TALAT Lectures 2710

Static Design Example

82 pages

Advanced Level

prepared during the TAS project:

TAS

Leonardo da Vinci
Training in Aluminium Alloy Structural Design

Example developed with the "Mathcad" Software

Date of Issue: 1999

2710 Static Design example

Table of Contents (Active)

Table of Contents (Complete)

6 CODE CHECKING

Software

The example is worked out using the MathCad software in which some symbols have special meani
according to the following

$x := 50.6 \cdot mm$	Assign value
$y \equiv 2.5 \cdot mm$	Global assignment
$x + y = 53.1 \circ mm$	Evaluate expression
$a = b$	Boolean equals
0.5	Decimal point
$c := (1 \quad 3 \quad 2)$	Vector
$d := (2 \quad 4 \quad 3)$	Vector
$a := \overrightarrow{(c \cdot d)}$	Vectorize (multiply the elements in vector c with corresponding elements in d)
$a = (2 \quad 12 \quad 6)$	Result

Structure

The structure was proposed by Steinar Lundberg, who also contributed with valuable suggestions.
Part 1 to 6.6 was worked out by Torsten Höglund and 6.7 by Myriam Bouet-Griffon.

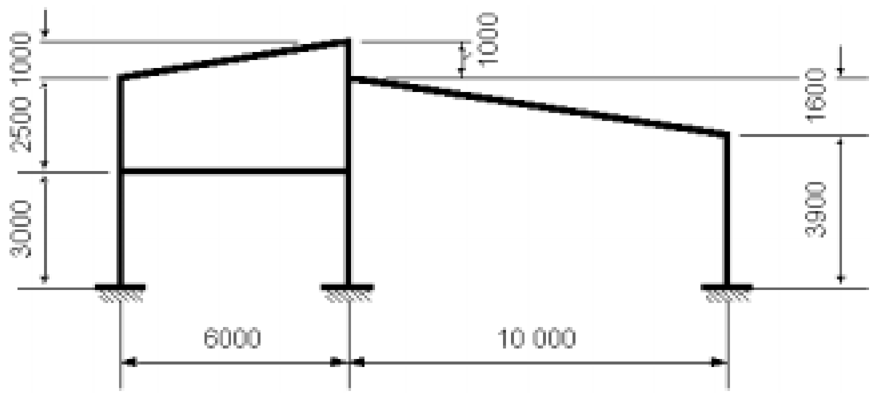
1. Introduction

1.1 Description

The industrial building contains an administration part with offices, wardrobe, meeting rooms etc. and a fabrication hall. The load bearing system consist of frames standing at a distance of 5000 m

In the serviceability limit state max. allowable deflection is 1/250 of span.

1.2 Sketches



1.3 References

[1] ENV 1999-1-1. Eurocode 9 - Design of aluminium structures - Part 1-1: General rules. 1997

[2] ENV 1991-2-1. Eurocode 1 - Basis of design and actions on structures - Part 2-1: Action on structures - Densities, self-weight and imposed loads. 1995

[3] ENV 1991-2-3. Eurocode 1 - Basis of design and actions on structures - Part 2-3: Action on structures - Snow loads. 1995

[4] ENV 1991-2-4. Eurocode 1 - Basis of design and actions on structures - Part 2-4: Action on structures - Wind loads. 1995

[5] ENV 1991-1. Eurocode 1 - Basis of design and actions on structures - Part 1: Basis of design. 1994

1.4 S.I. units

$kN \equiv 1000 \cdot N$ $\quad MPa \equiv 1000000 \cdot Pa$ $\quad kNm \equiv kN \cdot m$ $\quad MPa = 1 \circ \frac{N}{mm^2}$

2. Materials

2.1 Aluminium

[1], 3.2.2 The extrusions are alloy EN AW-6082, temper T6
The plates are EN AW-5083 temper H24

Strength of aluminium alloys

EN AW-6082 T6 $f_o := 260 \cdot MPa$ $f_u := 310 \cdot MPa$
EN AW-5083 H24 $f_o := 250 \cdot MPa$ $f_u := 340 \cdot MPa$

[1], 5.1.1 The partial safety factor for the members $\gamma_{M1} := 1.10$ $\gamma_{M2} := 1.25$

[1], 6.1.1 The partial safety factor for welded connections $\gamma_{Mw} := 1.25$

Design values of material coefficients

Modulus of elasticity $E := 700000 \cdot MPa$
Shear modulus $G := 27000 \cdot MPa$
Poisson´s ratio $\nu := 0.3$

Coefficient of linear thermal expansion $\alpha_T := 23 \cdot 10^{-6}$

Density $\rho := 2700 \cdot kg \cdot m^{-3}$

2.2 Other materials

Comment: Properties of any other materials to be filled in

3. Loads

3.1 Permanent loads

[3], ?? Permanent loads are self-weight of structure, insulation, surface materials and fixed equipment

Permanent load on roof $q'_{p.roof} := 0.5 \cdot kN \cdot m^{-2}$

Permanent load on floor $q'_{p.floor} := 0.7 \cdot kN \cdot m^{-2}$

3.2 Imposed loads

[2], 6.3.1 Office area => Category B

Uniform distributed load $q'_{k.floor} := 3 \cdot kN \cdot m^{-2}$

Concentrated load $Q_{k.floor} := 2 \cdot kN$

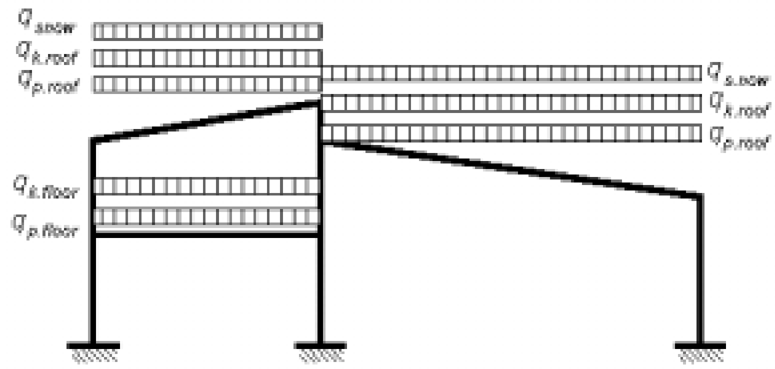
[2], 6.3.4 Roofs not accessible except for normal maintenance etc. => Category H =>

Uniform distributed load $q'_{k.roof} := 0.75 \cdot kN \cdot m^{-2}$

Concentrated load $Q_{k.roof} := 1.5 \cdot kN$

[?], ?? Load from crane, the crane located in the middle of the roof beam in the production hall

Concentrated load $Q_{crane} := 50 \cdot kN$



3.3 Environmental loads

3.3.1 Snow loads

[3] *Comment: The characteristic values of the snow loads vary from nation to nation. For simplicity for this design example, a snow load is chosen including shape coefficient, exposure coefficient and thermal coefficient. In a design report the calculation of the snow loads have to be shown.*

Snow load $q'_{snow} := 2\ kN \cdot m^{-2}$

3.3.2 Wind loads

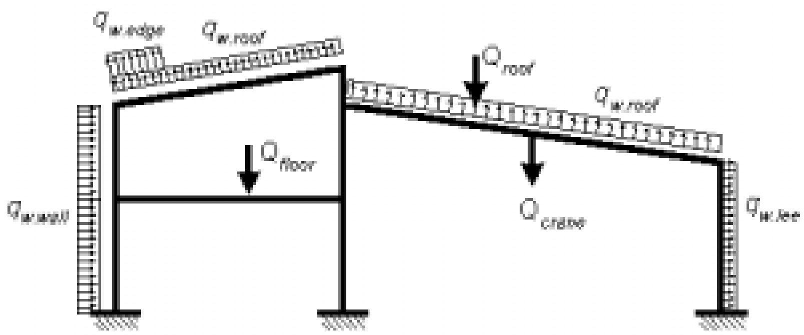
[3] *Comment: The characteristic values of the wind loads vary from nation to nation. For simplicity, for this design example, a wind load is chosen including all coefficients. In a design report the calculation of the wind loads including all coefficients have to be shown.*

Maximum wind load on the external walls $q'_{w.wall} := 0.70\ kN \cdot m^{-2}$

Wind suction on leeward side of walls $q'_{w.lee} := -0.27\ kN \cdot m^{-2}$

Wind suction on the roof $q'_{w.roof} := 0.70\ kN \cdot m^{-2}$

Max. wind suction at the lower edge of the roof and 1.6 m upwards $q'_{w.edge} := 1.0\ kN \cdot m^{-2}$



4. Load Combinations

4.1 Ultimate limit state

[3] To decide the section forces on the different members, the following load combinations to be calculated in the ultimate limit state

LC 1: Permanent + imposed + crane + snow loads imposed load dominant
LC 2: Permanent + imposed + crane + snow loads crane load dominant
LC 3: Permanent + imposed + crane + snow loads snow load dominant
LC 4: Reduced permanent + wind loads wind load dominant
LC 5: Permanent + imposed + crane + snow + wind imposed load dominant
LC 6: Permanent + imposed + crane + snow + wind wind load dominant

[3] *Comment: All possible load combinations to be calculated*

[5], 9.4 Partial load factors for different load combinations in the *ultimate limit state*

[5] Table 9.2 Partial factor for permanent action, unfavourable $\gamma_{Gsup} := 1.35$

Partial factor for permanent action, favourable $\gamma_{Ginf} := 1.0$

Partial factor for variable action, unfavourable $\gamma_Q := 1.5$

[5] Table 9.3 ψ factor for imposed loads $\psi_{0i} := 0.7$

ψ factor for snow loads $\psi_{0s} := 0.6$

ψ factor for wind loads $\psi_{0w} := 0.6$

[5] Eq (9.10b) In load combinations where the imposed load is dominating ξ in Eq (9.10b) is less than 1.0, say $\xi := 0.9$

Load combinations

$$\psi_u := \begin{array}{l} \\ \left[\begin{array}{cccccc} \xi\cdot\gamma_{Gsup} & \xi\cdot\gamma_{Gsup} & \xi\cdot\gamma_{Gsup} & \gamma_{Ginf} & \xi\cdot\gamma_{Gsup} & \xi\cdot\gamma_{Gsup} \\ \gamma_Q & \psi_{0i}\cdot\gamma_Q & \psi_{0i}\cdot\gamma_Q & 0 & \gamma_Q & \psi_{0i}\cdot\gamma_Q \\ \psi_{0i}\cdot\gamma_Q & \gamma_Q & \psi_{0i}\cdot\gamma_Q & 0 & \psi_{0i}\cdot\gamma_Q & \psi_{0i}\cdot\gamma_Q \\ \psi_{0s}\cdot\gamma_Q & \psi_{0s}\cdot\gamma_Q & \gamma_Q & 0 & \psi_{0s}\cdot\gamma_Q & \psi_{0s}\cdot\gamma_Q \\ 0 & 0 & 0 & \gamma_Q & \psi_{0w}\cdot\gamma_Q & \gamma_Q \end{array}\right]\end{array} \begin{array}{l} \text{Load case} \\ \text{1 permanent loads} \\ \text{2 distributed loads} \\ \text{3 crane load} \\ \text{4 snow loads} \\ \text{5 wind loads} \end{array}$$

Resulting load factors in the ultimate limit state

$$\psi_u = \begin{bmatrix} 1.215 & 1.215 & 1.215 & 1 & 1.215 & 1.215 \\ 1.5 & 1.05 & 1.05 & 0 & 1.5 & 1.05 \\ 1.05 & 1.5 & 1.05 & 0 & 1.05 & 1.05 \\ 0.9 & 0.9 & 1.5 & 0 & 0.9 & 0.9 \\ 0 & 0 & 0 & 1.5 & 0.9 & 1.5 \end{bmatrix}$$

4.2 Serviceability limit state

[5], 9.5.2 and 9.5.5

Partial load factors for *frequent load combinations* in the *serviceability limit state*

LC 1: imposed load dominant
LC 2: crane load dominant
LC 3: snow load dominant
LC 4: wind load dominant
LC 5: wind load only (for comparison)
LC 6: simplified, [5] (9.20) (for comparison)

[5] Table 9.3

ψ factor for imposed loads	$\psi_{1i} := 0.5$	$\psi_{2i} := 0.3$	(= 0 for roof)
ψ factor for crane loads	$\psi_{1c} := 0.5$	$\psi_{2c} := 0.3$	
ψ factor for snow loads	$\psi_{1s} := 0.2$	$\psi_{2s} := 0$	
ψ factor for wind loads	$\psi_{1w} := 0.5$	$\psi_{2w} := 0$	

Load combination

$$\psi_s := \begin{array}{c} \begin{array}{cccccc} 1 & 2 & 3 & 4 & 5 & 6 \end{array} \\ \begin{bmatrix} 1 & 1 & 1 & 1 & 0 & 1 \\ \psi_{1i} & 0 & 0 & 0 & 0 & 0.9 \\ \psi_{2c} & \psi_{1c} & \psi_{2c} & 0 & 0 & 0.9 \\ \psi_{2s} & \psi_{2s} & \psi_{1s} & \psi_{2s} & 0 & 0.9 \\ \psi_{2w} & \psi_{2w} & \psi_{2w} & \psi_{1w} & 1 & 0 \end{bmatrix} \end{array} \begin{array}{l} \text{Load case} \\ \text{permanent loads} \\ \text{imposed distributed loads} \\ \text{imposed crane load} \\ \text{snow loads} \\ \text{wind loads} \end{array}$$

Resulting partial load factors in the *serviceability limit state*

Load combination

$$\psi_s = \begin{array}{c} \begin{array}{cccccc} 1 & 2 & 3 & 4 & 5 & 6 \end{array} \\ \begin{bmatrix} 1 & 1 & 1 & 1 & 0 & 1 \\ 0.5 & 0 & 0 & 0 & 0 & 0.9 \\ 0.3 & 0.5 & 0.3 & 0 & 0 & 0.9 \\ 0 & 0 & 0.2 & 0 & 0 & 0.9 \\ 0 & 0 & 0 & 0.5 & 1 & 0 \end{bmatrix} \end{array}$$

For the floor, the load combination 1 is valid
For roofs, the load combinations 2, 3 and 4 are valid

Comment: Load combinations 5 and 6 for comparison only

5. Loads Effects

5.1. Loads per unit length and concentrated loads

Distance between all frames $c_{frame} := 5000 \cdot mm$

Because of continuous purlins and secondary floor beams the load on a beam in a frame is mo than the distance between the beam times the load per area. Therefore, for the second frame, t load is increased with a factor of k_f where

$k_f := 1.1$

5.1.1 Permanent loads

Permanent load on floor $q_{p.floor} := k_f \cdot c_{frame} \cdot q'_{p.floor}$ $q_{p.floor} = 3.85 \circ kN \cdot m^{-1}$

Permanent load on roof $q_{p.roof} := k_f \cdot c_{frame} \cdot q'_{p.roof}$ $q_{p.roof} = 2.75 \circ kN \cdot m^{-1}$

5.1.2 Imposed loads, uniform distributed

Distributed load on floor $q_{k.floor} := k_f \cdot c_{frame} \cdot q'_{k.floor}$ $q_{k.floor} = 16.5 \circ kN \cdot m^{-1}$

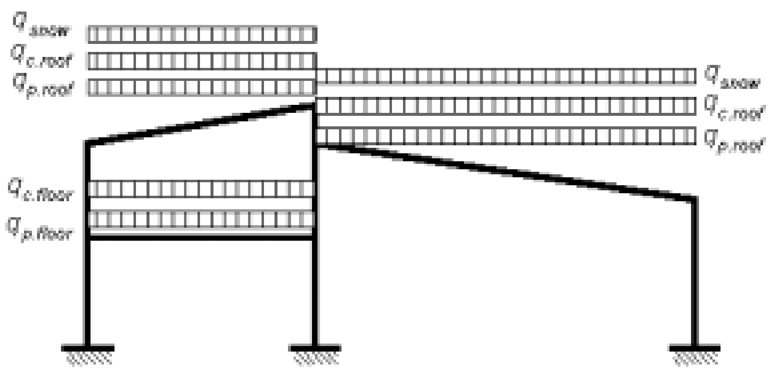
Distributed load on roof $q_{k.roof} := k_f \cdot c_{frame} \cdot q'_{k.roof}$ $q_{k.roof} = 4.125 \circ kN \cdot m^{-1}$

5.1.3 Imposed loads, concentrated

Concentrated load on floor $Q_{k.floor} := 2 \cdot kN$

Concentrated load on roof $Q_{k.roof} := 1.5 \cdot kN$

Concentrated load from crane $P_{crane} := 50 \cdot kN$

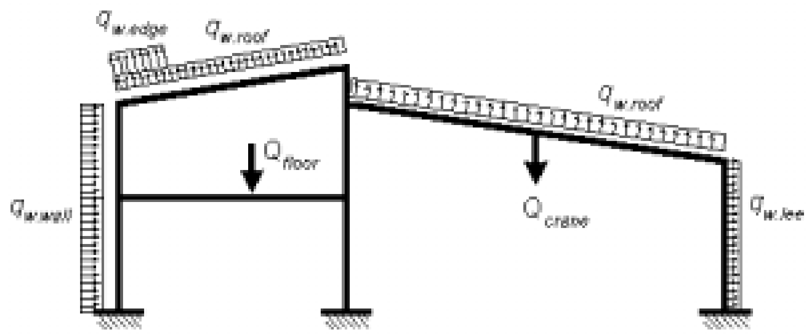


5.1.4 Snow loads

Snow load on roof $q_{snow} := k_f \cdot c_{frame} \cdot q'_{snow}$ $q_{snow} = 11 \circ kN \cdot m^{-1}$

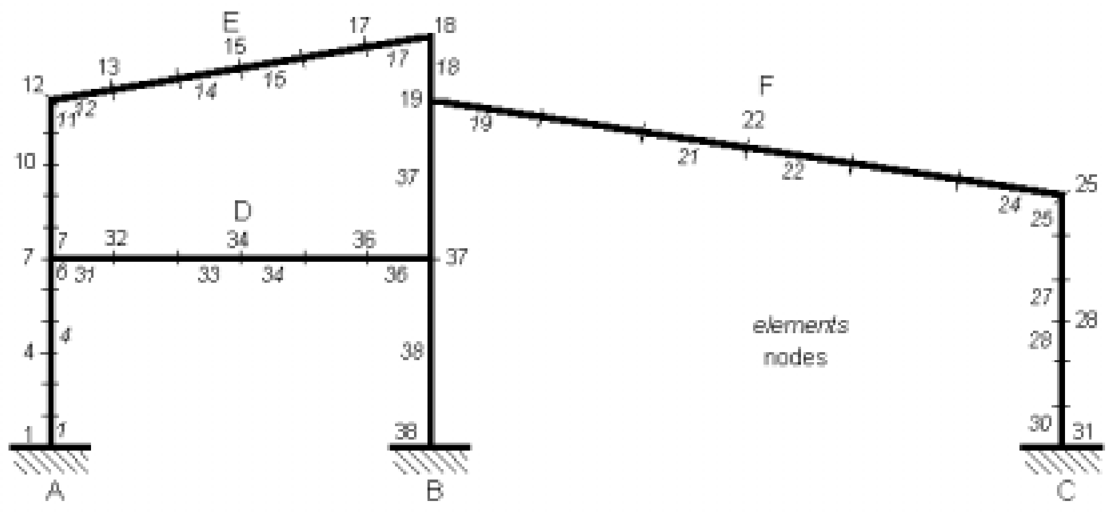
5.1.5 Wind loads

Maximum wind load on the external walls	$q_{w.wall} := k_f \cdot c_{frame} \cdot q'_{w.wall}$	$q_{w.wall} = 3.85 \circ kN \cdot m^{-1}$
Wind suction on leeward side of walls	$q_{w.lee} := k_f \cdot c_{frame} \cdot q'_{w.lee}$	$q_{w.lee} = -1.485 \circ kN \cdot m^{-1}$
Wind suction on the roof	$q_{w.roof} := k_f \cdot c_{frame} \cdot q'_{w.roof}$	$q_{w.roof} = 3.85 \circ kN \cdot m^{-1}$
Max. wind suction at the lower edge of the roof and 1.6 m upwards	$q_{w.edge} := k_f \cdot c_{frame} \cdot q'_{w.edge}$	$q_{w.edge} = 5.5 \circ kN \cdot m^{-1}$



5.2 Finite element calculations

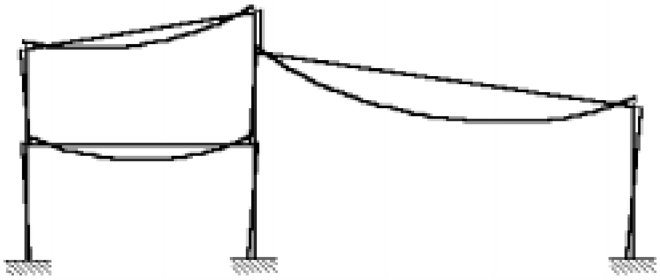
Nodes and elements



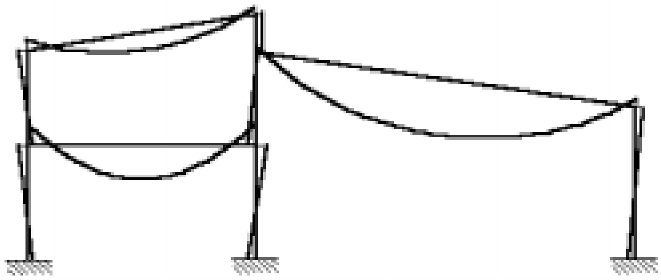
Moment diagrams

5.2.1 Permanent loads

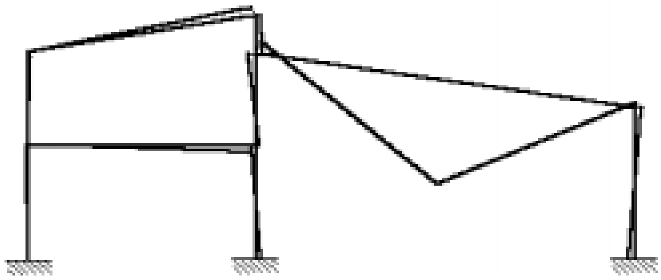
Values of moments and shear forces for separate columns and beams are given in 5.3



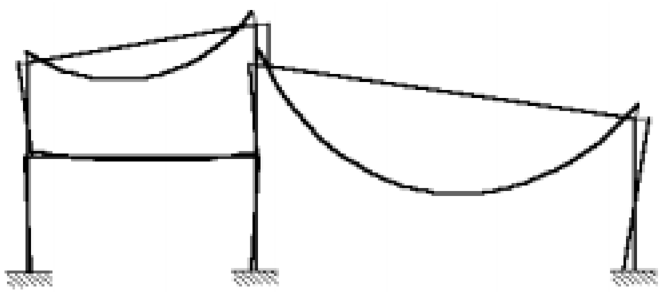
5.2.2 Imposed loads, uniformly distributed



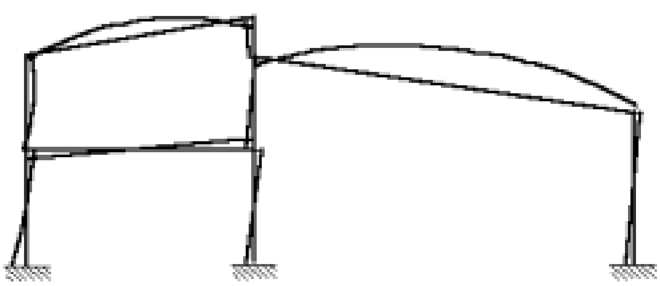
5.2.3 Imposed loads, concentrated



5.2.4 Snow loads

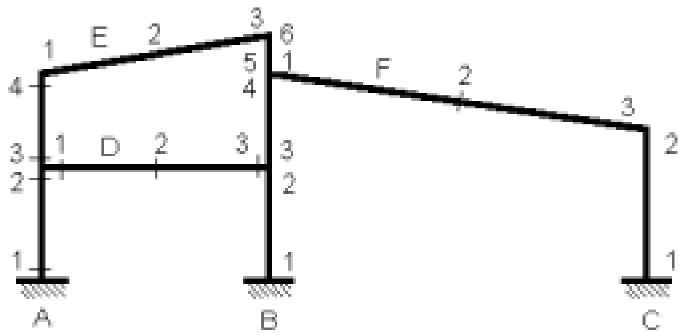


5.2.5 Wind loads



5.3. Section forces for characteristic loads

5.3.1 Column A



(FE-calculation)

Bending moments, section 1, 2, 3 and 4

Sections in columns, load cases in rows

row 1, permanent loads
row 2, distributed loads
row 3, crane load
row 4, snow loads
row 5, wind loads

$$M_A := \begin{bmatrix} 2.90 & -4.40 & 2.14 & -1.88 \\ 9.43 & -15.6 & 12.1 & -5.96 \\ 3.65 & -4.60 & -5.22 & 5.81 \\ 4.28 & -4.63 & -4.22 & -3.04 \\ -12.8 & 6.45 & -2.52 & 1.95 \end{bmatrix} \cdot kNm$$

Axial force, part 1-2 and 3-4

Parts in columns, load cases in rows

$$N_A := \begin{bmatrix} -18.8 & -7.32 \\ -60.5 & -12.0 \\ 1.42 & 2.84 \\ -28.5 & -27.8 \\ 15.6 & 12.0 \end{bmatrix} \cdot kN$$

Deflection in section 4 $\delta_A := \begin{bmatrix} 0.56 \\ -0.50 \\ 2.48 \\ 4.19 \\ 8.36 \end{bmatrix} \cdot mm$

5.3.2 Column B

(FE-calculation)

Bending moments, section 1, 2, 3, 4,5 and 6

Sections in columns, load cases in rows

row 1, permanent loads
row 2, distributed loads
row 3, crane load
row 4, snow loads
row 5, wind loads

$$M_B := \begin{bmatrix} -0.07 & -2.73 & 4.51 & 4.35 & -7.46 & -9.07 \\ 4.26 & -15.4 & 18.2 & -4.69 & -8.29 & -15.5 \\ -3.27 & 0.96 & -2.15 & 19.27 & -15.7 & -11.2 \\ -6.57 & 5.24 & 1.59 & 20.0 & -34.6 & -33.5 \\ 17.0 & -13.5 & 0.49 & -10.95 & 11.4 & 12.7 \end{bmatrix} \cdot kNm$$

Sections in columns, load cases in rows

Axial force, parts 1-2, 3-4 and 5-6

$$N_B := \begin{bmatrix} -35.1 & -23.5 & -9.18 \\ -84.4 & -33.9 & -12.8 \\ -31.7 & -31.1 & -4.34 \\ -95.0 & -95.7 & -38.2 \\ 31.2 & 34.8 & 13.8 \end{bmatrix} kN$$

Deflection in section 6 $\delta_B := \begin{bmatrix} 0.57 \\ -0.53 \\ 2.57 \\ 4.30 \\ 8.32 \end{bmatrix} \cdot mm$

5.3.3 Column C

Comment: To reduce the extent of this example, this column is left out. It can be given, conservatively the same section as column B

5.3.4 Floor beam D

(FE-calculation)

Bending moments, section 1, 2 and 3

Sections in columns, load cases in rows

row 1, permanent loads
row 2, distributed loads
row 3, crane load
row 4, snow loads
row 5, wind loads

