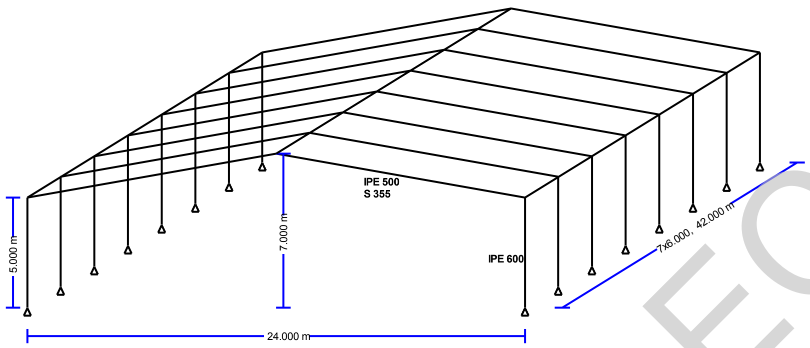
Example Report



1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
EN1991-1-1:2002, Eurocode 1-1 Actions on structures
EN1991-1-3:2003, Eurocode 1-3 Snow loads
EN1991-1-4:2005, Eurocode 1-4 Wind actions
EN1993-1-1:2005, Eurocode 3 1-1 Design of Steel structures
EN1993-1-3:2005, Eurocode 3 1-3 Cold-formed members
EN1993-1-5:2006, Eurocode 3 1-5 Plated structural elements
EN1993-1-8:2005, Eurocode 3 1-8 Design of Joints
CEN/TS 1992-4-1:2009, Design of fastenings in concrete, General
CEN/TS 1992-4-2:2009, Design of fastenings, Headed Fasteners
EN1992-1-1:2004, Eurocode 2 Reinforced concrete
EN1997-1-1:2004, Eurocode 7 Geotechnical design

2. Basic data

2.1. Geometry of frame structure

Bay width	L = 24.000 m
Total height(max)	H = 7.000 m
Column height	H1 = 5.000 m
Total length	B = 42.000 m (7x6.000m)
Spacing of frames	s = 6.000 m
Roof slope	α = 9.46°
Haunch size	L1 =L/10.0= 2.400 m
Cladding	Sheeting thickness tw=0.750 mm, Profile depth hw=40.0 mm
Purlin spacing	= 3.000 m
	Purlin laterally restrained, Continuous purlin

2.2. Steel sections

Column section	IPE 600 - S 355
Rafter section	IPE 500 - S 355
Purlin section	IPE 160 - S 355
Transverse restraint system	L100x100x10 - S 355
Lateral bracing of columns	Lm1= 4.250 m
Torsional restrains of rafters	Lm2= 3.201 m

Compression stiffener at the bottom of haunch

2.3. Steel joints

Type of connection	End-plate connection, non-preloaded bolts
Category of connection	Category A: Bearing type
	Category D: Non-preloded
End Plate	Thickness tp=20 mm, S 235
Bolts	M24, Grade 10.9

2.4. Column base and foundation

Base plate steel grade	Thickness tp=30 mm, S 235
Anchor bolts	M24, Grade 5.6
Concrete of foundament	C25/30-B500C
Concrete cover	Cnom= 35 mm
Soil bearing capacity	qu= 0.200 kN/mm²
Unit weight of soil	γ = 18.00 kN/m³
Foundation depth	hf= 2.700 m

3. Materials and Code parameters

3.1. Materials

Steel: S 355 (EN1993-1-1, §3.2)

t<= 40 mm, Yield strength fy= 355 N/mm², Ultimate strength fu= 510 N/mm²
40mm<t<= 80 mm, Yield strength fy= 335 N/mm², Ultimate strength fu= 470 N/mm²
Modulus of elasticity E=210000 N/mm², Poisson ratio ν=0.30, Unit mass ρ= 7850 Kg/m³

Partial factors for materials (EN1993-1-1, §6.1)

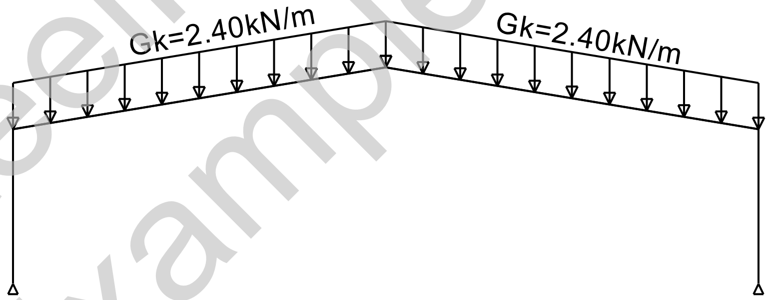
γM0= 1.00, γM1= 1.00, γM2= 1.25

4. Loads

4.1. Permanent loads (EN1991-1-1)

Self weight of purlins and finishing	gk1 = 0.200+0.155/3.000= 0.252kN/m²
Self weight of ceiling under the roof	gk2 = 0.000kN/m² gk =gk1+gk2 = 0.252kN/m²
Spacing of frames	s = 6.000 m
Roof load on frame	(gk1+gk2)·s = 0.252x6.000= 1.51kN/m
Self weight of Rafters	G(IPE 500)= 0.89kN/m
Permanent load on frame	Gk = 1.51+0.89= 2.40kN/m
Self weight of Columns	G(IPE 600)= 1.20kN/m

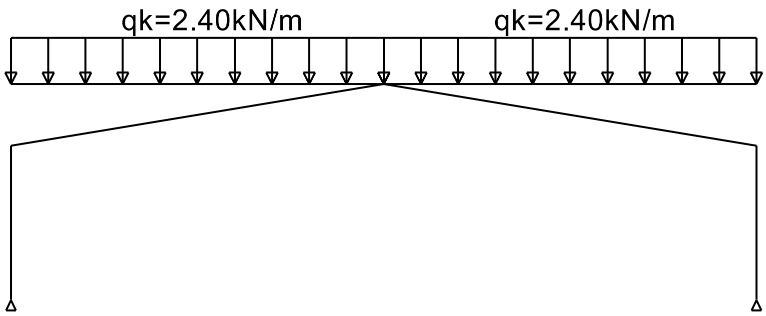
Load G (Permanent load)



4.2. Imposed loads (EC1 EN1991-1-1:2002 Tab.6.10)

Roof slope	α = 9.46°
Imposed load (category H)	qk = 0.40kN/m²
Roof load on frame	qk·s = 0.40x6.000= 2.40kN/m

Load Qk (Imposed load)



4.3. Snow load

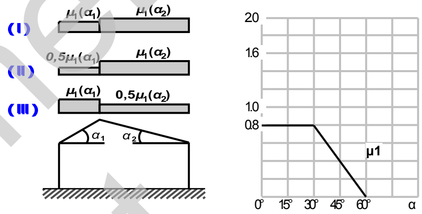
(EC1 EN1991-1-3:2003)

Snow load on the ground (EN1991-1-3 §4, Annex C)

Characteristic value of snow load on the ground: sk=0.800 kN/m²

Snow load on the roof (EC1 EN1991-1-3:2003, §5,)

pitched roof (EC1-1-3 §5.3.3))
Angle of pitch of roof : a1=9.462°
Angle of pitch of roof : a2=9.462°
Exposure coefficient : Ce=1.000 (EC1-1-3 §5.2(7))
Thermal coefficient : Ct=1.000 (EC1-1-3 §5.2(8))
Shape coefficients μ1(α1)=μ1(α2)=0.800 (EC1-1-3 T.5.2))
S(α1)=μ1(α1)·Ce·Ct·Sk=0.800x1.000x1.000x0.800=0.640kN/m²
S(α2)=μ1(α2)·Ce·Ct·Sk=0.800x1.000x1.000x0.800=0.640kN/m²



Snow load (EC1 EN1991-1-3:2003, §5.2, §5.3.3)

Load case (I) , S(Left)=S(α1) =0.640 kN/m², S(Right)=S(α2)= 0.640 kN/m²
Load case (II) , S(Left)=0.5xS(α1)=0.320 kN/m², S(Right)=S(α2)= 0.640 kN/m²
Load case (III), S(Left)=S(α1)= 0.640 kN/m², S(Right)=0.5xS(α2)=0.320 kN/m²

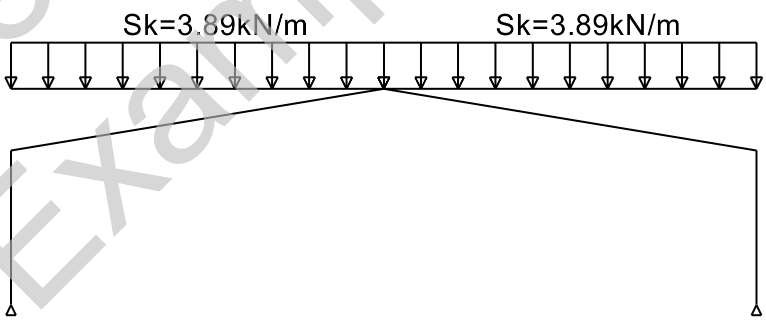
4.4. Snow load on frame

(EC1 EN1991-1-3:2003)

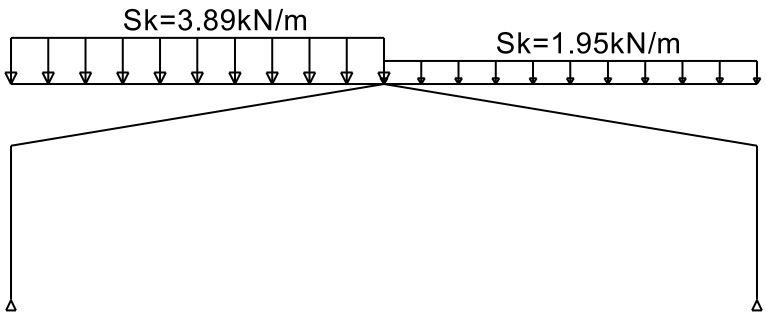
Snow load on the ground sk= 0.800 kN/m²
Snow load on the roof Sk= 0.8x0.800x1.00x1.00=0.640 kN/m²
Spacing of frames s= 6.000 m
Snow load on frame Sk1= 0.640x6.000/cos9.46°=3.89kN/m
Sk2=0.5x0.640x6.000/cos9.46°=1.95kN/m

Load case(I) Sk1= 3.89 kN/m, Sk2=3.89 kN/m
Load case(II) Sk1= 3.89 kN/m, Sk2=1.95 kN/m
Load case(III) Sk1= 1.95 kN/m, Sk2=3.89 kN/m

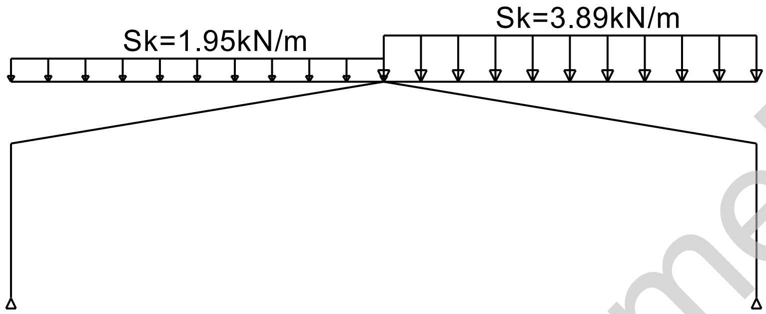
Load Qs1 (Snow load)



Load Qs2 (Snow load)



Load Qs3 (Snow load)



4.5. Wind load (EC1 EN1991-1-4:2005)

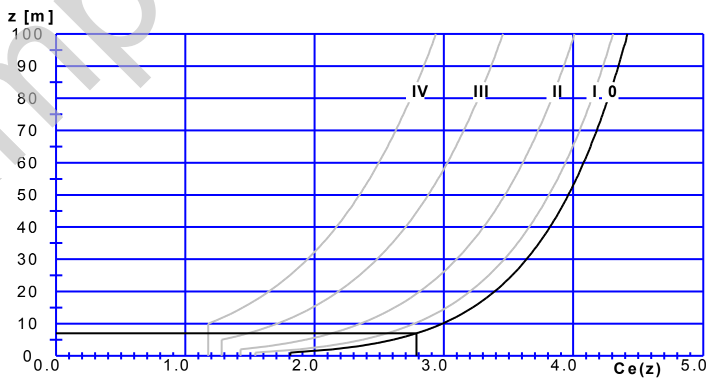
Terrain effects (EN1991-1-4, §4.3.2, Annex A)

Terrain category : 0 (EN1991-1-4, Tab. 4.1)
Sea, coastal area exposed to the open sea
Roughness factor Cr(z) (EN1991-1-4, §4.3.2)
Terrain category: 0, z=7.000m, zo=0.003m, zmin=1m, zmax:=200m, zoII=0.050m
kr=0.19·(0.003/0.05)$^{0.07}$=0.156, Cr(z)=kr·ln(z/zo)=0.156xln(7.000/0.003)=1.210
Orography factor Co(z) (EN1991-1-4, §4.3.3)
Co(z)=0.000 (EN1991-1-4, §4.3.3)
Turbulence factor Kl (EN1991-1-4, §4.4)
Kt=1.000
Exposure factor Ce(z) (EN1991-1-4, §4.5)
Terrain category : 0 (EN1991-1-4, Tab. 4.1)
z= 7.00 m, kr=0.156, lv(z)=0.129, Ce((EN1991-1-4, eq. A.4.8, eq. A.4.7, eq. A.4.4, eq. A.4.3)

Wind peak velocity pressure q(z)=Ce(z)·qb =Ce(z)·(0.625)·Vb² (EN1991-1-4, §4.5)

Vb=22.73m/sec
z=7.000m
Cr(z)=1.210
Co(z)=0.000
Kt=1.000
Ce(z)=2.786

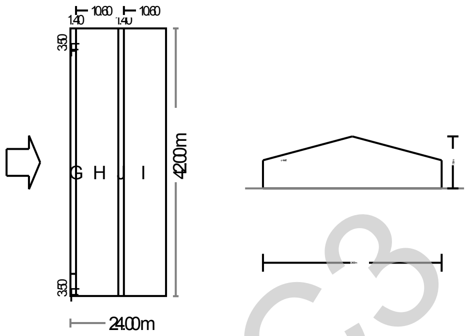
q(z)=Ce(z)·(½ρ)·Vb²
=Ce(z)x0.625x22.73²
= 0.900 N/m²



Wind forces on duopitch roof, wind direction: 0.00 (EN1991-1-4, §7.2.5)

Wind pressure coefficients Cpe (EN1991-1-4, Tab. 7.4a)

wind direction: θ=0.00
b=42.00m, d=24.00m, h=7.00m, e=min(b,2h)=14.00m
e/4=3.50m, e/10=1.40m, e/2=7.00m
Pitch angle: α=9.462



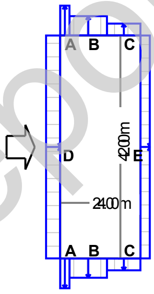
Zone : F, A= 4.97m², Cpe,10=-1.34, Cpe,1=-2.28
Zone : F, A= 4.97m², Cpe,10=+0.09, Cpe,1=+0.09
Zone : G, A= 49.68m², Cpe,10=-1.02, Cpe,1=-1.78
Zone : G, A= 49.68m², Cpe,10=+0.09, Cpe,1=+0.09
Zone : H, A= 451.34m², Cpe,10=-0.47, Cpe,1=-0.80
Zone : H, A= 451.34m², Cpe,10=+0.09, Cpe,1=+0.09
Zone : I, A= 451.34m², Cpe,10=-0.51, Cpe,1=-0.51
Zone : I, A= 451.34m², Cpe,10=-0.33, Cpe,1=-0.33
Zone : J, A= 59.61m², Cpe,10=-0.34, Cpe,1=-0.56
Zone : J, A= 59.61m², Cpe,10=-0.33, Cpe,1=-0.33

Wind pressure on roof surfaces we=q(z)·Cpe=0.900xCpe [kN/m²] (EN1991-1-4, 5.1)

F		G		H		I		J	
we,10	we,1	we,10	we,1	we,10	we,1	we,10	we,1	we,10	we,1
-1.209	-2.049	-0.919	-1.599	-0.420	-0.719	-0.460	-0.460	-0.302	-0.503
+0.080	+0.080	+0.080	+0.080	+0.080	+0.080	-0.299	-0.299	-0.299	-0.299

Wind forces on vertical walls (EN1991-1-4, §7.2.2)

Wind pressure coefficients Cpe (EN1991-1-4, Tab. 7.1)



h/d=5.00/24.00=0.208, e=10.00m
Zone : A, (2.00xh), Cpe,10=-1.20, Cpe,1=-1.40
Zone : B, (8.00xh), Cpe,10=-0.80, Cpe,1=-1.10
Zone : C, (14.00xh), Cpe,10=-0.50, Cpe,1=-0.50
Zone : D, (42.00xh), Cpe,10= 0.70, Cpe,1= 1.00
Zone : E, (42.00xh), Cpe,10=-0.30, Cpe,1=-0.30

Wind pressure on wall surfaces we=q(z)·Cpe [kN/m²] (EN1991-1-4, 5.1)

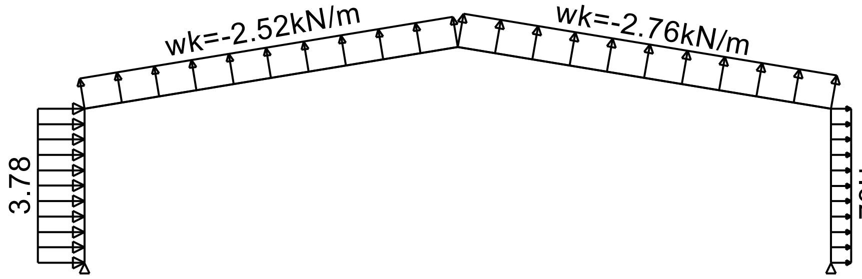
	A		B		C		D		E	
	we,10	we,1	we,10	we,1	we,10	we,1	we,10	we,1	we,10	we,1
z= 5.00~ 0.00m,	-1.080	-1.260	-0.720	-0.990	-0.450	-0.450	0.630	0.900	-0.270	-0.270

4.6. Wind load on frame

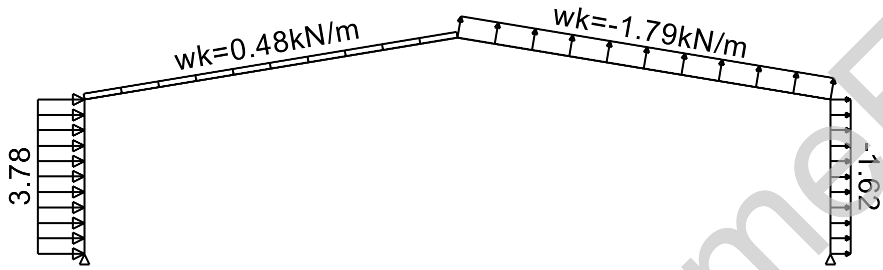
(EC1 EN1991-1-4:2005)

Wind pressure on vertical surface	wk= 0.900 kN/m²	
Wind internal pressure	wi= 0.000 kN/m²	
Spacing of frames	s= 6.000 m	
Left column	Wk1= 0.630x6.000= 3.78kN/m,	Wk1= 0.630x6.000= 3.78kN/m
Left rafter	Wk2=-0.420x6.000= -2.52kN/m,	Wk2= 0.080x6.000= 0.48kN/m
Right rafter	Wk3=-0.460x6.000= -2.76kN/m,	Wk3=-0.299x6.000= -1.79kN/m
Right column	Wk4=-0.270x6.000= -1.62kN/m,	Wk4=-0.270x6.000= -1.62kN/m

Load Qw1 (Wind load)



Load Qw2 (Wind load)



5. Design values of Actions (EN1990 NA Eurocode EN, §6.4, §6.5)

5.1. Load combination factors (EN1990 Tab.A1.1)

Category H (roofs)	*Qk*	$\psi_o=0.00$, $\psi_1=0.00$, $\psi_2=0.00$
Snow loads on buildings	*Qs*	$\psi_o=0.50$, $\psi_1=0.20$, $\psi_2=0.00$
Wind loads on buildings	*Qw*	$\psi_o=0.60$, $\psi_1=0.20$, $\psi_2=0.00$

5.2. Ultimate Limit State (ULS) (EQU)

$$Ed = \gamma G \cdot Gk + \gamma Q \cdot Qk1 + \gamma Q \cdot \psi o \cdot Qk2 \qquad \text{(Eq.6.10)}$$

$\gamma G,sup=1.10$ (Unfavourable)
$\gamma G,inf=0.90$ (Favourable)
$\gamma Q\ \ =1.50$ (Unfavourable)
$\gamma Q\ \ =0.00$ (Favourable)

Load combinations (ULS)(EQU),

Permanent load Gk, Imposed load Qk, Snow load Qs1,Qs2,Qs3, Wind load ,Qw2

L.C. 101: 1.10Gk+1.50Qk (Eq.6.10)
L.C. 102: 1.10Gk+1.50Qs1 (Eq.6.10)
L.C. 103: 1.10Gk+1.50Qs2 (Eq.6.10)
L.C. 104: 1.10Gk+1.50Qs3 (Eq.6.10)
L.C. 105: 1.10Gk+1.50Qw1 (Eq.6.10)
L.C. 106: 1.10Gk+1.50Qw2 (Eq.6.10)
L.C. 111: 0.90Gk+1.50Qw1 (Eq.6.10)
L.C. 121: 1.10Gk+1.50Qs1+0.60x1.50Qw1= 1.10xGk+1.50Qs1+0.90Qw1 (Eq.6.10)
L.C. 122: 1.10Gk+1.50Qs1+0.60x1.50Qw2= 1.10xGk+1.50Qs1+0.90Qw2 (Eq.6.10)
L.C. 123: 1.10Gk+1.50Qs2+0.60x1.50Qw1= 1.10xGk+1.50Qs2+0.90Qw1 (Eq.6.10)
L.C. 124: 1.10Gk+1.50Qs2+0.60x1.50Qw2= 1.10xGk+1.50Qs2+0.90Qw2 (Eq.6.10)
L.C. 125: 1.10Gk+1.50Qs3+0.60x1.50Qw1= 1.10xGk+1.50Qs3+0.90Qw1 (Eq.6.10)
L.C. 126: 1.10Gk+1.50Qs3+0.60x1.50Qw2= 1.10xGk+1.50Qs3+0.90Qw2 (Eq.6.10)
L.C. 127: 1.10Gk+1.50Qw1+0.50x1.50Qs1= 1.10xGk+1.50Qw1+0.75Qs1 (Eq.6.10)
L.C. 128: 1.10Gk+1.50Qw1+0.50x1.50Qs2= 1.10xGk+1.50Qw1+0.75Qs2 (Eq.6.10)
L.C. 129: 1.10Gk+1.50Qw1+0.50x1.50Qs3= 1.10xGk+1.50Qw1+0.75Qs3 (Eq.6.10)
L.C. 130: 1.10Gk+1.50Qw2+0.50x1.50Qs1= 1.10xGk+1.50Qw2+0.75Qs1 (Eq.6.10)
L.C. 131: 1.10Gk+1.50Qw2+0.50x1.50Qs2= 1.10xGk+1.50Qw2+0.75Qs2 (Eq.6.10)
L.C. 132: 1.10Gk+1.50Qw2+0.50x1.50Qs3= 1.10xGk+1.50Qw2+0.75Qs3 (Eq.6.10)

