Braced panel vertical post and bracing design in long term-duration loading

Wood type:	C24
Braced panel height:	$H := 3.0\ m$
Braced panel width:	$B := 0.8\ m$
Bracing angle:	$\alpha := 63.44\ deg$
Width of wooden element:	$b := 45\ mm$
Height of wooden element:	$h := 95\ mm$
Buckling length about x axis (minus minimal lintel height):	$l_x := 2.886\ m$
Buckling length about y axis:	$l_y := 0.628\ m$
Factor for support condition at the ends of the element:	$\mu_x := 1 \quad \mu_y := 1$
Factor for solid timber straightness:	$\beta_c := 0.2$
Charac. wood bending strength:	$f_{m.k} := 24\ MPa$
Charac. wood compression strength parallel to the grain:	$f_{c.0.k} := 21\ MPa$
Charac. wood tension strength parallel to the grain:	$f_{t.0.k} := 14\ MPa$
Wood modulus of elasticity parallel to the grain:	$E_{0.05} := 7400\ MPa$
Wood modulus of mean elasticity parallel to the grain:	$E_{0.mean} := 11000\ MPa$
Factor for duration loading and service:	$k_{mod} := 0.6$
Factor for load shearing:	$k_{sys} := 1.0$
Deformation factor for for solid timber class 1:	$k_{def} := 0.6$
Partial factor for wood properties:	$\gamma_M := 1.30$

Rectangular panel member design for compression

Design wood compression strength:

$$f_{c.0.d} := \frac{k_{mod} \cdot f_{c.0.k}}{\gamma_M} = 9.692 \ \frac{N}{mm^2}$$

Cross-sectional area:

$$A := b \cdot h = 42.75 \ cm^2$$

Second moment of area:

$$I_x := \frac{h^3 \cdot b}{12} = 321.516 \ cm^4 \qquad I_y := \frac{h \cdot b^3}{12} = 72.141 \ cm^4$$

Radius of gyration:

$$i_x := \sqrt{\frac{I_x}{A}} = 27.424 \ mm \qquad i_y := \sqrt{\frac{I_y}{A}} = 12.99 \ mm \qquad i := min\left(i_x, i_y\right) = 12.99 \ mm$$

Design element length:

$$l_{ef.x} := \mu_x \cdot l_x = 2.886 \ m \qquad l_{ef.y} := \mu_y \cdot l_y = 0.628 \ m$$

Slenderness ratio:

$$\lambda_x := \frac{l_{ef.x}}{i_x} = 105.236 \qquad \lambda_y := \frac{l_{ef.y}}{i_y} = 48.343 \qquad \lambda := \max\left(\lambda_x, \lambda_y\right) = 105.236$$

Relative slenderness:

$$\lambda_{rel} := \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.784$$

Instability factor:

$$k := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + \lambda_{rel}^{2}\right) = 2.241$$

$$k_c := \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^{2}}} = 0.278$$

Design buckling strength:

$$k_c \cdot f_{c.0.d} = 2.696 \ \frac{N}{mm^2}$$

Compressive stress/Design buckling strength equation :

$N_{c.d} := f_{c.0.k} \cdot A \cdot k_c = 24.968 \ kN$

$N_{c.d} := f_{c.0.d} \cdot A \cdot k_c = 11.524 \ kN$

Rectangular panel member design for tenssion

$$f_{t.0.d} := \frac{f_{t.0.k} \cdot k_{mod}}{\gamma_M} = 6.462 \ \frac{N}{mm^2}$$

$N_{T.d} := f_{t.0.d} \cdot A = 27.623 \ kN$

Vertical desidn loads: $F_{v.d} := 0 \ kN$

Compression

$F_{h.d.1} := N_{c.d} \cdot \cos(\alpha) = 5.153 \ kN$

$$F_{h.d.2} := \frac{N_{c.d} \cdot \cos(\alpha) - F_{v.d} \cdot \cos(\alpha)}{2 \cdot \sin(\alpha)} = 2.88 \ kN$$

$F_{h.d.c} := \min(F_{h.d.1}, F_{h.d.2}) = 2.88 \ kN$

Tension

$$F_{h.d.3} := -\left(\frac{F_{v.d} \cdot \cos(\alpha) - N_{T.d}}{\sin(\alpha)}\right) = 30.882 \ kN$$

$F_{h.d.4} := N_{T.d} \cdot \cos(\alpha) = 12.351 \ kN$

$F_{h.d.t} := min(F_{h.d.3}, F_{h.d.4}) = 12.351 \ kN$

Maximal lateral load

$F_{h.d} := min(F_{h.d.c}, F_{h.d.t}) = 2.88 \ kN$

Compression

$N_{2\_5} := \frac{F_{h.d}}{\cos(\alpha)} = 6.442\ kN$

$N_{3\_5} := F_{v.d} = 0\ kN$

$N_{4\_5} := F_{v.d} + \frac{2 \cdot F_{h.d}}{\cos(\alpha)} \cdot \sin(\alpha) = 11.524\ kN$

Tension

$N_{1\_2} := F_{v.d} - \frac{F_{h.d} \cdot \sin(\alpha)}{\cos(\alpha)} = -5.762\ kN$

$N_{1\_5} := \frac{-F_{h.d}}{\cos(\alpha)} = -6.442\ kN$

$N_{2\_3} := 0$

Reactions

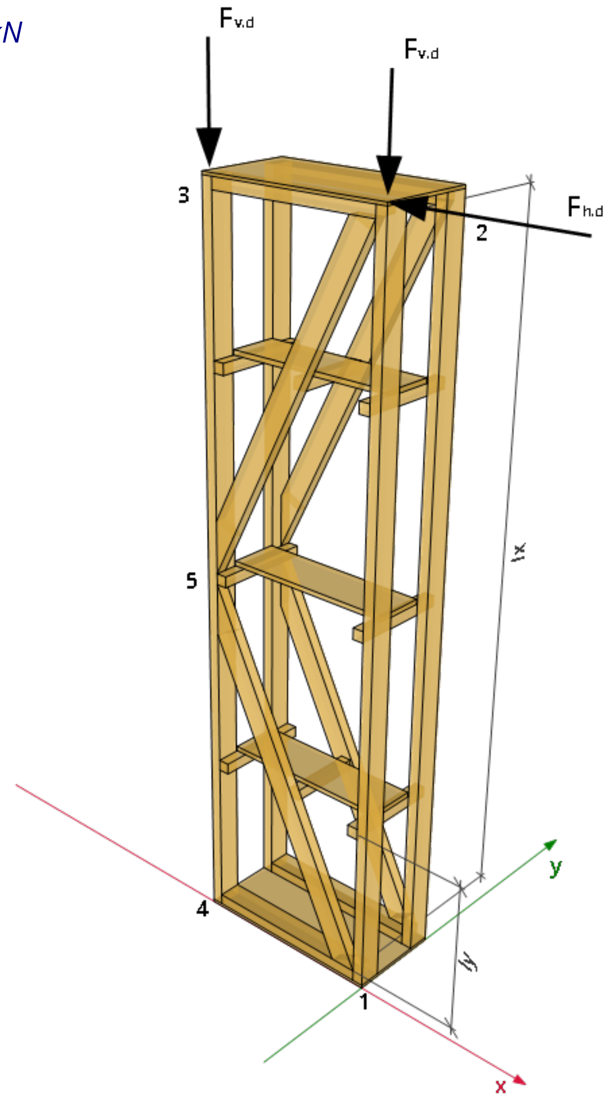
$R_{1y} := F_{v.d} - \frac{F_{h.d} \cdot \sin(\alpha)}{\cos(\alpha)} \cdot 2 = -11.524\ kN$

$R_{2y} := F_{v.d} + \frac{F_{h.d} \cdot \sin(\alpha)}{\cos(\alpha)} \cdot 2 = 11.524\ kN$

$R_{1x} := F_{h.d} = 2.88\ kN$

$R_{2x} := 0$

$F_{h.d} = 2.88\ kN$



Braced panel vertical post and bracing design in long term-duration loading

Wood type:	C24
Braced panel height:	$H := 3.0\ m$
Braced panel width:	$B := 0.8\ m$
Bracing angle:	$\alpha := 63.44\ deg$
Width of wooden element:	$b := 45\ mm$
Height of wooden element:	$h := 95\ mm$
Buckling length about x axis (minus minimal lintel height):	$l_x := 2.886\ m$
Buckling length about y axis:	$l_y := 0.628\ m$
Factor for support condition at the ends of the element:	$\mu_x := 1 \quad \mu_y := 1$
Factor for solid timber straightness:	$\beta_c := 0.2$
Charac. wood bending strength:	$f_{m.k} := 24\ MPa$
Charac. wood compression strength parallel to the grain:	$f_{c.0.k} := 21\ MPa$
Charac. wood tension strength parallel to the grain:	$f_{t.0.k} := 14\ MPa$
Wood modulus of elasticity parallel to the grain:	$E_{0.05} := 7400\ MPa$
Wood modulus of mean elasticity parallel to the grain:	$E_{0.mean} := 11000\ MPa$
Factor for duration loading and service:	$k_{mod} := 0.6$
Factor for load shearing:	$k_{sys} := 1.0$
Deformation factor for for solid timber class 1:	$k_{def} := 0.6$
Partial factor for wood properties:	$\gamma_M := 1.30$

Rectangular panel member design for compression

Design wood compression strength:

$$f_{c.0.d} := \frac{k_{mod} \cdot f_{c.0.k}}{\gamma_M} = 9.692 \ \frac{N}{mm^2}$$

Cross-sectional area:

$$A := b \cdot h = 42.75 \ cm^2$$

Second moment of area:

$$I_x := \frac{h^3 \cdot b}{12} = 321.516 \ cm^4 \qquad I_y := \frac{h \cdot b^3}{12} = 72.141 \ cm^4$$

Radius of gyration:

$$i_x := \sqrt{\frac{I_x}{A}} = 27.424 \ mm \qquad i_y := \sqrt{\frac{I_y}{A}} = 12.99 \ mm \qquad i := min\left(i_x, i_y\right) = 12.99 \ mm$$

Design element length:

$$l_{ef.x} := \mu_x \cdot l_x = 2.886 \ m \qquad l_{ef.y} := \mu_y \cdot l_y = 0.628 \ m$$

