Appendix A

Loads

A1. Live Load

Table shows how the buildings vary from different live load cases in accordance with NS-EN1995-1-1, Table 6.2.

Category	Load
A – Residence Area	2,0 kN/m^2
B – Office Area	3,0 kN/m^2
D – Commercial Area	5,0 kN/m^2

In addition, live load values are given by *Sweco* (Appendix D). This load is in compliance with Eurocode values. First two floors are assumed for commercial use, remaining floors are for office or residential use.
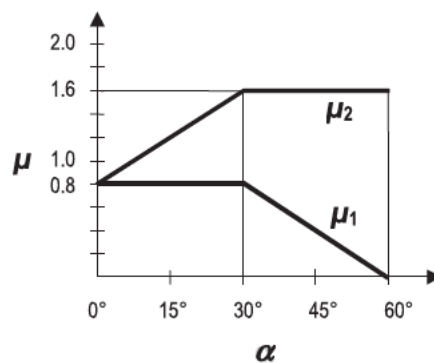
A2. Snow Load

Snow load is calculated in NS-EN1991-1-3. Location, height and shape of roof decides the characteristic value of snow load. Calculation of snow load on roof is given in Equation (EC1-1-3, 5.1). It is worth to mention that s has been used as s_k in this assignment.

$$s = \mu_i C_e C_t s_k$$

where,

μ_i	is shape factor, equal 0,8 for flat roof (Table 5.2)
C_e	is the exposure factor, equal 1
C_t	is the thermal coefficient, equal 1
s_k	is characteristic snow load

In figure below, one can see how shape factor vary with roof angle α. Figure is given in (EC1-1-1-3, Figure 5.1).

Characteristic snow load in National Annex (NA.4.1, Table NA.4.1(901)) located in Oslo is varying in different heights above sea level.

Height (meters above sea level, m.s.l)	$s_{k,0}$
0 – 150 m.s.l	$3,5\ kN/m^2$
151 – 250 m.s.l	$4,5\ kN/m^2$
251 – 350 m.s.l	$5,5\ kN/m^2$
> 350 m.s.l	$6,5\ kN/m^2$

Sweco has given value for snow load as characteristic value on **2,8 kN/m²** in Appendix D. This value is the one used for this project.

A3. Calculation of static wind load

General remarks:

Units used in script:

- Length/height: $[m]$

- Force: $[N]$

- Velocity: $\left[\dfrac{m}{s}\right]$

- Density: $\left[\dfrac{kg}{m^3}\right]$

All equations- and chapter-references are from the EC1-1-4.

Geometry of the building:

Height: $h := 66$

Width: $b := 32$

Depth: $d := 19.2$

Basic values:

The fundamental value of basic wind velocity: $v_{b.0} := 22$ (Table NA.4)

Directional factor: $c_{dir} := 1$ (Chapter 4.2(2), NOTE2)

Season factor: $c_{season} := 1$ (Chapter 4.2(2), NOTE3)

Probability factor:
For characteristic wind combination (EN1990, eq.(6.14), SLS) we set return period, T=50, which gives cprob=1. (this is an irreversible load combination happening rarely, every 50 years, therefor damage should be limited when it happens).

$c_{prob} := 1$

Mean wind:

Height above sea level at construction site:

$$H := 0$$

Height above sea level where level correction begins:
(When c_alt = 1)

$$H_0 := 900 \qquad \text{(Table NA.4 (901.2))}$$

The height above sea level where max. level correction is reached
(When c_alt is at its max.):

$$H_{topp} := 1500 \quad \text{(Table NA.4 (901.2))}$$

Threshold value for wind velocity:

$$v_0 := 30 \qquad \text{(NA.4.2(2)P (901.1))}$$

Factor for the wind increasing with the height over the sea:

$$c_{alt} := 1 \qquad \text{(Table NA.4(901.3))}$$

Basic wind velocity: $\qquad v_b := c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b.0} \quad$ (eq. NA.4.1)

$$v_b = 22$$

Referance height:
As a conservative assumption we make the windload uniformly distributed over the height of the building with the peak value. This means the only referance height needed is the total height of the building, both for internal and external pressure.

$$z_e := h \qquad \text{(Figure 7.2)}$$

$$z_i := h \qquad \text{(Figure 7.2)}$$

Terrain category:	$TK := 4$	(Table 4.1)

Orography factor: $\qquad c_0 := 1 \qquad$ (Chapter 4.3.1, NOTE 1)

Roughness length:

$$z_0 := \left. \begin{array}{l} \text{if } TK = 0 \\ \quad \| \, 0.003 \\ \text{else if } TK = 1 \\ \quad \| \, 0.01 \\ \text{else if } TK = 2 \\ \quad \| \, 0.05 \\ \text{else if } TK = 3 \\ \quad \| \, 0.3 \\ \text{else if } TK = 4 \\ \quad \| \, 1 \end{array} \right| = 1 \qquad \text{(Chapter 4.3.2)}$$

Minimum height:

$$z_{min} := \left. \begin{array}{l} \text{if } TK = 0 \\ \quad \| \, 2 \\ \text{else if } TK = 1 \\ \quad \| \, 2 \\ \text{else if } TK = 2 \\ \quad \| \, 4 \\ \text{else if } TK = 3 \\ \quad \| \, 8 \\ \text{else if } TK = 4 \\ \quad \| \, 16 \end{array} \right| = 16 \qquad \text{(Table 4.1)}$$

Max. height: $\qquad z_{max} := 200 \qquad$ (Chapter 4.3.2)

Terrain factor: $\qquad k_r := 0.24 \qquad$ (Table NA.4.1)

Roughness factor:
$$c_r(z) := \text{if } (z \geq z_{min}) \wedge (z \leq z_{max})$$
$$\left\| k_r \cdot \ln\left(\frac{z}{z_0}\right) \right.$$
$$\text{else if } (z \leq z_{min})$$
$$\left\| c_r(z_{min}) \right.$$
(Eq. 4.4)

Mean wind velocity:
$$v_m(z) := c_0 \cdot c_r(z) \cdot v_b$$
(Eq. 4.3)

$$v_m(z_e) = 22.121$$

Wind turbulence:

Turbulensfaktor:
$$k_I := 1$$
(Eq. 4.7)

Standard deviation:
$$\sigma_v := k_r \cdot v_b \cdot k_I$$
(Eq. 4.6)

Turbulence intensity:
$$I_v(z) := \text{if } (z \geq z_{min}) \wedge (z \leq z_{max})$$
$$\left\| \frac{\sigma_v}{v_m(z)} \right.$$
$$\text{else if } (z \leq z_{min})$$
$$\left\| I_v(z_{min}) \right.$$
(Eq. 4.7)

Peak velocity pressure:

Air density: $\quad\quad\quad\quad \rho := 1.25 \quad\quad\quad\quad\quad\quad$ (Chapter 4.5)

Peak factor: $\quad\quad\quad\quad k_p := 3.5 \quad\quad\quad\quad\quad\quad$ (Chapter NA.4.4)

Mean velocity pressure: $\quad q_m(z) := 0.5 \cdot \rho \cdot v_m(z)^2 \quad$ (Chapter NA. 4.5)

Peak velocity pressure: $\quad q_p(z) := (1 + 2 \cdot k_p \cdot I_v(z)) \cdot q_m(z) \quad$ (Eq. NA. 4.8)
(4.5, eq. 4.8)

$$q_p(z_e) = 816.851$$

Wind pressure on surfaces:

External pressure coefficients for buildings:

External wall surfaces: (Table 7.1)

$$c_{pe.10.A} := -1.2$$

$$c_{pe.10.B} := -0.8$$

$$c_{pe.10.C} := -0.5$$

$$c_{pe.10.D} := 0.8$$

$$c_{pe.10.E} := -0.7 \quad \text{(varies)}$$

External flat roof: (Table 7.2)

$$c_{pe.10.F} := -1.8$$

$$c_{pe.10.G} := -1.2$$

$$c_{pe.10.H} := -0.7$$

$$c_{pe.10.I} := 0.2$$

Internal pressure coefficients for buildings is ignored

Wind pressure when the pressure is at the longest surface:

Longest surfaces:

Sone D: $\quad q_{p.D} := q_p\left(z_e\right) \cdot c_{pe.10.D} = 653.481 \quad$ (pressure)

Sone E: $\quad q_{p.E} := q_p\left(z_e\right) \cdot c_{pe.10.E} = -571.796 \quad$ (suction)

Short surfaces:

Sone A and B:

$$q_{p.A} := q_p\left(z_e\right) \cdot c_{pe.10.A} = -980.221 \quad \text{(suction)}$$

Wind pressure when the pressure is at the shortest side:

Longest sides:

Sone A and B:

$$q_{p.A} := q_p\left(z_e\right) \cdot c_{pe.10.A} = -980.221 \quad \text{(suction)}$$

Short sides:

Sone D: $\quad q_{p.D} := q_p\left(z_e\right) \cdot c_{pe.10.D} = 653.481 \quad$ (pressure)

Sone E: $\quad q_{p.E} := q_p\left(z_e\right) \cdot c_{pe.10.E} = -571.796 \quad$ (suction)

Wind pressure on roof (neglected in this project):

Sone F/G/H/I - Pressure on sone I (Suction on rest)
$$q_{p.I} := q_p\left(z_e\right) \cdot c_{pe.10.I} = 163.37$$

A4. Load Combinations

All load combinations are determined in accordance with NS-EN-1990. Only a set of most decisive load combinations are chosen out.

Combination equation as in Eurocode (6.10a and 6.10b)

	Permanent	**Dominant**	**Non-dominant**
6.10a	$\gamma_{Gj,sup} \cdot g_{kj,sup}$	$\gamma_{Q,1} \cdot \psi_{0,1} \cdot q_{k,1}$	$\gamma_{Q,i} \cdot \psi_{0,i} \cdot q_{k,i}$
6.10b	$\xi \cdot \gamma_{Gj,sup} \cdot g_{kj,sup}$	$\gamma_{Q,1} \cdot q_{k,1}$	$\gamma_{Q,i} \cdot \psi_{0,i} \cdot q_{k,i}$

Firstly, we define our load- cases and factors in project.

Load Cases	**Symbol**
Dead Load	g
Snow Load	s
Wind Load	w
Live Load	q

Load Factor	**Value**
ξ	*0,89*
$\gamma_{Gj,sup}$	*1,35*
$\gamma_{Q,1}$ *(unfavourable)*	*1,5*
$\gamma_{Q,i}$ *(unfavourable)*	*1,5*
$\psi_{0,1}$ *(snow/live)*	*0,7*
$\psi_{0,i}$ *(snow/live)*	*0,7*
$\psi_{0,1}$ *(wind)*	*0,6*
$\psi_{0,i}$ *(wind)*	*0,6*

We present now the combinations used for this project.

	Permanent	**Dominant**	**Non-dominant**
Combination 1 and 2	*g*	*w*	*q+s*
Combination 3 and 4	*g*	*q*	*w+s*
Combination 5	*g*	*q*	*s*

Wind load is divided into w1 and w2 where

w1   is pressure on longitudinal surface
w2   is pressure on transversal surface

Load combinations will now be portrayed for both, Ultimate Limit State and Serviceability Limit State with all factors involved (defined in tables above). Combinations are expressed in tables, where alle are in accordance with Eurocode.

ULS Combinations

	Permanent	**Dominant**	**Non-dominant**
Combination 1a	$1,35g$	$(1,5 \cdot 0,6)w1$	$(1,5 \cdot 0,7)(q + s)$
Combination 1b	$(0,89 \cdot 1,35)g$	$1,5w1$	$(1,5 \cdot 0,7)(q + s)$
Combination 2a	$1,35g$	$1,5 \cdot 0,6)w2$	$(1,5 \cdot 0,7)(q + s)$
Combination 2b	$(0,89 \cdot 1,35)g$	$1,5w2$	$(1,5 \cdot 0,7)(q + s)$
Combination 3a	$1,35g$	$(1,5 \cdot 0,7)q$	$(1,5 \cdot 0,6)w1 + (1,5 \cdot 0,7)s$
Combination 3b	$(0,89 \cdot 1,35)g$	$1,5q$	$(1,5 \cdot 0,6)w1 + (1,5 \cdot 0,7)s$
Combination 4a	$1,35g$	$(1,5 \cdot 0,6)q$	$(1,5 \cdot 0,6)w2 + (1,5 \cdot 0,7)s$
Combination 4b	$(0,89 \cdot 1,35)g$	$1,5q$	$(1,5 \cdot 0,6)w2 + (1,5 \cdot 0,7)s$
Combination 5a	$1,35g$	$(1,5 \cdot 0,7)q$	$(1,5 \cdot 0,7)s$
Combination 5b	$(0,89 \cdot 1,35)g$	$1,5q$	$(1,5 \cdot 0,7)s$

*$0,89 \cdot 1,35 = 1,20$
*$1,5 \cdot 0,7 = 1,05$
*$1,5 \cdot 0,6 = 0,90$

SLS Combinations

	Permanent	**Dominant**	**Non-dominant**
Combination 1	g	$w1$	$0,7(q + s)$
Combination 2	g	$w2$	$0,7(q + s)$
Combination 3	g	q	$0,6w1 + 0,7s$
Combination 4	g	q	$0,6w1 + 0,7s$
Combination 5	g	q	$0,7s$

Appendix B

Procedures in Software

B1. Procedure of *Dynamo Sandbox* for Parametric Study

In this appendix, the *Dynamo* script is followed step by step. First out is the illustration of how the footprint base is defined.
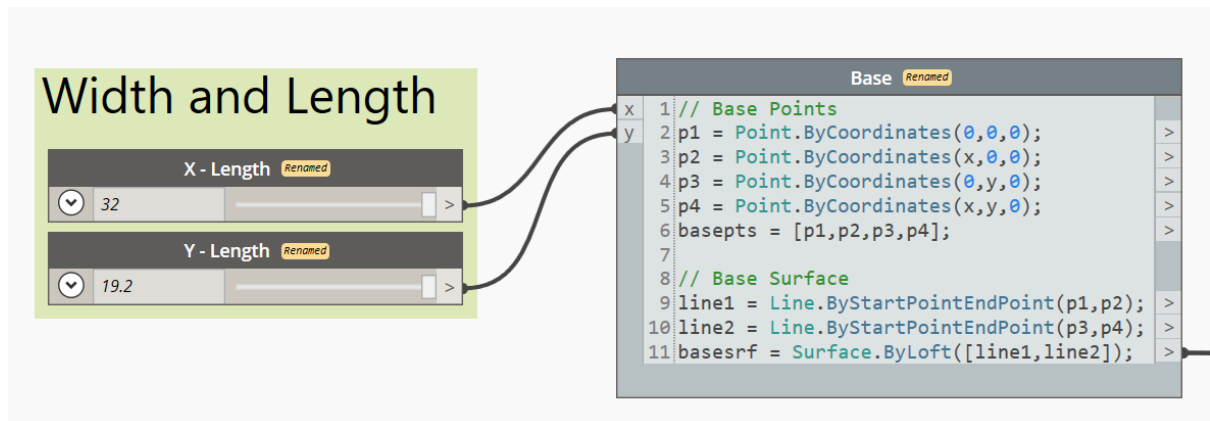

Figure 1: Base surface

The base surface is defining the footprint area. Width and length can be decided parametrically as shown. This code boxes are defining and used for our geometrical structure. Further on, the structure is made with lines and panels shown below.
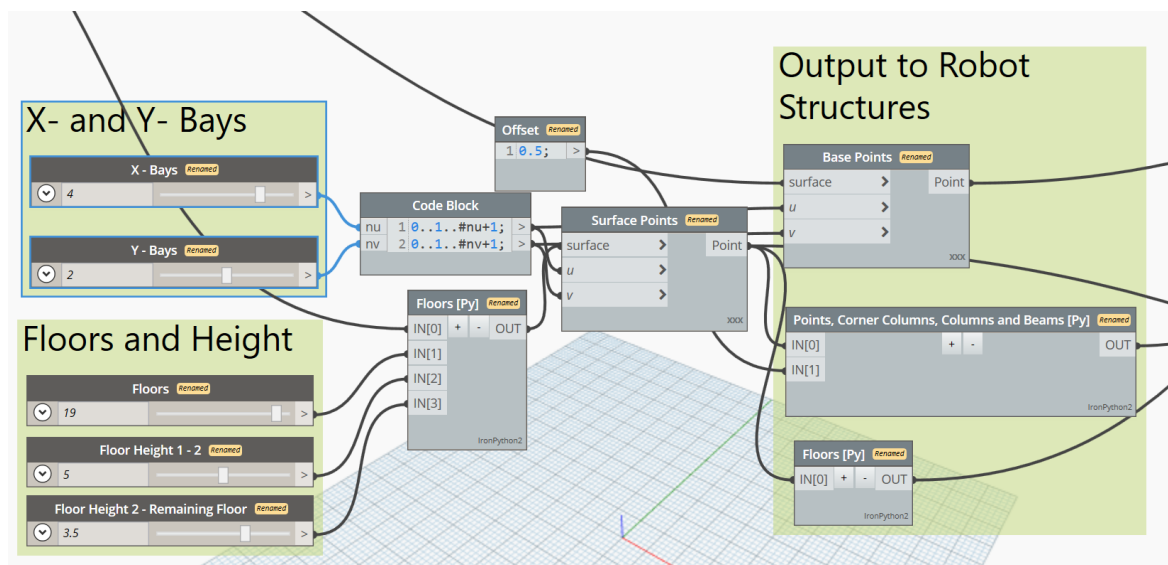

Figure 2: Geometry of structure

The geometry of structure is decided by defining number of floors, number of x- and y-bays and the height of commercial floors and residential. This is resulting in separate floors between bays, separate beams between columns and continuously columns. Lines (beams and columns) and surfaces (floors) are scripted in Python as shown in Figure above. Example of an output would look like.
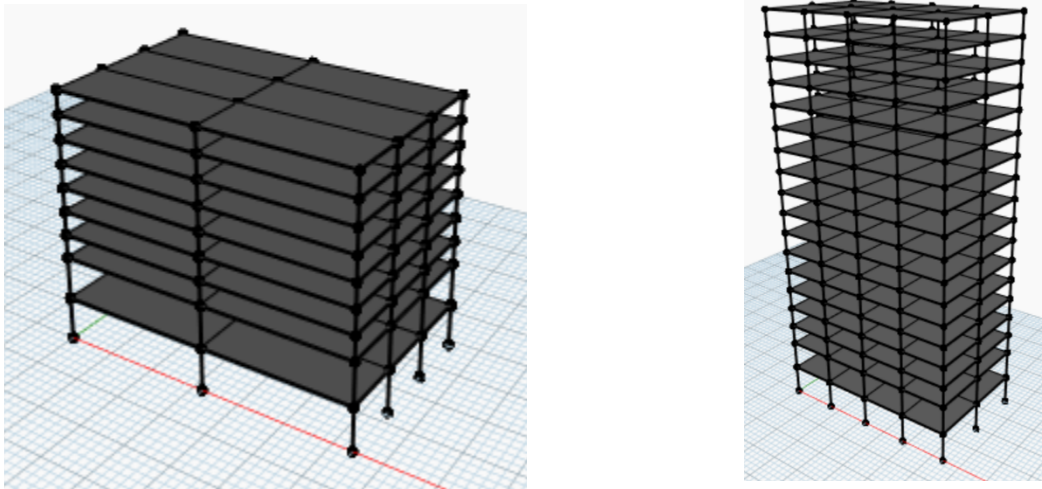

Figure 3: Geometrical output

This figure shows how one can change bays, floors and floor height easily in structure. By defining the geometry of the structure as shown, the next procedure is to connect and transfer to *Robot Structures* by using the package *Structural Analysis for Dynamo*. Diagonals are manually modelled in *Robot Structures*. The following next steps are converting the geometry over to *Robot*.

In *Dynamo* with help of the package, we create analytical nodes, bars and panels. Surface can be done as shown in figure below. It is also possible to assign thickness to the surface by defining one in *Robot*, and it will automatically be shown up in *Dynamo*.
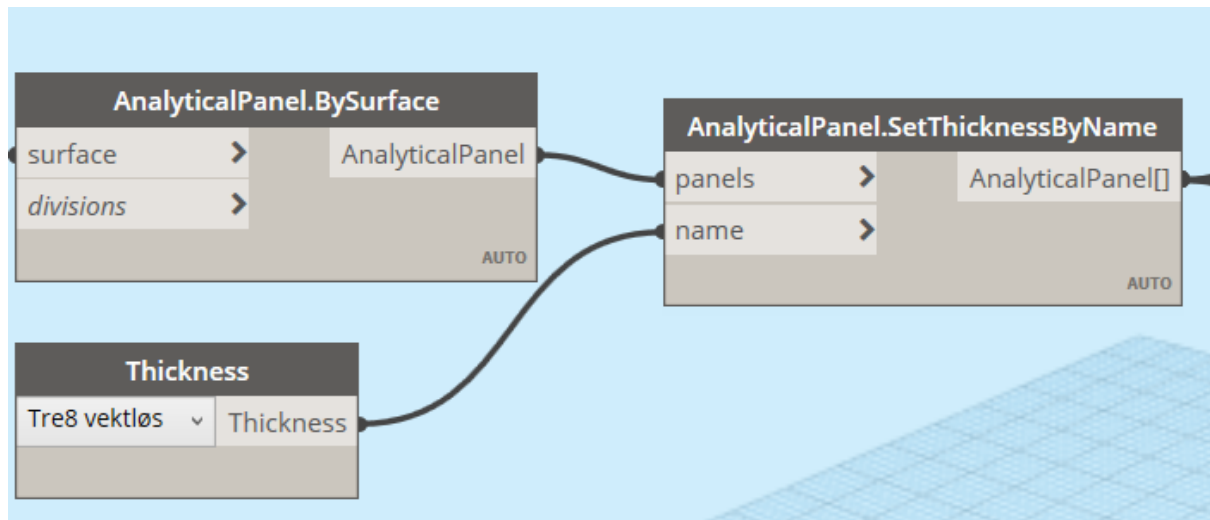
Figure 4: Floors from Dynamo to Robot

Further releases on the surfaces are also manually done in *Robot Structures*. There is no attribute in the structural package to linear release slabs, which is needed for floors.

Following the surfaces, beams and columns are done similarly. However, it is possible to give releases on lines, and this is also done in *Dynamo*. Base supports are also defined in *Dynamo*, as it is possible by help of the package to convert nodes into one of boundary conditions. This is shown in a figure.
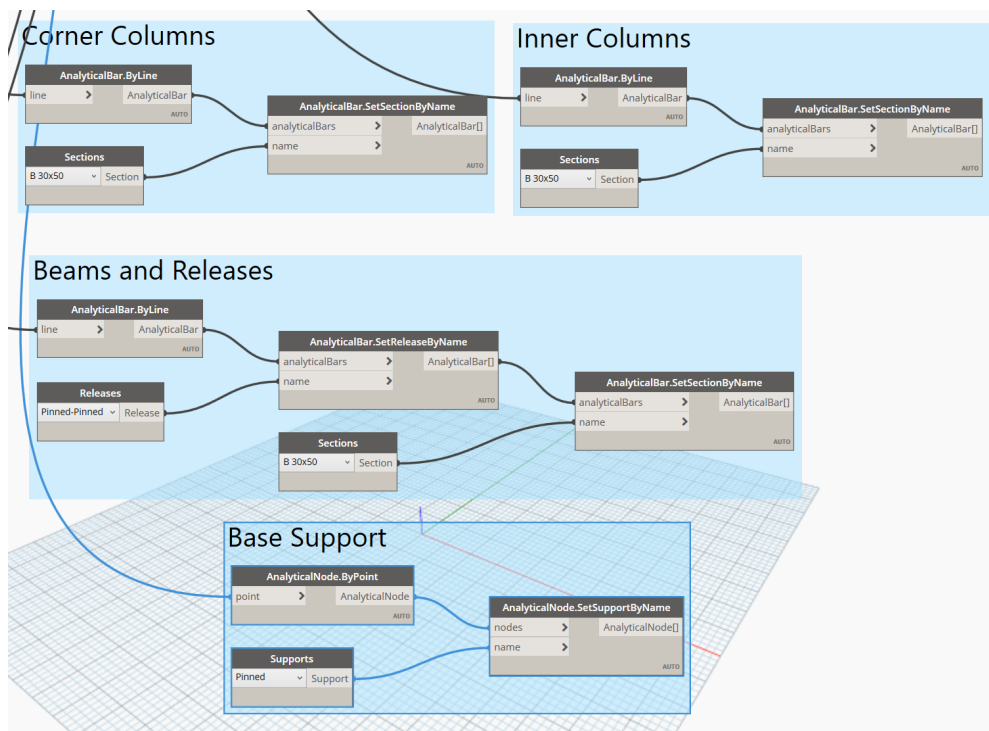

Figure 5: Code for transfer beams, columns and nodes from Dynamo to Robot

Appendix B

As for deciding thickness in Robot for floors, by adding cross section for beams and columns it will automatically be listed in Dynamo. The same procedure goes for the boundary conditions. Another important parameter is the rigid link, which is done with help of Python script for separating master and slave nodes. This is shown here.
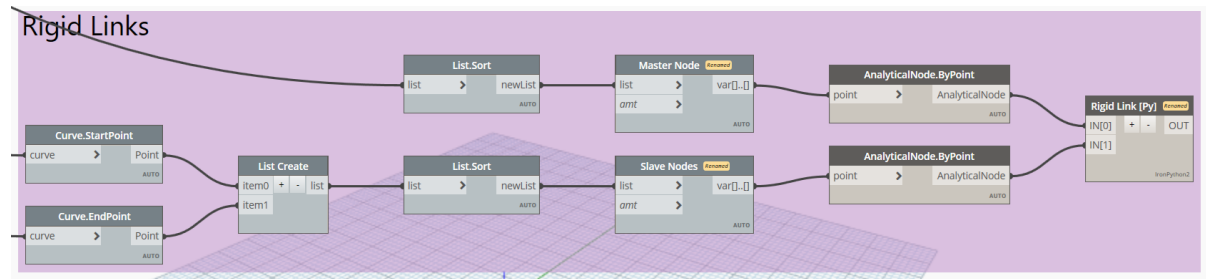

Figure 6 Rigid links in Dynamo

By doing all this, the structure is quite good modelled and transferred to *Robot*. Further, the diagonals are manually modelled in *Robot* as mentioned. However, it is important to know that the load cases for dead, snow and live is also done in *Dynamo* with help of the package, and the visual programming would look like.

After the structural model is introduced to *Robot*, the load analysis is given by adding it to *Dynamo*. Notation is that wind load and added mass as concrete, slab weight and green roof is done in *Robot* manually. The combinations of the loads are also not done in *Dynamo*. Live loads, dead loads and snow load is given in *Dynamo*.
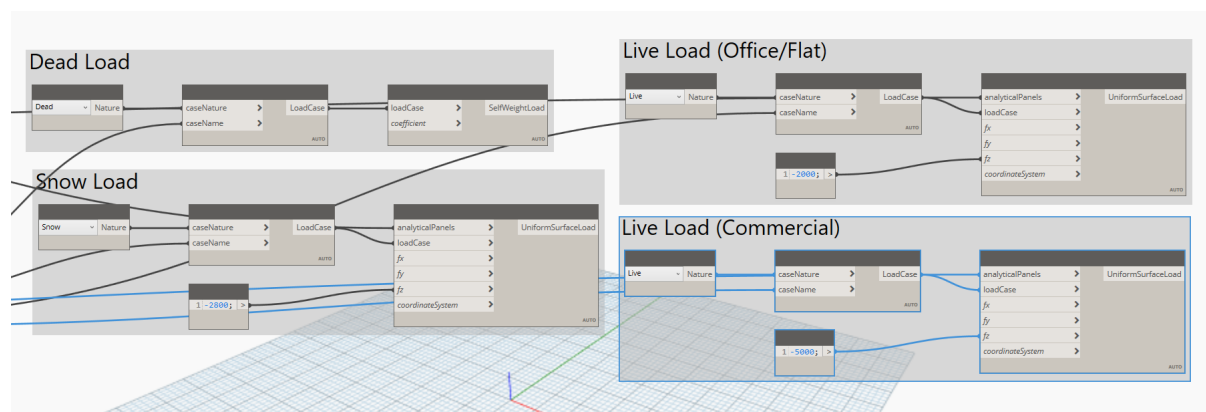

Figure 7: Load cases in Dynamo

Ending this Appendix, we clearly see that this is quite helpful for the efficiency for a parametric study. One can easily do modifications to obtain best optimized structure.