5.3. Ultimate Limit State (ULS) (STR)

$Ed = \gamma G \cdot Gk + \gamma Q \cdot Qk1 + \gamma Q \cdot \psi o \cdot Qk2$ (Eq.6.10)
$Ed = \gamma G \cdot Gk + \gamma Q \cdot \psi o \cdot Qk1 + \gamma Q \cdot \psi o \cdot Qk2$ (Eq.6.10a)
$Ed = \xi \cdot \gamma G \cdot Gk + \gamma Q \cdot Qk1 + \gamma Q \cdot \psi o \cdot Qk2$ (Eq.6.10b)

γG,sup=1.35 (Unfavourable)
γG,inf=1.00 (Favourable)
γQ =1.50 (Unfavourable)
γQ =0.00 (Favourable)
ξ=0.850, ξ·γG=0.850x1.35=1.15

Load combinations (ULS)(STR),
Permanent load Gk, Imposed load Qk, Snow load Qs1,Qs2,Qs3, Wind load ,Qw2

L.C. 201: 1.35Gk+1.50Qk (Eq.6.10)
L.C. 202: 1.35Gk+1.50Qs1 (Eq.6.10)
L.C. 203: 1.35Gk+1.50Qs2 (Eq.6.10)
L.C. 204: 1.35Gk+1.50Qs3 (Eq.6.10)
L.C. 205: 1.35Gk+1.50Qw1 (Eq.6.10)
L.C. 206: 1.35Gk+1.50Qw2 (Eq.6.10)
L.C. 210: 1.00Gk+1.50Qw1 (Eq.6.10)
L.C. 211: 1.35Gk+1.50Qs1+0.60x1.50Qw1= 1.35xGk+1.50Qs1+0.90Qw1 (Eq.6.10)
L.C. 212: 1.35Gk+1.50Qs1+0.60x1.50Qw2= 1.35xGk+1.50Qs1+0.90Qw2 (Eq.6.10)
L.C. 213: 1.35Gk+1.50Qs2+0.60x1.50Qw1= 1.35xGk+1.50Qs2+0.90Qw1 (Eq.6.10)
L.C. 214: 1.35Gk+1.50Qs2+0.60x1.50Qw2= 1.35xGk+1.50Qs2+0.90Qw2 (Eq.6.10)
L.C. 215: 1.35Gk+1.50Qs3+0.60x1.50Qw1= 1.35xGk+1.50Qs3+0.90Qw1 (Eq.6.10)
L.C. 216: 1.35Gk+1.50Qs3+0.60x1.50Qw2= 1.35xGk+1.50Qs3+0.90Qw2 (Eq.6.10)
L.C. 217: 1.35Gk+1.50Qw1+0.50x1.50Qs1= 1.35xGk+1.50Qw1+0.75Qs1 (Eq.6.10)
L.C. 218: 1.35Gk+1.50Qw1+0.50x1.50Qs2= 1.35xGk+1.50Qw1+0.75Qs2 (Eq.6.10)
L.C. 219: 1.35Gk+1.50Qw1+0.50x1.50Qs3= 1.35xGk+1.50Qw1+0.75Qs3 (Eq.6.10)
L.C. 220: 1.35Gk+1.50Qw2+0.50x1.50Qs1= 1.35xGk+1.50Qw2+0.75Qs1 (Eq.6.10)
L.C. 221: 1.35Gk+1.50Qw2+0.50x1.50Qs2= 1.35xGk+1.50Qw2+0.75Qs2 (Eq.6.10)
L.C. 222: 1.35Gk+1.50Qw2+0.50x1.50Qs3= 1.35xGk+1.50Qw2+0.75Qs3 (Eq.6.10)
L.C. 231: 1.35Gk+1.50x0.50Qs1 +1.50x0.60Qw1= 1.35xG+0.75Qs1+0.90Qw1 (Eq.6.10a)
L.C. 232: 1.35Gk+1.50x0.50Qs1 +1.50x0.60Qw2= 1.35xG+0.75Qs1+0.90Qw2 (Eq.6.10a)
L.C. 233: 1.35Gk+1.50x0.50Qs2 +1.50x0.60Qw1= 1.35xG+0.75Qs2+0.90Qw1 (Eq.6.10a)
L.C. 234: 1.35Gk+1.50x0.50Qs2 +1.50x0.60Qw2= 1.35xG+0.75Qs2+0.90Qw2 (Eq.6.10a)
L.C. 235: 1.35Gk+1.50x0.50Qs3 +1.50x0.60Qw1= 1.35xG+0.75Qs3+0.90Qw1 (Eq.6.10a)
L.C. 236: 1.35Gk+1.50x0.50Qs3 +1.50x0.60Qw2= 1.35xG+0.75Qs3+0.90Qw2 (Eq.6.10a)
L.C. 251: 0.850x1.35Gk+1.50Qs1+1.50x0.60Qw1= 1.15xG+1.50Qs1+0.90Qw1 (Eq.6.10b)
L.C. 252: 0.850x1.35Gk+1.50Qs1+1.50x0.60Qw2= 1.15xG+1.50Qs1+0.90Qw2 (Eq.6.10b)
L.C. 253: 0.850x1.35Gk+1.50Qs2+1.50x0.60Qw1= 1.15xG+1.50Qs2+0.90Qw1 (Eq.6.10b)
L.C. 254: 0.850x1.35Gk+1.50Qs2+1.50x0.60Qw2= 1.15xG+1.50Qs2+0.90Qw2 (Eq.6.10b)
L.C. 255: 0.850x1.35Gk+1.50Qs3+1.50x0.60Qw1= 1.15xG+1.50Qs3+0.90Qw1 (Eq.6.10b)
L.C. 256: 0.850x1.35Gk+1.50Qs3+1.50x0.60Qw2= 1.15xG+1.50Qs3+0.90Qw2 (Eq.6.10b)
L.C. 257: 0.850x1.35Gk+1.50Qw1+1.50x0.50Qs1= 1.15xG+1.50Qw1+0.75Qs1 (Eq.6.10b)
L.C. 258: 0.850x1.35Gk+1.50Qw1+1.50x0.50Qs2= 1.15xG+1.50Qw1+0.75Qs2 (Eq.6.10b)
L.C. 259: 0.850x1.35Gk+1.50Qw1+1.50x0.50Qs3= 1.15xG+1.50Qw1+0.75Qs3 (Eq.6.10b)
L.C. 260: 0.850x1.35Gk+1.50Qw2+1.50x0.50Qs1= 1.15xG+1.50Qw2+0.75Qs1 (Eq.6.10b)
L.C. 261: 0.850x1.35Gk+1.50Qw2+1.50x0.50Qs2= 1.15xG+1.50Qw2+0.75Qs2 (Eq.6.10b)
L.C. 262: 0.850x1.35Gk+1.50Qw2+1.50x0.50Qs3= 1.15xG+1.50Qw2+0.75Qs3 (Eq.6.10b)

5.4. Serviceability Limit State (SLS)

$Ed = Gk + Qk1 + \psi o \cdot Qk2 + \psi o \cdot Qk3$ (Characteristic combination) (Eq.6.14b)
$Ed = Gk + \psi 1 \cdot Qk1 + \psi 2 \cdot Qk2 + \psi 2 \cdot Qk3$ (Frequent combination) (Eq.6.15b)
$Ed = Gk + \psi 2 \cdot Qk1 + \psi 2 \cdot Qk2 + \psi 2 \cdot Qk3$ (Quasi-permanent combination) (Eq.6.16b)

Load combinations (SLS)

Permanent load Gk, Imposed load Qk, Snow load Qs1,Qs2,Qs3, Wind load ,Qw2

L.C. 301: Gk+Qk (Eq.6.14a)
L.C. 302: Gk+Qs1 (Eq.6.14a)
L.C. 303: Gk+Qs2 (Eq.6.14a)
L.C. 304: Gk+Qs3 (Eq.6.14a)
L.C. 305: Gk+Qw1 (Eq.6.14a)
L.C. 306: Gk+Qw2 (Eq.6.14a)
L.C. 311: G + Qs1 + 0.60Qw1 (Eq.6.14a)
L.C. 312: G + Qs1 + 0.60Qw2 (Eq.6.14a)
L.C. 313: G + Qs2 + 0.60Qw1 (Eq.6.14a)
L.C. 314: G + Qs2 + 0.60Qw2 (Eq.6.14a)
L.C. 315: G + Qs3 + 0.60Qw1 (Eq.6.14a)
L.C. 316: G + Qs3 + 0.60Qw2 (Eq.6.14a)
L.C. 317: G + Qw1 + 0.50Qs1 (Eq.6.14a)
L.C. 318: G + Qw1 + 0.50Qs2 (Eq.6.14a)
L.C. 319: G + Qw1 + 0.50Qs3 (Eq.6.14a)
L.C. 320: G + Qw2 + 0.50Qs1 (Eq.6.14a)
L.C. 321: G + Qw2 + 0.50Qs2 (Eq.6.14a)
L.C. 322: G + Qw2 + 0.50Qs3 (Eq.6.14a)
L.C. 331: G + 0.50Qs1 + 0.30Qw1 (Eq.6.15a)
L.C. 332: G + 0.50Qs1 + 0.30Qw2 (Eq.6.15a)
L.C. 333: G + 0.50Qs2 + 0.30Qw1 (Eq.6.15a)
L.C. 334: G + 0.50Qs2 + 0.30Qw2 (Eq.6.15a)
L.C. 335: G + 0.50Qs3 + 0.30Qw1 (Eq.6.15a)
L.C. 336: G + 0.50Qs3 + 0.30Qw2 (Eq.6.15a)
L.C. 337: G + 0.20Qw1 + 0.00Qs1 (Eq.6.15a)
L.C. 338: G + 0.20Qw1 + 0.00Qs2 (Eq.6.15a)
L.C. 339: G + 0.20Qw1 + 0.00Qs3 (Eq.6.15a)
L.C. 340: G + 0.20Qw2 + 0.00Qs1 (Eq.6.15a)
L.C. 341: G + 0.20Qw2 + 0.00Qs2 (Eq.6.15a)
L.C. 342: G + 0.20Qw2 + 0.00Qs3 (Eq.6.15a)
L.C. 351: G + 0.00Qs1 + 0.30Qw1 (Eq.6.16a)
L.C. 352: G + 0.00Qs1 + 0.30Qw2 (Eq.6.16a)
L.C. 353: G + 0.00Qs2 + 0.30Qw1 (Eq.6.16a)
L.C. 354: G + 0.00Qs2 + 0.30Qw2 (Eq.6.16a)
L.C. 355: G + 0.00Qs3 + 0.30Qw1 (Eq.6.16a)
L.C. 356: G + 0.00Qs3 + 0.30Qw2 (Eq.6.16a)

5.5. Ultimate Limit State (ULS)Seismic situation

$Ed = Gk + Aed + \psi 2 \cdot Qk1 + \psi 2 \cdot Qk2 + \psi 2 \cdot Qk3$ (Eq.6.12b)

Snow load Qs, Wind load Qw, Seismic load Aed

L.C. 601: Gk + 0.20Qs1 + Aed (Eq.6.14a)

5.6. Summary of load combination

Permanent load Gk, Imposed load Qk, Snow load Qs1,Qs2,Qs3, Wind load ,Qw2

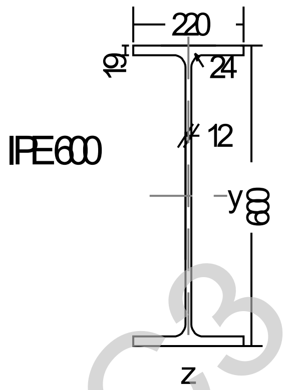
```
 1 L.C. 101 (ULS)(EQU) 1.10Gk+1.50Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2
 2 L.C. 102 (ULS)(EQU) 1.10Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2
 3 L.C. 103 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.00Qw1+0.00Qw2
 4 L.C. 104 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.00Qw1+0.00Qw2
 5 L.C. 105 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+1.50Qw1+0.00Qw2
 6 L.C. 106 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+1.50Qw2
 7 L.C. 111 (ULS)(EQU) 0.90Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+1.50Qw1+0.00Qw2
 8 L.C. 121 (ULS)(EQU) 1.10Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.90Qw1+0.00Qw2
 9 L.C. 122 (ULS)(EQU) 1.10Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.90Qw2
10 L.C. 123 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.90Qw1+0.00Qw2
11 L.C. 124 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.00Qw1+0.90Qw2
12 L.C. 125 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.90Qw1+0.00Qw2
13 L.C. 126 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.00Qw1+0.90Qw2
14 L.C. 127 (ULS)(EQU) 1.10Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+1.50Qw1+0.00Qw2
15 L.C. 128 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+1.50Qw1+0.00Qw2
16 L.C. 129 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+1.50Qw1+0.00Qw2
17 L.C. 130 (ULS)(EQU) 1.10Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+0.00Qw1+1.50Qw2
18 L.C. 131 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+0.00Qw1+1.50Qw2
19 L.C. 132 (ULS)(EQU) 1.10Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+0.00Qw1+1.50Qw2

20 L.C. 201 (ULS)(STR) 1.35Gk+1.50Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2
21 L.C. 202 (ULS)(STR) 1.35Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2
22 L.C. 203 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.00Qw1+0.00Qw2
23 L.C. 204 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.00Qw1+0.00Qw2
24 L.C. 205 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+1.50Qw1+0.00Qw2
25 L.C. 206 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+1.50Qw2
26 L.C. 210 (ULS)(STR) 1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+1.50Qw1+0.00Qw2
27 L.C. 211 (ULS)(STR) 1.35Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.90Qw1+0.00Qw2
28 L.C. 212 (ULS)(STR) 1.35Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.90Qw2
29 L.C. 213 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.90Qw1+0.00Qw2
30 L.C. 214 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.00Qw1+0.90Qw2
31 L.C. 215 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.90Qw1+0.00Qw2
32 L.C. 216 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.00Qw1+0.90Qw2
33 L.C. 217 (ULS)(STR) 1.35Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+1.50Qw1+0.00Qw2
34 L.C. 218 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+1.50Qw1+0.00Qw2
35 L.C. 219 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+1.50Qw1+0.00Qw2
36 L.C. 220 (ULS)(STR) 1.35Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+0.00Qw1+1.50Qw2
37 L.C. 221 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+0.00Qw1+1.50Qw2
38 L.C. 222 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+0.00Qw1+1.50Qw2
39 L.C. 231 (ULS)(STR) 1.35Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+0.90Qw1+0.00Qw2
40 L.C. 232 (ULS)(STR) 1.35Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.90Qw2
41 L.C. 233 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+0.90Qw1+0.00Qw2
42 L.C. 234 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+0.00Qw1+0.90Qw2
43 L.C. 235 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+0.90Qw1+0.00Qw2
44 L.C. 236 (ULS)(STR) 1.35Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+0.00Qw1+0.90Qw2
45 L.C. 251 (ULS)(STR) 1.15Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.90Qw1+0.00Qw2
46 L.C. 252 (ULS)(STR) 1.15Gk+0.00Qk+1.50Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.90Qw2
47 L.C. 253 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.90Qw1+0.00Qw2
48 L.C. 254 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+1.50Qs2+0.00Qs3+0.00Qw1+0.90Qw2
49 L.C. 255 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.90Qw1+0.00Qw2
50 L.C. 256 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+0.00Qs2+1.50Qs3+0.00Qw1+0.90Qw2
51 L.C. 257 (ULS)(STR) 1.15Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+1.50Qw1+0.00Qw2
52 L.C. 258 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+1.50Qw1+0.00Qw2
53 L.C. 259 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+1.50Qw1+0.00Qw2
54 L.C. 260 (ULS)(STR) 1.15Gk+0.00Qk+0.75Qs1+0.00Qs2+0.00Qs3+0.00Qw1+1.50Qw2
55 L.C. 261 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+0.75Qs2+0.00Qs3+0.00Qw1+1.50Qw2
56 L.C. 262 (ULS)(STR) 1.15Gk+0.00Qk+0.00Qs1+0.00Qs2+0.75Qs3+0.00Qw1+1.50Qw2

57 L.C. 301 (SLS)      1.00Gk+1.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2
58 L.C. 302 (SLS)      1.00Gk+0.00Qk+1.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2
59 L.C. 303 (SLS)      1.00Gk+0.00Qk+0.00Qs1+1.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2
60 L.C. 304 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+1.00Qs3+0.00Qw1+0.00Qw2
61 L.C. 305 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+1.00Qw1+0.00Qw2
62 L.C. 306 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+1.00Qw2
63 L.C. 311 (SLS)      1.00Gk+0.00Qk+1.00Qs1+0.00Qs2+0.00Qs3+0.60Qw1+0.00Qw2
64 L.C. 312 (SLS)      1.00Gk+0.00Qk+1.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.60Qw2
65 L.C. 313 (SLS)      1.00Gk+0.00Qk+0.00Qs1+1.00Qs2+0.00Qs3+0.60Qw1+0.00Qw2
66 L.C. 314 (SLS)      1.00Gk+0.00Qk+0.00Qs1+1.00Qs2+0.00Qs3+0.00Qw1+0.60Qw2
67 L.C. 315 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+1.00Qs3+0.60Qw1+0.00Qw2
68 L.C. 316 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+1.00Qs3+0.00Qw1+0.60Qw2
69 L.C. 317 (SLS)      1.00Gk+0.00Qk+0.50Qs1+0.00Qs2+0.00Qs3+1.00Qw1+0.00Qw2
70 L.C. 318 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.50Qs2+0.00Qs3+1.00Qw1+0.00Qw2
71 L.C. 319 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.50Qs3+1.00Qw1+0.00Qw2
72 L.C. 320 (SLS)      1.00Gk+0.00Qk+0.50Qs1+0.00Qs2+0.00Qs3+0.00Qw1+1.00Qw2
73 L.C. 321 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.50Qs2+0.00Qs3+0.00Qw1+1.00Qw2
74 L.C. 322 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.50Qs3+0.00Qw1+1.00Qw2
75 L.C. 331 (SLS)      1.00Gk+0.00Qk+0.50Qs1+0.00Qs2+0.00Qs3+0.30Qw1+0.00Qw2
76 L.C. 332 (SLS)      1.00Gk+0.00Qk+0.50Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.30Qw2
77 L.C. 333 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.50Qs2+0.00Qs3+0.30Qw1+0.00Qw2
78 L.C. 334 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.50Qs2+0.00Qs3+0.00Qw1+0.30Qw2
79 L.C. 335 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.50Qs3+0.30Qw1+0.00Qw2
80 L.C. 336 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.50Qs3+0.00Qw1+0.30Qw2
81 L.C. 337 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.20Qw1+0.00Qw2
82 L.C. 338 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.20Qw1+0.00Qw2
83 L.C. 339 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.20Qw1+0.00Qw2
84 L.C. 340 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.20Qw2
85 L.C. 341 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.20Qw2
86 L.C. 342 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.20Qw2
87 L.C. 351 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.30Qw1+0.00Qw2
88 L.C. 352 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.30Qw2
89 L.C. 353 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.30Qw1+0.00Qw2
90 L.C. 354 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.30Qw2
91 L.C. 355 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.30Qw1+0.00Qw2
92 L.C. 356 (SLS)      1.00Gk+0.00Qk+0.00Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.30Qw2

93 L.C. 601 (SESM)     1.00Gk+0.00Qk+0.20Qs1+0.00Qs2+0.00Qs3+0.00Qw1+0.00Qw2 + Aed
```

6. Steel sections

6.1. Column section

Cross-section properties

Cross-section IPE 600-S 355

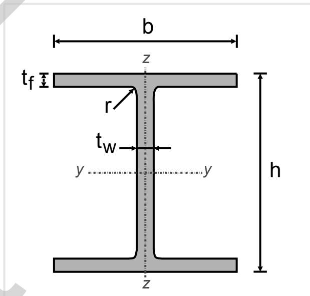


Dimensions of cross section

Depth of cross section	h=	600.00 mm
Width of cross section	b=	220.00 mm
Web depth	hw=	581.00 mm
Depth of straight portion of web	dw=	514.00 mm
Web thickness	tw=	12.00 mm
Flange thickness	tf=	19.00 mm
Radius of root fillet	r=	24.00 mm
Mass	=	122.00 Kg/m

Properties of cross section

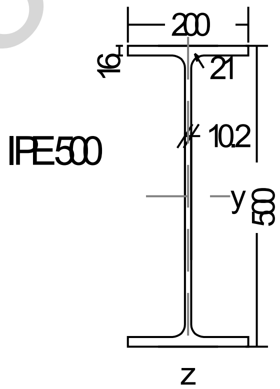
Area	A= 15600 mm^2		
Second moment of area	Iy=920.80×10^6 mm^4	Iz=33.870×10^6 mm^4	
Section modulus	Wy=3069.0×10^3 mm^3	Wz=307.90×10^3 mm^3	
Plastic section modulus	Wpy=3512.0×10^3 mm^3	Wpz=485.60×10^3 mm^3	
Radius of gyration	iy= 243.0 mm	iz= 46.6 mm	
Shear area	Avz= 8380 mm^2	Avy= 8360 mm^2	
Torsional constant	It= 1.654×10^6 mm^4	ip= 247 mm	
Warping constant	Iw=2845.5×10^9 mm^6		



6.2. Rafter section

Cross-section properties

Cross-section IPE 500-S 355

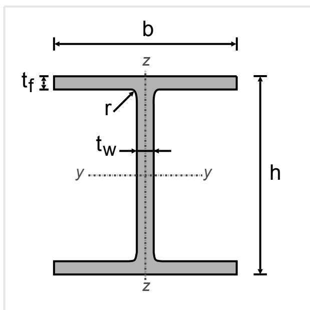


Dimensions of cross section

Depth of cross section	h=	500.00 mm
Width of cross section	b=	200.00 mm
Web depth	hw=	484.00 mm
Depth of straight portion of web	dw=	426.00 mm
Web thickness	tw=	10.20 mm
Flange thickness	tf=	16.00 mm
Radius of root fillet	r=	21.00 mm
Mass	=	90.70 Kg/m

Properties of cross section

Area	A= 11550 mm^2		
Second moment of area	Iy=482.00×10^6 mm^4	Iz=21.420×10^6 mm^4	
Section modulus	Wy=1928.0×10^3 mm^3	Wz=214.20×10^3 mm^3	
Plastic section modulus	Wpy=2194.0×10^3 mm^3	Wpz=335.90×10^3 mm^3	
Radius of gyration	iy= 204.3 mm	iz= 43.1 mm	
Shear area	Avz= 5985 mm^2	Avy= 6400 mm^2	
Torsional constant	It= 0.893×10^6 mm^4	ip= 209 mm	
Warping constant	Iw=1249.4×10^9 mm^6		

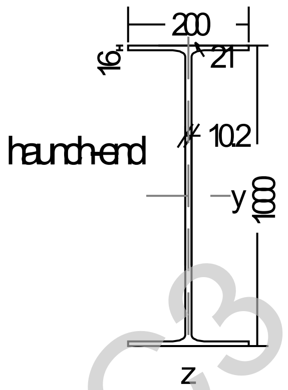


6.3. Haunch section at haunch end

Cross-section properties

Welded section

Cross-section haunch-end-S 355

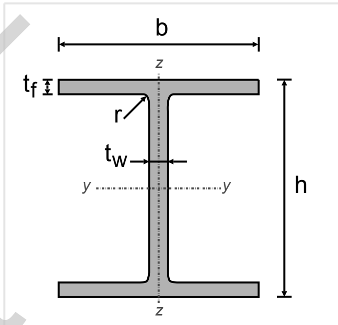


Dimensions of cross section

Depth of cross section	h=	1000.00	mm
Width of cross section	b=	200.00	mm
Web depth	hw=	984.00	mm
Depth of straight portion of web	dw=	908.60	mm
Web thickness	tw=	10.20	mm
Flange thickness	tf=	16.00	mm
Radius of root fillet	r=	21.00	mm
Mass	=	127.83	Kg/m

Properties of cross section

Area	A= 16274	mm^2		
Second moment of area	Iy=$2320.3x10^6$	mm^4	Iz=$21.419x10^6$	mm^4
Section modulus	Wy=$4640.7x10^3$	mm^3	Wz=$214.19x10^3$	mm^3
Plastic section modulus	Wpy=$5538.2x10^3$	mm^3	Wpz=$345.18x10^3$	mm^3
Radius of gyration	iy= 377.6	mm	iz= 36.3	mm
Shear area	Avz= 10037	mm^2	Avy= 6400	mm^2
Torsional constant	It= $0.879x10^6$	mm^4	ip= 379	mm
Warping constant	Iw=$5164.0x10^9$	mm^6		
Weld	a= $21.0x10^9$	mm		

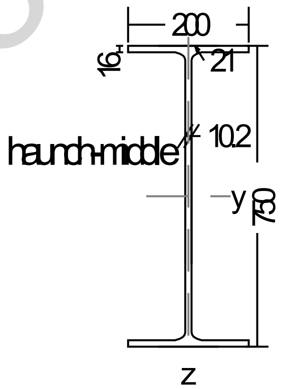


6.4. Haunch section at haunch-middle

Cross-section properties

Welded section

Cross-section haunch-middle-S 355

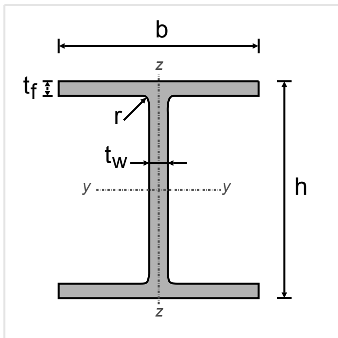


Dimensions of cross section

Depth of cross section	h=	750.00	mm
Width of cross section	b=	200.00	mm
Web depth	hw=	734.00	mm
Depth of straight portion of web	dw=	658.60	mm
Web thickness	tw=	10.20	mm
Flange thickness	tf=	16.00	mm
Radius of root fillet	r=	21.00	mm
Mass	=	107.80	Kg/m

Properties of cross section

Area	A= 13724	mm^2		
Second moment of area	Iy=$1176.8x10^6$	mm^4	Iz=$21.397x10^6$	mm^4
Section modulus	Wy=$3138.1x10^3$	mm^3	Wz=$213.97x10^3$	mm^3
Plastic section modulus	Wpy=$3663.4x10^3$	mm^3	Wpz=$338.68x10^3$	mm^3
Radius of gyration	iy= 292.8	mm	iz= 39.5	mm
Shear area	Avz= 7487	mm^2	Avy= 6400	mm^2
Torsional constant	It= $0.790x10^6$	mm^4	ip= 295	mm
Warping constant	Iw=$2873.4x10^9$	mm^6		
Weld	a= $21.0x10^9$	mm		



7. Finite Element Analysis

(EN1993-1-1, §5.1)

The 2-dimensional finite element program FRAME2Dexpres© RUNET is used for the analysis.
The column bases are assumed to be pinned.
The connection of rafter to column are assumed to be fully rigid.
The increased stiffness of the haunches is taken into account.
The global or local imperfections are taken into accountby by equivalent loads.