Slenderness ratio:

$$\lambda_x := \frac{l_{ef.x}}{i_x} = 105.236 \qquad \lambda_y := \frac{l_{ef.y}}{i_y} = 48.343 \qquad \lambda := \max\left(\lambda_x, \lambda_y\right) = 105.236$$

Relative slenderness:

$$\lambda_{rel} := \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.784$$

Instability factor:

$$k := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + \lambda_{rel}^2\right) = 2.241$$

$$k_c := \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} = 0.278$$

Design buckling strength:

$$k_c \cdot f_{c.0.d} = 2.696 \ \frac{N}{mm^2}$$

Compressive stress/Design buckling strength equation :

$$N_{c.d} := f_{c.0.k} \cdot A \cdot k_c = 24.968\ kN$$

$$N_{c.d} := f_{c.0.d} \cdot A \cdot k_c = 11.524\ kN$$

Rectangular panel member design for tenssion

$$f_{t.0.d} := \frac{f_{t.0.k} \cdot k_{mod}}{\gamma_M} = 6.462\ \frac{N}{mm^2}$$

$$N_{T.d} := f_{t.0.d} \cdot A = 27.623\ kN$$

Vertical desidn loads: $F_{v.d} := 0\ kN$

Compression

$$F_{h.d.1} := \frac{N_{c.d} \cdot \cos(\alpha) - F_{v.d} \cdot \cos(\alpha)}{\sin(\alpha)} = 5.761\ kN$$

$$F_{h.d.2} := N_{c.d} \cdot \cos(\alpha) = 5.153\ kN$$

$$F_{h.d.c} := \min\left(F_{h.d.1}, F_{h.d.2}\right) = 5.153\ kN$$

Tension

$$F_{h.d.4} := N_{T.d} \cdot \cos(\alpha) = 12.351\ kN$$

$$F_{h.d.3} := -\left(\frac{F_{v.d} \cdot \cos(\alpha) - N_{T.d} \cdot \cos(\alpha)}{2 \cdot \sin(\alpha)}\right) = 6.904\ kN$$

$$F_{h.d.t} := min\left(F_{h.d.3}, F_{h.d.4}\right) = 6.904\ kN$$

Maximal lateral load

$$F_{h.d} := min\left(F_{h.d.c}, F_{h.d.t}\right) = 5.153\ kN$$

Compression

$$N_{1\_2} := F_{v.d} + \frac{F_{h.d} \cdot \sin(\alpha)}{\cos(\alpha)} = 10.308 \ kN$$

$$N_{1\_5} := \frac{F_{h.d}}{\cos(\alpha)} = 11.524 \ kN$$

$$N_{2\_3} := F_{h.d} = 5.153 \ kN$$

Tension

$$N_{2\_5} := \frac{-F_{h.d}}{\cos(\alpha)} = -11.524 \ kN$$

$$N_{3\_5} := F_{v.d} = 0 \ kN$$

$$N_{4\_5} := F_{v.d} - \frac{F_{h.d} \cdot \sin(\alpha)}{\cos(\alpha)} \cdot 2 = -20.615 \ kN$$

Reactions

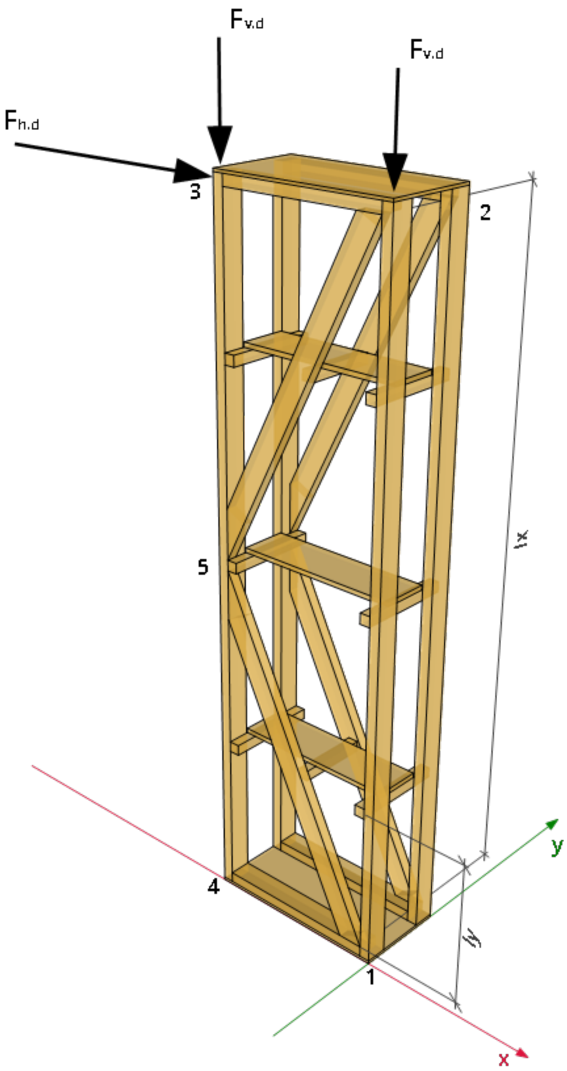
$$R_{1y} := F_{v.d} - \frac{F_{h.d} \cdot \sin(\alpha)}{\cos(\alpha)} \cdot 2 = -20.615 \ kN$$

$$R_{2y} := F_{v.d} + \frac{F_{h.d} \cdot \sin(\alpha)}{\cos(\alpha)} \cdot 2 = 20.615 \ kN$$

$$R_{1x} := F_{h.d} = 5.153 \ kN$$

$$R_{2x} := 0$$

$$F_{h.d} = 5.153 \ kN$$



Panel in whole construction and weakest part of panel

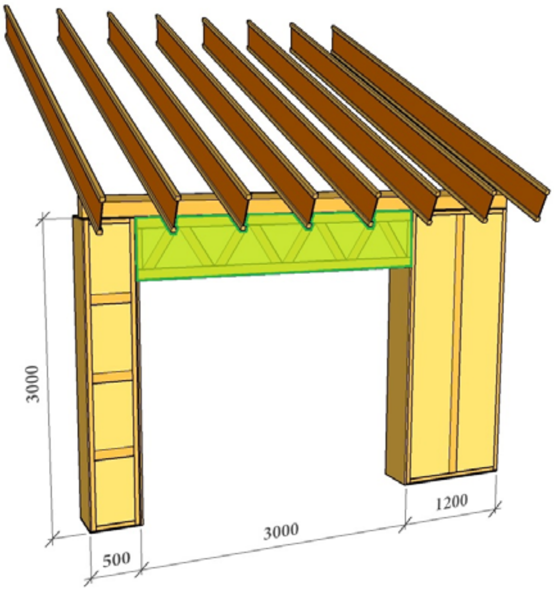


Fig 1. Total view

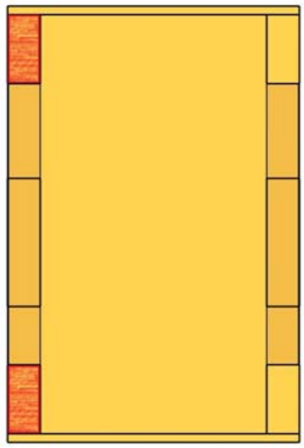


Fig 2. Side section of panel

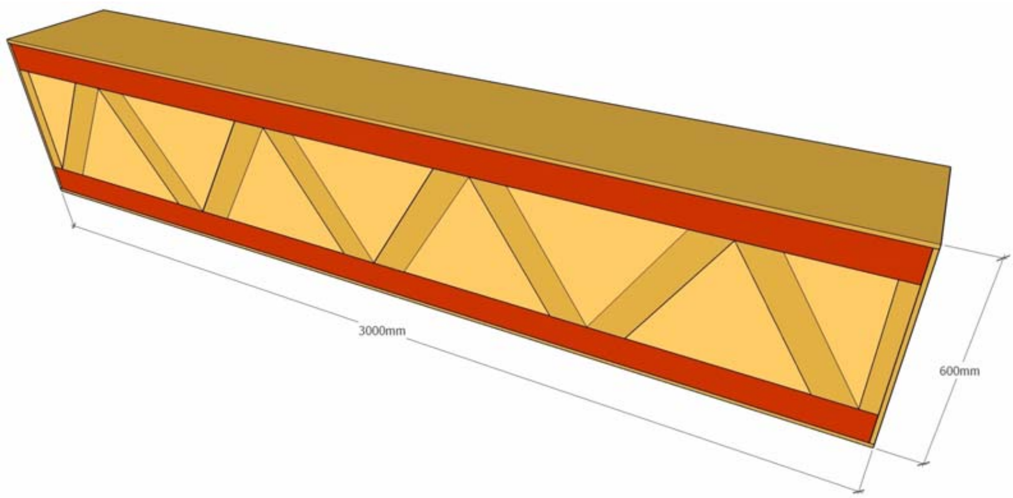


Fig 3. 3D view of panel

Lintel members design

Geometric parameters:

Height: $h := 95 \cdot mm$

Width: $b := 45 \cdot mm$

Section area: $A := h \cdot b = 0.004\ m^2$

Span: $B := 6.0\ m$

Moment of inertia: $I := \frac{b \cdot h^3}{12} = 321.516\ cm^4$

Section modulus about the strong axis: $W := \frac{b \cdot h^2}{6} = 67.688\ cm^3$

Factor for constant load: $\gamma_G := 1.35$

Factor for variable load: $\gamma_Q := 1.30$

Permanent action:

$g_{k1} := 2.4\ kPa$

$g_{d1} := g_{k1} \cdot \gamma_G = 3.24\ kPa$

Variable action:

$q_{ks} := 0.8 \cdot 1.0 \cdot 1.0 \cdot 1.6 \cdot kPa = 1.28\ kPa$ $\quad q_{kw} := 0.89 \cdot kPa$