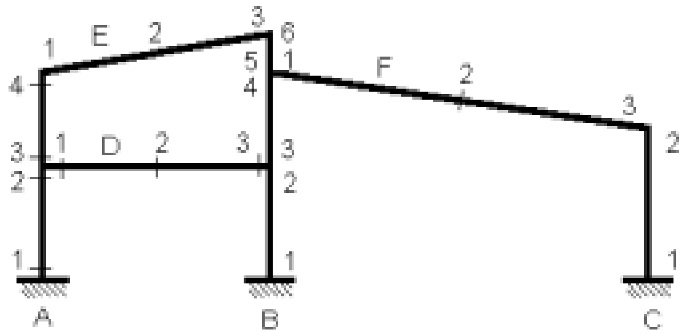
$$M_D := \begin{bmatrix} -6.56 & 10.4 & -7.25 \\ -27.8 & 43.6 & -33.6 \\ 0.62 & 4.86 & 3.11 \\ -0.40 & 1.62 & 3.65 \\ 8.97 & 2.00 & -13.0 \end{bmatrix} \cdot kNm$$

Sections in columns, load cases in rows

Shear force, section 1 and 3

$$V_D := \begin{bmatrix} 11.7 & 11.4 \\ 48.5 & 50.5 \\ 1.4 & -0.6 \\ 0.67 & -0.67 \\ -3.66 & 3.66 \end{bmatrix} \cdot kN$$

Deflection in section 2 $\delta_D := \begin{bmatrix} 3.21 \\ 13.1 \\ 1.63 \\ 0.92 \\ -0.91 \end{bmatrix} \cdot mm$



5.3.5 Roof beam E

Comment: To reduce the extent of this example, this column is left out

5.3.6 Roof beam F

(FE-calculation)

Bending moments, section 1, 2 and 3

Sections in columns, load cases in rows

row 1, permanent loads
row 2, distributed loads
row 3, crane load
row 4, snow loads
row 5, wind loads

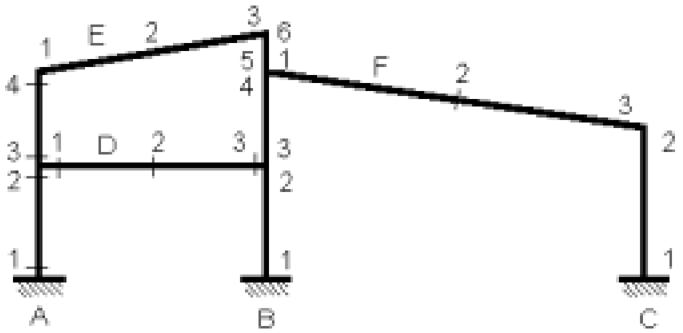
$$M_F := \begin{bmatrix} -11.8 & 26.4 & -4.08 \\ -13.0 & 42.4 & -5.28 \\ -35.0 & 102.0 & -10.94 \\ -54.1 & 101.7 & -17.5 \\ 22.4 & -38.0 & 0.26 \end{bmatrix} kNm$$

Shear force, section 1 and 3

Sections in columns, load cases in rows

$$V_F := \begin{bmatrix} 14.3 & 13.2 \\ 21.1 & 20.1 \\ 43.4 & 23.3 \\ 57.6 & 52.5 \\ -21.1 & -17.4 \end{bmatrix} \cdot kN$$

Deflection in section 2 $$\delta_F := \begin{bmatrix} 7.71 \\ 12.6 \\ 22.2 \\ 29.3 \\ -11.1 \end{bmatrix} \cdot mm$$



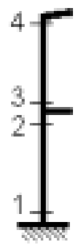
5.4. Design moments, shear forces and deflections

5.4.1 Column A

Bending moments

For the load cases 1 - 5 the bending moment in section 1 of column B is:

(5.3.1) $$M_A^{<1>} = \begin{bmatrix} 2.9 \\ 9.43 \\ 3.65 \\ 4.28 \\ -12.8 \end{bmatrix} \circ kNm$$



Column A

The load factor matrix is

(4.1) $$\psi_{\bar{u}} \begin{bmatrix} 1.215 & 1.215 & 1.215 & 1 & 1.215 & 1.215 \\ 1.5 & 1.05 & 1.05 & 0 & 1.5 & 1.05 \\ 1.05 & 1.5 & 1.05 & 0 & 1.05 & 1.05 \\ 0.9 & 0.9 & 1.5 & 0 & 0.9 & 0.9 \\ 0 & 0 & 0 & 1.5 & 0.9 & 1.5 \end{bmatrix}$$

The values in the moment vector shall be multiplied with the corresponding load factor for every lo
combination

$i := 1 .. cols(\psi_u)$ ($cols(\psi_u)$ is the number of columns in the matrix ψ_u)

$M^{<i>} := \overrightarrow{\left(M_A{}^{<1>} \cdot \psi_u{}^{<i>}\right)}$

$$M = \begin{bmatrix} 3.524 & 3.524 & 3.524 & 2.9 & 3.524 & 3.524 \\ 14.145 & 9.901 & 9.901 & 0 & 14.145 & 9.901 \\ 3.832 & 5.475 & 3.832 & 0 & 3.832 & 3.832 \\ 3.852 & 3.852 & 6.42 & 0 & 3.852 & 3.852 \\ 0 & 0 & 0 & -19.2 & -11.52 & -19.2 \end{bmatrix} \cdot kNm$$

The moments in the columns of the matrix M (= load combination) are added

$M_{sum_i} := \sum M^{<i>}$

$M_{sum}{}^T = (25.353 \;\; 22.752 \;\; 23.677 \;\; -16.3 \;\; 13.833 \;\; 1.909) \cdot kNm$

Maximum and minimum of moment are

$M_{Amax_1} := max(M_{sum})$ $M_{Amax_1} = 25.353 \cdot kNm$

$M_{Amin_1} := min(M_{sum})$ $M_{Amin_1} = -16.3 \cdot kNm$

(5.3.1) Moments in section 2 $s := 2$ $M^{<i>} := \overrightarrow{\left(M_A{}^{<s>} \cdot \psi_u{}^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}{}^T = (-37.743 \;\; -32.793 \;\; -33.501 \;\; 5.275 \;\; -31.938 \;\; -21.048) \cdot kNm$ $M_{Amax_s} := max(M_{sum})$

$M_{Amin_s} := min(M_{sum})$

(5.3.1) Moments in section 3 $s := 3$ $M^{<i>} := \overrightarrow{\left(M_A{}^{<s>} \cdot \psi_u{}^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}{}^T = (11.471 \;\; 3.677 \;\; 3.494 \;\; -1.64 \;\; 9.203 \;\; 2.246) \cdot kNm$ $M_{Amax_s} := max(M_{sum})$

$M_{Amin_s} := min(M_{sum})$

(5.3.1) Moments in section 4 $s := 4$ $M^{<i>} := \overrightarrow{\left(M_A{}^{<s>} \cdot \psi_u{}^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}{}^T = (-7.86 \;\; -2.563 \;\; -7.002 \;\; 1.045 \;\; -6.105 \;\; -2.253) \cdot kNm$ $M_{Amax_s} := max(M_{sum})$

$M_{Amin_s} := min(M_{sum})$

Resulting maximum moments and minimum moments in section 1 to 4 are

$M_{Amax}^T = (25.353 \;\; 5.275 \;\; 11.471 \;\; 1.045) \cdot kNm$

$M_{Amin}^T = (-16.3 \;\; -37.743 \;\; -1.64 \;\; -7.86) \cdot kNm$

Axial force

Axial force in part 1-2 $\quad s := 1$

(5.3.1) $\quad N_A^{\langle s \rangle} = \begin{bmatrix} -18.8 \\ -60.5 \\ 1.42 \\ -28.5 \\ 15.6 \end{bmatrix} \cdot kN \qquad N^{\langle i \rangle} := \overrightarrow{\left(N_A^{\langle s \rangle} \cdot \psi_u^{\langle i \rangle}\right)} \qquad N_{sum_i} := \sum N^{\langle i \rangle}$

$N_{Amax_s} := max\left(N_{sum}\right)$

$N_{sum}^T = (-137.751 \;\; -109.887 \;\; -127.626 \;\; 4.6 \;\; -123.711 \;\; -87.126) \cdot kN \qquad N_{Amin_s} := min\left(N_{sum}\right)$

(5.3.1) Axial force in part 3-4 $\quad s := 2 \qquad N^{\langle i \rangle} := \overrightarrow{\left(N_A^{\langle s \rangle} \cdot \psi_u^{\langle i \rangle}\right)} \qquad N_{sum_i} := \sum N^{\langle i \rangle}$

$N_{sum}^T = (-48.932 \;\; -42.254 \;\; -60.212 \;\; 10.68 \;\; -38.132 \;\; -25.532) \cdot kN \qquad N_{Amax_s} := max\left(N_{sum}\right)$

$N_{Amin_s} := min\left(N_{sum}\right)$

Resulting maximum and minimum axial forces in part 1, 2 and 3 are:

$N_{Amax}^T = (4.6 \;\; 10.68) \cdot kN \qquad N_{Amin}^T = (-137.8 \;\; -60.2) \cdot kN$

Deflection

Deflection in section 6 $\quad s := 1$

(5.3.1) $\quad \delta_A^{\langle s \rangle} = \begin{bmatrix} 0.56 \\ -0.5 \\ 2.48 \\ 4.19 \\ 8.36 \end{bmatrix} \cdot mm \qquad \delta^{\langle i \rangle} := \overrightarrow{\left(\delta_A^{\langle s \rangle} \cdot \psi_s^{\langle i \rangle}\right)} \qquad \delta_{sum_i} := \sum \delta^{\langle i \rangle}$

$\delta_{Amax_s} := max\left(\delta_{sum}\right) \qquad \delta_{Amin_s} := min\left(\delta_{sum}\right)$

$\delta_{sum}^T = (1.054 \;\; 1.8 \;\; 2.142 \;\; 4.74 \;\; 8.36 \;\; 6.113) \cdot mm$

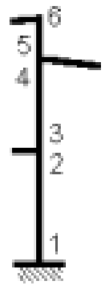
Resulting maximum and minimum deflection in section 6 $\qquad \delta_{Amax} = (8.36) \cdot mm$

$\delta_{Amin} = (1.054) \cdot mm$

5.4.2 Column B

Bending moments

For the load cases 1 - 5 the bending moment in section 1 of column B is:



Column B

(5.3.2) $$M_B^{<1>} = \begin{bmatrix} -0.07 \\ 4.26 \\ -3.27 \\ -6.57 \\ 17 \end{bmatrix} \cdot kNm$$

The load factor matrix is

(4.1) $$\psi_u := \begin{bmatrix} 1.215 & 1.215 & 1.215 & 1 & 1.215 & 1.215 \\ 1.5 & 1.05 & 1.05 & 0 & 1.5 & 1.05 \\ 1.05 & 1.5 & 1.05 & 0 & 1.05 & 1.05 \\ 0.9 & 0.9 & 1.5 & 0 & 0.9 & 0.9 \\ 0 & 0 & 0 & 1.5 & 0.9 & 1.5 \end{bmatrix}$$

The values in the moment vector shall be multiplied with the corresponding load factor for every l combination

$i := 1 .. cols(\psi_u)$ ($cols(\psi_u)$ is the number of columns in the matrix ψ_u)

$$M^{<i>} := \overrightarrow{\left(M_B^{<1>} \cdot \psi_u^{<i>}\right)}$$

$$M = \begin{bmatrix} -0.085 & -0.085 & -0.085 & -0.07 & -0.085 & -0.085 \\ 6.39 & 4.473 & 4.473 & 0 & 6.39 & 4.473 \\ -3.433 & -4.905 & -3.433 & 0 & -3.433 & -3.433 \\ -5.913 & -5.913 & -9.855 & 0 & -5.913 & -5.913 \\ 0 & 0 & 0 & 25.5 & 15.3 & 25.5 \end{bmatrix} \cdot kNm$$

The moments in the columns of the matrix M (= load combination) are added

$$M_{sum_i} := \sum M^{<i>}$$

$$M_{sum}^T = (-3.042 \ \ -6.43 \ \ -8.901 \ \ 25.43 \ \ 12.258 \ \ 20.541) \cdot kNm$$

Maximum and minimum of moment are

$M_{Bmax_1} := max(M_{sum})$ $M_{Bmax_1} = 25.43 \cdot kNm$

$M_{Bmin_1} := min(M_{sum})$ $M_{Bmin_1} = -8.901 \cdot kNm$

(5.3.2) Moments in section 2 $s := 2$ $M^{<i>} := \overrightarrow{\left(M_B^{<s>} \cdot \psi_u^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}^T = (-20.693\ \ -13.331\ \ -10.619\ \ -22.98\ \ -32.843\ \ -34.013) \circ kNm$ $M_{Bmax_s} := max(M_{sum})$

$M_{Bmin_s} := min(M_{sum})$

(5.3.2) Moments in section 3 $s := 3$ $M^{<i>} := \overrightarrow{\left(M_B^{<s>} \cdot \psi_u^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}^T = (31.953\ \ 22.796\ \ 24.717\ \ 5.245\ \ 32.394\ \ 24.498) \circ kNm$ $M_{Bmax_s} := max(M_{sum})$

$M_{Bmin_s} := min(M_{sum})$

(5.3.2) Moments in section 4 $s := 4$ $M^{<i>} := \overrightarrow{\left(M_B^{<s>} \cdot \psi_u^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}^T = (36.484\ \ 47.266\ \ 50.594\ \ -12.075\ \ 26.629\ \ 22.169) \circ kNm$ $M_{Bmax_s} := max(M_{sum})$

$M_{Bmin_s} := min(M_{sum})$

(5.3.2) Moments in section 5 $s := 5$ $M^{<i>} := \overrightarrow{\left(M_B^{<s>} \cdot \psi_u^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}^T = (-69.124\ \ -72.458\ \ -86.153\ \ 9.64\ \ -58.864\ \ -48.293) \circ kNm$ $M_{Bmax_s} := max(M_{sum})$

$M_{Bmin_s} := min(M_{sum})$

(5.3.2) Moments in section 6 $s := 6$ $M^{<i>} := \overrightarrow{\left(M_B^{<s>} \cdot \psi_u^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}^T = (-76.18\ \ -74.245\ \ -89.305\ \ 9.98\ \ -64.75\ \ -50.155) \circ kNm$ $M_{Bmax_s} := max(M_{sum})$

$M_{Bmin_s} := min(M_{sum})$

Resulting maximum moments and minimum moments in section 1 to 6 are

$M_{Bmax}^T = (25.43\ \ -10.619\ \ 32.394\ \ 50.594\ \ 9.64\ \ 9.98) \circ kNm$

$M_{Bmin}^T = (-8.901\ \ -34.013\ \ 5.245\ \ -12.075\ \ -86.153\ \ -89.305) \circ kNm$

Axial force

Axial force in part 1-2 $s := 1$

(5.3.2) $N_B^{<s>} = \begin{bmatrix} -35.1 \\ -84.4 \\ -31.7 \\ -95 \\ 31.2 \end{bmatrix} \circ kN$ $N^{<i>} := \overrightarrow{\left(N_B^{<s>} \cdot \psi_u^{<i>}\right)}$ $N_{sum_i} := \sum N^{<i>}$

$N_{Bmax_s} := max(N_{sum})$

$N_{sum}^T = (-288.031\ \ -264.317\ \ -307.051\ \ 11.7\ \ -259.951\ \ -203.251) \circ kN$ $N_{Bmin_s} := min(N_{sum})$

(5.3.2) Axial force in part 3-4 $s := 2$ $N^{<i>} := \overrightarrow{\left(N_B{}^{<s>} \cdot \psi_u{}^{<i>}\right)}$ $N_{sum_i} := \sum N^{<i>}$

$N_{sum}{}^T = (-198.187\ \ -196.927\ \ -240.352\ \ 28.7\ \ -166.868\ \ -130.732) \circ kN$ $N_{Bmax_s} := max\left(N_{sum}\right)$

$N_{Bmin_s} := min\left(N_{sum}\right)$

(5.3.2) Axial force in part 5-6 $s := 3$ $N^{<i>} := \overrightarrow{\left(N_B{}^{<s>} \cdot \psi_u{}^{<i>}\right)}$ $N_{sum_i} := \sum N^{<i>}$

$N_{sum}{}^T = (-69.291\ \ -65.484\ \ -86.451\ \ 11.52\ \ -56.871\ \ -42.831) \circ kN$ $N_{Bmin_s} := min\left(N_{sum}\right)$

$N_{Bmax_s} := max\left(N_{sum}\right)$

Resulting maximum and minimum axial forces in part 1, 2 and 3 are:

$N_{Bmax}{}^T = (11.7\ \ 28.7\ \ 11.52) \circ kN$ $N_{Bmin}{}^T = (-307.1\ \ -240.4\ \ -86.5) \circ kN$

Deflection

Deflection in section 6 $s := 1$

(5.3.2) $\delta_B{}^{<s>} = \begin{bmatrix} 0.57 \\ -0.53 \\ 2.57 \\ 4.3 \\ 8.32 \end{bmatrix} \circ mm$ $\delta^{<i>} := \overrightarrow{\left(\delta_B{}^{<s>} \cdot \psi_s{}^{<i>}\right)}$ $\delta_{sum_i} := \sum \delta^{<i>}$

$\delta_{Bmax_s} := max\left(\delta_{sum}\right)$ $\delta_{Bmin_s} := min\left(\delta_{sum}\right)$

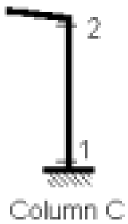
$\delta_{sum}{}^T = (1.076\ \ 1.855\ \ 2.201\ \ 4.73\ \ 8.32\ \ 6.276) \circ mm$

Resulting maximum and minimum deflection in section 6 $\delta_{Bmax} = (8.32) \circ mm$

$\delta_{Bmin} = (1.076) \circ mm$

5.4.3 Column C

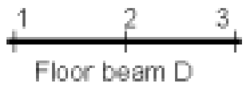
Comment: To reduce the extent of this example, calculation of this column is left out. It can, conservatively, be given the same dimensions as column B



5.4.4 Floor beam D

Bending moments

For the load cases 1 - 5 the bending moment in section 1 of beam D is:



(5.3.4) $$M_D^{<1>} = \begin{bmatrix} -6.56 \\ -27.8 \\ 0.62 \\ -0.4 \\ 8.97 \end{bmatrix} \circ kNm$$

The load factor matrix is

(4.1) $$\psi_u = \begin{bmatrix} 1.215 & 1.215 & 1.215 & 1 & 1.215 & 1.215 \\ 1.5 & 1.05 & 1.05 & 0 & 1.5 & 1.05 \\ 1.05 & 1.5 & 1.05 & 0 & 1.05 & 1.05 \\ 0.9 & 0.9 & 1.5 & 0 & 0.9 & 0.9 \\ 0 & 0 & 0 & 1.5 & 0.9 & 1.5 \end{bmatrix}$$

The values in the moment vector shall be multiplied with the corresponding load factor for every l combination

$i := 1 .. cols(\psi_u)$ ($cols(\psi_u)$ is the number of columns in the matrix ψ_u)

$$M^{<i>} := \overrightarrow{\left(M_D^{<1>} \cdot \psi_u^{<i>}\right)}$$

$$M = \begin{bmatrix} -7.97 & -7.97 & -7.97 & -6.56 & -7.97 & -7.97 \\ -41.7 & -29.19 & -29.19 & 0 & -41.7 & -29.19 \\ 0.651 & 0.93 & 0.651 & 0 & 0.651 & 0.651 \\ -0.36 & -0.36 & -0.6 & 0 & -0.36 & -0.36 \\ 0 & 0 & 0 & 13.455 & 8.073 & 13.455 \end{bmatrix} \circ kNm$$

The moments in the columns of the matrix M (= load combination) are added

$$M_{sum_i} := \sum M^{<i>}$$

$$M_{sum}^T = (-49.379 \ \ -36.59 \ \ -37.109 \ \ 6.895 \ \ -41.306 \ \ -23.414) \circ kNm$$

Maximum and minimum of moment are

$M_{Dmax_1} := max(M_{sum})$ $\qquad M_{Dmax_1} = 6.895 \circ kNm$

$M_{Dmin_1} := min(M_{sum})$ $\qquad M_{Dmin_1} = -49.379 \circ kNm$

(5.3.4) Moments in section 2 $s := 2$ $M^{<i>} := \overrightarrow{\left(M_D^{<s>} \cdot \psi_u^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}^T = (84.597 \;\; 67.164 \;\; 65.949 \;\; 13.4 \;\; 86.397 \;\; 67.977)\ \mathrm{m} \circ kN$ $M_{Dmin_s} := min\left(M_{sum}\right)$

$M_{Dmax_s} := max\left(M_{sum}\right)$

(5.3.4) Moments in section 3 $s := 3$ $M^{<i>} := \overrightarrow{\left(M_D^{<s>} \cdot \psi_u^{<i>}\right)}$ $M_{sum_i} := \sum M^{<i>}$

$M_{sum}^T = (-52.658 \;\; -36.139 \;\; -35.348 \;\; -26.75 \;\; -64.358 \;\; -57.038)\ \mathrm{m} \circ kN$ $M_{Dmin_s} := min\left(M_{sum}\right)$

$M_{Dmax_s} := max\left(M_{sum}\right)$

Resulting maximum moments and minimum moments in section 1 to 3 are

$$M_{Dmax} = \begin{bmatrix} 6.895 \\ 86.397 \\ -26.75 \end{bmatrix} \circ kNm \qquad M_{Dmin} = \begin{bmatrix} -49.379 \\ 13.4 \\ -64.358 \end{bmatrix} \circ kNm$$

Shear force

Shear force in section 1 $s := 1$

(5.3.4) $V_D^{<s>} = \begin{bmatrix} 11.7 \\ 48.5 \\ 1.4 \\ 0.67 \\ -3.66 \end{bmatrix} \circ kN$ $V^{<i>} := \overrightarrow{\left(V_D^{<s>} \cdot \psi_u^{<i>}\right)}$ $V_{sum_i} := \sum V^{<i>}$

$V_{Dmax_s} := max\left(V_{sum}\right)$ $V_{Dmin_s} := min\left(V_{sum}\right)$

$V_{sum}^T = (89.038 \;\; 67.844 \;\; 67.615 \;\; 6.21 \;\; 85.745 \;\; 61.723) \circ kN$

Shear force in section 2 $s := 2$

(5.3.4) $V_D^{<s>} = \begin{bmatrix} 11.4 \\ 50.5 \\ -0.6 \\ -0.67 \\ 3.66 \end{bmatrix} \circ kN$ $V^{<i>} := \overrightarrow{\left(V_D^{<s>} \cdot \psi_u^{<i>}\right)}$ $V_{sum_i} := \sum V^{<i>}$

$V_{Dmax_s} := max\left(V_{sum}\right)$ $V_{Dmin_s} := min\left(V_{sum}\right)$

$V_{sum}^T = (88.368 \;\; 65.373 \;\; 65.241 \;\; 16.89 \;\; 91.662 \;\; 71.133) \circ kN$

Resulting maximum and minimum shear forces in section 1 and 3 are

$$V_{Dmax} = \begin{bmatrix} 89.038 \\ 91.662 \end{bmatrix} \circ kN \qquad V_{Dmin} = \begin{bmatrix} 6.21 \\ 16.89 \end{bmatrix} \circ kN$$

Deflection

$$V_{Dmax} = \begin{bmatrix} 89.038 \\ 91.662 \end{bmatrix} \circ kN \qquad V_{Dmin} = \begin{bmatrix} 6.21 \\ 16.89 \end{bmatrix} \circ kN$$

Deflection

Deflection in section 2 $\quad s := 1$

(5.3.4) $\quad \delta_D^{<s>} = \begin{bmatrix} 3.21 \\ 13.1 \\ 1.63 \\ 0.92 \\ -0.91 \end{bmatrix} \circ mm \qquad \delta^{<i>} := \overrightarrow{\left(\delta_D^{<s>} \cdot \psi_s^{<i>}\right)} \qquad \delta_{sum_i} := \sum \delta^{<i>}$

$$\delta_{Dmax_s} := max\left(\delta_{sum}\right) \qquad \delta_{Dmin_s} := min\left(\delta_{sum}\right)$$

$$\delta_{sum}^T = (10.249 \;\; 4.025 \;\; 3.883 \;\; 2.755 \;\; -0.91 \;\; 17.295) \circ mm$$

Resulting maximum and minimum deflection in section 2

$$\delta_{Dmax} = (17.295) \circ mm \qquad \delta_{Dmin} = (-0.91) \circ mm$$

5.4.5 Roof beam E

Comment: To reduce the extent of this example, calculation of this beam is left out. It can be giver the same dimensions as floor beam D

5.4.6 Roof beam F

Moment

For the load cases 1 - 5 the bending moments in section 1 to 3 of the beam F are

(5.3.6) Moments in section 1 $\quad s := 1 \qquad M^{<i>} := \overrightarrow{\left(M_F^{<s>} \cdot \psi_u^{<i>}\right)} \qquad M_{sum_i} := \sum M^{<i>}$

$$M_{sum}^T = (-119.277 \;\; -129.177 \;\; -145.887 \;\; 21.8 \;\; -99.117 \;\; -79.827) \text{ m} \circ kN \qquad M_{Fmin_s} := min\left(M_{sum}\right)$$

$$M_{Fmax_s} := max\left(M_{sum}\right)$$

(5.3.6) Moments in section 2 $\quad s := 2 \qquad M^{<i>} := \overrightarrow{\left(M_F^{<s>} \cdot \psi_u^{<i>}\right)} \qquad M_{sum_i} := \sum M^{<i>}$

$$M_{sum}^T = (294.306 \;\; 321.126 \;\; 336.246 \;\; -30.6 \;\; 260.106 \;\; 218.226) \text{ m} \circ kN \qquad M_{Fmin_s} := min\left(M_{sum}\right)$$

$$M_{Fmax_s} := max\left(M_{sum}\right)$$

(5.3.6) Moments in section 3 $\quad s := 3 \qquad M^{<i>} := \overrightarrow{\left(M_F^{<s>} \cdot \psi_u^{<i>}\right)} \qquad M_{sum_i} := \sum M^{<i>}$

$$M_{sum}^T = (-40.114 \;\; -42.661 \;\; -48.238 \;\; -3.69 \;\; -39.88 \;\; -37.348) \text{ m} \circ kN \qquad M_{Fmin_s} := min\left(M_{sum}\right)$$

$$M_{Fmax_s} := max\left(M_{sum}\right)$$

Resulting maximum moments and minimum moments in section 1 to 3 are

$$M_{Fmax} = \begin{bmatrix} 21.8 \\ 336.246 \\ -3.69 \end{bmatrix} \circ kNm \qquad M_{Fmin} = \begin{bmatrix} -145.887 \\ -30.6 \\ -48.238 \end{bmatrix} \circ kNm$$

Shear force

Shear force in section 1 $\quad s := 1$

(5.3.4) $$V_F^{\langle s\rangle} = \begin{bmatrix} 14.3 \\ 21.1 \\ 43.4 \\ 57.6 \\ -21.1 \end{bmatrix} \circ kN \qquad V^{\langle i\rangle} := \overrightarrow{\left(V_F^{\langle s\rangle} \cdot \psi_u^{\langle i\rangle}\right)} \qquad V_{sum_i} := \sum V^{\langle i\rangle}$$

$$V_{Fmax_s} := max\left(V_{sum}\right) \qquad V_{Fmin_s} := min\left(V_{sum}\right)$$

$$V_{sum}^T = (146.435 \quad 156.47 \quad 171.5 \quad -17.35 \quad 127.444 \quad 105.289) \circ kN$$