Linear-elastic analysis is used for the design of static loads.

7.1. Data used for elastic analysis

Nodal points

Node	x [mm]	y[mm]
1	0	0
2	0	5000
3	12000	7000
4	24000	5000
5	24000	0

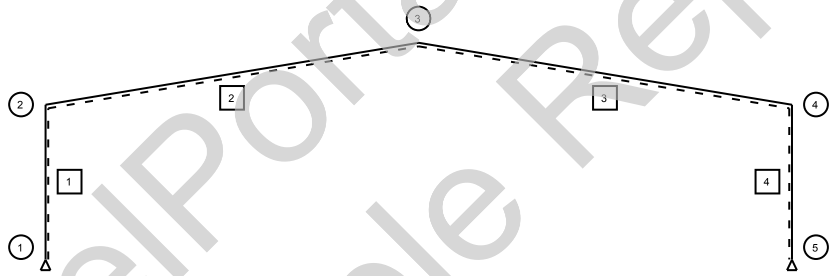
Supports

Node	kind	ux[mm]	uy[mm]	ur[rad]
1	pin ux=uy=0			
5	pin ux=uy=0			

Elements

Element	node 1	node 2	length(mm)	angle(°)	E(GPa)	A(mm^2)	I(mm^4)
1	1	2	5000	90.00	210	15600	920800x10^3
2	2	3	12166	9.46	210	11550	482000x10^3
3	3	4	12166	350.54	210	11550	482000x10^3
4	4	5	5000	270.00	210	15600	920800x10^3

Finite element model (FEM) Linear elastic analysis



8. Results of static-linear-elastic analysis

8.1. Displacements [mm]

L.C.	Horiz. defl. Column Dx mm	Vert. defl. Apex Dy mm	Bending Defl. of Rafter w mm
101 ULS-EQU	6.896	43.241	3.469
102 ULS-EQU	9.370	58.758	4.714
103 ULS-EQU	11.321	48.643	4.714
104 ULS-EQU	11.321	48.643	4.714
105 ULS-EQU	6.204	11.521	0.864
106 ULS-EQU	6.765	9.337	1.875
111 ULS-EQU	6.734	14.848	1.131
121 ULS-EQU	9.043	40.867	3.437
122 ULS-EQU	11.679	53.382	4.958
123 ULS-EQU	10.995	30.752	3.437
124 ULS-EQU	13.630	43.267	4.958
125 ULS-EQU	5.849	30.752	3.315
126 ULS-EQU	7.224	43.267	3.804
127 ULS-EQU	5.599	8.709	0.963
128 ULS-EQU	6.575	3.652	0.963
129 ULS-EQU	3.010	3.652	0.759
130 ULS-EQU	9.991	29.568	3.498
131 ULS-EQU	10.967	24.510	3.498
132 ULS-EQU	7.402	24.510	2.686
201 ULS-STR	7.559	47.400	3.802
202 ULS-STR	10.033	62.917	5.047
203 ULS-STR	11.984	52.802	5.047
204 ULS-STR	11.984	52.802	5.047
205 ULS-STR	5.541	7.363	0.530
206 ULS-STR	7.428	13.496	2.208
210 ULS-STR	6.469	13.185	0.997
211 ULS-STR	9.707	45.026	3.770
212 ULS-STR	12.342	57.541	5.291
213 ULS-STR	11.658	34.911	3.770
214 ULS-STR	14.293	47.425	5.291
215 ULS-STR	6.512	34.911	3.648
216 ULS-STR	7.887	47.425	4.137
217 ULS-STR	6.262	12.868	1.296
218 ULS-STR	7.238	7.810	1.296
219 ULS-STR	3.673	7.810	1.093
220 ULS-STR	10.654	33.726	3.831
221 ULS-STR	11.630	28.668	3.831
222 ULS-STR	8.066	28.668	3.020
231 ULS-STR	6.480	24.795	2.147
232 ULS-STR	9.115	37.310	3.668
233 ULS-STR	7.456	19.738	2.147
234 ULS-STR	10.091	32.253	3.668
235 ULS-STR	3.891	19.738	2.025
236 ULS-STR	6.527	32.253	2.857
251 ULS-STR	9.176	41.699	3.504
252 ULS-STR	11.811	54.214	5.025
253 ULS-STR	11.127	31.584	3.504
254 ULS-STR	13.762	44.099	5.025
255 ULS-STR	5.981	31.584	3.381
256 ULS-STR	7.356	44.099	3.870
257 ULS-STR	5.732	9.541	1.029
258 ULS-STR	6.707	4.483	1.029
259 ULS-STR	3.143	4.483	0.826
260 ULS-STR	10.124	30.399	3.565
261 ULS-STR	11.099	25.342	3.565
262 ULS-STR	7.535	25.342	2.753
301 SLS	5.304	33.263	2.668
302 SLS	6.954	43.608	3.498
303 SLS	8.255	36.865	3.498
304 SLS	8.255	36.864	3.498
305 SLS	3.429	3.245	0.220
306 SLS	5.217	10.660	1.606
311 SLS	6.736	31.681	2.647
312 SLS	8.493	40.024	3.661
313 SLS	8.037	24.937	2.647
314 SLS	9.794	33.280	3.661
315 SLS	4.606	24.937	2.566
316 SLS	5.523	33.280	2.891
317 SLS	4.440	10.242	0.998
318 SLS	5.090	6.870	0.998
319 SLS	2.714	6.870	0.862
320 SLS	7.368	24.147	2.688
321 SLS	8.018	20.776	2.688
322 SLS	5.642	20.776	2.147
331 SLS	4.694	24.157	1.991
332 SLS	5.573	28.329	2.498
333 SLS	5.345	20.786	1.991
334 SLS	6.223	24.957	2.498
335 SLS	3.629	20.786	1.950
336 SLS	4.088	24.957	2.113
337 SLS	2.580	12.658	1.050
338 SLS	2.580	12.658	1.050
339 SLS	2.580	12.658	1.050
340 SLS	3.165	15.439	1.388
341 SLS	3.165	15.439	1.388
342 SLS	3.165	15.439	1.388
351 SLS	2.543	10.670	0.909
352 SLS	3.422	14.842	1.416
353 SLS	2.543	10.670	0.909
354 SLS	3.422	14.842	1.416
355 SLS	2.543	10.670	0.909
356 SLS	3.422	14.842	1.416

8.2. Reactions at the supports

Horizontal Force Hed [kN], Vertical Force Ved [kN], Moment Med [kNm]

	Left support 1			Right support 2		
L.C.	Hed,1 kN	Ved,1 kN	Med,1 kNm	Hed,2 kN	Ved,2 kN	Med,2 kNm
101 ULS-EQU	51.7	82.5	0.0	-51.7	82.5	0.0
102 ULS-EQU	70.3	109.8	0.0	-70.3	109.8	0.0
103 ULS-EQU	58.2	100.9	0.0	-58.2	83.1	0.0
104 ULS-EQU	58.2	83.1	0.0	-58.2	100.9	0.0
105 ULS-EQU	-31.0	-9.2	0.0	3.3	-8.3	0.0
106 ULS-EQU	-2.9	37.2	0.0	-18.6	16.6	0.0
111 ULS-EQU	-35.0	-16.2	0.0	7.3	-15.4	0.0
121 ULS-EQU	38.6	81.0	0.0	-55.2	81.5	0.0
122 ULS-EQU	55.4	108.8	0.0	-68.3	96.5	0.0
123 ULS-EQU	26.5	72.1	0.0	-43.1	54.9	0.0
124 ULS-EQU	43.3	99.9	0.0	-56.2	69.9	0.0
125 ULS-EQU	26.5	54.4	0.0	-43.1	72.6	0.0
126 ULS-EQU	43.3	82.2	0.0	-56.2	87.6	0.0
127 ULS-EQU	-6.8	26.3	0.0	-20.9	27.2	0.0
128 ULS-EQU	-12.8	21.9	0.0	-14.8	13.9	0.0
129 ULS-EQU	-12.8	13.0	0.0	-14.8	22.7	0.0
130 ULS-EQU	21.3	72.7	0.0	-42.8	52.2	0.0
131 ULS-EQU	15.2	68.2	0.0	-36.7	38.8	0.0
132 ULS-EQU	15.2	59.4	0.0	-36.7	47.7	0.0
201 ULS-STR	56.7	91.3	0.0	-56.7	91.3	0.0
202 ULS-STR	75.3	118.6	0.0	-75.3	118.6	0.0
203 ULS-STR	63.2	109.7	0.0	-63.2	91.9	0.0
204 ULS-STR	63.2	91.9	0.0	-63.2	109.7	0.0
205 ULS-STR	-26.0	-0.4	0.0	-1.6	0.5	0.0
206 ULS-STR	2.0	46.0	0.0	-23.6	25.4	0.0
210 ULS-STR	-33.0	-12.7	0.0	5.3	-11.9	0.0
211 ULS-STR	43.6	89.8	0.0	-60.1	90.3	0.0
212 ULS-STR	60.4	117.6	0.0	-73.3	105.3	0.0
213 ULS-STR	31.5	80.9	0.0	-48.0	63.7	0.0
214 ULS-STR	48.3	108.7	0.0	-61.2	78.7	0.0
215 ULS-STR	31.5	63.2	0.0	-48.0	81.4	0.0
216 ULS-STR	48.3	91.0	0.0	-61.2	96.4	0.0
217 ULS-STR	-1.8	35.1	0.0	-25.8	36.0	0.0
218 ULS-STR	-7.8	30.7	0.0	-19.8	22.7	0.0
219 ULS-STR	-7.8	21.8	0.0	-19.8	31.5	0.0
220 ULS-STR	26.2	81.5	0.0	-47.8	61.0	0.0
221 ULS-STR	20.2	77.0	0.0	-41.7	47.6	0.0
222 ULS-STR	20.2	68.2	0.0	-41.7	56.5	0.0
231 ULS-STR	19.4	54.3	0.0	-35.9	54.8	0.0
232 ULS-STR	36.2	82.1	0.0	-49.1	69.8	0.0
233 ULS-STR	13.3	49.9	0.0	-29.9	41.5	0.0
234 ULS-STR	30.1	77.7	0.0	-43.0	56.5	0.0
235 ULS-STR	13.3	41.0	0.0	-29.9	50.4	0.0
236 ULS-STR	30.1	68.8	0.0	-43.0	65.4	0.0
251 ULS-STR	39.6	82.8	0.0	-56.2	83.3	0.0
252 ULS-STR	56.4	110.6	0.0	-69.3	98.3	0.0
253 ULS-STR	27.5	73.9	0.0	-44.1	56.6	0.0
254 ULS-STR	44.3	101.7	0.0	-57.2	71.6	0.0
255 ULS-STR	27.5	56.1	0.0	-44.1	74.4	0.0
256 ULS-STR	44.3	83.9	0.0	-57.2	89.4	0.0
257 ULS-STR	-5.8	28.1	0.0	-21.9	28.9	0.0
258 ULS-STR	-11.8	23.7	0.0	-15.8	15.6	0.0
259 ULS-STR	-11.8	14.8	0.0	-15.8	24.5	0.0
260 ULS-STR	22.3	74.4	0.0	-43.8	53.9	0.0
261 ULS-STR	16.2	70.0	0.0	-37.7	40.6	0.0
262 ULS-STR	16.2	61.1	0.0	-37.7	49.5	0.0
301 SLS	39.8	64.4	0.0	-39.8	64.4	0.0
302 SLS	52.2	82.6	0.0	-52.2	82.6	0.0
303 SLS	44.1	76.6	0.0	-44.1	64.8	0.0
304 SLS	44.1	64.8	0.0	-44.1	76.6	0.0
305 SLS	-15.3	3.3	0.0	-3.1	3.8	0.0
306 SLS	3.4	34.2	0.0	-17.7	20.5	0.0
311 SLS	31.0	63.4	0.0	-42.1	63.7	0.0
312 SLS	42.2	81.9	0.0	-50.9	73.7	0.0
313 SLS	23.0	57.5	0.0	-34.0	46.0	0.0
314 SLS	34.2	76.0	0.0	-42.8	56.0	0.0
315 SLS	23.0	45.6	0.0	-34.0	57.8	0.0
316 SLS	34.2	64.2	0.0	-42.8	67.8	0.0
317 SLS	0.8	26.9	0.0	-19.2	27.5	0.0
318 SLS	-3.2	24.0	0.0	-15.2	18.6	0.0
319 SLS	-3.2	18.1	0.0	-15.2	24.5	0.0
320 SLS	19.5	57.8	0.0	-33.8	44.2	0.0
321 SLS	15.5	54.9	0.0	-29.8	35.3	0.0
322 SLS	15.5	49.0	0.0	-29.8	41.2	0.0
331 SLS	25.5	49.3	0.0	-31.0	49.5	0.0
332 SLS	31.1	58.6	0.0	-35.4	54.5	0.0
333 SLS	21.4	46.3	0.0	-27.0	40.6	0.0
334 SLS	27.0	55.6	0.0	-31.3	45.6	0.0
335 SLS	21.4	40.4	0.0	-27.0	46.5	0.0
336 SLS	27.0	49.7	0.0	-31.3	51.5	0.0
337 SLS	12.9	28.8	0.0	-16.5	28.9	0.0
338 SLS	12.9	28.8	0.0	-16.5	28.9	0.0
339 SLS	12.9	28.8	0.0	-16.5	28.9	0.0
340 SLS	16.6	35.0	0.0	-19.5	32.3	0.0
341 SLS	16.6	35.0	0.0	-19.5	32.3	0.0
342 SLS	16.6	35.0	0.0	-19.5	32.3	0.0
351 SLS	9.3	25.6	0.0	-14.9	25.8	0.0
352 SLS	14.9	34.9	0.0	-19.2	30.8	0.0
353 SLS	9.3	25.6	0.0	-14.9	25.8	0.0
354 SLS	14.9	34.9	0.0	-19.2	30.8	0.0
355 SLS	9.3	25.6	0.0	-14.9	25.8	0.0
356 SLS	14.9	34.9	0.0	-19.2	30.8	0.0

8.3. Axial forces Ned [kN]

L.C.		Left column 1 Ned,1	Left rafter 2 Ned,2	Right rafter 3 Ned,3	Right column 4 Ned,4
101	ULS-EQU	-79.2	-57.3	-57.3	-79.2
102	ULS-EQU	-106.5	-77.8	-77.8	-106.5
103	ULS-EQU	-97.6	-64.4	-64.4	-79.8
104	ULS-EQU	-79.8	-64.4	-64.4	-97.6
105	ULS-EQU	12.5	0.4	0.2	11.6
106	ULS-EQU	-33.9	-26.0	-22.7	-13.3
111	ULS-EQU	18.9	4.8	4.6	18.1
121	ULS-EQU	-77.7	-63.1	-63.1	-78.2
122	ULS-EQU	-105.5	-78.9	-76.9	-93.2
123	ULS-EQU	-68.8	-49.7	-49.7	-51.6
124	ULS-EQU	-96.6	-65.5	-63.5	-66.6
125	ULS-EQU	-51.1	-49.7	-49.7	-69.3
126	ULS-EQU	-78.9	-65.5	-63.5	-84.3
127	ULS-EQU	-23.0	-26.4	-26.6	-23.9
128	ULS-EQU	-18.6	-19.7	-19.9	-10.6
129	ULS-EQU	-9.7	-19.7	-19.9	-19.4
130	ULS-EQU	-69.4	-52.8	-49.5	-48.9
131	ULS-EQU	-64.9	-46.1	-42.8	-35.5
132	ULS-EQU	-56.1	-46.1	-42.8	-44.4
201	ULS-STR	-87.3	-62.8	-62.8	-87.3
202	ULS-STR	-114.5	-83.3	-83.3	-114.5
203	ULS-STR	-105.6	-69.9	-69.9	-87.9
204	ULS-STR	-87.9	-69.9	-69.9	-105.6
205	ULS-STR	4.4	-5.1	-5.3	3.6
206	ULS-STR	-41.9	-31.5	-28.2	-21.4
210	ULS-STR	15.7	2.6	2.4	14.9
211	ULS-STR	-85.8	-68.6	-68.7	-86.3
212	ULS-STR	-113.6	-84.4	-82.4	-101.3
213	ULS-STR	-76.9	-55.2	-55.3	-59.6
214	ULS-STR	-104.7	-71.0	-69.0	-74.6
215	ULS-STR	-59.1	-55.2	-55.3	-77.4
216	ULS-STR	-86.9	-71.0	-69.0	-92.4
217	ULS-STR	-31.1	-31.9	-32.1	-31.9
218	ULS-STR	-26.6	-25.2	-25.4	-18.6
219	ULS-STR	-17.8	-25.2	-25.4	-27.5
220	ULS-STR	-77.4	-58.3	-55.0	-56.9
221	ULS-STR	-73.0	-51.6	-48.3	-43.6
222	ULS-STR	-64.1	-51.6	-48.3	-52.5
231	ULS-STR	-50.2	-41.8	-41.9	-50.8
232	ULS-STR	-78.1	-57.6	-55.6	-65.7
233	ULS-STR	-45.8	-35.1	-35.2	-37.4
234	ULS-STR	-73.6	-50.9	-48.9	-52.4
235	ULS-STR	-36.9	-35.1	-35.2	-46.3
236	ULS-STR	-64.7	-50.9	-48.9	-61.3
251	ULS-STR	-79.3	-64.2	-64.3	-79.8
252	ULS-STR	-107.1	-80.0	-78.0	-94.8
253	ULS-STR	-70.4	-50.8	-50.9	-53.2
254	ULS-STR	-98.3	-66.6	-64.6	-68.2
255	ULS-STR	-52.7	-50.8	-50.9	-71.0
256	ULS-STR	-80.5	-66.6	-64.6	-85.9
257	ULS-STR	-24.6	-27.5	-27.7	-25.5
258	ULS-STR	-20.2	-20.8	-21.0	-12.2
259	ULS-STR	-11.3	-20.8	-21.0	-21.1
260	ULS-STR	-71.0	-53.9	-50.6	-50.5
261	ULS-STR	-66.6	-47.2	-43.9	-37.2
262	ULS-STR	-57.7	-47.2	-43.9	-46.0
301	SLS	-61.4	-44.1	-44.1	-61.4
302	SLS	-79.6	-57.8	-57.8	-79.6
303	SLS	-73.6	-48.8	-48.8	-61.8
304	SLS	-61.8	-48.8	-48.8	-73.6
305	SLS	-0.3	-5.6	-5.7	-0.8
306	SLS	-31.2	-23.2	-21.0	-17.5
311	SLS	-60.4	-47.9	-48.0	-60.7
312	SLS	-78.9	-58.5	-57.1	-70.7
313	SLS	-54.5	-39.0	-39.0	-43.0
314	SLS	-73.0	-49.5	-48.2	-53.0
315	SLS	-42.6	-39.0	-39.0	-54.8
316	SLS	-61.2	-49.5	-48.2	-64.8
317	SLS	-23.9	-23.5	-23.6	-24.5
318	SLS	-21.0	-19.0	-19.1	-15.6
319	SLS	-15.1	-19.0	-19.1	-21.5
320	SLS	-54.8	-41.1	-38.8	-41.2
321	SLS	-51.9	-36.6	-34.4	-32.3
322	SLS	-46.0	-36.6	-34.4	-38.2
331	SLS	-46.3	-35.0	-35.0	-46.5
332	SLS	-55.6	-40.3	-39.6	-51.5
333	SLS	-43.3	-30.5	-30.5	-37.6
334	SLS	-52.6	-35.8	-35.1	-42.6
335	SLS	-37.4	-30.5	-30.5	-43.5
336	SLS	-46.7	-35.8	-35.1	-48.5
337	SLS	-25.8	-18.7	-18.8	-25.9
338	SLS	-25.8	-18.7	-18.8	-25.9
339	SLS	-25.8	-18.7	-18.8	-25.9
340	SLS	-32.0	-22.3	-21.8	-29.3
341	SLS	-32.0	-22.3	-21.8	-29.3
342	SLS	-32.0	-22.3	-21.8	-29.3
351	SLS	-22.6	-17.1	-17.1	-22.8
352	SLS	-31.9	-22.4	-21.7	-27.8
353	SLS	-22.6	-17.1	-17.1	-22.8
354	SLS	-31.9	-22.4	-21.7	-27.8
355	SLS	-22.6	-17.1	-17.1	-22.8
356	SLS	-31.9	-22.4	-21.7	-27.8