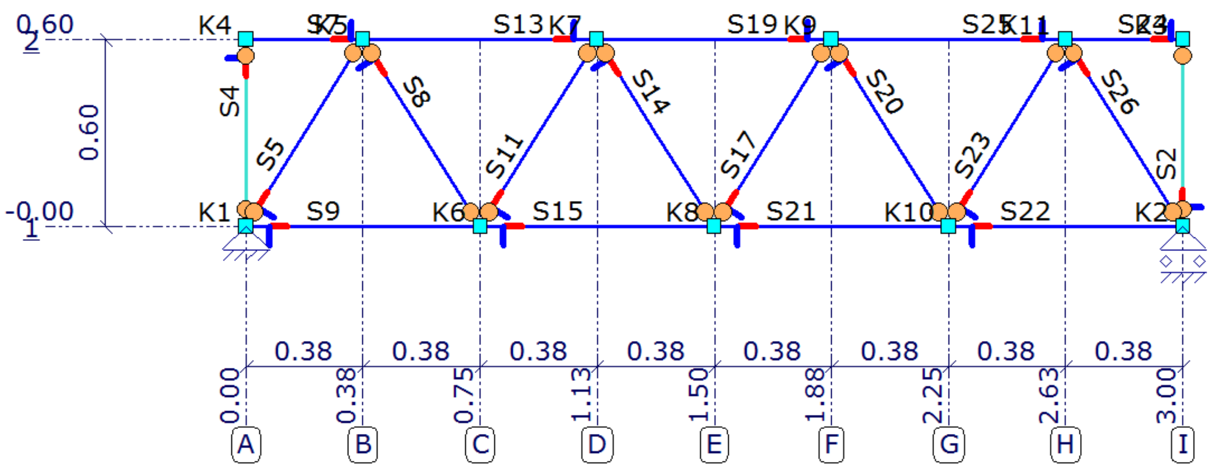
$q_{k1} := q_{ks} + q_{kw} = 2.17\ kPa$

$q_{d1} := q_{k1} \cdot \gamma_Q = 2.821\ kPa$

Sum of roof actions: $P_1 := g_{d1} + q_{d1} = 6.061\ kPa$

Loading on the wall: $p := P_1 \cdot B \cdot 0.5 = 18.183\ \frac{kN}{m}$

Calculation is made according to finite element program "MatrixFrame":



Max forces in the lintel web:

Max Axial compressive force in S19: $N_{C.max.d} := 33.20\ kN$

Max Axial tension force in S15 and S21: $N_{T.max.d} := 33.20\ kN$

Max Shear force in S13 and S25: $V_{max.d} := 7.10\ kN$

Max Bending force in S19: $M_{max.d} := 0.60\ kN \cdot m$

Physical/mechanical wood properties:

Characteristic bending strength $f_{m.k} := 24 \cdot \frac{N}{mm^2}$

Mean value of modulus of elasticity parallel to the grain $E_{0.mean} := 11000 \cdot \frac{N}{mm^2}$

Characteristic shear strength $f_{v.k} := 2.5 \cdot \frac{N}{mm^2}$ $f_{v.k.ply} := 2.9 \cdot \frac{N}{mm^2}$

Characteristic tenssion strength parallel to the grain $f_{t.0.k} := 14 \cdot \frac{N}{mm^2}$

Characteristic compressive strength parallel to the grain $f_{c.0.k} := 21 \cdot \frac{N}{mm^2}$

Mean value of shear modulus $G_{mean} := 690 \cdot \frac{N}{mm^2}$

Factors in use:

Partial factor for massive wood $\gamma_M := 1.30$

Deformation factor $k_{def} := 0.6$
Modification factor for duration of load and moisture content $k_{mod} := 0.7$
$k_{mod.short} := 0.9$

Depth factor for massive wood

h <150mm
$$k_h := min\left(\left(\frac{150 \cdot mm}{h}\right)^{0.2}, 1.3\right) = 1.096$$

Ultimate Limit States (ULS)
Bending strength

Design moment $M_d := M_{max.d}$

Design bending strength
$$f_{m.d} := \frac{f_{m.k} \cdot k_{mod} \cdot k_h}{\gamma_M} = 14.159 \ \frac{N}{mm^2}$$

Design section modulus about strong axis
$$W_d := \frac{M_d}{f_{m.d}} = 42.375 \ cm^3$$

$$\sigma_{m.y.d} := \frac{M_d}{W} = 8.864 \ \frac{N}{mm^2}$$

$$h := \sqrt{\frac{6 \cdot W_d}{b}} = 0.075 \ m$$

$$h := 95 \cdot mm$$

Shear strength

Design shear force $V_d := V_{max.d}$

Design shear stress $\tau_{v.d} := \frac{3}{2} \cdot \frac{V_d}{A + 12\ mm \cdot b} = 2.212\ \frac{N}{mm^2}$

Design shear strength $f_{v.d} := \frac{k_{mod} \cdot \left(f_{v.k} + f_{v.k.ply}\right)}{\gamma_M} = 2.908\ \frac{N}{mm^2}$

Compressive stress

Design value of the end reaction $N_d := N_{C.max.d}$

Design compressive stress perpendicular to the grain $\sigma_{c.0.d} := \frac{N_d}{A} = 7.766\ \frac{N}{mm^2}$

Design compressive stress $f_{c.0.d} := \frac{k_{mod} \cdot f_{c.0.k}}{\gamma_M} = 11.308\ \frac{N}{mm^2}$

Tenssion stress

Design value of the end reaction $N_{T.d} := N_{T.max.d}$

Design compressive stress perpendicular to the grain $\sigma_{t.0.d} := \frac{N_{T.d}}{A} = 7.766\ \frac{N}{mm^2}$

Design compressive stress $f_{t.0.d} := \frac{k_{mod} \cdot f_{t.0.k} \cdot k_h}{\gamma_M} = 8.26\ \frac{N}{mm^2}$

Serviceability Limit States (SLS)

Limiting values for deflections

$u_{fin.g} := 3 \cdot mm$

$u_{lim} := \frac{3\ m}{300} = 10\ mm$ $\Psi_2 := 0$

$u_{inst.G} := u_{fin.g} \cdot \left(1 + k_{def}\right) = 4.8\ mm$

$u_{net.lim} := \frac{3\ m}{250} = 12\ mm$

Design plywood flat pressure resistance for long term duration load

Timber geometric properties:

Thickness of plywood: $t := 12\ mm$

Width of wooden element: $b_1 := 45\ mm$ $b_2 := 90\ mm$

Height of wooden element: $h_1 := 95\ mm$

Bearing on plywood face strength small area: $f_{c.face.k} := 9.0\ \frac{N}{mm^2}$

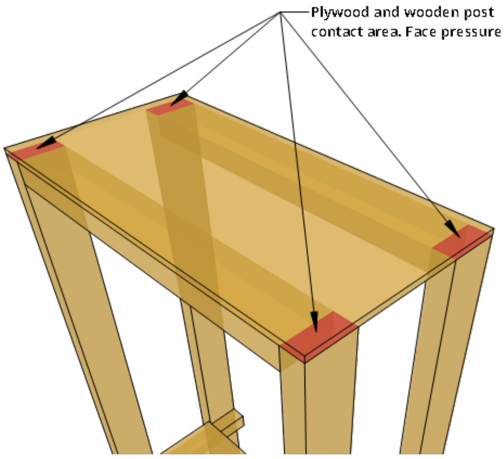
Partial safety factor for plywood: $\gamma_{M.plywood} := 1.2$

2nd service class factor for long duration loading: $k_{mod} := 0.7$

Design compressive stress on face area: $f_{c.face.d} := \frac{k_{mod} \cdot f_{c.face.k}}{\gamma_{M.plywood}} = 5.25\ \frac{N}{mm^2}$

Maximum load on plywood face: $F_{d.face} := f_{c.face.d} \cdot b_1 \cdot h_1 = 22.444\ kN$

$F_{d.face} := f_{c.face.d} \cdot b_2 \cdot h_1 = 44.888\ kN$



1Pic. Plywood and wood contact area

Screws in Breced Ecococon panel calculation

Wood type:	C24
Width of wooden element:	$t_1 := 45 \cdot mm$
Height of wooden element:	$t_2 := 95 \cdot mm$
Bracing angle:	$\alpha := 63.44 \cdot deg$
Screw length:	$l_s := 120 \cdot mm$
Screw diameter:	$d := 8 \cdot mm$
Threaded part of the screw 8,0x120:	$l_{g.1} := 80\ mm$
	$l_{ef.1} := l_{g.1} - d = 72\ mm$
Threaded part of the screw 8,0x100:	$l_{g.2} := 60\ mm$
	$l_{ef.2} := l_{g.2} - d = 52\ mm$
Threaded part of the screw 8,0x80:	$l_{g.3} := 50\ mm$
	$l_{ef.3} := l_{g.3} - d = 42\ mm$
Charac. density of the timber	$\rho_k := 350 \cdot \frac{kg}{m^3}$
Partial factor for material properties:	$\gamma_M := 1.30$
Charac. tensile strength of each screw:	$f_{u.k} := 600 \cdot MPa$
Charac. load-carrying capacity of axially loaded screw:	$f_{c.90.k} := 5.1 \cdot MPa$
Material factor for connections:	$\gamma_{M.connection} := 1.3$
Factor for duration loading and service:	$k_{mod} := 0.6$

Axial design withdrawal capacity of the screw

$$f_{ax.k} := 3.6 \cdot 10^{-3} \cdot \left(q_k \cdot \frac{m^3}{kg}\right)^{1.5} \cdot N \cdot mm^{-2} = 23.572 \ MPa$$

$$f_{ax.\alpha.k} := \frac{f_{ax.k}}{(\sin(\alpha))^2 + 1.5 \cdot (\cos(\alpha))^2} = 21.43 \ MPa$$

For screw 8,0x120mm: $\beta := 1$

$$F_{ax.\alpha.Rk.1} := \left(\pi \cdot d \cdot l_{ef.1} \cdot mm^{-2}\right)^{0.8} \cdot f_{ax.\alpha.k} \cdot mm^2 = 8.651 \ kN$$

$$F_{ax.Rd.1} := \frac{F_{ax.\alpha.Rk.1} \cdot k_{mod}}{\gamma_{M.connection}} = 3.993 \ kN$$

For screw 8,0x100mm:

$$F_{ax.\alpha.Rk.2} := \left(\pi \cdot d \cdot l_{ef.2} \cdot mm^{-2}\right)^{0.8} \cdot f_{ax.\alpha.k} \cdot mm^2 = 6.668 \ kN$$

$$F_{ax.Rd.2} := \frac{F_{ax.\alpha.Rk.2} \cdot k_{mod}}{\gamma_{M.connection}} = 3.078 \ kN$$