Shear force in section 3 $\quad s := 2$

(5.3.4) $$V_F^{\langle s\rangle} = \begin{bmatrix} 13.2 \\ 20.1 \\ 23.3 \\ 52.5 \\ -17.4 \end{bmatrix} \circ kN \qquad V^{\langle i\rangle} := \overrightarrow{\left(V_F^{\langle s\rangle} \cdot \psi_u^{\langle i\rangle}\right)} \qquad V_{sum_i} := \sum V^{\langle i\rangle}$$

$$V_{Fmax_s} := max\left(V_{sum}\right) \qquad V_{Fmin_s} := min\left(V_{sum}\right)$$

$$V_{sum}^T = (117.903 \quad 119.343 \quad 140.358 \quad -12.9 \quad 102.243 \quad 82.758) \circ kN$$

Resulting maximum and minimum shear force in section 1 and 3 are

$$V_{Fmax} = \begin{bmatrix} 171.5 \\ 140.358 \end{bmatrix} \circ kN \qquad V_{Fmin} = \begin{bmatrix} -17.35 \\ -12.9 \end{bmatrix} \circ kN$$

Deflection

Deflection in section 2 $\quad s := 1$

(5.3.4) $$\delta_F^{\langle s\rangle T} = (7.71 \quad 12.6 \quad 22.2 \quad 29.3 \quad -11.1) \circ mm \qquad \delta^{\langle i\rangle} := \overrightarrow{\left(\delta_F^{\langle s\rangle} \cdot \psi_s^{\langle i\rangle}\right)} \qquad \delta_{sum_i} := \sum \delta^{\langle i\rangle}$$

$$\delta_{Fmin_s} := min\left(\delta_{sum}\right)$$

$$\delta_{sum}^T = (20.67 \quad 18.81 \quad 20.23 \quad 2.16 \quad -11.1 \quad 65.4) \circ mm \qquad \delta_{Fmax_s} := max\left(\delta_{sum}\right)$$

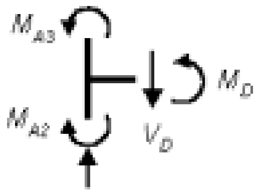
[5] (9.20) Simplified verification $\quad \delta_{Fmax} = (65.4) \circ mm \qquad \delta_{Fmin} = (-11.1) \circ mm$

[5] (9.16) Load combination 3 $\quad \delta_{sum_3} = 20.2 \circ mm$

5.4.7 Joint A-D

(5.4.1) and (5.5.4)

Moment



$M_{Amax_3} = 11.5 \cdot kNm$

$M_{Dmin_1} = -49.4 \cdot kNm$

$M_{Amin_2} = -37.7 \cdot kNm$

Shear

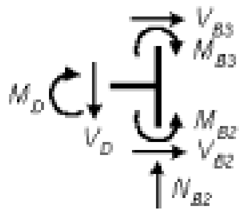
$V_{Dmax_1} = 89 \cdot kN$

Check: $M_{Amax_3} + M_{Dmin_1} + \left(-M_{Amin}\right)_2 = -0.17 \cdot kNm$

5.4.8 Joint B-D

(5.4.2) and (5.5.4)

Moment



$M_{Bmax_3} = 32.4 \cdot kNm$ $\quad M_{Bmin_2} = -34 \cdot kNm$

$M_{Dmin_3} = -64.4 \cdot kNm$

Check: $M_{Bmax_3} + M_{Dmin_3} + \left(-M_{Bmin}\right)_2 = 2.05 \cdot kNm$

Shear

$V_{Dmax_2} = 91.7 \cdot kN$

$V_{B3} = 2.306 \cdot kN$ $\quad V_{B2} = -18.184 \cdot kN$

Axial

$N_{B3} = -240.4 \cdot kN$ $\quad N_{B2} = -307.1 \cdot kN$

5.4.9 Joint A-E and joint B-E

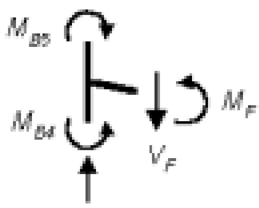
5.4.10

Comment: To reduce the extent of this example, calculation of this joint is left out

5.4.11 Joint B-F

(5.4.2) and (5.5.6)

Moment



$M_{Bmin_5} = -86.153 \cdot kNm$

$M_{Fmin_1} = -145.9 \cdot kNm$

$M_{Bmax_4} = 50.6 \cdot kNm$

Shear

$V_{Fmax_1} = 171.5 \cdot kN$

Check: $M_{Bmin_5} - M_{Fmin_1} - M_{Bmax_4} = 9.14 \cdot kNm$

(The reason why the sum of the moments is not = 0 is the fact that all the moments does not belong to the same load combination)

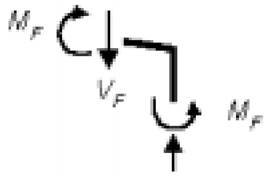
5.4.12 Joint F-C

(5.4.6) Moment $M_{Fmin_3} = -48.238 \circ kNm$

$M_{Fmax_3} = -3.7 \circ kNm$

Shear $V_{Fmin_2} = -12.9 \circ kN$

$V_{Fmax_2} = 140.4 \circ kN$

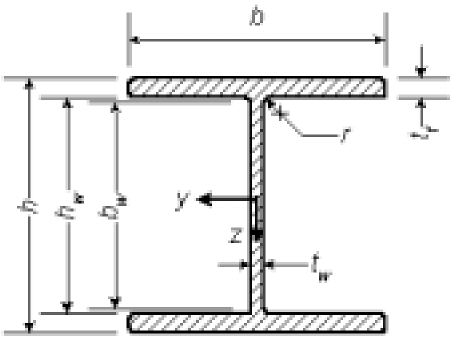


5.4.13 Column bases See 6.1.13 and 6.2.13

6. Code Checking

6.1 Column A

6.1.1 Dimensions and material properties



Section height: $h := 160 \cdot mm$

Flange depth: $b := 150 \cdot mm$

Web thickness: $t_w := 5 \cdot mm$

Flange thickness: $t_f := 14 \cdot mm$

Overall length: $L_1 := 3 \cdot m$

Distance between purlins: $c_p := 1 \cdot m$

[1] Table 3.2b Alloy: **EN AW-6082 T6** EP/O t > 5 mm

$f_{0.2} := 260 \cdot \dfrac{newton}{mm^2}$ $f_u := 310 \cdot \dfrac{newton}{mm^2}$

$heat\_treated := 1$ (if heat-treated then 1 else 0)

[1] (5.4), (5.5) $f_o := f_{0.2}$ $f_a := f_u$

[1] (5.6) $f_v := \dfrac{f_o}{\sqrt{3}}$ $f_v = 150 \circ \dfrac{newton}{mm^2}$ $E := 70000 \cdot \dfrac{newton}{mm^2}$ $G := 27000 \cdot \dfrac{newton}{mm^2}$

Partial safety factors: $\gamma_{M1} \equiv 1.10$ $\gamma_{M2} \equiv 1.25$

Inner radius: $r := 5 \cdot mm$

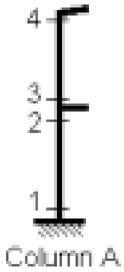
Web height: $b_w := h - 2 \cdot t_f - 2 \cdot r$ $b_w = 122 \circ mm$

S.I. units: $kN \equiv 1000 \cdot newton$ $kNm \equiv kN \cdot m$ $MPa \equiv 1000000 \cdot Pa$

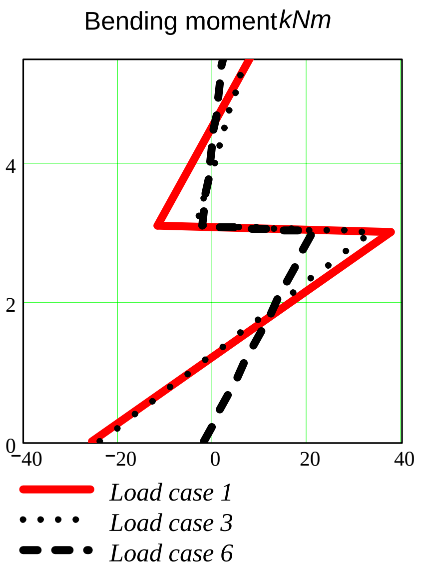
6.1.2 Internal moments and forces

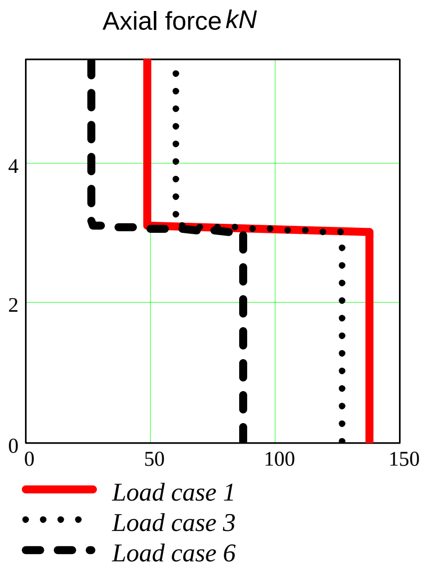
(5.4.2) Bending moments and axial forces for load case LC1, LC3 and LC6 in section 1, 2, 3 and 4

$$x := \begin{bmatrix} 0 \\ 3.0 \\ 3.1 \\ 5.5 \end{bmatrix} \qquad M_{LC1} := \begin{bmatrix} 25.4 \\ -37.7 \\ 11.5 \\ -7.86 \end{bmatrix} \cdot kNm \qquad M_{LC3} := \begin{bmatrix} 23.8 \\ -33.5 \\ 3.49 \\ -7.00 \end{bmatrix} \cdot kNm \qquad M_{LC6} := \begin{bmatrix} 1.91 \\ -21.1 \\ 2.24 \\ -2.25 \end{bmatrix} \cdot kNm$$



$$N_{LC1} := \begin{bmatrix} 138 \\ 138 \\ 49 \\ 49 \end{bmatrix} \cdot kN \qquad N_{LC3} := \begin{bmatrix} 127 \\ 127 \\ 60 \\ 60 \end{bmatrix} \cdot kN \qquad N_{LC6} := \begin{bmatrix} 87 \\ 87 \\ 26 \\ 26 \end{bmatrix} \cdot kN$$





	Load case 1	*Load case 3*	*Load case 6*
Moment in section 2	$-M_{LC1_2} = 37.7 \cdot kNm$	$-M_{LC3_2} = 33.5 \cdot kNm$	$-M_{LC6_2} = 21.1 \cdot kNm$
Moment at column base 1	$-M_{LC1_1} = -25.4 \cdot kNm$	$-M_{LC3_1} = -23.8 \cdot kNm$	$-M_{LC6_1} = -1.91 \cdot kNm$
Axial force in part 1-2	$N_{LC1_1} = 138 \cdot kN$	$N_{LC3_1} = 127 \cdot kN$	$N_{LC6_1} = 87 \cdot kN$

Preliminary calculations show that load case 1 is governing. Study part 1-2 from column base to floor beam. Moment in top of part 1-2 (section 2) is larger than at column base (section 1) why $M_{1.Ed}$ (below) correspond to section 2 and $M_{2.Ed}$ to section 1 of the column.

Load case 1

Bending moment in section 2 $M_{1.Ed} := -M_{LC1_2}$ $M_{1.Ed} = 37.7 \cdot kNm$

Bending moment at column base (1) $M_{2.Ed} := -M_{LC1_1}$ $M_{2.Ed} = -25.4 \cdot kNm$

Axial force in part 1-2 (compression) $N_{Ed} := N_{LC1_1}$ $N_{Ed} = 138 \cdot kN$

6.1.3 Classification of the cross section in y-y-axis bending

$\beta_w :=$ *bending*

a) Web

[1] 5.4.3 $b_1 := b_w$ $t_1 := t_w$ $\beta_w := 0.40 \cdot \dfrac{b_1}{t_1}$ $\beta_w = 9.76$

[1] Tab. 5.1 $\varepsilon := \sqrt{\dfrac{250}{f_o} \cdot \dfrac{newton}{mm^2}}$ $\beta_{1w} := 11 \cdot \varepsilon$ $\beta_{1w} = 10.786$

Heat treated, unwelded = no longitudinal weld

$\beta_{2w} := 16 \cdot \varepsilon$ $\beta_{2w} = 15.689$

$\beta_{3w} := 22 \cdot \varepsilon$ $\beta_{3w} = 21.573$

$class_w := if\left(\beta_w \le \beta_{1w}, if\left(\beta_w > \beta_{2w}, if\left(\beta_w > \beta_{3w}, 4, 3\right), 2\right), 1\right)$ $class_w = 1$

[1] 5.4.5 Local buckling

$$\rho_{cw} := if\left[\frac{\beta_w}{\varepsilon} \le 22, 1.0, \frac{32}{\left(\frac{\beta_w}{\varepsilon}\right)} - \frac{220}{\left(\frac{\beta_w}{\varepsilon}\right)^2}\right]$$ $\rho_{cw} = 1$

$t_{w.ef.b} := if\left(class_w \ge 4, t_w \cdot \rho_{cw}, t_w\right)$ ($_b$ = bending) $t_{w.ef.b} = 5.0 \cdot mm$

b) Flanges

[1] 5.4.3 $\psi := 1$

[1] (5.7.),(5.8.) $g := if\left(\psi > -1, 0.7 + 0.3 \cdot \psi, \dfrac{0.8}{1 - \psi}\right)$ $g = 1$

$b_2 := \dfrac{b - t_w - 2 \cdot r}{2}$ $t_2 := t_f$ $\beta_f := g \cdot \dfrac{b_2}{t_2}$ $\beta_f = 4.821$

[1] Tab. 5.1 $\varepsilon = 0.981$ $\beta_{1f} := 3 \cdot \varepsilon$ $\beta_{1f} = 2.942$

$\beta_{2f} := 4.5 \cdot \varepsilon$ $\beta_{2f} = 4.413$

$\beta_{3f} := 6 \cdot \varepsilon$ $\beta_{3f} = 5.883$

$class_f := if\left(\beta_f \le \beta_{1f}, if\left(\beta_f > \beta_{2f}, if\left(\beta_f > \beta_{3f}, 4, 3\right), 2\right), 1\right)$ $class_f = 3$

[1] 5.4.5 Local buckling:

$$\rho_{cf} := if\left[\frac{\beta_f}{\varepsilon} \le 6, 1.0, \frac{10}{\left(\frac{\beta_f}{\varepsilon}\right)} - \frac{24}{\left(\frac{\beta_f}{\varepsilon}\right)^2}\right] \qquad \rho_{cf} = 1$$

$$t_{f.ef} := if\left(class_f \ge 4, t_f \cdot \rho_{cf}, t_f\right) \qquad t_{f.ef} = 14.0 \circ mm$$

Classification of the cross-section in y-y axis bending

$$class_y := if\left(class_f > class_w, class_f, class_w\right) \qquad class_y = 3$$

6.1.4 Classification of the cross section in z-z-axis bending

Cross section class of web: No bending stresses $class_w := 1$

Cross section class for flanges: According to above $class_f = 3$

$$class_z := if\left(class_f > class_w, class_f, class_w\right) \qquad class_z = 3$$

6.1.5 Classification of the cross section in axial compression

a) Web

$\beta_{wc} :=$ compression

$b_1 := b_w$ $\quad t_1 := t_w$ $\quad \beta_{wc} := \frac{b_1}{t_1}$ $\qquad \beta_{wc} = 24.4$

[1] Tab. 5.1

$\beta_{1w} = 10.786$

$\beta_{2w} = 15.689$

$\beta_{3w} = 21.573$

$$class_{wc} := if\left(\beta_{wc} \le \beta_{1w}, if\left(\beta_{wc} > \beta_{2w}, if\left(\beta_{wc} > \beta_{3w}, 4, 3\right), 2\right), 1\right) \qquad class_{wc} = 4$$

[1] 5.4.5 Local buckling

$$\rho_{cw} := if\left[\frac{\beta_{wc}}{\varepsilon} \le 22, 1.0, \frac{32}{\left(\frac{\beta_{wc}}{\varepsilon}\right)} - \frac{220}{\left(\frac{\beta_{wc}}{\varepsilon}\right)^2}\right] \qquad \rho_{cw} = 0.931$$

$$t_{w.ef} := if\left(class_{wc} \ge 4, t_w \cdot \rho_{cw}, t_w\right) \qquad t_w = 5 \circ mm \qquad t_{w.ef} = 4.7 \circ mm$$

b) Flanges

Same as in bending $\qquad t_{f.ef} = 14.0 \circ mm \qquad class_f = 3$

Classification of the total cross-section in axial compression

$$class_c := if\left(class_f > class_{wc}, class_f, class_{wc}\right) \qquad class_c = 4$$

6.1.6. Welds

[1] 5.5 HAZ softening at column ends

[1] Tab.5.2 $\rho_{haz} := 0.65$

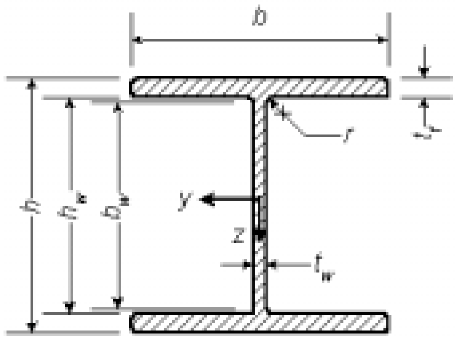
[1] Fig.5.6 Extent of HAZ (MIG-weld) $t_1 := t_f$

$$b_{haz} := if\left(t_1 > 6 \cdot mm, if\left(t_1 > 12 \cdot mm, if\left(t_1 > 25 \cdot mm, 40 \cdot mm, 35 \cdot mm\right), 30 \cdot mm\right), 20 \cdot mm\right)$$

$$b_{haz} = 35 \cdot mm$$

6.1.7 Design resistance, y-y-axis bending

[1] 5.6.1 Elastic modulus of the gross cross section W_{el}:



$A_g := 2 \cdot b \cdot t_f + \left(h - 2 \cdot t_f\right) \cdot t_w$ $\qquad A_g = 4.86 \cdot 10^3 \cdot mm^2$

$$I_g := \frac{1}{12} \cdot \left[b \cdot h^3 - \left(b - t_w\right) \cdot \left(h - 2 \cdot t_f\right)^3\right]$$

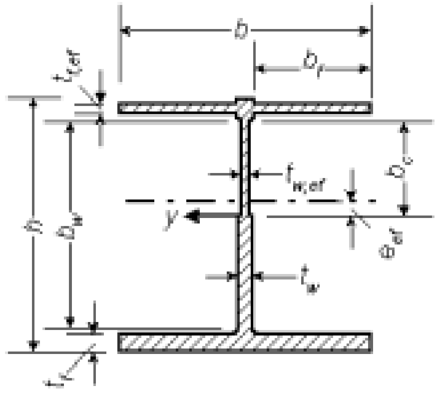
$$I_g = 2.341 \cdot 10^7 \cdot mm^4$$

$W_{el} := \frac{I_g \cdot 2}{h}$ $\qquad W_{el} = 2.926 \cdot 10^5 \cdot mm^3$

Plastic modulus of the gross cross section W_{pl}:

$W_{pl} := \frac{1}{4} \cdot \left[b \cdot h^2 - \left(b - t_w\right) \cdot \left(h - 2 \cdot t_f\right)^2\right]$ $\qquad W_{pl} = 3.284 \cdot 10^5 \cdot mm^3$

Elastic modulus of the effective cross section W_{effe}:



$t_f = 14 \cdot mm$ $\quad$ $t_{f.ef} = 14 \cdot mm$

As $t_{f.ef} = t_f$ then $\quad$ $b_c := \frac{b_w}{2}$ $\quad$ $b_c = 61 \cdot mm$

$t_w = 5 \cdot mm$

$t_{w.ef.b} = 5 \cdot mm$

$b_f := 0.5 \cdot (b - t_w - 2 \cdot r)$ $\quad$ $b_f = 67.5 \cdot mm$

$A_{eff} := A_g - 2 \cdot b_f \cdot (t_f - t_{f.ef}) - b_c \cdot (t_w - t_{w.ef.b})$ $\quad$ $A_{eff} = 4.86 \cdot 10^3 \cdot mm^2$

Shift of gravity centre:

$$e_{ef} := \left[2 \cdot b_f \cdot (t_f - t_{f.ef}) \cdot \left(\frac{h}{2} - \frac{t_f}{2} \right) + \frac{b_c^{\ 2}}{2} \cdot (t_w - t_{w.ef.b}) \right] \cdot \frac{1}{A_{eff}} \qquad e_{ef} = 0 \cdot mm$$

Second moment of area wiht respect to centre of gross cross section:

$$I_{eff} := I_g - 2 \cdot b_f \cdot (t_f - t_{f.ef}) \cdot \left(\frac{h}{2} - \frac{t_f}{2} \right)^2 - \frac{b_c^{\ 3}}{3} \cdot (t_w - t_{w.ef.b}) \qquad I_{eff} = 2.341 \cdot 10^7 \cdot mm^4$$

Second moment of area wiht respect to centre of effective gross section:

$$I_{eff} := I_{eff} - e_{ef}^{\ 2} \cdot A_{eff} \qquad I_{eff} = 2.341 \cdot 10^7 \cdot mm^4$$

$$W_{eff} := \frac{I_{eff}}{\frac{h}{2} + e_{ef}} \qquad W_{eff} = 2.926 \cdot 10^5 \cdot mm^3$$

[1] Tab. 5.3 $\quad$ Shape factor α

- for class 1 or 2 cross-sections:

$$\alpha_{1.2.w} := \frac{W_{pl}}{W_{el}} \qquad \alpha_{1.2.w} = 1.122$$

- for welded, class 3 cross-sections:

[1] Tab. 5.3 Shape factor α

- for class 1 or 2 cross-sections:

$$\alpha_{1.2.w} := \frac{W_{pl}}{W_{el}} \qquad \alpha_{1.2.w} = 1.122$$

- for welded, class 3 cross-sections:

[1] (5.16) $$\alpha_{3.ww} := \left[1 + \left(\frac{\beta_{3w} - \beta_w}{\beta_{3w} - \beta_{2w}}\right) \cdot \left(\frac{W_{pl} - W_{el}}{W_{el}}\right)\right] \qquad \alpha_{3.ww} = 1.245$$

[1] (5.16) $$\alpha_{3.wf} := \left[1 + \left(\frac{\beta_{3f} - \beta_f}{\beta_{3f} - \beta_{2f}}\right) \cdot \left(\frac{W_{pl} - W_{el}}{W_{el}}\right)\right] \qquad \alpha_{3.wf} = 1.088$$

β, β_2, β_3 are the slenderness parameter and the limiting values for the most critical element in the cross-section, so it is the smaller value of $\alpha_{3.ww}$ and $\alpha_{3.wf}$

$$\alpha_{3.w} := if\left(\alpha_{3.ww} \le \alpha_{3.wf}, \alpha_{3.ww}, \alpha_{3.wf}\right) \qquad \alpha_{3.w} = 1.088$$

- for class 4 cross-sections: $$\alpha_{4.w} := \frac{W_{eff}}{W_{el}} \qquad \alpha_{4.w} = 1$$

$class_y = 3$

$$\alpha_y := if\left(class_y > 2, if\left(class_y > 3, \alpha_{4.w}, \alpha_{3.w}\right), \alpha_{1.2.w}\right) \qquad \alpha_y = 1.088$$

Design moment of resistance of the cross section $M_{c,Rd}$

[1] (5.14) $$M_{y.Rd} := \frac{f_o \cdot \alpha_y \cdot W_{el}}{\gamma_{M1}} \qquad M_{y.Rd} = 75.3 \circ kNm$$

6.1.8 Design resistance, z-z-axis bending

Cross section class $class_z = 3$

Gross cross section: $$I_z := 2 \cdot \frac{t_f \cdot b^3}{12} \qquad I_z = 7.875 \cdot 10^6 \circ mm^4$$

Effective cross section: $$I_{z.ef} := 2 \cdot \frac{t_{f.ef} \cdot b^3}{12} \qquad I_{z.ef} = 7.875 \cdot 10^6 \circ mm^4$$

Section moduli: $$W_z := \frac{I_z \cdot 2}{b} \qquad W_{z.ef} := \frac{I_{z.ef} \cdot 2}{b}$$

Shape factor: $$\alpha_z := \frac{W_z}{W_{z.ef}} \qquad \alpha_z = 1$$

Bending resistance: $$M_{z.Rd} := \frac{f_o \cdot \alpha_z \cdot W_z}{\gamma_{M1}} \qquad M_{z.Rd} = 24.818 \circ kNm$$

6.1.9 Axial force resistance, y-y buckling

[1] 5.8.4 Cross section area of gross cross section A_{gr}

$$A_{gr} := b \cdot h - (b - t_w) \cdot (h - 2 \cdot t_f) \qquad A_{gr} = 4.86 \cdot 10^3 \circ mm^2$$

Cross section area of effective cross section A_{ef} $\qquad t_{w.ef} = 4.653 \circ mm$

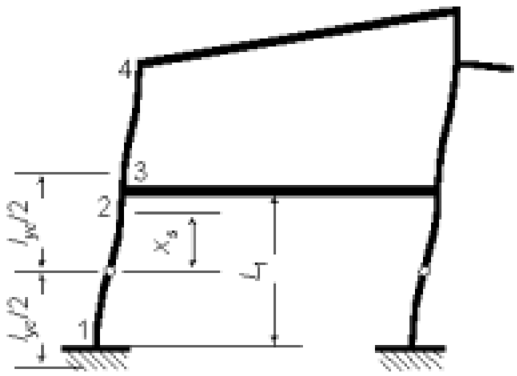
$$A_{ef} := A_{gr} - 2 \cdot b_2 \cdot (t_f - t_{f.ef}) - b_w \cdot (t_w - t_{w.ef}) \qquad A_{ef} = 4.818 \cdot 10^3 \circ mm^2$$

$$(t_f = 14 \circ mm \quad t_w = 5 \circ mm \quad 2 \cdot b_2 = 135 \circ mm \quad t_{f.ef} = 14 \circ mm \quad t_{w.ef} = 4.653 \circ mm)$$

Effective cross section factor $\eta := \dfrac{A_{ef}}{A_{gr}}$ $\qquad \eta = 0.991$

Second moment of area of gross cross section I_y:

$$I_y := \frac{2}{12} \cdot b \cdot t_f^3 + 2 \cdot b \cdot t_f \cdot \left(\frac{h - t_f}{2}\right)^2 + \frac{1}{12} \cdot (h - 2 \cdot t_f)^3 \cdot t_w$$



[1] Table 5.7 Case 5 Buckling length factor $K_y := 1.5 \qquad L_1 = 3 \circ m$

$$l_{yc} := K_y \cdot L_1 \qquad l_{yc} = 4.5 \text{ m}$$

Buckling load $N_{cr} := \dfrac{\pi^2 \cdot E \cdot I_y}{l_{yc}^2}$

$$N_{cr} = 798.639 \circ kN$$

[1] 5.8.4.1 Slenderness parameter $\lambda_y := \sqrt{\dfrac{A_{gr} \cdot \eta \cdot f_o}{N_{cr}}}$ $\qquad \lambda_y = 1.252$