8.4. Shearing forces Ved [kN]

	L.C.	Left column 1		Left rafter 2			Right rafter 3			Right column 4	
		VedA,1	VedB,1	VedA,2	VedC,2	VedB,2	VedA,3	VedC,3	VedB,3	VedA,4	VedB,4
101	ULS-EQU	-51.7	-51.7	66.4	51.6	-8.5	8.5	-51.6	-66.4	51.7	51.7
102	ULS-EQU	-70.3	-70.3	90.2	70.1	-11.6	11.6	-70.1	-90.2	70.3	70.3
103	ULS-EQU	-58.2	-58.2	83.4	63.4	-18.3	0.8	-52.7	-65.9	58.2	58.2
104	ULS-EQU	-58.2	-58.2	65.9	52.7	-0.8	18.3	-63.4	-83.4	58.2	58.2
105	ULS-EQU	31.0	2.6	-16.4	-13.6	-2.1	-3.1	11.9	15.5	-15.5	-3.3
106	ULS-EQU	2.9	-25.4	26.2	18.2	-14.3	-7.0	-3.4	-6.0	6.4	18.6
111	ULS-EQU	35.0	6.6	-21.5	-17.5	-1.5	-3.8	15.8	20.6	-19.5	-7.3
121	ULS-EQU	-38.6	-55.6	63.5	48.9	-10.7	7.5	-49.9	-64.0	47.9	55.2
122	ULS-EQU	-55.4	-72.4	89.1	68.0	-18.0	5.2	-60.7	-76.9	61.0	68.3
123	ULS-EQU	-26.5	-43.5	56.8	42.1	-17.4	-3.2	-32.5	-39.7	35.8	43.1
124	ULS-EQU	-43.3	-60.3	82.3	61.2	-24.7	-5.6	-43.4	-52.6	48.9	56.2
125	ULS-EQU	-26.5	-43.5	39.2	31.5	0.1	14.3	-43.1	-57.3	35.8	43.1
126	ULS-EQU	-43.3	-60.3	64.8	50.6	-7.2	12.0	-54.0	-70.2	48.9	56.2
127	ULS-EQU	6.8	-21.6	14.7	10.6	-6.1	0.9	-12.3	-15.5	8.7	20.9
128	ULS-EQU	12.8	-15.5	11.3	7.2	-9.5	-4.5	-1.8	-3.4	2.7	14.8
129	ULS-EQU	12.8	-15.5	2.5	1.9	-0.7	4.3	-8.9	-12.1	2.7	14.8
130	ULS-EQU	-21.3	-49.6	57.3	42.4	-18.2	-3.0	-30.3	-37.0	30.6	42.8
131	ULS-EQU	-15.2	-43.6	53.9	39.0	-21.6	-8.4	-21.6	-24.9	24.6	36.7
132	ULS-EQU	-15.2	-43.6	45.1	33.7	-12.9	0.3	-26.9	-33.6	24.6	36.7
201	ULS-STR	-56.7	-56.7	72.8	56.6	-9.3	9.3	-56.6	-72.8	56.7	56.7
202	ULS-STR	-75.3	-75.3	96.6	75.1	-12.4	12.4	-75.1	-96.6	75.3	75.3
203	ULS-STR	-63.2	-63.2	89.8	68.3	-19.1	1.6	-57.7	-72.3	63.2	63.2
204	ULS-STR	-63.2	-63.2	72.3	57.7	-1.6	19.1	-68.3	-89.8	63.2	63.2
205	ULS-STR	26.0	-2.3	-10.0	-8.6	-2.9	-2.3	6.9	9.2	-10.5	1.6
206	ULS-STR	-2.0	-30.4	32.6	23.2	-15.1	-6.2	-11.1	-12.3	11.4	23.6
210	ULS-STR	33.0	4.6	-18.9	-15.5	-1.8	-3.4	13.8	18.1	-17.5	-5.3
211	ULS-STR	-43.6	-60.6	69.9	53.8	-11.5	8.4	-54.9	-70.4	52.8	60.1
212	ULS-STR	-60.4	-77.4	95.5	72.9	-18.8	6.0	-65.7	-83.3	66.0	73.3
213	ULS-STR	-31.5	-48.5	63.1	47.1	-18.3	-2.4	-37.5	-46.1	40.7	48.0
214	ULS-STR	-48.3	-65.3	88.7	66.2	-25.5	-4.7	-48.3	-59.0	53.9	61.2
215	ULS-STR	-31.5	-48.5	45.6	36.5	-0.7	15.1	-48.1	-63.6	40.7	48.0
216	ULS-STR	-48.3	-65.3	71.2	55.5	-8.0	12.8	-58.9	-76.5	53.9	61.2
217	ULS-STR	1.8	-26.6	21.1	15.5	-6.9	1.7	-17.2	-21.9	13.7	25.8
218	ULS-STR	7.8	-20.5	17.7	12.2	-10.3	-3.7	-8.6	-9.8	7.6	19.8
219	ULS-STR	7.8	-20.5	8.9	6.9	-1.5	5.1	-13.9	-18.5	7.6	19.8
220	ULS-STR	-26.2	-54.6	63.7	47.3	-19.1	-2.2	-35.3	-43.4	35.6	47.8
221	ULS-STR	-20.2	-48.5	60.3	43.9	-22.4	-7.6	-26.6	-31.3	29.6	41.7
222	ULS-STR	-20.2	-48.5	51.5	38.6	-13.7	1.2	-31.9	-40.0	29.6	41.7
231	ULS-STR	-19.4	-36.4	38.8	29.7	-7.5	4.4	-30.7	-39.3	28.6	35.9
232	ULS-STR	-36.2	-53.2	64.4	48.8	-14.8	2.0	-41.5	-52.3	41.8	49.1
233	ULS-STR	-13.3	-30.3	35.5	26.3	-10.9	-1.0	-22.0	-27.2	22.6	29.9
234	ULS-STR	-30.1	-47.1	61.0	45.4	-18.2	-3.4	-32.9	-40.1	35.7	43.0
235	ULS-STR	-13.3	-30.3	26.7	21.0	-2.1	7.8	-27.3	-36.0	22.6	29.9
236	ULS-STR	-30.1	-47.1	52.3	40.1	-9.4	5.4	-38.2	-48.9	35.7	43.0
251	ULS-STR	-39.6	-56.6	64.8	49.9	-10.8	7.7	-50.9	-65.3	48.9	56.2
252	ULS-STR	-56.4	-73.4	90.3	69.0	-18.1	5.3	-61.7	-78.2	62.0	69.3
253	ULS-STR	-27.5	-44.5	58.0	43.1	-17.6	-3.1	-33.5	-41.0	36.8	44.1
254	ULS-STR	-44.3	-61.3	83.6	62.2	-24.9	-5.4	-44.3	-53.9	49.9	57.2
255	ULS-STR	-27.5	-44.5	40.5	32.5	-0.1	14.5	-44.1	-58.5	36.8	44.1
256	ULS-STR	-44.3	-61.3	66.1	51.6	-7.4	12.1	-55.0	-71.4	49.9	57.2
257	ULS-STR	5.8	-22.6	16.0	11.6	-6.3	1.0	-13.3	-16.8	9.7	21.9
258	ULS-STR	11.8	-16.5	12.6	8.2	-9.7	-4.3	-2.9	-4.6	3.7	15.8
259	ULS-STR	11.8	-16.5	3.8	2.9	-0.9	4.4	-9.9	-13.4	3.7	15.8
260	ULS-STR	-22.3	-50.6	58.5	43.4	-18.4	-2.9	-31.3	-38.3	31.6	43.8
261	ULS-STR	-16.2	-44.6	55.2	40.0	-21.8	-8.3	-22.6	-26.2	25.6	37.7
262	ULS-STR	-16.2	-44.6	46.4	34.7	-13.0	0.5	-27.9	-34.9	25.6	37.7
301	SLS	-39.8	-39.8	51.1	39.7	-6.5	6.5	-39.7	-51.1	39.8	39.8
302	SLS	-52.2	-52.2	66.9	52.0	-8.6	8.6	-52.0	-66.9	52.2	52.2
303	SLS	-44.1	-44.1	62.4	47.5	-13.1	1.4	-40.5	-50.7	44.1	44.1
304	SLS	-44.1	-44.1	50.7	40.5	-1.4	13.1	-47.5	-62.4	44.1	44.1
305	SLS	15.3	-3.6	-4.1	-3.8	-2.3	-1.2	2.6	3.6	-5.0	3.1
306	SLS	-3.4	-22.3	24.3	17.4	-10.4	-3.8	-9.4	-10.8	9.6	17.7
311	SLS	-31.0	-42.4	49.2	37.9	-8.0	5.9	-38.6	-49.5	37.2	42.1
312	SLS	-42.2	-53.6	66.2	50.6	-12.8	4.3	-45.8	-58.1	46.0	50.9
313	SLS	-23.0	-34.3	44.6	33.4	-12.5	-1.3	-27.0	-33.3	29.2	34.0
314	SLS	-34.2	-45.5	61.7	46.1	-17.4	-2.8	-34.2	-41.9	37.9	42.8
315	SLS	-23.0	-34.3	33.0	26.3	-0.8	10.4	-34.0	-45.0	29.2	34.0
316	SLS	-34.2	-45.5	50.0	39.0	-5.7	8.8	-41.3	-53.6	37.9	42.8
317	SLS	-0.8	-19.7	16.6	12.3	-4.9	1.5	-13.5	-17.1	11.1	19.2
318	SLS	3.2	-15.7	14.3	10.1	-7.2	-2.1	-7.7	-9.1	7.1	15.2
319	SLS	3.2	-15.7	8.5	6.6	-1.4	3.7	-11.2	-14.9	7.1	15.2
320	SLS	-19.5	-38.4	45.0	33.5	-13.0	-1.2	-25.5	-31.5	25.7	33.8
321	SLS	-15.5	-34.4	42.7	31.3	-15.3	-4.7	-19.7	-23.4	21.7	29.8
322	SLS	-15.5	-34.4	36.9	27.7	-9.5	1.1	-23.3	-29.2	21.7	29.8
331	SLS	-25.5	-31.1	37.3	28.9	-5.6	4.6	-29.2	-37.5	28.6	31.0
332	SLS	-31.1	-36.7	45.9	35.2	-8.1	3.8	-32.8	-41.8	32.9	35.4
333	SLS	-21.4	-27.1	35.1	26.6	-7.9	1.0	-23.4	-29.4	24.5	27.0
334	SLS	-27.0	-32.7	43.6	33.0	-10.3	0.2	-27.0	-33.7	28.9	31.3
335	SLS	-21.4	-27.1	29.2	23.1	-2.0	6.8	-26.9	-35.3	24.5	27.0
336	SLS	-27.0	-32.7	37.8	29.4	-4.5	6.1	-30.6	-39.6	28.9	31.3
337	SLS	-12.9	-16.6	19.6	15.1	-3.1	2.4	-15.4	-19.7	14.9	16.5
338	SLS	-12.9	-16.6	19.6	15.1	-3.1	2.4	-15.4	-19.7	14.9	16.5
339	SLS	-12.9	-16.6	19.6	15.1	-3.1	2.4	-15.4	-19.7	14.9	16.5
340	SLS	-16.6	-20.4	25.3	19.4	-4.7	1.9	-17.8	-22.6	17.8	19.5
341	SLS	-16.6	-20.4	25.3	19.4	-4.7	1.9	-17.8	-22.6	17.8	19.5
342	SLS	-16.6	-20.4	25.3	19.4	-4.7	1.9	-17.8	-22.6	17.8	19.5
351	SLS	-9.3	-15.0	16.6	12.8	-3.0	1.9	-13.1	-16.8	12.4	14.9
352	SLS	-14.9	-20.6	25.2	19.1	-5.4	1.1	-16.7	-21.1	16.8	19.2
353	SLS	-9.3	-15.0	16.6	12.8	-3.0	1.9	-13.1	-16.8	12.4	14.9
354	SLS	-14.9	-20.6	25.2	19.1	-5.4	1.1	-16.7	-21.1	16.8	19.2
355	SLS	-9.3	-15.0	16.6	12.8	-3.0	1.9	-13.1	-16.8	12.4	14.9
356	SLS	-14.9	-20.6	25.2	19.1	-5.4	1.1	-16.7	-21.1	16.8	19.2

A:left end, C:haunch end, B: right end

8.5. Bending moments Med [kNm]

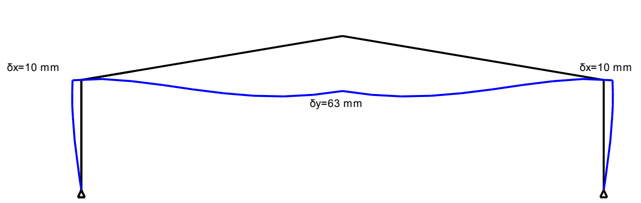
L.C.		Left column 1 MedA,1	MedM,1	MedB,1	Right column 4 MedA,4	MedM,4	MedB,4
101	ULS-EQU	0.0	-129.3	-258.7	-258.7	-129.3	0.0
102	ULS-EQU	0.0	-175.8	-351.5	-351.5	-175.8	0.0
103	ULS-EQU	0.0	-145.5	-291.0	-291.0	-145.5	0.0
104	ULS-EQU	0.0	-145.5	-291.0	-291.0	-145.5	0.0
105	ULS-EQU	0.0	42.0	84.0	47.1	23.6	0.0
106	ULS-EQU	0.0	0.8	-56.2	-62.6	-31.3	0.0
111	ULS-EQU	0.0	52.0	103.9	67.0	33.5	0.0
121	ULS-EQU	0.0	-117.7	-235.5	-257.6	-128.8	0.0
122	ULS-EQU	0.0	-159.8	-319.6	-323.4	-161.7	0.0
123	ULS-EQU	0.0	-87.5	-174.9	-197.1	-98.5	0.0
124	ULS-EQU	0.0	-129.5	-259.1	-262.9	-131.4	0.0
125	ULS-EQU	0.0	-87.5	-174.9	-197.1	-98.5	0.0
126	ULS-EQU	0.0	-129.5	-259.1	-262.9	-131.4	0.0
127	ULS-EQU	0.0	4.0	-37.0	-73.9	-37.0	0.0
128	ULS-EQU	0.0	14.5	-6.8	-43.7	-21.8	0.0
129	ULS-EQU	0.0	14.5	-6.8	-43.7	-21.8	0.0
130	ULS-EQU	0.0	-88.6	-177.2	-183.6	-91.8	0.0
131	ULS-EQU	0.0	-73.5	-147.0	-153.3	-76.7	0.0
132	ULS-EQU	0.0	-73.5	-147.0	-153.3	-76.7	0.0
201	ULS-STR	0.0	-141.8	-283.6	-283.6	-141.8	0.0
202	ULS-STR	0.0	-188.2	-376.4	-376.4	-188.2	0.0
203	ULS-STR	0.0	-157.9	-315.9	-315.9	-157.9	0.0
204	ULS-STR	0.0	-157.9	-315.9	-315.9	-157.9	0.0
205	ULS-STR	0.0	59.6	59.1	22.2	-0.5	0.0
206	ULS-STR	0.0	-40.5	-81.1	-87.4	-43.7	0.0
210	ULS-STR	0.0	47.0	94.0	57.1	28.5	0.0
211	ULS-STR	0.0	-130.2	-260.3	-282.5	-141.2	0.0
212	ULS-STR	0.0	-172.2	-344.5	-348.3	-174.1	0.0
213	ULS-STR	0.0	-99.9	-199.8	-222.0	-111.0	0.0
214	ULS-STR	0.0	-142.0	-283.9	-287.8	-143.9	0.0
215	ULS-STR	0.0	-99.9	-199.8	-222.0	-111.0	0.0
216	ULS-STR	0.0	-142.0	-283.9	-287.8	-143.9	0.0
217	ULS-STR	0.0	0.3	-61.9	-98.8	-49.4	0.0
218	ULS-STR	0.0	5.4	-31.7	-68.5	-34.3	0.0
219	ULS-STR	0.0	5.4	-31.7	-68.5	-34.3	0.0
220	ULS-STR	0.0	-101.1	-202.1	-208.5	-104.2	0.0
221	ULS-STR	0.0	-85.9	-171.9	-178.2	-89.1	0.0
222	ULS-STR	0.0	-85.9	-171.9	-178.2	-89.1	0.0
231	ULS-STR	0.0	-69.6	-139.3	-161.4	-80.7	0.0
232	ULS-STR	0.0	-111.7	-223.4	-227.2	-113.6	0.0
233	ULS-STR	0.0	-54.5	-109.0	-131.2	-65.6	0.0
234	ULS-STR	0.0	-96.6	-193.2	-197.0	-98.5	0.0
235	ULS-STR	0.0	-54.5	-109.0	-131.2	-65.6	0.0
236	ULS-STR	0.0	-96.6	-193.2	-197.0	-98.5	0.0
251	ULS-STR	0.0	-120.2	-240.4	-262.6	-131.3	0.0
252	ULS-STR	0.0	-162.3	-324.6	-328.4	-164.2	0.0
253	ULS-STR	0.0	-90.0	-179.9	-202.1	-101.0	0.0
254	ULS-STR	0.0	-132.0	-264.0	-267.9	-133.9	0.0
255	ULS-STR	0.0	-90.0	-179.9	-202.1	-101.0	0.0
256	ULS-STR	0.0	-132.0	-264.0	-267.9	-133.9	0.0
257	ULS-STR	0.0	2.9	-42.0	-78.9	-39.5	0.0
258	ULS-STR	0.0	12.3	-11.8	-48.6	-24.3	0.0
259	ULS-STR	0.0	12.3	-11.8	-48.6	-24.3	0.0
260	ULS-STR	0.0	-91.1	-182.2	-188.6	-94.3	0.0
261	ULS-STR	0.0	-76.0	-152.0	-158.3	-79.2	0.0
262	ULS-STR	0.0	-76.0	-152.0	-158.3	-79.2	0.0
301	SLS	0.0	-99.5	-199.0	-199.0	-99.5	0.0
302	SLS	0.0	-130.4	-260.9	-260.9	-130.4	0.0
303	SLS	0.0	-110.3	-220.5	-220.5	-110.3	0.0
304	SLS	0.0	-110.3	-220.5	-220.5	-110.3	0.0
305	SLS	0.0	31.1	29.5	4.9	-2.9	0.0
306	SLS	0.0	-32.0	-64.0	-68.2	-34.1	0.0
311	SLS	0.0	-91.8	-183.5	-198.3	-99.1	0.0
312	SLS	0.0	-119.8	-239.6	-242.1	-121.1	0.0
313	SLS	0.0	-71.6	-143.2	-157.9	-79.0	0.0
314	SLS	0.0	-99.6	-199.2	-201.8	-100.9	0.0
315	SLS	0.0	-71.6	-143.2	-157.9	-79.0	0.0
316	SLS	0.0	-99.6	-199.2	-201.8	-100.9	0.0
317	SLS	0.0	-25.6	-51.2	-75.8	-37.9	0.0
318	SLS	0.0	1.4	-31.1	-55.6	-27.8	0.0
319	SLS	0.0	1.4	-31.1	-55.6	-27.8	0.0
320	SLS	0.0	-72.3	-144.7	-148.9	-74.5	0.0
321	SLS	0.0	-62.3	-124.5	-128.8	-64.4	0.0
322	SLS	0.0	-62.3	-124.5	-128.8	-64.4	0.0
331	SLS	0.0	-70.8	-141.5	-148.9	-74.4	0.0
332	SLS	0.0	-84.8	-169.5	-170.8	-85.4	0.0
333	SLS	0.0	-60.7	-121.3	-128.7	-64.4	0.0
334	SLS	0.0	-74.7	-149.4	-150.6	-75.3	0.0
335	SLS	0.0	-60.7	-121.3	-128.7	-64.4	0.0
336	SLS	0.0	-74.7	-149.4	-150.6	-75.3	0.0
337	SLS	0.0	-36.9	-73.7	-78.6	-39.3	0.0
338	SLS	0.0	-36.9	-73.7	-78.6	-39.3	0.0
339	SLS	0.0	-36.9	-73.7	-78.6	-39.3	0.0
340	SLS	0.0	-46.2	-92.4	-93.2	-46.6	0.0
341	SLS	0.0	-46.2	-92.4	-93.2	-46.6	0.0
342	SLS	0.0	-46.2	-92.4	-93.2	-46.6	0.0
351	SLS	0.0	-30.4	-60.8	-68.2	-34.1	0.0
352	SLS	0.0	-44.4	-88.8	-90.1	-45.1	0.0
353	SLS	0.0	-30.4	-60.8	-68.2	-34.1	0.0
354	SLS	0.0	-44.4	-88.8	-90.1	-45.1	0.0
355	SLS	0.0	-30.4	-60.8	-68.2	-34.1	0.0
356	SLS	0.0	-44.4	-88.8	-90.1	-45.1	0.0

