K_D\, K_{SF}\, K_T$$

$$J_F = J_E\, J_A$$

where:

$K_D$ = duration of load factor

$K_{SF}$ = service condition factor

$K_T$ = fire-retardant treatment factor

$J_E$ = factor for fastening into end grain

$J_A$ = factor for toe-screwing

In many cases K′ and $J_F$ will equal 1.0. Review the following checklist to determine if each factor is equal to 1.0. Where a factor does not equal 1.0 determine the appropriate value and calculate K′ and $J_F$.

Screws may be designed for withdrawal under wind or earthquake loads only. Design requirements for withdrawal are given in CSA O86 Clause 10.11.5. The factored withdrawal resistance is the lesser of the factored withdrawal resistance of the main member, $P_{rw'}$ or the head pull-through resistance of the side member, $P_{pt}$. Withdrawal resistance of the main member is given in Table 7.4 and head pull-through resistance of the side member is given in Table 7.5.

Minimum required penetration for single shear connections are:

- The side member with the screw head:
  - 3 screw diameters for wood members
  - 0.9mm for steel members

Minimum required penetration for double shear connections are:

- 3 screw diameters for the side members with the screw head
- 8 screw diameters for the centre member
- 5 screw diameters for the side member with the screw point.

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### Calculation

Try 6 gauge 2” screws driven from alternate sides of the main member. For double shear connections, penetration into the point side member must be 5 screw diameters (17.5mm) or greater. The connection cannot be considered double shear.

From the Screw Selection Tables:

Minimum penetration into the main member is 18 mm and corresponding $N'_r\ n_s$ is 0.373 kN

Maximum penetration into the main member is 40 mm and corresponding $N'_r\ n_s$ is 0.546 kN

Actual penetration is 38 mm.

$N'_r\ n_s$ based on an actual penetration of 38 mm can be calculated by interpolation.
For a 38 mm penetration in the main member $N'_r\ n_s$ is equal to 0.530 kN.

$N_r = N'_r\ n_s\ n_F\ K'\ J_F$

$N_r = 0.530 \times n_F \times 1 \times 1 = 0.530\ n_F$ kN

$n_F$ required = 8/0.530 = 14.1

Therefore use sixteen 2” screws per side

Determine minimum spacing, end distance and edge distance from Figure 7.5:

Minimum spacing perpendicular to grain c = 28 mm

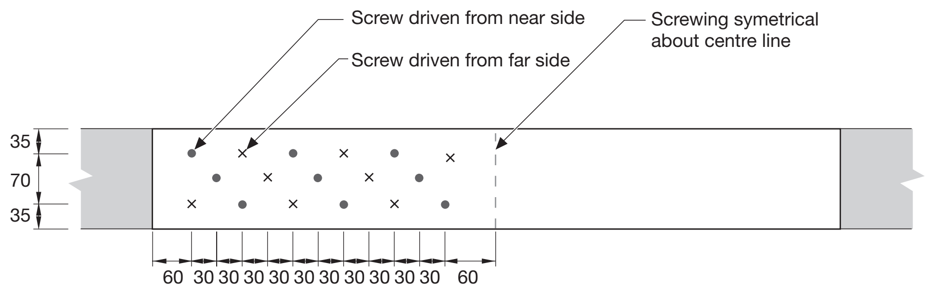
Minimum edge distance d = 14 mm

Therefore, use two rows of screws spaced at 70 mm, with 35 mm edge distance:

Minimum spacing parallel to grain a = 56 mm, use 60 mm

Minimum end distance is 42 mm, use 60 mm

The final connection geometry is shown below (dimensions in mm). Note that the tensile resistance of the OSB and lumber members should also be checked.