[1] Table 5.6 $\alpha := if(heat\_treated = 1, 0.2, 0.32)$ $\qquad \alpha = 0.2$

$\lambda_o := if(heat\_treated = 1, 0.1, 0)$ $\qquad \lambda_o = 0.1$

$$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda_y - \lambda_o) + \lambda_y^2\right] \qquad \phi = 1.399$$

$$\chi_y := \frac{1}{\phi + \sqrt{\phi^2 - \lambda_y^2}} \qquad \chi_y = 0.494$$

[1] Table 5.5 Symmetric profile $\qquad k_1 := 1$

[1] Table 5.5 No longitudinal welds $\qquad k_2 := 1$

Axial force resistance $N_{y.Rd} := \chi_y \cdot \eta \cdot k_1 \cdot k_2 \cdot \dfrac{f_o}{\gamma_{M1}} \cdot A_{gr}$ $\qquad N_{y.Rd} = 562.6 \circ kN$

6.1.10 Axial force resistance, z-z axis buckling

Buckling length = distance between purlins $K_z := \frac{c_p}{L_1}$ $K_z \cdot L_1 = 1 \circ m$

Buckling load $N_{cr} := \frac{\pi^2 \cdot E \cdot I_z}{\left(K_z \cdot L_1\right)^2}$ $N_{cr} = 5.4 \cdot 10^3 \circ kN$

[1] 5.8.4.1 Slenderness factor $\lambda_z := \sqrt{\frac{A_{gr} \cdot \eta \cdot f_o}{N_{cr}}}$ $\lambda_z = 0.48$

[1] Table 5.6 $\alpha := if(heat\_treated = 1, 0.2, 0.32)$ $\alpha = 0.2$

$\lambda_o := if(heat\_treated = 1, 0.1, 0)$ $\lambda_o = 0.1$

[1] 5.8.4.1 $\phi := 0.5 \cdot \left[1 + \alpha \cdot \left(\lambda_z - \lambda_o\right) + \lambda_z^2\right]$ $\phi = 0.653$

$\chi_z := \frac{1}{\phi + \sqrt{\phi^2 - \lambda_z^2}}$ $\chi_z = 0.912$

[1] Table 5.5 Symmetric profile $k_1 := 1$

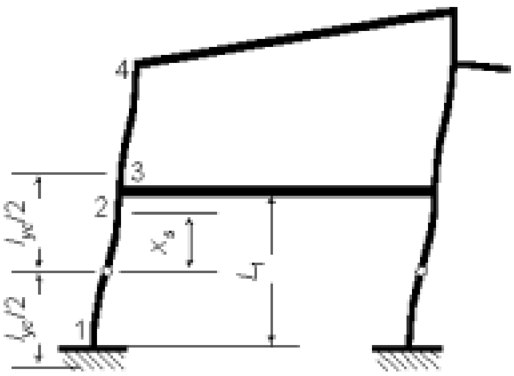
[1] Table 5.5 No longitudinal welds $k_2 = 1$

[1] 5.8.4.1 Axial load resistance $N_{z.Rd} := \chi_z \cdot \eta \cdot k_1 \cdot k_2 \cdot \frac{f_o}{\gamma_{M1}} \cdot A_{gr}$ $N_{z.Rd} = 1.039 \cdot 10^3 \circ kN$

(6.1.9) Compare y-y axis buckling $N_{y.Rd} = 562.622 \circ kN$

and without column buckling $N_{Rd} := \eta \cdot \frac{f_o}{\gamma_{M1}} \cdot A_{gr}$ $N_{Rd} = 1.139 \cdot 10^3 \circ kN$

6.1.11 Flexural buckling of beam-column



[1] Table 5.5 Buckling length factor, frame buckling

Case 5 $K_y = 1.5$ $l_{yc} := K_y \cdot L_1$ $l_{yc} = 4.5 \text{ m}$

[1] 5.8.4.1 The ends of column part 1-2 is designing $x_s := \frac{L_1}{2}$ $\frac{x_s}{l_{yc}} = 0.333$ $x_s = 1.5 \text{ m}$

[1] 5.9.4.5 HAZ reduction factors $\rho_{haz} = 0.65$

[1] (5.51) $\omega_0 := \rho_{haz} \cdot \frac{f_u}{\gamma_{M2}} \cdot \frac{\gamma_{M1}}{f_o}$ $\omega_0 := if(\omega_0 > 1, 1, \omega_0)$ $\omega_0 = 0.682$

[1] (5.49) $\omega_x := \frac{\omega_0}{\chi_y + (1 - \chi_y) \cdot \sin\left(\frac{\pi \cdot x_s}{l_{yc}}\right)}$ $\omega_x = 0.732$

Exponents in interaction formulae

[1] (5.42c) $\xi_0 := \alpha_y^2$ $\xi_0 := if(\xi_0 < 1, 1, \xi_0)$ $\xi_0 = 1.184$

[1] 5.9.4.2 $\xi_{yc} := \xi_0 \cdot \chi_y$ $\xi_{yc} := if(\xi_{yc} < 0.8, 0.8, \xi_{yc})$ $\xi_{yc} = 0.8$

Flexural buckling check

Bending moment $M_{y.Ed} := M_{1.Ed}$ $M_{y.Ed} = 37.7 \cdot kNm$

[1] 5.4.4 $U_y := \left(\frac{N_{Ed}}{\chi_z \cdot \omega_x \cdot N_{Rd}}\right)^{\xi_{yc}} + \frac{M_{y.Ed}}{\omega_0 \cdot M_{y.Rd}}$ $U_y = 0.99$

or with simplified exponents

$U_{ys} := \left(\frac{N_{Ed}}{\chi_z \cdot \omega_x \cdot N_{Rd}}\right)^{0.8} + \left(\frac{M_{y.Ed}}{\omega_0 \cdot M_{y.Rd}}\right)^{1.0}$ $U_{ys} = 0.99$

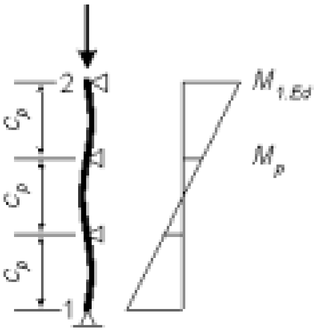
6.1.12 Lateral-torsional buckling between purlins

Moment in section 2 $M_{1.Ed} = 37.7 \cdot kNm$

Moment in section 1 $M_{2.Ed} = -25.4 \cdot kNm$

Moment c_p from section 2 $c_p = 1000 \cdot mm$

$M_p := M_{1.Ed} - \frac{M_{1.Ed} - M_{2.Ed}}{L_1} \cdot c_p$ $M_p = 16.667 \cdot kNm$



[1] 5.9.4.3 Lateral-torsional buckling

[1] Figure J.2 Varping constant: $I_w := \frac{(h - t_f)^2 \cdot I_z}{4}$ $I_w = 4.197 \cdot 10^{10} \cdot mm^6$

Torsional constant: $I_t := \frac{2 \cdot b \cdot t_f^3 + h \cdot t_w^3}{3}$ $I_t = 2.811 \cdot 10^5 \cdot mm^4$

Lateral buckling length $L := c_p$ $W_y := W_{el}$ $W_y = 2.926 \cdot 10^5 \cdot mm^3$

[1] H.1.2 Moment relation $\psi := \frac{M_p}{M_{1.Ed}}$ $\psi = 0.442$

[1] H.1.2(6) C_1 - constant $C_1 := 1.88 - 1.4 \cdot \psi + 0.52 \cdot \psi^2$ $C_1 = 1.363$

Shear modulus $G := \dfrac{E}{2.6}$ $G = 2.692 \cdot 10^4 \circ MPa$

[1] H.1.3(3) $M_{cr} := \dfrac{C_1 \cdot \pi^2 \cdot E \cdot I_z}{L^2} \cdot \sqrt{\dfrac{I_w}{I_z} + \dfrac{L^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z}}$ $L = 1000 \circ mm$ $M_{cr} = 607.759 \circ kNm$

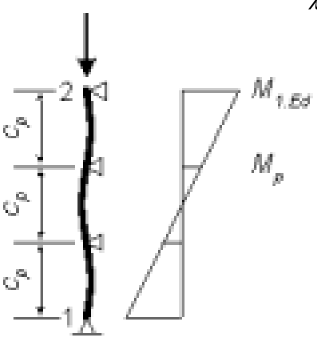
[1] 5.6.6.3(3) $\lambda_{LT} := \sqrt{\dfrac{\alpha_y \cdot W_y \cdot f_o}{M_{cr}}}$ $\lambda_{LT} = 0.369$

[1] 5.6.6.3(2) $\alpha_{LT} := if(class_z > 2, 0.2, 0.1)$ $\alpha_{LT} = 0.2$

$\lambda_{0LT} := if(class_z > 2, 0.4, 0.6)$ $\lambda_{0LT} = 0.4$

[1] 5.6.6.3(1) $\phi_{LT} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{LT} - \lambda_{0LT}) + \lambda_{LT}^2\right]$ $\phi_{LT} = 0.565$

$\chi_{LT} := \dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}}$ $\chi_{LT} = 1.007$



Check sections

$l_{zc} := c_p$

$i := 1 .. 8$ $x_{s_i} := \dfrac{i-2}{5} \cdot l_{zc}$

$x_{s_1} := 0 \cdot m$ $x_{s_2} := b_{haz}$

$x_{s_7} := l_{zc} - b_{haz}$ $x_{s_8} := l_{zc}$

$\dfrac{x_s}{l_{zc}}^T = (0 \;\; 0.035 \;\; 0.2 \;\; 0.4 \;\; 0.6 \;\; 0.8 \;\; 0.965 \;\; 1)$

HAZ reduction factors (ω_0 = 1 except at column ends)

[1] (5.51) $\omega_{0_i} := \overrightarrow{\left(\rho_{haz} \cdot \dfrac{f_u}{\gamma_{M2}} \cdot \dfrac{\gamma_{M1}}{f_o}\right)}$ $\omega_{0_i} := if(\omega_{0_i} > 1, 1, \omega_{0_i})$

Weld at section i = 1 (column end) and at section i = 7 (fixing of purlin) $\omega_{0_i} := if\left[(i < 2) + (i > 7), \omega_{0_i}, 1\right]$

$\omega_0^T = (0.682 \;\; 1 \;\; 1 \;\; 1 \;\; 1 \;\; 1 \;\; 1 \;\; 0.682)$

[1] (5.49) or (5.52) $\omega_x := \overrightarrow{\dfrac{\omega_0}{\chi_z + (1 - \chi_z) \cdot sin\left(\dfrac{\pi \cdot x_s}{l_{zc}}\right)}}$ $\omega_x^T = (0.75 \;\; 1.08 \;\; 1.04 \;\; 1 \;\; 1 \;\; 1.04 \;\; 1.08 \;\; 0.75)$

[1] (5.50) or (5.53) $\omega_{xLT} := \overrightarrow{\frac{\omega_0}{\chi_{LT} + (1 - \chi_{LT})\cdot sin\left(\frac{\pi \cdot x_s}{l_{zc}}\right)}}$ $\omega_{xLT}^T = (0.68\ \ 0.99\ \ 1\ \ 1\ \ 1\ \ 1\ \ 0.99\ \ 0.68)$

[1] (5.42a) $\eta_0 := \alpha_z^2 \cdot \alpha_y^2$ $\eta_0 := if(\eta_0 < 1, 1, if(\eta_0 > 2, 2, \eta_0))$ $\eta_0 = 1.184$

[1] (5.42b) $\gamma_0 := \alpha_z^2$ $\gamma_0 := if(\gamma_0 < 1, 1, if(\gamma_0 > 2, 2, \gamma_0))$ $\gamma_0 = 1$

[1] (5.42c) $\xi_0 := \alpha_y^2$ $\xi_0 := if(\xi_0 < 1, 1, \xi_0)$ $\xi_0 = 1.184$

[1] 5.9.4.3 $\eta_c := \eta_0 \cdot \chi_z$ $\eta_c := if(\eta_c < 0.8, 0.8, \eta_c)$ $\chi_y = 0.494$ $\eta_c = 1.08$

$\gamma_c := \gamma_0$ $\chi_z = 0.912$ $\gamma_c = 1$

$\xi_{zc} := \xi_0 \cdot \chi_z$ $\xi_{zc} := if(\xi_{zc} < 0.8, 0.8, \xi_{zc})$ $\xi_{zc} = 1.08$

Lateral-torsional buckling of beam-column

Bending moment in section x_s $M_{y.Ed} := M_{1.Ed} - (M_{1.Ed} - M_p)\cdot\frac{x_s}{l_{zc}}$

$M_{z.Ed} := 0\cdot kNm$ $\left(\frac{M_{y.Ed}}{M_{1.Ed}}\right)^T = (1\ \ 0.98\ \ 0.888\ \ 0.777\ \ 0.665\ \ 0.554\ \ 0.462\ \ 0.442)$

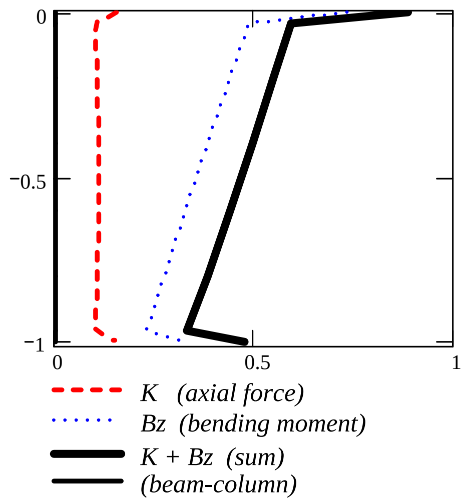
[1] (5.43) $U_{LT} := \overrightarrow{\left[\left(\frac{N_{Ed}}{\chi_z \cdot \omega_x \cdot N_{Rd}}\right)^{\eta_c} + \left(\frac{M_{y.Ed}}{\chi_{LT} \cdot \omega_{xLT} \cdot M_{y.Rd}}\right)^{\gamma_c} + \left(\frac{M_{z.Ed}}{\omega_0 \cdot M_{z.Rd}}\right)^{\xi_{zc}}\right]}$

$U_{LT}^T = (0.889\ \ 0.594\ \ 0.552\ \ 0.499\ \ 0.443\ \ 0.385\ \ 0.334\ \ 0.479)$

Max utilisation, lateral-torsional buckling $U_{z.max} := max\left(U_{LT}\right)$ $U_{z.max} = 0.889$

Max utilisation, flexural buckling $U_y = 0.99$

$K := \overrightarrow{\left(\frac{N_{Ed}}{\chi_z \cdot \varphi_x \cdot N_{Rd}}\right)^{\eta_c}}$ $B_z := \overrightarrow{\left(\frac{M_{y.Ed}}{\chi_{LT} \cdot \omega_{xLT} \cdot M_{y.Rd}}\right)^{\gamma_c}}$ $O_i := 0.005$



6.1.13 Design moment at column base

Design section $x_s := \frac{L_1}{2}$ $l_{yc} = 4.5\ \mathrm{m}$ $x_s = 1.5 \circ m$

Second order bending moment $\Delta M := \frac{N_{Ed} \cdot W_y}{A_{ef}} \cdot \left(\frac{1}{\chi_y} - 1\right) \cdot sin\left(\frac{\pi \cdot x_s}{l_{yc}}\right)$ $\Delta M = 7.43 \circ kNm$

Design moment at column base $M_{A.base} := \left|M_{2.Ed}\right| + \Delta M$ $M_{A.base} = 32.8 \circ kNm$

Axial force corresponding to $M_{D.base}$ $N_{A.corre} := N_{Ed}$ $N_{A.corre} = 138 \circ kN$

6.1.14 Deflections

[1] 4.2.4 $I_{gr} := \frac{1}{12}\cdot\left[b\cdot h^3 - (b - t_w)\cdot(h - 2\cdot t_f)^3\right]$ $I_{gr} = 2.341\cdot10^7 \cdot mm^4$

To calculate the fictive second moment of area I_{fic}, the bending moment in the serviceability limit state is supposed to be half the maximum bending moment at the ultimate limit state.

$\sigma_{gr} := \frac{0.5\cdot M_{1.Ed}}{I_{gr}}\cdot\frac{h}{2}$ $\sigma_{gr} = 64 \cdot MPa$

Allowing for a reduced stress level I_{fic} may be used constant along the beam.

[1] (4.2) $I_{fic} := I_{gr} - \frac{\sigma_{gr}}{f_o}\cdot(I_{gr} - I_{eff})$ $I_{fic} = 2.341\cdot10^7 \cdot mm^4$

$I := if(class_y = 4, I_{fic}, I_{gr})$ $class_y = 3$ $I = 2.341\cdot10^7 \cdot mm^4$

Horisontal deformation according to FEM calculation $\delta_1 := 0.6\cdot mm$

$\delta_2 := 4.1\cdot mm$

Pre-camber $\delta_0 := 0\cdot mm$

[1] (4.1) $\delta_{max} := \delta_1 + \delta_2 - \delta_0$ $\delta_{max} = 4.7 \cdot mm$

[1] 4.2.3 Limit horizontal deformation for building frame with $h_{top} := x_4\cdot m$ $h_{top} = 5.5$ m

$\delta_{limit} := \frac{h_{top}}{300}$ $\delta_{limit} = 18 \cdot mm$

6.1.15 Summary

$M_{1.Ed} = 38 \cdot kNm$ $M_{y.Rd} = 75 \cdot kNm$ $\omega_{\rho} = 0.682$ $\omega_{x} = 0.748$ $\frac{M_{1.Ed}}{\omega_{\rho} M_{y.Rd}} = 0.734$

$N_{Ed} = 138 \cdot kN$ $N_{y.Rd} = 562.6 \cdot kN$ $\chi_y = 0.494$ $\frac{N_{Ed}}{\chi_y\cdot\omega_{x_1}\cdot N_{y.Rd}} = 0.664$

Utilisation, flexural buckling - HAZ at column base $U_y = 0.99$

Utilisation, lateral-torsional buckling $U_{z.max} = 0.889$

$\delta_{limit} = 18.3 \cdot mm$ $\delta_{max} = 5 \cdot mm$ $\frac{\delta_{max}}{\delta_{limit}} = 0.256$

Effective second moment of area $I_{fic} = 2.341\cdot10^7 \cdot mm^4$

Cross section $h = 160 \cdot mm$ $b = 150 \cdot mm$ $t_w = 5 \cdot mm$ $t_f = 14 \cdot mm$ $A_{gr} = 4.86\cdot10^3 \cdot mm^2$

6.2 Column B

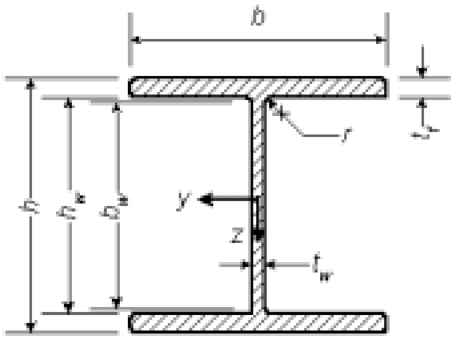
6.2.1 Dimensions and material properties

Flange height: $h := 200 \cdot mm$

Flange depth: $b := 160 \cdot mm$

Web thickness: $t_w := 7 \cdot mm$

Flange thickness: $t_f := 16 \cdot mm$



Overall length: $L_1 := 3 \cdot m$

Distance between purlins: $c_p := 3 \cdot m$

[1] Table 3.2b Alloy: **EN AW-6082 T6** EP/O t > 5 mm

$f_{0.2} := 260 \cdot MPa$ $f_u := 310 \cdot MPa$

$heat\_treated := 1$ (if heat-treated then 1 else 0)

[1] (5.4), (5.5) $f_o := f_{0.2}$ $f_a := f_u$

[1] (5.6) $f_v := \dfrac{f_o}{\sqrt{3}}$ $f_v = 150 \circ MPa$ $E := 70000 \cdot MPa$ $G := 27000 \cdot MPa$

Partial safety factors: $\gamma_{M1} \equiv 1.10$ $\gamma_{M2} \equiv 1.25$

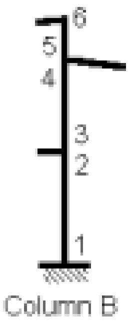
Inner radius: $r := 5 \cdot mm$

Web width: $b_w := h - 2 \cdot t_f - 2 \cdot r$ $b_w = 158 \circ mm$

S.I. units: $kN \equiv 1000 \cdot newton$ $kNm \equiv kN \cdot m$ $MPa \equiv 1000000 \cdot Pa$

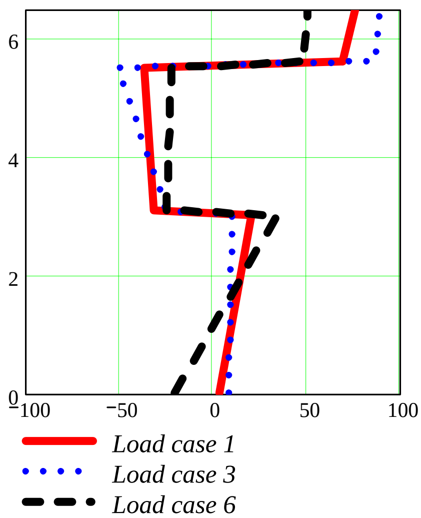
6.2.2 Internal moments and forces

(5.4.2) Bending moments and axial forces for LC1, LC3 and LC4 in section 1 to 6 (axial compression force = +) $i := 1..6$

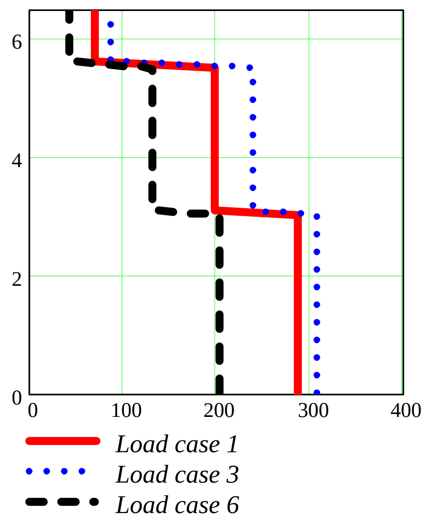


$i =$	$\frac{x_i}{m} =$	$\frac{M_{LC1_i}}{kNm} =$	$\frac{M_{LC3_i}}{kNm} =$	$\frac{M_{LC4_i}}{kNm} =$	$\frac{N_{LC1_i}}{kN} =$	$\frac{N_{LC3_i}}{kN} =$	$\frac{N_{LC4_i}}{kN} =$
1	0	-3.04	-8.9	25.4	288	307	-11.7
2	3	-20.7	-10.6	-23	288	307	-11.7
3	3.1	32	24.7	5.25	198	240	-28.7
4	5.5	36.5	50.6	-12.1	198	240	-28.7
5	5.6	-69.1	-86.2	9.64	69.3	86.5	-11.5
6	6.5	-76.2	-89.3	9.98	69.3	86.5	-11.5

Bending moment kNm



Axial compressive force kN



	Load case 1	*Load case 3*	*Load case 4*
Moment in section 2	$-M_{LC1_2} = 20.7 \cdot kNm$	$-M_{LC3_2} = 10.6 \cdot kNm$	$-M_{LC4_2} = 23 \cdot kNm$
Moment at column base 1	$-M_{LC1_1} = 3.04 \cdot kNm$	$-M_{LC3_1} = 8.9 \cdot kNm$	$-M_{LC4_1} = -25.4 \cdot kNm$
Axial force in part 1-2	$N_{LC1_1} = 288 \cdot kN$	$N_{LC3_1} = 307 \cdot kN$	$N_{LC4_1} = -11.7 \cdot kN$

Preliminary calculations show that load case 1 is governing (except for welds in column base). Study part 1-2 from column base to floor beam. Moment in top of part 1-2 (section 2) is larger than at column base (section 1) why $M_{1.Ed}$ below correspond to section 2 and $M_{2.Ed}$ to section 1 of the column.