A:left end, B: right end

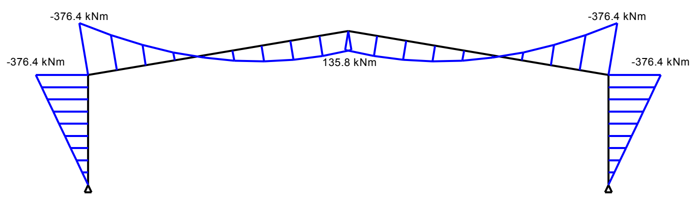
L.C.		Left rafter 2 MedA,2	MedC2	MedM,2	MedB,2	Right rafter 3 MedA,3	MedM,3	MedC3	MedB,3
101	ULS-EQU	-258.7	-117.1	99.2	93.3	93.3	99.2	-117.1	-258.7
102	ULS-EQU	-351.5	-159.2	134.8	126.8	126.8	134.8	-159.2	-351.5
103	ULS-EQU	-291.0	-114.9	125.0	105.0	105.0	105.0	-148.6	-291.0
104	ULS-EQU	-291.0	-148.6	105.0	105.0	105.0	125.0	-114.9	-291.0
105	ULS-EQU	84.0	48.1	27.7	-28.5	-28.5	-31.7	14.2	47.1
106	ULS-EQU	-56.2	-2.9	47.1	16.5	16.5	-23.0	-47.0	-62.6
111	ULS-EQU	103.9	57.1	34.1	-35.7	-35.7	-39.2	23.2	67.0
121	ULS-EQU	-235.5	-100.6	95.3	86.0	86.0	90.8	-120.9	-257.6
122	ULS-EQU	-319.6	-131.2	131.3	113.0	113.0	115.0	-158.2	-323.4
123	ULS-EQU	-174.9	-56.3	89.1	64.2	64.2	-66.5	-110.4	-197.1
124	ULS-EQU	-259.1	-86.9	125.9	91.2	91.2	-85.9	-147.7	-262.9
125	ULS-EQU	-174.9	-90.1	-55.4	64.2	64.2	81.5	-76.6	-197.1
126	ULS-EQU	-259.1	-120.6	95.6	91.2	91.2	101.8	-113.9	-262.9
127	ULS-EQU	-37.0	-6.7	26.0	15.1	15.1	15.4	-40.6	-73.9
128	ULS-EQU	-6.8	15.4	30.6	4.2	4.2	-19.7	-34.2	-43.7
129	ULS-EQU	-6.8	-1.5	5.2	4.2	4.2	10.9	-18.4	-43.7
130	ULS-EQU	-177.2	-57.7	87.0	60.1	60.1	-61.7	-102.8	-183.6
131	ULS-EQU	-147.0	-35.5	86.9	49.2	49.2	-52.1	-97.5	-153.3
132	ULS-EQU	-147.0	-52.4	66.6	49.2	49.2	49.3	-80.7	-153.3
201	ULS-STR	-283.6	-128.4	108.7	102.3	102.3	108.7	-128.4	-283.6
202	ULS-STR	-376.4	-170.4	144.3	135.8	135.8	144.3	-170.4	-376.4
203	ULS-STR	-315.9	-126.1	134.4	113.9	113.9	114.1	-159.9	-315.9
204	ULS-STR	-315.9	-159.9	114.1	113.9	113.9	134.4	-126.1	-315.9
205	ULS-STR	59.1	36.8	19.8	-19.6	-19.6	-22.3	3.0	22.2
206	ULS-STR	-81.1	-14.1	54.5	25.5	25.5	-31.0	-59.2	-87.4
210	ULS-STR	94.0	52.6	30.9	-32.1	-32.1	-35.5	18.7	57.1
211	ULS-STR	-260.3	-111.8	104.8	95.0	95.0	100.3	-132.2	-282.5
212	ULS-STR	-344.5	-142.4	140.7	122.0	122.0	124.4	-169.5	-348.3
213	ULS-STR	-199.8	-67.6	98.0	73.1	73.1	-74.4	-121.6	-222.0
214	ULS-STR	-283.9	-98.1	134.9	100.1	100.1	-93.8	-159.0	-287.8
215	ULS-STR	-199.8	-101.3	73.2	73.1	73.1	90.8	-87.9	-222.0
216	ULS-STR	-283.9	-131.9	105.1	100.1	100.1	111.2	-125.2	-287.8
217	ULS-STR	-61.9	-18.0	34.5	24.1	24.1	24.8	-51.8	-98.8
218	ULS-STR	-31.7	4.1	36.3	13.2	13.2	-27.7	-46.6	-68.5
219	ULS-STR	-31.7	-12.7	14.6	13.2	13.2	19.8	-29.7	-68.5
220	ULS-STR	-202.1	-68.9	95.8	69.1	69.1	-69.7	-114.1	-208.5
221	ULS-STR	-171.9	-46.8	95.3	58.2	58.2	-60.0	-108.8	-178.2
222	ULS-STR	-171.9	-63.7	75.7	58.2	58.2	58.4	-91.9	-178.2
231	ULS-STR	-139.3	-57.0	58.7	51.3	51.3	54.0	-77.3	-161.4
232	ULS-STR	-223.4	-87.6	95.1	78.3	78.3	78.8	-114.7	-227.2
233	ULS-STR	-109.0	-34.9	56.0	40.4	40.4	-45.4	-72.1	-131.2
234	ULS-STR	-193.2	-65.5	92.8	67.4	67.4	-64.8	-109.4	-197.0
235	ULS-STR	-109.0	-51.8	41.3	40.4	40.4	48.8	-55.2	-131.2
236	ULS-STR	-193.2	-82.3	76.1	67.4	67.4	70.7	-92.5	-197.0
251	ULS-STR	-240.4	-102.8	97.2	87.8	87.8	92.7	-123.1	-262.6
252	ULS-STR	-324.6	-133.4	133.2	114.8	114.8	116.9	-160.5	-328.4
253	ULS-STR	-179.9	-58.6	90.9	65.9	65.9	-68.1	-112.6	-202.1
254	ULS-STR	-264.0	-89.1	127.7	93.0	93.0	-87.4	-149.9	-267.9
255	ULS-STR	-179.9	-92.3	65.9	65.9	65.9	83.4	-78.9	-202.1
256	ULS-STR	-264.0	-122.9	97.5	93.0	93.0	103.6	-116.2	-267.9
257	ULS-STR	-42.0	-9.0	27.7	16.9	16.9	17.3	-42.8	-78.9
258	ULS-STR	-11.8	13.2	31.5	6.0	6.0	-21.3	-37.9	-48.6
259	ULS-STR	-11.8	-3.7	7.0	6.0	6.0	12.7	-20.7	-48.6
260	ULS-STR	-182.2	-59.9	88.7	61.9	61.9	-63.3	-105.0	-188.6
261	ULS-STR	-152.0	-37.8	88.6	51.0	51.0	-53.6	-99.8	-158.3
262	ULS-STR	-152.0	-54.7	68.4	51.0	51.0	51.1	-82.9	-158.3
301	SLS	-199.0	-90.1	76.3	71.8	71.8	76.3	-90.1	-199.0
302	SLS	-260.9	-118.1	100.0	94.1	94.1	100.0	-118.1	-260.9
303	SLS	-220.5	-88.6	93.3	79.5	79.5	79.8	-111.1	-220.5
304	SLS	-220.5	-111.1	79.8	79.5	79.5	93.3	-88.6	-220.5
305	SLS	29.5	20.0	10.0	-9.5	-9.5	-11.3	-2.5	4.9
306	SLS	-64.0	-13.9	39.5	20.6	20.6	-23.8	-44.0	-68.2
311	SLS	-183.5	-79.1	73.7	66.9	66.9	70.7	-92.6	-198.3
312	SLS	-239.6	-99.4	97.6	84.9	84.9	86.7	-117.5	-242.1
313	SLS	-143.2	-49.6	69.0	52.3	52.3	-52.8	-85.6	-157.9
314	SLS	-199.2	-69.9	93.5	70.3	70.3	-65.7	-110.5	-201.8
315	SLS	-143.2	-72.1	52.5	52.3	52.3	64.2	-63.1	-157.9
316	SLS	-199.2	-92.4	73.9	70.3	70.3	78.0	-88.0	-201.8
317	SLS	-51.2	-16.5	26.5	19.7	19.7	20.3	-39.1	-75.8
318	SLS	-31.1	-1.7	27.0	12.4	12.4	-21.6	-35.6	-55.6
319	SLS	-31.1	-13.0	13.5	12.4	12.4	16.9	-24.3	-55.6
320	SLS	-144.7	-50.5	67.5	49.7	49.7	-49.6	-80.5	-148.9
321	SLS	-124.5	-35.7	66.9	42.4	42.4	-43.2	-77.0	-128.8
322	SLS	-124.5	-47.0	54.1	42.4	42.4	42.6	-65.8	-128.8
331	SLS	-141.5	-62.1	55.9	51.4	51.4	54.4	-68.8	-148.9
332	SLS	-169.5	-72.2	67.7	60.4	60.4	62.3	-81.3	-170.8
333	SLS	-121.3	-47.3	52.9	44.1	44.1	44.3	-65.3	-128.7
334	SLS	-149.4	-57.5	65.1	53.1	53.1	53.1	-77.8	-150.6
335	SLS	-121.3	-58.5	44.9	44.1	44.1	50.9	-54.1	-128.7
336	SLS	-149.4	-68.7	56.0	53.1	53.1	58.0	-66.5	-150.6
337	SLS	-73.7	-32.0	29.4	26.8	26.8	28.4	-36.5	-78.6
338	SLS	-73.7	-32.0	29.4	26.8	26.8	28.4	-36.5	-78.6
339	SLS	-73.7	-32.0	29.4	26.8	26.8	28.4	-36.5	-78.6
340	SLS	-92.4	-38.8	37.3	32.8	32.8	33.7	-44.8	-93.2
341	SLS	-92.4	-38.8	37.3	32.8	32.8	33.7	-44.8	-93.2
342	SLS	-92.4	-38.8	37.3	32.8	32.8	33.7	-44.8	-93.2
351	SLS	-60.8	-25.5	25.0	22.3	22.3	23.5	-32.3	-68.2
352	SLS	-88.8	-35.7	37.1	31.3	31.3	31.6	-44.7	-90.1
353	SLS	-60.8	-25.5	25.0	22.3	22.3	23.5	-32.3	-68.2
354	SLS	-88.8	-35.7	37.1	31.3	31.3	31.6	-44.7	-90.1
355	SLS	-60.8	-25.5	25.0	22.3	22.3	23.5	-32.3	-68.2
356	SLS	-88.8	-35.7	37.1	31.3	31.3	31.6	-44.7	-90.1

-A:left end, C:haunch end, M: span, B: right end

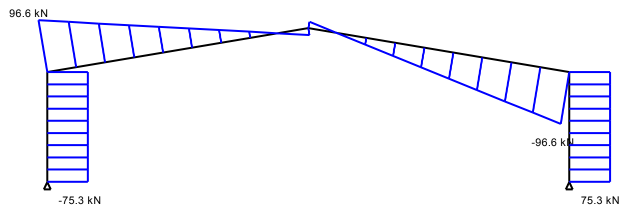
L.C. 202 Displacements mm



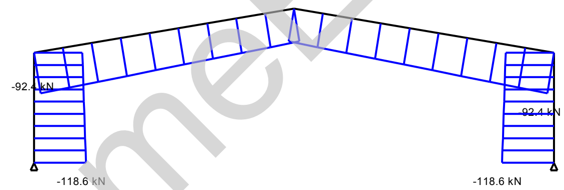
L.C. 202 Bend. moments kNm



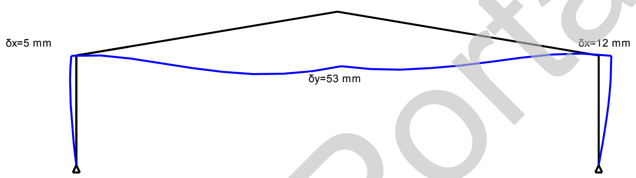
L.C. 202 Shear forces kN



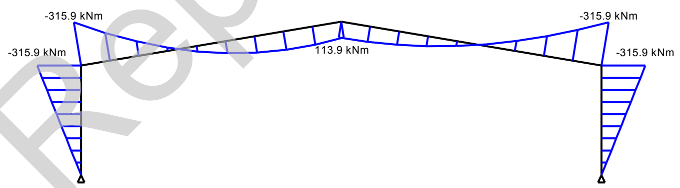
L.C. 202 Axial forces kN



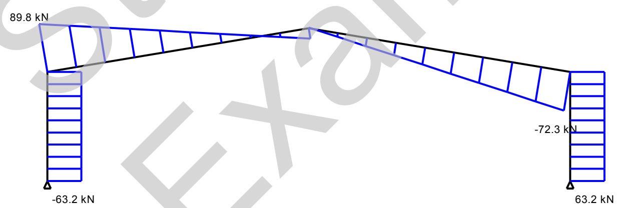
L.C. 203 Displacements mm



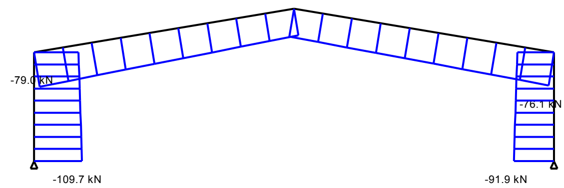
L.C. 203 Bend. moments kNm



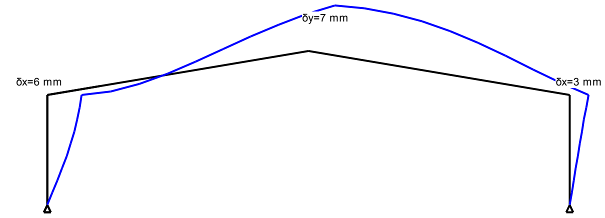
L.C. 203 Shear forces kN



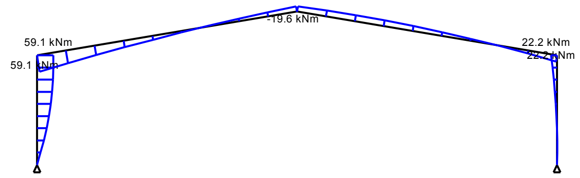
L.C. 203 Axial forces kN



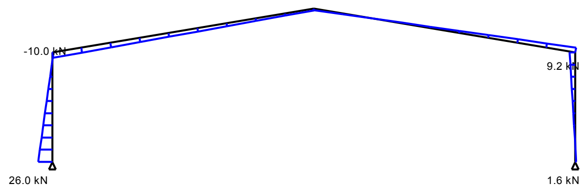
L.C. 205 Displacements mm



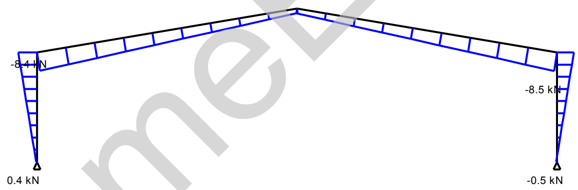
L.C. 205 Bend. moments kNm



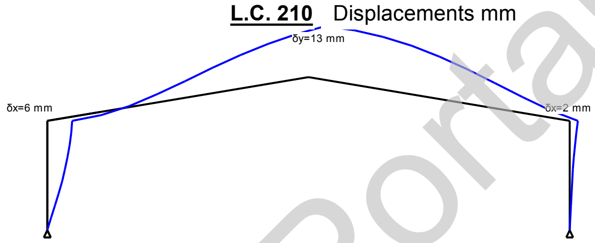
L.C. 205 Shear forces kN



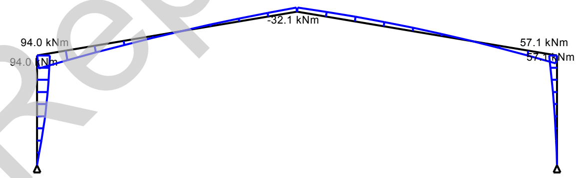
L.C. 205 Axial forces kN



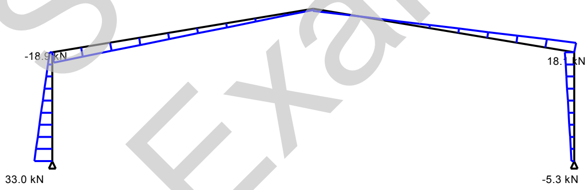
L.C. 210 Displacements mm



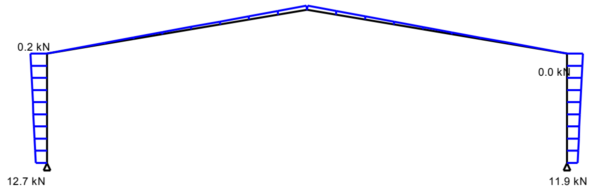
L.C. 210 Bend. moments kNm



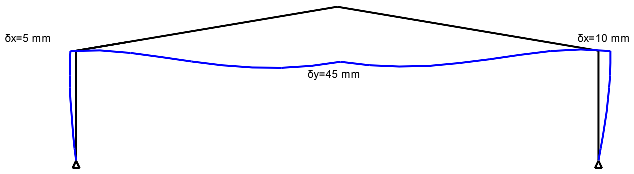
L.C. 210 Shear forces kN



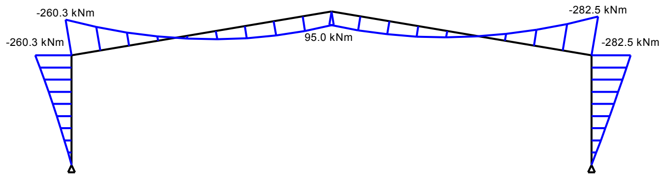
L.C. 210 Axial forces kN



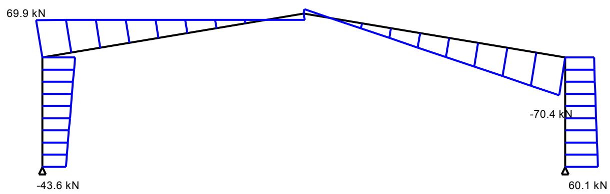
L.C. 211 Displacements mm



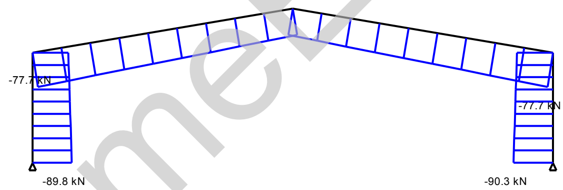
L.C. 211 Bend. moments kNm



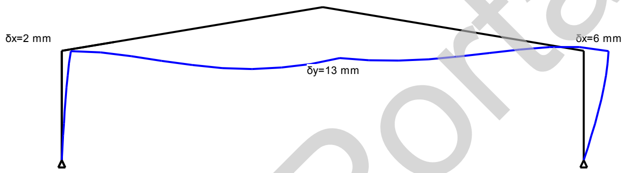
L.C. 211 Shear forces kN



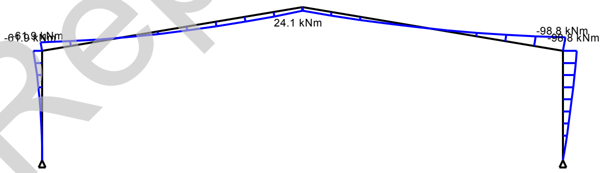
L.C. 211 Axial forces kN



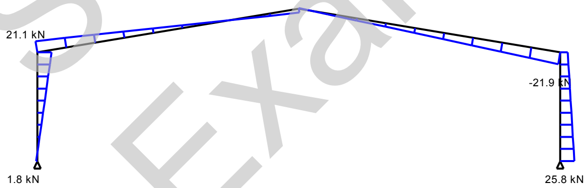
L.C. 217 Displacements mm



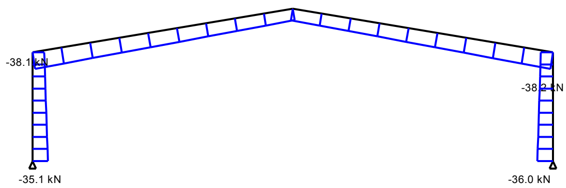
L.C. 217 Bend. moments kNm



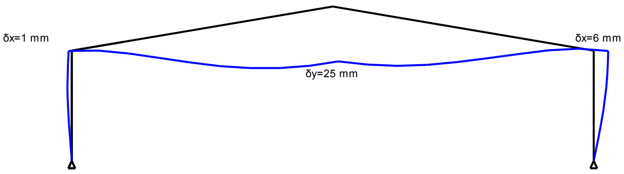
L.C. 217 Shear forces kN



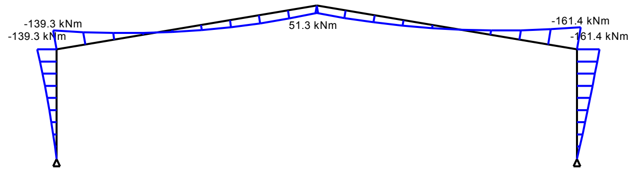
L.C. 217 Axial forces kN



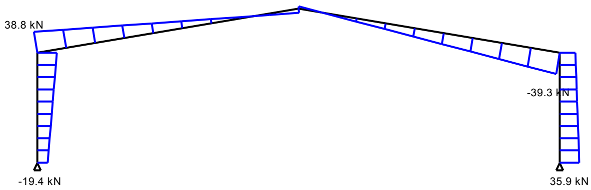
L.C. 231 Displacements mm



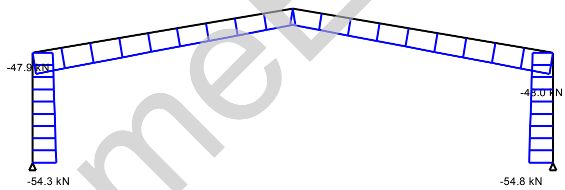
L.C. 231 Bend. moments kNm



L.C. 231 Shear forces kN

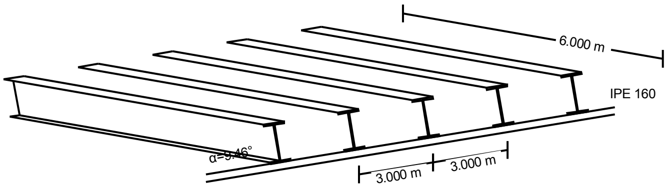


L.C. 231 Axial forces kN



9. Design of Purlins

Purlin laterally restrained, IPE 160 S 355
Continuous purlin, L= 6.000 m, s= 3.000 m



9.1. Materials

Steel: S 355 (EN1993-1-1, §3.2)
t<= 40 mm, Yield strength fy= 355 N/mm², Ultimate strength fu= 510 N/mm²
40mm<t<= 80 mm, Yield strength fy= 335 N/mm², Ultimate strength fu= 470 N/mm²
Modulus of elasticity E=210000 N/mm², Poisson ratio ν=0.30, Unit mass ρ= 7850 Kg/m³

Partial safety factors for actions (EN1990, Annex A1)
$\gamma G,sup= 1.35$, $\gamma Q= 1.50$, $\gamma G,inf= 1.00$, $\psi 0= 0.70$

Partial factors for materials (EN1993-1-1, §6.1)
$\gamma M0= 1.00$, $\gamma M1= 1.00$, $\gamma M2= 1.25$

9.2. Loading (EN1991-1-1)

Roof loads

Roof slope	α = 9.46°	
Load of roof covering	gk1 = 0.200 kN/m²	(EN1991-1-1 §5)
Imposed load (category H)	qk = 0.400 kN/m²	(EN1991-1-1 §6.3.4.2)
Snow load	sk = 0.640 kN/m²	(EN1991-1-3 §5.3)
Wind pressure	wk = 0.000 kN/m²	(EN1991-1-4 §7.2)
Wind uplift	wk =-0.460 kN/m²	

Load on purlin

Purlin spacing	s= 3.000 m
Load of roof covering	Gk1= 3.000x0.200= 0.60kN/m
Purlin weight	Gk2= 0.15 kN/m
Permanent load	Gk =Gk1+Gk2=0.60+0.15=0.75 kN/m
Imposed load (category H)	Qkk= 3.000x0.400= 1.20kN/m
Snow load	Qsk= 3.000x0.640= 1.92kN/m
Wind uplift	Qwk=-3.000x0.460=-1.38kN/m

Load on purlin main axis(z) and transverse direction(y)

Permanent load	Gk,z = 0.75xcos(9.46)= 0.74kN/m,	Gk,y = 0.75xsin(9.46)= 0.12kN/m
Imposed load (category H)	Qkk,z= 1.20xcos(9.46)= 1.18kN/m,	Qkk,y= 1.20xsin(9.46)= 0.20kN/m
Snow load	Qsk,z= 1.92xcos(9.46)= 1.89kN/m,	Qsk,y= 1.92xsin(9.46)= 0.32kN/m
Wind pressure	Qwk,z= 0.00kN/m,	Qwk,y= 0.00kN/m
Wind uplift	Qwk,z= - 1.38kN/m,	Qwk,y= 0.00kN/m

9.3. Design values of Actions, Load combinations

Ultimate Limit State, Load combinations (EN1990 §6.4.3.2, T.A1.2A, T.A1.2B)

Sagging	$\gamma G,sup \cdot Gk,z+\gamma Q \cdot Qk,z+\gamma Q \cdot \psi o \cdot Qwk,z$ = 1.35x0.74 +1.50x1.89 +1.50x0.60x0.00	= 3.84kN/m
Hogging	$\gamma G,inf \cdot Gk,z-\gamma Q \cdot Qwk,z$ = 1.00x0.74 -1.50x1.38	= -1.33kN/m
Transverse	$\gamma G,sup \cdot Gk,y+\gamma Q \cdot Qk,y$ = 1.35x0.12 +1.50x0.32	= 0.64kN/m

Serviceability Limit State (SLS), Load combinations (EN1990 §6.5.3, T.A1.4)

Sagging	$Gk,z+Qk,z+\psi o \cdot Qwk,z$ = 0.74+1.89+0.60x0.00	= 2.63kN/m
Hogging	$Gk,z+Qwk,z$ = 0.74-1.38	= -0.64kN/m

9.4. Design actions

Design actions, Ultimate Limit State

Sagging Myed,o= 0.078x 3.84x6.000²= 10.78kNm, Myed,s=-0.105x 3.84x6.000²= -14.51kNm
Vzed=0.605x3.84x6.000= 13.94kN