**Use sixteen 6 gauge 2” screws.**

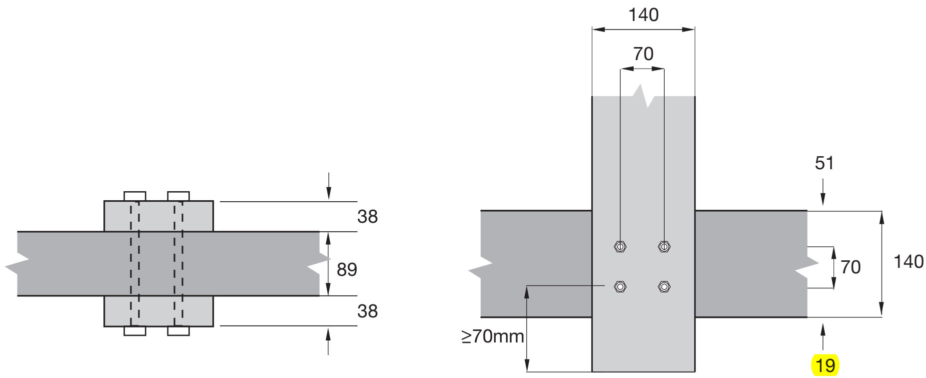
7
Fastenings

July 2011 Errata



## Example 2: Bolts

Determine if two rows of 1/2" bolts are adequate for the attachment of the beam to the wooden tension member. The wood tension member consists of two 38 x 140 mm D.Fir-L No. 2 grade sawn lumber members. The beam is 89 x 140 mm D.Fir-L No. 2 grade sawn lumber. The total factored tension force is 20 kN.




**Modification Factors**

$K_D$ = 1.0 (loading is standard term)

$K_{SF}$ = 1.0 (service conditions are dry, material is seasoned)

$K_T$ = 1.0 (wood members are not treated)

Therefore:

$K'$ = 1.0

**Calculation**

The configuration is Type 5 – double shear with wood side plates loaded parallel to the grain and the main member loaded perpendicular to grain.

Try two rows of two ½" diameter bolts:

$P_r$ is equal to the lesser of row shear resistance, $PR_{rT}$, group tear-out resistance $PG_{rT}$, and tension resistance parallel to grain.

$Q_r$ is equal to perpendicular to grain splitting resistance $QS_{rT}$.

The maximum bolt spacing parallel to grain is 140 – 19 – 51 = 70 mm.

For an efficient connection, the loaded end distance should be the same as the bolt spacing parallel to grain. The spacing between rows of bolts is 70 mm.

The factored shear resistance of panels on both sides of the same shearwall may be added together. Where wood structural panel are applied over 12.7 mm or 15.9 mm thick gypsum wallboard, the shear resistance of the wood-based panel may be considered in the shearwall design, provided minimum nail penetration requirements are met. In all other cases with multiple layers of panels on the same side of the shearwall, only the shear resistance of the panel closest to the studs is considered in design.

**Shearwalls With Segments**

Where shearwalls have openings for windows or doors, only the full height shearwall segments between the openings and at the ends of the shearwalls are considered in the shearwall design. In the case of blocked shearwalls where the height of the shearwall segment, $H_s$, measured from the bottom of the bottom plate to the top of the top plate, is greater than 3.5 times the length of the shearwall segment, $L_s$, the segment is not included in the resistance calculation. For unblocked shearwalls, where $H_s$ is greater than $2L_s$, the segment is not included in the resistance calculation.

A shearwall is designed to resist the sum of factored shear forces from upper storey shearwalls and diaphragms. Where materials or construction details vary between shearwall segments, the capacity of the shearwall may either be conservatively estimated based on the minimum segment capacity. Alternatively the total factored shear force on the shearwall, $V_{fs}$, may be distributed to each of the segments. Different methods are used to distribute the shear force on a shearline to each of the shearwall segments. The distribution of lateral forces to the shearwall segments may be based on the relative strength or relative stiffness of the segments. See the information on Lateral Force Distribution to Shearwall Segments later in this section.

The shearwall sheathing and the connection of the sheathing to the framing members must be designed so that the factored shear resistance of the shearwall segment per unit length, $v_r$, is equal to or greater than the maximum shear force in the shearwall segment per unit length, $v_{fsi}$. The maximum shear force is assumed to be uniformly distributed the full length of the shearwall segment parallel to the applied load, $L_s$, and $v_{fsi} = \dfrac{V_{fsi}}{L_s}$ .