Load case 1

Bending moment in section 2	$M_{1.Ed} := -M_{LC1_2}$	$M_{1.Ed} = 20.7 \cdot kNm$
Bending moment at column base (1)	$M_{2.Ed} := -M_{LC1_1}$	$M_{2.Ed} = 3.04 \cdot kNm$
Axial force in part 1-2 (compression)	$N_{Ed} := N_{LC1_1}$	$N_{Ed} = 288 \cdot kN$

6.2.3 Classification of the cross section in y-y-axis bending

$\psi_w \equiv bending$

a) Web

[1] 5.4.3 $b_1 := b_w$ $\quad t_1 := t_w$ $\quad \beta_w := 0.40 \cdot \frac{b_1}{t_1}$ $\quad \beta_w = 9.029$

[1] Tab. 5.1 $\varepsilon := \sqrt{\frac{250}{f_o} \cdot \frac{newton}{mm^2}}$ $\quad \beta_{1w} := 11 \cdot \varepsilon$ $\quad \beta_{1w} = 10.786$

Heat treated, unwelded = no longitudinal weld

$\beta_{2w} := 16 \cdot \varepsilon$ $\quad \beta_{2w} = 15.689$

$\beta_{3w} := 22 \cdot \varepsilon$ $\quad \beta_{3w} = 21.573$

$class_w := if(\beta_w \le \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1)$ $\quad class_w = 1$

[1] 5.4.5 Local buckling

$\rho_{cw} := if\left[\frac{\beta_w}{\varepsilon} \le 22, 1.0, \frac{32}{\left(\frac{\beta_w}{\varepsilon}\right)} - \frac{220}{\left(\frac{\beta_w}{\varepsilon}\right)^2}\right]$ $\quad \rho_{cw} = 1$

$t_{w.ef.b} := if(class_w \ge 4, t_w \cdot \rho_{cw}, t_w)$ $\quad$ ($_b$ = bending) $\quad t_{w.ef.b} = 7.0 \circ mm$

b) Flanges

[1] 5.4.3 $\psi := 1$

[1] (5.7.),(5.8.) $g := if\left(\psi > -1, 0.7 + 0.3 \cdot \psi, \frac{0.8}{1 - \psi}\right)$ $\quad g = 1$

$b_2 := \frac{b - t_w - 2 \cdot r}{2}$ $\quad t_2 := t_f$ $\quad \beta_f := g \cdot \frac{b_2}{t_2}$ $\quad \beta_f = 4.469$

[1] Tab. 5.1 $\varepsilon = 0.981$ $\quad \beta_{1f} := 3 \cdot \varepsilon$ $\quad \beta_{1f} = 2.942$

$\beta_{2f} := 4.5 \cdot \varepsilon$ $\quad \beta_{2f} = 4.413$

$\beta_{3f} := 6 \cdot \varepsilon$ $\quad \beta_{3f} = 5.883$

$class_f := if(\beta_f > \beta_{1f}, if(\beta_f > \beta_{2f}, if(\beta_f > \beta_{3f}, 4, 3), 2), 1)$ $\quad class_f = 3$

[1] 5.4.5 Local buckling:

$\rho_{cf} := if\left[\frac{\beta_f}{\varepsilon} \le 6, 1.0, \frac{10}{\left(\frac{\beta_f}{\varepsilon}\right)} - \frac{24}{\left(\frac{\beta_f}{\varepsilon}\right)^2}\right]$ $\quad \rho_{cf} = 1$

$t_{f.ef} := if(class_f \ge 4, t_f \cdot \rho_{cf}, t_f)$ $\quad t_{f.ef} = 16.0 \circ mm$

Classification of the cross-section in y-y axis bending

$class_y := if(class_f > class_w, class_f, class_w)$ $\quad class_y = 3$

6.2.4 Classification of the cross section in z-z-axis bending

Cross section class of web: No bending stresses $class_w := 1$

Cross section class for flanges: According to above $class_f = 3$

$class_z := if\left(class_f > class_w, class_f, class_w\right)$ $class_z = 3$

6.2.5 Classification of the cross section in axial compression

a) Web

$\beta_{wc} := \text{compression}$

$b_1 := b_w$ $t_1 := t_w$ $\beta_{wc} := \dfrac{b_1}{t_1}$ $\beta_{wc} = 22.571$

[1] Tab. 5.1

$\beta_{1w} = 10.786$

$\beta_{2w} = 15.689$

$\beta_{3w} = 21.573$

$class_{wc} := if\left(\beta_{wc} \le \beta_{1w}, if\left(\beta_{wc} > \beta_{2w}, if\left(\beta_{wc} > \beta_{3w}, 4, 3\right), 2\right), 1\right)$ $class_{wc} = 4$

[1] 5.4.5 Local buckling

$$\rho_{cw} := if\left[\frac{\beta_{wc}}{\varepsilon} \le 22, 1.0, \frac{32}{\left(\frac{\beta_{wc}}{\varepsilon}\right)} - \frac{220}{\left(\frac{\beta_{wc}}{\varepsilon}\right)^2}\right]$$

$\rho_{cw} = 0.975$

$t_{w.ef} := if\left(class_{wc} \ge 4, t_w \cdot \rho_{cw}, t_w\right)$ $t_w = 7 \circ mm$ $t_{w.ef} = 6.8 \circ mm$

b) Flanges

Same as in bending $t_{f.ef} = 16.0 \circ mm$ $class_f = 3$

Classification of the total cross-section in axial compression

$class_c := if\left(class_f > class_{wc}, class_f, class_{wc}\right)$ $class_c = 4$

6.2.6. Welds

[1] 5.5

[1] Tab. 5.2 HAZ softening factor at column ends

$\rho_{haz} := 0.65$

[1] Fig. 5.6 Extent of HAZ (MIG-weld)

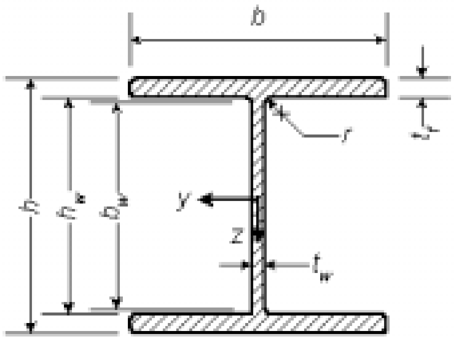
$$b_{haz} := if\left(t_1 > 6 \cdot mm, if\left(t_1 > 12 \cdot mm, if\left(t_1 > 25 \cdot mm, 40 \cdot mm, 35 \cdot mm\right), 30 \cdot mm\right), 20 \cdot mm\right)$$

$$b_{haz} = 30 \, mm$$

6.2.7 Design resistance, y-y-axis bending

[1] 5.6.2

Elastic modulus of gross cross section W_{el}:



$$A_g := 2 \cdot b \cdot t_f + \left(h - 2 \cdot t_f\right) \cdot t_w \qquad A_g = 6.296 \cdot 10^3 \, mm^2$$

$$I_{gr} := \frac{1}{12} \cdot \left[b \cdot h^3 - \left(b - t_w\right) \cdot \left(h - 2 \cdot t_f\right)^3\right]$$

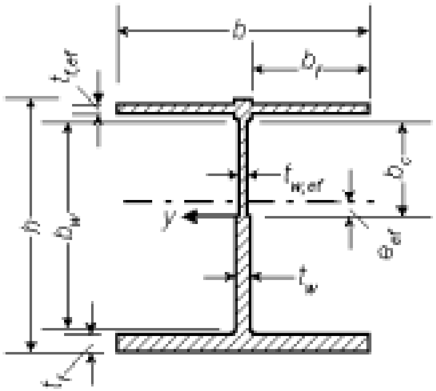
$$I_{gr} = 4.621 \cdot 10^7 \, mm^4$$

$$W_{el} := \frac{I_{gr} \cdot 2}{h} \qquad W_{el} = 4.621 \cdot 10^5 \, mm^3$$

Plastic modulus

$$W_{pl} := \frac{1}{4} \cdot \left[b \cdot h^2 - \left(b - t_w\right) \cdot \left(h - 2 \cdot t_f\right)^2\right] \qquad W_{pl} = 5.204 \cdot 10^5 \, mm^3$$

Elastic modulus of the effective cross section W_{eff}:



$t_f = 16 \, mm$ $\qquad t_{f.ef} = 16 \, mm$

As $t_{f.ef} = t_f$ then $\qquad b_c := \frac{b_w}{2} \qquad b_c = 79 \, mm$

$t_w = 7 \, mm$ $\qquad t_{w.ef.b} = 7 \, mm$

$$b_f := 0.5 \cdot \left(b - t_w - 2 \cdot r\right) \qquad b_f = 71.5 \, mm$$

$$A_{eff} := A_g - 2 \cdot b_f \cdot \left(t_f - t_{f.ef}\right) - b_c \cdot \left(t_w - t_{w.ef.b}\right) \qquad A_{eff} = 6.296 \cdot 10^3 \, mm^2$$

Shift of gravity centre:

$$e_{ef} := \left[2 \cdot b_f \cdot \left(t_f - t_{f.ef}\right) \cdot \left(\frac{h}{2} - \frac{t_f}{2}\right) + \frac{b_c^{\,2}}{2} \cdot \left(t_w - t_{w.ef.b}\right)\right] \cdot \frac{1}{A_{eff}} \qquad e_{ef} = 0 \, mm$$

Centre of gross cross section:

$$I_{eff} := I_{gr} - 2 \cdot b_f \cdot \left(t_f - t_{f.ef}\right) \cdot \left(\frac{h}{2} - \frac{t_f}{2}\right)^2 - \frac{b_c^{\,3}}{3} \cdot \left(t_w - t_{w.ef.b}\right) \qquad I_{eff} = 4.621 \cdot 10^7 \, mm^4$$

Centre of effective gross section:

$I_{eff} := I_{eff} - e_{ef}^{2} \cdot A_{eff}$ $\qquad I_{eff} = 4.621 \cdot 10^{7} \, mm^{4}$

$W_{eff} := \dfrac{I_{eff}}{\dfrac{h}{2} + e_{ef}}$ $\qquad W_{eff} = 4.621 \cdot 10^{5} \, mm^{3}$

[1] Tab. 5.3 Shape factor α

- for welded, class 1 or 2 cross-sections:

$\alpha_{1.2.w} := \dfrac{W_{pl}}{W_{el}}$ $\qquad \alpha_{1.2.w} = 1.126$

- for welded, class 3 cross-sections:

[1] (5.16) $\alpha_{3.ww} := \left[1 + \left(\dfrac{\beta_{3w} - \beta_{w}}{\beta_{3w} - \beta_{2w}}\right) \cdot \left(\dfrac{W_{pl} - W_{el}}{W_{el}}\right)\right]$ $\qquad \alpha_{3.ww} = 1.269$

[1] (5.16) $\alpha_{3.wf} := \left[1 + \left(\dfrac{\beta_{3f} - \beta_{f}}{\beta_{3f} - \beta_{2f}}\right) \cdot \left(\dfrac{W_{pl} - W_{el}}{W_{el}}\right)\right]$ $\qquad \alpha_{3.wf} = 1.121$

β, β_2, β_3 are the slenderness parameter and the limiting values for the most critical element in the cross-section, so it is the smaller value of $\alpha_{3.ww}$ and $\alpha_{3.wf}$

$\alpha_{3.w} := if\left(\alpha_{3.ww} \le \alpha_{3.wf}, \alpha_{3.ww}, \alpha_{3.wf}\right)$ $\qquad \alpha_{3.w} = 1.121$

- for welded, class 4 cross-sections: $\alpha_{4.w} := \dfrac{W_{eff}}{W_{el}}$ $\qquad \alpha_{4.w} = 1$

$class_y = 3$

$\alpha_y := if\left(class_y > 2, if\left(class_y > 3, \alpha_{4.w}, \alpha_{3.w}\right), \alpha_{1.2.w}\right)$ $\qquad \alpha_y = 1.121$

Design moment of resistance of the cross section $M_{c,Rd}$

[1] (5.14) $M_{y.Rd} := \dfrac{f_o \cdot \alpha_y \cdot W_{el}}{\gamma_{M1}}$ $\qquad M_{y.Rd} = 122.5 \, kNm$

6.2.8 Design resistance, z-z-axis bending

Cross section class $\qquad class_z = 3$

Gross cross section: $I_z := 2 \cdot \dfrac{t_f \cdot b^3}{12}$ $\qquad I_z = 1.092 \cdot 10^{7} \, mm^{4}$

Effective cross section: $I_{z.ef} := 2 \cdot \dfrac{t_{f.ef} \cdot b^3}{12}$ $\qquad I_{z.ef} = 1.092 \cdot 10^{7} \, mm^{4}$

Section moduli: $W_z := \dfrac{I_z \cdot 2}{b}$ $\qquad W_{z.ef} := \dfrac{I_{z.ef} \cdot 2}{b}$

Shape factor: $\alpha_z := \frac{W_z}{W_{z.ef}}$ $\qquad \alpha_z = 1$

Bending resistance: $M_{z.Rd} := \frac{f_o \cdot \alpha_z \cdot W_z}{\gamma_{M1}}$ $\qquad M_{z.Rd} = 32.272 \circ kNm$

6.2.9 Axial force resistance, y-y buckling

[1] 5.8.4 Cross section area of gross cross section A_{gr}

$$A_{gr} := b \cdot h - (b - t_w) \cdot (h - 2 \cdot t_f) \qquad A_{gr} = 6.296 \cdot 10^3 \circ mm^2$$

Cross section area of effective cross section A_{ef}

$$A_{ef} := A_{gr} - 2 \cdot b_f \cdot (t_f - t_{f.ef}) - b_w \cdot (t_w - t_{w.ef}) \qquad A_{ef} = 6.268 \cdot 10^3 \circ mm^2$$

($t_f = 16 \circ mm$ $\quad t_w = 7 \circ mm$ $\quad 2 \cdot b_2 = 143 \circ mm$ $\quad t_{w.ef} = 6.825 \circ mm$ $\quad t_{w.ef} = 6.825 \circ mm$)

Effective cross section factor $\eta := \frac{A_{ef}}{A_{gr}}$ $\qquad \eta = 0.996$

Second moment of area of gross cross section I_y:

$$I_y := \frac{2}{12} \cdot b \cdot t_f^3 + 2 \cdot b \cdot t_f \cdot \left(\frac{h - t_f}{2}\right)^2 + \frac{1}{12} \cdot (h - 2 \cdot t_f)^3 \cdot t_w$$

[1] Table 5.7 Buckling length factor $K_y := 1.5$ $\quad L_1 = 3 \circ m$ $\quad l_{yc} := K_y \cdot L_1$ $\quad l_{yc} = 4.5\ \mathrm{m}$

Case 5.
See also
6.2.11 below

Buckling load $N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{l_{yc}^2}$ $\qquad N_{cr} = 1.577 \cdot 10^3 \circ kN$

[1] 5.8.4.1 Slenderness parameter $\lambda_y := \sqrt{\frac{A_{gr} \cdot \eta \cdot f_o}{N_{cr}}}$ $\qquad \lambda_y = 1.017$

[1] Table 5.6 $\alpha := if(heat\_treated = 1, 0.2, 0.32)$ $\qquad \alpha = 0.2$

$\lambda_o := if(heat\_treated = 1, 0.1, 0)$ $\qquad \lambda_o = 0.1$

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda_y - \lambda_o) + \lambda_y^2\right]$ $\qquad \phi = 1.109$

$\chi_y := \frac{1}{\phi + \sqrt{\phi^2 - \lambda_y^2}}$ $\qquad \chi_y = 0.645$

[1] Table 5.5 Symmetric profile $\qquad k_1 := 1$

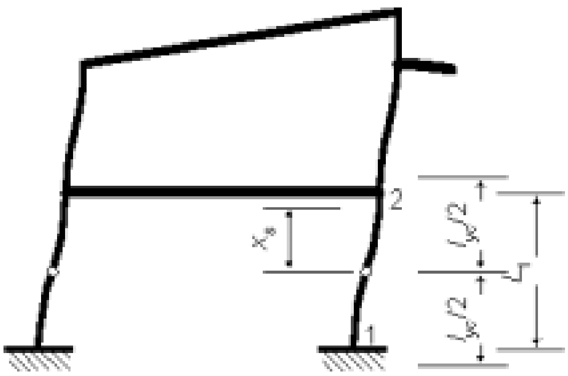
[1] Table 5.5 No longitudinal welds $\qquad k_2 := 1$

Axial force resistance $N_{y.Rd} := \chi_y \cdot \eta \cdot k_1 \cdot k_2 \cdot \frac{f_o}{\gamma_{M1}} \cdot A_{gr}$ $\qquad N_{y.Rd} = 955.708 \circ kN$

6.2.10 Axial force resistance, z-z axis buckling

[1] Table 5.5 Case 3	Buckling length factor	$K := 1$ $\quad L_1 = 3 \circ m$	$K \cdot L_1 = 3 \circ m$
	Buckling load	$N_{cr} := \dfrac{\pi^2 \cdot E \cdot I_z}{\left(K \cdot L_1\right)^2}$	$N_{cr} = 838.5 \circ kN$
[1] 5.8.4.1	Slenderness factor	$\lambda_z := \sqrt{\dfrac{A_{gr} \cdot \eta \cdot f_o}{N_{cr}}}$	$\lambda_z = 1.394$
[1] Table 5.6	$\alpha := if(heat\_treated = 1, 0.2, 0.32)$		$\alpha = 0.2$
	$\lambda_o := if(heat\_treated = 1, 0.1, 0)$		$\lambda_o = 0.1$
[1] 5.8.4.1	$\phi := 0.5 \cdot \left[1 + \alpha \cdot \left(\lambda_z - \lambda_o\right) + \lambda_z^2\right]$		$\phi = 1.601$
	$\chi_z := \dfrac{1}{\phi + \sqrt{\phi^2 - \lambda_z^2}}$		$\chi_z = 0.419$
[1] Table 5.5	Symmetric profile		$k_1 := 1$
[1] Table 5.5	No longitudinal welds		$k_2 := 1$
[1] 5.8.4.1	Axial load resistance	$N_{z.Rd} := \chi_z \cdot \eta \cdot k_1 \cdot k_2 \cdot \dfrac{f_o}{\gamma_{M1}} \cdot A_{gr}$	$N_{z.Rd} = 620.192 \circ kN$
(6.2.9)	Compare y-y axis buckling		$N_{y.Rd} = 955.708 \circ kN$
	and without column buckling	$N_{Rd} := \eta \cdot \dfrac{f_o}{\gamma_{M1}} \cdot A_{gr}$	$N_{Rd} = 1.482 \cdot 10^3 \circ kN$

6.2.11 Flexural buckling of beam-column



[1] Table 5.5	Buckling length			
(6.2.9)	$K_y = 1.5$	$l_{yc} := K_y \cdot L_1$		$l_{yc} = 4.5\ \mathrm{m}$
[1] 5.8.4.1	The ends of column part 1-2 is designing	$x_s := \dfrac{L_1}{2}$	$\dfrac{x_s}{l_{yc}} = 0.333$	$x_s = 1.5\ \mathrm{m}$

[1] 5.9.4.5	HAZ reduction factors	$\rho_{haz} = 0.65$
[1] (5.51)	$\omega_0 := \rho_{haz} \cdot \frac{f_u}{\gamma_{M2}} \cdot \frac{\gamma_{M1}}{f_o}$ $\quad \omega_0 := if(\omega_0 > 1, 1, \omega_0)$	$\omega_0 = 0.682$
[1] (5.49)	$\omega_x := \frac{\omega_0}{\chi_y + (1 - \chi_y) \cdot \sin\left(\frac{\pi \cdot x_s}{l_{yc}}\right)}$	$\omega_x = 0.716$

Exponents in interaction formulae

[1] (5.42c)	$\xi_0 := \alpha_y^{\,2}$ $\quad \xi_0 := if(\xi_0 < 1, 1, \xi_0)$	$\xi_0 = 1.258$
[1] 5.9.4.2	$\xi_{yc} := \xi_0 \cdot \chi_y$ $\quad \xi_{yc} := if(\xi_{yc} < 0.8, 0.8, \xi_{yc})$	$\xi_{yc} = 0.811$

Flexural buckling check

Bending moment $\quad M_{y.Ed} := M_{1.Ed} \quad\quad M_{y.Ed} = 20.7 \cdot kNm$

[1] 5.4.4

$$U_y := \left(\frac{N_{Ed}}{\chi_z \cdot \varphi_x \cdot N_{Rd}}\right)^{\xi_{yc}} + \frac{M_{y.Ed}}{\omega_0 \cdot M_{y.Rd}} \qquad U_y = 0.952$$

or with simplified exponents

$$U_{ys} := \left(\frac{N_{Ed}}{\chi_z \cdot \varphi_x \cdot N_{Rd}}\right)^{0.8} + \left(\frac{M_{y.Ed}}{\omega_0 \cdot M_{y.Rd}}\right)^{1.0} \qquad U_{ys} = 0.955$$

6.2.12 Lateral-torsional buckling of beam-column

[1] 5.9.4.3

[1] Figure J.2	Varping constant:	$I_w := \frac{(h - t_f)^2 \cdot I_z}{4}$	$I_w = 9.245 \cdot 10^{10} \cdot mm^6$
	Torsional constant:	$I_t := \frac{2 \cdot b \cdot t_f^{\,3} + h \cdot t_w^{\,3}}{3}$	$I_t = 4.598 \cdot 10^5 \cdot mm^4$
		$L := L_1 \quad W_y := W_{el}$	$W_y = 4.621 \cdot 10^5 \cdot mm^3$
[1] H.1.2	Moment relation	$\psi := \frac{M_{2.Ed}}{M_{1.Ed}}$	$\psi = 0.147$
[1] H.1.2(6)	C_1 - constant	$C_1 := 1.88 - 1.4 \cdot \psi + 0.52 \cdot \psi^2$	$C_1 = 1.686$

Shear modulus $G = 2.7 \cdot 10^4 \circ MPa$

[1] H.1.3(3) $M_{cr} := \frac{C_1 \cdot \pi^2 \cdot E \cdot I_z}{L^2} \cdot \sqrt{\frac{I_w}{I_z} + \frac{L^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z}}$ $M_{cr} = 215.593 \circ kN \cdot m$

[1] 5.6.6.3(3) $\lambda_{LT} := \sqrt{\frac{\alpha_y \cdot W_y \cdot f_o}{M_{cr}}}$ $\lambda_{LT} = 0.791$

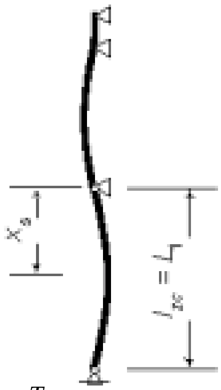
[1] 5.6.6.3(2) $\alpha_{LT} := if(class_z > 2, 0.2, 0.1)$ $\alpha_{LT} = 0.2$

$\lambda_{0LT} := if(class_z > 2, 0.4, 0.6)$ $\lambda_{0LT} = 0.4$

[1] 5.6.6.3(1) $\phi_{LT} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{LT} - \lambda_{0LT}) + \lambda_{LT}^2\right]$ $\phi_{LT} = 0.852$

$\chi_{LT} := \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}}$ $\chi_{LT} = 0.856$

Check sections



$l_{zc} := L_1$

$i := 1 .. 7$

$x_{s_i} := \frac{i-2}{10} \cdot l_{zc}$ $x_{s_1} := 0 \cdot m$ $x_{s_2} := b_{haz}$ $\frac{x_s^T}{l_{zc}} = (0 \;\; 0.01 \;\; 0.1 \;\; 0.2 \;\; 0.3 \;\; 0.4 \;\; 0.5)$

HAZ reduction factors (ω_0 = 1 except at column ends with cross welds)

[1] (5.51) $\omega_0 := \overrightarrow{\left(\rho_{haz} \cdot \frac{f_u}{\gamma_{M2}} \cdot \frac{\gamma_{M1}}{f_o}\right)}$ $\omega_{0_i} := if(\omega_{0_i} > 1, 1, \omega_{0_i})$

Weld at section i = 0 (column end) $\omega_{0_i} := if(i > 1, 1, \omega_{0_i})$

$\omega_0^T = (0.682 \;\; 1 \;\; 1 \;\; 1 \;\; 1 \;\; 1 \;\; 1)$

[1] (5.49)
or (5.52) $\omega_x := \overrightarrow{\frac{\omega_0}{\chi_z + (1-\chi_z) \cdot \sin\left(\frac{\pi \cdot x_s}{l_{zc}}\right)}}$ $\omega_x^T = (1.63 \;\; 2.29 \;\; 1.67 \;\; 1.32 \;\; 1.12 \;\; 1.03 \;\; 1)$

[1] (5.50)
or (5.53) $\omega_{xLT} := \overrightarrow{\frac{\omega_0}{\chi_{LT} + (1-\chi_{LT}) \cdot \sin\left(\frac{\pi \cdot x_s}{l_{zc}}\right)}}$ $\omega_{xLT}^T = (0.8 \;\; 1.16 \;\; 1.11 \;\; 1.06 \;\; 1.03 \;\; 1.01 \;\; 1)$

[1] (5.42a)	$\eta_0 := \alpha_z^2 \cdot \alpha_y^2$	$\eta_0 := if(\eta_0 < 1, 1, if(\eta_0 > 2, 2, \eta_0))$		$\eta_0 = 1.258$
[1] (5.42b)	$\gamma_0 := \alpha_z^2$	$\gamma_0 := if(\gamma_0 < 1, 1, if(\gamma_0 > 2, 2, \gamma_0))$		$\gamma_0 = 1$
[1] (5.42c)	$\xi_0 := \alpha_y^2$	$\xi_0 := if(\xi_0 < 1, 1, \xi_0)$		$\xi_0 = 1.258$
[1] 5.9.4.3	$\eta_c := \eta_0 \cdot \chi_z$	$\eta_c := if(\eta_c < 0.8, 0.8, \eta_c)$	$\chi_y = 0.645$	$\eta_c = 0.8$
[1] 5.9.4.3	$\gamma_c := \gamma_0$		$\chi_z = 0.419$	$\gamma_c = 1$
[1] 5.9.4.3	$\xi_{zc} := \xi_0 \cdot \chi_z$	$\xi_{zc} := if(\xi_{zc} < 0.8, 0.8, \xi_{zc})$		$\xi_{zc} = 0.8$

Lateral-torsional buckling check

Bending moment in section x_s $\qquad M_{y.Ed} := M_{1.Ed} - (M_{1.Ed} - M_{2.Ed}) \cdot \frac{x_s}{l_{zc}}$

$$\frac{M_{y.Ed}}{M_{1.Ed}}^T = (1 \quad 0.991 \quad 0.915 \quad 0.829 \quad 0.744 \quad 0.659 \quad 0.573)$$

$M_{z.Ed} := 0 \cdot kNm$

[1] (5.43)

$$U_{LT} := \overrightarrow{\left[\left(\frac{N_{Ed}}{\chi_z \cdot \varphi_x \cdot N_{Rd}}\right)^{\eta_c} + \left(\frac{M_{y.Ed}}{\chi_{LT} \cdot \omega_{xLT} \cdot M_{y.Rd}}\right)^{\gamma_c} + \left(\frac{M_{z.Ed}}{\omega_0 \cdot M_{z.Rd}}\right)^{\xi_{zc}}\right]}$$

$$U_{LT}^T = (0.614 \quad 0.448 \quad 0.522 \quad 0.589 \quad 0.636 \quad 0.658 \quad 0.655)$$

or with simplified exponents

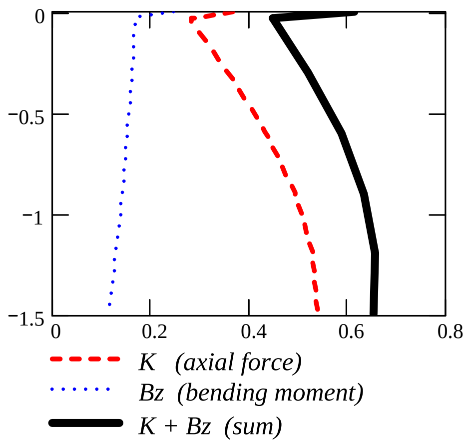
$$U_{LTs} := \overrightarrow{\left[\left(\frac{N_{Ed}}{\chi_z \cdot \varphi_x \cdot N_{Rd}}\right)^{0.8} + \left(\frac{M_{y.Ed}}{\chi_{LT} \cdot \omega_{xLT} \cdot M_{y.Rd}}\right)^{1} + \left(\frac{M_{z.Ed}}{\omega_0 \cdot M_{z.Rd}}\right)^{0.8}\right]}$$

$$U_{LTs}^T = (0.614 \quad 0.448 \quad 0.522 \quad 0.589 \quad 0.636 \quad 0.658 \quad 0.655)$$

Max utilisation, lateral-torsional buckling $\qquad U_{z.max} := max(U_{LT}) \qquad U_{z.max} = 0.658$

Compare utilisation, flexural buckling $\qquad U_y = 0.952$

$$K := \overrightarrow{\left(\frac{N_{Ed}}{\chi_z \cdot \varphi_x \cdot N_{Rd}}\right)^{\eta_c}} \qquad B_z := \overrightarrow{\left(\frac{M_{y.Ed}}{\chi_{LT} \cdot \omega_{xLT} \cdot M_{y.Rd}}\right)^{\gamma_c}}$$



Section 2, HAZ
Section 2, no HAZ

Middle of part 1-2

6.2.13 Design moment in column base

Design section $x_s := \frac{L_1}{2}$ $l_{yc} = 4.5\ \mathrm{m}$ $x_s = 1.5 \cdot m$

Second order bending moment $\Delta M := \frac{N_{Ed} \cdot W_y}{A_{ef}} \cdot \left(\frac{1}{\chi_y} - 1\right) \cdot \sin\left(\frac{\pi \cdot x_s}{l_{yc}}\right)$ $\Delta M = 10.12 \cdot kNm$

Design moment at column base $M_{D.base} := \left| M_{2.Ed} \right| + \Delta M$ $M_{D.base} = 13.2 \cdot kNm$

Axial force corresponding to $M_{D.base}$ (+ = compression) $N_{D.corre} := N_{Ed}$ $N_{D.corre} = 288 \cdot kN$

Minimum axial force, LC4 $N_{D.max} := N_{LC4_1}$ $N_{D.max} = -11.7 \cdot kN$

Corresponding moment $M_{D.corre} := M_{LC4_1}$ $M_{D.corre} = 25.4 \cdot kNm$

The shear force is small why the first order moments are used to calculate V

Load case 1 $V := \left(M_{LC1_2} - M_{LC1_1}\right) \cdot \frac{1}{L}$ $V = -5.89 \cdot kN$

Load case 4 $V := \left(M_{LC4_2} - M_{LC4_1}\right) \cdot \frac{1}{L}$ $V = -16.1 \cdot kN$

6.2.14 Deflections

To calculate the fictive second moment of area I_{fic}, the bending moment in the serviceability limit state is supposed to be half the maximum bending moment at the ultimate limit state.