B2. Seismic design in *Robot Structural*

The seismic design will be performed in *Robot Structural*. In this software there are two options as to how the seismic calculation should be performed:

- Lateral force method of analysis
- Response spectrum method

The response spectrum method is based upon the modal analysis and is more accurate than the Lateral force method of analysis and is thus chosen for this thesis. In this Appendix the method for how to perform this type of analysis in *Robot Structural* will be presented.
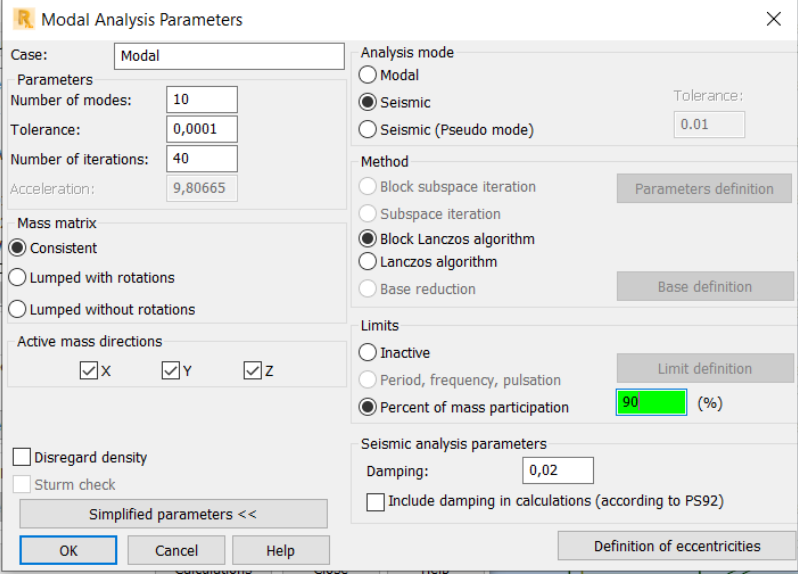
For seismic analysis based on the response spectrum method, all data is defined the same way as in modal analysis. Additionally, parameters required by a specific national code to establish the response spectrum shape must be specified. Calculations and results are the same as those for spectral analysis.

Step 1: Defining modal analysis.
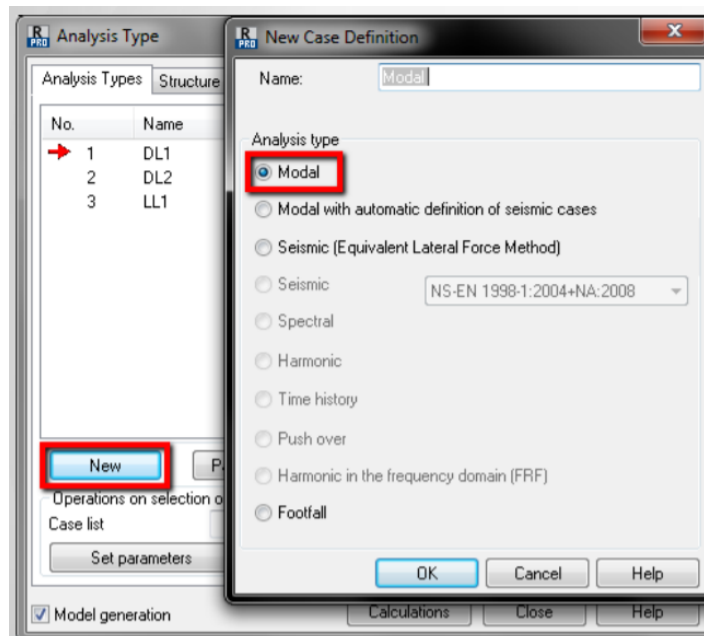
The modal analysis is defined as a load case under "Analysis Type". For the seismic design it is convenient to define the no. of modes based on the demand that over 90% of mass participation is accounted for.

Step 2: Load to mass conversion.

Defining added masses manually using "Load Definition", or by converting existing load cases to masses.

Step 3: Defining seismic analysis
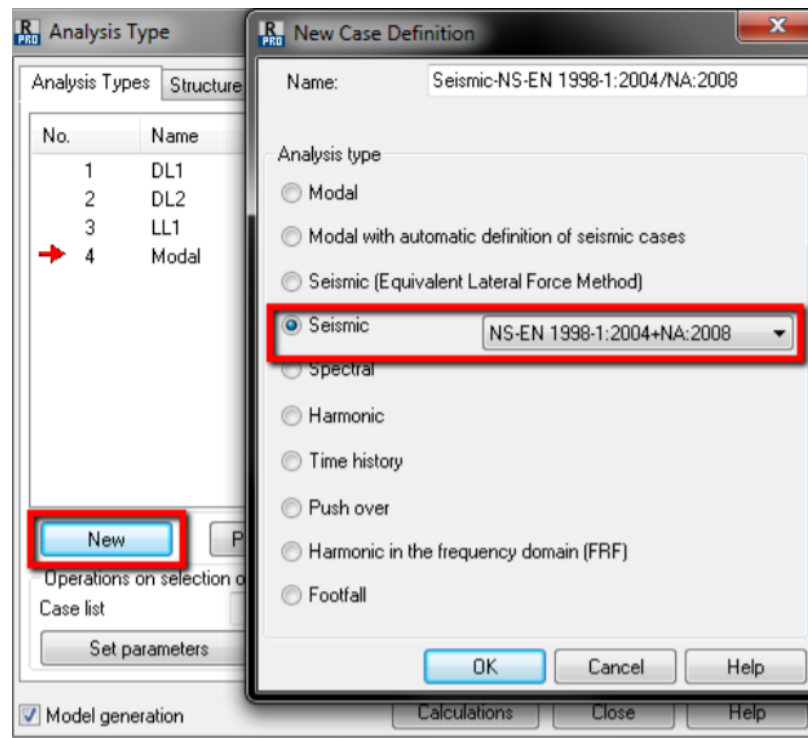
The seismic analysis is defined as a new load case.
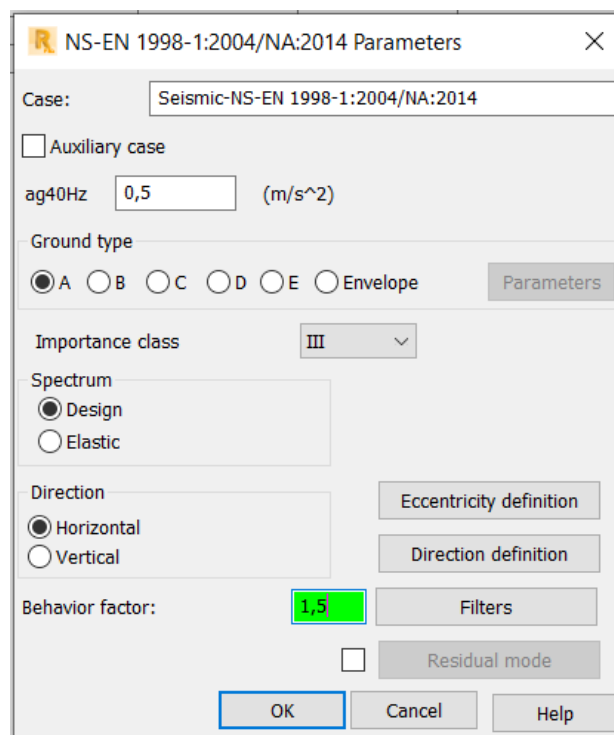
Step 4: Set combination signs.

In case of using signed quadratic combinations, it will be necessary to set main modes for each of the directions. Usually, the main criterion to select such modes is their contribution to participation mass for given direction. This contribution can be checked in Dynamic Analysis Results with appropriate columns added. In this thesis we have only looked at the seismic effect in x and y direction and neglected the z direction.
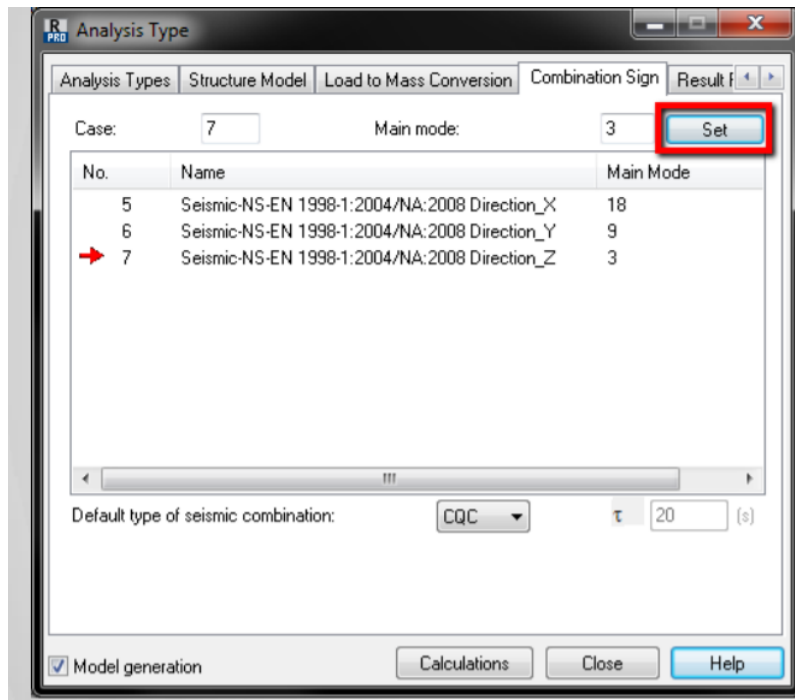
Step 5: Making seismic code combinations.

Defining design combinations (manual or automatic ones) considering static load cases and dynamic combinations.
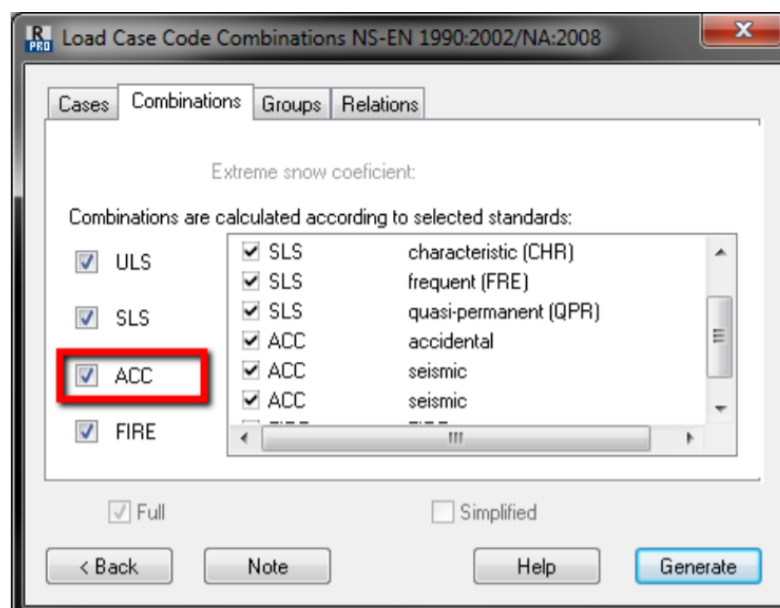
Step 6: Calculations

Run calculations.
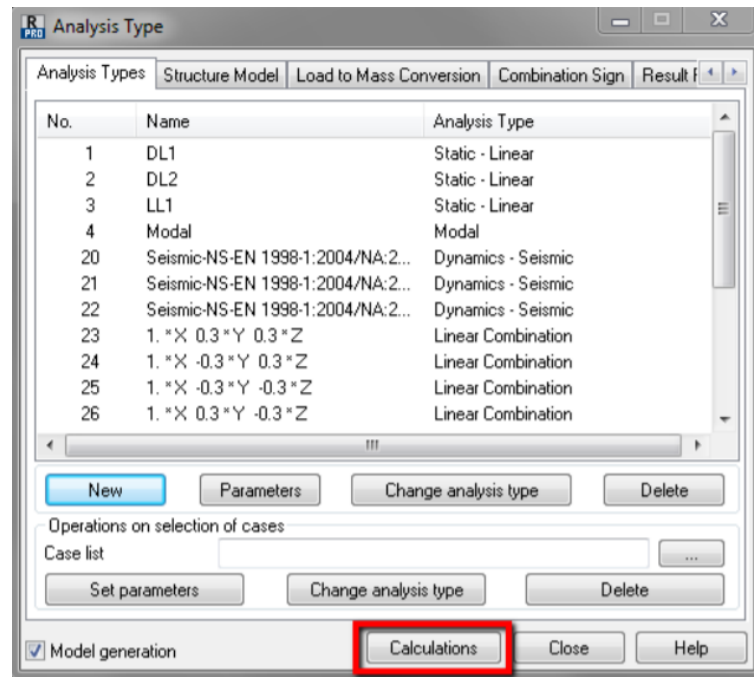
Step 7: Analyse the results
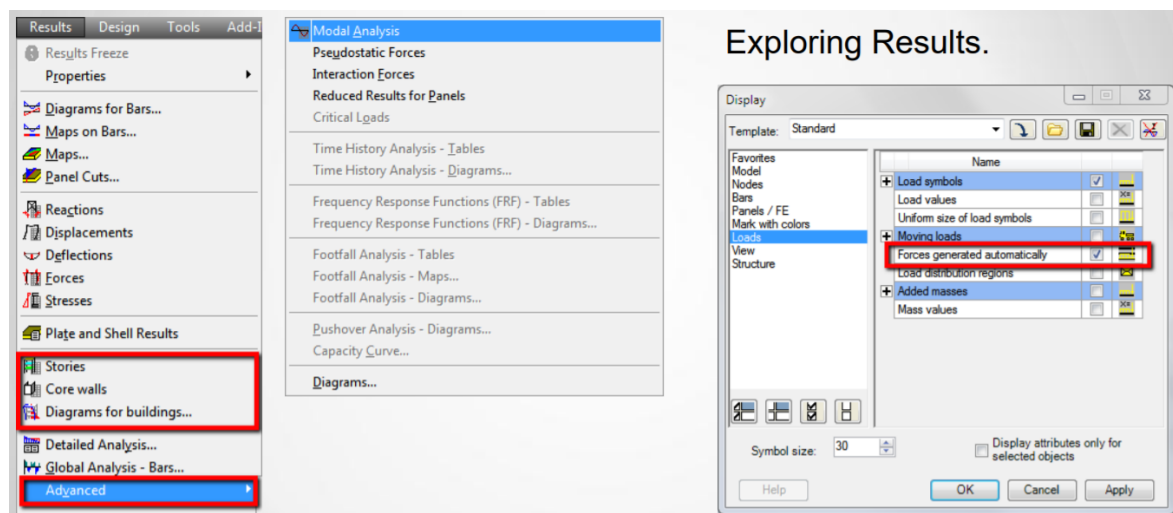
Appendix C

Manual Calculations

C1. Pre-liminary design of slabs

Formulas for this calculation has been extrated from NS-EN 1995-1-1 (2004).
Material properties for Glulam elements have been extracted from EN 14080 (2013).
Material properties for LVL Kerto Q elements have been extracted from Metsa Wood's catalogue.

Units used in script:

- Dimensions/lengths: $[mm]$

- Forces: $[N]$

- Moments: $[Nmm]$

- Stresses/strengths: $\left[\dfrac{N}{mm^2}\right]$

- Areas: $[mm^2]$

- 2nd moment of inertia: $[mm^4]$

- Densities: $\left[\dfrac{kg}{m^3}\right]$

Appendix C

General data:

Span-length:	$l := 8500$
Width of 1 slab element:	$b := 2400$
Distance between Glulam members:	$CC := 565$
Service class:	$Sc := 2$
Load-duration:	$Ld := \text{“Medium”}$

Loads:

ULS:

$$q_{Ed.ULS} := 9.9 \cdot 10^{-3} \cdot CC = 5.594$$

SLS-characteristic:

$$q_{Ed.SLS1} := 7 \cdot 10^{-3} \cdot CC = 3.955$$

> The combinations that have given these design loads are the ones that can be found in Appendix A4.
> ULS COMB5 has been used for the ULS load
> SLS COMB5 has been used for the SLS characteristic load

Internal forces:
Formulas corresponding to simply supported floors.

Bending moments

$$M_{Ed.ULS} := \frac{1}{8} \cdot q_{Ed.ULS} \cdot l^2 = 5.052 \cdot 10^7$$

$$M_{Ed.SLS1} := \frac{1}{8} \cdot q_{Ed.SLS1} \cdot l^2 = 3.572 \cdot 10^7$$

Shear forces

$$V_{Ed.ULS} := \frac{1}{2} \cdot q_{Ed.ULS} \cdot l = 2.377 \cdot 10^4$$

$$V_{Ed.SLS1} := \frac{1}{2} \cdot q_{Ed.SLS1} \cdot l = 1.681 \cdot 10^4$$

Material properties:

Safety-factor for Glulam and LVL: $\gamma_M := 1.15$
NS-EN 1995-1-1, table NA.2.3

Webs: GL30c

Dimensions:

Middle beams:	$h_w := 405$	$b_w := 66$
Edge beams:	$h_{w.edge} := 405$	$b_{w.edge} := 140$

Factors:

Modification factor: $k_{mod.web} := 0.8$
NS-EN 1995-1-1, table 3.1

Time-property factor: $k_{def.web} := 0.8$
NS-EN 1995-1-1, table 3.2

Cracking factor: $k_{cr.w} := 0.8$

Bending strength:

$$f_{mk.w} := 30$$

$$f_{md.w} := \frac{f_{mk.w}}{\gamma_M} \cdot k_{mod.web} = 20.87$$

Shear strength:

$$f_{vk.w} := 3.5$$

$$f_{vd.w} := \frac{f_{vk.w}}{\gamma_M} \cdot k_{mod.web} = 2.435$$

Axial compr. strength:

$$f_{c.0.k.w} := 24.5$$

$$f_{c.0.d.w} := \frac{f_{c.0.k.w}}{\gamma_M} \cdot k_{mod.web} = 17.043$$

Appendix C

Axial tension strength:	$f_{t.0.k.w} := 19.5$

$$f_{t.0.d.w} := \frac{f_{t.0.k.w}}{\gamma_M} \cdot k_{mod.web} = 13.565$$

Mean young`s modulus: $E_{0.mean.w} := 13000$

Mean shear modulus: $G_{web.mean} := 650$

Characteristic density: $\rho_{k.w} := 390$

Mean density: $\rho_{mean.w} := 430$

Top flange: LVL Kerto Q

Height: $h_{t.f} := 43$

The width of the flange will be an effective width that is chosen according to NS-EN 1995-1-1, table 9.1, where shear lag and plate-buckling will be accounted for. This will be done later in the calculation-process.
Properties for LVL found in EN 13986 (2004).

Factors:

Modification factor:
NS-EN 1995-1-1, table 3.1 $k_{mod.t.f} := 0.8$

Time-property factor:
NS-EN 1995-1-1, table 3.2 $k_{def.t.f} := 1$

Bending strength: $f_{mk.t.f} := 36$

$$f_{md.t.f} := \frac{f_{mk.t.f}}{\gamma_M} \cdot k_{mod.t.f} = 25.043$$

Appendix C

Shear strengths:	$f_{v.0.edge.k.tf} := 4.5$
	$f_{vd.f.tf} := \dfrac{f_{v.0.edge.k.tf}}{\gamma_M} \cdot k_{mod.t.f} = 3.13$
	$f_{v.0.flat.k.tf} := 1.3$
	$f_{v.0.flat.d.tf} := \dfrac{f_{v.0.flat.k.tf}}{\gamma_M} \cdot k_{mod.t.f} = 0.904$
Axial compr. strength:	$f_{c.0.k.tf} := 26$
	$f_{c.0.d.tf} := \dfrac{f_{c.0.k.tf}}{\gamma_M} \cdot k_{mod.t.f} = 18.087$
Axial tension strength:	$f_{t.0.k} := 26$
	$f_{t.0.d.f} := \dfrac{f_{t.0.k}}{\gamma_M} \cdot k_{mod.t.f} = 18.087$
Young`s modulus:	$E_{0.mean.t.f} := 10500$
Shear modulus:	$G_{t.f.mean} := 600$
Mean density:	$\rho_{mean.t.f} := 510$

Bottom flange: LVL, Kerto Q

Height: $\qquad h_{b.f} := 61$

Factors:

    Modification factor: $\qquad k_{mod.b.f} := 0.8$
    NS-EN 1995-1-1, table 3.1

    Time-property factor: $\qquad k_{def.b.f} := 1$
    NS-EN 1995-1-1, table 3.2

Bending strength: $\qquad f_{mk.b.f} := 36$

$$f_{md.b.f} := \frac{f_{mk.b.f}}{\gamma_M} \cdot k_{mod.b.f} = 25.043$$

Shear strengths: $\qquad f_{v.0.edge.k.bf} := 4.5$

$$f_{v0.edge.d.bf} := \frac{f_{v.0.edge.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 3.13$$

$$f_{v.0.flat.k.bf} := 1.3$$

$$f_{v.0.flat.d.bf} := \frac{f_{v.0.flat.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 0.904$$

Axial compr. strength: $\qquad f_{c.0.k.bf} := 26$

$$f_{c.0.d.bf} := \frac{f_{c.0.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 18.087$$

Axial tension strength: $\qquad f_{t.0.k.bf} := 19.5$

$$f_{t.0.d.bf} := \frac{f_{t.0.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 13.565$$

Young`s modulus: $\qquad E_{0.mean.b.f} := 10500$

Shear modulus: $\qquad G_{b.f.mean} := 600$

Densities: $\qquad \rho_{k.b.f} := 510 \qquad \rho_{mean.b.f} := 510$

Effective width of top and bottom flange:

Acc. to NS-EN 1995-1-1, table 9.1

The effective flange widths found in table 9.1 is the maximum allowable flange widths that we can have so that we avoid shear lag and plate buckling.
Another thing we should avoid is the overlapping of flange widths. If the effective width used is larger than the center distance of webs, then they overlap, which is not allowed. Therefor, we choose flange widths below the demand in table 9.1, and also below the centre distance.
EC5-1-1, §9.1.2.(5): Unrestrained flange width is smaller than twice the plate buckling value in Table 9.1 -> no detailed buckling investigation required.

Tensile flange:

Must account for shear lag

$$b_{ef.tensile} := min\left(0.1 \cdot l, 20 \cdot h_{b.f}\right) = 850$$

Compression flange:

both shear lag and plate buckling must be accounted for

$$b_{ef.compr} := min\left(0.1 \cdot l, 20 \cdot h_{t.f}\right) = 850$$

- Max. effective width top flange:

  Middle beams: $\qquad b_{ef.t} := b_{ef.compr} + b_w = 916$

  Edge beams: $\qquad b_{ef.t.edge} := 0.5 \cdot b_{ef.compr} + b_{w.edge} = 565$

- Max. effective width bottom flange:

  Middle beams: $\qquad b_{ef.b} := b_{ef.tensile} + b_w = 916$

  Edge beams: $\qquad b_{ef.b.edge} := 0.5 \cdot b_{ef.tensile} + b_{w.edge} = 565$

- Chosen width of bottom flange:

    Middle beams: $b_{b.f} := 565$

    Edge beams: $b_{b.f.edge} := 282$

- Chosen width of top flange:

    Middle beams: $b_{t.f} := 565$

    Edge beams: $b_{t.f.edge} := 282$
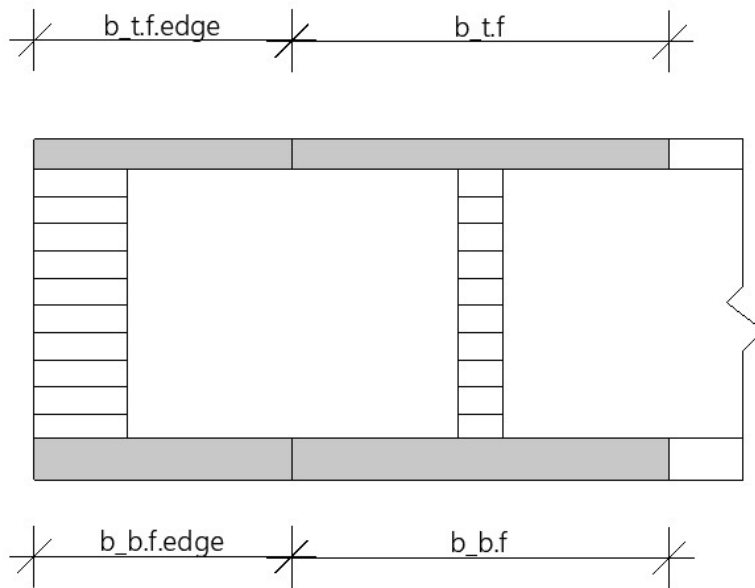
- Control of shear lag and plate buckling:

Middle beams:

$$\text{if } \left(b_{b.f} \leq b_{ef.tensile}\right) \wedge \left(b_{t.f} \leq b_{ef.compr}\right) \bigg| = \text{“OK”}$$
$$\quad \bigg\| \text{“OK”}$$
$$\text{else}$$
$$\quad \bigg\| \text{“Not OK”}$$

Edge beams:

$$\text{if } \left(b_{b.f.edge} \leq b_{ef.tensile}\right) \wedge \left(b_{t.f.edge} \leq b_{ef.compr}\right) \bigg| = \text{“OK”}$$
$$\quad \bigg\| \text{“OK”}$$
$$\text{else}$$
$$\quad \bigg\| \text{“Not OK”}$$

The chosen flange-widths will not encounter any shear lag nor plate buckling.

- Control of overlapping flanges:

Top flange:

Middle beams:
$$\text{if } b_{t.f} > CC \quad \left| = \text{``No overlap''} \right.$$
$$\left\| \text{``Overlap''} \right.$$
$$\text{else}$$
$$\left\| \text{``No overlap''} \right.$$

Edge beams:
$$\text{if } b_{t.f.edge} > \frac{CC}{2} \quad \left| = \text{``No overlap''} \right.$$
$$\left\| \text{``Overlap''} \right.$$
$$\text{else}$$
$$\left\| \text{``No overlap''} \right.$$

Bottom flange:

Middle beams:
$$\text{if } b_{b.f} > CC \quad \left| = \text{``No overlap''} \right.$$
$$\left\| \text{``Overlap''} \right.$$
$$\text{else}$$
$$\left\| \text{``No overlap''} \right.$$

Edge beams:
$$\text{if } b_{b.f.edge} > \frac{CC}{2} \quad \left| = \text{``No overlap''} \right.$$
$$\left\| \text{``Overlap''} \right.$$
$$\text{else}$$
$$\left\| \text{``No overlap''} \right.$$

After finding effective flange width and controlled it for overlapping, the sections may be interpreted as a thin-flanged beam, and the controls needed can be done acc. to NS-EN 1995-1-1.

Only the thin-flanged beams for the middle webs will be controlled, assuming that the edge beams will be ok since they have bigger cross-sections and less moment acting on them. But, the stiffness of the edge beams will be extracted for the purpose of finding a more accurate stiffness of the slab-element.

ULS: Instantaneous

<mark>After finding effective flange width and controlled it for overlapping, we may interpret the section as a thin-flanged beam.