Hogging Myed,o=-0.078x 1.33x6.000²= -3.74kNm, Myed,s= 0.105x 1.33x6.000²= 5.03kNm
Vzed=0.605x1.33x6.000= 4.83kN

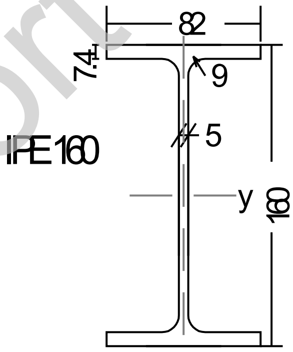
Transverse Laterally restrained

Design actions, Serviceability Limit State (SLS)

Sagging Myed,o= 0.078x 2.63x6.000²= 7.40kNm, Myed,s=-0.105x 2.63x6.000²= -9.96kNm
Hogging Myed,o=-0.078x 0.64x6.000²= -1.80kNm, Myed,s= 0.105x 0.64x6.000²= 2.42kNm

9.5. Cross-section properties, Purlins

Cross-section IPE 160-S 355

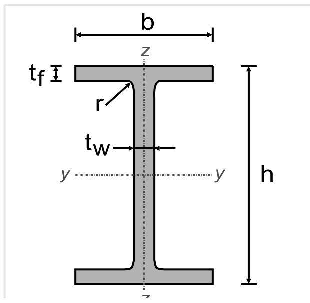


Dimensions of cross section

Depth of cross section	h=	160.00	mm
Width of cross section	b=	82.00	mm
Web depth	hw=	152.60	mm
Depth of straight portion of web	dw=	127.20	mm
Web thickness	tw=	5.00	mm
Flange thickness	tf=	7.40	mm
Radius of root fillet	r=	9.00	mm
Mass	=	15.80	Kg/m

Properties of cross section

Area	A=	2009	mm^2			
Second moment of area	Iy=	$8.693x10^6$	mm^4	Iz=	$0.683x10^6$	mm^4
Section modulus	Wy=	$108.70x10^3$	mm^3	Wz=	$16.660x10^3$	mm^3
Plastic section modulus	Wpy=	$123.90x10^3$	mm^3	Wpz=	$26.100x10^3$	mm^3
Radius of gyration	iy=	65.8	mm	iz=	18.4	mm
Shear area	Avz=	966	mm^2	Avy=	1214	mm^2
Torsional constant	It=	$0.036x10^6$	mm^4	ip=	68	mm
Warping constant	Iw=	$3.959x10^9$	mm^6			



9.6. Serviceability Limit State (SLS), Purlins (EN1993-1-1, §7)

Purlin deflections, Sagging

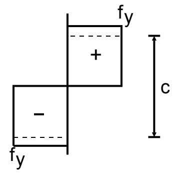
Loading G+Q: w= $2.63x6000^4/(153.6x2.1x10^5x8.693x10^6)$= 12.17mm = L/493<L/200
Loading Q: w= $1.89x6000^4/(153.6x2.1x10^5x8.693x10^6)$= 8.75mm = L/686<L/250

Purlin deflections, Hogging

Loading G+Q: w=$-0.64x6000^4/(153.6x2.1x10^5x8.693x10^6)$= -2.96mm = L/2026<L/200
Loading Q: w=$-1.38x6000^4/(153.6x2.1x10^5x8.693x10^6)$= -6.38mm = L/939<L/250

Purlin deflections, Serviceability Limit State (SLS), Is verified

9.7. Classification of cross-sections, Bending My (Purlin section) (EN1993-1-1, §5.5)



Web

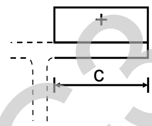
c=160.0-2x7.4-2x9.0=127.2 mm, t=5.0 mm, c/t=127.2/5.0=25.44

S 355 , t= 5.0<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81

c/t=25.44<=72ε=72x0.81=58.32

The web is class 1 (EN1993-1-1, Tab.5.2)

Flange



c=82.0/2-5.0/2-9.0=29.5 mm, t=7.4 mm, c/t=29.5/7.4=3.99

S 355 , t= 7.4<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81

c/t=3.99<=9ε=9x0.81=7.29

The flange is class 1 (EN1993-1-1, Tab.5.2)

Overall classification of cross-section is Class 1, Bending My,ed

9.8. Resistance of cross-section, Purlin section (EN1993-1-1, §6.2)

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)

My.ed= 14.51 kNm

Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x123.90x10^{3}x355/1.00= 43.98kNm

My,ed= 14.51 kNm < 43.98 kNm =My,rd=Mply,rd, Is verified

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)

Vz.ed= 13.94 kNm

Av=A-2b·tf+(tw+2r)tf=2009-2x82.0x7.4+(5.0+2x9.0)x7.4=966mm² (EC3 §6.2.6.3)

Av= 966mm² > η·hw·tw= 1.00x(160.0-2x7.4)x5.0=1.00x152.6x5.0= 763mm²

Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x966x(355/1.73)/1.00= 197.91kN

Vz,ed= 13.94 kN < 197.91 kN =Vz,rd=Vpl,z,rd, Is verified

hw/tw=(160.0-2x7.4)/5.0=152.6/5.0=30.52<=72x0.81/1.00=72ε/η=58.32 (η=1.00)

S 355 , t= 5.0<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81

Shear buckling resistance is not necessary to be verified (EC3 §6.2.6.6)

Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)

N.ed= 0.00kN, Vz.ed= 13.94kNm, My.ed= 14.51kN

Mpl,y,rd=43.98kNm, Vpl,z,rd=197.91kN

Ned=0 kN, Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

Ved=13.94kN <= 0.50x197.91=0.50xVpl,rd=98.96kN

Effect of shear force is neglected (EC3 §6.2.8.2)

9.9. Lateral restraining of sheeting (EC3 EN1993-1-3:2005, §10.1)

Sheeting thickness tw=0.750 mm, Profile depth hw=40.0 mm

Shear stiffness of sheeting (EN1993-1-3, §10.1.1Eq.10.1b)

S=t$^{1.5}$(50+10b$^{0.33}$)s/hw=0.750$^{1.5}$x(50+10x6000$^{0.33}$x3000/40.0=11288 kNm/m

Minimum required shear stiffness, for lateraly restrained purlin (§10.1.1Eq.10.1a)

Smin=[π²E·Iw/L²+G·It+π²E·Iz(h/2)²/L²]·70/h= [π²x2.1x10^{5}x3.959x10^{9}/6000²
+8.1x10^{4}x0.036x10^{6}+π²x2.1x10^{5}x0.683x10^{6}x80²/6000²]x70/160²]x[10^{-3}]=9272kNm/m

s=11288 kNm/m > 9272 kNm/m

The sheeting can be considered as sufficientely stiff to restrain the purlins

Rotational restraint given by the sheeting Cd=1/(1/Cd,a+1/Cd,c) (EN1993-1-3, §10.1.5.2)

Cd,c=k·E·Ieff/s, k=2, Ieff=0.3x0.75x39.25²=347mm^{4}/m, s=3000mm (Eq.10.16)

Cd,c=[10^{-3}]2x2.1x10^{5}x346.6/3000=48.5 kNm/m

Cd,a=C100·kba·kt·kbr·ka·kbt (EN1993-1-3, Eq.10.17)

C100=2.0, kba=1.25x160/100=2.00, kt=(0.75/0.75)$^{1.5}$=1.00, kbr=1.0, ka=1.0, kbt=1.0

Cd,a=2.0x2.00x1.00x1.0x1.0x1.0=4.0kNm/m

Cd=Cd,a= 3.7kNm/m

9.10. Lateral torsional buckling(Purlin laterally restrained) (EN1993-1-1, §6.3.2)

Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko,S.P, Gere,J.M, Theory of elastic stability, McGraw-Hill, 1961
$Mcr=C1\cdot[\pi^2EIz/(kL)^2]\{\sqrt{[(kz/kw)^2(Iw/Iz)+(kL)^2GIt,eq/(\pi^2EIz)+(C2\cdot zg-C3\cdot zj)^2]}-(C2\cdot zg-C3\cdot zj)\}$
$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4$ N/mm², $It,eq=It+Cd\cdot(kL)^2/(\pi^2G)$

Hogging
k·L=6000mm, zg=-80mm, zj=0mm (EN1993:2002 T.C.1)
kz=1.0, kw=1.0, C1=1.680, C2=0.809, C3=0.000 (EN1993:2002 T.C.1)
$Mcr=[10^{-6}]1.680x[\pi^2x2.1x10^5x0.683x10^6/6000^2]$
$x\{[1.0x(3.959x10^9/0.683x10^6)$
$+6000^2x8.1x10^4x0.203x10^6/(\pi^2x2.1x10^5x0.683x10^6)$
$+(-0.809x80)^2]^{0.5}-(-0.809x80)\}=47.4$ kNm
$It,eq=(0.036x10^6+10^3x3.7x6000^2/(\pi^2 8.1x10^4))=0.203x10^6$ mm^4

$\bar{\lambda},lt=\sqrt{(Wpl,y\cdot fy/Mcr)}=\sqrt{\{[10^{-6}]x123.90x10^3x355/47.4\}}=0.963$ (EC3 Eq.6.56)
h/b=160/82=1.95<=2.00 buckling curve:b
imperfection factor:α,lt=0.34, β=0.75, χ,lt=0.722 (T.6.3, T.6.5, Fig.6.4)
$\Phi,lt=0.5[1+\alpha,lt(\bar{\lambda},lt-\bar{\lambda},lto)+\beta\bar{\lambda},lt^2]=0.5x[1+0.34x(0.963-0.40)+0.75x0.963^2]=0.944$
$\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar{\lambda},lt^2)}]=1/[0.944+\sqrt{(0.944^2-0.75x0.944^2)}]=0.722$
Reduction factor $\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar{\lambda},lt^2)}]$, $\chi,lt<=1.0$, $1/\bar{\lambda},lt^2$, χ,lt=0.722 (Eq.6.57)

$Mb,rd=\chi,lt\cdot Wpl,y\cdot fy/\gamma M1=0.722x[10^{-6}]x123.90x10^3x355/1.00=31.76$kNm (EC3 Eq.6.55)
My,ed= 5.03 kNm < 31.76 kNm =Mb,rd, Is verified

10. Global analysis (EN1993-1-1, §5.2)

10.1. Effects of deformed geometry of the structure (EN1993-1-1, §5.2.1)

$\alpha_{cr}=(H_{nhf}/V_{ed})(h/\delta_{h,ed})$ (Eq.5.2)

From elastic analysis we obtain, L.C. 202: 1.35Gk+1.50Qs1

Vertical reaction at the base of column	Ved=	118.6 kN
Horizontal reaction at the base of column	Hed=	75.3 kN
Axial force at rafters	Nred=	92.4 kN
Notional horizontal force applied at the top of the columns	Hnhf=	1.0 kN
Horizontal deflection at column top for notional force	δh,ed=	0.93 mm

$\alpha_{cr}=(1.0/118.6)(5000/0.93)=45.47$ (Eq.5.2)

Check axial compression of rafters. Axial compression is significant if (§5.2.1,(4)B)

$\lambda = \sqrt{(A\cdot fy/Ncr)} > 0.3\sqrt{(A\cdot fy/Ned)}$, $Ned>0.09Ncr$ (§5.2.1 Eq.5.3)

Development length of the rafter pair from column to column L=24000/cos9.46° =24331mm

$Ncr=\pi^2EI/L^2=\pi^2x210x482.00x10^6/(24331)^2=1687.5kN$

Maximum axial force in the rafters Ned=92.4 kN, L.C. 202: 1.35Gk+1.50Qs1

$\lambda= \sqrt{(11550x355/1687502)} =1.56 <= 0.3 \sqrt{(11550x355/92417)}=2.00$

Axial compression of rafters is not significant, we can use Eq.5.2

α_{cr}=45.47 > 10 (Eq.5.1)

First-order elastic analysis may be used (§5.2.2.1)

Amplification factor for design moments $\delta=1/(1-1/\alpha_{cr})=1/(1-1/45.47)=1.02$ (Eq.5.4)

10.2. Imperfections for global analysis (EN1993-1-1, §5.3.2)

$\varphi=\varphi_o\cdot\alpha_h\cdot\alpha_m\cdot\delta=(1/200)x0.894x0.866x1.022=3.960x10^{-3}=1/253$ (Eq.5.5)

$\varphi_o=1/200$, $\alpha_h=2/\sqrt{h}=2/\sqrt{5.000}=0.894$ $2/3<=\alpha_h<=1.0$, $\alpha_m=\sqrt{(0.5(1+1/2))}=0.866$

Sway imperfection may be disregarded where Hed>0.15Ved (§5.3.2(4) Eq.5.7)

Effect of initial sway imperfection $Heq=3.960x10^{-3}xVed$ (§5.3.2 (5))

10.3. Sway imperfections for columns

(EN1993-1-1, §5.3.2)

Reactions at the supports, Horizontal Force Hed [kN], Vertical Force Ved [kN]

		Left column 1		Right column 2		Hed1+Hed2	Ved1+Ved2		φ·Ved
		Hed,1	Ved,1	Hed,2	Ved,2	Hed	Ved	Hed/Vhe	Heq kN
201	ULS-STR	56.7	91.3	-56.7	91.3	0.0	182.6	0.00	0.362
202	ULS-STR	75.3	118.6	-75.3	118.6	0.0	237.1	0.00	0.469
203	ULS-STR	63.2	109.7	-63.2	91.9	0.0	201.6	0.00	0.434
204	ULS-STR	63.2	91.9	-63.2	109.7	0.0	201.6	0.00	0.434
205	ULS-STR	-26.0	-0.4	-1.6	0.5	-27.6	0.1	347.55	0.000
206	ULS-STR	2.0	46.0	-23.6	25.4	-21.5	71.4	0.30	0.000
210	ULS-STR	-33.0	-12.7	5.3	-11.9	-27.6	-24.6	1.12	0.000
211	ULS-STR	43.6	89.8	-60.1	90.3	-16.6	180.1	0.09	0.358
212	ULS-STR	60.4	117.6	-73.3	105.3	-12.9	222.9	0.06	0.466
213	ULS-STR	31.5	80.9	-48.0	63.7	-16.6	144.6	0.11	0.321
214	ULS-STR	48.3	108.7	-61.2	78.7	-12.9	187.4	0.07	0.431
215	ULS-STR	31.5	63.2	-48.0	81.4	-16.6	144.6	0.11	0.323
216	ULS-STR	48.3	91.0	-61.2	96.4	-12.9	187.4	0.07	0.382
217	ULS-STR	-1.8	35.1	-25.8	36.0	-27.6	71.1	0.39	0.000
218	ULS-STR	-7.8	30.7	-19.8	22.7	-27.6	53.4	0.52	0.000
219	ULS-STR	-7.8	21.8	-19.8	31.5	-27.6	53.4	0.52	0.000
220	ULS-STR	26.2	81.5	-47.8	61.0	-21.5	142.5	0.15	0.000
221	ULS-STR	20.2	77.0	-41.7	47.6	-21.5	124.7	0.17	0.000
222	ULS-STR	20.2	68.2	-41.7	56.5	-21.5	124.7	0.17	0.000
231	ULS-STR	19.4	54.3	-35.9	54.8	-16.6	109.1	0.15	0.000
232	ULS-STR	36.2	82.1	-49.1	69.8	-12.9	151.9	0.09	0.325
233	ULS-STR	13.3	49.9	-29.9	41.5	-16.6	91.3	0.18	0.000
234	ULS-STR	30.1	77.7	-43.0	56.5	-12.9	134.1	0.10	0.308
235	ULS-STR	13.3	41.0	-29.9	50.4	-16.6	91.3	0.18	0.000
236	ULS-STR	30.1	68.8	-43.0	65.4	-12.9	134.1	0.10	0.272
251	ULS-STR	39.6	82.8	-56.2	83.3	-16.6	166.1	0.10	0.330
252	ULS-STR	56.4	110.6	-69.3	98.3	-12.9	208.9	0.06	0.438
253	ULS-STR	27.5	73.9	-44.1	56.6	-16.6	130.5	0.13	0.293
254	ULS-STR	44.3	101.7	-57.2	71.6	-12.9	173.3	0.07	0.403
255	ULS-STR	27.5	56.1	-44.1	74.4	-16.6	130.5	0.13	0.295
256	ULS-STR	44.3	83.9	-57.2	89.4	-12.9	173.3	0.07	0.354
257	ULS-STR	-5.8	28.1	-21.9	28.9	-27.6	57.0	0.48	0.000
258	ULS-STR	-11.8	23.7	-15.8	15.6	-27.6	39.3	0.70	0.000
259	ULS-STR	-11.8	14.8	-15.8	24.5	-27.6	39.3	0.70	0.000
260	ULS-STR	22.3	74.4	-43.8	53.9	-21.5	128.4	0.17	0.000
261	ULS-STR	16.2	70.0	-37.7	40.6	-21.5	110.6	0.19	0.000
262	ULS-STR	16.2	61.1	-37.7	49.5	-21.5	110.6	0.19	0.000

10.4. Internal forces and bending moments with imperfection effect

10.5. Axial forces Ned [kN]

L.C.		Left column 1 Ned,1	Left rafter 2 Ned,2	Right rafter 3 Ned,3	Right column 4 Ned,4
201	ULS-STR	-87.2	-62.9	-62.9	-87.3
202	ULS-STR	-114.4	-83.5	-83.5	-114.6
203	ULS-STR	-105.5	-70.1	-70.1	-88.0
204	ULS-STR	-87.8	-70.1	-70.1	-105.7
205	ULS-STR	4.4	-5.1	-5.3	3.6
206	ULS-STR	-41.9	-31.5	-28.2	-21.4
210	ULS-STR	15.7	2.6	2.4	14.9
211	ULS-STR	-85.7	-68.7	-68.8	-86.3
212	ULS-STR	-113.5	-84.6	-82.6	-101.4
213	ULS-STR	-76.8	-55.3	-55.4	-59.7
214	ULS-STR	-104.6	-71.2	-69.2	-74.7
215	ULS-STR	-59.1	-55.3	-55.4	-77.5
216	ULS-STR	-86.9	-71.2	-69.2	-92.5
217	ULS-STR	-31.1	-31.9	-32.1	-31.9
218	ULS-STR	-26.6	-25.2	-25.4	-18.6
219	ULS-STR	-17.8	-25.2	-25.4	-27.5
220	ULS-STR	-77.4	-58.3	-55.0	-56.9
221	ULS-STR	-73.0	-51.6	-48.3	-43.6
222	ULS-STR	-64.1	-51.6	-48.3	-52.5
231	ULS-STR	-50.2	-41.8	-41.9	-50.8
232	ULS-STR	-78.0	-57.7	-55.7	-65.8
233	ULS-STR	-45.8	-35.1	-35.2	-37.4
234	ULS-STR	-73.6	-51.0	-49.0	-52.5
235	ULS-STR	-36.9	-35.1	-35.2	-46.3
236	ULS-STR	-64.7	-51.0	-49.0	-61.4
251	ULS-STR	-79.3	-64.3	-64.4	-79.9
252	ULS-STR	-107.0	-80.2	-78.2	-94.9
253	ULS-STR	-70.4	-50.9	-51.0	-53.3
254	ULS-STR	-98.2	-66.8	-64.8	-68.3
255	ULS-STR	-52.6	-50.9	-51.0	-71.0
256	ULS-STR	-80.4	-66.7	-64.7	-86.0
257	ULS-STR	-24.6	-27.5	-27.7	-25.5
258	ULS-STR	-20.2	-20.8	-21.0	-12.2
259	ULS-STR	-11.3	-20.8	-21.0	-21.1
260	ULS-STR	-71.0	-53.9	-50.6	-50.5
261	ULS-STR	-66.6	-47.2	-43.9	-37.2
262	ULS-STR	-57.7	-47.2	-43.9	-46.0

10.6. Shearing forces Ved [kN]

L.C.		Left column 1		Left rafter 2			Right rafter 3			Right column 4	
		VedA,1	VedB,1	VedA,2	VedC,2	VedB,2	VedA,3	VedC,3	VedB,3	VedA,4	VedB,4
201	ULS-STR	-56.5	-56.5	72.7	56.5	-9.4	9.3	-56.6	-72.8	56.9	56.9
202	ULS-STR	-75.0	-75.0	96.4	75.0	-12.5	12.3	-75.1	-96.6	75.5	75.5
203	ULS-STR	-62.9	-62.9	89.7	68.2	-19.3	1.6	-57.8	-72.3	63.4	63.4
204	ULS-STR	-62.9	-62.9	72.2	57.6	-1.7	19.1	-68.4	-89.9	63.4	63.4
205	ULS-STR	26.0	-2.3	-10.0	-8.6	-2.9	-2.3	6.9	9.2	-10.5	1.6
206	ULS-STR	-2.0	-30.4	32.6	23.2	-15.1	-6.2	-11.1	-12.3	11.4	23.6
210	ULS-STR	33.0	4.6	-18.9	-15.5	-1.8	-3.4	13.8	18.1	-17.5	-5.3
211	ULS-STR	-43.4	-60.4	69.8	53.7	-11.6	8.3	-54.9	-70.5	53.0	60.3
212	ULS-STR	-60.1	-77.1	95.3	72.8	-18.9	5.9	-65.8	-83.4	66.2	73.5
213	ULS-STR	-31.3	-48.3	63.0	47.0	-18.3	-2.4	-37.5	-46.2	40.9	48.2
214	ULS-STR	-48.0	-65.0	88.6	66.0	-25.7	-4.8	-48.4	-59.1	54.1	61.4
215	ULS-STR	-31.3	-48.3	45.5	36.4	-0.8	15.1	-48.1	-63.7	40.9	48.2
216	ULS-STR	-48.1	-65.1	71.1	55.4	-8.1	12.7	-59.0	-76.6	54.1	61.4
217	ULS-STR	1.8	-26.6	21.1	15.5	-6.9	1.7	-17.2	-21.9	13.7	25.8
218	ULS-STR	7.8	-20.5	17.7	12.2	-10.3	-3.7	-8.6	-9.8	7.6	19.8
219	ULS-STR	7.8	-20.5	8.9	6.9	-1.5	5.1	-13.9	-18.5	7.6	19.8
220	ULS-STR	-26.2	-54.6	63.7	47.3	-19.1	-2.2	-35.3	-43.4	35.6	47.8
221	ULS-STR	-20.2	-48.5	60.3	43.9	-22.4	-7.6	-26.6	-31.3	29.6	41.7
222	ULS-STR	-20.2	-48.5	51.5	38.6	-13.7	1.2	-31.9	-40.0	29.6	41.7
231	ULS-STR	-19.4	-36.4	38.8	29.7	-7.5	4.4	-30.7	-39.3	28.6	35.9
232	ULS-STR	-36.0	-53.0	64.3	48.7	-14.9	2.0	-41.6	-52.3	41.9	49.2
233	ULS-STR	-13.3	-30.3	35.5	26.3	-10.9	-1.0	-22.0	-27.2	22.6	29.9
234	ULS-STR	-29.9	-47.0	60.9	45.3	-18.3	-3.4	-32.9	-40.2	35.9	43.2
235	ULS-STR	-13.3	-30.3	26.7	21.0	-2.1	7.8	-27.3	-36.0	22.6	29.9
236	ULS-STR	-30.0	-47.0	52.2	40.0	-9.5	5.4	-38.2	-48.9	35.9	43.2
251	ULS-STR	-39.4	-56.4	64.7	49.8	-10.9	7.7	-50.9	-65.3	49.0	56.3
252	ULS-STR	-56.1	-73.2	90.2	68.8	-18.2	5.3	-61.8	-78.3	62.2	69.5
253	ULS-STR	-27.3	-44.3	57.9	43.0	-17.7	-3.1	-33.6	-41.0	36.9	44.2
254	ULS-STR	-44.1	-61.1	83.5	62.1	-25.0	-5.5	-44.4	-54.0	50.1	57.4
255	ULS-STR	-27.3	-44.3	40.4	32.4	-0.2	14.4	-44.2	-58.6	36.9	44.2
256	ULS-STR	-44.1	-61.1	66.0	51.5	-7.5	12.1	-55.0	-71.5	50.1	57.4
257	ULS-STR	5.8	-22.6	16.0	11.6	-6.3	1.0	-13.3	-16.8	9.7	21.9
258	ULS-STR	11.8	-16.5	12.6	8.2	-9.7	-4.3	-2.9	-4.6	3.7	15.8
259	ULS-STR	11.8	-16.5	3.8	2.9	-0.9	4.4	-9.9	-13.4	3.7	15.8
260	ULS-STR	-22.3	-50.6	58.5	43.4	-18.4	-2.9	-31.3	-38.3	31.6	43.8
261	ULS-STR	-16.2	-44.6	55.2	40.0	-21.8	-8.3	-22.6	-26.2	25.6	37.7
262	ULS-STR	-16.2	-44.6	46.4	34.7	-13.0	0.5	-27.9	-34.9	25.6	37.7