[1] 4.2.4 $\sigma_{gr} := \frac{0.5 \cdot M_{1.Ed}}{I_{gr}} \cdot \frac{h}{2}$ $I_{gr} = 4.621 \cdot 10^7 \cdot mm^4$ $\sigma_{gr} = 22 \cdot MPa$

Allowing for a reduced stress level I_{fic} may be used constant along the beam.

[1] (4.2) $I_{fic} := I_{gr} - \frac{\sigma_{gr}}{f_o} \cdot (I_{gr} - I_{eff})$ $I_{fic} = 4.621 \cdot 10^7 \cdot mm^4$

$I := if(class_y = 4, I_{fic}, I_{gr})$ $class_y = 3$ $I = 4.621 \cdot 10^7 \cdot mm^4$

$\delta_1 := 0 \cdot mm$ $\delta_1 = 0 \cdot mm$

$\delta_2 := 4.7 \cdot mm$ $\delta_2 = 4.7 \cdot mm$

Pre-camber $\delta_0 := 0 \cdot mm$

$\delta_{max} := \delta_1 + \delta_2 - \delta_0$ $\delta_{max} = 4.7 \cdot mm$

Limit horizontal deformation for building frame with $h_{building} := 6.5 \cdot m$

$\delta_{limit} := \frac{h_{building}}{300}$ $\delta_{limit} = 22 \cdot mm$

$Check := if(\delta_{max} \le \delta_{limit}, \text{"OK!"}, \text{"Not OK!"})$ $Check = \text{"OK!"}$

6.2.15 Summary

$M_{1.Ed} = 21 \cdot kNm$ $M_{y.Rd} = 122 \cdot kNm$ $\omega_{\rho 1} = 0.682$ $\frac{M_{1.Ed}}{\omega_{\rho 1} M_{y.Rd}} = 0.248$

$N_{Ed} = 288 \cdot kN$ $N_{y.Rd} = 955.7 \cdot kN$ $\omega_{x1} = 1.629$ $\chi_y = 0.645$ $\frac{N_{Ed}}{\chi_y \cdot \omega_{x_1} \cdot N_{y.Rd}} = 0.287$

Utilisation, flexural buckling - HAZ at column base $U_y = 0.952$

Utilisation, lateral-torsional buckling $U_{z.max} = 0.658$

$\delta_{limit} = 21.7 \cdot mm$ $\delta_{max} = 4.7 \cdot mm$ $\frac{\delta_{max}}{\delta_{limit}} = 0.217$

Effective second moment of area $I_{fic} = 4.621 \cdot 10^7 \cdot mm^4$

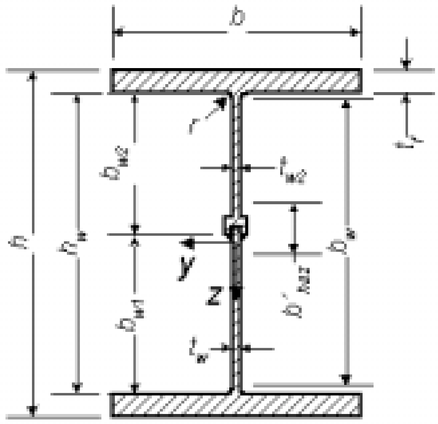
Cross section $h = 200 \cdot mm$ $b = 160 \cdot mm$ $t_w = 7 \cdot mm$ $t_f = 16 \cdot mm$ $A_{gr} = 6.296 \cdot 10^3 \cdot mm^2$

6.3 Column C

Comment: To reduce the extent of this example the check of column C is left out. It is given the same cross section as column A.

6.4 Floor Beam D

6.4.1 Dimensions and material properties



Flange height: $h := 300 \cdot mm$

Flange depth: $b := 120 \cdot mm$

Web thickness: $t_w := 4 \cdot mm$ $\quad t_{w2} := t_w$

Flange thickness: $t_f := 12 \cdot mm$

Flange web part: $b_{w1} := \left(h - 2 \cdot t_f\right) \cdot 0.5$

Overall length: $L := 6 \cdot m$

Distance between joists: $c_p := 0.6 \cdot m$

Depth of web plate:

$h_w := h - 2 \cdot t_f$ $\qquad h_w = 276 \circ mm$

[1] Table 3.2b Alloy: **EN AW-6082 T6** EP/O t > 5 mm

$f_{0.2} := 260 \cdot MPa$ $\qquad f_u := 310 \cdot MPa$

[1] (5.4), (5.5) $f_o := f_{0.2}$ $\qquad f_a := f_u$

[1] (5.6) $f_v := \dfrac{f_o}{\sqrt{3}}$ $\qquad f_v = 150 \circ \dfrac{newton}{mm^2}$ $\qquad E := 70000 \cdot \dfrac{newton}{mm^2}$ $\qquad G := 27000 \cdot \dfrac{newton}{mm^2}$

Partial safety factors: $\gamma_{M1} \equiv 1.10$ $\qquad \gamma_{M2} \equiv 1.25$

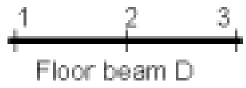
Radius: $r := 3 \cdot mm$

Web width: $b_w := h - 2 \cdot t_f - 2 \cdot r$ $\qquad b_w = 270 \circ mm$

S.A.E. units: $kN \equiv 1000 \cdot newton$ $\qquad MPa \equiv 1000000 \cdot Pa$ $\qquad MPa = 1 \circ \dfrac{newton}{mm^2}$

$kNm \equiv kN \cdot m$

6.4.2 Internal moments and forces



(5.4.4)	Bending moment in section 2	$M_{Ed} := 86.4 \cdot kNm$
	Shear force at support 3	$V_{Ed} := 91.6 \cdot kN$
	Bending moment at support 3	$M_{1Ed} := -64.4 \cdot kNm$
(3.1, 3.2 and 3.3)	Concentrated load	$Q_{k.floor} := 1.5 \cdot kN$
	Distributed load, characteristic value	
	Permanent load	$q_{p.floor} := 3.85 \cdot kN \cdot m^{-1}$
	Imposed load	$q_{k.floor} := 16.5 \cdot kN \cdot m^{-1}$
	Distributed load, design value in the serviceability limit state	$q_{Ed} := 1.0 \cdot q_{p.floor} + 1.0 \cdot q_{k.floor}$

6.4.3 Classification of the cross section

a) Web

Comment 1: As the flanges belong to a class < 4 (se below) and the cross section is symmetric, no iteration is needed to find the final neutral axis to calculate ψ and g. See [1] Figure 5.17

Comment 2: As the longitudinal weld is close to the neutral axis, the web might have beenclassified as unwelded. However, on the safe side, it is classified as welded.

[1] 5.4.3

$\psi := -1$ $\quad \beta_w := 0.40 \cdot \dfrac{b_w}{t_w}$ $\quad \beta_w = 27$ $\quad \varepsilon := \sqrt{\dfrac{250}{f_o} \cdot \dfrac{newton}{mm^2}}$

[1] Tab. 5.1
Heat treated, *welded web*

$\beta_{1w} := 9 \cdot \varepsilon$ $\quad \beta_{2w} := 13 \cdot \varepsilon$ $\quad \beta_{3w} := 18 \cdot \varepsilon$

$\beta_{1w} = 8.825$ $\quad \beta_{2w} = 12.748$ $\quad \beta_{3w} = 17.65$

$class_w := if\left(\beta_w \geq \beta_{1w}, if\left(\beta_w > \beta_{2w}, if\left(\beta_w > \beta_{3w}, 4, 3\right), 2\right), 1\right)$ $\qquad class_w = 4$

[1] 5.4.5 Local buckling

$$\rho_{cw} := if\left[\frac{\beta_w}{\varepsilon} \leq 18, 1.0, \frac{29}{\left(\frac{\beta_w}{\varepsilon}\right)} - \frac{198}{\left(\frac{\beta_w}{\varepsilon}\right)^2}\right] \qquad \rho_{cw} = 0.792$$

$$t_{w.ef.b} := if\left(class_w \geq 4, t_w \cdot \rho_{cw}, t_w\right) \qquad t_{w.ef.b} = 3.2 \circ mm$$

[1] 5.4.3 **b) Flanges**

[1] (5.7.),(5.8.) $\psi := 1$ $\quad g := 1$ $\quad \beta_f := g \cdot \dfrac{b - t_w - 2 \cdot r}{2 \cdot t_f}$ $\quad \beta_f = 4.583$

Heat treated, *unwelded flange*

[1] Tab. 5.1 $\beta_{1f} := 3 \cdot \varepsilon$ $\quad \beta_{2f} := 4.5 \cdot \varepsilon$ $\quad \beta_{3f} := 6 \cdot \varepsilon$

$\beta_{1f} = 2.942$ $\quad \beta_{2f} = 4.413$ $\quad \beta_{3f} = 5.883$

$class_f := if\left(\beta_f > \beta_{1f}, if\left(\beta_f > \beta_{2f}, if\left(\beta_f > \beta_{3f}, 4, 3\right), 2\right), 1\right)$ $\quad class_f = 3$

[1] 5.4.5 Local buckling:

$$\rho_{cf} := if\left[\frac{\beta_f}{\varepsilon} \le 6, 1.0, \frac{10}{\left(\frac{\beta_f}{\varepsilon}\right)} - \frac{24}{\left(\frac{\beta_f}{\varepsilon}\right)^2}\right] \qquad \rho_{cf} = 1$$

$$t_{f.ef} := if\left(class_f \ge 4, t_f \cdot \rho_{cf}, t_f\right) \qquad t_{f.ef} = 12.0 \circ mm$$

Classification of the total cross-section:

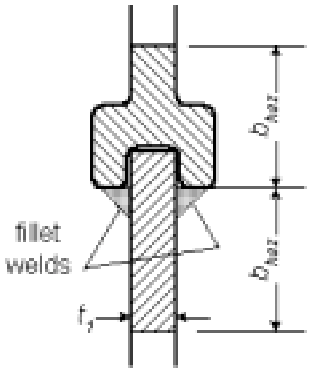
$$class := if\left(class_f > class_w, class_f, class_w\right) \qquad class = 4$$

c) Flange induced buckling

[1] 5.12.9 Elastic moment resistance utilised $\quad k := 0.55$ $\quad f_{of} := f_o$

[1] (5.115) $check := if\left(\dfrac{h_w}{t_w} < \dfrac{k \cdot E}{f_{of}} \cdot \sqrt{\dfrac{h_w \cdot t_w}{b \cdot t_f}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\quad check = \text{"OK!"}$

6.4.4 Welds



[1] 5.5

[1] Tab. 5.2 1. HAZ softening factor

$\rho_{haz} := 0.65$

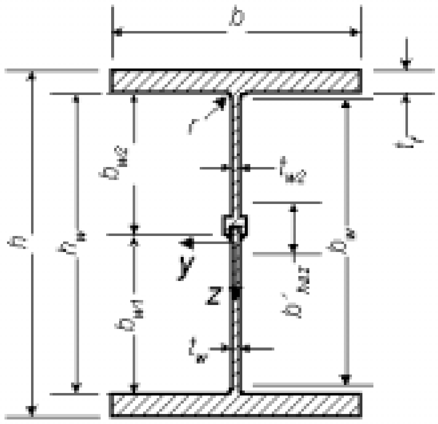
[1] Fig. 5.6 2. Extent of HAZ (MIG-weld) $\quad t_1 := t_w$

$$b_{haz} := if\left(t_1 > 6 \cdot mm, if\left(t_1 > 12 \cdot mm, if\left(t_1 > 25 \cdot mm, 40 \cdot mm, 35 \cdot mm\right), 30 \cdot mm\right), 20 \cdot mm\right)$$

$b_{haz} = 20 \circ mm$ $\quad b'_{haz} := 2 \cdot b_{haz}$ $\quad b'_{haz} = 40 \circ mm$

6.4.5 Bending resistance

[1] 5.6.1 Elastic modulus of the gross cross section W_{gr}:



$A_{gr} := 2 \cdot b \cdot t_f + (h - 2 \cdot t_f) \cdot t_w \qquad A_{gr} = 3.984 \cdot 10^3 \circ mm^2$

$I_{gr} := \frac{1}{12} \cdot \left[b \cdot h^3 - (b - t_w) \cdot (h - 2 \cdot t_f)^3 \right]$

$I_{gr} = 6.676 \cdot 10^7 \circ mm^4$

$W_{el} := \frac{I_{gr} \cdot 2}{h} \qquad W_{el} = 4.451 \cdot 10^5 \circ mm^3$

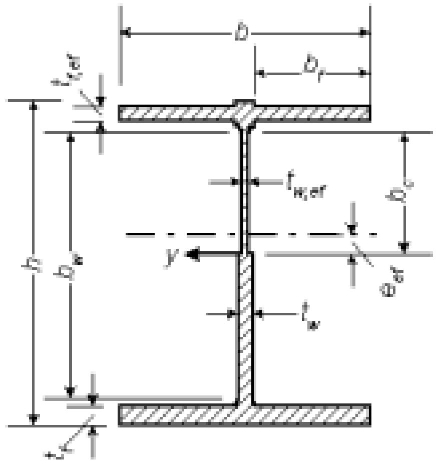
As the weld is close to the centroidal axis there is no reduction due to HAZ

$W_{ele} := W_{el} \qquad W_{ele} = 4.451 \cdot 10^5 \circ mm^3$

Plastic modulus of cross section W_{ple}:

$W_{ple} := \frac{1}{4} \cdot \left[b \cdot h^2 - (b - t_w) \cdot (h - 2 \cdot t_f)^2 \right] \qquad W_{ple} = 4.909 \cdot 10^5 \circ mm^3$

Elastic modulus of the effective cross section W_{effe}:



$t_f = 12 \circ mm \qquad t_{f.ef} = 12 \circ mm$

As $t_{f.ef} = t_f$ then $\quad b_c := \frac{b_w}{2} \qquad b_c = 135 \circ mm$

$t_w = 4 \circ mm$

$t_{w.ef.b} = 3.2 \circ mm$

$b_f := 0.5 \cdot (b - t_w - 2 \cdot r) \qquad b_f = 55 \circ mm$

$A_{effe} := A_{gr} - 2 \cdot b_f \cdot (t_f - t_{f.ef}) - b_c \cdot (t_w - t_{w.ef.b}) \qquad A_{effe} = 3.872 \cdot 10^3 \circ mm^2$

Shift of gravity centre:

$e_{ef} := \left[2 \cdot b_f \cdot (t_f - t_{f.ef}) \cdot \left(\frac{h}{2} - \frac{t_f}{2} \right) + \frac{b_c^{\,2}}{2} \cdot (t_w - t_{w.ef.b}) \right] \cdot \frac{1}{A_{effe}} \qquad e_{ef} = 1.958 \circ mm$

Second moment of area with respect to centre of gross cross section:

$I_{effe} := I_{gr} - 2 \cdot b_f \cdot (t_f - t_{f.ef}) \cdot \left(\frac{h}{2} - \frac{t_f}{2} \right)^2 - \frac{b_c^{\,3}}{3} \cdot (t_w - t_{w.ef.b}) \qquad I_{effe} = 6.608 \cdot 10^7 \circ mm^4$

Second moment of area with respect to centre of effective gross section:

$$I_{effe} := I_{effe} - e_{ef}^{2} \cdot A_{effe} \qquad I_{effe} = 6.607 \cdot 10^{7} \circ mm^{4}$$

$$W_{effe} := \frac{I_{effe}}{\frac{h}{2} + e_{ef}} \qquad W_{effe} = 4.348 \cdot 10^{5} \circ mm^{3}$$

[1] Tab. 5.3 Shape factor α

- for welded, class 1 or 2 cross-sections:

$$\alpha_{1.2.w} := \frac{W_{ple}}{W_{el}} \qquad \alpha_{1.2.w} = 1.103$$

- for welded, class 3 cross-sections:

[1] (5.16) $$\alpha_{3.ww} := \left[\frac{W_{ele}}{W_{el}} + \left(\frac{\beta_{3w} - \beta_{w}}{\beta_{3w} - \beta_{2w}}\right) \cdot \left(\frac{W_{ple} - W_{ele}}{W_{el}}\right)\right] \qquad \alpha_{3.ww} = 0.804$$

[1] (5.16) $$\alpha_{3.wf} := \left[\frac{W_{ele}}{W_{el}} + \left(\frac{\beta_{3f} - \beta_{f}}{\beta_{3f} - \beta_{2f}}\right) \cdot \left(\frac{W_{ple} - W_{ele}}{W_{el}}\right)\right] \qquad \alpha_{3.wf} = 1.091$$

β, β_2, β_3 are the slenderness parameter and the limiting values for the most critical element in the cross-section, so it is the smaller value of $\alpha_{3.ww}$ and $\alpha_{3.wf}$

$$\alpha_{3.w} := if\left(\alpha_{3.ww} \le \alpha_{3.wf}, \alpha_{3.ww}, \alpha_{3.wf}\right) \qquad \alpha_{3.w} = 0.804$$

- for welded, class 4 cross-sections: $\alpha_{4.w} := \frac{W_{effe}}{W_{el}}$ $\qquad \alpha_{4.w} = 0.977$

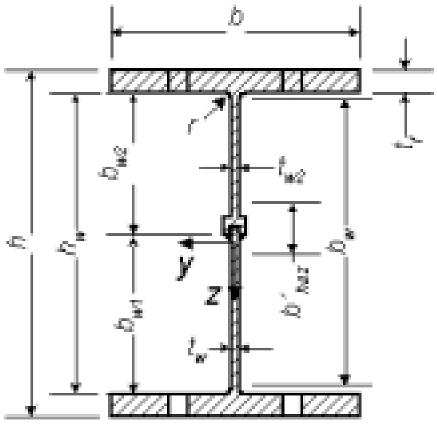
$class = 4$

$$\alpha := if\left(class > 2, if\left(class > 3, \alpha_{4.w}, \alpha_{3.w}\right), \alpha_{1.2.w}\right) \qquad \alpha = 0.977$$

Design moment of resistance of the cross section $M_{c,Rd}$

[1] (5.14) $$M_{c.Rd} := \frac{f_{o} \cdot \alpha \cdot W_{el}}{\gamma_{M1}} \qquad M_{c.Rd} = 102.8 \circ kNm$$

6.4.6 Bending resistance in a section with holes



[1] (5.13) The resistance is based on the effective elastic modulus of the net section W_{net} times the ultimate strength.

Allowance for bolt holes on the tension flange is made by reducing the width of the flanges with two bolt diameters d_b.

$d_b := 12 \cdot mm$

Holes are supposed in both flanges

$$I_{net} := I_{gr} - 2 \cdot d_b \cdot t_f \cdot \left(\frac{h - t_f}{2}\right)^2 \cdot 2 - 2 \cdot d_b \cdot \frac{t_f^{\,3}}{12} \cdot 2 \qquad I_{net} = 5.481 \cdot 10^7 \, mm^4$$

No allowance for HAZ

$$W_{net} := \frac{2 \cdot I_{net}}{h} \qquad \frac{f_o}{\gamma_{M1}} = 236 \, MPa \qquad W_{net} = 3.654 \cdot 10^5 \, mm^3$$

[1] (5.13) $$M_{a.Rd} := \frac{f_u \cdot W_{net}}{\gamma_{M2}} \qquad \frac{f_u}{\gamma_{M2}} = 248 \, MPa \qquad M_{a.Rd} = 90.6 \, kNm$$

The bending resistance is the lesser of $M_{a.Rd}$ and $M_{c.Rd}$

$$M_{Rd} := if\left(M_{a.Rd} < M_{c.Rd}, M_{a.Rd}, M_{c.Rd}\right) \qquad M_{Rd} = 90.6 \, kNm$$

6.4.7 Shear force resistance

[1] 5.12.4 Design shear resistance $V_{w.Rd}$ for web. A rigid end post is assumed.

[1] (5.93) $\lambda_w := 0.35 \cdot \frac{h_w}{t_w} \cdot \sqrt{\frac{f_o}{E}}$ $h_w = 276 \cdot mm$ $\lambda_w = 1.472$

$\eta := 0.4 + 0.2 \cdot \frac{f_u}{f_o}$ $\eta = 0.638$

[1] Tab. 5.12 $\rho_v := if\left(\lambda_w > \frac{0.48}{\eta}, if\left(\lambda_w \geq 0.949, \frac{0.48}{\lambda_w}, \frac{0.48}{\lambda_w}\right), \eta\right)$ $\rho_v = 0.326$

$\rho_v := if\left(\lambda_w > 0.949, \frac{1.32}{1.66 + \lambda_w}, \rho_v\right)$ $\rho_v = 0.421$

[1] (5.95) $V_{w.Rd} := \rho_v \cdot t_w \cdot h_w \cdot \frac{f_o}{\gamma_{M1}}$ $V_{w.Rd} = 110 \cdot kN$

Shear resistance contribution $V_{f.Rd}$ of the flanges is small and is omitted.

$V_{Rd} := V_{w.Rd}$ $V_{Rd} = 110 \cdot kN$

6.4.8 Deflections

To calculate the fictive second moment of area I_{fic}, the bending moment in the serviceability limit state is supposed to be half the maximum bending moment at the ultimate limit state.

[1] 4.2.4 $\sigma_{gr} := \frac{0.5 \cdot M_{Ed}}{I_{gr}} \cdot \frac{h}{2}$ $I_{gr} = 6.676 \cdot 10^7 \cdot mm^4$ $\sigma_{gr} = 97 \cdot MPa$

Allowing for a reduced stress level I_{fic} may be used constant along the beam.

[1] (4.2) $I_{fic} := I_{gr} - \frac{\sigma_{gr}}{f_o} \cdot \left(I_{gr} - I_{effe}\right)$ $I_{fic} = 6.65 \cdot 10^7 \cdot mm^4$

$I := if\left(class = 4, I_{fic}, I_{gr}\right)$ $class = 4$ $I = 6.65 \cdot 10^7 \cdot mm^4$

(Coefficient 0.45 from FE-calculation)

$\delta_1 := 0.45 \cdot \frac{5 \cdot q_{p.floor} \cdot L^4}{384 \cdot E \cdot I}$ $q_{p.floor} = 3.85 \cdot kN \cdot m^{-1}$ $\delta_1 = 6.3 \cdot mm$

$\delta_2 := 0.45 \cdot \frac{5 \cdot q_{k.floor} \cdot L^4}{384 \cdot E \cdot I}$ $q_{k.floor} = 16.5 \cdot kN \cdot m^{-1}$ $\delta_2 = 26.9 \cdot mm$

Pre-camber $\delta_0 := 0 \cdot mm$

Load combination 2, imposed load dominant

$\delta_{max} := \delta_1 + 0.5 \cdot \delta_2 - \delta_0$ $\qquad \delta_{max} = 19.7 \circ mm$

$\delta_{limit} := \dfrac{L}{250}$ for beams carrying floors $\qquad \delta_{limit} = 24 \circ mm$

Summary

$M_{Ed} = 86 \circ kNm$ $\qquad M_{Rd} = 90.6 \circ kNm$ $\qquad M_{1Ed} = -64 \circ kNm$ $\qquad \dfrac{M_{Ed}}{M_{Rd}} = 0.953$

$V_{Ed} = 91.6 \circ kN$ $\qquad V_{Rd} = 110 \circ kN$ $\qquad \dfrac{V_{Ed}}{V_{Rd}} = 0.833$

$\delta_{limit} = 24 \circ mm$ $\qquad \delta_{max} = 20 \circ mm$ $\qquad I_{fic} = 6.65 \cdot 10^7 \circ mm^4$

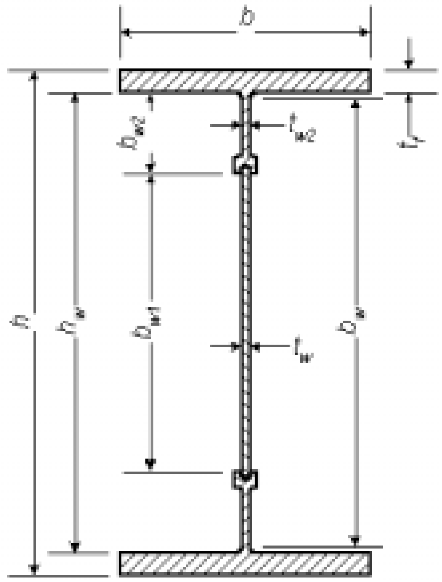
Cross section $\quad h = 300 \circ mm \quad b = 120 \circ mm \quad t_w = 4 \circ mm \quad t_f = 12 \circ mm \quad A_{gr} = 3.984 \cdot 10^3 \circ mm^2$

6.5 Roof Beam E

Comment: To reduce the extent of this example the check of roof beam E is left out. It is given the same cross section as roof beam D.