Maximal axial tensional load in bracing:

$$F_{ax.Rd} := \left(F_{ax.Rd.1} + F_{ax.Rd.2}\right) = 7.071 \ kN$$

Embedment strength of timber

$$f_{h.k} := 0.082 \cdot \left(q_k \cdot \frac{m^3}{kg}\right) \cdot \left(d \cdot mm^{-1}\right)^{-0.3} \cdot N \cdot mm^{-2} = 15.38 \ MPa$$

Yield moment of a screw

$$M_{y.Rk} := 0.15 \cdot \left(f_{u.k} \cdot \frac{mm^2}{N}\right) \cdot \left(d \cdot mm^{-1}\right)^{2.6} \cdot N \cdot mm = \left(2.006 \cdot 10^4\right) \ N \cdot mm$$

Shear strength for the screw 8,0x120

Failure mode (a): $F_{v.Rk.a} := f_{h.k} \cdot t_1 \cdot d = 5.537\ kN$

Failure mode (b): $F_{v.Rk.b} := f_{h.k} \cdot t_2 \cdot d = 11.689\ kN$

Failure mode (c):

$$F_{v.Rk.c} := \frac{f_{h.k} \cdot t_1 \cdot d}{1+\beta} \cdot \left(\sqrt{\beta + 2 \cdot \beta^2 \cdot \left(1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right) + \beta^3 \cdot \left(\frac{t_2}{t_1}\right)^2} - \beta \cdot \left(1 + \left(\frac{t_2}{t_1}\right)\right)\right) + \frac{F_{ax.\alpha.Rk.1}}{4} = 6.113\ kN$$

$$F_{v.Rk.d} := 1.05 \cdot \frac{f_{h.k} \cdot t_1 \cdot d}{2+\beta} \cdot \left(\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y.Rk}}{f_{h.k} \cdot {t_1}^2 \cdot d}} - \beta\right) + \frac{F_{ax.\alpha.Rk.1}}{4} = 4.543\ kN$$

$$F_{v.Rk.e} := 1.05 \cdot \frac{f_{h.k} \cdot t_2 \cdot d}{1+2 \cdot \beta} \cdot \left(\sqrt{2 \cdot \beta^2 \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (1+2 \cdot \beta) \cdot M_{y.Rk}}{f_{h.k} \cdot {t_2}^2 \cdot d}} - \beta\right) + \frac{F_{ax.\alpha.Rk.1}}{4} = 6.473\ kN$$

$$F_{v.Rk.f} := 1.15 \left(\sqrt{\frac{2 \cdot \beta}{1+\beta} \cdot (2 \cdot M_{y.Rk} \cdot f_{h.k} \cdot d)}\right) + \frac{F_{ax.\alpha.Rk.1}}{4} = 4.718\ kN$$

$$F_{v.Rk} := min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}\right) = 4.543\ kN \qquad F_{v.Rd.1} := \frac{F_{v.Rk} \cdot k_{mod}}{\gamma_{M.connection}} = 2.097\ kN$$

Shear strength for the screw 8,0x100

Failure mode (a): $F_{v.Rk.a} := f_{h.k} \cdot t_1 \cdot d = 5.537\ kN$

Failure mode (b): $F_{v.Rk.b} := f_{h.k} \cdot t_2 \cdot d = 11.689\ kN$

Failure mode (c):

$$F_{v.Rk.c} := \frac{f_{h.k} \cdot t_1 \cdot d}{1+\beta} \cdot \left(\sqrt{\beta + 2 \cdot \beta^2 \cdot \left(1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right) + \beta^3 \cdot \left(\frac{t_2}{t_1}\right)^2} - \beta \cdot \left(1 + \left(\frac{t_2}{t_1}\right)\right)\right) + \frac{F_{ax.\alpha.Rk.2}}{4} = 5.617\ kN$$

$$F_{v.Rk.d} := 1.05 \cdot \frac{f_{h.k} \cdot t_1 \cdot d}{2+\beta} \cdot \left(\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y.Rk}}{f_{h.k} \cdot {t_1}^2 \cdot d}} - \beta\right) + \frac{F_{ax.\alpha.Rk.2}}{4} = 4.048\ kN$$

$$F_{v.Rk.e} := 1.05 \cdot \frac{f_{h.k} \cdot t_2 \cdot d}{1+2 \cdot \beta} \cdot \left(\sqrt{2 \cdot \beta^2 \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (1+2 \cdot \beta) \cdot M_{y.Rk}}{f_{h.k} \cdot {t_2}^2 \cdot d}} - \beta\right) + \frac{F_{ax.\alpha.Rk.2}}{4} = 5.977\ kN$$

$$F_{v.Rk.f} := 1.15 \left(\sqrt{\frac{2 \cdot \beta}{1+\beta} \cdot (2 \cdot M_{y.Rk} \cdot f_{h.k} \cdot d)}\right) + \frac{F_{ax.\alpha.Rk.2}}{4} = 4.222\ kN$$

$$F_{v.Rk} := min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}\right) = 4.048\ kN \qquad F_{v.Rd.2} := \frac{F_{v.Rk} \cdot k_{mod}}{\gamma_{M.connection}} = 1.868\ kN$$

Shear strength of the connection

$F_{v.Rd} := F_{v.Rd.1} + F_{v.Rd.2} = 3.965\ kN$

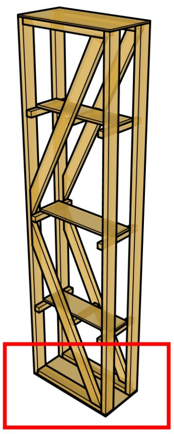
Axial strength and shear strength combined in connection

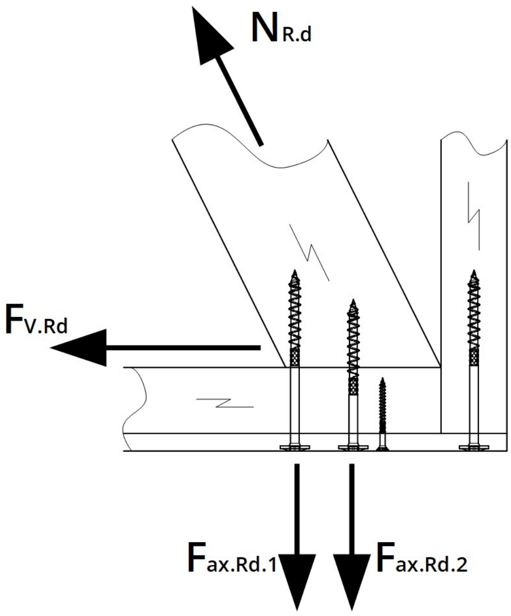
By taking as a foundation formula EC5 (8.28):

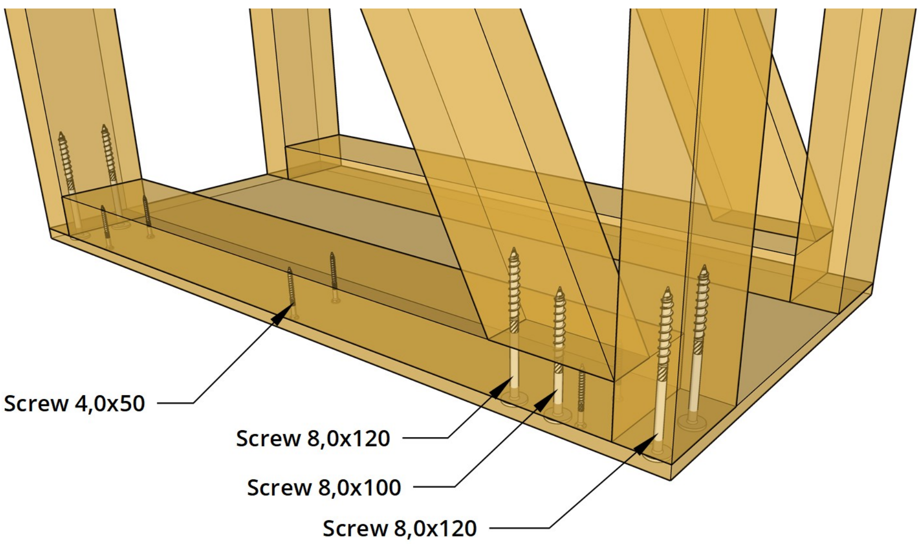
$$\left(\frac{F_{ax.Ed}}{F_{ax.Rd}}\right)^2 + \left(\frac{F_{v.Ed}}{F_{v.Rd}}\right)^2$$

Maximal axial load in one bracing is being calculated:

$$N_{R.d} := \sqrt{\frac{F_{ax.Rd}^{\ 2} \cdot F_{v.Rd}^{\ 2}}{F_{v.Rd}^{\ 2} \cdot \sin(\alpha)^2 + F_{ax.Rd}^{\ 2} \cdot \cos(\alpha)^2}} = 5.901\ kN$$







Screws in Ecococon panel calculation for a traverse connection

Wood type:	C24
Width of wooden element:	$t_1 := 25 \cdot mm$
Height of wooden element:	$t_2 := 45 \cdot mm$
Screw angle:	$\alpha := 0 \cdot deg$
Screw length:	$l_s := 80 \cdot mm$
Screw diameter:	$d := 8 \cdot mm$
Threaded part of the screw 8,0x80:	$l_{g.3} := 50\ mm$
	$l_{ef.3} := l_{g.3} - d = 42\ mm$
Charac. density of the timber	$q_k := 350 \cdot \frac{kg}{m^3}$
Partial factor for material properties:	$\gamma_M := 1.30$
Charac. tensile strength of each screw:	$f_{u.k} := 600 \cdot MPa$
Charac. load-carrying capacity of axially loaded screw:	$f_{c.90.k} := 5.1 \cdot MPa$
Material factor for connections:	$\gamma_{M.connection} := 1.3$
Factor for duration loading and service:	$k_{mod} := 0.6$