Design checks for flanges according to EC5, 9.1.2(7)
Design checks for web according to EC5, 9.1.2(9)</mark>

Calculation of stresses according to Annex B in EC5-1-1

- Cross sectional parameters:

2nd moment of area:

Middle beams:
$$I_1 := \frac{1}{12} \cdot b_{t.f} \cdot h_{t.f}{}^3 = 3.743 \cdot 10^6$$

$$I_2 := \frac{1}{12} \cdot b_w \cdot h_w{}^3 = 3.654 \cdot 10^8$$

$$I_3 := \frac{1}{12} \cdot b_{b.f} \cdot h_{b.f}{}^3 = 1.069 \cdot 10^7$$

Edge beams:
$$I_{1.edge} := \frac{1}{12} \cdot b_{t.f.edge} \cdot h_{t.f}{}^3$$

$$I_{2.edge} := \frac{1}{12} \cdot b_{w.edge} \cdot h_w{}^3$$

$$I_{3.edge} := \frac{1}{12} \cdot b_{b.f.edge} \cdot h_{b.f}{}^3$$

Areas:

Middle beams:
$$A_1 := b_{t.f} \cdot h_{t.f} = 2.43 \cdot 10^4$$

$$A_2 := b_w \cdot h_w = 2.673 \cdot 10^4$$

$$A_3 := b_{b.f} \cdot h_{b.f} = 3.447 \cdot 10^4$$

Edge beams:
$$A_{1.edge} := b_{t.f.edge} \cdot h_{t.f} = 1.213 \cdot 10^4$$

$$A_{2.edge} := b_{w.edge} \cdot h_w = 5.67 \cdot 10^4$$

$$A_{3.edge} := b_{b.f.edge} \cdot h_{b.f} = 1.72 \cdot 10^4$$

Gamma-values:
(Glued interfaces)

$$\gamma_1 := 1 \qquad \gamma_2 := 1 \qquad \gamma_3 := 1$$

Steiner-distances:
Calculated acc. to Annex B in EC5.
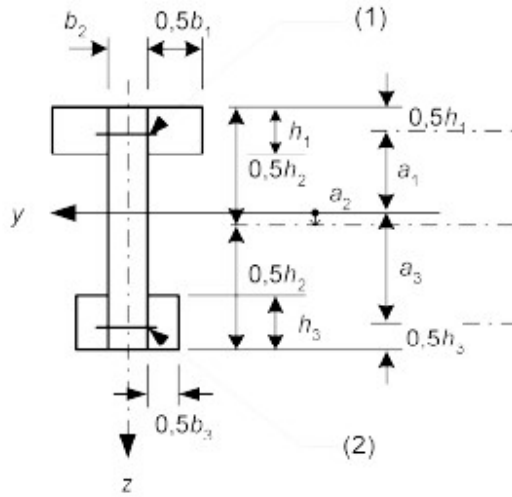
The figure shows the Steiner-distances as illustrated in EC5-1-1, Annex B.

Middle beams:

$$a_2 := \frac{\gamma_1 \cdot E_{0.mean.t.f} \cdot A_1 \cdot (h_{t.f} + h_w) - \gamma_3 \cdot E_{0.mean.b.f} \cdot A_3 \cdot (h_{b.f} + h_w)}{2 \cdot ((\gamma_1 \cdot E_{0.mean.t.f} \cdot A_1) + (\gamma_2 \cdot E_{0.mean.w} \cdot A_2) + (\gamma_3 \cdot E_{0.mean.b.f} \cdot A_3))}$$

$$a_2 = -28.178$$

$$a_1 := \frac{h_w + h_{t.f}}{2} - a_2 = 252.178$$

$$a_3 := \frac{h_w + h_{b.f}}{2} + a_2 = 204.822$$

Edge beams:

$$a_{2.edge} := \frac{\gamma_1 \cdot E_{0.mean.t.f} \cdot A_{1.edge} \cdot (h_{t.f} + h_w) - \gamma_3 \cdot E_{0.mean.b.f} \cdot A_{3.edge} \cdot (h_{b.f} + h_w)}{2 \cdot \left((\gamma_1 \cdot E_{0.mean.t.f} \cdot A_{1.edge}) + (\gamma_2 \cdot E_{0.mean.w} \cdot A_{2.edge}) + (\gamma_3 \cdot E_{0.mean.b.f} \cdot A_{3.edge}) \right)}$$

$$a_{2.edge} = -12.98$$

$$a_{1.edge} := \frac{h_w + h_{t.f}}{2} - a_{2.edge} = 236.98$$

$$a_{3.edge} := \frac{h_w + h_{b.f}}{2} + a_{2.edge} = 220.02$$

Effective bending stiffness:
Equation B.1 in EC5-1-1

Middle beams:

$$EI_{ef.inst1} := E_{0.mean.t.f} \cdot I_1 + \gamma_1 \cdot E_{0.mean.t.f} \cdot A_1 \cdot a_1^2 = 1.626 \cdot 10^{13}$$

$$EI_{ef.inst2} := E_{0.mean.w} \cdot I_2 + \gamma_2 \cdot E_{0.mean.w} \cdot A_2 \cdot a_2^2 = 5.026 \cdot 10^{12}$$

$$EI_{ef.inst3} := E_{0.mean.b.f} \cdot I_3 + \gamma_3 \cdot E_{0.mean.b.f} \cdot A_3 \cdot a_3^2 = 1.529 \cdot 10^{13}$$

$$EI_{ef.inst} := EI_{ef.inst1} + EI_{ef.inst2} + EI_{ef.inst3} = 3.658 \cdot 10^{13}$$

Edge beams:

$$EI_{ef.inst1.edge} := E_{0.mean.t.f} \cdot I_{1.edge} + \gamma_1 \cdot E_{0.mean.t.f} \cdot A_{1.edge} \cdot a_{1.edge}^2 = 7.17 \cdot 10^{12}$$

$$EI_{ef.inst2.edge} := E_{0.mean.w} \cdot I_{2.edge} + \gamma_2 \cdot E_{0.mean.w} \cdot A_{2.edge} \cdot a_{2.edge}^2 = 1.02 \cdot 10^{13}$$

$$EI_{ef.inst3.edge} := E_{0.mean.b.f} \cdot I_{3.edge} + \gamma_3 \cdot E_{0.mean.b.f} \cdot A_{3.edge} \cdot a_{3.edge}^2 = 8.8 \cdot 10^{12}$$

$$EI_{ef.inst.edge} := EI_{ef.inst1.edge} + EI_{ef.inst2.edge} + EI_{ef.inst3.edge} = 2.617 \cdot 10^{13}$$

Effective 2nd moment of inertia:

Middle beams:

$$I_{ef.inst} := \left(I_1 + \left(A_1 \cdot a_1{}^2\right)\right) + \left(I_2 + \left(A_2 \cdot a_2{}^2\right)\right) + \left(I_3 + \left(A_3 \cdot a_3{}^2\right)\right) = 3.392 \cdot 10^9$$

Edge beams:

$$I_{1.ef.edge} := I_{1.edge} + \left(A_{1.edge} \cdot a_{1.edge}{}^2\right)$$

$$I_{2.ef.edge} := I_{2.edge} + \left(A_{2.edge} \cdot a_{2.edge}{}^2\right)$$

$$I_{3.ef.edge} := I_{3.edge} + \left(A_{3.edge} \cdot a_{3.edge}{}^2\right)$$

$$I_{ef.inst.edge} := I_{1.ef.edge} + I_{2.ef.edge} + I_{3.ef.edge}$$

Now we have the stiffness of the middle beams and edge beams, and from here there will only be done checks for the <u>middle beams.</u>

<u>Calculation of stresses:</u>
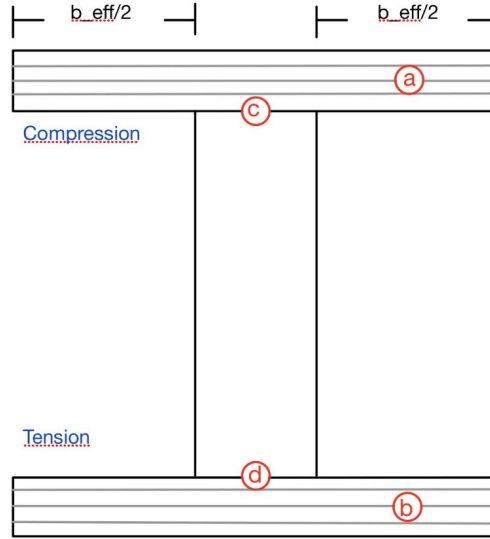
Axial stresses:
Eq. B.7 in EC5-1-1

The figure shows the position of the points in which the stresses are found.

Point a) Design compression stress

$$\sigma_1 := \frac{\left(\gamma_1 \cdot E_{0.mean.t.f} \cdot a_1 \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} + \frac{\left(0.5 \cdot E_{0.mean.t.f} \cdot h_{t.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} = 3.968$$

Point b) Design tension stress

$$\sigma_3 := \frac{\left(\gamma_1 \cdot E_{0.mean.b.f} \cdot a_3 \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} + \frac{\left(0.5 \cdot E_{0.mean.b.f} \cdot h_{b.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} = 3.412$$

Point c) Design bending + axial stress in the web (same as in point d)

$$\sigma_2 := \frac{\left(\gamma_1 \cdot E_{0.mean.w} \cdot a_2 \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} + \frac{\left(0.5 \cdot E_{0.mean.w} \cdot h_w \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} = 3.129$$

Shear stress:
(Eq. B.9 in EC5-1-1)

Distance from center of web to the place of zero normal stress:

$$h := \frac{h_w}{2} + a_2$$

$$\tau_{2.max} := \frac{\left(\gamma_3 \cdot E_{0.mean.b.f} \cdot A_3 \cdot a_3 + 0.5 \cdot E_{0.mean.w} \cdot b_w \cdot h^2\right)}{k_{cr.w} \cdot b_w \cdot EI_{ef.inst}} \cdot V_{Ed.ULS} = 1.073$$

Appendix C

Design checks:

Normal stresses:

Utilizations:

Point a)

if $\sigma_1 < f_{c.0.d.tf}$ = "OK"
$\quad \|$ "OK"
else
$\quad \|$ "Not OK"

$$\frac{\sigma_1}{f_{c.0.d.tf}} = 0.219$$

Point b)

if $\sigma_3 < f_{t.0.d.bf}$ = "OK"
$\quad \|$ "OK"
else
$\quad \|$ "Not OK"

$$\frac{\sigma_3}{f_{t.0.d.bf}} = 0.252$$

Point c)

if $\sigma_2 < f_{c.0.d.w}$ = "OK"
$\quad \|$ "OK"
else
$\quad \|$ "Not OK"

$$\frac{\sigma_2}{f_{c.0.d.w}} = 0.184$$

Point d)

if $\sigma_2 < f_{t.0.d.w}$ = "OK"
$\quad \|$ "OK"
else
$\quad \|$ "Not OK"

$$\frac{\sigma_2}{f_{t.0.d.w}} = 0.231$$

Shear stress:

if $\tau_{2.max} < f_{vd.w}$ = "OK"
$\quad \|$ "OK"
else
$\quad \|$ "Not OK"

$$\frac{\tau_{2.max}}{f_{vd.w}} = 0.441$$

Glue-line check:

We assume that the glue itself will be ok, but a check of the shear that arises in the LVL flanges along the grain in the interface will have to be done. The corresponding shear along the grain in the Glulam bottom flange can also be assumed to be ok because it will not be as critical as the shear in the Glulam web, and the strength will be the same for the two cases, fvk. We will only check the top flange because this is thinner than the bottom flange and therefor more critical.
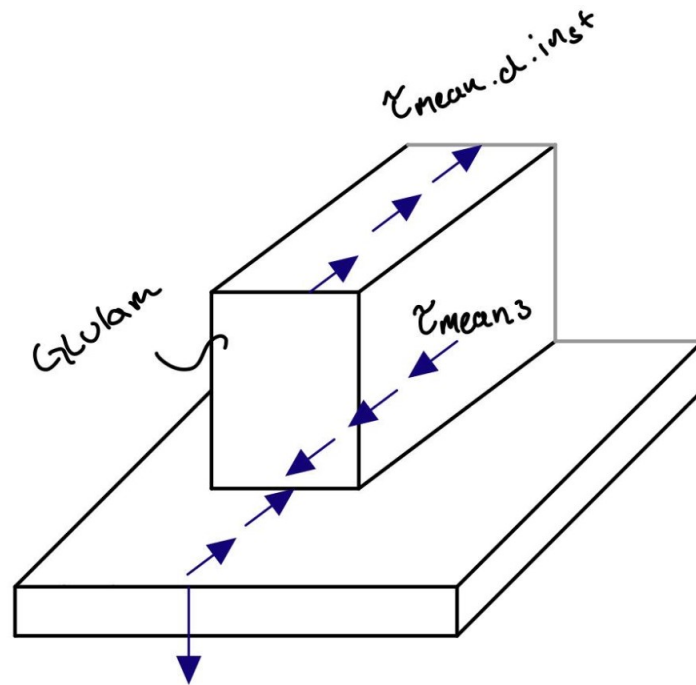
Shear that arises in the flange over the width of the web. This can be found using equation B.5 in EC5, which is equivalent as load on a fastener.

- Shear stress (assumed uniform) in the flange-area over the width of the web:
Acc. to eq. B.5 in EC5

$$\tau_{mean.d.inst} := \frac{E_{0.mean.t.f} \cdot A_1 \cdot a_1}{EI_{ef.inst} \cdot b_w} \cdot V_{Ed.ULS} = 0.633$$

- Flatwise shear strength of LVL:

$$f_{v.0.flat.d.tf} = 0.904$$

- Check acc. to NS-EN 1995-1-1, §9.1.2(6):

Capacity:

$$f_{v.0d.check} := \text{if } b_w \leq 8 \cdot h_{t.f} \\ \quad \left\| f_{v.0.flat.d.tf} \right. \\ \text{else} \\ \quad \left\| f_{v.0.flat.d.tf} \cdot \left(\frac{8 \cdot h_{t.f}}{b_w} \right)^{0.8} \right. = 0.904$$

Check:

$$\text{if } f_{v.0d.check} \geq \tau_{mean.d.inst} = \text{"OK"} \\ \quad \left\| \text{"OK"} \right. \\ \text{else} \\ \quad \left\| \text{"Not OK"} \right.$$

Utilization:

$$\frac{\tau_{mean.d.inst}}{f_{v.0d.check}} = 0.7$$

ULS: Final

$$\text{if } k_{def.t.f} = k_{def.web} \quad \left| = \text{``Final cond. needed''} \right.$$
$$\left\| \text{``Final cond. not needed''} \right.$$
$$\text{else}$$
$$\left\| \text{``Final cond. needed''} \right.$$

Since the time-dependent factors kdef is not the same between two parts of the composite, we must check the final condition. This means that we must consider the time-dependent effects (such as creep) on the different parts.

Stiffness in final condition:

NS-EN 1990-1-1,
Table A1.1:
$\psi_2 := 0.3$

Flanges:
$$E_{mean.fin.1} := \frac{E_{0.mean.t.f}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 8.077 \cdot 10^3$$

$$E_{mean.fin.3} := \frac{E_{0.mean.b.f}}{\left(1 + \psi_2 \cdot k_{def.b.f}\right)} = 8.077 \cdot 10^3$$

$$G_{mean.fin.1} := \frac{G_{t.f.mean}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 461.538$$

$$G_{mean.fin.3} := \frac{G_{b.f.mean}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 461.538$$

Web:
$$E_{mean.fin.2} := \frac{E_{0.mean.w}}{\left(1 + \psi_2 \cdot k_{def.web}\right)} = 1.048 \cdot 10^4$$

$$G_{mean.fin.2} := \frac{G_{web.mean}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 500$$

Appendix C

<u>Calculation of stresses in final condition:</u>
according to Annex B in EC5-1-1

- Cross sectional parameters:

2nd moment of area:

$$I_1 := \frac{1}{12} \cdot b_{ef.t} \cdot h_{t.f}^3 = 6.069 \cdot 10^6$$

$$I_2 := \frac{1}{12} \cdot b_w \cdot h_w^3 = 3.654 \cdot 10^8$$

$$I_3 := \frac{1}{12} \cdot b_{ef.b} \cdot h_{b.f}^3 = 1.733 \cdot 10^7$$

Areas:

$$A_1 := b_{t.f} \cdot h_{t.f} = 2.43 \cdot 10^4$$

$$A_2 := b_w \cdot h_w = 2.673 \cdot 10^4$$

$$A_3 := b_{b.f} \cdot h_{b.f} = 3.447 \cdot 10^4$$

Gamma-values: $\qquad \gamma_1 := 1 \qquad \gamma_2 := 1 \qquad \gamma_3 := 1$
(Glued interfaces)

- Steiner-distances:
Calculated acc. to Annex B in EC5.

*The figure that shows the Steiner-distances as illustrated in EC5-1-1,
Annex B can be found in the corresponding instantaneous check.*

$$a_2 := \frac{\gamma_1 \cdot E_{mean.fin.1} \cdot A_1 \cdot (h_{t.f} + h_w) - \gamma_3 \cdot E_{mean.fin.3} \cdot A_3 \cdot (h_{b.f} + h_w)}{2 \cdot ((\gamma_1 \cdot E_{mean.fin.1} \cdot A_1) + (\gamma_2 \cdot E_{mean.fin.2} \cdot A_2) + (\gamma_3 \cdot E_{mean.fin.3} \cdot A_3))}$$

$$a_2 = -27.695$$

$$a_1 := \frac{h_w + h_{t.f}}{2} - a_2 = 251.695$$

$$a_3 := \frac{h_w + h_{b.f}}{2} + a_2 = 205.305$$

Effective bending stiffness:
Acc. to Equation B.1 in EC5-1-1

$$EI_{ef.fin1} := E_{mean.fin.1} \cdot I_1 + \gamma_1 \cdot E_{mean.fin.1} \cdot A_1 \cdot a_1{}^2$$

$$EI_{ef.fin2} := E_{mean.fin.2} \cdot I_2 + \gamma_2 \cdot E_{mean.fin.2} \cdot A_2 \cdot a_2{}^2$$

$$EI_{ef.fin3} := E_{mean.fin.3} \cdot I_3 + \gamma_3 \cdot E_{mean.fin.3} \cdot A_3 \cdot a_3{}^2$$

$$EI_{ef.fin} := EI_{ef.fin1} + EI_{ef.fin2} + EI_{ef.fin3} = 2.84 \cdot 10^{13}$$

Effective 2nd moment of inertia:

$$I_{ef.fin} := (I_1 + (A_1 \cdot a_1{}^2)) + (I_2 + (A_2 \cdot a_2{}^2)) + (I_3 + (A_3 \cdot a_3{}^2)) = 3.401 \cdot 10^9$$

- Calculation of stresses:

Axial stresses:
(Eq. B.7 in EC5-1-1)

The figure shows the position of the points in which the stresses are found.

Point a) Design compression stress

$$\sigma_1 := \frac{\left(\gamma_1 \cdot E_{mean.fin.1} \cdot a_1 \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} + \frac{\left(0.5 \cdot E_{mean.fin.1} \cdot h_{t.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} = 3.925$$

Point b) Design tension stress

$$\sigma_3 := \frac{\left(\gamma_3 \cdot E_{mean.fin.3} \cdot a_3 \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} + \frac{\left(0.5 \cdot E_{mean.fin.3} \cdot h_{b.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} = 3.388$$

Point c) Design bending + axial stress in the web (same as in point d)

$$\sigma_2 := \frac{\left(\gamma_2 \cdot E_{mean.fin.2} \cdot a_2 \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} + \frac{\left(0.5 \cdot E_{mean.fin.2} \cdot h_w \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} = 3.26$$

Shear stress:
(Eq. B.9 in EC5-1-1)

Distance from center of web to the place of zero normal stress:

$$h := \frac{h_w}{2} + a_2$$

$$\tau_{2.max} := \frac{\left(\gamma_3 \cdot E_{mean.fin.3} \cdot A_3 \cdot a_3 + 0.5 \cdot E_{mean.fin.2} \cdot b_w \cdot h^2\right)}{k_{cr.w} \cdot b_w \cdot EI_{ef.fin}} \cdot V_{Ed.ULS} = 1.074$$

Design checks: Utilizations:

- Normal stresses:

Point a)
$$\text{if } \sigma_1 < f_{c.0.d.tf} = \text{``OK''}$$
$$\left\| \text{``OK''} \right.$$
$$\text{else}$$
$$\left\| \text{``Not OK''} \right.$$

$$\frac{\sigma_1}{f_{c.0.d.tf}} = 0.217$$

Point b)
$$\text{if } \sigma_3 < f_{t.0.d.bf} = \text{``OK''}$$
$$\left\| \text{``OK''} \right.$$
$$\text{else}$$
$$\left\| \text{``Not OK''} \right.$$

$$\frac{\sigma_3}{f_{t.0.d.bf}} = 0.25$$

Point c)
$$\text{if } \sigma_2 < f_{c.0.d.w} = \text{``OK''}$$
$$\left\| \text{``OK''} \right.$$
$$\text{else}$$
$$\left\| \text{``Not OK''} \right.$$

$$\frac{\sigma_2}{f_{c.0.d.w}} = 0.191$$

Point d)
$$\text{if } \sigma_2 < f_{t.0.d.w} = \text{``OK''}$$
$$\left\| \text{``OK''} \right.$$
$$\text{else}$$
$$\left\| \text{``Not OK''} \right.$$

$$\frac{\sigma_2}{f_{t.0.d.w}} = 0.24$$

-Shear stress

$$\text{if } \tau_{2.max} < f_{vd.w} = \text{``OK''}$$
$$\left\| \text{``OK''} \right.$$
$$\text{else}$$
$$\left\| \text{``Not OK''} \right.$$

$$\frac{\tau_{2.max}}{f_{vd.w}} = 0.441$$

Glue-line check:

> We assume that the glue itself will be ok, but a check of the shear that arises in the LVL flanges along the grain in the interface will have to be done. The corresponding shear along the grain in the Glulam bottom flange can also be assumed to be ok because it will not be as critical as the shear in the Glulam web, and the strength will be the same for the two cases, fvk.

- Shear stress (assumed uniform) in the flange-area over the width of the web:
Acc. to eq. B.5 in EC5

$$\tau_{mean.d.fin} := \frac{E_{mean.fin.1} \cdot A_1 \cdot a_1}{EI_{ef.inst} \cdot k_{cr.w} \cdot b_w} \cdot V_{Ed.ULS} = 0.608$$

- Flatwise shear strength of LVL:

$$f_{v.0.flat.d.tf} = 0.904$$

- Check acc. to NS-EN 1995-1-1, §9.1.2(6):

Capacity:
$$f_{v.90d.check} := \text{if } b_w \le 8 \cdot h_{t.f} \qquad \bigg| = 0.904$$
$$\left\|\, f_{v.0.flat.d.tf} \right.$$
$$\text{else}$$
$$\left\|\, f_{v.0.flat.d.tf} \cdot \left(\frac{8 \cdot h_{t.f}}{b_w}\right)^{0.8} \right.$$

Check:
$$\text{if } f_{v.90d.check} \ge \tau_{mean.d.fin} \bigg| = \text{"OK"}$$
$$\left\|\, \text{"OK"} \right.$$
$$\text{else}$$
$$\left\|\, \text{"Not OK"} \right.$$

Utilization:
$$\frac{\tau_{mean.d.fin}}{f_{v.90d.check}} = 0.672$$

SLS: Instantaneous deformation

- Bending deformation:
$$w_{inst.bending} := \frac{5}{384} \cdot \frac{q_{Ed.SLS1} \cdot l^4}{EI_{ef.inst}} = 7.349$$

-Shear deformation acc. to Timoshenko beam theory:

Shear correction factor: $\qquad \kappa := 0.83$

The shear correction factor is for rectangular cross sections, since we assume only web takes shear.

Shear stiffness:

$$S_T := \left(G_{t.f.mean} \cdot b_{ef.t} \cdot h_{t.f}\right) + \left(G_{web.mean} \cdot b_w \cdot h_w\right) + \left(G_{b.f.mean} \cdot b_{ef.b} \cdot h_{b.f}\right) = 7.453 \cdot 10^7$$

The shear deformation becomes:

$$w_{inst.shear} := q_{Ed.SLS1} \cdot \frac{l^2}{8} \cdot \frac{1}{\kappa \cdot S_T} = 0.577$$

-Total deformation:
Acc. to Timoshenko beam theory

$$w_{inst} := w_{inst.bending} + w_{inst.shear} = 7.926$$

- Allowed deformation: $\qquad w_{max} := \dfrac{l}{500} = 17 \qquad$ (most conservative demand)
(Acc. to EC5-1-1, table 7.2)

$$\begin{aligned} &\text{if } w_{inst} < w_{max} \\ &\quad \| \text{``Ok''} \\ &\text{else} \\ &\quad \| \text{``Not ok''} \end{aligned} = \text{``Ok''}$$

Utilization:

$$\frac{w_{inst}}{w_{max}} = 0.466$$

The instantaneous deformation is ok.

SLS: Final deformation

<mark>Since the time-property factors are not the same, we must use the elasticity-modulus for the final condition and then calculate the deformation in the same way as for instantaneous.</mark>

SLS characteristic load:
$$q_{Ed.SLS1} = 3.955$$

- Bending deformation:
(formula from handbooks)
$$w_{fin.bending} := \frac{5}{384} \cdot \frac{q_{Ed.SLS1} \cdot l^4}{EI_{ef.fin}} = 9.466$$

-Shear deformation:

Shear correction factor:
$$\kappa := 0.83$$

Shear stiffness:

$$S_T := \left(G_{mean.fin.1} \cdot b_{ef.t} \cdot h_{t.f}\right) + \left(G_{mean.fin.2} \cdot b_w \cdot h_w\right) + \left(G_{mean.fin.3} \cdot b_{ef.b} \cdot h_{b.f}\right) = 5.733 \cdot 10^7$$

Shear deformation:
$$w_{fin.shear} := q_{Ed.SLS1} \cdot \frac{l^2}{8} \cdot \frac{1}{\kappa \cdot S_T} = 0.751$$

-Total deformation:
$$w_{fin} := w_{fin.bending} + w_{fin.shear} = 10.216$$

- Allowed deformation:
Acc. to EC5-1-1, table 7.2
$$w_{max} := \frac{l}{500} = 17$$

$$\text{if } w_{fin} < w_{max} = \text{"Ok"}$$
$$\left\| \text{"Ok"} \right.$$
$$\text{else}$$
$$\left\| \text{"Not ok"} \right.$$

Utilization:

$$\frac{w_{fin}}{w_{max}} = 0.601$$

The final deformation is ok.

SLS: Vibration check
(Human induced vibr.)

Weight of the deck: $\gamma := 2$ $\left[\dfrac{kN}{m^2}\right]$

Dead-load: $q_{dead} := \gamma \cdot CC \cdot 10^{-3} = 1.13$ $\left[\dfrac{N}{mm}\right]$

Gravitational acceleration: $g := 9.81$ $\left[\dfrac{m}{s^2}\right]$

Mass: $m := \dfrac{q_{dead}}{g} = 0.115$ $\left[\dfrac{kg}{mm}\right]$

We calculate the mass based on the dead-load (EC5-1-1, §7.3)

No. of beam elements per meter: $n_{beams} := \dfrac{1000}{CC} = 1.77$

Equivalent bending stiffness: $EI_L := EI_{ef.inst} = 3.658 \cdot 10^{13}$ $\left[\dfrac{Nmm^2}{m}\right]$

Fundamental frequency: $f_{n.1} := \dfrac{\pi}{2 \cdot l^2} \cdot \sqrt[2]{\dfrac{EI_L}{m \cdot 10^{-3}}} = 12.252$ $\left[Hz\right]$

Appendix C

EN1995-1-1, §7.3.3.(1):

$$\begin{array}{l} \text{if } f_{n.1} > 8 \\ \quad \| \text{ "Simplified method allowed"} \\ \text{else} \\ \quad \| \text{ "Special investigation needed"} \end{array} = \text{"Simplified method allowed"}$$

1kN static deflection: $w_{static.1kN} := \dfrac{1000 \cdot l^3}{48 \cdot EI_L} = 0.35$

Hu & Chui criterion:

$$\begin{array}{l} \text{if } \dfrac{\left(\dfrac{f_{n.1}}{18.7}\right)^{2.27}}{w_{static.1kN}} \geq 1 \\ \quad \| \text{ "OK"} \\ \text{else} \\ \quad \| \text{ "Not OK"} \end{array} = \text{"OK"}$$

$$\dfrac{\left(\dfrac{f_{n.1}}{18.7}\right)^{2.27}}{w_{static.1kN}} = 1.095$$

By using the quasi-permanent load combination one would obtain more vibrations.

Effective stiffness to be used for the modelling of the shell elements: (using the instantaneous values)

Width of deck: $\qquad b = 2.4 \cdot 10^3$

Number of glulam elements over the width of the deck:

Middle beams: $\qquad n_{mid} := 3$

Edge beams: $\qquad n_{edge} := 2$

Total bending stiffness of the deck:

$$EI_{ef.tot.inst} := \left(EI_{ef.inst} \cdot n_{mid}\right) + \left(EI_{ef.inst.edge} \cdot n_{edge}\right) = 1.621 \cdot 10^{14}$$

Total 2nd moment of area of the deck:

$$I_{ef.tot.inst} := \left(I_{ef.inst} \cdot n_{mid}\right) + \left(I_{ef.inst.edge} \cdot n_{edge}\right) = 1.479 \cdot 10^{10}$$

Total elasticity modulus of the deck (grain direction):

$$E_{inst.1} := \frac{EI_{ef.tot.inst}}{I_{ef.tot.inst}} = 1.096 \cdot 10^4$$

Note: The stiffness values extracted here (both for instantaneous and final condition) are for the grain direction (direction 1). To get the values in the direction perpendicular to the grain, we may divide E1 by 4.