6.6 Roof Beam F

6.6.1 Dimensions and material properties



Flange height: $h := 570 \cdot mm$

Flange depth: $b := 160 \cdot mm$

Flange web part: $b_{w2} := 50 \cdot mm$

Web thickness: $t_{w2} := 5 \cdot mm$ $t_w := 5 \cdot mm$

Flange thickness: $t_f := 15.4 \cdot mm$

Overall length: $L := 10 \cdot m$

Distance between purlins: $c_p := 1.0 \cdot m$

Width of web plate:

$b_{w1} := h - 2 \cdot b_{w2} - 2 \cdot t_f$ $b_{w1} = 439.2 \circ mm$

[1] Table 3.2b Alloy: **EN AW-6082 T6** EP/O t > 5 mm

$f_{0.2} := 260 \cdot MPa$ $f_u := 310 \cdot MPa$

[1] (5.4), (5.5) $f_o := f_{0.2}$ $f_a := f_u$

[1] (5.6) $f_v := \dfrac{f_o}{\sqrt{3}}$ $f_v = 150 \circ \dfrac{newton}{mm^2}$ $E := 70000 \cdot MPa$ $G := 27000 \cdot MPa$

Partial safety factors: $\gamma_{M1} \equiv 1.10$ $\gamma_{M2} \equiv 1.25$

6.6.2 Internal moments and forces

Bending moment in section 2 $M_{Ed} := 336 \cdot kNm$

Shear force at support 1 $V_{Ed} := 172 \cdot kN$

Bending moment at support 1 $M_{1Ed} := -146 \cdot kNm$

Concentrated load $P_{crane} := 50 \cdot kN$

Distributed loads, characteristic value

Permanent load $q_{p.roof} := 2.75 \cdot kN \cdot m^{-1}$

Imposed load $q_{k.roof} := 4.125 \cdot kN \cdot m^{-1}$

Snow load $q_{snow} := 11 \cdot kN \cdot m^{-1}$

Wind load $q_{w.roof} := -3.85 \cdot kN \cdot m^{-1}$

Web-flange corner radius $r := 5 \cdot mm$

Welds: $a := 4 \cdot mm$

Web height: $b_w := h - 2 \cdot t_f - 2 \cdot r$ $b_w = 529 \circ mm$

S.I. units $kN \equiv 1000 \cdot newton$ $kNm \equiv kN \cdot m$ $MPa \equiv 1000000 \cdot Pa$

6.6.3 Classification of the cross section

a) Web

[1] 5.4.3

$\beta_w := 0.40 \cdot \frac{b_w}{t_w}$ $\qquad \beta_w = 42.336$ $\qquad \varepsilon := \sqrt{\frac{250}{f_o} \cdot \frac{newton}{mm^2}}$

Comment: As the flanges belong to a class < 4 (se below) and the cross section is symmetric, no iteration is needed to find the final neutral axis to calculate ψ and g. See [1] Figure 5.17

[1] Tab. 5.1
Heat treated, *welded web*

$\beta_{1w} := 9 \cdot \varepsilon$ $\qquad \beta_{2w} := 13 \cdot \varepsilon$ $\qquad \beta_{3w} := 18 \cdot \varepsilon$

$\beta_{1w} = 8.825$ $\qquad \beta_{2w} = 12.748$ $\qquad \beta_{3w} = 17.65$

$class_w := if(\beta_w > \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1)$ $\qquad class_w = 4$

[1] 5.4.5 Local buckling

$$\rho_{cw} := if\left[\frac{\beta_w}{\varepsilon} \le 18, 1.0, \frac{29}{\left(\frac{\beta_w}{\varepsilon}\right)} - \frac{198}{\left(\frac{\beta_w}{\varepsilon}\right)^2}\right] \qquad \rho_{cw} = 0.565$$

$t_{w.ef.b} := if(class_w \ge 4, t_w \cdot \rho_{cw}, t_w)$ $\qquad t_{w.ef.b} = 2.8 \cdot mm$

($_b$ = bending)

b) Flange

[1] 5.4.3

$\psi := 1$ $\qquad g := 1$ $\qquad \beta_f := g \cdot \frac{b - t_w - 2 \cdot r}{2 \cdot t_f}$ $\qquad \beta_f = 4.708$

[1] Tab. 5.1
Heat treated, *unwelded flange*

$\beta_{1f} := 3 \cdot \varepsilon$ $\qquad \beta_{2f} := 4.5 \cdot \varepsilon$ $\qquad \beta_{3f} := 6 \cdot \varepsilon$

$\beta_{1f} = 2.942$ $\qquad \beta_{2f} = 4.413$ $\qquad \beta_{3f} = 5.883$

$class_f := if(\beta_f > \beta_{1f}, if(\beta_f > \beta_{2f}, if(\beta_f > \beta_{3f}, 4, 3), 2), 1)$ $\qquad class_f = 3$

[1] 5.4.5 Local buckling:

$$\rho_{cf} := if\left[\frac{\beta_f}{\varepsilon} \le 6, 1.0, \frac{10}{\left(\frac{\beta_f}{\varepsilon}\right)} - \frac{24}{\left(\frac{\beta_f}{\varepsilon}\right)^2}\right] \qquad \rho_{cf} = 1$$

$t_{f.ef} := if(class_f \ge 4, t_f \cdot \rho_{cf}, t_f)$ $\qquad t_{f.ef} = 15.4 \cdot mm$

Classification of the total cross-section:

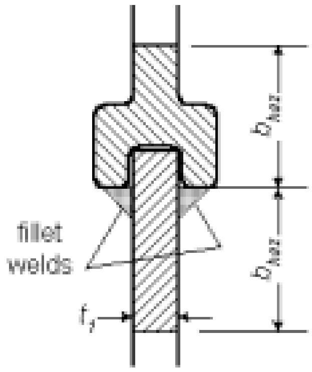
$class := if(class_f > class_w, class_f, class_w)$ $\qquad class = 4$

c) Flange induced buckling

[1] 5.12.9 Elastic moment resistance utilised $k := 0.55$ $f_{of} := f_o$ $h_w := h - 2 \cdot t_f$

[1] (5.115) $Check := if\left(\frac{h_w}{t_w} < \frac{k \cdot E}{f_{of}} \cdot \sqrt{\frac{h_w \cdot t_w}{b \cdot t_f}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $Check = \text{"OK!"}$

6.6.4 Welds



[1] 5.5

[1] Tab. 5.2 1. HAZ softening factor

$\rho_{haz} := 0.65$

[1] Fig. 5.6 2. Extent of HAZ (MIG-weld)

$t_1 := t_w$

$b_{haz} := if(t_1 > 6 \cdot mm, if(t_1 > 12 \cdot mm, if(t_1 > 25 \cdot mm, 40 \cdot mm, 35 \cdot mm), 30 \cdot mm), 20 \cdot mm)$

$b_{haz} = 20 \circ mm$ $b'_{haz} := 2 \cdot b_{haz}$ $b'_{haz} = 40 \circ mm$

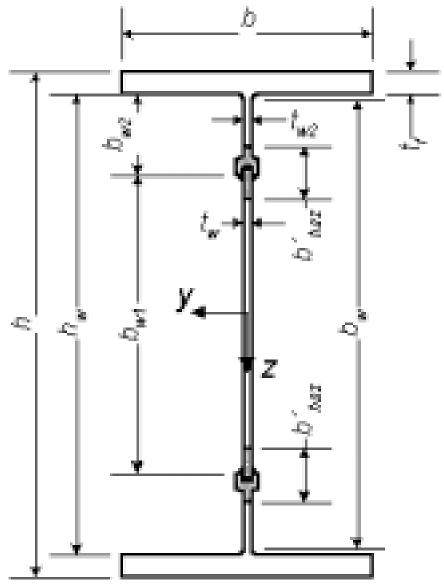
Due to added material: $t_{w..ef.b} := t_w$

$\rho_{haz} := \frac{(b'_{haz} - 2.5 \cdot t_w) \cdot t_w + 2.5 \cdot t_w \cdot 2.5 \cdot t_w}{b'_{haz} \cdot t_w} \cdot \rho_{haz}$ $\rho_{haz} = 0.955$

$\rho'_{haz} := if\left(\rho_{haz} < \frac{t_{w.ef.b}}{t_w}, \rho_{haz}, \frac{t_{w.ef.b}}{t_w}\right)$ $\rho'_{haz} = 0.565$

6.6.5 Bending moment resistance

[1] 5.6.2



Elastic modulus of gross cross section W_{el}:

$A_{gr} := 2 \cdot b \cdot t_f + (h - 2 \cdot t_f) \cdot t_w$ $A_{gr} = 7.624 \cdot 10^3 \circ mm^2$

$I_{gr} := \frac{1}{12} \cdot \left[b \cdot h^3 - (b - t_w) \cdot (h - 2 \cdot t_f)^3\right]$

$I_{gr} = 4.444 \cdot 10^8 \circ mm^4$

$W_{el} := \frac{I_{gr} \cdot 2}{h}$ $W_{el} = 1.559 \cdot 10^6 \circ mm^3$

Plastic modulus

$W_{ple} := \frac{1}{4} \cdot \left[b \cdot h^2 - (b - t_w) \cdot (h - 2 \cdot t_f)^2\right] - b'_{haz} \cdot t_w \cdot (1 - \rho_{haz}) \cdot \frac{b_w}{2}$ $W_{ple} = 1.728 \cdot 10^6 \circ mm^3$

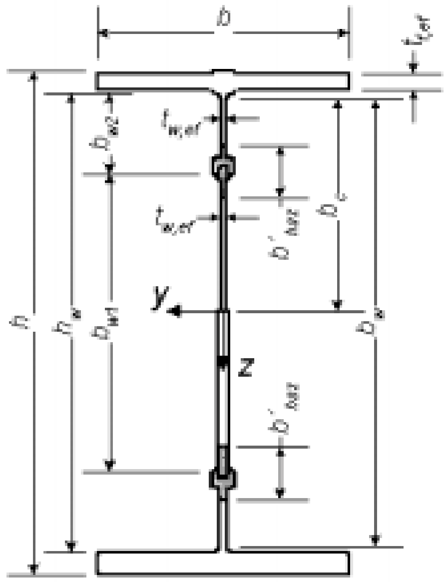
Elastic modulus of the effective cross section W_{effe}:

$t_f = 15.4 \cdot mm$ $\quad t_{f.ef} = 15.4 \cdot mm$

As $t_{f.ef} = t_f$ then $\quad b_c := \dfrac{b_w}{2} \quad b_c = 264.6 \cdot mm$

$t_w = 5 \cdot mm$

$t_{w.ef.b} = 2.8 \cdot mm$



Allowing for local buckling:

$$A_{effe} := A_{gr} - b \cdot \left(t_f - t_{f.ef}\right) - b_c \cdot \left(t_w - t_{w.ef.b}\right)$$

$$A_{effe} = 7.049 \cdot 10^3 \cdot mm^2$$

Allowing for HAZ:

$$A_{effe} := A_{effe} - 2 \cdot b'_{haz} \cdot \left(t_{w.ef.b} - \rho_{haz} \cdot t_w\right)$$

$$A_{effe} = 7.049 \cdot 10^3 \cdot mm^2$$

Shift of gravity centre:

$$e_{ef} := \left[b \cdot \left(t_f - t_{f.ef}\right) \cdot \left(\frac{h}{2} - \frac{t_f}{2}\right) + \frac{b_c^2}{2} \cdot \left(t_w - t_{w.ef.b}\right) + 2 \cdot b'_{haz} \cdot \left(t_{w.ef.b} - \rho_{haz} \cdot t_w\right) \cdot \frac{b_{w1}}{2} \right] \cdot \frac{1}{A_{effe}}$$

$$e_{ef} = 10.79 \cdot mm$$

Second moment of area with respect to centre of gross cross section:

$$I_{effe} := I_{gr} - b \cdot \left(t_f - t_{f.ef}\right) \cdot \left(\frac{h}{2} - \frac{t_f}{2}\right)^2 - \frac{b_c^3}{3} \cdot \left(t_w - t_{w.ef.b}\right) - 2 \cdot b'_{haz} \cdot \left(t_{w.ef.b} - \rho_{haz} \cdot t_w\right) \cdot \left(\frac{b_{w1}}{2}\right)^2$$

$$I_{effe} = 4.309 \cdot 10^8 \cdot mm^4$$

Second moment of area with respect to centre of effective cross section:

$$I_{effe} := I_{effe} - e_{ef}^2 \cdot A_{effe} \qquad I_{effe} = 4.301 \cdot 10^8 \cdot mm^4$$

$$W_{effe} := \frac{I_{effe}}{\frac{h}{2} + e_{ef}} \qquad W_{ele} := W_{effe} \qquad W_{effe} = 1.454 \cdot 10^6 \cdot mm^3$$

$$W_{ele} = 1.454 \cdot 10^6 \cdot mm^3$$

Plastic modulus of the welded section W_{ple}: $\quad \rho_{haz} = 0.955$

$$W_{ple} := 2 \cdot b \cdot t_f \cdot \frac{h - t_f}{2} + \frac{\left(h - 2 \cdot t_f\right)^2}{4} \cdot t_w - b'_{haz} \cdot t_w \cdot \left(1 - \rho_{haz}\right) \cdot \left(\frac{b_{w1}}{2}\right) \qquad W_{ple} = 1.728 \cdot 10^6 \cdot mm^3$$

[1] Tab. 5.3 Shape factor α

- for welded, class 1 or 2 cross-sections:

$$\alpha_{1.2.w} := \frac{W_{ple}}{W_{el}} \qquad \alpha_{1.2.w} = 1.108$$

- for welded, class 3 cross-sections:

[1] (5.16)
$$\alpha_{3.ww} := \left[\frac{W_{ele}}{W_{el}} + \left(\frac{\beta_{3w} - \beta_w}{\beta_{3w} - \beta_{2w}}\right)\cdot\left(\frac{W_{ple} - W_{ele}}{W_{el}}\right)\right] \qquad \alpha_{3.ww} = 0.048$$

[1] (5.16)
$$\alpha_{3.wf} := \left[\frac{W_{ele}}{W_{el}} + \left(\frac{\beta_{3f} - \beta_f}{\beta_{3f} - \beta_{2f}}\right)\cdot\left(\frac{W_{ple} - W_{ele}}{W_{el}}\right)\right] \qquad \alpha_{3.wf} = 1.073$$

β, β_2, β_3 are the slenderness parameter and the limiting values for the most critical element in the cross-section, so it is the smaller value of $\alpha_{3.ww}$ and $\alpha_{3.wf}$

$$\alpha_{3.w} := if\left(\alpha_{3.ww} \le \alpha_{3.wf}, \alpha_{3.ww}, \alpha_{3.wf}\right) \qquad \alpha_{3.w} = 0.048$$

- for welded, class 4 cross-sections: $\alpha_{4.w} := \dfrac{W_{effe}}{W_{el}}$ $\qquad \alpha_{4.w} = 0.933$

$class = 4$

$$\alpha := if\left(class > 2, if\left(class > 3, \alpha_{4.w}, \alpha_{3.w}\right), \alpha_{1.2.w}\right) \qquad \alpha = 0.933$$

Design moment of resistance of the cross section $M_{c,Rd}$

[1] (5.14)
$$M_{c.Rd} := \frac{f_o \cdot \alpha \cdot W_{el}}{\gamma_{M1}} \qquad M_{c.Rd} = 343.7 \circ kNm$$

6.6.6 Lateral-torsional buckling between purlins

[1] 5.9.4.3

Lateral stiffness constant
$$I_z := \frac{2 \cdot b^3 \cdot t_f}{12} + \frac{h \cdot t_w^{\,3}}{12} \qquad I_z = 1.052 \cdot 10^7 \circ mm^4$$

[1] Figure J.2 Varping constant:
$$I_w := \frac{\left(h - t_f\right)^2 \cdot I_z}{4} \qquad I_w = 8.089 \cdot 10^{11} \circ mm^6$$

Torsional constant:
$$I_t := \frac{2 \cdot b \cdot t_f^{\,3} + h \cdot t_w^{\,3}}{3} \qquad I_t = 4.133 \cdot 10^5 \circ mm^4$$

Length between purlins c_p $\qquad c_p = 1 \cdot 10^3 \circ mm$

[1] H.1.2	Moment relation	$\psi := 1$	$\psi = 1$
[1] H.1.2(6)	C_1 - constant	$C_1 := 1.88 - 1.4\cdot\psi + 0.52\cdot\psi^2$	$C_1 = 1$
	Shear modulus		$G = 2.7\cdot10^4\,MPa$
[1] H.1.3(3)	$M_{cr} := \dfrac{C_1\cdot\pi^2\cdot E\cdot I_z}{c_p^2}\cdot\sqrt{\dfrac{I_w}{I_z}+\dfrac{c_p^2\cdot G\cdot I_t}{\pi^2\cdot E\cdot I_z}}$	$W_y := W_{el}$	$M_{cr} = 2.035\cdot10^3\,kN\cdot m$
			$W_y = 1.559\cdot10^6\,mm^3$
[1] 5.6.6.3(3)	$\lambda_{LT} := \sqrt{\dfrac{\alpha\cdot W_y\cdot f_o}{M_{cr}}}$		$\lambda_{LT} = 0.431$
[1] 5.6.6.3(2)	$\alpha_{LT} := if(class>2, 0.2, 0.1)$		$\alpha_{LT} = 0.2$
	$\lambda_{0LT} := if(class>2, 0.4, 0.6)$		$\lambda_{0LT} = 0.4$
[1] 5.6.6.3(1)	$\phi_{LT} := 0.5\cdot\left[1+\alpha_{LT}\cdot\left(\lambda_{LT}-\lambda_{0LT}\right)+\lambda_{LT}^2\right]$		$\phi_{LT} = 0.596$
	$\chi_{LT} := \dfrac{1}{\phi_{LT}+\sqrt{\phi_{LT}^2-\lambda_{LT}^2}}$		$\chi_{LT} = 0.992$

Design moment of resistance of the cross section $M_{c,Rd}$

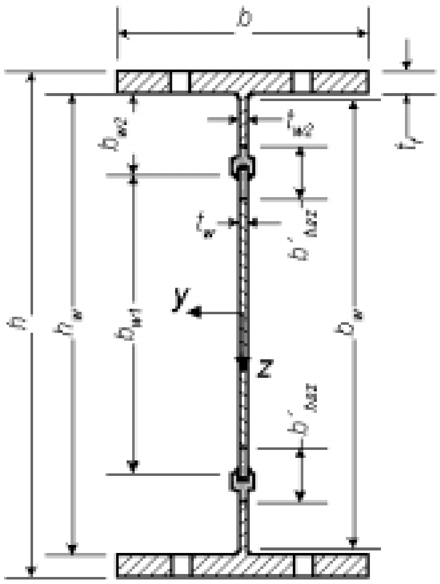
[1] (5.14) $M_{c.Rd} := \chi_{LT}\cdot\dfrac{f_o\cdot\alpha\cdot W_{el}}{\gamma_{M1}}$ $M_{c.Rd} = 341.1\,kNm$

6.6.7 Bending resistance in a section with holes

[1] (5.13) The resistance is based on the effective elastic modulus of the net section W_{net} times the ultimate strength.

Allowance for bolt holes on the tension flange is made by reducing the width of the flanges with two bolt diameters d_b.

$d_b := 12\cdot mm$



Holes are supposed in both flanges

$$I_{net} := I_{gr} - 2\cdot d_b\cdot t_f\cdot\left(\frac{h-t_f}{2}\right)^2\cdot 2 - 2\cdot d_b\cdot\frac{t_f^3}{12}\cdot 2$$

$I_{net} = 3.875\cdot10^8\,mm^4$

Allowance for HAZ $\rho_{haz} = 0.955$

$$I_{net} := I_{net} - 2 \cdot t_w \cdot b'^3_{haz} \cdot \left(1 - \rho_{haz}\right) - 2 \cdot t_w \cdot b'_{haz} \cdot \left(1 - \rho_{haz}\right) \cdot \left(\frac{b_{w1}}{2}\right)^2 \qquad I_{net} = 3.866 \cdot 10^8 \circ mm^4$$

$$W_{net} := \frac{2 \cdot I_{net}}{h} \qquad \frac{f_o}{\gamma_{M1}} = 236 \circ MPa \qquad W_{net} = 1.356 \cdot 10^6 \circ mm^3$$

[1] (5.13) $$M_{a.Rd} := \frac{f_u \cdot W_{net}}{\gamma_{M2}} \qquad \frac{f_u}{\gamma_{M2}} = 248 \circ MPa \qquad M_{a.Rd} = 336.4 \circ kNm$$

The bending resistance is the lesser of $M_{a.Rd}$ and $M_{c.Rd}$

$$M_{Rd} := if\left(M_{a.Rd} < M_{c.Rd}, M_{a.Rd}, M_{c.Rd}\right) \qquad M_{Rd} = 336.4 \circ kNm$$

6.6.8 Shear force resistance

[1] 5.12.4 Design shear resistance $V_{w.Rd}$ for the web: A rigid end post is assumed

[1] (5.93) $$\lambda_w := 0.35 \cdot \frac{b_w}{t_w} \cdot \sqrt{\frac{f_o}{E}} \qquad b_w = 529 \circ mm \qquad \lambda_w = 2.258$$

$$\eta := 0.4 + 0.2 \cdot \frac{f_u}{f_o} \qquad \eta = 0.638$$

[1] Tab. 5.12 $$\rho_v := if\left(\lambda_w > \frac{0.48}{\eta}, if\left(\lambda_w \geq 0.949, \frac{0.48}{\lambda_w}, \frac{0.48}{\lambda_w}\right), \eta\right) \qquad \rho_v = 0.213$$

$$\rho_v := if\left(\lambda_w > 0.949, \frac{1.32}{1.66 + \lambda_w}, \rho_v\right) \qquad \rho_v = 0.337$$

$$h_w := h - 2 \cdot t_w \qquad h_w = 560 \circ mm$$

[1] (5.95) $$V_{w.Rd} := \rho_v \cdot t_w \cdot h_w \cdot \frac{f_o}{\gamma_{M1}} \qquad V_{w.Rd} = 222.99 \circ kN$$

[1] 5.12.5(7) Shear resistance contribution $V_{f.Rd}$ of the flanges.

$$M_{f.Rd} := b \cdot t_f \left(h_w + t_f\right) \cdot \frac{f_o}{\gamma_{M1}} \qquad \frac{\left|M_{1Ed}\right|}{M_{f.Rd}} = 0.436 \qquad M_{f.Rd} = 335 \circ kNm$$

$$c := \left(0.08 + \frac{4.4 \cdot b \cdot t_f^2 \cdot f_o}{t_w \cdot b^2 \cdot f_o}\right) \cdot a \qquad a := L \qquad a = 1 \cdot 10^4 \circ mm \qquad c = 1.384 \cdot 10^4 \circ mm$$

[1] (5.101) $V_{f.Rd} := if\left[\left|M_{1Ed}\right| < M_{f.Rd}, \frac{b \cdot t_f^2 \cdot f_o}{c \cdot \gamma_{M1}} \cdot \left[1 - \left(\frac{M_{1Ed}}{M_{f.Rd}}\right)^2\right], 0\right]$ $V_{f.Rd} = 0.5 \circ kN$

$V_{Rd} := V_{w.Rd} + V_{f.Rd}$ $V_{Rd} = 223.5 \circ kN$

6.6.9 Concentrated transverse force

[1] 5.12.8

[1] Figure 5.24 Length of stiff bearing $s_s := 50 \cdot mm$ $h_w = 560 \circ mm$

[1] (5.109) Parameter m_1 $b_f := b$ $f_{of} := f_o$ $f_{ow} := f_o$ $m_1 := \frac{f_{of} \cdot b_f}{f_{ow} \cdot t_w}$ $m_1 = 32$

[1] Figure 5.24 Buckling coefficient $k_F := 6 + 2 \cdot \left(\frac{h_w}{a}\right)^2$ $k_F = 6.01$

[1] (5.110) Parameter m_2 $m_2 := if\left[\frac{(s_s + 4 \cdot t_f) \cdot h_w \cdot f_{ow}}{k_F \cdot E \cdot t_w^2} > 0.2, 0.02 \cdot \left(\frac{h_w}{t_f}\right)^2, 0\right]$ $m_2 = 26.4$

[1] (5.111) Effective loaded length l_y $l_y := s_s + 2 \cdot t_f \cdot \left(1 + \sqrt{m_1 + m_2}\right)$ $l_y = 316.3 \circ mm$

[1] (5.108) Design resistance F_{Rd} $F_{Rd} := 0.57 \cdot t_w^2 \cdot \sqrt{\frac{k_F \cdot l_y \cdot f_{ow} \cdot E}{h_w}} \cdot \frac{1}{\gamma_{M1}}$ $F_{Rd} = 101.8 \circ kN$

$F_{Rd} := if\left[F_{Rd} > \left(t_w \cdot l_y \cdot \frac{f_{ow}}{\gamma_{M1}}\right), \left(t_w \cdot l_y \cdot \frac{f_{ow}}{\gamma_{M1}}\right), F_{Rd}\right]$ $F_{Rd} = 101.8 \circ kN$

Applied load $F_{Ed} := (q_{p.roof} + q_{snow}) \cdot c_p$ $c_p = 1$ m $F_{Ed} = 13.75 \circ kN$

$F_{Ed} << F_{Rd}$ OK!

6.6.10 Deflections

To calculate the fictive second moment of area I_{fic}, the bending moment in the serviceability limi state is supposed to be half the maximum bending moment at the ultimate limit state.

[1] 4.2.4 $\sigma_{gr} := \frac{0.5 \cdot M_{Ed}}{I_{gr}} \cdot \frac{h}{2}$ $I_{gr} = 4.444 \cdot 10^8 \circ mm^4$ $\sigma_{gr} = 108 \circ MPa$

Allowing for a reduced stress level I_{fic} may be used constant along the beam.

[1] (4.2) $I_{fic} := I_{gr} - \frac{\sigma_{gr}}{f_o} \cdot (I_{gr} - I_{effe})$ $I_{fic} = 4.385 \cdot 10^8 \circ mm^4$

$I := if(class = 4, I_{fic}, I_{gr})$ $class = 4$ $I = 4.385 \cdot 10^8 \circ mm^4$

Approximate deflections (coefficients from FE calculations)

$\delta_1 := 0.7 \cdot \dfrac{5 \cdot q_{p.roof} \cdot L^4}{384 \cdot E \cdot I}$ $\quad q_{p.roof} = 2.75 \circ kN \cdot m^{-1}$ $\quad \delta_1 = 8.2 \circ mm$

$\delta_{k.roof} := 0.75 \cdot \dfrac{5 \cdot q_{k.roof} \cdot L^4}{384 \cdot E \cdot I}$ $\quad q_{k.roof} = 4.125 \circ kN \cdot m^{-1}$ $\quad \delta_{k.roof} = 13.1 \circ mm$

$\delta_{crane} := 0.75 \cdot \dfrac{P_{crane} \cdot L^3}{48 \cdot E \cdot I}$ $\quad P_{crane} = 50 \circ kN$ $\quad \delta_{crane} = 25.45 \circ mm$

$\delta_{snow} := 0.65 \cdot \dfrac{5 \cdot q_{snow} \cdot L^4}{384 \cdot E \cdot I}$ $\quad q_{snow} = 11 \circ kN \cdot m^{-1}$ $\quad \delta_{snow} = 30.3 \circ mm$

$\delta_{w.roof} := 0.75 \cdot \dfrac{5 \cdot q_{w.roof} \cdot L^4}{384 \cdot E \cdot I}$ $\quad q_{w.roof} = -3.85 \circ kN \cdot m^{-1}$ $\quad \delta_{w.roof} = -12.3 \circ mm$

[5] 9.5.2 Frequent load combination no. 3, snow load dominant

[5] (9.16) $\delta_2 := 0 \cdot \delta_{k.roof} + 0.3 \cdot \delta_{crane} + 0.2 \cdot \delta_{snow} + 0 \cdot \delta_{w.roof}$ $\quad \delta_2 = 14 \circ mm$

Pre-camber $\quad \delta_0 := 0 \cdot mm$

[1] (4.1) $\delta_{max} := \delta_1 + \delta_2 - \delta_0$ $\quad \delta_{max} = 21.9 \circ mm$

The FEM calculation gives for the same load combination $\delta_{max} := 20.2 \cdot mm$

(1.1) $\delta_{limit} := \dfrac{L}{250}$ for beams carrying roof $\quad \delta_{limit} = 40 \circ mm$

6.6.11 Summary

$M_{Rd} = 336 \circ kNm$

$M_{Ed} = 336 \circ kNm$ $\quad M_{c.Rd} = 341 \circ kNm$ $\quad M_{1Ed} = -146 \circ kNm$ $\quad \dfrac{M_{Ed}}{M_{Rd}} = 0.999$

$V_{Ed} = 172 \circ kN$ $\quad V_{Rd} = 223.5 \circ kN$ $\quad \dfrac{V_{Ed}}{V_{Rd}} = 0.77$

$\delta_{limit} = 40 \circ mm$ $\quad \delta_{max} = 20 \circ mm$ $\quad I_{fic} = 4.385 \cdot 10^8 \circ mm^4$

Cross section $\quad h = 570 \circ mm$ $\quad b = 160 \circ mm$ $\quad t_w = 5 \circ mm$ $\quad t_f = 15.4 \circ mm$ $\quad A_{gr} = 7.624 \cdot 10^3 \circ mm^2$

6.7 Welded Connections

6.7.1 Weld properties

The checking of welded connections includes two parts:
- the design of the welds
- the check of HAZ adjacent to welds ([1] 6.6.3.5)

In this design example three connections are checked. Other connections are treated in a similar wa

EN-AW 6082 T6 welded with 5356 filler metal

[1] Tab 5.5.1 $\rho_{haz} := 0.65$

S.I. units: $kN \equiv 1000 \cdot newton$

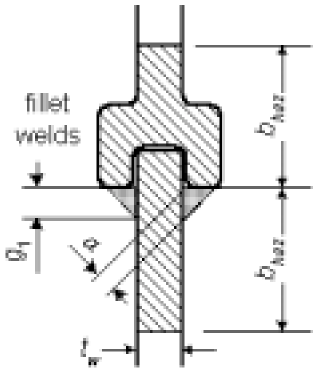
[1] 5.5.2 $f_{a.haz} := 220 \cdot MPa$ $f_{v.haz} := 97 \cdot MPa$

$kNm \equiv kN \cdot m$ $MPa \equiv 10^6 \cdot Pa$

[1] 6.6.1 $\gamma_{Mw} := 1.25$

[1] Tab 6.8 $f_w := 210 \cdot MPa$

6.7.2 Longitudinal weld of floor beam D



a) Design of weld

[1] 6.6.3.3 (10) NOTE

The longitudinal weld of floor beam D is submitted to shear stress and normal stress acting along the weld axis. In design the normal stress does not have to be considered.