Axial design withdrawal capacity of the screw

$$f_{ax.k} := 3.6 \cdot 10^{-3} \cdot \left(q_k \cdot \frac{m^3}{kg}\right)^{1.5} \cdot N \cdot mm^{-2} = 23.572\ MPa$$

$$f_{ax.\alpha.k} := \frac{f_{ax.k}}{(\sin(\alpha))^2 + 1.5 \cdot (\cos(\alpha))^2} = 15.715\ MPa$$

For screw 8,0x80mm: $\beta := 1$

$$F_{ax.\alpha.Rk} := \left(\pi \cdot d \cdot l_{ef.3} \cdot mm^{-2}\right)^{0.8} \cdot f_{ax.\alpha.k} \cdot mm^2 = 4.122\ kN$$

$$F_{ax.Rd.1} := \frac{F_{ax.\alpha.Rk} \cdot k_{mod}}{\gamma_{M.connection}} = 1.902\ kN$$

Maximal axial tensional load in connection:

$$F_{ax.Rd} := F_{ax.Rd.1} \cdot 2 = 3.805\ kN$$

Embedment strength of timber

$$f_{h.k} := 0.082 \cdot \left(q_k \cdot \frac{m^3}{kg} \right) \cdot \left(d \cdot mm^{-1} \right)^{-0.3} \cdot N \cdot mm^{-2} = 15.38\ MPa$$

Yield moment of a screw

$$M_{y.Rk} := 0.15 \cdot \left(f_{u.k} \cdot \frac{mm^2}{N} \right) \cdot \left(d \cdot mm^{-1} \right)^{2.6} \cdot N \cdot mm = \left(2.006 \cdot 10^4 \right)\ N \cdot mm$$

Shear strength for the screw 8,0x80

Failure mode (a): $F_{v.Rk.a} := f_{h.k} \cdot t_1 \cdot d = 3.076\ kN$

Failure mode (b): $F_{v.Rk.b} := f_{h.k} \cdot t_2 \cdot d = 5.537\ kN$

Failure mode (c):

$$F_{v.Rk.c} := \frac{f_{h.k} \cdot t_1 \cdot d}{1+\beta} \cdot \left(\sqrt{\beta + 2 \cdot \beta^2 \cdot \left(1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 \right) + \beta^3 \cdot \left(\frac{t_2}{t_1} \right)^2} - \beta \cdot \left(1 + \left(\frac{t_2}{t_1} \right) \right) \right) + \frac{F_{ax.a.Rk}}{4} = 2.937\ kN$$

$$F_{v.Rk.d} := 1.05 \cdot \frac{f_{h.k} \cdot t_1 \cdot d}{2+\beta} \cdot \left(\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y.Rk}}{f_{h.k} \cdot t_1^2 \cdot d}} - \beta \right) + \frac{F_{ax.a.Rk}}{4} = 2.829\ kN$$

$$F_{v.Rk.e} := 1.05 \cdot \frac{f_{h.k} \cdot t_2 \cdot d}{1+2 \cdot \beta} \cdot \left(\sqrt{2 \cdot \beta^2 \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (1+2 \cdot \beta) \cdot M_{y.Rk}}{f_{h.k} \cdot t_2^2 \cdot d}} - \beta \right) + \frac{F_{ax.a.Rk}}{4} = 3.411\ kN$$

$$F_{v.Rk.f} := 1.15 \left(\sqrt{\frac{2 \cdot \beta}{1+\beta} \cdot \left(2 \cdot M_{y.Rk} \cdot f_{h.k} \cdot d \right)} \right) + \frac{F_{ax.a.Rk}}{4} = 3.585\ kN$$

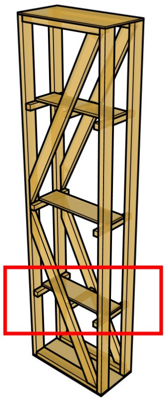
$$F_{v.Rk} := min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f} \right) = 2.829\ kN$$

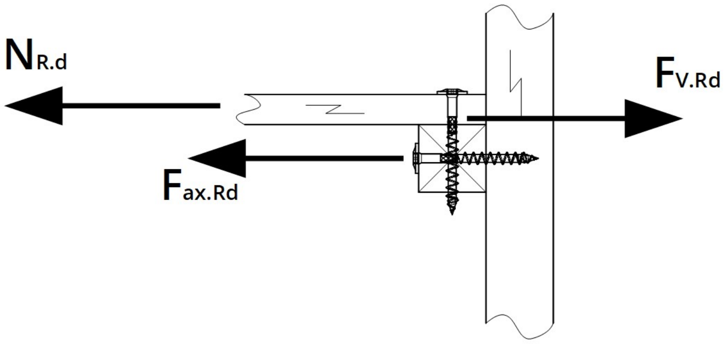
$$F_{v.Rd.1} := \frac{F_{v.Rk} \cdot k_{mod}}{\gamma_{M.connection}} = 1.306\ kN$$

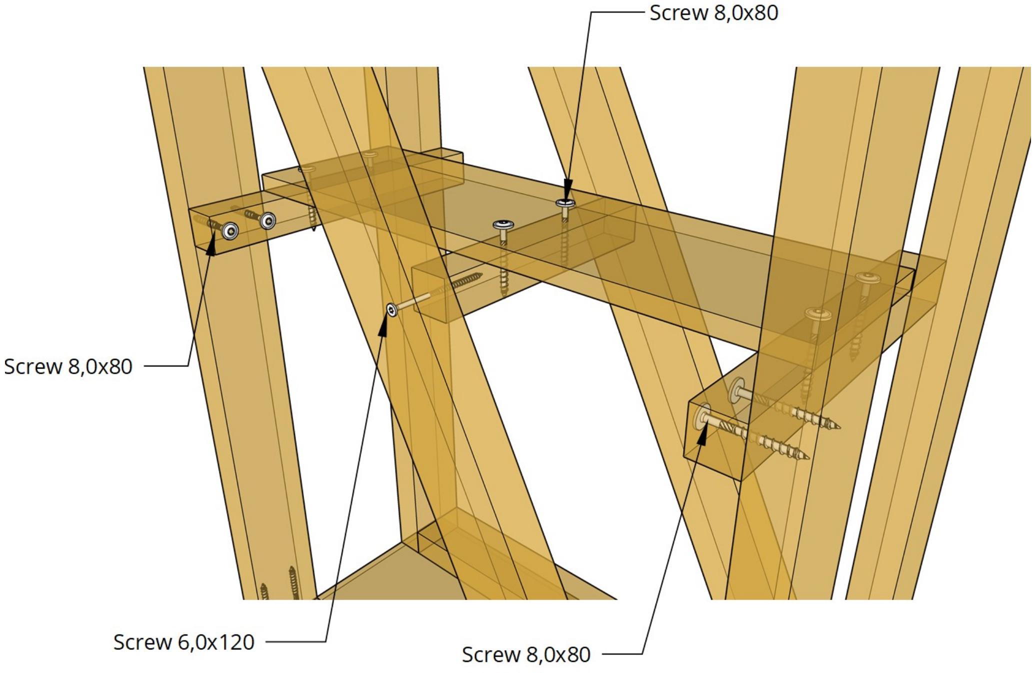
Shear strength of the connection

$$F_{v.Rd} := F_{v.Rd.1} \cdot 2 = 2.611\ kN$$

$$N_{R.d} := min\left(F_{v.Rd}, F_{ax.Rd}\right) = 2.611\ kN$$







Double vertical post load bearing in long term-duration loading

Wood type:	C24
Width of wooden element:	$b := 90\ mm$
Height of wooden element:	$h := 95\ mm$
Buckling length about x axis:	$l_x := 2.886\ m$
Buckling length about y axis:	$l_y := 0.962\ m$
Factor for support condition at the ends of the element:	$\mu_x := 1 \quad \mu_y := 1$
Factor for solid timber straightness:	$\beta_c := 0.2$
Charac. wood compression strength parallel to the grain:	$f_{c.0.k} := 21\ MPa$
Charac. wood modulus of elasticity parallel to the grain:	$E_{0.05} := 7400\ MPa$
Factor for duration loading and service:	$k_{mod} := 0.6$
Partial factor for material properties:	$\gamma_M := 1.30$

Design wood compression strength:

$$f_{c.0.d} := \frac{k_{mod} \cdot f_{c.0.k}}{\gamma_M} = 9.692\ \frac{N}{mm^2}$$

Cross-sectional area:

$$A := b \cdot h = 85.5\ cm^2$$

Second moment of area:

$$I_x := \frac{h^3 \cdot b}{12} = 643.031\ cm^4 \qquad I_y := \frac{h \cdot b^3}{12} = 577.125\ cm^4$$

Radius of gyration:

$$i_x := \sqrt{\frac{I_x}{A}} = 27.424\ mm \qquad i_y := \sqrt{\frac{I_y}{A}} = 25.981\ mm \qquad i := min\left(i_x, i_y\right) = 25.981\ mm$$

Design element length:

$$l_{ef.x} := \mu_x \cdot l_x = 2.886\ m \qquad l_{ef.y} := \mu_y \cdot l_y = 0.962\ m$$

Slenderness ratio:

$$\lambda_x := \frac{l_{ef.x}}{i_x} = 105.236 \qquad \lambda_y := \frac{l_{ef.y}}{i_y} = 37.027 \qquad \lambda := \max\left(\lambda_x, \lambda_y\right) = 105.236$$