Total elasticity modulus of the deck (perpendicular to grain direction):

$$E_{inst.2} := \frac{E_{inst.1}}{4} = 2.74 \cdot 10^3$$

C2. Connection Calculations

Part 1 consists of the data needed for the calculation
Part 2 is the ULS checks.
Part 3 is the calculation of stiffness for the given configuration.

Formulas for structural connection design has been extrated from NS-EN 1995-1-1.

Units used in script:

- Dimensions/lengths: $[mm]$

- Forces: $[N]$

- Moments: $[Nmm]$

- Stresses/strengths: $\left[\dfrac{N}{mm^2}\right]$

- Areas: $[mm^2]$

- 2nd moment of inertia: $[mm^4]$

- Densities: $\left[\dfrac{kg}{m^3}\right]$

PART 1)

Geometry:

Assumed dimensions of column and diagonal:

$$b_{col} := 765 \qquad\qquad h_{col} := 765$$

$$b_{diag} := 540 \qquad\qquad h_{diag} := 585$$

Lengths of members: $\qquad\qquad L_1 := 5000$

$$L_2 := 11650$$

Angle between diagonal and column: $\qquad\qquad \alpha := 42 \; \textbf{\textit{deg}}$

Diameter of all the dowels: $\qquad\qquad d := 12$

Angle between diagonal force and grain: $\qquad\qquad \alpha_1 := \alpha$

$$\alpha_2 := 0$$

Number of dowels: $\qquad\qquad n_{dowels.1} := 35$

$$n_{dowels.2} := 66$$

Number of rows: $\qquad\qquad n_{rows.1} := 5$

$$n_{rows.2} := 6$$

Number of steel plates in
column and diagonal:

$$n_{plates} := 4$$

Thickness of steel plates:

$$t_{plate} := 16$$

Classification of steel plates according to NS-EN 1995-1-1, §8.2.3(1):

$$Plate\_id := \text{if } t_{plate} \leq 0.5 \cdot d \quad \bigg| = \text{"Thick plate"}$$
$$\left\| \text{"Thin plate"} \right.$$
$$\text{else if } t_{plate} \geq d$$
$$\left\| \text{"Thick plate"} \right.$$
$$\text{else}$$
$$\left\| \text{"Not clear"} \right.$$

*Note: For multiple internal slotted-in steel plate connections we can
always assume to use thick plates.*

Factors:

-Partial factors:

For glulam: $\gamma_{M.GL} := 1.15$

For connections: $\gamma_{M.con} := 1.3$

- Modification factors:

Factor for medium-
duration loading: $k_{mod} := 1.1$ (since wind is included,
(EC5, Table 3.1)   ULS COMB1)

Bearing factor: $k_{90} := 1.35 + (0.015 \cdot d) = 1.53$
(EC5, eq. (8.33))

Material properties:

-Column and diagonal:

Density:
$$\rho_{m.1} := 480$$

$$\rho_{m.2} := 480$$

$$\rho_m := \sqrt[2]{\rho_{m.1} \cdot \rho_{m.2}} = 480$$

Area:
$$A_1 := h_{col} \cdot b_{col}$$

$$A_2 := h_{diag} \cdot b_{diag}$$

Strength:
$$f_{c.0.k} := 24.5$$

$$f_{t.0.k} := 19.5$$

$$f_{v.k} := 3.5$$

Youngs modulus:
$$E_{0.mean} := 13600$$

2nd moment of area:
$$I_{col} := \frac{1}{12} \cdot b_{col} \cdot h_{col}^3 = 2.854 \cdot 10^{10}$$

$$I_{diag} := \frac{1}{12} \cdot b_{diag} \cdot h_{diag}^3 = 9.009 \cdot 10^9$$

Embedment strength of timber:

(EC5, eq. 8.32)
$$f_{h.0.k} := 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_m = 34.637$$

(EC5, eq. 8.31)
$$f_{h.\alpha.k} := \frac{f_{h.0.k}}{k_{90} \cdot (\sin(\alpha))^2 + (\cos(\alpha))^2} = 27.994$$

Appendix C

Column:
$$f_{h.1.k.col} := f_{h.\alpha.k} = 27.994$$

$$f_{h.2.k.col} := f_{h.\alpha.k} = 27.994$$

$$\beta_{col} := \frac{f_{h.2.k.col}}{f_{h.1.k.col}} = 1$$

Diagonal:
$$f_{h.1.k.diag} := f_{h.0.k} = 34.637$$

$$f_{h.2.k.diag} := f_{h.0.k} = 34.637$$

Ratio:
$$\beta_{diag} := \frac{f_{h.2.k.diag}}{f_{h.1.k.diag}} = 1$$

-Fasteners (dowels):

Tensile strength: $f_{uk} := 650$

Yield moment:
(EC5, eq. 8.30)
$$M_{y.Rk} := 0.3 \cdot f_{uk} \cdot d^{2.6} = 1.247 \cdot 10^5$$

- Steel plates:

Elasticity modulus: $E_s := 210000$

Safety factor: $\gamma_0 := 1.15 \qquad \gamma_{M2} := 1.25$

Dimensions:
$$l_{plate.1} := 0.8 \cdot h_{col} \qquad b_{plate.1} := 0.8 \cdot b_{diag}$$

$$l_{plate.2} := 0.8 \cdot h_{diag} \qquad b_{plate.2} := b_{plate.1}$$

The dimensions of the steel plates are assumed to be 80% of the cross-sectional heights/widths as a conservative simplification.

34

Yielding strength: $f_y := 355$

Fracture strength: $f_u := 510$ (Stålprofiler håndbok)

$$f_{ub} := 0.9 \cdot f_u = 459$$

Design loads:

- Diagonal: $F_{diag} := 2385 \cdot 10^3$

- Column: $F_{col} := 4000 \cdot 10^3$

- Total load in column connection:

$$F_{Ed.col} := F_{diag} \cdot \sin(\alpha) = 1.596 \cdot 10^6$$

- Total load in diagonal connection:

$$F_{Ed.diag} := F_{diag} = 2.385 \cdot 10^6$$

In the connection it is assumed to be no eccentricities

Concentric connection

<div align="center">
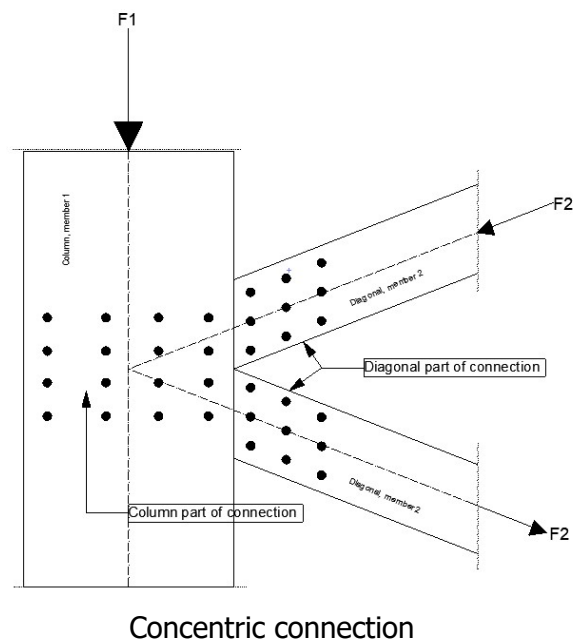
PART 2)
Capacity of Connection

</div>

In this part the ULS checks of the connection will be presented.
First the column part of the connection, then the diagonal part.

Column part:

Embedment strength:
Acc. to NS-EN 1995-1-1, §8.2

<div style="background-color: yellow;">

In connections with multiple shear planes the load-carrying capacity is determined by assuming that the external members are in single shear and the middle members in double shear. The total load-carrying capacity is determined by adding the contributions of compatible failure modes. Some of the failure modes cannot occur simultaneously due to deformation compatibility, meaning that they occur at different deformation levels: either small ('brittle') or large ('ductile').

</div>

Number of middle members:

$$n_{mid} := n_{plates} - 1 = 3$$

Total nr. of shear planes for middle members:

$$n_{sp.mid} := n_{mid} \cdot 2 = 6$$

Total nr. of shear planes for outer members:

$$n_{sp.out} := 2$$

Thickness of middle members:

$$t_2 := \frac{b_{col}}{\left(n_{plates} + 1\right)} = 153$$

Thickness of outer members:

$$t_1 := \frac{\left(b_{col} - \left(n_{mid} \cdot t_2\right) - \left(n_{rows.1} \cdot t_{plate}\right)\right)}{2} = 113$$
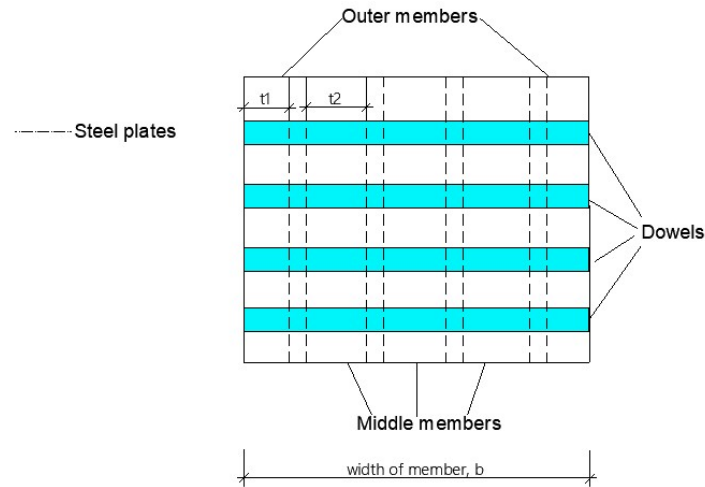
Figure: Section showing the outer and middle members of a connection with multiple slotted-in steel plates and dowels. Representative for both the column part and the diagonal part of the connection.

Outer members:

Formulas for single shear failure modes, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode c) $F_{v.Rk.c} := f_{h.1.k.col} \cdot t_1 \cdot d$

Failure mode d) $F_{v.Rk.d} := f_{h.1.k.col} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \dfrac{4 \cdot M_{y.Rk}}{f_{h.1.k.col} \cdot d \cdot t_1^{\,2}}} - 1 \right)$

Failure mode e) $F_{v.Rk.e} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.col} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

Total capacity per fastener per shear plane for outer members:

$$F_{v.Rk.outer} := min\left(F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}\right) = 1.489 \cdot 10^4$$

Middle members:

Formulas for double shear failure modes with external plates, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode f) $\quad F_{v.Rk.f} := f_{h.1.k.col} \cdot t_1 \cdot d$

Failure mode g) $\quad F_{v.Rk.g} := f_{h.1.k.col} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \dfrac{4 \cdot M_{y.Rk}}{f_{h.1.k.col} \cdot d \cdot t_1^{\,2}}} - 1 \right)$

Failure mode h) $\quad F_{v.Rk.h} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.col} \cdot d}$

Failure mode l) $\quad F_{v.Rk.l} := 0.5 \cdot f_{h.2.k.col} \cdot t_2 \cdot d$

Failure mode m) $\quad F_{v.Rk.m} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.2.k.col} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

Total capacity per fastener per shear plane for middle members:

$$F_{v.Rk.middle} := min\left(F_{v.Rk.f}, F_{v.Rk.g}, F_{v.Rk.h}, F_{v.Rk.l}, F_{v.Rk.m}\right) = 1.489 \cdot 10^4$$

Appendix C

Total capacity per fastener per shear plane:

We can only combine compatible failure modes.
For the practical purpose of being able to program the comparison, the failure modes will be identified as 1, 2, 3 etc. instead of a, b ,c etc.

Failure mode for middle members will hereafter be named "mode_mid".
Failure mode for outer members will hereafter be named "mode_outer".
It must also be noted that ▯∧▯is the logical operator "AND", and ▯∨▯is the logical operator "OR", which will both be used in the if-else-statements below.

Failure mode for
middle members:

$$mode\_mid := \text{if } F_{v.Rk.middle} = F_{v.Rk.f} \quad \Big\| 6 \quad = 8$$
$$\text{else if } F_{v.Rk.middle} = F_{v.Rk.g} \quad \Big\| 7$$
$$\text{else if } F_{v.Rk.middle} = F_{v.Rk.h} \quad \Big\| 8$$
$$\text{else if } F_{v.Rk.middle} = F_{v.Rk.l} \quad \Big\| 12$$
$$\text{else if } F_{v.Rk.middle} = F_{v.Rk.m} \quad \Big\| 13$$

Failure mode for
outer members:

$$mode\_outer := \text{if } F_{v.Rk.outer} = F_{v.Rk.c} \quad \Big\| 3 \quad = 5$$
$$\text{else if } F_{v.Rk.outer} = F_{v.Rk.d} \quad \Big\| 4$$
$$\text{else if } F_{v.Rk.outer} = F_{v.Rk.e} \quad \Big\| 5$$

Comaptibility check:
Acc. to EC5, §8.1.3.2

Now we must check if the failure mode for outer members are the same type as for the middle members. We can do this through an if-else statement, as shown below.

$$Failure \coloneqq \text{if } (mode\_outer = 3) \wedge ((mode\_mid = 6) \vee (mode\_mid = 12))$$
$$\qquad \| \text{ "Brittle modes"}$$
$$\quad \text{else if } ((mode\_outer = 4) \vee (mode\_outer = 5)) \wedge ((mode\_mid = 7) \vee (mode\_mid = 8) \vee (mode\_mid = 13))$$
$$\qquad \| \text{ "Ductile modes"}$$
$$\quad \text{else}$$
$$\qquad \| \text{ "Incompatible modes"}$$

$$Failure = \text{"Ductile modes"}$$

As we can see, both the outer failure modes and the inner failure modes are ductile, which is what we want. In situations where both ductile and brittle types are possible it is good practice to try to ensure that the design condition is based on the ductile failure mechanism. Therefor, we will not proceed until ductile compatibility is achieved in the above code.

Total capacity per fastener for the column part is:

Shear planes for outer members: $n_{sp.out} = 2$

Shear planes for middle members: $n_{sp.mid} = 6$

$$F_{v.Rk.col} \coloneqq n_{sp.out} \cdot F_{v.Rk.outer} + n_{sp.mid} \cdot F_{v.Rk.middle} = 1.191 \cdot 10^5$$

Capacity per fastener in ULS:

$$F_{v.Rd.col} \coloneqq \frac{F_{v.Rk.col}}{\gamma_{M.con}} \cdot (k_{mod}) = 1.008 \cdot 10^5$$

Amount of dowels needed in the column part of the connection:

$$n_{col} := \left(\frac{F_{Ed.col}}{F_{v.Rd.col}}\right) = 15.836$$

Check if the chosen amount of dowels (in the start of the script) is sufficient:

$$
\begin{array}{l|l}
\text{if } n_{dowels.1} \geq n_{col} & = \text{``OK''} \\
\quad \| \text{ ``OK''} & \\
\text{else} & \\
\quad \| \text{ ``Must increase nr. of dowels''} &
\end{array}
$$

Chosen configuration for column part (member 1):

$$a_{1.col} := 100 \qquad a_{2.col} := 141$$

$$a_{3.t.col} := 1000 \qquad a_{3.c.col} := 1000$$

$$a_{4.t.col} := 1000 \qquad a_{4.c.col} := 100$$
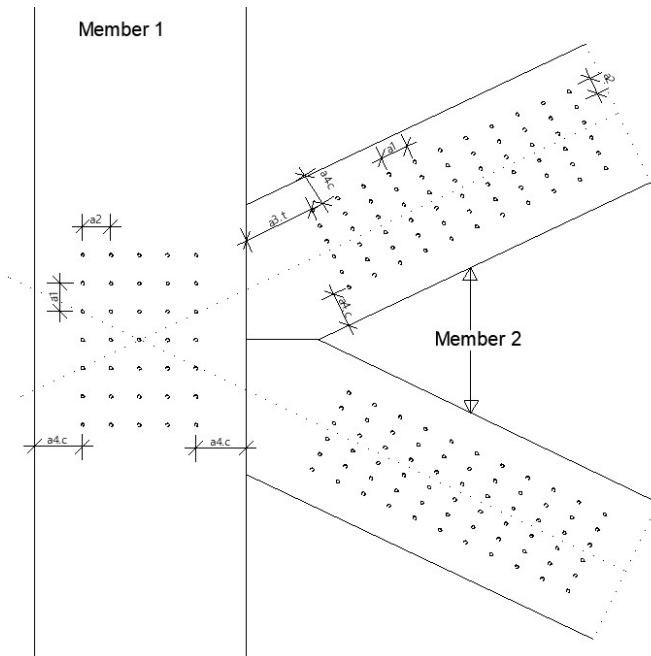
Figure: Connection Configuration

The a3 distances (and a4.t) are actually bigger, but there is no point in measuring them because they will satisfy the distance-demand, as seen in the figure. Therefor a random number (which is big enough to make the code run) has been implemented.

Check of minimum distances:

The minimum spacings given in EC5 have been derived to prevent splitting failure when connection is subjected to lateral load. With too small spacings we get increasing tension perpendicular to grain.

EC5, table 8.5:

$$\text{if } a_{1.col} \geq \left(3 + \left(2 \cdot \cos\left(\alpha\right)\right)\right) \cdot d \;\middle|\; = \text{"OK"}$$
$$\left\| \text{"OK"} \right.$$

$$\text{if } a_{2.col} \geq 3 \cdot d \;\middle|\; = \text{"OK"}$$
$$\left\| \text{"OK"} \right.$$

$$\text{if } a_{3.t.col} \geq \max\left(7 \cdot d, 80\right) \;\middle|\; = \text{"OK"}$$
$$\left\| \text{"OK"} \right.$$

$$\text{if } a_{3.c.col} \geq \max\left(\sin\left(\alpha\right) \cdot d, 3 \cdot d\right) \;\middle|\; = \text{"OK"}$$
$$\left\| \text{"OK"} \right.$$

$$\text{if } a_{4.t.col} \geq \max\left(\left(2 + 2 \cdot \sin\left(\alpha\right)\right) \cdot d, 3 \cdot d\right) \;\middle|\; = \text{"OK"}$$
$$\left\| \text{"OK"} \right.$$

$$\text{if } a_{4.c.col} \geq 3 \cdot d \;\middle|\; = \text{"OK"}$$
$$\left\| \text{"OK"} \right.$$

All the minimum distances are fulfilled!

Splitting check: (Parallell to grain)

<mark>a) The first thing that needs to be sorted is the distances between the fasteners. This has been verified.

b) The second thing that must be verified is that the effective number of fasteners in a row has sufficient capacity to carry the load parallell to grain. This will be controlled in accordance with NS-EN 1995-1-1, §8.1.2(5):</mark>

Total amount of fasteners in one row in grain direction:

$$n_{row.col} := \frac{n_{dowels.1}}{n_{rows.1}} = 7$$

The effective number of fasteners in one row in grain direction:

EC5, eq. (8.34) $n_{ef.row.col} := min\left(n_{row.col}, n_{row.col}^{0.9} \cdot \sqrt[4]{\frac{a_{1.col}}{13 \cdot d}}\right) = 5.156$

The effective load-carrying capacity of each row then becomes:

$$F_{v.ef.Rk.col} := F_{v.Rk.col} \cdot n_{ef.row.col}$$

For entire connection:

$$F_{v.ef.Rk.col.tot} := F_{v.ef.Rk.col} \cdot n_{rows.1} = 3.07 \cdot 10^6$$

Design capacity in ULS:

$$F_{v.ef.Rd.col} := \frac{F_{v.ef.Rk.col.tot} \cdot k_{mod}}{\gamma_{M.con}} = 2.598 \cdot 10^6$$

Control of splitting parallell to grain: Utility:

$$\left. \begin{array}{l} \text{if } F_{Ed.col} \leq F_{v.ef.Rd.col} \\ \quad \| \text{ "OK"} \\ \text{else} \\ \quad \| \text{ "Not OK"} \end{array} \right| = \text{"OK"}$$

$$\frac{F_{Ed.col}}{F_{v.ef.Rd.col}} = 0.614$$

Splitting check: (Perpendicular to grain)

Since the connection considered has two diagonals hitting the column it means that there will be one compression force from one of the diagonals, and a tension force from the other. Looking at the force resultants on the dowels from the diagonal forces, we see that they point in the direction parallell to grain. This means that there will not be any forces perpendicular to grain, and we will not have to check it.
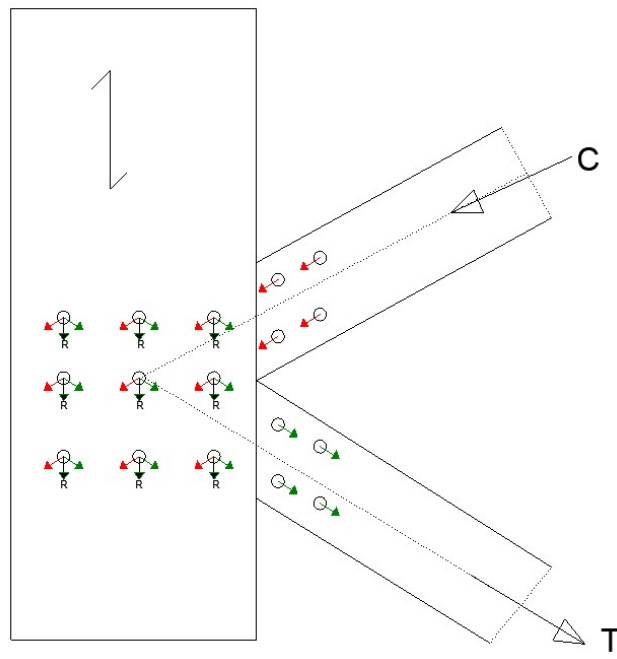
Demonstration of resultant force from diagonal forces

Control of compression of net cross section:

Net area: $A_{net.col} := A_1 - \left(n_{plates} \cdot h_{col} \cdot t_{plate} \right) - \left(d \cdot \dfrac{n_{dowels.1}}{n_{rows.1}} \cdot b_{col} \right) = 4.72 \cdot 10^5$
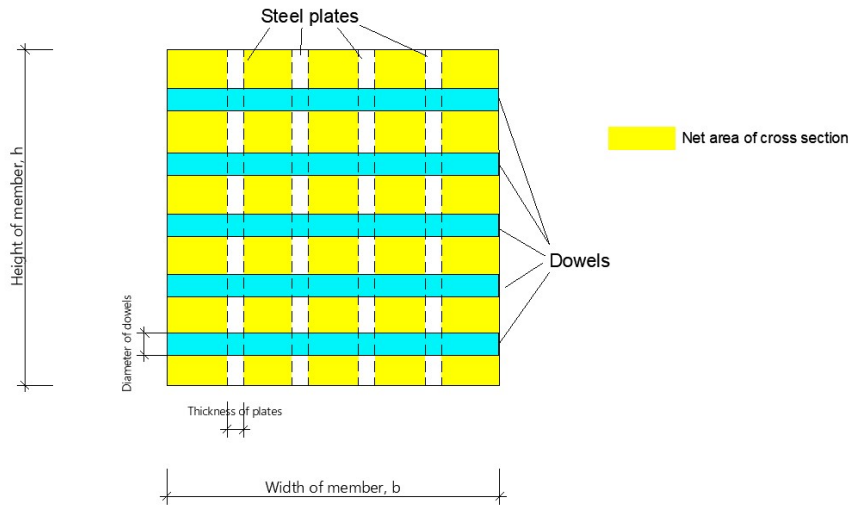
Figure: Net area of cross section.

Strength: $\qquad f_{c.0.d} := \dfrac{f_{c.0.k} \cdot k_{mod}}{\gamma_{M.GL}} = 23.435$

Compr. stress: $\qquad \sigma_{compr.col} := \dfrac{F_{col}}{A_{net.col}} = 8.474$

Utility: $\qquad u := \dfrac{\sigma_{compr.col}}{f_{c.0.d}} = 0.362$

Check:
$$\begin{array}{l|l} \text{if } u \leq 1 & = \text{``OK''} \\ \quad \left\| \text{``OK''} \right. & \\ \text{else} & \\ \quad \left\| \text{``Not OK''} \right. & \end{array}$$

Block failure:
Acc. to EC5, §8.2.3(5)

<mark>For dowel type connections we should verify that block failure will not arise, by use of Annex A in EC5.</mark>

<mark>For the column part of the connection this will not be a problem since there is <u>no tension force.</u></mark>

Check of steel plates:
Acc. to EC5, 8.2.3(2)

The capacity of the steel plates will be found according to EC3-1-1.

<mark>- Cross section classification:</mark>
Acc. to EC3-1-1, table 5.2

Factor: $\varepsilon := 0.81$

Length: $C := b_{plate.1} = 432$

Thickness: $t := t_{plate} = 16$

Slenderness: $\lambda := \dfrac{C}{t \cdot \varepsilon} = 33.333$

Classification:

$$\begin{array}{l} \text{if } \lambda \leq 33 \\ \quad \left\| \text{``Class 1''} \right. \\ \text{else if } \lambda \leq 38 \\ \quad \left\| \text{``Class 2''} \right. \\ \text{else if } \lambda \leq 42 \\ \quad \left\| \text{``Class 3''} \right. \\ \text{else} \\ \quad \left\| \text{``Class 4''} \right. \end{array} = \text{``Class 2''}$$

- Compression check:

Net area of steel plates:

$$A_{plate.net} := t \cdot \left(b_{plate.1} - n_{rows.1} \cdot d\right)$$

Capacity of steel plates:

$$N_{c.Rd} := A_{plate.net} \cdot \frac{f_y}{\gamma_0} = 1.837 \cdot 10^6$$

Control of compression capacity acc. to EC3-1-1, §6.2.4

$$\left. \begin{array}{l} \text{if } n_{plates} \cdot N_{c.Rd} \geq F_{Ed.col} \\ \quad \left\| \text{``OK''} \right. \\ \text{else} \\ \quad \left\| \text{``Not OK''} \right. \end{array} \right| = \text{``OK''}$$

Utility:

$$u := \frac{F_{Ed.col}}{n_{plates} \cdot N_{c.Rd}} = 0.217$$

The compression capacity of the steel plates in the column is OK.

Appendix C

Acc. to EC3-1-8, table 3.3

Edge distances:

$$e_1 := a_{3.t.col} = 1 \cdot 10^3$$

$$e_2 := a_{4.t.col} = 1 \cdot 10^3$$

Spacings:

$$p_1 := a_{1.col} = 100$$

$$p_2 := a_{2.col} = 141$$

Hole width for dowels: $\quad d_0 := d + 2 = 14$

Checks:

if $e_1 \geq 1.2 \cdot d_0$ = "OK"
∥ "OK"
else
∥ "Not OK"

if $e_2 \geq 1.2 \cdot d_0$ = "OK"
∥ "OK"
else
∥ "Not OK"

if $p_1 \geq 2.2 \cdot d_0$ = "OK"
∥ "OK"
else
∥ "Not OK"

if $p_2 \geq 2.4 \cdot d_0$ = "OK"
∥ "OK"
else
∥ "Not OK"

acc. to EC3-1-8, §3.5

Buckling will not occur if:

if $a_{1.col} \leq 9 \cdot t_{plate} \cdot \sqrt[2]{\dfrac{235}{f_y}}$ = "No buckling"
∥ "No buckling"
else
∥ "Buckling check must be performed"

==Control of bearing resistance in the plate:==

Acc. to EC3-1-8, table 3.4

α-values:

$$\alpha_{d.end} := \frac{e_1}{3 \cdot d_0} \qquad\qquad \alpha_{d.inner} := \frac{p_1}{3 \cdot d_0} - \frac{1}{4}$$

$$\alpha_{b.end} := min\left(\alpha_{d.end}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

$$\alpha_{b.inner} := min\left(\alpha_{d.inner}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

k-values:

$$k_{1.edge} := min\left(2.8 \cdot \frac{e_2}{d_0} - 1.7, 2.5\right) = 2.5$$

$$k_{1.inner} := min\left(1.4 \cdot \frac{p_2}{d_0}, 2.5\right) = 2.5$$

Bearing resistance:

Parallell to force direction

End dowels: $\qquad F_{b.Rd.par.end} := \dfrac{k_{1.edge} \cdot \alpha_{b.end} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$

Inner dowels: $\quad F_{b.Rd.par.inner} := \dfrac{k_{1.inner} \cdot \alpha_{b.inner} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$

Total: $\qquad F_{b.Rd.tot} := min\left(F_{b.Rd.par.end}, F_{b.Rd.par.inner}\right) \cdot n_{dowels.1} = 6.169 \cdot 10^6$

Utility: $\qquad\qquad u := \dfrac{F_{Ed.col}}{F_{b.Rd.tot}} = 0.259$

Check: $\qquad\qquad$ if $u \leq 1$ $\qquad$ $= $ "OK"
$\qquad\qquad\qquad\qquad \|$ "OK"
$\qquad\qquad\qquad$ else
$\qquad\qquad\qquad\qquad \|$ "Not OK"

Diagonal part:
We will check only the diagonal in tension since this one is more critical than the one in compression.