A:left end, C:haunch end, B: right end

10.7. Bending moments Med [kNm]

L.C.		Left column 1			Right column 4		
		MedA,1	MedM,1	MedB,1	MedA,4	MedM,4	MedB,4
201	ULS-STR	0.0	-141.3	-282.5	-284.3	-142.2	0.0
202	ULS-STR	0.0	-187.5	-375.0	-377.4	-188.7	0.0
203	ULS-STR	0.0	-157.3	-314.6	-316.8	-158.4	0.0
204	ULS-STR	0.0	-157.3	-314.6	-316.8	-158.4	0.0
205	ULS-STR	0.0	59.6	59.1	22.2	-0.5	0.0
206	ULS-STR	0.0	-40.5	-81.1	-87.4	-43.7	0.0
210	ULS-STR	0.0	47.0	94.0	57.1	28.5	0.0
211	ULS-STR	0.0	-129.6	-259.3	-283.2	-141.6	0.0
212	ULS-STR	0.0	-171.5	-343.1	-349.2	-174.6	0.0
213	ULS-STR	0.0	-99.4	-198.9	-222.6	-111.3	0.0
214	ULS-STR	0.0	-141.3	-282.7	-288.6	-144.3	0.0
215	ULS-STR	0.0	-99.4	-198.9	-222.6	-111.3	0.0
216	ULS-STR	0.0	-141.4	-282.8	-288.5	-144.3	0.0
217	ULS-STR	0.0	0.3	-61.9	-98.8	-49.4	0.0
218	ULS-STR	0.0	5.4	-31.7	-68.5	-34.3	0.0
219	ULS-STR	0.0	5.4	-31.7	-68.5	-34.3	0.0
220	ULS-STR	0.0	-101.1	-202.1	-208.5	-104.2	0.0
221	ULS-STR	0.0	-85.9	-171.9	-178.2	-89.1	0.0
222	ULS-STR	0.0	-85.9	-171.9	-178.2	-89.1	0.0
231	ULS-STR	0.0	-69.6	-139.3	-161.4	-80.7	0.0
232	ULS-STR	0.0	-111.2	-222.5	-227.9	-114.0	0.0
233	ULS-STR	0.0	-54.5	-109.0	-131.2	-65.6	0.0
234	ULS-STR	0.0	-96.1	-192.3	-197.6	-98.8	0.0
235	ULS-STR	0.0	-54.5	-109.0	-131.2	-65.6	0.0
236	ULS-STR	0.0	-96.2	-192.4	-197.5	-98.8	0.0
251	ULS-STR	0.0	-119.7	-239.5	-263.3	-131.6	0.0
252	ULS-STR	0.0	-161.6	-323.3	-329.3	-164.6	0.0
253	ULS-STR	0.0	-89.5	-179.1	-202.7	-101.3	0.0
254	ULS-STR	0.0	-131.4	-262.9	-268.7	-134.3	0.0
255	ULS-STR	0.0	-89.5	-179.1	-202.7	-101.3	0.0
256	ULS-STR	0.0	-131.5	-263.0	-268.6	-134.3	0.0
257	ULS-STR	0.0	2.9	-42.0	-78.9	-39.5	0.0
258	ULS-STR	0.0	12.3	-11.8	-48.6	-24.3	0.0
259	ULS-STR	0.0	12.3	-11.8	-48.6	-24.3	0.0
260	ULS-STR	0.0	-91.1	-182.2	-188.6	-94.3	0.0
261	ULS-STR	0.0	-76.0	-152.0	-158.3	-79.2	0.0
262	ULS-STR	0.0	-76.0	-152.0	-158.3	-79.2	0.0

A:left end, B: right end

L.C.		Left rafter 2				Right rafter 3			
		MedA,2	MedC2	MedM,2	MedB,2	MedA,3	MedM,3	MedC3	MedB,3
201	ULS-STR	-282.5	-127.6	108.7	102.1	102.1	108.5	-129.0	-284.3
202	ULS-STR	-375.0	-169.3	144.3	135.6	135.6	144.0	-171.2	-377.4
203	ULS-STR	-314.6	-125.1	134.5	113.8	113.8	114.0	-160.6	-316.8
204	ULS-STR	-314.6	-158.9	114.0	113.8	113.8	134.1	-126.9	-316.8
205	ULS-STR	59.1	36.8	19.8	-19.6	-19.6	-22.3	3.0	22.2
206	ULS-STR	-81.1	-14.1	54.5	25.5	25.5	-31.0	-59.2	-87.4
210	ULS-STR	94.0	52.6	30.9	-32.1	-32.1	-35.5	18.7	57.1
211	ULS-STR	-259.3	-111.0	104.9	94.8	94.8	100.1	-132.8	-283.2
212	ULS-STR	-343.1	-141.3	140.8	121.8	121.8	124.2	-170.3	-349.2
213	ULS-STR	-198.9	-66.8	98.2	73.0	73.0	-74.8	-122.2	-222.6
214	ULS-STR	-282.7	-97.2	135.0	100.0	100.0	-94.3	-159.7	-288.6
215	ULS-STR	-198.9	-100.6	73.1	73.0	73.0	90.6	-88.4	-222.6
216	ULS-STR	-282.8	-131.0	105.1	100.0	100.0	111.0	-125.9	-288.5
217	ULS-STR	-61.9	-18.0	34.5	24.1	24.1	24.8	-51.8	-98.8
218	ULS-STR	-31.7	4.1	36.3	13.2	13.2	-27.7	-46.6	-68.5
219	ULS-STR	-31.7	-12.7	14.6	13.2	13.2	19.8	-29.7	-68.5
220	ULS-STR	-202.1	-68.9	95.8	69.1	69.1	-69.7	-114.1	-208.5
221	ULS-STR	-171.9	-46.8	95.3	58.2	58.2	-60.0	-108.8	-178.2
222	ULS-STR	-171.9	-63.7	75.7	58.2	58.2	58.4	-91.9	-178.2
231	ULS-STR	-139.3	-57.0	58.7	51.3	51.3	54.0	-77.3	-161.4
232	ULS-STR	-222.5	-86.9	95.2	78.2	78.2	78.6	-115.2	-227.9
233	ULS-STR	-109.0	-34.9	56.0	40.4	40.4	-45.4	-72.1	-131.2
234	ULS-STR	-192.3	-64.8	92.9	67.3	67.3	-65.2	-109.9	-197.6
235	ULS-STR	-109.0	-51.8	41.3	40.4	40.4	48.8	-55.2	-131.2
236	ULS-STR	-192.4	-81.7	76.2	67.3	67.3	70.5	-93.0	-197.5
251	ULS-STR	-239.5	-102.1	97.3	87.7	87.7	92.5	-123.7	-263.3
252	ULS-STR	-323.3	-132.4	133.3	114.6	114.6	116.7	-161.2	-329.3
253	ULS-STR	-179.1	-57.9	91.0	65.8	65.8	-68.4	-113.1	-202.7
254	ULS-STR	-262.9	-88.2	127.9	92.8	92.8	-87.9	-150.6	-268.7
255	ULS-STR	-179.1	-91.6	65.8	65.8	65.8	83.2	-79.4	-202.7
256	ULS-STR	-263.0	-122.1	97.4	92.8	92.8	103.4	-116.8	-268.6
257	ULS-STR	-42.0	-9.0	27.7	16.9	16.9	17.3	-42.8	-78.9
258	ULS-STR	-11.8	13.2	31.5	6.0	6.0	-21.3	-37.9	-48.6
259	ULS-STR	-11.8	-3.7	7.0	6.0	6.0	12.7	-20.7	-48.6
260	ULS-STR	-182.2	-59.9	88.7	61.9	61.9	-63.3	-105.0	-188.6
261	ULS-STR	-152.0	-37.8	88.6	51.0	51.0	-53.6	-99.8	-158.3
262	ULS-STR	-152.0	-54.7	68.4	51.0	51.0	51.1	-82.9	-158.3

A:left end, C:haunch end, M: span, B: right end

11. Serviceability Limit State (SLS) (EN1993-1-1, §7)

11.1. Vertical deflection at the apex (EN1993-1-1, §7.2.1)

```
Maximum  vertical deflection, L.C. 302: Gk+Qs1     Dy= 44 mm=24000/545=L/545
Vertical deflection due to imposed load only      Dy= 17 mm=24000/889=L/1412
Vertical deflection due to snow only              Dy= 27 mm=24000/889=L/889
Limit for vertical deflection L/200,  Is verified
```

11.2. Horizontal deflection at the top of column (EN1993-1-1, §7.2.2)

```
Maximum  horizontal deflection, L.C. 304: Gk+Qs3    Dx=  8 mm= 5000/625=h/625
Horizontal deflection due to wind only              Dx=  6 mm= 5000/833=h/833
Limit for horizontal deflection H/150,  Is verified
```

11.3. Dynamic effects (EN1993-1-1, §7.2.3)

Eigenfrequencies and Eigenperiods of the structure

```
Mass of building, for loading: L.C. 601: Gk + 0.20Qs1
  1  f=   2.616 Hz  T=   0.382 sec
  2  f=   3.896 Hz  T=   0.257 sec
  3  f=  10.732 Hz  T=   0.093 sec
  4  f=  29.103 Hz  T=   0.034 sec
  5  f=  42.721 Hz  T=   0.023 sec
  6  f=  59.397 Hz  T=   0.017 sec
  7  f= 112.457 Hz  T=   0.009 sec
  8  f= 149.552 Hz  T=   0.007 sec
  9  f= 153.281 Hz  T=   0.007 sec
```

12. Column verification (Ultimate Limit State) (EN1993-1-1, §6)

Profile : IPE 600-S 355

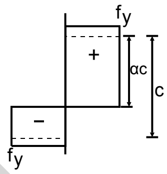
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

Ned = 118.7 kN
Ved = 75.5 kN
Myed = 377.4 kNm, Mzed =0.0 kNm
Myed = 320.8 kNm (Column top under the haunch)
Buckling length,In-plane buckling Lcr,y =5000mm (System length) (EC3 §5.5.2.(7))
Buckling length,Out-of-plane buckling Lcr,z =4250mm (Column height without haunch)
Buckling length,Torsional buckling Lcr,t =4250mm
Buckling length,Lateral torsional buckling Lcr,lt=4250mm

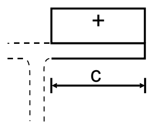
12.1. Classification of cross-sections, Column (EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
σ=[10^{-3}]119/15600 ± [10^{-6}]377/3069.0x10³ ± [10^{-6}]0/307.9x10³
σ1=131 N/mm², σ2=-115 N/mm² (comression positive)

Web
c=600.0-2x19.0-2x24.0=514.0 mm, t=12.0 mm, c/t=514.0/12.0=42.83
S 355 , t=12.0<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
Position of neutral axis for combined Bending and compression
Ned/(2tw·fy/γM0)=118700/(2x12.0x355/1.00)=13.9 mm
α=(514.0/2+13.9)/514.0=0.527>0.5
c/t=42.83<=396x0.81/(13x0.527-1)=54.81
The web is class 1 (EN1993-1-1, Tab.5.2)



Flange
c=220.0/2-12.0/2-24.0=80.0 mm, t=19.0 mm, c/t=80.0/19.0=4.21
S 355 , t=19.0<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
c/t=4.21<=9ε=9x0.81=7.29
The flange is class 1 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 1, Bending and compression Nc,ed + My,ed

12.2. Resistance of cross-section, Column (Ultimate Limit State) (EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Nc.ed=118.70 kN
Compression Resistance Nplrd= A·fy/γM0=[10^{-3}]x15600x355/1.00=5538.00kN
Ned= 118.70 kN < 5538.00 kN =Nc,rd=Nplrd, Is verified

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
My.ed=377.40 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x3512.0x10^{3}x355/1.00= 1246.76kNm
My,ed= 377.40 kNm < 1246.76 kNm =My,rd=Mply,rd, Is verified

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)
Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2
Vz.ed= 77.10 kNm
Av=A-2b·tf+(tw+2r)tf=15600-2x220.0x19.0+(12.0+2x24.0)x19.0=8380mm² (EC3 §6.2.6.3)
Av= 8380mm² > η·hw·tw= 1.00x(600.0-2x19.0)x12.0=1.00x581.0x12.0= 6972mm²
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x8380x(355/1.73)/1.00= 1717.56kN
Vz,ed= 77.10 kN < 1717.56 kN =Vz,rd=Vpl,z,rd, Is verified

hw/tw=(600.0-2x19.0)/12.0=581.0/12.0=48.42<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t=12.0<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
Shear buckling resistance is not necessary to be verified (EC3 §6.2.6.6)

Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)

```
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
N.ed= 118.70kN (Compression), Vz.ed= 75.50kNm,  My.ed= 377.40kN
Nplrd=5538.00kN, Mpl,y,rd=1246.76kNm, Vpl,z,rd=1717.56kN
Ned=118.70kN <= 0.25x5538.00=0.25xNplrd=1384.50kN
Ned=118.70kN <= [10^-3]x0.5x581.0x12.0x355/1.00=0.5hw·tw·fy/γM0=1237.53 kN
n=Ned/Nplrd=119/5538= 0.021
Effect of axial force is neglected                (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=75.50kN <= 0.50x1717.56=0.50xVpl,rd=858.78kN
Effect of shear force is neglected                                           (EC3 §6.2.8.2)

My,ed= 377.40 kNm < 1246.76 kNm =Mply,rd,  Is verified
```

12.3. Flexural Buckling, Column (Ultimate Limit State) (EN1993-1-1, §6.3.1)

```
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Buckling lengths: Lcr,y=1.000x5000=5000mm, Lcr,z=0.850x5000=4250mm
Non-dimensional slenderness (Cross-section Class: 1 )                       (EC3 §6.3.1.3)
λ̄y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(5000/243.0)x(1/76.06)=0.271
λ̄z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(4250/ 46.6)x(1/76.06)=1.199
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=600/220=2.73>=1.20, tf=19.0mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.984          (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(λ̄y-0.2)+λ̄y^2]=0.5x[1+0.21x(0.271-0.2)+0.271^2]=0.544
χy=1/[Φy+√(Φy^2-λ̄y^2)]=1/[0.544+√(0.544^2-0.271^2)]=0.984 <=1 χy=0.984
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.479
Φz=0.5[1+αz(λ̄z-0.2)+λ̄z^2]=0.5x[1+0.34x(1.199-0.2)+1.199^2]=1.389
χz=1/[Φz+√(Φz^2-λ̄z^2)]=1/[1.389+√(1.389^2-1.199^2)]=0.479 <=1 χz=0.479

Reduction factor χ=1/[Φ+√(Φ^2-λ̄^2)], χ<=1.0, Φ=0.5[1+α(λ̄-0.2)+λ̄^2], χ=0.479       (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.479x[10^-3]x15600x355/1.00=2652.70kN                          (EC3 Eq.6.47)
Nc,ed= 118.70 kN < 2652.70 kN =Nb,rd,  Is verified
```

12.4. Lateral torsional buckling , Column (EN1993-1-1, §6.3.2)

```
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Elastic critical moment for lateral-torsional buckling    (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
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Timoshenko,S.P, Gere,J.M, Theory of elastic stability, McGraw-Hill, 1961

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Mcr=C1·[π^2EIz/(kL)^2]{√[(kz/kw)^2(Iw/Iz)+(kL)^2GIt/(π^2EIz)+(C2·zg-C3·zj)^2] -(C2·zg-C3·zj)}
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^4 N/mm^2
k·L=4250mm,  zg=h/2=600/2=300mm, zj=0mm                                (EN1993:2002 T.C.1)
kz=1.0, kw=1.0, ψ=0.000, C1=1.847, C2=0.000, C3=0.000                   (EN1993:2002 T.C.1)
Mcr=[10^-6]1.847x[π^2x2.1x10^5x33.870x10^6/4250^2]
   x{ [1.0x(2845.5x10^9/33.870x10^6)
   +4250^2x8.1x10^4x1.654x10^6/(π^2x2.1x10^5x33.870x10^6)]^0.5 }= 2469.9 kNm

λ̄,lt=√(Wpl,y·fy/Mcr)=√{[10^-6]x3512.0x10^3x355/2469.9}=0.710                  (EC3 Eq.6.56)
h/b=600/220=2.73>2.00 buckling curve:c
imperfection factor:α,lt=0.49, β=0.75, χ,lt=0.819                     (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt(λ̄,lt-λ̄,lto)+βλ̄,lt^2]=0.5x[1+0.49x(0.710-0.40)+0.75x0.710^2]=0.765
χ,lt=1/[Φ,lt+√(Φ,lt^2-βλ̄,lt^2)]=1/[0.765+√(0.765^2-0.75x0.765^2)]=0.819
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt^2-βλ̄,lt^2)], χ,lt<=1.0, 1/λ̄,lt^2, χ,lt=0.819     (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/λ̄,lt^2=1/0.710^2=1.98     (EC3 §6.3.2.3(2),Eq.6.58)
Kc=1/(1.33-0.33ψ)=0.752, ψ=0.00                                                   (EC3 Tab.6.6)
f=1-0.5(1-kc)[1-2.0(λ̄,lt-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.710-0.8)^2]=0.878, f<=1.0
χ,lt,mod=χ,lt/f=0.819/0.878=0.933, χ,lt,mod<=1.0, χ,lt,mod<=1.98,  χ,lt,mod=0.933

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.933x[10^-6]x3512.0x10^3x355/1.00=1163.23kNm            (EC3 Eq.6.55)
My,ed= 320.77 kNm < 1163.23 kNm =Mb,rd,  Is verified
```

12.5. Axial force and bending moment, Column (EN1993-1-1, §6.3.3)

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.61)
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.62)
Nrk=A·fy=[10^{-3}]x15600x355=5538.0 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x3512.0x10^{3}x355=1246.8 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.984x[10^{-3}]x15600x355/1.00=5449.4kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.479x[10^{-3}]x15600x355/1.00=2652.7kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.933x[10^{-6}]x3512.0x10^{3}x355/1.00=1163.2kNm

Interaction factors, Method of computation: Method 1 Annex A (EC3 AnnexA)

kyy=Cmy·CmLT(μy/(1-Ned/Ncr,y)(1/Cyy), μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y) (EC3 Tab.A.1)
kzy=Cmy·CmLT(μz/(1-Ned/Ncr,y)(1/Czy)0.60√(wy/wz), μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)

Ncr,y=π²EIy/lcr,y²=3.14²x[10^{-3}]x210000x920.80x10^{6}/ 5000²= 76339 kN
Ncr,z=π²EIz/lcr,z²=3.14²x[10^{-3}]x210000x33.870x10^{6}/ 4250²= 3886 kN
Ncr,t=(1/ip²)x(G·It + π²EIw/Lcr,t²) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10^{-3}]x(1/247²)[80769x 1.654x10^{6}+π²x210000x2845.5x10^{9}/4250²]=7519 kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 118.7/ 76339)/(1-0.984x 118.7/ 76339)=1.000
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 118.7/ 3886)/(1-0.479x 118.7/ 3886)=0.984
alt=1-It/Iy>=0=1- 1.654x10^{6}/920.80x10^{6}=0.998 (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 3.512x10^{6}/ 3.069x10^{6}=1.144 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.486x10^{6}/ 0.308x10^{6}=1.577 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=118.70/(5538.00/1.00)=0.021

$\bar{\lambda}$max=max(0.271,1.199)=1.200 (EC3 Annex A.1)
Mcr,o=(1.00/1.85)x2469.90=1337.3, C1=1.00
$\bar{\lambda}$o=√([10^{-6}]x3512.0x10^{3}x355/1337.3)=0.970
$\bar{\lambda}$o,lim=0.2√C1 [(1-Ned/Ncr,z)(1-Ned/Ncr,t)]$^{0.25}$ (EC3 Annex A.1)
$\bar{\lambda}$o,lim=0.2√1.847 [(1-118.7/3886)(1-118.7/7519)]$^{0.25}$=0.269
εy=(My,ed/Ned)(A/Wel)=([10^{3}]x320.77/118.70)x(15600.0/3069.0x10^{3})=13.74

Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(118.70/76339.0)=0.790, (ψ=0.00) (EC3 Annex A, T.A.1)
$\bar{\lambda}$o=0.970 > $\bar{\lambda}$o,lim=0.269
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
=0.790+(1-0.790)x(√13.736x0.998)/(1+√13.736x0.998) = 0.955
Cmlt=Cmy²·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=0.955²x0.998/√[(1-118.7/3886.0)(1-118.7/7519.0)]=0.932, Cmlt=1.000

Cyy=1+(wy-1)[(2-1.6Cmy²·$\bar{\lambda}$max/wy-1.6Cmy²·$\bar{\lambda}$max²/wy)npl-blt]>=Wel,y/Wpl,y (Annex A, T.A.1)
blt=0.5alt·$\bar{\lambda}$o²[My,ed/(χ,lt·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
=0.5x0.998x0.970²[320.8/(0.933x1089.5)](0.0/109.3) = 0.000
Cyy=1+(1.144-1)[(2-1.6x0.955²x1.200/1.144-1.6x0.955²x1.200²/1.144)x0.021-0.000]=0.996
Cyy>=3069.0x10^{3}/3512.0x10^{3}=0.874, Cyy=0.996

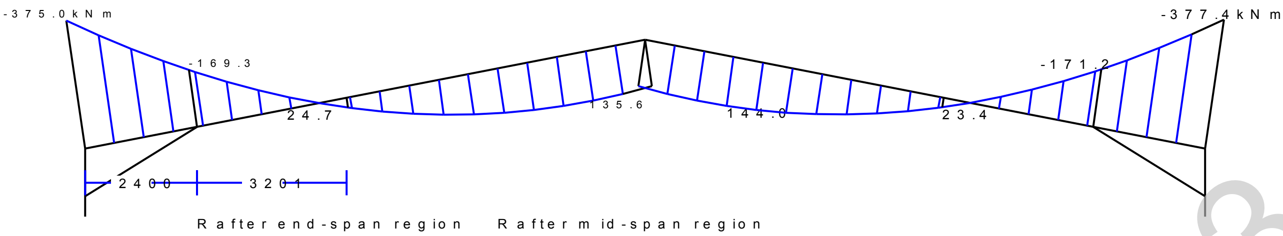
Czy=1+(wy-1)[(2-14.0Cmy²·$\bar{\lambda}$max²/wy^{5})npl-dlt]>=0.6√(wy/wz)(Wel,y/Wpl,y) (Annex A, T.A.1)
dlt=2alt·[$\bar{\lambda}$o/(0.1+$\bar{\lambda}$z^{4})][My,ed/(Cmy·χ,lt·Mpl,y,rd)][Mz,ed/(Cmz·Mpl,z,rd)]=
=20.998x[0.970/(0.1+1.199^{4})][320.8/(0.955x0.933x1089.5)][0.0/(0.000x109.3)]=0.000
Czy=1+(1.144-1)[(2-14.0x0.955²x1.200²/1.144^{5})0.021-0.000]=0.978
Czy>=0.6√(1.144/1.500)(3069.0x10^{3}/3512.0x10^{3})=0.458, Czy=0.978