Capacity of shearwalls may vary with load direction and shearwalls must be designed for lateral loads acting in opposite directions. The resistance on the shearwall in each direction must be greater than the corresponding lateral load.

**Shearwall Segments Without Hold-downs**

Traditionally, shearwalls have been designed using chords and hold-down connections at the ends of all shearwall segments. Hold-downs are designed to transfer the chord segment overturning force, $T_f$, to the shearwall or foundation below. CSA O86 includes provisions for design of shearwall segments without hold-down connections.

Without hold-downs, the overturning tension force is transferred from the top wall plate to the bottom wall plate through the shearwall sheathing. Since a portion of the shearwall sheathing is used to resist the overturning force, the shear capacity of the sheathing is reduced. Even though hold-down anchors are not used, anchorage is still required to transfer the uplift force from the wall plate to the foundation or shearwall below (see Commentary).

2. Adequate shear resistance must be provided

   a) For sawn timber and SCL:

      Factored shear resistance $V_r \geq$ maximum factored shear force $V_f$

      In the calculation of $V_f$, loads within a distance from the support equal to the depth of the member may be neglected.

      For sawn timber only, factored notch shear force resistance $F_r \geq$ maximum reaction force $Q_f$ when a member has a notch on the tension side at a support.

   b) For glulam only:

      i) Beams with a volume less than 2.0 $m^3$
      If the beam volume is less than 2.0 $m^3$, shear resistance may be checked by the procedure described in ii) or the tabulated shear resistance $V_r$ may simply be compared to the maximum factored shear force $V_f$. In the calculation of $V_f$, loads within a distance from the support equal to the depth of the member may be neglected.

      ii) Beams of any volume
      Shear resistance $W_r \geq$ total of all factored loads acting on beam $W_f$

where:

$W_r = (W_r\, L^{0.18})\, L_o^{-0.18}$

$(W_r\, L^{0.18})$ = unit factored shear resistance from Beam Selection Tables

$L_o$ = beam length in m

3. Factored bearing resistance $Q_r \geq$ maximum factored reaction $Q_f$ (refer to Chapter 6)

4. Maximum deflection under specified loads < deflection criteria (actual $E_S I$ > required $E_S I$)

   Examples:

   For L/180 deflection limit based on total load:

   $$E_S I_{REQ'D} = 180 \left[\frac{5 w L^3}{384}\right]$$

   For L/360 deflection limit based on live load:

   $$E_S I_{REQ'D} = 360 \left[\frac{5 w_L L^3}{384}\right]$$

where:

w = total specified load

$w_L$ = specified live load

The Beam Selection Tables for sawn timbers and SCL provide $M_r$, $V_r$ and $E_SI$ values.

The Beam Selection Tables for glulam provide $M'_r$, $V_r$ and $E_SI$ values. To obtain $M_r$, for glulam, multiply $M'_r$ (from the Glulam Beam Selection Tables) by the lesser of $K_L$ (from Table 2.9) or $K_{Zbg}$ (from Table 2.10).

Note that in certain cases the *National Building Code of Canada* permits a reduction in the loads due to use and occupancy depending upon the size of the tributary area (refer to Article 4.1.6.9 of the *NBCC*).

## Modification Factors

The tabulated resistances for sawn timber and glulam are based upon the following formulae and modification factors. When using the tables, be certain that the member conforms exactly to the modification factors used in the tables; if not, use the factors below and make the necessary adjustments to the tabulated values.