Relative slenderness:

$$\lambda_{rel} := \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.784$$

Instability factor:

$$k := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + {\lambda_{rel}}^2\right) = 2.241$$

$$k_c := \frac{1}{k + \sqrt{k^2 - {\lambda_{rel}}^2}} = 0.278$$

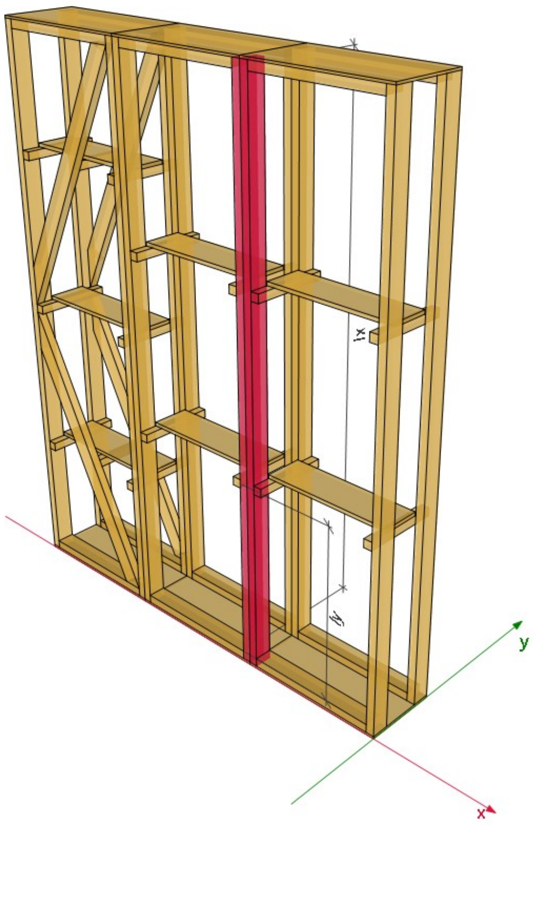
Design buckling strength:

$$k_c \cdot f_{c.0.d} = 2.696 \ \frac{N}{mm^2}$$

Compressive stress/Design buckling strength equation :

$$N_{c.k} := f_{c.0.k} \cdot A \cdot k_c = 49.936 \ kN$$

$$N_{c.d} := f_{c.0.d} \cdot A \cdot k_c = 23.048 \ kN$$



Single vertical post load bearing in long term-duration loading

Wood type:	C24
Width of wooden element:	$b := 45\ mm$
Height of wooden element:	$h := 95\ mm$
Buckling length about x axis:	$l_x := 2.886\ m$
Buckling length about y axis:	$l_y := 0.962\ m$
Factor for support condition at the ends of the element:	$\mu_x := 1 \quad \mu_y := 1$
Factor for solid timber straightness:	$\beta_c := 0.2$
Charac. wood compression strength parallel to the grain:	$f_{c.0.k} := 21\ MPa$
Charac. wood modulus of elasticity parallel to the grain:	$E_{0.05} := 7400\ MPa$
Factor for duration loading and service:	$k_{mod} := 0.6$
Partial factor for material properties:	$\gamma_M := 1.30$

Design wood compression strength:

$$f_{c.0.d} := \frac{k_{mod} \cdot f_{c.0.k}}{\gamma_M} = 9.692\ \frac{N}{mm^2}$$

Cross-sectional area:

$$A := b \cdot h = 42.75\ cm^2$$

Second moment of area:

$$I_x := \frac{h^3 \cdot b}{12} = 321.516\ cm^4 \qquad I_y := \frac{h \cdot b^3}{12} = 72.141\ cm^4$$

Radius of gyration:

$$i_x := \sqrt{\frac{I_x}{A}} = 27.424\ mm \qquad i_y := \sqrt{\frac{I_y}{A}} = 12.99\ mm \qquad i := min\left(i_x, i_y\right) = 12.99\ mm$$

Design element length:

$$l_{ef.x} := \mu_x \cdot l_x = 2.886\ m \qquad l_{ef.y} := \mu_y \cdot l_y = 0.962\ m$$

Slenderness ratio:

$$\lambda_x := \frac{l_{ef.x}}{i_x} = 105.236 \qquad \lambda_y := \frac{l_{ef.y}}{i_y} = 74.055 \qquad \lambda := \max\left(\lambda_x, \lambda_y\right) = 105.236$$

Relative slenderness:

$$\lambda_{rel} := \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.784$$

Instability factor:

$$k := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + \lambda_{rel}^{\ 2}\right) = 2.241$$

$$k_c := \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^{\ 2}}} = 0.278$$

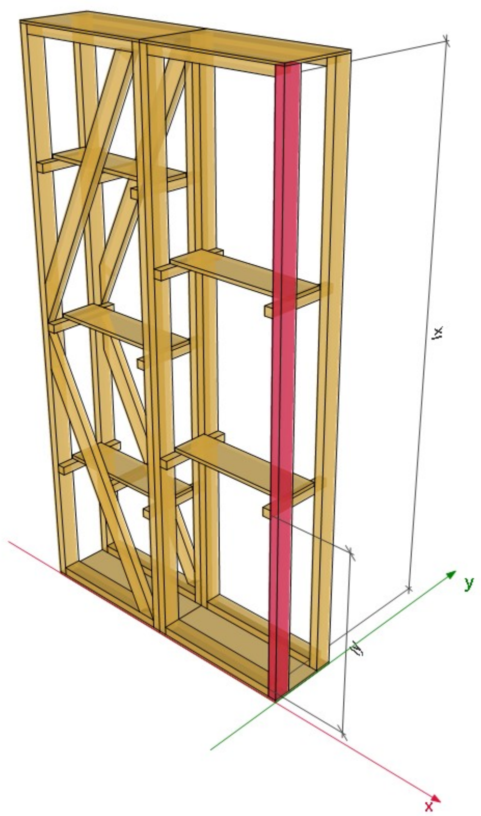
Design buckling strength:

$$k_c \cdot f_{c.0.d} = 2.696\ \frac{N}{mm^2}$$

Compressive stress/Design buckling strength equation :

$$N_{c.k} := f_{c.0.k} \cdot A \cdot k_c = 24.968\ kN$$

$$N_{c.d} := f_{c.0.d} \cdot A \cdot k_c = 11.524\ kN$$



Composite vertical post load bearing in long term-duration loading

Wood type:	C24
Width of wooden element:	$b := 45\ mm$
Height of wooden element:	$h := 95\ mm$
Thickness of plywood element:	$b_w := 12\ mm$
Width of plywood element:	$H := 400\ mm$
Clear width between two wood elements:	$h_w := 210\ mm$
Plywood between two posts area:	$A_w := b_w \cdot H = 0.005\ m^2$
For I-beam section [EC5 9.1.1 (8)]:	$b_{ef} := \frac{b_w}{2} = 6\ mm$
Buckling length about x axis:	$l_x := 2.886\ m$
Buckling length about y axis:	$l_y := 0.689\ m$
Factor for support condition at the ends of the element:	$\mu_x := 1 \quad \mu_y := 1$
Factor for solid timber straightness:	$\beta_c := 0.2$
Charac. wood bending strength:	$f_{m.k} := 24\ MPa$
Charac. wood compression strength parallel to the grain:	$f_{c.0.k} := 21\ MPa$
Charac. wood tension strength parallel to the grain:	$f_{t.0.k} := 14\ MPa$
Wood modulus of elasticity parallel to the grain:	$E_{0.05} := 7400\ MPa$
Wood modulus of mean elasticity parallel to the grain:	$E_{0.mean} := 11000\ MPa$
Charac. plywood compression strength:	$f_{p.c.90.k} := 24.3\ MPa$
Charac. plywood tensile strength :	$f_{p.t.90.k} := 35\ MPa$
Charac. plywood panel shear strength :	$f_{p.v.k} := 9.5\ MPa$
Charac. plywood rolling shear strength :	$f_{p.r.k} := 2.78\ MPa$
Plywood modulus of mean elasticity:	$E_{p.c.90.mean} := 6781\ MPa$
Plywood mean modulus of rigidity:	$G_{w.mean} := 620\ MPa$

Factor for duration loading and service: $k_{mod} := 0.7$

Factor for load shearing: $k_{sys} := 1.0$

Factor for solid timber - bending and axial tension: $k_h := min\left(\left(\frac{150\ mm}{h}\right)^{0.2}, 1.3\right) = 1.096$

Deformation factor for for solid timber class 1: $k_{def.f} := 0.6$

Deformation factor for for plywood class 1: $k_{def.w} := 0.8$

Partial factor for wood properties: $\gamma_{M.f} := 1.30$

Partial factor for plywood properties: $\gamma_{M.w} := 1.25$

Buckling resistance condition with beam laterally supported along its compression flange: $k_c := 1$

Geometric properties - transformed sections

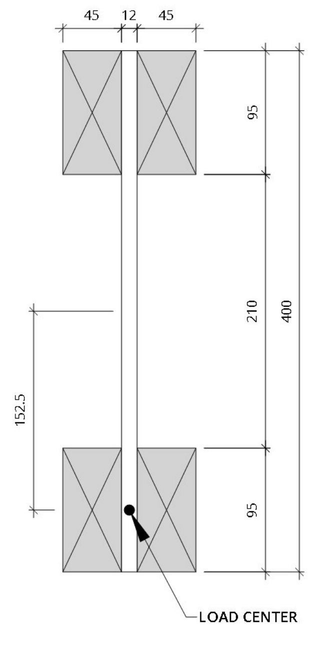
Transformed web thickness (into wood):

$$b_{w.tfd} := \left(b_w \cdot \frac{E_{p.c.90.mean}}{E_{0.mean}}\right) = 7.397\ mm$$

Second moment of area of the flanges (into wood):

$$I_{ef.f.x} := \frac{2 \cdot b \cdot H^3}{12} - \frac{2 \cdot b \cdot (H - 2 \cdot h)^3}{12} = \left(4.105 \cdot 10^8\right)\ mm^4$$

$$I_{ef.f.y} := \frac{2 \cdot b^3 \cdot H}{12} - \frac{2 \cdot b^3 \cdot (H - 2 \cdot h)}{12} = \left(2.886 \cdot 10^6\right)\ mm^4$$



Pic1. Analyed cross-section

Second moment of area of the web:

$$I_{ef.w.x} := \frac{b_{w.tfd} \cdot H^3}{12} = \left(3.945 \cdot 10^7\right)\ mm^4 \qquad I_{ef.w.y} := \frac{b_{w.tfd}^3 \cdot H}{12} = \left(1.349 \cdot 10^4\right)\ mm^4$$

Instantaneous second moment of the tranformed section:

$$I_{ef.x} := I_{ef.f.x} + I_{ef.w.x} = \left(4.5 \cdot 10^8\right)\ mm^4 \qquad I_{ef.y} := I_{ef.f.y} + I_{ef.w.y} = \left(2.899 \cdot 10^6\right)\ mm^4$$