Embedment strength:
Acc. to NS-EN 1995-1-1, §8.2

<mark>In connections with multiple shear planes the load-carrying capacity is determined by assuming that the external members are in single shear and the middle members in double shear. The total load-carrying capacity is determined by adding the contributions of compatible failure modes. Some of the failure modes cannot occur simultaneously due to deformation compatibility, meaning that they occur at different deformation levels: either small ("brittle") or large ("ductile").</mark>

Number of middle members: $\quad n_{mid} := n_{plates} - 1 = 3$

Total nr. of shear planes for middle members: $\quad n_{sp.mid} := n_{mid} \cdot 2 = 6$

$$n_{out} := 2$$

Total nr. of shear planes for outer members: $\quad n_{sp.out} := 2$

Thickness of middle members: $\quad t_2 := \dfrac{b_{diag}}{\left(n_{plates} + 1\right)} = 108$

Thickness of outer members: $\quad t_1 := \dfrac{\left(b_{diag} - \left(n_{mid} \cdot t_2\right) - \left(n_{rows.2} \cdot t_{plate}\right)\right)}{2} = 60$

Middle members:

Formulas for double shear failure modes with external plates, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode f) $\quad F_{v.Rk.f} := f_{h.1.k.diag} \cdot t_1 \cdot d$

Failure mode g) $\quad F_{v.Rk.g} := f_{h.1.k.diag} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \dfrac{4 \cdot M_{y.Rk}}{f_{h.1.k.diag} \cdot d \cdot t_1^{\,2}}} - 1 \right)$

Failure mode h) $\quad F_{v.Rk.h} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.diag} \cdot d}$

Failure mode l) $\quad F_{v.Rk.l} := 0.5 \cdot f_{h.2.k.diag} \cdot t_2 \cdot d$

Failure mode m) $\quad F_{v.Rk.m} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.2.k.diag} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

$$F_{v.Rk.middle} := min\left(F_{v.Rk.f}, F_{v.Rk.g}, F_{v.Rk.h}, F_{v.Rk.l}, F_{v.Rk.m}\right) = 1.316 \cdot 10^4$$

Outer members:
Formulas for single shear failure modes, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode c) $\quad F_{v.Rk.c} := f_{h.1.k.diag} \cdot t_1 \cdot d$

Failure mode d) $\quad F_{v.Rk.d} := f_{h.1.k.diag} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \dfrac{4 \cdot M_{y.Rk}}{f_{h.1.k.diag} \cdot d \cdot t_1^{\,2}}} - 1 \right)$

Failure mode e) $\quad F_{v.Rk.e} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.diag} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

<mark>Total capacity per fastener per shear plane for outer members:</mark>

$$F_{v.Rk.outer} := min\left(F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}\right) = 1.316 \cdot 10^4$$

Total capacity per fastener per shear plane:

<mark>We can only combine compatible failure modes.
For the practical purpose of being able to program the comparison, the
failure modes will be identified as 1, 2, 3 etc. instead of a, b ,c etc.</mark>

<mark>Failure mode for middle members will hereafter be named "mode_mid"
Failure mode for outer members will hereafter be named "mode_outer"
It must also be noted that ∧ is the logical operator "AND", and ∨ is the
logical operator "OR", which will both be used in the if-else-statements below.</mark>

Failure mode for middle members:

$$mode\_mid := \begin{array}{l} \text{if } F_{v.Rk.middle} = F_{v.Rk.f} \\ \quad \| 6 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.g} \\ \quad \| 7 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.h} \\ \quad \| 8 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.l} \\ \quad \| 12 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.m} \\ \quad \| 13 \end{array} \quad = 7$$

Failure mode for outer members:

$$mode\_outer := \begin{array}{l} \text{if } F_{v.Rk.outer} = F_{v.Rk.c} \\ \quad \| 3 \\ \text{else if } F_{v.Rk.outer} = F_{v.Rk.d} \\ \quad \| 4 \\ \text{else if } F_{v.Rk.outer} = F_{v.Rk.e} \\ \quad \| 5 \end{array} \quad = 4$$

Comaptibility check:
Acc. to EC5, §8.1.3.2:

Now we must check if the failure mode for outer members are the same type as for the middle members. We can do this through an if-else statement:

$$Failure := \text{if } (mode\_outer = 3) \wedge ((mode\_mid = 6) \vee (mode\_mid = 12))$$
$$\qquad \left\| \text{``Brittle modes''} \right.$$
$$\qquad \text{else if } ((mode\_outer = 4) \vee (mode\_outer = 5)) \wedge ((mode\_mid = 7) \vee (mode\_mid = 8) \vee (mode\_mid = 13))$$
$$\qquad \left\| \text{``Ductile modes''} \right.$$
$$\qquad \text{else}$$
$$\qquad \left\| \text{``Incompatible modes''} \right.$$

$$Failure = \text{``Ductile modes''}$$

As we can see, both the outer failure modes and the inner failure modes are ductile, which is what we want. In situations where both ductile and brittle types are possible it is good practice to try to ensure that the design condition is based on the ductile failure mechanism. Therefor, we will not proceed until ductile compatibility is achieved in the above code.

Total capacity per fastener for the column part is:

Shear planes for outer members: $\qquad n_{sp.out} = 2$

Shear planes for middle members: $\qquad n_{sp.mid} = 6$

$$F_{v.Rk.diag} := n_{sp.out} \cdot F_{v.Rk.outer} + n_{sp.mid} \cdot F_{v.Rk.middle} = 1.052 \cdot 10^5$$

Capacity per fastener in ULS:

$$F_{v.Rd.diag} := \frac{F_{v.Rk.diag}}{\gamma_{M.con}} \cdot (k_{mod}) = 8.906 \cdot 10^4$$

Now that the capacity per fastener has been found in ULS, we can check it for the design load to see if enough dowels have been selected.

<u>Min. amount of dowels needed in the diagonal part of the connection:</u>

$$n_{diag} := \left(\frac{F_{Ed.diag}}{F_{v.Rd.diag}} \right) = 26.781$$

Check if the chosen amount of dowels (in the start of the script) is sufficient:

$$\begin{array}{l} \text{if } n_{dowels.2} \geq n_{diag} \\ \quad \left\| \text{ "OK"} \right. \\ \text{else} \\ \quad \left\| \text{ "Must increase nr. of dowels"} \right. \end{array} \right| = \text{"OK"}$$

Chosen configuration:

$$a_{1.diag} := 100 \qquad a_{2.diag} := 101$$

$$a_{3.t.diag} := 330 \qquad a_{3.c.diag} := 1000$$

$$a_{4.t.diag} := 1000 \qquad a_{4.c.diag} := 40$$

The figure of the configuration is found in the column part calculation.

The a3.c distance (and a4.t) are actually bigger, but there is no point in measuring them because they will satisfy the distance-demand. Therefor a random number (which is big enough) has been implemented.

Minimum distances:

<mark>The minimum spacings given in EC5 have been derived to prevent splitting failure when connection is subjected to lateral load. With too small spacings we get increasing tension perpendicular to grain.</mark>

EC5, table 8.5:

$$\text{if } a_{1.diag} \geq \left(3 + \left(2 \cdot \cos(\alpha)\right)\right) \cdot d \quad = \text{"OK"}$$
$$\left\| \text{"OK"}\right.$$
$$\text{else}$$
$$\left\| \text{"Not OK"}\right.$$

$$\text{if } a_{2.diag} \geq 3 \cdot d \quad = \text{"OK"}$$
$$\left\| \text{"OK"}\right.$$
$$\text{else}$$
$$\left\| \text{"Not OK"}\right.$$

$$\text{if } a_{3.t.diag} \geq \max\left(7 \cdot d, 80\right) \quad = \text{"OK"}$$
$$\left\| \text{"OK"}\right.$$
$$\text{else}$$
$$\left\| \text{"Not ok"}\right.$$

$$\text{if } a_{3.c.diag} \geq \max\left(\sin(\alpha) \cdot d, 3 \cdot d\right) \quad = \text{"OK"}$$
$$\left\| \text{"OK"}\right.$$
$$\text{else}$$
$$\left\| \text{"Not OK"}\right.$$

$$\text{if } a_{4.t.diag} \geq \max\left(\left(2 + 2 \cdot \sin(\alpha)\right) \cdot d, 3 \cdot d\right) \quad = \text{"OK"}$$
$$\left\| \text{"OK"}\right.$$
$$\text{else}$$
$$\left\| \text{"Not OK"}\right.$$

$$\text{if } a_{4.c.diag} \geq 3 \cdot d \quad = \text{"OK"}$$
$$\left\| \text{"OK"}\right.$$
$$\text{else}$$
$$\left\| \text{"Not OK"}\right.$$

All the minimum distances are fulfilled!

Appendix C

Splitting check: (Parallell to grain)

Total amount of fasteners in one row in grain direction:

$$n_{row.diag} := \frac{n_{dowels.2}}{n_{rows.2}} = 11$$

The effective number of fasteners in one row in grain direction:
EC5, eq. (8.34)

$$n_{ef.row.diag} := min\left(n_{row.diag}, n_{row.diag}^{0.9} \cdot \sqrt[4]{\frac{a_{1.diag}}{13 \cdot d}}\right) = 7.744$$

The effective load-carrying capacity of each row then becomes:

$$F_{v.ef.Rk.diag} := F_{v.Rk.diag} \cdot n_{ef.row.diag} = 8.151 \cdot 10^5$$

For entire connection: $F_{v.ef.Rk.diag.tot} := F_{v.ef.Rk.diag} \cdot n_{rows.2} = 4.89 \cdot 10^6$

Design load in ULS: $F_{v.ef.Rd.diag} := \dfrac{F_{v.ef.Rk.diag.tot} \cdot k_{mod}}{\gamma_{M.con}} = 4.138 \cdot 10^6$

Control of splitting parallell to grain:
$$\text{if } F_{Ed.diag} \leq F_{v.ef.Rd.diag} \left\| \text{``OK''} \right. = \text{``OK''}$$
$$\text{else} \left\| \text{``Not OK''} \right.$$

Utility: $\dfrac{F_{Ed.diag}}{F_{v.ef.Rd.diag}} = 0.576$

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Splitting check: (Perpendicular to grain)

No force components perpendicular to grain for the diagonals as explained in the calculation of the column part.

Control of tension of net cross section:

Net area: $A_{net.diag} := A_2 - \left(n_{plates} \cdot h_{diag} \cdot t_{plate}\right) - \left(d \cdot \dfrac{n_{dowels.2}}{n_{rows.2}} \cdot b_{diag}\right) = 2.072 \cdot 10^5$

Strength: $f_{t.0.d} := \dfrac{f_{t.0.k} \cdot k_{mod}}{\gamma_{M.GL}} = 18.652$

Compr. stress: $\sigma_{tens.diag} := \dfrac{F_{Ed.diag}}{A_{net.diag}} = 11.512$

Utility: $u := \dfrac{\sigma_{tens.diag}}{f_{t.0.d}} = 0.617$

Check:
$$\left. \begin{array}{l} \text{if } u \leq 1 \\ \quad \left\| \text{``OK''} \right. \\ \text{else} \\ \quad \left\| \text{``Not OK''} \right. \end{array} \right| = \text{``OK''}$$

Block failure:

Acc. to EC5, §8.2.3(5)

<mark>For dowel type connections we should verify that block failure will not arise, by use of <u>Annex A in EC5</u>.</mark>

Lengths in v-direction:

$$l_{v.1} := a_{3.t.diag} \quad l_{v.2} := a_{1.diag} - d = 88 \quad l_{v.3} := l_{v.2} \quad l_{v.4} := l_{v.2} \quad l_{v.5} := l_{v.2}$$

$$l_{v.6} := l_{v.2} \quad\quad l_{v.7} := l_{v.2} \quad l_{v.8} := l_{v.2} \quad l_{v.9} := l_{v.2} \quad l_{v.10} := l_{v.2} \quad l_{v.11} := l_{v.2}$$

Lengths in t-direction:

$$l_{t.1} := a_{2.diag} - d = 89 \quad\quad l_{t.2} := l_{t.1} \quad\quad l_{t.3} := l_{t.1} \quad\quad l_{t.4} := l_{t.1} \quad\quad l_{t.5} := l_{t.1}$$
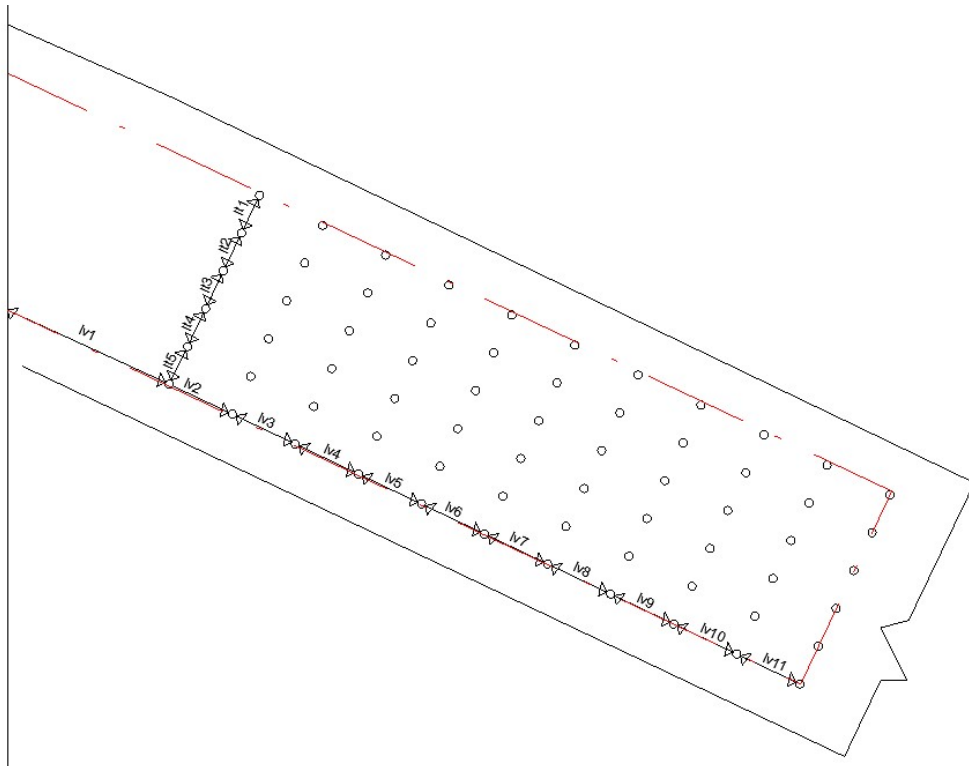
Showing distances in t- and v-direction of the diagonal. The red line is the boundary of the block shear area.

Eq. A.4:

$$L_{net.v} := l_{v.1} + l_{v.2} + l_{v.3} + l_{v.4} + l_{v.5} + l_{v.6} + l_{v.7} + l_{v.8} + l_{v.9} + l_{v.10} + l_{v.11} = 1.21 \cdot 10^3$$

Eq. A.5: $\qquad L_{net.t} := l_{t.1} + l_{t.2} + l_{t.3} + l_{t.4} + l_{t.5} = 445$

==Since we have failure modes (e) and (h) for outer and middle members, we get the following effective thickness (steel to timber connection with thick steel plates):==

Eq. A.7 $\qquad t_{ef} := 2 \cdot \sqrt[2]{\dfrac{M_{y.Rk}}{f_{h.2.k.diag} \cdot d}} = 34.644$

Thickness of diagonal:

$$t_{1.outer} := t_1 = 60$$

$$t_{1.middle} := t_2 = 108$$

Net areas:

Eq. A.2:

Outer members: $\quad A_{net.t.outer} := L_{net.t} \cdot t_{1.outer} \cdot n_{out} = 5.34 \cdot 10^4$

Middle members: $\quad A_{net.t.middle} := L_{net.t} \cdot t_{1.middle} \cdot n_{mid} = 1.442 \cdot 10^5$

$$A_{net.t} := A_{net.t.outer} + A_{net.t.middle} = 1.976 \cdot 10^5$$

Eq. A.3:

Outer members: $\quad A_{net.v.outer} := \dfrac{L_{net.v}}{2} \cdot \left(L_{net.v} + 2 \cdot t_{ef} \right) \cdot n_{out} = 1.548 \cdot 10^6$

Middle members: $\quad A_{net.v.middle} := \dfrac{L_{net.v}}{2} \cdot \left(L_{net.v} + 2 \cdot t_{ef} \right) \cdot n_{mid} = 2.322 \cdot 10^6$

$$A_{net.v} := A_{net.v.outer} + A_{net.v.middle} = 3.87 \cdot 10^6$$

Block shear capacity:

Eq. A.1:
$$F_{bs.Rk} := \max\left(1.5 \cdot A_{net.t} \cdot f_{t.0.k}, 0.7 \cdot A_{net.v} \cdot f_{v.k}\right)$$

ULS:
$$F_{bs.Rd} := \frac{F_{bs.Rk}}{\gamma_{M.con}} \cdot k_{mod} = 8.022 \cdot 10^6$$

Control of block shear:

Utilization:
$$u := \frac{F_{Ed.diag}}{F_{bs.Rd}} = 0.297$$

Check:
$$\begin{array}{l} \text{if } u \leq 1 \\ \quad \left\| \text{``OK''} \right. \\ \text{else} \\ \quad \left\| \text{``Not OK''} \right. \end{array} \quad = \text{``OK''}$$

Check of steel plates:
Acc. to EC5, 8.2.3(2)
The capacity of the steel plates will be found according to EC3.

Cross section classification:
EC3-1-1, table 5.2

Length: $\quad C := b_{plate.2} = 432$

Thickness: $\quad t := t_{plate} = 16$

Factor: $\quad \varepsilon := 0.81$

Slenderness: $\quad \lambda := \dfrac{C}{t \cdot \varepsilon} = 33.333$

$$\begin{array}{l} \text{if } \lambda \leq 33 \\ \quad \left\| \text{``Class 1''} \right. \\ \text{else if } \lambda \leq 38 \\ \quad \left\| \text{``Class 2''} \right. \\ \text{else if } \lambda \leq 42 \\ \quad \left\| \text{``Class 3''} \right. \\ \text{else} \\ \quad \left\| \text{``Class 4''} \right. \end{array} \quad = \text{``Class 2''}$$

Appendix C

Net area of steel plates:
$$A_{plate} := t \cdot (b_{plate.2} - n_{rows.2} \cdot d)$$

Capacity of steel plate:
$$N_{t.Rd} := A_{plate} \cdot \frac{f_y}{\gamma_0} = 1.778 \cdot 10^6$$

Control of compression capacity acc. to EC3-1-1, §6.2.4

$$
\begin{array}{l}
\text{if } n_{plates} \cdot N_{t.Rd} \geq F_{Ed.diag} \quad = \text{"OK"} \\
\quad \left\| \text{"OK"} \right. \\
\text{else} \\
\quad \left\| \text{"Not OK"} \right.
\end{array}
$$

Utility:
$$u := \frac{F_{Ed.diag}}{n_{plates} \cdot N_{t.Rd}} = 0.335$$

- Control of bearing resistance in the plate

Acc. to EC3-1-8, table 3.4:

α-values:
$$\alpha_{d.end} := \frac{e_1}{3 \cdot d_0}$$

$$\alpha_{d.inner} := \frac{p_1}{3 \cdot d_0} - \frac{1}{4}$$

$$\alpha_{b.end} := min\left(\alpha_{d.end}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

$$\alpha_{b.inner} := min\left(\alpha_{d.inner}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

k-values:
$$k_{1.edge} := min\left(2.8 \cdot \frac{e_2}{d_0} - 1.7, 2.5\right) = 2.5$$

$$k_{1.inner} := min\left(1.4 \cdot \frac{p_2}{d_0}, 2.5\right) = 2.5$$

Bearing resistance:
Parallell to force direction

End dowels:

$$F_{b.Rd.par.end} := \frac{k_{1.edge} \cdot \alpha_{b.end} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$$

Inner dowels:

$$F_{b.Rd.par.inner} := \frac{k_{1.inner} \cdot \alpha_{b.inner} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$$

Total:

$$F_{b.Rd.tot} := min\left(F_{b.Rd.par.end}, F_{b.Rd.par.inner}\right) \cdot n_{dowels.1} = 6.169 \cdot 10^6$$

Utility:

$$u := \frac{F_{Ed.diag}}{F_{b.Rd.tot}} = 0.387$$

Check:

$$\begin{array}{l|l} \text{if } u \leq 1 & = \text{"OK"} \\ \quad \| \text{ "OK"} & \\ \text{else} & \\ \quad \| \text{ "Not OK"} & \end{array}$$

PART 3)
Connection Stiffness

Stiffness in SLS:
Acc. to NS-EN 1995-1-1, §7.1

Total number of shear planes: $\quad n_{sp} := n_{sp.mid} + n_{sp.out} = 8$

Stiffness per shear plane per fastener:

$$K_{ser} := \frac{\rho_m^{1.5} \cdot d}{23} = 5.487 \cdot 10^3$$

Slip modulus in column part: $\quad K_{1.transl} := K_{ser} \cdot 2 \cdot n_{sp} \cdot n_{dowels.1} = 3.073 \cdot 10^6$

Slip modulus in diagonal part: $\quad K_{2.transl} := K_{ser} \cdot 2 \cdot n_{sp} \cdot n_{dowels.2} = 5.794 \cdot 10^6$

Effective length of steel plates: $\quad l_{plate.ef} := \frac{h_{col}}{2} + \frac{l_{plate.2}}{2} = 616.5$
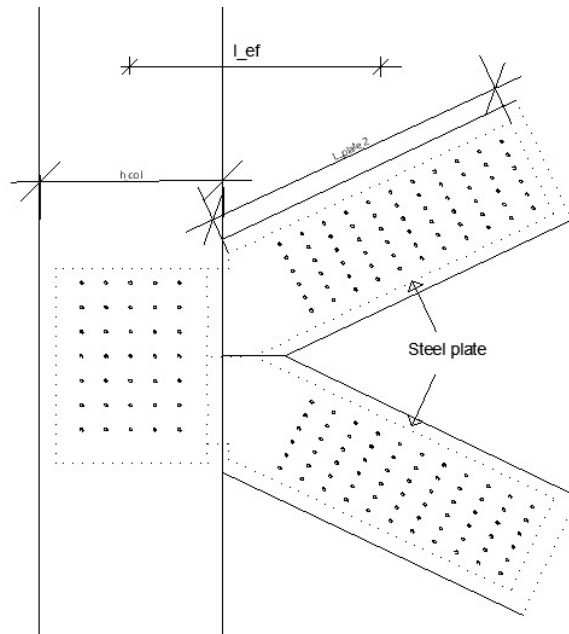
The effective length of steel plates has been found by an approximation, based on the illustration shown in the figure.

Appendix C

$$b_{plate.1} = 432 \qquad b_{plate.2} = 432$$

Width of steel plates:

$$b_{plates} := b_{plate.1} = 432$$

Net area of steel plates:

$$A_{net.plate} := t_{plate} \cdot \left(b_{plates} - n_{rows.1} \cdot d\right) = 5.952 \cdot 10^3$$

Slip modulus of steel plates:

$$K_{3.transl} := E_s \cdot \frac{A_{net.plate}}{l_{plate.ef}} \cdot n_{plates} = 8.11 \cdot 10^6$$

Total slip modulus:
The total slip modulus is calculated considering the different parts of the connection as springs in series.

$$K_{tot.con.SLS} := \left(\frac{1}{K_{1.transl}} + \frac{1}{K_{2.transl}} + \frac{1}{K_{3.transl}}\right)^{-1} = 1.609 \cdot 10^6$$

<mark>Check if the stiffness is sufficient:</mark>
Assuming same connection configuration in both ends of diagonals.

Effective length of diagonal:

$$L_{2.ef} := L_2 = 1.165 \cdot 10^4$$

Stiffness of diagonal member:

$$K_{diag} := \frac{\left(h_{diag} \cdot b_{diag}\right) \cdot E_{0.mean}}{L_{2.ef}} = 3.688 \cdot 10^5$$

System stiffness (translational):

$$K_{system.ser} := \frac{K_{diag} \cdot K_{tot.con.SLS}}{\left(K_{tot.con.SLS} + 2 \cdot K_{diag}\right)} = 2.529 \cdot 10^5$$

Ratio:
$$SLS\_ratio := \frac{K_{system.ser}}{K_{diag}} = 0.686$$

Stiffness in ULS:
Since the members have the same time-dependent properties, we may use the mean stiffness values for the calculations.

Stiffness per plane per fastener: $\quad K_u := \dfrac{2}{3} \cdot K_{ser} = 3.658 \cdot 10^3$

Slip modulus in part 1: $\quad K_{transl.1} := K_u \cdot 2 \cdot n_{sp} \cdot n_{dowels.1} = 2.048 \cdot 10^6$

Slip modulus in part 2: $\quad K_{transl.2} := K_u \cdot 2 \cdot n_{sp} \cdot n_{dowels.2} = 3.863 \cdot 10^6$

Slip modulus of steel plates: $\quad K_{transl.3} := E_s \cdot \dfrac{A_{net.plate}}{l_{plate.ef}} \cdot n_{plates} = 8.11 \cdot 10^6$

Total stiffness (slip modulus):

$$K_{tot.con.uls} := \left(\dfrac{1}{K_{transl.1}} + \dfrac{1}{K_{transl.2}} + \dfrac{1}{K_{transl.3}} \right)^{-1} = 1.149 \cdot 10^6$$

Check if the stiffness is sufficient:
Assuming same connection configuration in both ends.