For sawn timbers:

$$M_r = \phi F_b S K_{zb} K_L \text{ (N•mm)}$$

$$V_r = \phi F_v \frac{2A_n}{3} K_{Zv} \text{ (N)}$$

$$F_r = \phi F_f A K_N \text{ (N)} \quad \text{for beams with notches on the tension side at supports}$$

For straight glulam:

$$M_r = \text{lesser of } M'_r K_L \text{ or } M'_r K_{Zbg} \text{ (N•mm)}$$

$$M'_r = \phi F_b S \text{ (N•mm)}$$

$$V_r = \phi F_v \frac{2A}{3} K_N \text{ (N)}$$

$$W_r = \phi F_v 0.48 A K_N C_v Z^{-0.18} \text{ (N)}$$

which may be expressed as:

$$W_r L^{0.18} = \phi F_v 0.48 A K_N C_v (b\ d)^{-0.18} \text{ (N•m}^{0.18}\text{)}$$

(as shown in the Beam Selection Tables)

For sawn timber and glulam:

$$E_SI = E (K_{SE} K_T) I \text{ (N•mm}^2\text{)}$$

March 2011 Errata



Table 2.10
**Values of $K_{Zbg}$ for glulam beams**

| L m | B (mm) 80 | 130 | 175 | 215 | 265 | 315 | 365 |
|---|---|---|---|---|---|---|---|
| 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| 3.5 | 1 | 1 | 1 | 1 | 1 | 1 | 0.986 |
| 4 | 1 | 1 | 1 | 1 | 1 | 0.988 | 0.962 |
| 4.5 | 1 | 1 | 1 | 1 | 0.998 | 0.967 | 0.942 |
| 5 | 1 | 1 | 1 | 1 | 0.979 | 0.949 | 0.924 |
| 5.5 | 1 | 1 | 1 | 1 | 0.962 | 0.933 | 0.909 |
| 6 | 1 | 1 | 1 | 0.984 | 0.948 | 0.918 | 0.894 |
| 6.5 | 1 | 1 | 1 | 0.970 | 0.934 | 0.905 | 0.882 |
| 7 | 1 | 1 | 0.993 | 0.957 | 0.922 | 0.893 | 0.870 |
| 7.5 | 1 | 1 | 0.981 | 0.945 | 0.910 | 0.882 | 0.859 |
| 8 | 1 | 1 | 0.969 | 0.934 | 0.900 | 0.872 | 0.849 |
| 8.5 | 1 | 1 | 0.959 | 0.924 | 0.890 | 0.863 | 0.840 |
| 9 | 1 | 1 | 0.949 | 0.915 | 0.881 | 0.854 | 0.831 |
| 9.5 | 1 | 0.992 | 0.940 | 0.906 | 0.872 | 0.846 | 0.823 |
| 10 | 1 | 0.982 | 0.931 | 0.897 | 0.864 | 0.838 | 0.816 |
| 10.5 | 1 | 0.974 | 0.923 | 0.890 | 0.857 | 0.830 | 0.809 |
| 11 | 1 | 0.966 | 0.915 | 0.882 | 0.850 | 0.824 | 0.802 |
| 11.5 | 1 | 0.958 | 0.908 | 0.875 | 0.843 | 0.817 | 0.796 |
| 12 | 1 | 0.951 | 0.901 | 0.868 | 0.836 | 0.811 | 0.790 |
| 12.5 | 1 | 0.944 | 0.895 | 0.862 | 0.830 | 0.805 | 0.784 |
| 13 | 1 | 0.937 | 0.888 | 0.856 | 0.824 | 0.799 | 0.778 |
| 13.5 | 1 | 0.931 | 0.882 | 0.850 | 0.819 | 0.794 | 0.773 |
| 14 | 1 | 0.925 | 0.877 | 0.845 | 0.813 | 0.789 | 0.768 |
| 14.5 | 1 | 0.919 | 0.871 | 0.839 | 0.808 | 0.784 | 0.763 |
| 15 | 0.997 | 0.913 | 0.866 | 0.834 | 0.803 | 0.779 | 0.758 |

Notes:
1. B is the member width for single piece laminations, mm. For laminations consisting of multiple pieces, B is the width of the widest piece. Glulam laminations wider than 175 mm are typically constructed with multiple pieces. Check with glulam supplier.
2. L is the length of the glulam from point of zero moment to zero moment, m. For simple span beams, L is the span of the beam.
3. $K_{zbg} = 1.03(B/1000 \times L)^{-0.18}$ but not greater than 1.0.
4. Use the lesser of $K_{Zbg}$ and $K_L$ to determine $M_r$.