Factor for quasi-permanent value of variable action:

$$\psi_2 := 0.3$$

Transformed web thickness (into wood):

$$b_{c.w.tfd} := \left(b_w \cdot \frac{E_{p.c.90.mean}}{E_{0.mean}}\right) \cdot \frac{1+\psi_2 \cdot k_{def.f}}{1+\psi_2 \cdot k_{def.w}} = 7.04\ mm$$

$$I_{c.ef.w.x} := \frac{b_{c.w.tfd} \cdot H^3}{12} = \left(3.754 \cdot 10^7\right)\ mm^4 \qquad I_{c.ef.w.y} := \frac{b_{c.w.tfd}^{\ 3} \cdot H}{12} = \left(1.163 \cdot 10^4\right)\ mm^4$$

$$I_{c.ef.x} := I_{ef.f.x} + I_{c.ef.w.x} = \left(4.481 \cdot 10^8\right)\ mm^4 \qquad I_{c.ef.y} := I_{ef.f.y} + I_{c.ef.w.y} = \left(2.897 \cdot 10^6\right)\ mm^4$$

Cross-sectional area:

$$A := b \cdot h \cdot 4 + A_w = 219\ cm^2$$

Second moment of area:

$$I_x := I_{c.ef.x} \qquad I_x = \left(4.481 \cdot 10^8\right)\ mm^4 \qquad I_y := I_{c.ef.y} \qquad I_y = \left(2.897 \cdot 10^6\right)\ mm^4$$

$$W_x := \frac{b_{c.w.tfd} \cdot H^2 + 2 \cdot b \cdot H^2 - 2 \cdot b \cdot (H - 2 \cdot h)^2}{6} = \left(1.926 \cdot 10^6\right)\ mm^3$$

$$W_y := \frac{b_{c.w.tfd}^{\ 2} \cdot H + (2 \cdot b)^2 \cdot H - (2 \cdot b)^2 \cdot (H - 2 \cdot h)}{6} = \left(2.598 \cdot 10^5\right)\ mm^3$$

Radius of gyration:

$$i_x := \sqrt{\frac{I_x}{A}} = 143.04\ mm \qquad i_y := \sqrt{\frac{I_y}{A}} = 11.502\ mm \qquad i := min\left(i_x, i_y\right) = 11.502\ mm$$

Design element length:

$$l_{ef.x} := \mu_x \cdot l_x = 2.886\ m \qquad l_{ef.y} := \mu_y \cdot l_y = 0.689\ m$$

Slenderness ratio:

$$\lambda_x := \frac{l_{ef.x}}{i_x} = 20.176 \qquad \lambda_y := \frac{l_{ef.y}}{i_y} = 59.903 \qquad \lambda := \max\left(\lambda_x, \lambda_y\right) = 59.903$$

Relative slenderness:

$$\lambda_{rel.x} := \frac{\lambda_x}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 0.342$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.016$$

Instability factor:

$$k_x := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.x} - 0.3\right) + {\lambda_{rel.x}}^2\right) = 0.563$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 1.087$$

$$k_{c.x} := \frac{1}{k_x + \sqrt{{k_x}^2 - {\lambda_{rel.x}}^2}} = 0.991$$

$$k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.678$$

Axial strength in compression:

$$f_{c.0.d} := \frac{k_{mod} \cdot k_{sys} \cdot f_{c.0.k}}{\gamma_{M.f}} = 11.308 \; MPa$$

Axial strength in tension:

$$f_{t.0.d} := \frac{k_{mod} \cdot k_h \cdot k_{sys} \cdot f_{t.0.k}}{\gamma_{M.f}} = 8.26 \; MPa$$

Compressive stress/Design buckling strength equation :

$$N_{c.k.x} := f_{c.0.k} \cdot A \cdot k_{c.x} = 455.559 \; kN$$

$$N_{c.k.y} := f_{c.0.k} \cdot A \cdot k_{c.y} = 311.629 \; kN$$

$$N_{c.d.x} := f_{c.0.d} \cdot A \cdot k_{c.x} = 245.301 \; kN$$

$$N_{c.d.y} := f_{c.0.d} \cdot A \cdot k_{c.y} = 167.8 \; kN$$

$$\sigma_{c.0.d.x} := \frac{N_{c.d.x}}{A} = 11.201 \; MPa$$

$$\sigma_{c.0.d.y} := \frac{N_{c.d.y}}{A} = 7.662 \; MPa$$

Design wood bending strength:

$$f_{m.d} := \frac{k_{mod} \cdot k_h \cdot k_{sys} \cdot f_{m.k}}{\gamma_{M.f}} = 14.159 \; MPa$$

Moment about the axis x-x:

$$e_y := \frac{H}{2} - \frac{h}{2} = 152.5 \; mm$$

$$e_x := 0 \; mm$$

$$M_{x.d} := N_{c.d.x} \cdot e_y = 37.408 \; kN \cdot m$$

$$M_{y.d} := N_{c.d.y} \cdot e_x = 0 \; kN \cdot m$$

Bending strength of the web in compression:

$$f_{c.w.d} := \frac{k_{mod} \cdot k_{sys} \cdot f_{p.c.90.k}}{\gamma_{M.w}} = 13.608 \; MPa$$

Bending stress in top and the bottom flange:

$$\sigma_{f.c.max.d.x} := \frac{M_{x.d}}{I_{c.ef.x}} \cdot \frac{H}{2} = 16.697 \ MPa \qquad \sigma_{f.c.max.d.y} := \frac{M_{y.d}}{I_{c.ef.y}} \cdot \frac{b}{2} = 0 \ MPa$$

Bending strength of the web in tension:

$$f_{c.w.d} := \frac{k_{mod} \cdot k_{sys} \cdot f_{p.t.90.k}}{\gamma_{M.w}} = 19.6 \ MPa$$

Bending stress in the web:

$$\sigma_{w.c.max.d.x} := \frac{M_{x.d}}{I_{ef.x}} \cdot \frac{H}{2} \cdot \left(\frac{E_{p.c.90.mean}}{E_{0.mean}} \right) = 10.249 \ MPa \qquad \sigma_{w.c.max.d.y} := \frac{M_{y.d}}{I_{ef.y}} \cdot \frac{b}{2} \cdot \left(\frac{E_{p.c.90.mean}}{E_{0.mean}} \right) = 0 \ MPa$$

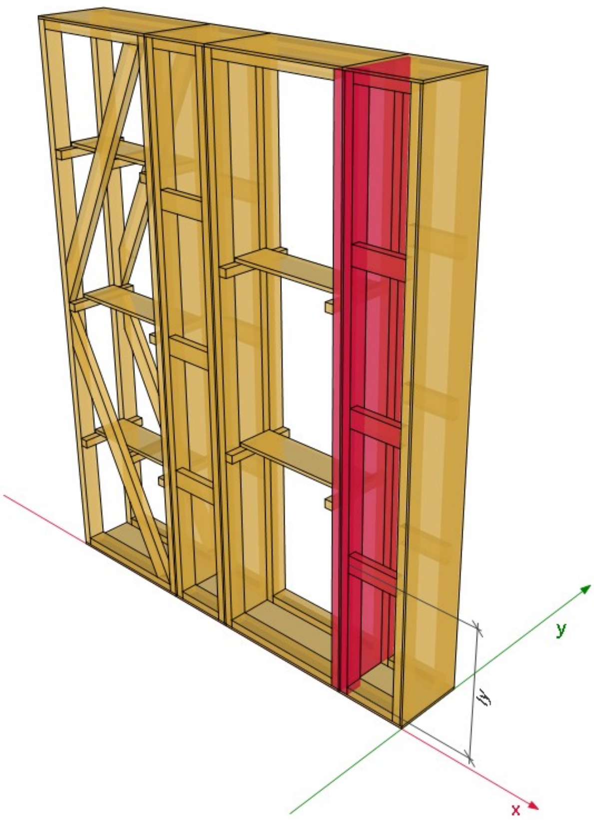
Axial stress in the top and bottom flange:

$$\sigma_{f.c.max.d.c} := \frac{M_{x.d}}{I_{c.ef.x}} \cdot e_y = 12.731 \ MPa$$

Design buckling strength:

$$k_c \cdot f_{c.0.d} = 11.308 \ \frac{N}{mm^2}$$

Checking combined compression and bending stress condition about the x-x axis:



$$a := \frac{\sigma_{c.0.d.x}}{k_{c.x} \cdot f_{c.0.d}} + \frac{\sigma_{f.c.max.d.x}}{f_{m.d}} = 2.179$$

$$N_{c.d.x.final} := \frac{N_{c.d.x}}{a} = 112.563 \ kN$$

$$b := \frac{\sigma_{c.0.d.y}}{k_{c.y} \cdot f_{c.0.d}} = 1$$

$$N_{c.d.y.final} := \frac{N_{c.d.y}}{b} = 167.8 \ kN$$

$$N_{c.d.max} := N_{c.d.x.final} = 112.563 \ kN$$

Composite vertical post near window load bearing in long term-duration loading

Wood type:	C24
Width of wooden element:	$b := 45\ mm$
Height of wooden element:	$h := 95\ mm$
Thickness of plywood element:	$b_w := 21\ mm$
Width of plywood element:	$H := 400\ mm$
Clear width between two wood elements:	$h_w := 210\ mm$
Plywood between two posts area:	$A_w := b_w \cdot H = 84\ cm^2$
For I-beam section [EC5 9.1.1 (8)]:	$b_{ef} := \frac{b_w}{2} = 10.5\ mm$
Buckling length about x axis (minus minimal lintel height):	$l_x := 2.886\ m - 0.4\ m = 2.486\ m$
Buckling length about y axis:	$l_y := 1.007\ m$
Factor for support condition at the ends of the element:	$\mu_x := 1 \quad \mu_y := 1$
Factor for solid timber straightness:	$\beta_c := 0.2$
Charac. wood bending strength:	$f_{m.k} := 24\ MPa$
Charac. wood compression strength parallel to the grain:	$f_{c.0.k} := 21\ MPa$
Charac. wood tension strength parallel to the grain:	$f_{t.0.k} := 14\ MPa$
Wood modulus of elasticity parallel to the grain:	$E_{0.05} := 7400\ MPa$
Wood modulus of mean elasticity parallel to the grain:	$E_{0.mean} := 11000\ MPa$
Charac. plywood compression strength:	$f_{p.c.90.k} := 25\ MPa$
Charac. plywood tensile strength :	$f_{p.t.90.k} := 36\ MPa$
Charac. plywood panel shear strength :	$f_{p.v.k} := 9.5\ MPa$
Charac. plywood rolling shear strength :	$f_{p.r.k} := 2.59\ MPa$
Plywood modulus of mean elasticity:	$E_{p.c.90.mean} := 7642\ MPa$
Plywood mean modulus of rigidity:	$G_{w.mean} := 620\ MPa$