(6.4.1) $t_w := 4 \cdot mm$ $h := 300 \cdot mm$ $t_f := 12 \cdot mm$ $b := 120 \cdot mm$

(6.4.2) Shear force acting along the beam $V_{Ed} := 91.6 \cdot kN$

Shear stress $\tau := \dfrac{V_{Ed}}{t_w \cdot (h - t_f)}$ $\tau = 79.5 \circ MPa$

Choose weld throat $a := 3 \cdot mm$, two welds

[1] (6.42) $Check := if\left[a > 0.85 \cdot \dfrac{\tau \cdot t_w}{\dfrac{f_w}{\gamma_{Mw}}}, \text{"OK!"}, \text{"Not OK!"}\right]$ $0.85 \cdot \dfrac{\tau \cdot t_w}{\dfrac{f_w}{\gamma_{Mw}}} = 1.6 \circ mm$ $Check = \text{"OK!"}$

b) Design strength in HAZ

Tensile force perpendicular to the failure plane: As the weld is located close to the neutral axis, ther no need to check this point.

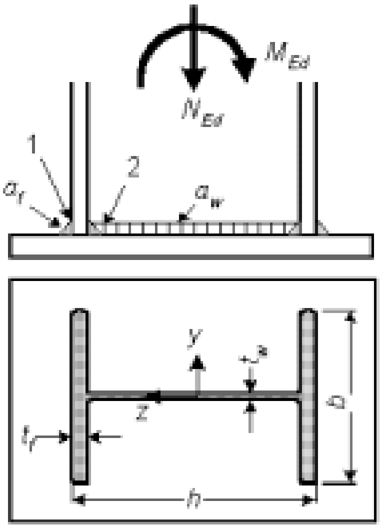
[1] (6.50) Shear stress in the failure plane at the toe of the weld:

$\tau = 79.5 \circ MPa$ is at the same level as $\dfrac{f_{v.haz}}{\gamma_{Mw}} = 77.6 \circ MPa$. Can be accepted!

[1] (6.51) Shear stress at the failure plane at the fusion boundary:

As in practice $g_1 > t_w$, there is no need to check this formula

6.7.3 Base of column B



Note!
As the notations in [1] for perpendicular and parallel cannot be used in Mathcad expressions the notations τ_{per} and τ_{ll} are used.

Cross section, see 6.2.1 and 6.2.7

$h := 200 \cdot mm$ $\quad$ $b := 160 \cdot mm$

$t_w := 7 \cdot mm$ $\quad$ $t_f := 16 \cdot mm$

$A := 6296 \cdot mm^2$

$I_y := 4.621 \cdot 10^7 \cdot mm^4$

a) Design of welds

In order to use the expression for combined stress components ([1] (6.37)), we calculate the normal stress σ_{per} and the two shear stresses τ_{per} and τ_{ll} which are induced by the normal stress σ and the shear stress τ in the connected member column B.

The base of column B is submitted to normal force N_{Ed}, bending moment M_{Ed} and shear force V_{Ed}. Two cases are to be considered (see 6.2.13 of this example)

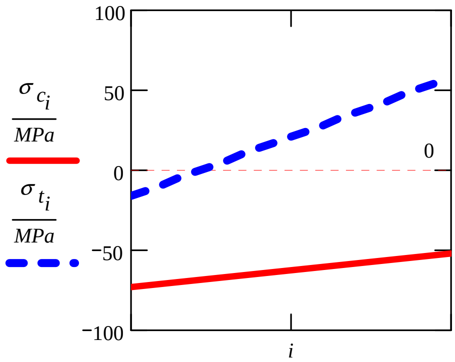
$N_{Ed} := \begin{bmatrix} -288 \\ 11.7 \end{bmatrix} \cdot kN$ $\quad$ $M_{Ed} := \begin{bmatrix} 13.2 \\ 25.5 \end{bmatrix} \cdot kNm$ $\quad$ $V_{Ed} := \begin{bmatrix} 5.9 \\ 16.1 \end{bmatrix} \cdot kN$ $\quad$ LC1, max N, compression
LC4, min N, tension

The normal stress σ in the member is defined by

On the compression side, in extreme fibre $\quad$ $z := -\frac{h}{2}$ $\quad$ $\sigma_c := \frac{N_{Ed}}{A} + \frac{M_{Ed}}{I_y} \cdot z$ $\quad$ $\sigma_c = \begin{bmatrix} -74.3 \\ -53.3 \end{bmatrix} \circ MPa$

On the tension side in extreme fibre 1 $\quad$ $z := \frac{h}{2}$ $\quad$ $\sigma_t := \frac{N_{Ed}}{A} + \frac{M_{Ed}}{I_y} \cdot z$ $\quad$ $\sigma_t = \begin{bmatrix} -17.2 \\ 57 \end{bmatrix} \circ MPa$

The compression stresses are transmitted through the contact surface between the column and the plate. No check of the weld is needed. The HAZ is checked in 6.2.11 (flexural buckling) and 6.2.12 (lateral-torsional buckling)

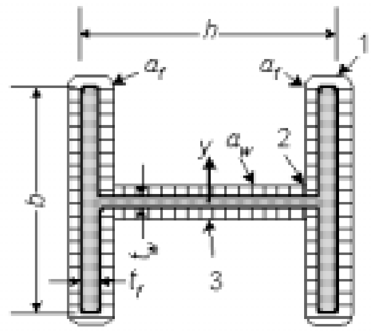


Check the tensile stress $\sigma_{t_2} = 57 \cdot MPa$

Weld throat $a_f := 10 \cdot mm$ around the flange.

Comment: For process reasons weld throat $a_f > t_{min}$ which is larger than needed by the calculation.

The normal stress σ_t in the extreme fibre of the member induces in the weld the normal stress σ_{per} and the shear stress τ_{per}



$$\sigma_{per} := \frac{\sigma_{t_2} \cdot t_f}{2 \cdot \sqrt{2} \cdot a_f} \qquad \sigma_{per} = 32.3 \cdot MPa$$

$$\tau_{per} := \sigma_{per}$$

The shear stress τ_{ll} in the *flange* is neglected, but see comment below.

[1] (6.37) The resulting stress in the fillet weld is $\sigma_c := \sqrt{\sigma_{per}^2 + 3 \cdot \left(\tau_{ll}^2 + \tau_{per}^2\right)}$ $\quad \sigma_c = 65 \cdot MPa$

[1] (6.38) $Check := if\left(\sigma_c \le \frac{f_w}{\gamma_{Mw}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\quad \frac{f_w}{\gamma_{Mw}} = 168 \cdot MPa$ $\quad Check = \text{"OK!"}$

At point 2 in the web there is a tensile stress for $z := \frac{h}{2} - t_f - a_f \cdot \sqrt{2}$

$$\sigma_t := \frac{N_{Ed}}{A} + \frac{M_{Ed}}{I_y} \cdot z \qquad \sigma_t = \begin{bmatrix} -25.8 \\ 40.4 \end{bmatrix} \cdot MPa$$

In the welds, throat $a_w := 6 \cdot mm$, $\quad \sigma_{per} := \frac{\sigma_{t_2} \cdot t_w}{2 \cdot \sqrt{2} \cdot a_w}$ $\quad \sigma_{per} = 16.7 \cdot MPa$ $\quad \tau_{per} := \sigma_{per}$

The shear stress is: (S_z is the first moment of area for the flange) $\quad S_z := b \cdot t_f \cdot \left(0.5 \cdot h - 0.5 \cdot t_f\right)$

$$\tau := \frac{V_{Ed_2} \cdot S_z}{t_w \cdot I_y} \qquad \tau = 11.7 \cdot MPa$$

Compare: $\frac{V_{Ed_2}}{t_w \cdot \left(h - t_f\right)} = 12.5 \cdot MPa$

In the welds $\quad \tau_{ll} := \frac{\tau \cdot t_w}{2 \cdot a_w} \qquad \tau_{ll} = 6.8 \cdot MPa$

[1] (6.37) The resulting stressin the fillet weld is $\sigma_c := \sqrt{\sigma_{per}^2 + 3\cdot\left(\tau_{ll}^2 + \tau_{per}^2\right)}$ $\sigma_c = 35\cdot MPa$

[1] (6.38) $Check := if\left(\sigma_c \le \frac{f_w}{\gamma_{Mw}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\frac{f_w}{\gamma_{Mw}} = 168\cdot MPa$ $Check = \text{"OK!"}$

Comment: The shear stress in the flange can be calculated with the expression:

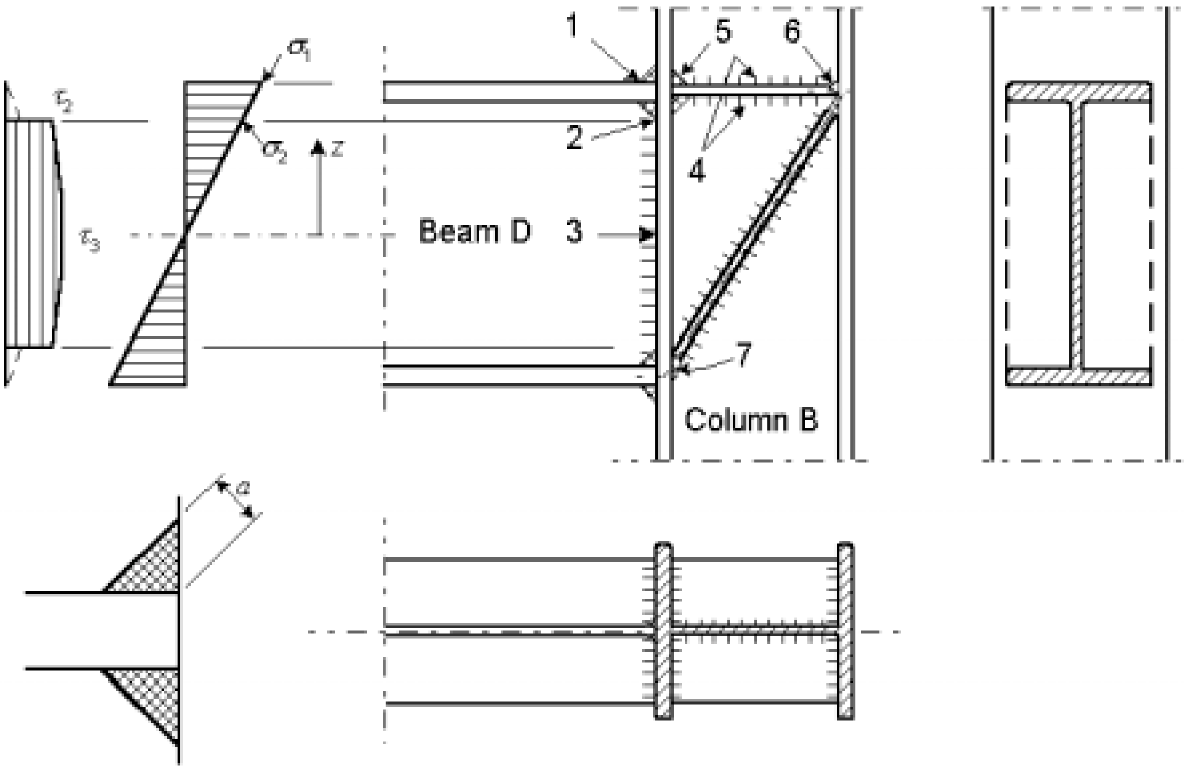
$$\tau := \frac{V_{Ed}}{2\cdot t_f\cdot\left(h - t_f\right)\cdot\left[1 + \frac{t_w\cdot\left(h - t_f\right)}{6\cdot t_f\cdot b}\right]} \qquad \tau = \begin{bmatrix} 0.925 \\ 2.523 \end{bmatrix}\cdot MPa$$

b) Check of HAZ

[1] (6.46) The HAZ is checked in 6.2.11 (flexural buckling) and 6.2.12 (lateral-torsional buckling).

[1] (6.47) As $g_1 > a$, no check at the fusion boundary is needed.

6.7.4 Connection between floor beam D and column B



The diagonal stiffeners (on both sides) are welded first with unsymmetric fillet welds. Then the horizontal stiffeners are welded with butt weld to the flanges close to the diagonals and with fillet w to the web.

a) Beam D, design of welds

Cross section, see 6.4.1 and 6.4.7 $h := 300 \cdot mm$ $b := 120 \cdot mm$ $A := 3984 \cdot mm^2$
$t_w := 4 \cdot mm$ $t_f := 12 \cdot mm$ $I_y := 6.676 \cdot 10^7 \cdot mm^4$

As the notations in [1] for perpendicular and parallel cannot be used in Mathcad expressions the notations τ_{per} and τ_{ll} are used.

In order to use the expression for combined stress components ([1] (6.37)), we calculate the normal stress σ_{per} and the two shear stresses τ_{per} and τ_{ll} which are induced by the normal stress σ and the shear stress τ in the connected member beam D.

The end of beam D is submitted to a shear force V_{Ed} and a bending moment M_{Ed}. The normal force N_{Ed} is small and is neglected.

$M_{Ed} := -64.4 \cdot kNm$ $V_{Ed} := 91.7 \cdot kN$

The normal stress σ_t in the member is defined by:

On the tension side, in extreme fibre 1 $z := -\frac{h}{2}$ $\sigma_t := \frac{M_{Ed}}{I_y} \cdot z$ $\sigma_t = 145 \circ MPa$

The normal stress σ_t in the extreme fibre of the member induces in the weld the normal stress σ_{per} and the shear stress τ_{per}. Weld throat $a_f := 8 \cdot mm$

$\sigma_{per} := \frac{\sigma_t \cdot t_f}{2 \cdot \sqrt{2} \cdot a_f}$ $\sigma_{per} = 76.7 \circ MPa$ $\tau_{per} := \sigma_{per}$

The shear stress τ_{ll} in the *flange* is neglected. Then the resulting stress in the fillet weld is

[1] (6.37) $\sigma_c := \sqrt{\sigma_{per}^2 + 3 \cdot \tau_{per}^2}$ $\sigma_c = 153 \circ MPa$

[1] (6.38) $Check := if\left(\sigma_c \le \frac{f_w}{\gamma_{Mw}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\frac{f_w}{\gamma_{Mw}} = 168 \circ MPa$ $Check = \text{"OK!"}$

At point 2 in the web there is a tensile stress for $z := -\left(\frac{h}{2} - t_f - a_f \cdot \sqrt{2}\right)$ $z = -127 \circ mm$

$\sigma_t := \frac{M_{Ed}}{I_y} \cdot z$ $\sigma_t = 122 \circ MPa$ *Comment: For process reasons a weld throat larger than needed by the calculation is used.*

Welds, throat $a_w := 5 \cdot mm$, $\sigma_{per} := \frac{\sigma_t \cdot t_w}{2 \cdot \sqrt{2} \cdot a_w}$ $\sigma_{per} = 34.6 \circ MPa$ $\tau_{per} := \sigma_{per}$

The shear stress is: (S_z is the first moment of area for the flange) $S_z := b \cdot t_f \cdot \left(0.5 \cdot h - 0.5 \cdot t_f\right)$

$\tau := \dfrac{V_{Ed} \cdot S_z}{t_w \cdot I_y}$ $\tau = 71.2 \circ MPa$ Compare: $\dfrac{V_{Ed}}{t_w \cdot \left(h - t_f\right)} = 79.6 \circ MPa$

In the welds $\tau_{ll} := \dfrac{\tau \cdot t_w}{2 \cdot a_w}$ $\tau_{ll} = 28.5 \circ MPa$

[1] (6.37) The resulting stress in the fillet weld is $\sigma_c := \sqrt{\sigma_{per}^2 + 3 \cdot \left(\tau_{ll}^2 + \tau_{per}^2\right)}$ $\sigma_c = 85 \circ MPa$

[1] (6.38) $Check := if\left(\sigma_c \le \dfrac{f_w}{\gamma_{Mw}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\dfrac{f_w}{\gamma_{Mw}} = 168 \circ MPa$ $Check = \text{"OK!"}$

In the middle of the web, point 3

$\sigma_t := 0 \cdot MPa$ $\tau := \dfrac{V_{Ed}}{t_w \cdot \left(h - t_f\right)}$ $\tau_{ll} := \dfrac{\tau \cdot t_w}{2 \cdot a_w}$ $\tau_{ll} = 31.8 \circ MPa$

[1] (6.37) The resulting stress in the fillet weld is $\sigma_c := \sqrt{3} \cdot \tau_{ll}$ $\sigma_c = 55 \circ MPa$

[1] (6.38) $Check := if\left(\sigma_c \le \dfrac{f_w}{\gamma_{Mw}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\dfrac{f_w}{\gamma_{Mw}} = 168 \circ MPa$ $Check = \text{"OK!"}$

b) Beam D, design strength HAZ

Verify in the HAZ

[1] (6.54) - at the toe of the weld: $\sqrt{\sigma^2 + 3 \cdot \tau^2} \le \dfrac{f_{a.haz}}{\gamma_{Mw}}$

[1] (6.55) - at the fusion boundary: $\sqrt{\sigma^2 + 3 \cdot \tau^2} \le \dfrac{g_1 \cdot f_{a.haz}}{t \cdot \gamma_{Mw}}$

As in practice $g_1 > t$, there is no need to check the formula at the fusion boundary.

In the flange: $\sigma := \sigma_1 := 145 \cdot MPa$ $\tau := 0 \cdot MPa$ $\sigma_c := \sqrt{\sigma_1^2 + 3 \cdot \tau^2}$ $\sigma_c = 145 \circ MPa$

[1] (6.54) $Check := if\left(\sigma_c \le \dfrac{f_{a.haz}}{\gamma_{Mw}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\dfrac{f_{a.haz}}{\gamma_{Mw}} = 176 \circ MPa$ $Check = \text{"OK!"}$

In the web: $\sigma := 122 \cdot MPa$ $\tau := 71 \cdot MPa$ $\sigma_c := \sqrt{\sigma^2 + 3 \cdot \tau^2}$ $\sigma_c = 173 \circ MPa$

[1] (6.54) $Check := if\left(\sigma_c \le \dfrac{f_{a.haz}}{\gamma_{Mw}}, \text{"OK!"}, \text{"Not OK!"}\right)$ $\dfrac{f_{a.haz}}{\gamma_{Mw}} = 176 \circ MPa$ $Check = \text{"OK!"}$

c) Column B, design of welds and stiffeners

(5.4.8) Moment $M_{B3} := 32.4\cdot kNm$ $M_{B2} := -34\cdot kNm$

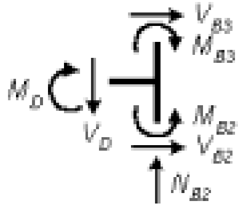
$M_D - 64.4\cdot kNm$

Shear force $V_{B3} := 2.31\cdot kN$ $V_{B2} := -18.2\cdot kN$

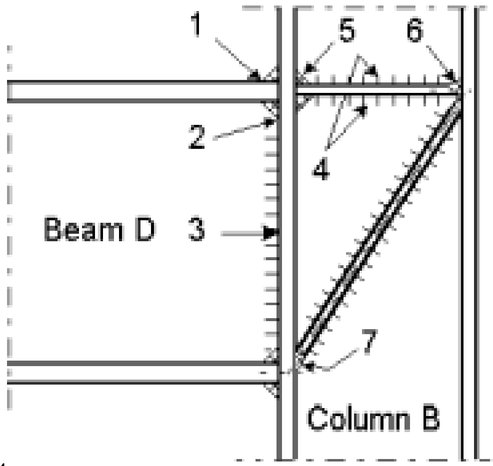
$V_D := 91.7$

Axial force $N_{B3} := 240\cdot kN$ $N_{B2} := -307\cdot kN$

$N_D := 0\cdot kN$



The tensile force F_5 in the upper flange of beam D and the distance h_{fD} between the centre of the flanges are needed to check the stiffeners in column B



$F_5 := \left(1 - \frac{t_f}{h}\right)\cdot \sigma_1 \cdot b\cdot t_f$ $F_5 = 200\circ kN$

$h_{fD} := h - t_f$ $b_D := b$

Cross section column B see 6.2.1 and 6.2.7

$h := 200\cdot mm$ $b := 160\cdot mm$ $A := 6296\cdot mm^2$

$t_w := 7\cdot mm$ $t_f := 16\cdot mm$ $I_y := 4.621\cdot 10^7\cdot mm^4$

Welds 5 between the upper stiffener and the column flange

These welds are given the same dimensions as weld 1. No check is needed

Welds 4 between the upper stiffener and the column web

The tensile force in the stiffener is the tensile force in the upper flange $F_5 = 200\circ kN$

This force is in equilibrium with the shear force V_{B3} in column B above the stiffener, the shear force V_w in the web below the stiffener and the horizontal component F_6 of the force in the diagonal stiffener.

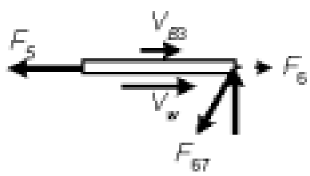
$$F_5 - V_{B3} - V_w - F_6 = 0$$

The shear resistance in the web below the stiffener is the least of ($f_o := 255\cdot MPa$ $\gamma_{M1} := 1.1$)

$V_{wo} := \left(h - 2\cdot t_f\right)\cdot t_w\cdot \frac{f_o}{\sqrt{3}\cdot \gamma_{M1}}$ and $V_{w.haz} := \left(h - 2\cdot t_f\right)\cdot t_w\cdot \frac{f_{v.haz}}{\gamma_{Mw}}$

$V_w := if\left(V_{wo} < V_{w.haz}, V_{wo}, V_{w.haz}\right)$ $V_w = 91\circ kN$

As the web panel is stiffened, there is no risk of shear buckling.



The welds 4 are designed for the shear force $V_{B3} + V_w = 93.6 \cdot kN$ Choose $a_4 := 4 \cdot mm$

$\tau := \dfrac{V_{B3} + V_w}{4 \cdot a_4 \cdot (h - 2 \cdot t_f)}$ $\tau = 34.8 \cdot MPa$ < $\dfrac{f_w}{\sqrt{3} \cdot \gamma_{Mw}} = 97 \cdot MPa$ OK !

Stiffener 4 and weld 6

The stiffener is in tension. Tension force $F_5 = 200 \cdot kN$, width $b_D = 120 \cdot mm$, thickness $t_4 := 12 \cdot mm$

Stiffener, HAZ close to 5 $\sigma := \dfrac{F_5}{b_D \cdot t_4}$ $\sigma = 139 \cdot MPa$ < $\dfrac{f_{a.haz}}{\gamma_{Mw}} = 176 \cdot MPa$ OK !

Weld 6, "butt weld" $\sigma := \dfrac{F_5 - V_{B3} - V_w}{b_D \cdot t_4}$ $\sigma = 74 \cdot MPa$ < $\dfrac{f_w}{\gamma_{Mw}} = 168 \cdot MPa$ OK !

Diagonal stiffener 6-7

Compression force F_{67}

$F_6 := F_5 - V_{B3} - V_w$ $F_{67} := F_6 \cdot \dfrac{\sqrt{(h - t_f)^2 + h_{fD}^2}}{h - t_f}$ $F_{67} = 199 \cdot kN$

Width $b_D = 120 \cdot mm$, thickness $t_{67} := 12 \cdot mm$ OK !

$\sigma := \dfrac{F_{67}}{b_D \cdot t_{67}}$ $\sigma = 138 \cdot MPa$ < $\dfrac{f_{a.haz}}{\gamma_{Mw}} = 176 \cdot MPa$ OK !

Weld 6 and 7, "Unsymmertic butt weld" $\sigma = 138 \cdot MPa$ < $\dfrac{f_w}{\gamma_{Mw}} = 168 \cdot MPa$

Weld between stiffener and column web: Make it as small as possible!

[1] 5.4.4 Local buckling, heat treated, welded stiffener

$\beta := \dfrac{b_D}{2 \cdot t_{67}}$ $\beta = 5$ $\varepsilon := \sqrt{\dfrac{250 \cdot MPa}{f_o}}$ $\beta_3 := 5 \cdot \varepsilon$ $\beta_3 = 4.95$

β is close to β_3 Cross section class 3 OK !

Eventual horizontal stiffener from point 7

Check if a stiffener is needed due to web crippling according to [1] 5.12.8.

Force $F_7 := F_5 - F_6$ $F_7 = 94 \cdot kN$

[1] 5.12.8 $f_{of} := f_o$ $f_{ow} := f_o$ $b_f := b$ $a := 3 \cdot m$ $h_w := h - t_f$

$s_s := 12 \cdot mm + 2 \cdot \sqrt{2} \cdot 8 \cdot mm$ $s_s = 35 \cdot mm$ $E := 70000 \cdot MPa$

[1] (5.109) $m_1 := \frac{f_{of} \cdot b_f}{f_{ow} \cdot t_w}$ $m_1 = 22.9$

[1] Fig 5.23 $k_F := 6 + 2 \cdot \left(\frac{h_w}{a}\right)^2$ $k_F = 6.01$

[1] (5.110) $m_2 := if\left[\frac{(s_s + 4 \cdot t_f) \cdot h_w \cdot f_{ow}}{k_F \cdot E \cdot t_w^{\ 2}} > 0.2, 0.02 \cdot \left(\frac{h_w}{t_f}\right)^2, 0\right]$ $m_2 = 2.6$

[1] (5.113) $l_y := s_s + 2 \cdot t_f \cdot \left(1 + \sqrt{m_1 + m_2}\right)$ $l_y = 228 \circ mm$

[1] (5.108) $F_{Rd} := 0.57 \cdot t_w^{\ 2} \cdot \sqrt{\frac{k_F \cdot l_y \cdot f_{ow} \cdot E}{h_w}} \cdot \frac{1}{\gamma_{M1}}$ $F_{Rd} = 293 \circ kN$

[1] (5.108) $F_{Rd} := if\left[F_{Rd} > \left(t_w \cdot l_y \cdot \frac{f_{ow}}{\gamma_{M1}}\right), \left(t_w \cdot l_y \cdot \frac{f_{ow}}{\gamma_{M1}}\right), F_{Rd}\right]$ $F_{Rd} = 293 \circ kN$

$F_{Rd} > F_7$ No stiffener is needec