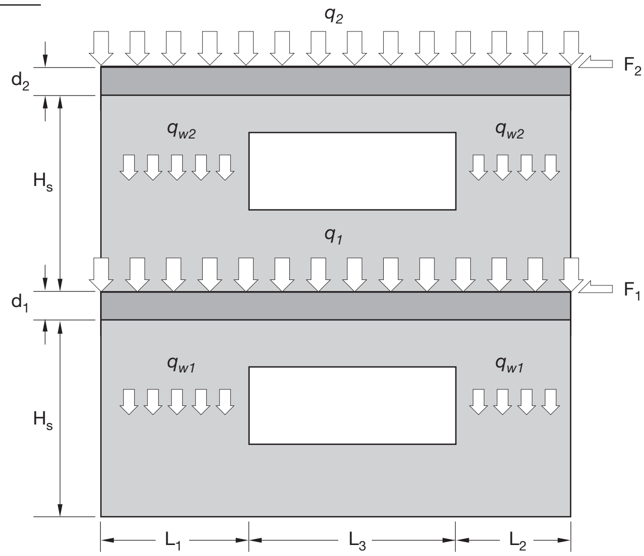
**In the lower storeys of a structure:**

$P_{tj}$ = the factored floor dead load acting on the top of the end stud in the shearwall segment minus $R_j$ from the storey above (kN)

$P_j$ = $P_{tj}$ + the factored wall dead load acting at the bottom of the end stud in the shearwall segment (kN)

$$R_j = \frac{V_{fsi}\,(H_s + d)}{L_s} - P_j \text{ (kN)}$$

Figure 8.5
**Steps in Calculation of $J_{hd}$**




Steps:
Calculate $J_{hd}$ for all segments of the upper storey shearwall before proceeding to the next lower storey.

1. Divide the shearwall into segments ($L_1$, $L_2$)
2. Calculate the dead load to resist overturning at the top of the segments ($P_{tj}$), and the bottom of the segments ($P_j$).
3. Calculate $J_{hd}$ for each of the segments.
4. Calculate the shear resistance of each segment ($V_{rs}$).
5. Distribute the storey shear load ($V_{fs}$) to the segments based on their relative resistance.
6. Calculate the resultant overturning force at the end of each segment ($R_j$).
7. Repeat steps 2 to 6 for lateral loads acting in the opposite direction.
8. Repeat steps 1 to 7 for each lower storey shearwall.

Hold-down connections are required at the ends of each wall segment to resist the $R_1$ uplift forces. If the hold-down connections are installed at the ends of each of the shearwall segments, they may be designed to resist the $R_1$ uplift forces shown above. In general, if the hold-down connections are offset from the ends of the shearwall segments, the hold-down forces are calculated as:

$$T_f = \frac{V_{fs1}\left(H_s + d\right)}{h} - P_j$$

where:

h = centre-to-centre spacing between stud chords

## Lateral Force Distribution to Shearwall Segments

Different methods are used to distribute the shear force on a shearline to each of the shearwall segments. The distribution of lateral forces to the shearwall segments may be based on the relative strength of the segments. This approach is less accurate than distributing forces based on the relative stiffness of the segments. Distributing forces by the relative strength of the segments is improved when the segments in the shearwall have similar configurations and ductility.

The most rigorous approach is to distribute the forces based on the relative stiffness of each segment using the shearwall deflection equations to determine the stiffness of each segment. This approach typically requires an iterative process to arrive at a solution where all of the segments in the shearwall have the same displacement at the load level being considered.

## Example 2: Methods of lateral force distribution to a line of shearwall segments.

Determine the forces in the one storey office building shearwall shown below for the following conditions:
Design information:

- dry service conditions ($K_{SF}$ = 1)
- factored roof diaphragm wind load reaction (F) = 42 kN
- specified roof diaphragm wind load reaction = 30 kN
- specified shearwall force, $V_{fs}$ = 30 kN
- walls are 3 m high
- depth of the roof framing is 0.3 m