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

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

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

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

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

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