Effective length of diagonal: $\quad L_{2.ef} := L_2 = 1.165 \cdot 10^4$

Stiffness of diagonal member: $\quad K_{diag} := \dfrac{(h_{diag} \cdot b_{diag}) \cdot E_{0.mean}}{L_{2.ef}} = 3.688 \cdot 10^5$

System stiffness (translational): $\quad K_{system.uls} := \dfrac{K_{diag} \cdot K_{tot.con.uls}}{(K_{tot.con.uls} + 2 \cdot K_{diag})} = 2.246 \cdot 10^5$

Ratio: $\quad ULS\_ratio := \dfrac{K_{system.uls}}{K_{diag}} = 0.609$

C3.1. Design of Beam

- Ultimate Limit State
- NS-EN-1995-1-1

Cross Section:

$$H := 540 \quad [mm] \qquad B := 360 \quad [mm]$$

Material: GL30c

Characteristic bending strength	$f_{mk} := 30$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic shear strength	$f_{vk} := 3.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength // grain	$f_{t0k} := 19.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength // grain	$f_{c0k} := 24.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength perpendicular to grain	$f_{c90k} := 2.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength perpendicular to grain	$f_{t90k} := 0.5$	$\left[\dfrac{N}{mm^2}\right]$

Bucklling length about y-axis (strong axis) $L_{ky} := 9600 \quad [mm]$

Buckling length about z-axis (weak axis) $L_{kz} := 0 \quad [mm]$

Beams are assumed restrained in slabs

Modification factor (Tab. 3.1) $k_{mod} := 0.8$

Safety factor $\gamma := 1.15$

5% - fractile Elacticity modulus $E_{0.05} := 10800 \quad \left[\dfrac{N}{mm^2}\right]$

Cross-section area

$$A := H \cdot B = 194400 \quad \left[mm^2 \right]$$

Moment of Inertia

$$I_y := \frac{1}{12} \cdot B \cdot H^3 = 4.724 \cdot 10^9 \quad \left[mm^4 \right]$$

$$I_z := \frac{1}{12} \cdot B^3 \cdot H = 2.1 \cdot 10^9 \quad \left[mm^4 \right]$$

Moment of Resistance

$$W_y := \frac{1}{6} \cdot B \cdot H^2 = 1.75 \cdot 10^7 \quad \left[mm^3 \right]$$

$$W_z := \frac{1}{6} \cdot B^2 \cdot H = 1.166 \cdot 10^7 \quad \left[mm^3 \right]$$

Height Factor (EC5: 3.3)

$$k_h := \begin{array}{l} \text{if } H < 600 \\ \quad \left\| min\left(\left(\frac{600}{H}\right)^{0.1}, 1.1\right) \right. \\ \text{else if } H \geq 600 \\ \quad \left\| 1.0 \right. \end{array} = 1.011$$

Design strength

Design bending strength (y-axis)

$$f_{myd} := \frac{f_{mk} \cdot k_{mod}}{\gamma} \cdot k_h = 21.091 \quad \left[\frac{N}{mm^2}\right]$$

Design bending strength (z-axis)

$$f_{mzd} := f_{myd} = 21.091 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength // grain

$$f_{t0d} := \frac{f_{t0k} \cdot k_{mod}}{\gamma} \cdot k_h = 13.709 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength // grain

$$f_{c0d} := \frac{f_{c0k} \cdot k_{mod}}{\gamma} = 17.043 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength perpendicular to grain

$$f_{t90d} := \frac{f_{t90k} \cdot k_{mod}}{\gamma} = 0.348 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength perpendicular to grain

$$f_{c90d} := \frac{f_{c90k} \cdot k_{mod}}{\gamma} = 1.739 \quad \left[\frac{N}{mm^2}\right]$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod}}{\gamma} = 2.435 \quad \left[\frac{N}{mm^2}\right]$$

Acting Forces (Extracted from *Robot Analysis*)

Moment: $M_{ed.y} := 325 \cdot 10^6 \quad [Nmm]$

$M_{ed.z} := 0 \cdot 10^6 \quad [Nmm]$

Bending stress (y-axis): Bending stress (z-axis):

$$\sigma_{myd} := \frac{M_{ed.y}}{W_y} = 18.576 \left[\frac{N}{mm^2}\right] \qquad \sigma_{mzd} := \frac{M_{ed.z}}{W_z} = 0$$

Shear: $V_{ed.z} := 114 \cdot 10^3 \quad [N]$

$V_{ed.y} := 0 \cdot 10^3 \quad [N]$

Shear stress:

$k_{cr} := 0.80 \quad \text{(Glulam)} \qquad \text{(EC5: 6.1.7(2))}$

$b_{ef} := k_{cr} \cdot B = 288 \quad [mm] \qquad \text{(6.13a)}$

$h_{ef} := k_{cr} \cdot H = 432 \quad [mm]$

$$\tau_{d.z} := \frac{3}{2} \cdot \frac{V_{ed.z}}{b_{ef} \cdot H} = 1.1 \quad \left[\frac{N}{mm^2}\right] \qquad \tau_{d.y} := \frac{3}{2} \cdot \frac{V_{ed.y}}{h_{ef} \cdot B} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Axial: $N_{c.ed} := 0 \cdot 10^3 \quad [N]$

$N_{t.ed} := 0 \cdot 10^3 \quad [N]$

Axial stress:

$$\sigma_{c0d} := \frac{N_{c.ed}}{A} = 0 \quad \left[\frac{N}{mm^2}\right] \qquad \sigma_{t0d} := \frac{N_{t.ed}}{A} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Appendix C

Bending - 6.1.6

Control check:

$$k_m := 0.7 \quad \text{(Glulam)} \qquad \text{(EC5 6.1.6(2))}$$

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_d}{f_{vd}} \leq 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{d.z}}{f_{vd}} = 0.452 \qquad \frac{\tau_{d.y}}{f_{vd}} = 0 \qquad (6.13)$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{t0d}}{f_{t0d}} \leq 1 \tag{6.1}$$

Utilization

$$\frac{\sigma_{t0d}}{f_{t0d}} = 0 \tag{6.1}$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{c0d}}{f_{c0d}} \leq 1 \tag{6.2}$$

Utilization

$$\frac{\sigma_{c0d}}{f_{c0d}} = 0 \tag{6.2}$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$$k_m := 0.7 \qquad\qquad (\text{EC5: 6.1.6(2)})$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.18)$$

Utilizations

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881 \qquad\qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617 \qquad\qquad (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$$k_m := 0.7 \hspace{6cm} \text{(EC5: 6.1.6(2))}$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{4cm} \text{(6.19)}$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{4cm} \text{(6.20)}$$

Utilizations

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881 \hspace{3cm} \text{(6.19)}$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617 \hspace{3cm} \text{(6.20)}$$

Stability - Buckling - 6.3.2

Buckling length (y-axis) $\qquad\qquad L_{ky} = 9600 \; [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 61.584$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.934 \qquad\qquad (6.21)$$

Buckling length (z-axis) $\qquad\qquad L_{kz} = 0 \qquad [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 0$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0 \qquad\qquad (6.22)$$

EC5: 6.3.2(3)

$$\beta_c := 0.1 \qquad \text{Glulam} \qquad\qquad (6.29)$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\;2}\right) = 0.968 \qquad\qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\;2}\right) = 0.485 \qquad\qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^{\;2} - \lambda_{rel.y}^{\;2}}} = 0.819 \qquad\qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^{\;2} - \lambda_{rel.z}^{\;2}}} = 1.031 \qquad\qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7 \qquad\qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad \text{(6.23)}$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad \text{(6.24)}$$

Utilizations

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881 \qquad\qquad \text{(6.23)}$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617 \qquad\qquad \text{(6.24)}$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 9600 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 9720 \quad [mm] \qquad \text{(Table 6.1)}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 208 \qquad \left[\frac{N}{mm^2}\right] \qquad \text{(6.32)}$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.38 \qquad \text{(6.30)}$$

$$k_{crit} := \text{if } \lambda_{rel.m} \leq 0.75 \qquad = 1$$
$$\qquad \| 1.0$$
$$\quad \text{else if } 0.75 < \lambda_{rel.m} \leq 1.4$$
$$\qquad \| 1.56 - 0.75 \cdot \lambda_{rel.m}$$
$$\quad \text{else if } 1.4 < \lambda_{rel.m}$$
$$\qquad \left\| \frac{1.0}{\lambda_{rel.m}^2} \right.$$

$$\text{(6.34)}$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \tag{6.34}$$

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \le 1 \tag{6.33}$$

Utilization

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} = 0.881 \tag{6.33}$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 \le 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 = 0.776 \tag{6.35}$$

Appendix C

C3.2. Design of Columns

- Ultimate Limit State
- NS-EN1995-1-1

Cross Section:

$$H := 720 \quad [mm] \qquad B := 720 \quad [mm] \qquad \alpha := 0 \quad [deg]$$

Material: GL30C

Characteristic bending strength	$f_{mk} := 30$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic shear strength	$f_{vk} := 3.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength // grain	$f_{t0k} := 19.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength // grain	$f_{c0k} := 24.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength perpendicular to grain	$f_{c90k} := 2.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength perpendicular to grain	$f_{t90k} := 0.5$	$\left[\dfrac{N}{mm^2}\right]$
Buckling about y-axis (strong axis)	$L_{ky} := 3500$	$[mm]$
Buckling about z-axis (weak axis)	$L_{kz} := 3500$	$[mm]$
Modification Factor (Tab. 3.1)	$k_{mod} := 0.8$	
Safety Factor	$\gamma := 1.15$	
5% - fractile Elasticity Modulus	$E_{0.05} := 10800$	$\left[\dfrac{N}{mm^2}\right]$

Appendix C

Cross Section Area	$A := H \cdot B = 5.184 \cdot 10^5 \quad \left[mm^2 \right]$

Moment of Inertia (y-axis)

$$I_y := \frac{1}{12} \cdot B \cdot H^3 = 2.239 \cdot 10^{10} \quad \left[mm^4 \right]$$

$$I_z := \frac{1}{12} \cdot B^3 \cdot H = 2.239 \cdot 10^{10} \quad \left[mm^4 \right]$$

Moment of Resistance (y-axis)

$$W_y := \frac{1}{6} \cdot B \cdot H^2 = 6.221 \cdot 10^7 \quad \left[mm^3 \right]$$

$$W_z := \frac{1}{6} \cdot B^2 \cdot H = 6.221 \cdot 10^7 \quad \left[mm^3 \right]$$

Height Factor (EC5: 3.3)

$$k_h := \begin{array}{l} \text{if } H < 600 \\ \quad \left\| min \left(\left(\frac{600}{H} \right)^{0.1}, 1.1 \right) \right. \\ \text{else if } H \geq 600 \\ \quad \left\| 1.0 \right. \end{array} \right| = 1$$

Design strength

Design bending strength (y-axis)

$$f_{myd} := \frac{f_{mk} \cdot k_{mod}}{\gamma} \cdot k_h = 20.87 \quad \left[\frac{N}{mm^2}\right]$$

Design bending strength (z-axis)

$$f_{mzd} := f_{myd} = 20.87 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength // grain

$$f_{t0d} := \frac{f_{t0k} \cdot k_{mod}}{\gamma} \cdot k_h = 13.565 \left[\frac{N}{mm^2}\right]$$

Design compression strength // grain

$$f_{c0d} := \frac{f_{c0k} \cdot k_{mod}}{\gamma} = 17.043 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength perpendiculat to grain

$$f_{t90d} := \frac{f_{t90k} \cdot k_{mod}}{\gamma} = 0.348 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength perpendiculat to grain

$$f_{c90d} := \frac{f_{c90k} \cdot k_{mod}}{\gamma} = 1.739 \quad \left[\frac{N}{mm^2}\right]$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod}}{\gamma} = 2.435 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength angle to grain

$$f_{cad} := \frac{f_{c0d}}{\frac{f_{c0d}}{f_{c90d}} \cdot \sin(\alpha \cdot \boldsymbol{deg})^2 + \cos(\alpha \cdot \boldsymbol{deg})^2}$$

Acting Forces (Extracted from *Robot Structutal Analysis*)

Moment:

$$M_{ed.y} := 0 \cdot 10^6 \qquad [Nmm]$$

$$M_{ed.z} := 0 \cdot 10^6 \qquad [Nmm]$$

Bending stress (y-axis):   Bending stress (z-axis):

$$\sigma_{myd} := \frac{M_{ed.y}}{W_y} = 0 \qquad \left[\frac{N}{mm^2}\right] \qquad\qquad \sigma_{mzd} := \frac{M_{ed.z}}{W_z} = 0$$

Shear:

$$V_{ed.z} := 0 \cdot 10^3 \qquad [N]$$

$$V_{ed.y} := 0 \cdot 10^3 \qquad [N]$$

Shear stress:

$$k_{cr} := 0.80 \quad \text{(Glulam)} \qquad \text{(EC5: 6.1.7(2))}$$

$$b_{ef} := k_{cr} \cdot B = 576 \quad [mm] \qquad \text{(6.13a)}$$

$$h_{ef} := k_{cr} \cdot H = 576 \quad [mm]$$

$$\tau_{d.z} := \frac{3}{2} \cdot \frac{V_{ed.z}}{b_{ef} \cdot H} = 0 \qquad \left[\frac{N}{mm^2}\right] \qquad\qquad \tau_{d.y} := \frac{3}{2} \cdot \frac{V_{ed.y}}{h_{ef} \cdot B} = 0 \qquad \left[\frac{N}{mm^2}\right]$$

Axial:

$$N_{c.ed} := 7600 \cdot 10^3 \quad [N]$$

$$N_{t.ed} := 0 \cdot 10^3 \qquad [N]$$

Axial stress:

$$\sigma_{c0d} := \frac{N_{c.ed}}{A} = 14.66 \qquad \left[\frac{N}{mm^2}\right] \qquad\qquad \sigma_{t0d} := \frac{N_{t.ed}}{A} = 0 \qquad \left[\frac{N}{mm^2}\right]$$

Appendix C

Bending - 6.1.6

Requirements:

$$k_m := 0.7 \quad \text{(Rectangle cross-section, Glulam)} \quad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \tag{6.11}$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \tag{6.12}$$

Utilization

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0 \tag{6.11}$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0 \tag{6.12}$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_d}{f_{vd}} \leq 1 \tag{6.13}$$

Utilization

$$\frac{\tau_{d.z}}{f_{vd}} = 0 \qquad\qquad \frac{\tau_{d.y}}{f_{vd}} = 0 \tag{6.13}$$

Axial (tensile) - 6.1.2

Requirements:

$$\frac{\sigma_{t0d}}{f_{t0d}} \leq 1 \tag{6.1}$$

Utilization

$$\frac{\sigma_{t0d}}{f_{t0d}} = 0 \tag{6.1}$$

Axial (compression) - 6.1.4

Requirements:

$$\frac{\sigma_{c0d}}{f_{c0d}} \leq 1 \tag{6.2}$$

Utilization

$$\frac{\sigma_{c0d}}{f_{c0d}} = 0.86 \tag{6.2}$$

Appendix C

Combined bending and axial tension - 6.2.3

Requirements:

$$k_m := 0.7 \qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.18)$$

Calculations

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.18)$$

Combined bending and axial compression - 6.2.4

Requirements:
$$k_m := 0.7 \qquad \text{(EC5: 6.1.6(2))}$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.20)$$

Utilization

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.74 \qquad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.74 \qquad (6.20)$$

Stability - Buckling - 6.3.2

Combined axial and bending

Buckling length (y-axis) $L_{ky} = 3500 \quad [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 16.839$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.255 \tag{6.21}$$

Buckling length (z-axis) $L_{kz} = 3500 \quad [mm]$

Slenderness

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 16.839$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.255 \tag{6.22}$$

EC5: 6.3.2(3)

$$\beta_c := 0.1 \quad \text{Glulam} \tag{6.29}$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3 \right) + \lambda_{rel.y}^2 \right) = 0.53 \tag{6.27}$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3 \right) + \lambda_{rel.z}^2 \right) = 0.53 \tag{6.28}$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^2 - \lambda_{rel.y}^2}} = 1.005 \tag{6.25}$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^2 - \lambda_{rel.z}^2}} = 1.005 \tag{6.26}$$

Appendix C

Control - Combined bending and axial

Requirements:

$$k_m := 0.7 \qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad (6.23)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad (6.24)$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.856 \qquad (6.23)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.856 \qquad (6.24)$$

Stability - LTB - 6.3.3

Simply supported beams with uniform distributed load

$$L := L_{ky} = 3500 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 4590 \quad [mm] \qquad \text{(Tabell 6.1)}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 1.321 \cdot 10^3 \left[\frac{N}{mm^2} \right] \qquad \text{(6.32)}$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.151 \qquad \text{(6.30)}$$

$$
k_{crit} := \text{if } \lambda_{rel.m} \leq 0.75 \qquad\qquad\quad\; = 1 \qquad \text{(6.34)}
$$
$$
\left\| \, 1.0 \right.
$$
$$
\text{else if } 0.75 < \lambda_{rel.m} \leq 1.4
$$
$$
\left\| \, 1.56 - 0.75 \cdot \lambda_{rel.m} \right.
$$
$$
\text{else if } 1.4 < \lambda_{rel.m}
$$
$$
\left\| \, \frac{1.0}{\lambda_{rel.m}^{\,2}} \right.
$$

Control - Bending
Requirements:

$$k_{crit} = 1 \qquad \text{(6.34)}$$

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \leq 1 \qquad \text{(6.33)}$$

Utilization

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} = 0 \qquad \text{(6.33)}$$

Control- Combination bending and compression

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \right)^2 \leq 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \right)^2 = 0.856 \tag{6.35}$$

Appendix C

C3.3. Design of Diagonals

- Ultimate Limit State
- NS-EN1995-1-1

Cross Section:

$$H := 585 \quad [mm] \qquad B := 540 \quad [mm]$$

Material: GL30C

Characteristic bending strength $\qquad\qquad\qquad f_{mk} := 30 \quad \left[\dfrac{N}{mm^2}\right]$

Characteristic shear strength $\qquad\qquad\qquad f_{vk} := 3.5 \quad \left[\dfrac{N}{mm^2}\right]$

Characteristic tension strength // grain $\qquad\quad f_{t0k} := 19.5 \left[\dfrac{N}{mm^2}\right]$

Characteristic compression strength // grain $\qquad f_{c0k} := 24.5 \left[\dfrac{N}{mm^2}\right]$

Characteristic compression strength perpendicular to grain $\qquad\qquad\qquad f_{c90k} := 2.5 \left[\dfrac{N}{mm^2}\right]$

Characteristic tension strength perpendicular to grain $\qquad\qquad\qquad f_{t90k} := 0.5 \left[\dfrac{N}{mm^2}\right]$

Bucklling length about y-axis (strong axis) $\qquad L_{ky} := 11650 \quad [mm]$

Buckling length about z-axis (weak axis) $\qquad L_{kz} := 0 \qquad [mm]$

Diagonals are assumed restrained in weak axis

Modification factor (Tab. 3.1) $\qquad\qquad k_{mod} := 1.1$

Safety factor $\qquad\qquad\qquad\qquad\qquad \gamma := 1.15$

5% - fractile Elacticity modulus $\qquad\qquad E_{0.05} := 10800 \left[\dfrac{N}{mm^2}\right]$

Cross-section area

$$A := H \cdot B = 315900 \quad \left[mm^2\right]$$

Moment of Inertia

$$I_y := \frac{1}{12} \cdot B \cdot H^3 = 9.009 \cdot 10^9 \quad \left[mm^4\right]$$

$$I_z := \frac{1}{12} \cdot B^3 \cdot H = 7.676 \cdot 10^9 \quad \left[mm^4\right]$$

Moment of Resistance

$$W_y := \frac{1}{6} \cdot B \cdot H^2 = 3.08 \cdot 10^7 \quad \left[mm^3\right]$$

$$W_z := \frac{1}{6} \cdot B^2 \cdot H = 2.843 \cdot 10^7 \quad \left[mm^3\right]$$

Height Factor (EC5: 3.3)

$$k_h := \begin{array}{l} \text{if } H < 600 \\ \quad \left\| min\left(\left(\frac{600}{H}\right)^{0.1}, 1.1\right) \right. \\ \text{else if } H \geq 600 \\ \quad \left\| 1.0 \right. \end{array} \quad = 1.003$$

Design strength

Design bending strength (y-axis)

$$f_{myd} := \frac{f_{mk} \cdot k_{mod}}{\gamma} \cdot k_h = 28.768 \quad \left[\frac{N}{mm^2}\right]$$

Design bending strength (z-axis)

$$f_{mzd} := f_{myd} = 28.768 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength // grain

$$f_{t0d} := \frac{f_{t0k} \cdot k_{mod}}{\gamma} \cdot k_h = 18.699 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength // grain

$$f_{c0d} := \frac{f_{c0k} \cdot k_{mod}}{\gamma} = 23.435 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength
perpendicular to grain

$$f_{t90d} := \frac{f_{t90k} \cdot k_{mod}}{\gamma} = 0.478 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength
perpendicular to grain

$$f_{c90d} := \frac{f_{c90k} \cdot k_{mod}}{\gamma} = 2.391 \quad \left[\frac{N}{mm^2}\right]$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod}}{\gamma} = 3.348 \quad \left[\frac{N}{mm^2}\right]$$

Acting Forces extracted from *Robot Structural*

Moment: $M_{ed.y} := 0 \cdot 10^6$

$$[Nmm]$$

$M_{ed.z} := 0 \cdot 10^6$

Bending stress (y-axis): Bending stress (z-axis):

$$\sigma_{myd} := \frac{M_{ed.y}}{W_y} = 0 \qquad \left[\frac{N}{mm^2}\right] \qquad\qquad \sigma_{mzd} := \frac{M_{ed.z}}{W_z} = 0$$

Shear: $V_{ed.z} := 0 \cdot 10^3 \qquad [N]$

$$V_{ed.y} := 0 \cdot 10^3 \qquad [N]$$

Shear stress:

$k_{cr} := 0.80$ (Glulam) (EC5: 6.1.7(2))

$b_{ef} := k_{cr} \cdot B = 432 \qquad [mm]$ (6.13a)

$h_{ef} := k_{cr} \cdot H = 468 \qquad [mm]$

$$\tau_{d.z} := \frac{3}{2} \cdot \frac{V_{ed.z}}{b_{ef} \cdot H} = 0 \quad \left[\frac{N}{mm^2}\right] \qquad\qquad \tau_{d.y} := \frac{3}{2} \cdot \frac{V_{ed.y}}{h_{ef} \cdot B} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Aksial: $N_{c.ed} := 2400 \cdot 10^3 \quad [N]$

Assumed to be similar in both, compression and tension

$N_{t.ed} := 2400 \cdot 10^3$

Axial stress:

$$\sigma_{c0d} := \frac{N_{c.ed}}{A} = 7.597 \quad \left[\frac{N}{mm^2}\right] \qquad\qquad \sigma_{t0d} := \frac{N_{t.ed}}{A} = 7.597 \quad \left[\frac{N}{mm^2}\right]$$

Bending - 6.1.6

Control check:

$$k_m := 0.7 \quad \text{(Glulam)} \qquad\qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.12)$$

Utilization

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad\qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad\qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_d}{f_{vd}} \leq 1 \qquad\qquad (6.13)$$

Utilization

$$\frac{\tau_{d.z}}{f_{vd}} = 0 \qquad\qquad \frac{\tau_{d.y}}{f_{vd}} = 0 \qquad\qquad (6.13)$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{t0d}}{f_{t0d}} \leq 1 \tag{6.1}$$

Utilization

$$\frac{\sigma_{t0d}}{f_{t0d}} = 0.406 \tag{6.1}$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{c0d}}{f_{c0d}} \leq 1 \tag{6.2}$$

Utilization

$$\frac{\sigma_{c0d}}{f_{c0d}} = 0.324 \tag{6.2}$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$$k_m := 0.7 \qquad\qquad\qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.18)$$

Utilizations

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.406 \qquad\qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.406 \qquad\qquad (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$$k_m := 0.7 \qquad\qquad\qquad \text{(EC5: 6.1.6(2))}$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad\qquad \text{(6.19)}$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad\qquad \text{(6.20)}$$

Utilizations

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.105 \qquad\qquad\qquad \text{(6.19)}$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.105 \qquad\qquad\qquad \text{(6.20)}$$

Stability - Buckling - 6.3.2

Buckling length (y-axis) $\qquad\qquad\qquad L_{ky} = 11650 \quad \left[mm\right]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 68.986$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 1.046 \qquad\qquad\qquad\qquad (6.21)$$

Buckling length (z-aksen) $\qquad\qquad\qquad L_{kz} = 0 \qquad \left[mm\right]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 0$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0 \qquad\qquad\qquad\qquad (6.22)$$

EC5: 6.3.2(3)

$$\beta_c := 0.1 \qquad \text{Glulam} \qquad\qquad\qquad\qquad\qquad (6.29)$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\;2}\right) = 1.084 \qquad\qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\;2}\right) = 0.485 \qquad\qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^{\;2} - \lambda_{rel.y}^{\;2}}} = 0.73 \qquad\qquad\qquad\qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^{\;2} - \lambda_{rel.z}^{\;2}}} = 1.031 \qquad\qquad\qquad\qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7 \qquad\qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \qquad\qquad (6.24)$$

Utilizations

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.444 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.314 \qquad\qquad (6.24)$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 11650 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 11655 \quad [mm] \tag{Table 6.1}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 360.278 \quad \left[\frac{N}{mm^2}\right] \tag{6.32}$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.289 \tag{6.30}$$

$$
k_{crit} := \begin{array}{|l}
\text{if } \lambda_{rel.m} \le 0.75 \\
\quad \left\| 1.0 \right. \\
\text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\
\quad \left\| 1.56 - 0.75 \cdot \lambda_{rel.m} \right. \\
\text{else if } 1.4 < \lambda_{rel.m} \\
\quad \left\| \dfrac{1.0}{\lambda_{rel.m}^2} \right.
\end{array} \Bigg| = 1 \tag{6.34}
$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \qquad \text{(Trykkdelen er fastholdt sideveis av takskivene)} \qquad (6.34)$$

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \leq 1 \qquad (6.33)$$

Utilization

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} = 0 \qquad (6.33)$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \qquad (6.34)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 \leq 1 \qquad (6.35)$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 = 0.314 \qquad (6.35)$$

C4.1. Structural Fire Design - Beam

- EC5
- Reduced Cross Section Method

Dimension $\quad [mm] \qquad\qquad H := 540 \quad B := 360 \qquad L := 9600$

Action Forces from *Robot Structural*

$M_{yd} := 325 \cdot 10^6 \qquad\qquad M_{zd} := 0 \cdot 10^6 \qquad\qquad [Nmm]$

$V_{zd} := 114 \cdot 10^3 \qquad\qquad V_{yd} := 0 \cdot 10^3 \qquad\qquad [N]$

$N_{cd} := 0 \cdot 10^3 \qquad\qquad N_{td} := 0 \cdot 10^3 \qquad\qquad [N]$

Buckling $\quad [mm] \qquad L_{ky} := 9600 \qquad\qquad L_{kz} := 9600$
Length

Combination Factor $\qquad\qquad\qquad \psi_{fi} := 0.3$

Material Factor (NA.2.3) $\qquad\qquad \gamma_{M.fi} := 1.0$

Modification Factor $\qquad\qquad\qquad k_{mod.fi} := 1.0$

Modification Factor $\qquad\qquad\qquad k_{fi} := 1.15$
(Glulam) Table 2.1

Reduction Factor $\qquad\qquad\qquad \eta_{fi} := 0.6$

Characteristic strength

Characteristic bending strength

$$f_{mk} := 30 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic shear strength

$$f_{vk} := 3.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic tension strength // grain

$$f_{t0k} := 19.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic compression strength // grain

$$f_{c0k} := 24.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic compression strength perpendicular to grain

$$f_{c90k} := 2.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic tension strength perpendicular to grain

$$f_{t90k} := 0.5 \quad \left[\frac{N}{mm^2}\right]$$

Fire Design Strength

Design bending strength (y-axis)

$$f_{myd.fi} := \frac{f_{mk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 34.5$$

Design bending strength (z-axis)

$$f_{mzd.fi} := f_{myd.fi} = 34.5$$

Design tension strength // grain

$$f_{t0d.fi} := \frac{f_{t0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 22.425$$

Design compression strength // grain

$$f_{c0d.fi} := \frac{f_{c0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 28.175$$

Design tension strength
perpendicular to grain

$$f_{t90d.fi} := \frac{f_{t90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 0.575$$

Design compression strength
perpendicular to grain

$$f_{c90d.fi} := \frac{f_{c90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 2.875$$

Design shear strength

$$f_{vd.fi} := \frac{f_{vk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 4.025$$

Design Fire Load

$$M_{yd.fi} := \eta_{fi} \cdot M_{yd} = 1.95 \cdot 10^8 \qquad M_{zd.fi} := \eta_{fi} \cdot M_{zd} \qquad [Nmm]$$

$$V_{zd.fi} := \eta_{fi} \cdot V_{zd} = 6.84 \cdot 10^4 \qquad V_{yd.fi} := \eta_{fi} \cdot V_{yd} \qquad [N]$$

$$N_{cd.fi} := \eta_{fi} \cdot N_{cd} \qquad N_{td.fi} := \eta_{fi} \cdot N_{td} \qquad [N]$$

Parameters

$$t_{req} := 90 \qquad [min]$$

$$\beta_0 := 0.65 \qquad \left[\frac{mm}{min}\right]$$

$$d_0 := 7 \qquad [mm]$$

$$k_0 := 1.0$$

$$d_{char.0} := \beta_0 \cdot t_{req} \qquad [mm]$$

$$d_{ef} := d_{char.0} + k_0 \cdot d_0 \qquad [mm]$$

Reduced Cross Section

$$H_{ef} := H - d_{ef} = 474.5 \qquad [mm] \qquad\qquad B_{ef} := B = 360 \qquad [mm]$$

$$A_{ef} := H_{ef} \cdot B_{ef} \qquad [mm^2] \qquad\qquad k_{cr} := 0.8$$

$$W_{y.fi} := \frac{1}{6} \cdot A_{ef} \cdot H_{ef} \qquad [mm^3] \qquad\qquad W_{z.fi} := \frac{1}{6} \cdot A_{ef} \cdot B_{ef} \qquad [mm^3]$$