Factor for duration loading and service: $k_{mod} := 0.7$

Factor for load shearing: $k_{sys} := 1.0$

Factor for solid timber - bending and axial tension: $k_h := min\left(\left(\frac{150\ mm}{h}\right)^{0.2}, 1.3\right) = 1.096$

Deformation factor for for solid timber class 1: $k_{def.f} := 0.6$

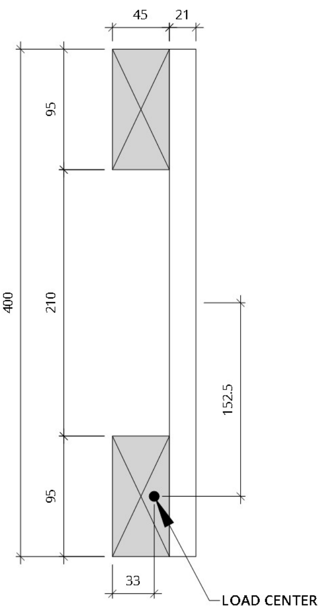
Deformation factor for for plywood class 1: $k_{def.w} := 0.8$

Partial factor for wood properties: $\gamma_{M.f} := 1.30$

Partial factor for plywood properties: $\gamma_{M.w} := 1.25$

Buckling resistance condition with beam laterally supported along its compression flange: $k_c := 1$

Geometric properties - transformed sections



Pic1. Analyed cross-section

Transformed web thickness (into wood):

$$b_{w.tfd} := \left(b_w \cdot \frac{E_{p.c.90.mean}}{E_{0.mean}}\right) = 14.589\ mm$$

Second moment of area of the flanges (into wood):

$$I_{ef.f.x} := \frac{b \cdot H^3}{12} - \frac{b \cdot (H - 2 \cdot h)^3}{12} = \left(2.053 \cdot 10^8\right)\ mm^4$$

$$I_{ef.f.y} := \frac{b^3 \cdot H}{12} - \frac{b^3 \cdot (H - 2 \cdot h)}{12} = \left(1.443 \cdot 10^6\right)\ mm^4$$

Second moment of area of the web:

$$I_{ef.w.x} := \frac{b_{w.tfd} \cdot H^3}{12} = \left(7.781 \cdot 10^7\right)\ mm^4 \qquad I_{ef.w.y} := \frac{b_{w.tfd}^{\ 3} \cdot h}{12} = \left(2.458 \cdot 10^4\right)\ mm^4$$

Instanteous second moment of the tranformed section:

$$I_{ef.x} := I_{ef.f.x} + I_{ef.w.x} = \left(2.831 \cdot 10^8\right)\ mm^4 \qquad I_{ef.y} := I_{ef.f.y} + I_{ef.w.y} = \left(1.467 \cdot 10^6\right)\ mm^4$$

Factor for quasi-permanent value of variable action:

$$\psi_2 := 0.3$$

Transformed web thickness (into wood):

$$b_{c.w.tfd} := \left(b_w \cdot \frac{E_{p.c.90.mean}}{E_{0.mean}} \right) \cdot \frac{1 + \psi_2 \cdot k_{def.f}}{1 + \psi_2 \cdot k_{def.w}} = 13.883\ mm$$

$$I_{c.ef.w.x} := \frac{b_{c.w.tfd} \cdot H^3}{12} = \left(7.404 \cdot 10^7\right)\ mm^4 \qquad I_{c.ef.w.y} := \frac{b_{c.w.tfd}^{\ 3} \cdot H}{12} = \left(8.92 \cdot 10^4\right)\ mm^4$$

$$I_{c.ef.x} := I_{ef.f.x} + I_{c.ef.w.x} = \left(2.793 \cdot 10^8\right)\ mm^4 \qquad I_{c.ef.y} := I_{ef.f.y} + I_{c.ef.w.y} = \left(1.532 \cdot 10^6\right)\ mm^4$$

Cross-sectional area:

$$A := b \cdot h \cdot 2 + A_w = 169.5\ cm^2$$

Second moment of area:

$$I_x := I_{c.ef.x} \qquad I_x = \left(2.793 \cdot 10^8\right)\ mm^4 \qquad I_y := I_{c.ef.y} \qquad I_y = \left(1.532 \cdot 10^6\right)\ mm^4$$

$$W_x := \frac{b_{c.w.tfd} \cdot H^2 + 2 \cdot b \cdot H^2 - 2 \cdot b \cdot (H - 2 \cdot h)^2}{6} = \left(2.109 \cdot 10^6\right)\ mm^3$$

$$W_y := \frac{b_{c.w.tfd}^{\ 2} \cdot H + (2 \cdot b)^2 \cdot H - (2 \cdot b)^2 \cdot (H - 2 \cdot h)}{6} = \left(2.693 \cdot 10^5\right)\ mm^3$$

Radius of gyration:

$$i_x := \sqrt{\frac{I_x}{A}} = 128.37\ mm \qquad i_y := \sqrt{\frac{I_y}{A}} = 9.507\ mm \qquad i := min\left(i_x, i_y\right) = 9.507\ mm$$

Design element length:

$$l_{ef.x} := \mu_x \cdot l_x = 2.486\ m \qquad l_{ef.y} := \mu_y \cdot l_y = 1.007\ m$$

Slenderness ratio:

$$\lambda_x := \frac{l_{ef.x}}{i_x} = 19.366 \qquad \lambda_y := \frac{l_{ef.y}}{i_y} = 105.921 \qquad \lambda := \max\left(\lambda_x, \lambda_y\right) = 105.921$$

Relative slenderness:

$$\lambda_{rel.x} := \frac{\lambda_x}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 0.328 \qquad \lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.796$$

Instability factor:

$$k_x := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.x} - 0.3\right) + {\lambda_{rel.x}}^2\right) = 0.557 \qquad k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 2.263$$

$$k_{c.x} := \frac{1}{k_x + \sqrt{{k_x}^2 - {\lambda_{rel.x}}^2}} = 0.994 \qquad k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.275$$

Axial strength in compression:

$$f_{c.0.d} := \frac{k_{mod} \cdot k_{sys} \cdot f_{c.0.k}}{\gamma_{M.f}} = 11.308\ MPa$$

Axial strength in tension:

$$f_{t.0.d} := \frac{k_{mod} \cdot k_h \cdot k_{sys} \cdot f_{t.0.k}}{\gamma_{M.f}} = 8.26\ MPa$$

Compressive stress/Design buckling strength equation :

$$N_{c.k.x} := f_{c.0.k} \cdot A \cdot k_{c.x} = 353.701\ kN \qquad N_{c.k.y} := f_{c.0.k} \cdot A \cdot k_{c.y} = 97.828\ kN$$

$$N_{c.d.x} := f_{c.0.d} \cdot A \cdot k_{c.x} = 190.454\ kN \qquad N_{c.d.y} := f_{c.0.d} \cdot A \cdot k_{c.y} = 52.676\ kN$$

$$\sigma_{c.0.d.x} := \frac{N_{c.d.x}}{A} = 11.236\ MPa \qquad \sigma_{c.0.d.y} := \frac{N_{c.d.y}}{A} = 3.108\ MPa$$

Design wood bending strength:

$$f_{m.d} := \frac{k_{mod} \cdot k_h \cdot k_{sys} \cdot f_{m.k}}{\gamma_{M.f}} = 14.159\ MPa$$

Moment about the axis x-x:

$$e_y := \frac{H}{2} - \frac{h}{2} = 152.5\ mm \qquad e_x := \frac{b_w}{2} + \frac{b}{2} = 33\ mm$$

$$M_{x.d} := N_{c.d.x} \cdot e_y = 29.044\ kN \cdot m \qquad M_{y.d} := N_{c.d.y} \cdot e_x = 1.738\ kN \cdot m$$

Bending strength of the web in compression:

$$f_{c.w.d} := \frac{k_{mod} \cdot k_{sys} \cdot f_{p.c.90.k}}{\gamma_{M.w}} = 14\ MPa$$

Bending stress in top and the bottom flange:

$$\sigma_{f.c.max.d.x} := \frac{M_{x.d}}{I_{c.ef.x}} \cdot \frac{H}{2} = 20.797 \; MPa \qquad \sigma_{f.c.max.d.y} := \frac{M_{y.d}}{W_y} = 6.454 \; MPa$$

Bending strength of the web in tension:

$$f_{c.w.d} := \frac{k_{mod} \cdot k_{sys} \cdot f_{p.t.90.k}}{\gamma_{M.w}} = 20.16 \; MPa$$

Bending stress in the web:

$$\sigma_{w.c.max.d.x} := \frac{M_{x.d}}{I_{ef.x}} \cdot \frac{H}{2} \cdot \left(\frac{E_{p.c.90.mean}}{E_{0.mean}}\right) = 14.256 \; MPa \qquad \sigma_{w.c.max.d.y} := \frac{M_{y.d}}{W_y} \cdot \left(\frac{E_{p.c.90.mean}}{E_{0.mean}}\right) = 4.484 \; MPa$$

Axial stress in the top and bottom flange:

$$\sigma_{f.c.max.d.c} := \frac{M_{x.d}}{I_{c.ef.x}} \cdot e_y = 15.858 \; MPa$$

Design buckling strength:

$$k_c \cdot f_{c.0.d} = 11.308 \; \frac{N}{mm^2}$$

Checking combined compression and
bending stress condition about the x-x axis:

$$a := \frac{\sigma_{c.0.d.x}}{k_{c.x} \cdot f_{c.0.d}} + \frac{\sigma_{f.c.max.d.x}}{f_{m.d}} = 2.469$$

$$N_{c.d.x.final} := \frac{N_{c.d.x}}{a} = 77.145 \; kN$$

$$b := \frac{\sigma_{c.0.d.y}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{f.c.max.d.y}}{f_{m.d}} = 1.456$$

$$N_{c.d.y.final} := \frac{N_{c.d.y}}{b} = 36.184 \; kN$$

$$N_{c.d.max} := N_{c.d.y.final} = 36.184 \; kN$$