Design Stresses in Fire $\qquad [MPa]$

Moment:
$$\sigma_{my.fi} := \frac{M_{yd.fi}}{W_{y.fi}} \qquad\qquad \sigma_{mz.fi} := \frac{M_{zd.fi}}{W_{z.fi}}$$

Axial:
$$\sigma_{c.fi} := \frac{N_{cd.fi}}{A_{ef}} \qquad\qquad \sigma_{t.fi} := \frac{N_{td.fi}}{A_{ef}}$$

Shear:
$$\tau_{Vz.fi} := \frac{3}{2} \cdot \frac{V_{zd.fi}}{k_{cr} \cdot A_{ef}} \qquad\qquad \tau_{Vy.fi} := \frac{3}{2} \cdot \frac{V_{yd.fi}}{k_{cr} \cdot A_{ef}}$$

Design Check in Accordance with EC5

Bending - 6.1.6

Control check:

$$k_m := 0.7 \quad \text{(Glulam)} \qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_{V.fi}}{f_{vd.fi}} \leq 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{Vz.fi}}{f_{vd.fi}} = 0.187 \qquad\qquad \frac{\tau_{Vy.fi}}{f_{vd.fi}} = 0 \qquad (6.13)$$

Appendix C

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{td.fi}}{f_{t0d.fi}} \leq 1 \tag{6.1}$$

Utilization

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} = 0 \tag{6.1}$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{cd.fi}}{f_{c0d.fi}} \leq 1 \tag{6.2}$$

Utilization

$$\frac{\sigma_{c.fi}}{f_{c0d.fi}} = 0 \tag{6.2}$$

Appendix C

Combination of Bending and Axial (tension) stress - 6.2.3
Requirements:

$$k_m := 0.7 \hspace{4cm} \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{3cm} (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{3cm} (6.18)$$

Utilizations

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \hspace{2cm} (6.17)$$

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \hspace{2cm} (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:
$$k_m := 0.7 \hspace{4cm} \text{(EC5: 6.1.6(2))}$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \hspace{2cm} (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \hspace{2cm} (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \hspace{2cm} (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \hspace{2cm} (6.20)$$

Stability - Buckling - 6.3.2

$$E_{0.05} := 10800 \qquad [MPa]$$

Buckling length (y-axis) $\qquad\qquad L_{ky} = 9600 \qquad [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 61.584$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.934 \qquad\qquad\qquad (6.21)$$

Buckling length (z-aksen) $\qquad\qquad L_{kz} = 9600 \qquad [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 92.376$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 1.4 \qquad\qquad\qquad (6.22)$$

EC5: 6.3.2(3)

$$\beta_c := 0.1 \qquad \text{Glulam} \qquad\qquad\qquad (6.29)$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^2\right) = 0.968 \qquad\qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^2\right) = 1.536 \qquad\qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^2 - \lambda_{rel.y}^2}} = 0.819 \qquad\qquad\qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^2 - \lambda_{rel.z}^2}} = 0.462 \qquad\qquad\qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7 \qquad\qquad\qquad\text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad\qquad (6.24)$$

Utilizations

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad\qquad (6.24)$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 9600 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 9720 \quad [mm] \tag{Table 6.1}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 208 \qquad \left[\frac{N}{mm^2}\right] \tag{6.32}$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.38 \tag{6.30}$$

$$k_{crit} := \begin{array}{|l}\text{if } \lambda_{rel.m} \leq 0.75 \\ \quad \left\| 1.0 \right. \\ \text{else if } 0.75 < \lambda_{rel.m} \leq 1.4 \\ \quad \left\| 1.56 - 0.75 \cdot \lambda_{rel.m} \right. \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \left\| \dfrac{1.0}{\lambda_{rel.m}^2} \right. \end{array} = 1 \tag{6.34}$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \tag{6.34}$$

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd}} \leq 1 \tag{6.33}$$

Utilization

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} = 0.418 \tag{6.33}$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} \right)^2 \leq 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} \right)^2 = 0.175 \tag{6.35}$$

C4.2. Structural Fire Design - Column

- Eurocode 5 1-2
- Reduced Cross Section Method

Dimension	$[mm]$	$B := 720$	$H := 720$

Length $\quad\quad\quad [mm] \quad\quad L := 9600$

Buckling Length $\quad [mm] \quad\quad L_{ky} := 9600 \quad\quad\quad L_{kz} := 9600$

Action Forces from *Robot Structural*

$M_{yd} := 0 \quad\quad\quad\quad\quad M_{zd} := 0 \quad\quad\quad [Nmm]$

$V_{zd} := 0 \quad\quad\quad\quad\quad V_{yd} := 0 \quad\quad\quad\quad [N]$

$N_{cd} := 7600 \cdot 10^3 \quad\quad\quad N_{td} := 0 \quad\quad\quad\quad [N]$

Combination Factor	$\psi_{fi} := 0.3$
Material Factor (NA.2.3)	$\gamma_{M.fi} := 1.0$
Modification Factor	$k_{mod.fi} := 1.0$
Modification Factor (Glulam) Table 2.1	$k_{fi} := 1.15$
Reduction Factor	$\eta_{fi} := 0.6$

Characteristic strength

Characteristic bending strength

$$f_{mk} := 30 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic shear strength

$$f_{vk} := 3.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic tension strength // grain

$$f_{t0k} := 19.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic compression strength // grain

$$f_{c0k} := 24.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic compression strength perpendicular to grain

$$f_{c90k} := 2.5 \quad \left[\frac{N}{mm^2}\right]$$

Characteristic tension strength perpendicular to grain

$$f_{t90k} := 0.5 \quad \left[\frac{N}{mm^2}\right]$$

Fire Design Strength

Design bending strength (y-axis)

$$f_{myd.fi} := \frac{f_{mk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 34.5$$

Design bending strength (z-axis)

$$f_{mzd.fi} := f_{myd.fi} = 34.5$$

Design tension strength // grain

$$f_{t0d.fi} := \frac{f_{t0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 22.425$$

Design compression strength // grain

$$f_{c0d.fi} := \frac{f_{c0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 28.175$$

Design tension strength perpendicular to grain

$$f_{t90d.fi} := \frac{f_{t90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 0.575$$

Design compression strength perpendicular to grain

$$f_{c90d.fi} := \frac{f_{c90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 2.875$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 4.025$$

Design Fire Load

$$M_{yd.fi} := \eta_{fi} \cdot M_{yd} = 0$$

$$V_{zd.fi} := \eta_{fi} \cdot V_{zd} = 0$$

$$N_{cd.fi} := \eta_{fi} \cdot N_{cd} = 4.56 \cdot 10^6$$

$$M_{zd.fi} := \eta_{fi} \cdot M_{zd} = 0 \qquad [Nmm]$$

$$V_{yd.fi} := \eta_{fi} \cdot V_{yd} = 0 \qquad [N]$$

$$N_{td.fi} := \eta_{fi} \cdot N_{td} = 0 \qquad [N]$$

Parameters

$$t_{req} := 90 \qquad [min]$$

$$\beta_n := 0.7 \qquad \left[\frac{mm}{min}\right]$$

$$d_0 := 7 \qquad [mm]$$

$$k_0 := 1.0$$

$$d_{char.n} := \beta_n \cdot t_{req} \qquad [mm]$$

$$d_{ef} := d_{char.n} + k_0 \cdot d_0 \qquad [mm]$$

Reduced Cross Section

$$H_{ef} := H - 2 \cdot d_{ef} = 580 \qquad [mm] \qquad B_{ef} := B - 2 \cdot d_{ef} = 580 \qquad [mm]$$

$$A_{ef} := H_{ef} \cdot B_{ef} \qquad [mm^2] \qquad k_{cr} := 0.8 \qquad [mm^2]$$

$$W_{y.fi} := \frac{1}{6} \cdot A_{ef} \cdot H_{ef} \qquad [mm^3] \qquad W_{z.fi} := \frac{1}{6} \cdot A_{ef} \cdot B_{ef} \qquad [mm^3]$$

Design Stresses in Fire $\qquad [MPa]$

Moment:
$$\sigma_{my.fi} := \frac{M_{yd.fi}}{W_{y.fi}} \qquad\qquad \sigma_{mz.fi} := \frac{M_{zd.fi}}{W_{z.fi}}$$

Axial:
$$\sigma_{c.fi} := \frac{N_{cd.fi}}{A_{ef}} \qquad\qquad \sigma_{t.fi} := \frac{N_{td.fi}}{A_{ef}}$$

Shear:
$$\tau_{Vz.fi} := \frac{3}{2} \cdot \frac{V_{zd.fi}}{k_{cr} \cdot A_{ef}} \qquad\qquad \tau_{Vy.fi} := \frac{3}{2} \cdot \frac{V_{yd.fi}}{k_{cr} \cdot A_{ef}}$$

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Design Check in Accordance to Eurocode 5

Bending - 6.1.6

Control check:

$$k_m := 0.7 \quad \text{(Glulam)} \qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \tag{6.11}$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \tag{6.12}$$

Utilization

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \tag{6.11}$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \tag{6.12}$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_{V.fi}}{f_{vd}} \leq 1 \tag{6.13}$$

Utilization

$$\frac{\tau_{Vz.fi}}{f_{vd}} = 0 \qquad\qquad \frac{\tau_{Vy.fi}}{f_{vd}} = 0 \tag{6.13}$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{td.fi}}{f_{t0d.fi}} \leq 1 \qquad\qquad (6.1)$$

Utilization

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} = 0 \qquad\qquad (6.1)$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{cd.fi}}{f_{c0d.fi}} \leq 1 \qquad\qquad (6.2)$$

Utilization

$$\frac{\sigma_{c.fi}}{f_{c0d.fi}} = 0.481 \qquad\qquad (6.2)$$

Combination of Bending and Axial (tension) stress - 6.2.3
Requirements:

$$k_m := 0.7 \hspace{4cm} \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{3cm} (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{3cm} (6.18)$$

Utilizations

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \hspace{2.5cm} (6.17)$$

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \hspace{2.5cm} (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$$k_m := 0.7 \hspace{4cm} \text{(EC5: 6.1.6(2))}$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \hspace{2.5cm} (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \hspace{2.5cm} (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.231 \hspace{2cm} (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.231 \hspace{2cm} (6.20)$$

Stability - Buckling - 6.3.2

$$E_{0.05} := 10800 \quad [MPa]$$

Buckling length (y-axis) $\quad\quad\quad\quad L_{ky} = 9600 \quad\quad [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 46.188$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.7 \quad\quad\quad\quad\quad (6.21)$$

Buckling length (z-aksen) $\quad\quad\quad L_{kz} = 9600 \quad\quad [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 46.188$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.7 \quad\quad\quad\quad\quad (6.22)$$

EC5: 6.3.2(3)

$$\beta_c := 0.1 \quad \text{Glulam} \quad\quad\quad\quad\quad\quad\quad\quad (6.29)$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^2\right) = 0.765 \quad\quad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^2\right) = 0.765 \quad\quad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^2 - \lambda_{rel.y}^2}} = 0.931 \quad\quad\quad\quad\quad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^2 - \lambda_{rel.z}^2}} = 0.931 \quad\quad\quad\quad\quad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7 \qquad\qquad\qquad (\text{EC5: } 6.1.6(2))$$

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad\qquad (6.24)$$

Utilizations

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.517 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.517 \qquad\qquad (6.24)$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 9600 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 10080 \quad [mm] \qquad \text{(Table 6.1)}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 601.714 \quad \left[\frac{N}{mm^2}\right] \qquad \text{(6.32)}$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.223 \qquad \text{(6.30)}$$

$$k_{crit} := \text{if } \lambda_{rel.m} \leq 0.75 \quad = 1$$
$$\qquad \left\| 1.0 \right.$$
$$\qquad \text{else if } 0.75 < \lambda_{rel.m} \leq 1.4$$
$$\qquad \left\| 1.56 - 0.75 \cdot \lambda_{rel.m} \right.$$
$$\qquad \text{else if } 1.4 < \lambda_{rel.m}$$
$$\qquad \left\| \frac{1.0}{\lambda_{rel.m}^2} \right. \qquad \text{(6.34)}$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \tag{6.34}$$

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd}} \leq 1 \tag{6.33}$$

Utilization

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} = 0 \tag{6.33}$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} \right)^2 \leq 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} \right)^2 = 0.517 \tag{6.35}$$

C4.3. Structural Fire Design - Diagonal

- EC5 1-2
- Reduced Cross Section Method

Dimension $\quad [mm] \qquad H := 585 \qquad B := 450$

Action Forces from *Robot Structural Analysis*

$M_{yd} := 0 \cdot 10^6 \qquad\qquad M_{zd} := 0 \cdot 10^6 \qquad [Nmm]$

$V_{zd} := 0 \cdot 10^3 \qquad\qquad V_{yd} := 0 \cdot 10^3 \qquad [N]$

$N_{cd} := 2400 \cdot 10^3 \qquad\quad N_{td} := 2400 \cdot 10^3 \qquad [N]$

Buckling Length $\quad [mm] \quad L_{ky} := 11650 \qquad\qquad L_{kz} := 0$ *Restrained in weak direction*

Combination Factor $\qquad\qquad\qquad\qquad \psi_{fi} := 0.3$

Material Factor (NA.2.3) $\qquad\qquad\qquad \gamma_{M.fi} := 1.0$

Modification Factor $\qquad\qquad\qquad\qquad k_{mod.fi} := 1.0$

Modification Factor $\qquad\qquad\qquad\qquad k_{fi} := 1.15$
(Glulam) Table 2.1

Reduction Factor $\qquad\qquad\qquad\qquad \eta_{fi} := 0.6$

Characteristic strength

Characteristic bending strength $\qquad f_{mk} := 30 \quad \left[\dfrac{N}{mm^2}\right]$

Characteristic shear strength $\qquad f_{vk} := 3.5 \quad \left[\dfrac{N}{mm^2}\right]$

Characteristic tension strength // grain $\qquad f_{t0k} := 19.5 \quad \left[\dfrac{N}{mm^2}\right]$

Characteristic compression strength // grain $\qquad f_{c0k} := 24.5 \quad \left[\dfrac{N}{mm^2}\right]$

Characteristic compression strength perpendicular to grain $\qquad f_{c90k} := 2.5 \quad \left[\dfrac{N}{mm^2}\right]$

Characteristic tension strength perpendicular to grain $\qquad f_{t90k} := 0.5 \quad \left[\dfrac{N}{mm^2}\right]$

Fire Design Strength

Design bending strength (y-axis)

$$f_{myd.fi} := \frac{f_{mk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 34.5$$

Design bending strength (z-axis)

$$f_{mzd.fi} := f_{myd.fi} = 34.5$$

Design tension strength // grain

$$f_{t0d.fi} := \frac{f_{t0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 22.425$$

Design compression strength // grain

$$f_{c0d.fi} := \frac{f_{c0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 28.175$$

Design tension strength perpendicular to grain

$$f_{t90d.fi} := \frac{f_{t90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 0.575$$

Design compression strength perpendicular to grain

$$f_{c90d.fi} := \frac{f_{c90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 2.875$$

Design shear strength

$$f_{vd.fi} := \frac{f_{vk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 4.025$$

Design Fire Load

$$M_{yd.fi} := \eta_{fi} \cdot M_{yd} = 0 \qquad\qquad M_{zd.fi} := \eta_{fi} \cdot M_{zd} \qquad [Nmm]$$

$$V_{zd.fi} := \eta_{fi} \cdot V_{zd} = 0 \qquad\qquad V_{yd.fi} := \eta_{fi} \cdot V_{yd} \qquad [N]$$

$$N_{cd.fi} := \eta_{fi} \cdot N_{cd} = 1.44 \cdot 10^6 \qquad N_{td.fi} := \eta_{fi} \cdot N_{td} = 1.44 \cdot 10^6 \qquad [N]$$

Parameters

$$t_{req} := 90 \qquad [min]$$

$$\beta_n := 0.7 \qquad \left[\frac{mm}{min}\right]$$

$$d_0 := 7 \qquad [mm]$$

$$k_0 := 1.0$$

$$d_{char.n} := \beta_n \cdot t_{req} \qquad [mm]$$

$$d_{ef} := d_{char.n} + k_0 \cdot d_0 \qquad [mm]$$

Reduced Cross Section

$$H_{ef} := H - 2 \cdot d_{ef} = 445 \qquad [mm] \qquad\qquad B_{ef} := B - d_{ef} = 380 \qquad [mm]$$

$$A_{ef} := H_{ef} \cdot B_{ef} \qquad [mm^2] \qquad\qquad k_{cr} := 0.8 \qquad\qquad [mm^2]$$

$$W_{y.fi} := \frac{1}{6} \cdot A_{ef} \cdot H_{ef} \qquad [mm^3] \qquad\qquad W_{z.fi} := \frac{1}{6} \cdot A_{ef} \cdot B_{ef} \qquad [mm^3]$$

Design Stresses in Fire $\qquad [MPa]$

Moment:
$$\sigma_{my.fi} := \frac{M_{yd.fi}}{W_{y.fi}} \qquad\qquad \sigma_{mz.fi} := \frac{M_{zd.fi}}{W_{z.fi}}$$

Axial:
$$\sigma_{c.fi} := \frac{N_{cd.fi}}{A_{ef}} \qquad\qquad \sigma_{t.fi} := \frac{N_{td.fi}}{A_{ef}}$$

Shear:
$$\tau_{Vz.fi} := \frac{3}{2} \cdot \frac{V_{zd.fi}}{k_{cr} \cdot A_{ef}} \qquad\qquad \tau_{Vy.fi} := \frac{3}{2} \cdot \frac{V_{yd.fi}}{k_{cr} \cdot A_{ef}}$$

Design Check in Accordance to EC5

Bending - 6.1.6

Control check:

$$k_m := 0.7 \quad \text{(Glulam)} \qquad \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \tag{6.11}$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \tag{6.12}$$

Utilization

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \tag{6.11}$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \tag{6.12}$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_{V.fi}}{f_{vd.fi}} \leq 1 \tag{6.13}$$

Utilization

$$\frac{\tau_{Vz.fi}}{f_{vd.fi}} = 0 \qquad\qquad \frac{\tau_{Vy.fi}}{f_{vd.fi}} = 0 \tag{6.13}$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{td.fi}}{f_{t0d.fi}} \leq 1 \tag{6.1}$$

Utilization

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} = 0.38 \tag{6.1}$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{cd.fi}}{f_{c0d.fi}} \leq 1 \tag{6.2}$$

Utilization

$$\frac{\sigma_{c.fi}}{f_{c0d.fi}} = 0.302 \tag{6.2}$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$$k_m := 0.7 \hspace{4cm} \text{(EC5: 6.1.6(2))}$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{3cm} (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \hspace{3cm} (6.18)$$

Utilizations

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.38 \hspace{2cm} (6.17)$$

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.38 \hspace{2cm} (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$$k_m := 0.7 \hspace{4cm} \text{(EC5: 6.1.6(2))}$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \hspace{2cm} (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \hspace{2cm} (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.091 \hspace{2cm} (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.091 \hspace{2cm} (6.20)$$

Appendix C

Stability - Buckling - 6.3.2

$$E_{0.05} := 10800$$

Buckling length (y-axis) $\qquad L_{ky} = 11650 \quad [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 68.986$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 1.046 \qquad (6.21)$$

Buckling length (z-aksen) $\qquad L_{kz} = 0 \qquad [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 0$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0 \qquad (6.22)$$

EC5: 6.3.2(3)

$$\beta_c := 0.1 \qquad \text{Glulam} \qquad (6.29)$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^2\right) = 1.084 \qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^2\right) = 0.485 \qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^2 - \lambda_{rel.y}^2}} = 0.73 \qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^2 - \lambda_{rel.z}^2}} = 1.031 \qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7 \qquad\qquad (\text{EC5: 6.1.6(2)})$$

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \qquad\qquad (6.24)$$

Utilizations

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.414 \qquad\qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad\qquad (6.24)$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 11650 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 11655 \quad [mm] \tag{Table 6.1}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 250.193 \quad \left[\frac{N}{mm^2}\right] \tag{6.32}$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.346 \tag{6.30}$$

$$k_{crit} := \text{if } \lambda_{rel.m} \leq 0.75 \quad \Big| = 1$$
$$\quad \left\| 1.0 \right.$$
$$\text{else if } 0.75 < \lambda_{rel.m} \leq 1.4$$
$$\quad \left\| 1.56 - 0.75 \cdot \lambda_{rel.m} \right.$$
$$\text{else if } 1.4 < \lambda_{rel.m}$$
$$\quad \left\| \frac{1.0}{\lambda_{rel.m}^2} \right. \tag{6.34}$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \tag{6.34}$$

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd}} \leq 1 \tag{6.33}$$

Utilization

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} = 0 \tag{6.33}$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} \right)^2 \leq 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} \right)^2 = 0.293 \tag{6.35}$$

C5. Peak Acceleration Calculation

- NS-EN1991-1-4

General remarks:

Units used in script:

- Length/height: $\left[m\right]$

- Force: $\left[N\right]$

- Velocity: $\left[\dfrac{m}{s}\right]$

- Density: $\left[\dfrac{kg}{m^3}\right]$

All equations- and chapter-references are from the EC1-1-4.

Geometry of the building:

Height: $h := 66$

Width: $b_x := 32$

Depth: $b_y := 19.2$

Fundamental values:

Damping coefficient: $\xi := \dfrac{1.9}{100} = 0.019$

Zeta faktor: $\zeta := 1.0$

Reference height: $z_s := 0.6 \cdot h$ (Figure 6.1)

Exact mode function value at top of building, extracted from *Robot:*

$$\phi_{1.x}(h) := 1$$

$$\phi_{1.y}(h) := 1$$

Exact mode function value at top floor of building, extracted from *Robot:*

$$\phi_{1.x.tfl} := 0.97$$

$$\phi_{1.y.tfl} := 0.96$$

Natural frequencies of building:

$$n_{1.x} := 0.45$$

$$n_{1.y} := 0.55$$

Air density: $\qquad \rho := 1.25$ $\qquad\qquad$ (Chapter 4.5)

Basic wind velocity:

Since the probability factor for the calculation of acceleration is different from the probability factor in the calculation of static wind load we must calculate the basic wind velocity once again.

The fundamental value of basic wind velocity:

$$v_{b.0} := 22 \qquad\qquad\text{(Table NA.4)}$$

Probability factor:
For acceleration calculation we set return period T=1, which gives cprob=0,73.

$$c_{prob} := 0.73$$

Directional factor: $\qquad c_{dir} := 1 \qquad$ (Chapter 4.2(2), NOTE2)

Season factor: $\qquad c_{season} := 1 \qquad$ (Chapter 4.2(2), NOTE3)

Factor for the wind increasing with the height over the sea:

$$c_{alt} := 1 \qquad\text{(Table NA.4(901.3))}$$

Basic wind velocity: $\qquad v_b := c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b.0} \qquad$ (eq. NA.4.1)

$$v_b = 16.06$$

Mean wind velocity:

Since the reference height for the calculation of acceleration (zs = 0,6h) is different from the reference height in the calculation of static wind load (ze = h) we must calculate the mean wind velocity once again.

Terrain category: $\qquad TK := 4 \qquad$ (Table 4.1)

Orography factor: $\qquad c_0 := 1 \qquad$ (Chapter 4.3.1, NOTE 1)

Roughness length: $\qquad\qquad\qquad$ (Chapter 4.3.2)

$$
z_0 := \left. \begin{array}{l} \text{if } TK = 0 \\ \quad \| \, 0.003 \\ \text{else if } TK = 1 \\ \quad \| \, 0.01 \\ \text{else if } TK = 2 \\ \quad \| \, 0.05 \\ \text{else if } TK = 3 \\ \quad \| \, 0.3 \\ \text{else if } TK = 4 \\ \quad \| \, 1 \end{array} \right| = 1
$$

Minimum height: $\quad z_{min} := \left. \begin{array}{l} \text{if } TK = 0 \\ \quad \| \, 2 \\ \text{else if } TK = 1 \\ \quad \| \, 2 \\ \text{else if } TK = 2 \\ \quad \| \, 4 \\ \text{else if } TK = 3 \\ \quad \| \, 8 \\ \text{else if } TK = 4 \\ \quad \| \, 16 \end{array} \right| = 16 \quad$ (Table 4.1)

Max. height: $\qquad z_{max} := 200 \qquad$ (Chapter 4.3.2)

Terrain factor: $\qquad k_r := 0.24 \qquad$ (Table NA.4.1)

Roughness factor:

$$c_r(z) := \text{if } (z \geq z_{min}) \wedge (z \leq z_{max})$$
$$\left\| k_r \cdot \ln\left(\frac{z}{z_0}\right) \right.$$
$$\text{else if } (z \leq z_{min})$$
$$\left\| c_r(z_{min}) \right.$$

(Eq. 4.4)

Mean wind velocity:

$$v_m(z) := c_0 \cdot c_r(z) \cdot v_b$$

(Eq. 4.3)

$$v_m(z_s) = 14.18$$

Wind turbulence:

Turbulence factor:

$$k_I := 1$$

(Eq. 4.7)

Standard deviation:

$$\sigma_v := k_r \cdot v_b \cdot k_I$$

(Eq. 4.6)

Turbulence intensity:

$$I_v(z) := \text{if } (z \geq z_{min}) \wedge (z \leq z_{max})$$
$$\left\| \frac{\sigma_v}{v_m(z)} \right.$$
$$\text{else if } (z \leq z_{min})$$
$$\left\| I_v(z_{min}) \right.$$

(Eq. 4.7)

Appendix C

Non-dimensional power spectral density function:

Roughness length: $\qquad\qquad\qquad z_0 = 1$

Reference height: $\qquad\qquad\qquad z_t := 200 \qquad\qquad\qquad$ (eq. B.1)

Reference length scale: $\qquad\qquad L_t := 300 \qquad\qquad\qquad$ (eq. B.1)

Factor: $\qquad\qquad\qquad\qquad \alpha := 0.67 + 0.05 \cdot \ln(z_0) = 0.67 \qquad$ (eq. B.1)

Turbulent length scale:
$$
L(z) := \text{if } z \geq z_{min} \\
\qquad \left\| L_t \cdot \left(\frac{z}{z_t}\right)^\alpha \right. \\
\quad \text{else} \\
\qquad \left\| L(z_{min}) \right.
$$

Non-dimensional frequency: $\qquad f_L(z,n) := \dfrac{n \cdot L(z)}{v_m(z)}$

Non-dimensional power spectral density function:

$$
S_L(z,n) := \frac{6.8 \cdot f_L(z,n)}{\left(1 + 10.2 \cdot f_L(z,n)\right)^{\frac{5}{3}}}
$$

Aerodynamic admittance factors:

The aerodynamic admittance functions R_h and R_b for a fundamental mode shape may be approximated using Expressions (B.7) and (B.8).

x-direction:
$$\eta_{h.x} := 4.6 \cdot \frac{h}{L\left(z_s\right)} \cdot f_L\left(z_s, n_{1.x}\right)$$

(eq. B.7)

$$\eta_{b.x} := 4.6 \cdot \frac{b_y}{L\left(z_s\right)} \cdot f_L\left(z_s, n_{1.x}\right)$$

y-direction:
$$\eta_{h.y} := 4.6 \cdot \frac{h}{L\left(z_s\right)} \cdot f_L\left(z_s, n_{1.y}\right)$$

$$\eta_{b.y} := 4.6 \cdot \frac{b_x}{L\left(z_s\right)} \cdot f_L\left(z_s, n_{1.y}\right)$$

x-direction:
$$R_{h.x} := \frac{1}{\eta_{h.x}} - \frac{1}{2 \cdot \eta_{h.x}^2} \cdot \left(1 - e^{-2 \cdot \eta_{h.x}}\right)$$

(eq. B.8)

$$R_{b.x} := \frac{1}{\eta_{b.x}} - \frac{1}{2 \cdot \eta_{b.x}^2} \cdot \left(1 - e^{-2 \cdot \eta_{b.x}}\right)$$

y-direction:
$$R_{h.y} := \frac{1}{\eta_{h.y}} - \frac{1}{2 \cdot \eta_{h.y}^2} \cdot \left(1 - e^{-2 \cdot \eta_{h.y}}\right)$$

$$R_{b.y} := \frac{1}{\eta_{b.y}} - \frac{1}{2 \cdot \eta_{b.y}^2} \cdot \left(1 - e^{-2 \cdot \eta_{b.y}}\right)$$

Equivalent mass and dimensionless coefficient:

The equivalent mass per unit length will be calculated acc. to equation F.14.
The dimensionless coefficient will be calculated acc. to equation B.11.
They both depend on the modal shape of the building, which will be extracted
from the modal analysis in Robot Structures. The mass of each floor, which is
needed for the calculation of equivalent mass, will also be extracted from
Robot. We place the degrees of freedom at the top of each floor. Since the
modal values are found directly from the modal analysis in Robot, we can use
summation instead of integration over the height of the building. These
summations, both for equivalent mass and for the dimensionless coefficient,
will be done in a separate excel sheet.

Mass from excel sheet (Appendix C5.1):

x-direction: $m_{e.x} := 53209$

y-direction: $m_{e.y} := 53218$

Dimensionless coefficient from excel sheet (Appendix C5.1):

x-direction: $K_x := 1.46$

y-direction: $K_y := 1.50$

Force coefficient: Calculated according to chapter 7, eq. 7.9.

Force coefficient for rectangular cross section and sharp edges, and
without free-end flow are given in figure 7.23:

Length-to-depth ratios:
(d/b in figure 7.23)

x-direction: $r_x := \dfrac{b_x}{b_y} = 1.667$

y-direction: $r_y := \dfrac{b_y}{b_x} = 0.6$

Force coefficients of rectangular cross sections with sharp corners and without free end flow:

We interpret the whole building as a rectangular cross section.

x-direction: $c_{f.0.x} := 1.8$

(Figure 7.23)

y-direction: $c_{f.0.y} := 2.35$

Reduction factor: $\psi_r := 1$ (NA.7)

Effective slenderness according to table 7.16:
For l=38 m we must interpolate by linear interpolation to find the slenderness.

Interpolation gives:

$$k(z) := \frac{1.4 \cdot z - 21}{35} - \frac{2 \cdot z - 100}{35}$$

$$\lambda_x := min\left(k(h) \cdot \frac{h}{b_x}, 70\right) = 2.322$$

$$\lambda_y := min\left(k(h) \cdot \frac{h}{b_y}, 70\right) = 3.87$$

Solidity ratio: $\varphi := 1$

End-effect factor:

The end effect factor accounts for reduced force caused by wind flow around the ends of a finite section. The force coefficients cf0 are based on measurements on structures without free-end flow away from the ground. The end-effect factor takes into account the reduced resistance of the structure due to the wind flow around the end (end-effect). We use figure 7.36 which is based on measurements in low-turbulent flow.

$$\psi_{\lambda.x} := 0.63$$

(Figure 7.36)

$$\psi_{\lambda.y} := 0.65$$

Force-factor:

$$c_{f.x} := c_{f.0.x} \cdot \psi_{\lambda.x} \cdot \psi_r = 1.134$$

(Eq. 7.19)

$$c_{f.y} := c_{f.0.y} \cdot \psi_{\lambda.y} \cdot \psi_r = 1.528$$

The force coefficients give overall loads on the whole structure. In effect, they represent the integration of the surface pressure distribution.

Logarithmic decrement of damping:

We will be putting the total damping of the structure equal to 1,9% based on the value given by Sweco. This damping is assumed to be the structural damping, meaning that we put aerodynamic damping and damping from special devices equal to zero.

Logarithmic decrement of structural damping:

$$\delta_s := 2 \cdot \pi \cdot \frac{\xi}{\sqrt[2]{1 - \xi^2}} = 0.119$$

Logarithmic decrement of Aerodynamic damping:

$$\delta_{a.x} := 0$$

Ignored for this project

$$\delta_{a.y} := 0$$

Logarithmic decrement of damping from special devices:

$$\delta_d := 0$$

Logarithmic decrement of damping:

$$\delta_x := \delta_s + \delta_{a.x} + \delta_d = 0.119$$

$$\delta_y := \delta_s + \delta_{a.y} + \delta_d = 0.119$$

(Eq. F.15)

$$\delta := \delta_s = 0.119 \qquad \text{(Same for both directions)}$$

Resonance response factor:

The resonance response factor squared R allowing for turbulence in resonance with the considered vibration mode of the structure should be determined using Expression (B.6)

x-direction: $\qquad R_x := \sqrt[2]{\left(\dfrac{\pi^2}{2 \cdot \delta} \cdot S_L\left(z_s, n_{1.x}\right) \cdot R_{h.x} \cdot R_{b.x}\right)}$ (Eq. B.6)

y-direction: $\qquad R_y := \sqrt[2]{\left(\dfrac{\pi^2}{2 \cdot \delta} \cdot S_L\left(z_s, n_{1.y}\right) \cdot R_{h.y} \cdot R_{b.y}\right)}$ (Eq. B.6)

Background factor:

x-direction: $\qquad B_x := \sqrt[2]{\left(\dfrac{1}{1 + 0.9 \cdot \left(\dfrac{b_y + h}{L\left(z_s\right)}\right)^{0.63}}\right)} = 0.744$ (Eq. B.3)

y-direction: $\qquad B_y := \sqrt[2]{\left(\dfrac{1}{1 + 0.9 \cdot \left(\dfrac{b_x + h}{L\left(z_s\right)}\right)^{0.63}}\right)} = 0.729$ (Eq. B.3)

Up-crossing frequency:

We put the up-crossing frequency equal to the natural frequency of the building.
(Annex B in EC1-1-4)

$$\nu_x := n_{1.x} = 0.45$$

$$\nu_y := n_{1.y} = 0.55$$

Peak factor:

Averaging time for the mean wind velocity:

$$T := 600$$

Peak factor:

x-direction:

$$k_{p1.x} := \sqrt[2]{2 \cdot \ln\left(\nu_x \cdot T\right)} + \frac{0.6}{\sqrt[2]{2 \cdot \ln\left(\nu_x \cdot T\right)}} = 3.525$$

$$k_{p2.x} := 3$$

$$k_{p.x} := \max\left(k_{p1.x}, k_{p2.x}\right) = 3.525 \qquad \text{(Eq. B.2)}$$

y-direction:

$$k_{p1.y} := \sqrt[2]{2 \cdot \ln\left(\nu_y \cdot T\right)} + \frac{0.6}{\sqrt[2]{2 \cdot \ln\left(\nu_y \cdot T\right)}} = 3.582$$

$$k_{p2.y} := 3$$

$$k_{p.y} := \max\left(k_{p1.y}, k_{p2.y}\right) = 3.582 \qquad \text{(Eq. B.2)}$$

Standard deviation:

Standard deviation for the top (roof) of the building: (Eq. B.10)

x-direction: $\sigma_{a.x}(z) := \dfrac{c_{f.x} \cdot \rho \cdot b_y \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.x}} \cdot R_x \cdot K_x \cdot \phi_{1.x}(z)$

$\sigma_{a.x} := \sigma_{a.x}(h) = 0.011$

y-direction: $\sigma_{a.y}(z) := \dfrac{c_{f.y} \cdot \rho \cdot b_x \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.y}} \cdot R_y \cdot K_y \cdot \phi_{1.y}(z)$

$\sigma_{a.y} := \sigma_{a.y}(h) = 0.016$

Standard deviation for the top floor of the building: (Eq. B.10)

x-direction: $\sigma_{a.x.tfl}(z) := \dfrac{c_{f.x} \cdot \rho \cdot b_y \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.x}} \cdot R_x \cdot K_x \cdot \phi_{1.x.tfl}$

$\sigma_{a.x.tfl} := \sigma_{a.x.tfl}(h) = 0.011$

y-direction: $\sigma_{a.y.tfl}(z) := \dfrac{c_{f.y} \cdot \rho \cdot b_x \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.y}} \cdot R_y \cdot K_y \cdot \phi_{1.y.tfl}$

$\sigma_{a.y.tfl} := \sigma_{a.y.tfl}(h) = 0.015$

Peak acceleration:

Peak acceleration on roof:
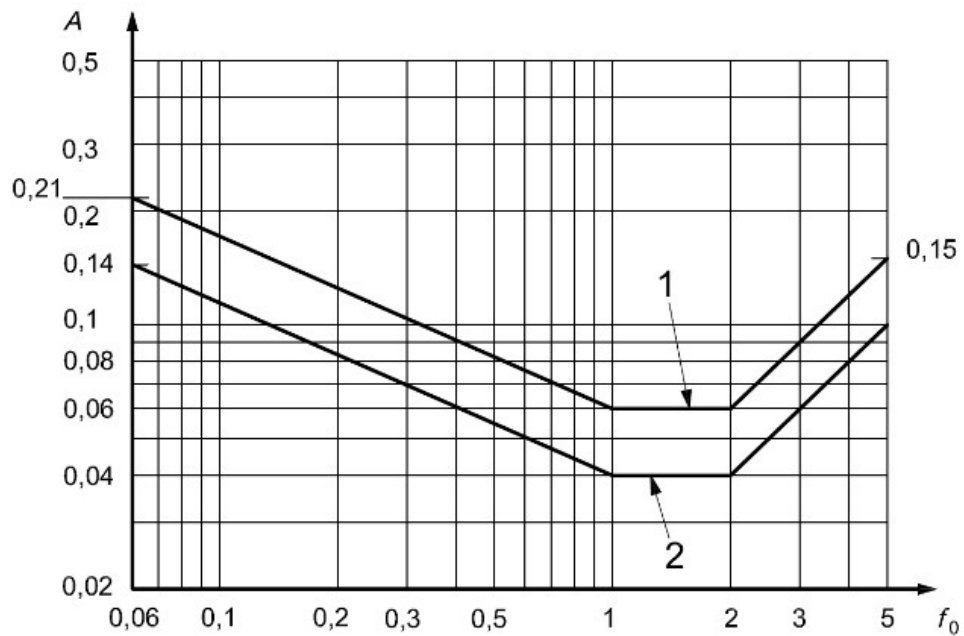
$$a_x := \sigma_{a.x} \cdot k_{p.x} = 0.039$$

$$a_y := \sigma_{a.y} \cdot k_{p.y} = 0.058$$

Peak acceleration on top Floor :

$$a_{x.tfl} := \sigma_{a.x.tfl} \cdot k_{p.x} = 0.038$$

$$a_{y.tfl} := \sigma_{a.y.tfl} \cdot k_{p.y} = 0.055$$

Requirement from ISO10137

C5.1 Equivalent Mass and Dimensionless Coefficient

Equivalent Mass and Dimensionless Coefficient in x-direction
Dimensionless Coefficient

n	z	$\Phi_{1.x}(z)$	$v_m(z)$	$v_m^2 \cdot \Phi_{1.x}(z)$	$\Phi_{1.x}(z)^2$	$v_m(z_s)^2$
1,00	5,00	0,07	14,64	15,86	0,01	377,00
2,00	10,00	0,15	14,64	32,57	0,02	
3,00	13,50	0,21	14,64	44,79	0,04	
4,00	17,00	0,27	14,96	59,52	0,07	
5,00	20,50	0,32	15,95	81,13	0,10	
6,00	24,00	0,38	16,78	106,71	0,14	
7,00	27,50	0,44	17,50	135,35	0,20	
8,00	31,00	0,51	18,13	166,01	0,26	
9,00	34,50	0,57	18,70	198,54	0,32	
10,00	38,00	0,63	19,21	231,65	0,39	
11,00	41,50	0,68	19,67	264,70	0,47	
12,00	45,00	0,73	20,10	296,51	0,54	
13,00	48,50	0,79	20,50	329,74	0,62	
14,00	52,00	0,84	20,86	363,45	0,70	
15,00	55,50	0,88	21,21	397,12	0,78	
16,00	59,00	0,93	21,53	429,66	0,86	
17,00	62,50	0,97	21,83	460,51	0,93	
18,00	66,00	1,00	22,12	489,34	1,00	
			Sum:	*4103,17*	*7,45*	
			Kx =		**1,46**	

Equivalent mass

n	[m]	[kg]	[kg/m]	$\Phi_{1.x}(z)$	$\Phi_{1.x}(z)^2$	$m(z) \cdot \Phi_{1.x}(z)^2$
1,00	5,00	352359,59	70471,92	0,07	0,01	385,90
2,00	5,00	201996,31	40399,26	0,15	0,02	933,38
3,00	3,50	185519,64	53005,61	0,21	0,04	2315,34
4,00	3,50	185519,64	53005,61	0,27	0,07	3750,47
5,00	3,50	185519,64	53005,61	0,32	0,10	5393,90
6,00	3,50	185519,64	53005,61	0,38	0,14	7613,78
7,00	3,50	185519,64	53005,61	0,44	0,20	10355,39
8,00	3,50	185519,64	53005,61	0,51	0,26	13517,76
9,00	3,50	185519,64	53005,61	0,57	0,32	17100,88
10,00	3,50	185519,64	53005,61	0,63	0,39	20904,57
11,00	3,50	185519,64	53005,61	0,68	0,47	24798,99
12,00	3,50	185519,64	53005,61	0,73	0,54	28557,09
13,00	3,50	185519,64	53005,61	0,79	0,62	32663,38
14,00	3,50	185519,64	53005,61	0,84	0,70	36956,84
15,00	3,50	185519,64	53005,61	0,88	0,78	41327,89
16,00	3,50	211493,72	60426,78	0,93	0,86	51926,48
17,00	3,50	206794,86	59084,25	0,97	0,93	55134,82
18,00	3,50	149321,94	42663,41	1,00	1,00	42663,41
				Sum:	*7,45*	*396300,28*
				me =		**53208,58**

Equivalent Mass and Dimensionless Coefficient in y-direction

Dimensionless Coefficient

n	z	$\Phi_{1.y}(z)$	$v_m(z)$	$v_m^2 \cdot \Phi_{1.y}(z)$	$\Phi_{1.y}(z)^2$	$v_m(z_s)^2$
1,00	5,00	-0,06	14,64	-11,79	0,00	377,00
2,00	10,00	-0,13	14,64	-27,00	0,02	
3,00	13,50	-0,18	14,64	-38,79	0,03	
4,00	17,00	-0,23	14,96	-52,14	0,05	
5,00	20,50	-0,29	15,95	-73,25	0,08	
6,00	24,00	-0,35	16,78	-97,42	0,12	
7,00	27,50	-0,41	17,50	-124,63	0,17	
8,00	31,00	-0,47	18,13	-153,52	0,22	
9,00	34,50	-0,53	18,70	-184,56	0,28	
10,00	38,00	-0,59	19,21	-216,53	0,34	
11,00	41,50	-0,65	19,67	-250,77	0,42	
12,00	45,00	-0,70	20,10	-283,99	0,49	
13,00	48,50	-0,76	20,50	-319,23	0,58	
14,00	52,00	-0,82	20,86	-354,74	0,66	
15,00	55,50	-0,87	21,21	-390,37	0,75	
16,00	59,00	-0,91	21,53	-423,64	0,84	
17,00	62,50	-0,96	21,83	-456,22	0,92	
18,00	66,00	-0,99	22,12	-484,45	0,98	
			Sum:	-3943,03	6,96	
				Ky =	**-1,50**	

Equivalent mass

n	[m]	[kN]	[kN/m]	$\Phi_{1.y}(z)$	$\Phi_{1.y}(z)^2$	$m(z) \cdot \Phi_{1.y}(z)^2$
1,00	5,00	352359,59	70471,92	-0,06	0,00	213,18
2,00	5,00	201996,31	40399,26	-0,13	0,02	641,38
3,00	3,50	185519,64	53005,61	-0,18	0,03	1736,52
4,00	3,50	185519,64	53005,61	-0,23	0,05	2877,62
5,00	3,50	185519,64	53005,61	-0,29	0,08	4396,50
6,00	3,50	185519,64	53005,61	-0,35	0,12	6345,62
7,00	3,50	185519,64	53005,61	-0,41	0,17	8780,33
8,00	3,50	185519,64	53005,61	-0,47	0,22	11559,94
9,00	3,50	185519,64	53005,61	-0,53	0,28	14777,12
10,00	3,50	185519,64	53005,61	-0,59	0,34	18264,09
11,00	3,50	185519,64	53005,61	-0,65	0,42	22257,27
12,00	3,50	185519,64	53005,61	-0,70	0,49	26195,85
13,00	3,50	185519,64	53005,61	-0,76	0,58	30616,04
14,00	3,50	185519,64	53005,61	-0,82	0,66	35207,65
15,00	3,50	185519,64	53005,61	-0,87	0,75	39935,70
16,00	3,50	211493,72	60426,78	-0,91	0,84	50480,29
17,00	3,50	206794,86	59084,25	-0,96	0,92	54112,25
18,00	3,50	149321,94	42663,41	-0,99	0,98	41814,41
				Sum:	*6,96*	*370211,74*
					me =	**53218,80**

Appendix D
Note – *Økern Sentrum*

NOTAT – ØKERN SENTRUM

KUNDE / PROSJEKT	PROSJEKTLEDER	DATO
Steen & Strøm	Daniel Adolfsson	01.07.2020

PROSJEKTNUMMER	OPPRETTET AV	REV. DATO
10214111	Jonas Johnstad og Cathrine Hafnor Revidert av Daniel Adolfsson	20.11.2020

Bakgrunn

I forbindelse med utvikling av område på Økern, er det bedt om et estimat av muligheter ang. antall etasjer som kan bygges over flere tunneler og kulverter på Økern. Det vises til et Premissnotat fra «Aas-Jakobsen» for Statens vegvesen (DOK.nr B014), som legger føringer for mulig ovenforliggende bebyggelse over Økerntunnelen, Lørentunnelen, rampe Grorud – Sinsen, samt en flere tekniske- og VA kulverter. Dette notatet tar for seg et grovt estimat over mulig antall etasjer over de ulike tunneler mht. fundamentering direkte på tunnelenes vegger og/eller tak med de kapasiteter gitt i Aas-Jakobsen sitt notat.

Forutsetninger

For beregningene er det gjort følgende forutsetninger:

- For konstruksjoner med bærende betongvegger (ikke søyler) settes det at veggkonstruksjonene opptar 6 % og massivtreveggene 9 % av BYA. Det er tatt utgangspunkt i et bygg med dimensjoner lik 20 m x 30 m for beregning av andel bærende vegger.

- Det er lagt inn restriksjonssoner som gjelder området over tunneler og arealer inntil ca.10 m utenfor tunneler. For å få et fornuftig areal i kjeller, må denne hensynssonen revurderes. Sweco har ansvar for plan-prosjektering av bebyggelse i sone 1, 2, 3 og 4. I disse sonene skal all byggeaktivitet godkjennes av Statens vegvesen før den kan igangsettes. Videre er krav, kapasiteter og andre forutsetninger hentet fra notat av «Aas-Jakobsen» for Statens vegvesen. (DOK.nr B014).

- I dette notat er det tatt utgangspunkt i at byggets last fordeles ned på tuneller i form av flatelast. Der det er behov og mulig, kan det ved hjelp av bjelker føres laster ned på utsiden av tunell/kulvert som kan gi en høyere kapasitet. Dette tas i utgangspunktet ikke med i dette notatet.

Laster

For beregningene er det brukt følgende laster:
Nyttelast: 5 kN/m2 for næring, 3 kN/m2 for kontor, og 2kN/m2 for bolig. Det antas 2 etasjer med næring, og resterende i bolig eller kontor.
Egenlaster: Hulldekker 4 kN/m2, bærevegg betong 25 kN/m3, lettvegger 0,5 kN/m2, teknisk + himling 1 kN/m2.
For tak det er beregnet 3 alternativer:

- Takkonstruksjon med ordinær taktekking (1,5kN/m2)

- Takkonstruksjon med 0,7m vannmettet «lettjord» fra Bergknapp (10kN/m2)

- Takkonstruksjon med 0,7m vannmettet ordinær jord (16,5kN/m2)

Snølast: 2,8 kN/m2 (dimensjonerende). Det er i tillegg medregnet en teknisk etasje (på tak) med 5kN/m2 nyttelast.

Materialer og bæresystemer

Det er gjort et overslag over antall mulige etasjer over de ulike tunnelene/kulvertene. Det er tatt utgangspunkt i 2 etasjer med næringslokaler og resterende etasjer som bolig eller kontorer (det er gjort beregninger for begge tilfeller). Disse 2 mulighetene er igjen kontrollert med ulike byggematerialer:

- **Betong**: komplett bæresystem i betongelementer, hulldekker og innvendige lettvegger.

- **Betong/stål**: betongvegger i trappehus/heissjakt, stålsøyler med lette vegger som yttervegger, samt hulldekker.

- **Massivtre**: massivtre veggelementer i yttervegger og bærevegger, massivtredekker, 4-5 betongdekker. For massivtre er det antatt flere bærevegger grunnet begrensning i spenn på massivtredekker (maks 7,5m).

Område 1

- Fra Aas-Jakobsens notat fremkommer det at lastene som påføres tunnelens fundamenter fra overliggende bebyggelse må påføres som uavhengige stripelaster over tunnelens vegger. Typisk verdi av stripelastene er 1000-1370 kN/m, avhengig av tunnelbredden. Laster fra fremtidig bebyggelse kan kun settes på oppstikkende betongvegger over tunneltaket. I våre beregninger brukes det da 11m lastbredde, og en maksimal linjelast på 1370kN/m.
- Teknisk kulvert er fundamentert i løsmasser, og er ikke dimensjonert for overliggende bygg. Kulverten er også sensitiv for setninger og det må vises varsomhet ved fundamentering rundt. Dvs. at ovenforeliggende bebyggelse må fundamenteres ned på hver side av kulvert, uten belastning på kulverts tak.
- Det kan være mulig å fundamentere på fjell ved siden av kulvert. Lastbredden over kulvert til tunnelfundament ser ut til å være omtrent like stor som tunnelveggene. Det tas derfor utgangspunkt i samme maksimale karakteristisk nyttelast som i område direkte over tunnel; 1370 kN/m.

Felt F9

Felt F9 er plassert over setningssensitiv kulvert, og det må vises varsomhet ved fundamentering rundt denne. Ellers kan ingen laster føres ned på kulvert. Her vil evt. utveksling av konstruksjon til siden av tunnel og kulvert være nødvendig, for å føre laster til siden for kulvert og ned gjennom fundamenter direkte til berg. Krav fra Statens Vegvesen for mulig lastnedføring rundt dette feltet vil være førende for hva som kan bygges. I tillegg er tilgjengelig plass mellom kulvert og andre konstruksjoner samt ovenforliggende bebyggelse av vesentlig betydning med tanke på tilstrekkelig høyde for bæresystem. Dette bør dog være løsbart slik at planlagt høyde på felt kan bygges. Adkomster til kulverter må ivaretas ved overbygging.

Område 2 - Økerntunnelen:

Fra notat av Statens vegvesen, er det ved dimensjonering av Økerntunnelen gjort antagelser om plasstøpte bygg med maksimalt 8 etg. Der er det beregnet egenvekt på 10kPa/etg og en nyttelast på 5kPa/etg. Ved å bygge i annet materiale og/eller bruke annen nyttelast, kan denne begrensningen økes. Se videre resultater.

Forutsetninger for beregninger:

- Det tas utgangspunkt i karakteristisk linjelast på 1370 kN/m når maksimalt antall etasjer beregnes.
- Det benyttes en lastbredde på 11 m, hvilket er verst tenkelige tilfelle. I enkelte tilfeller er det kanskje mulig å nedjustere denne da bygget ikke står fullstendig over tunnel. Her vil det også bli ekstra utfordringer med å overføre lasten ned på tunnelens fundamenter.

Felt F2 og F3

Hjørne på F3 er så vidt plassert over setningssensitiv kulvert, og det må vises varsomhet ved fundamentering rundt denne. Ellers kan ingen laster føres ned på kulvert. Her vil evt. utvekslingskonstruksjon være nødvendig, for å føre laster til siden for kulvert og ned gjennom fundamenter direkte til berg. Krav fra Statens Vegvesen rundt mulig lastnedføring rundt denne vil være førende for hva som kan bygges. I tillegg er tilgjengelig høyde mellom kulvert og ovenforliggende bebyggelse vesentlig mtp. plass til bæresystem. Dette bør dog være løsbart slik at planlagt høyde kan bygges.

Hjørnet av F3 og F2 mot rundkjøring er til dels plassert over teknisk bygg, der både ventilasjon og adkomst må ivaretas. Hjørne F3 er i tillegg plassert over adkomst til kulvert som også må ivaretas.

For øvrige bygg i dette området er det den maksimale karakteristiske stripelasten som begrenser høyden på bygget.

Område 3, 4 og 5 - Lørentunnelen, rampe Grorud – Sinsen

Lørentunnelen er ikke dimensjonert for overliggende bebyggelse. Det ser også ut til at det er lite sannsynlig at Statens vegvesen tillater overliggende bebyggelse pga. hindret adkomst fra oversiden ved en hendelse i tunnelen som krever vedlikehold/utbedring.

Grorud – Sinsen rampe er ifølge Aas-Jakobsens rapport dimensjonert for en karakteristisk terrenglast på 40kPa, og er generelt sensitiv for skjevlast pga. det sirkulære tverrsnittet. På bakgrunn av dette, bør trolig last fra overliggende bebyggelse føres ut på hver side av tunnelveggene, evt. bygge lave bygg i lette materialer. Ved lavere utgravninger på en side enn den andre, må det etableres spuntvegg langs tunnelen. Ifølge rapporten er det er fra Statens Vegvesens side ikke stilt spesifikke krav til tilkomst for vedlikehold av denne tunnelen. Dersom det mot formodning skulle skje en hendelse der det blir behov for tilkomst for tyngre vedlikehold vil dette kunne medføre at overliggende bebyggelse må rives. Sannsynligheten for en slik hendelse er svært liten.

SWECO

Felt F1

Felt F1 er plassert over påkjøringsrampe til Lørentunnelen, og det bør derfor kontrolleres mulighet for bygg her med Statens Vegvesen i tidlig fase.

Utenfor definerte soner

Felt F7 og F8

F8 vil bygges over innkjøring til tunnel samt ved siden av og vil måtte godkjennes av Statens Vegvesen for å kunne bygges. Det gjelder også forsiktighet med fundamentering rundt VA-kulvert. Det vil også her være bæresystem over kulvert som blir dimensjonerende for antall mulige etasjer, og tilgjengelig høyde mellom øvre del av kulvert og ovenforeliggende bygg er essensielt. Dette bør dog være løsbart slik at planlagt høyde kan bygges.

F7 vil bli inneklemt mellom Østre Aker Vei og Økernveien. Feltet ser ut til å ligge over en undergang/trapper ned til t-bane. Dette er ikke nevnt i notat fra Statens Vegvesen, men det antas at adkomst her også må ivaretas, og at bygget må fundamenteres utenfor dette.

Felt F4 og F10

Felt F4 og F10 antas i mindre grad å være begrenset av konstruksjoner i grunnen.

Konklusjon

For å komme i nærheten av de bygningshøyedene som er planlagt over Økern-området anbefales det benyttes en kombinasjon av massivtre og betongdekker. I sone 1 og 2 kan det da være mulig å oppnå omtrent 21 etasjer uten grønt tak, 17 etasjer med grønt tak og 19 etasjer dersom det benyttes «lettjord». For å oppnå disse høydene forutsettes det som tidligere nevnt at kun de to nederste etasjene er næring og at resterende etasjer benyttes til boligformål. Dersom etasjene over næringslokalene skal benyttes som kontor vil maksimalt antall etasjer være omtrent 18 uten grønne tak og 15 med grønne tak med lettjord. Byggene i sone 3 og 5 er planlagt over Lørentunnelen og/eller rampen ned til Grorud-Sinsen tunnelen. I disse sonene tåler fundamentet svært lave laster og med massivtre vil det maksimalt kunne bygges 3 etasjer. Ifølge notatet av «Aas-Jakobsen» for Statens vegvesen vil det trolig ikke være mulig å bygge noe som helst over Lørentunnelen.

Det må allikevel presiseres at dette kun er et estimat gjort ut ifra gitte forutsetninger. Dersom man optimaliserer byggene og har tilstrekkelig med tid og økonomiske midler er det sannsynligvis mulig å bygge høyere enn disse estimatene.