Cyy=0.996, Czy=0.978 (Annex A, T.A.1)
kyy=0.955x1.000x1.000/(1-118.70/76339.0)x(1/0.996)=0.960
kzy=0.955x1.000x0.984/(1-118.70/76339.0)x(1/0.978)x0.6x√(1.144/1.500)=0.504

```
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
 Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)=                                (EC3 Eq.6.61)
 118.7/(0.984x5538.0/1.00)+0.960x320.8/(0.933x1246.8/1.00)=0.022+0.265=0.287
               0.287< 1.000,  Is verified
 Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)=                                (EC3 Eq.6.62)
 118.7/(0.479x5538.0/1.00)+0.504x320.8/(0.933x1246.8/1.00)=0.045+0.139=0.184
               0.184< 1.000,  Is verified
```

13. Rafter verification (Ultimate Limit State) (EN1993-1-1, §6)

L.C. 202 Bend. moments kNm



Profile : IPE 500-S 355

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

Ned = 83.5 kN
Ved = 75.1 kN
Myed = 171.2 kNm, Mzed =0.0 kNm
Myed = 144.3 kNm (at mid-span)
Myed = -171.2 kNm (at haunch-start)
Myed = -348.4 kNm (at haunch end)
Myed = -377.4 kNm (at column axis point)

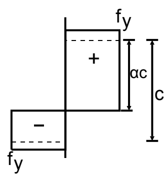
Maximum design valuesRafter-Uplift conditions: L.C. 210: 1.00Gk+1.50Qw1

Ned = 2.4 kN
Ved = 13.8 kN
Myed = -35.5 kNm

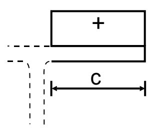
13.1. Classification of cross-sections, Rafter (EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
$\sigma=[10^{-3}]84/11550 \pm [10^{-6}]171/1928.0x10^3 \pm [10^{-6}]0/214.2x10^3$
σ1=96 N/mm², σ2=-82 N/mm² (comression positive)

Web
c=500.0-2x16.0-2x21.0=426.0 mm, t=10.2 mm, c/t=426.0/10.2=41.76
S 355 , t=10.2<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}$=0.81
Position of neutral axis for combined Bending and compression
Ned/(2tw·fy/γM0)=83500/(2x10.2x355/1.00)=11.5 mm
α=(426.0/2+11.5)/426.0=0.527>0.5
c/t=41.76<=396x0.81/(13x0.527-1)=54.81
The web is class 1 (EN1993-1-1, Tab.5.2)



Flange
c=200.0/2-10.2/2-21.0=73.9 mm, t=16.0 mm, c/t=73.9/16.0=4.62
S 355 , t=16.0<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}$=0.81
c/t=4.62<=9ε=9x0.81=7.29
The flange is class 1 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 1, Bending and compression Nc,ed + My,ed

13.2. Resistance of cross-section, Rafter (Ultimate Limit State) (EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)
Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2
Nc.ed= 84.60 kN
Compression Resistance Nplrd= A·fy/γM0=$[10^{-3}]$x11550x355/1.00=4100.25kN
Ned= 84.60 kN < 4100.25 kN =Nc,rd=Nplrd, Is verified

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
My.ed=171.20 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x2194.0x10^{3}x355/1.00= 778.87kNm
My,ed= 171.20 kNm < 778.87 kNm =My,rd=Mply,rd, Is verified

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Vz.ed= 75.10 kNm
Av=A-2b·tf+(tw+2r)tf=11550-2x200.0x16.0+(10.2+2x21.0)x16.0=5985mm^2 (EC3 §6.2.6.3)
Av= 5985mm^2 > η·hw·tw= 1.00x(500.0-2x16.0)x10.2=1.00x484.0x10.2= 4937mm^2
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x5985x(355/1.73)/1.00= 1226.72kN
Vz,ed= 75.10 kN < 1226.72 kN =Vz,rd=Vpl,z,rd, Is verified

hw/tw=(500.0-2x16.0)/10.2=484.0/10.2=47.45<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t=10.2<= 40 mm, fy=355 N/mm^2, ε=(235/355)$^{0.5}$=0.81
Shear buckling resistance is not necessary to be verified (EC3 §6.2.6.6)

Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
N.ed= 83.50kN (Compression), Vz.ed= 75.10kNm, My.ed= 171.20kN
Nplrd=4100.25kN, Mpl,y,rd=778.87kNm, Vpl,z,rd=1226.72kN
Ned=83.50kN <= 0.25x4100.25=0.25xNplrd=1025.06kN
Ned=83.50kN <= [10^{-3}]x0.5x484.0x10.2x355/1.00=0.5hw·tw·fy/γM0=876.28 kN
n=Ned/Nplrd=84/4100= 0.020
Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=75.10kN <= 0.50x1226.72=0.50xVpl,rd=613.36kN
Effect of shear force is neglected (EC3 §6.2.8.2)

My,ed= 171.20 kNm < 778.87 kNm =Mply,rd, Is verified

13.3. Buckling resistance, Rafter mid-span region (Ultimate Limit State)

Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2
Ned = 83.5 kN
Ved = 96.6 kN
Myed = 144.0 kNm, Mzed =0.0 kNm
Rafter length Lr=√[(24000/2)^2+(7000-5000)^2]=12042 mm
Buckling length,In-plane buckling
αcr=45.47, Ned=83.5kN, Lcr,y=π√[EI/αcr·Ned] >= Lr=12042 mm
Lcr,y=π√[210000x482.00x10^{6}/(45.47x83.5x10^{3})=16217mm], Lcr,y=16217mm
Buckling length,In-plane buckling Lcr,y=16217mm (System length)
Buckling length,Out-of-plane buckling Lcr,z=3000mm (Purlin spacing)

13.4. Flexural Buckling, Rafter mid-span region (Ultimate Limit State) (EN1993-1-1, §6.3.1)

Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2
Buckling lengths: Lcr,y=1.347x12042=16217mm, Lcr,z=0.249x12042=3000mm
Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)
$\bar{\lambda}$y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(16217/204.3)x(1/76.06)=1.044
$\bar{\lambda}$z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(3000/ 43.1)x(1/76.06)=0.916
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=500/200=2.50>=1.20, tf=16.0mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.635 (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy($\bar{\lambda}$y-0.2)+$\bar{\lambda}$y^2]=0.5x[1+0.21x(1.044-0.2)+1.044^2]=1.134
χy=1/[Φy+√(Φy^2-$\bar{\lambda}$y^2)]=1/[1.134+√(1.134^2-1.044^2)]=0.635 <=1 χy=0.635
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.651
Φz=0.5[1+αz($\bar{\lambda}$z-0.2)+$\bar{\lambda}$z^2]=0.5x[1+0.34x(0.916-0.2)+0.916^2]=1.041
χz=1/[Φz+√(Φz^2-$\bar{\lambda}$z^2)]=1/[1.041+√(1.041^2-0.916^2)]=0.651 <=1 χz=0.651

Reduction factor χ=1/[Φ+√(Φ^2-$\bar{\lambda}^2$)], χ<=1.0, Φ=0.5[1+α($\bar{\lambda}$-0.2)+$\bar{\lambda}^2$], χ=0.635 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.635x[10^{-3}]x11550x355/1.00=2603.66kN (EC3 Eq.6.47)
Nc,ed= 83.54 kN < 2603.66 kN =Nb,rd, Is verified

13.5. Lateral torsional buckling , Rafter mid-span region (EN1993-1-1, §6.3.2)

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko,S.P, Gere,J.M, Theory of elastic stability, McGraw-Hill, 1961
Mcr=C1·[π²EIz/(kL)²]{√[(kz/kw)²(Iw/Iz)+(kL)²GIt/(π²EIz)+(C2·zg-C3·zj)²] -(C2·zg-C3·zj)}
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^{4} N/mm²
k·L=3000mm, zg=h/2=500/2=250mm, zj=0mm (EN1993:2002 T.C.1)
kz=1.0, kw=1.0, ψ=1.000, C1=1.000, C2=0.000, C3=0.000 (EN1993:2002 T.C.1)
Mcr=[10^{-6}]1.000x[π²x2.1x10^{5}x21.420x10^{6}/3000²]
x{ [1.0x(1249.4x10^{9}/21.420x10^{6})
+3000²x8.1x10^{4}x0.893x10^{6}/(π²x2.1x10^{5}x21.420x10^{6})]$^{0.5}$ }= 1332.3 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{[10^{-6}]x2194.0x10^{3}x355/1332.3}=0.765 (EC3 Eq.6.56)
h/b=500/200=2.50>2.00 buckling curve:c
imperfection factor:α,lt=0.49, β=0.75, χ,lt=0.786 (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt($\bar{\lambda}$,lt-$\bar{\lambda}$,lto)+β$\bar{\lambda}$,lt²]=0.5x[1+0.49x(0.765-0.40)+0.75x0.765²]=0.809
χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)]=1/[0.809+√(0.809²-0.75x0.809²)]=0.786
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)], χ,lt<=1.0, 1/$\bar{\lambda}$,lt², χ,lt=0.786 (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/ $\bar{\lambda}$,lt²=1/0.765²=1.71 (EC3 §6.3.2.3(2),Eq.6.58)
Kc=1.00 (EC3 Tab.6.6)
f=1-0.5(1-kc)[1-2.0($\bar{\lambda}$,lt-0.8)²]=1-0.5x(1-1.000)[1-2.0x(0.765-0.8)²]=1.000, f<=1.0
χ,lt,mod=χ,lt/f=0.786/1.000=0.786, χ,lt,mod<=1.0, χ,lt,mod<=1.71, χ,lt,mod=0.786

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.786x[10^{-6}]x2194.0x10^{3}x355/1.00=612.19kNm (EC3 Eq.6.55)
My,ed= 144.05 kNm < 612.19 kNm =Mb,rd, Is verified

13.6. Axial force and bending moment, Rafter mid-span region (EN1993-1-1, §6.3.3)

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.61)
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.62)
Nrk=A·fy=[10^{-3}]x11550x355=4100.2 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x2194.0x10^{3}x355=778.9 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.635x[10^{-3}]x11550x355/1.00=2603.7kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.651x[10^{-3}]x11550x355/1.00=2669.3kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.786x[10^{-6}]x2194.0x10^{3}x355/1.00=612.2kNm

Interaction factors, Method of computation: Method 1 Annex A (EC3 AnnexA)

kyy=Cmy·CmLT(μy/(1-Ned/Ncr,y)(1/Cyy), μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y) (EC3 Tab.A.1)
kzy=Cmy·CmLT(μz/(1-Ned/Ncr,y)(1/Czy)0.60√(wy/wz), μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)

Ncr,y=π²EIy/lcr,y²=3.14²x[10^{-3}]x210000x482.00x10^{6}/16217²= 3799 kN
Ncr,z=π²EIz/lcr,z²=3.14²x[10^{-3}]x210000x21.420x10^{6}/ 3000²= 4933 kN
Ncr,t=(1/ip²)x(G·It + π²EIw/Lcr,t²) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10^{-3}]x(1/209²)[80769x 0.893x10^{6}+π²x210000x1249.4x10^{9}/4040²]=5294 kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 83.5/ 3799)/(1-0.635x 83.5/ 3799)=0.992
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 83.5/ 4933)/(1-0.651x 83.5/ 4933)=0.994
alt=1-It/Iy>=0=1- 0.893x10^{6}/482.00x10^{6}=0.998 (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 2.194x10^{6}/ 1.928x10^{6}=1.138 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.336x10^{6}/ 0.214x10^{6}=1.568 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=83.54/(4100.20/1.00)=0.020

$\bar{\lambda}$max=max(1.044,0.916)=1.040 (EC3 Annex A.1)
Mcr,o=(1.00/1.00)x1332.30=1332.3, C1=1.00
$\bar{\lambda}$o=√([10^{-6}]x2194.0x10^{3}x355/1332.3)=0.760
$\bar{\lambda}$o,lim=0.2√C1 [(1-Ned/Ncr,z)(1-Ned/Ncr,t)]$^{0.25}$ (EC3 Annex A.1)
$\bar{\lambda}$o,lim=0.2√1.000 [(1-83.5/4933)(1-83.5/5294)]$^{0.25}$=0.198
εy=(My,ed/Ned)(A/Wel)=([10^{3}]x144.05/83.54)x(11550.0/1928.0x10^{3})=10.33

```
Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(83.54/3799.0)=1.005, (ψ=1.00)            (EC3 Annex A, T.A.1)
λ̄o=0.760 > λ̄o,lim=0.198
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
   =1.005+(1-1.005)x(√10.329x0.998)/(1+√10.329x0.998) = 1.001
Cmlt=Cmy^2·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=1.001^2x0.998/√[(1-83.5/4933.0)(1-83.5/5294.0)]=1.017, Cmlt=1.017

Cyy=1+(wy-1)[(2-1.6Cmy^2·λ̄max/wy-1.6Cmy^2·λ̄max^2/wy)npl-blt]>=Wel,y/Wpl,y       (Annex A, T.A.1)
blt=0.5alt·λ̄o^2[My,ed/(χ,lt·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
   =0.5x0.998x0.760^2[0.0/(0.786x684.4)](0.0/76.0) = 0.000
Cyy=1+(1.138-1)[(2-1.6x1.001^2x1.040/1.138-1.6x1.001^2x1.040^2/1.138)x0.020-0.000]=0.997
Cyy>=1928.0x10^3/2194.0x10^3=0.879, Cyy=0.997

Czy=1+(wy-1)[(2-14.0Cmy^2·λ̄max^2/wy^5)npl-dlt]>=0.6√(wy/wz)(Wel,y/Wpl,y)        (Annex A, T.A.1)
dlt=2alt·[λ̄o/(0.1+λ̄z^4)][My,ed/(Cmy·χ,lt·Mpl,y,rd)][Mz,ed/(Cmz·Mpl,z,rd)]=
   =20.998x[0.760/(0.1+0.916^4)][0.0/(1.001x0.786x684.4)][0.0/(0.000x76.0)]=0.000
Czy=1+(1.138-1)[(2-14.0x1.001^2x1.040^2/1.138^5)0.020-0.000]=0.984
Czy>=0.6√(1.138/1.500)(1928.0x10^3/2194.0x10^3)=0.459, Czy=0.984

Cyy=0.997, Czy=0.984                                                            (Annex A, T.A.1)
kyy=1.001x1.017x0.992/(1-83.54/3799.0)x(1/0.997)=1.036
kzy=1.001x1.017x0.994/(1-83.54/3799.0)x(1/0.984)x0.6x√(1.138/1.500)=0.550
```

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

```
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)=                                   (EC3 Eq.6.61)
83.5/(0.635x4100.2/1.00)+1.036x144.0/(0.786x778.9/1.00)=0.032+0.244=0.276
               0.276< 1.000,  Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)=                                   (EC3 Eq.6.62)
83.5/(0.651x4100.2/1.00)+0.550x144.0/(0.786x778.9/1.00)=0.031+0.129=0.161
               0.161< 1.000,  Is verified
```

13.7. Buckling resistance, Rafter end-span region (Ultimate Limit State)

```
Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2
 Ned  =   92.6 kN
 Ved  =   96.6 kN
 Myed =  171.2 kNm,  Mzed =0.0 kNm
Rafter length Lr=√[(24000/2)^2+(7000-5000)^2]=12042 mm
Buckling length,In-plane buckling     Lcr,y=16217mm (System length)
Buckling length,Out-of-plane buckling Lcr,z=3201mm (Torsional restrains of rafters)
```

13.8. Flexural Buckling, Rafter end-span region (Ultimate Limit State) (EN1993-1-1, §6.3.1)

```
Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2
Buckling lengths: Lcr,y=1.347x12042=16217mm, Lcr,z=0.266x12042=3201mm
Non-dimensional slenderness (Cross-section Class: 1 )                         (EC3 §6.3.1.3)
λ̄y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(16217/204.3)x(1/76.06)=1.044
λ̄z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(3201/ 43.1)x(1/76.06)=0.977
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=500/200=2.50>=1.20, tf=16.0mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.635           (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(λ̄y-0.2)+λ̄y^2]=0.5x[1+0.21x(1.044-0.2)+1.044^2]=1.134
χy=1/[Φy+√(Φy^2-λ̄y^2)]=1/[1.134+√(1.134^2-1.044^2)]=0.635 <=1 χy=0.635
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.612
Φz=0.5[1+αz(λ̄z-0.2)+λ̄z^2]=0.5x[1+0.34x(0.977-0.2)+0.977^2]=1.109
χz=1/[Φz+√(Φz^2-λ̄z^2)]=1/[1.109+√(1.109^2-0.977^2)]=0.612 <=1 χz=0.612

Reduction factor χ=1/[Φ+√(Φ^2-λ̄^2)], χ<=1.0, Φ=0.5[1+α(λ̄-0.2)+λ̄^2], χ=0.612      (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.612x[10^-3]x11550x355/1.00=2509.35kN                      (EC3 Eq.6.47)
Nc,ed= 92.62 kN < 2509.35 kN =Nb,rd,  Is verified
```

13.9. Lateral torsional buckling , Rafter end-span region (EN1993-1-1, §6.3.2)

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko,S.P, Gere,J.M, Theory of elastic stability, McGraw-Hill, 1961

$Mcr=C1\cdot[\pi^2EIz/(kL)^2]\{\sqrt{[(kz/kw)^2(Iw/Iz)+(kL)^2GIt/(\pi^2EIz)+(C2\cdot zg-C3\cdot zj)^2]}-(C2\cdot zg-C3\cdot zj)\}$
$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4$ N/mm²
k·L=3201mm, zg=h/2=500/2=250mm, zj=0mm (EN1993:2002 T.C.1)
kz=1.0, kw=1.0, ψ=-0.137, C1=2.044, C2=0.000, C3=0.000 (EN1993:2002 T.C.1)
$Mcr=[10^{-6}]2.044x[\pi^2x2.1x10^5x21.420x10^6/3201^2]$
$x\{ [1.0x(1249.4x10^9/21.420x10^6)$
$+3201^2x8.1x10^4x0.893x10^6/(\pi^2x2.1x10^5x21.420x10^6)]^{0.5}\}=$ 2425.0 kNm

$\bar\lambda,lt=\sqrt{(Wpl,y\cdot fy/Mcr)}=\sqrt{\{[10^{-6}]x2194.0x10^3x355/2425.0\}}=0.567$ (EC3 Eq.6.56)
h/b=500/200=2.50>2.00 buckling curve:c
imperfection factor:α,lt=0.49, β=0.75, χ,lt=0.905 (T.6.3, T.6.5, Fig.6.4)
$\Phi,lt=0.5[1+\alpha,lt(\bar\lambda,lt-\bar\lambda,lto)+\beta\bar\lambda,lt^2]=0.5x[1+0.49x(0.567-0.40)+0.75x0.567^2]=0.661$
$\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar\lambda,lt^2)}]=1/[0.661+\sqrt{(0.661^2-0.75x0.661^2)}]=0.905$
Reduction factor $\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar\lambda,lt^2)}]$, χ,lt<=1.0, $1/\bar\lambda,lt^2$, χ,lt=0.905 (Eq.6.57)

$\chi,lt,mod=\chi,lt/f$, χ,lt,mod<=1, $\chi,lt,mod<=1/\bar\lambda,lt^2=1/0.567^2=3.11$ (EC3 §6.3.2.3(2),Eq.6.58)
Kc=1/(1.33-0.33ψ)=0.727, ψ=-0.14 (EC3 Tab.6.6)
$f=1-0.5(1-kc)[1-2.0(\bar\lambda,lt-0.8)^2]=1-0.5x(1-0.727)[1-2.0x(0.567-0.8)^2]=0.878$, f<=1.0
χ,lt,mod=χ,lt/f=0.905/0.878=1.030, χ,lt,mod<=1.0, χ,lt,mod<=3.11, χ,lt,mod=1.000

$Mb,rd=\chi,lt\cdot Wpl,y\cdot fy/\gamma M1=1.000x[10^{-6}]x2194.0x10^3x355/1.00=778.87kNm$ (EC3 Eq.6.55)
My,ed= 171.23 kNm < 778.87 kNm =Mb,rd, Is verified

13.10. Axial force and bending moment, Rafter end-span region (EN1993-1-1, §6.3.3)

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.61)
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.62)
$Nrk=A\cdot fy=[10^{-3}]x11550x355=4100.2$ kN (Tab.6.7)
$My,rk=Wpl,y\cdot fy=[10^{-6}]x2194.0x10^3x355=778.9$ kNm
$\chi y\cdot Nrk/\gamma M1=\chi y\cdot A\cdot fy/\gamma M1= 0.635x[10^{-3}]x11550x355/1.00=2603.7kN$
$\chi z\cdot Nrk/\gamma M1=\chi z\cdot A\cdot fy/\gamma M1= 0.612x[10^{-3}]x11550x355/1.00=2509.4kN$
$\chi LT\cdot My,rk/\gamma M1=\chi LT\cdot Wpl,y\cdot fy/\gamma M1= 1.000x[10^{-6}]x2194.0x10^3x355/1.00=778.9kNm$

Interaction factors, Method of computation: Method 1 Annex A (EC3 AnnexA)

kyy=Cmy·CmLT(μy/(1-Ned/Ncr,y)(1/Cyy), μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y) (EC3 Tab.A.1)
$kzy=Cmy\cdot CmLT(\mu z/(1-Ned/Ncr,y)(1/Czy)0.60\sqrt{(wy/wz)}$, μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)

$Ncr,y=\pi^2EIy/lcr,y^2=3.14^2x[10^{-3}]x210000x482.00x10^6/16217^2=$ 3799 kN
$Ncr,z=\pi^2EIz/lcr,z^2=3.14^2x[10^{-3}]x210000x21.420x10^6/3201^2=$ 4333 kN
$Ncr,t=(1/ip^2)x(G\cdot It+\pi^2EIw/Lcr,t^2)$ (EC3 NCCI SN003b-EN-EU)
$Ncr,t=[10^{-3}]x(1/209^2)[80769x 0.893x10^6+\pi^2x210000x1249.4x10^9/4311^2]=4851$ kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 92.6/ 3799)/(1-0.635x 92.6/ 3799)=0.991
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 92.6/ 4333)/(1-0.612x 92.6/ 4333)=0.992
$alt=1-It/Iy>=0=1- 0.893x10^6/482.00x10^6=0.998$ (EC3 Annex A.1)

$wy=Wpl,y/Wel,y<=1.50, wy= 2.194x10^6/ 1.928x10^6=1.138 <= 1.50$ (EC3 Annex A.1)
$wz=Wpl,z/Wel,z<=1.50, wz= 0.336x10^6/ 0.214x10^6=1.568 > 1.50$, wz=1.50
npl=Ned/(Nrk/γM1)=92.62/(4100.20/1.00)=0.023

$\bar\lambda max=max(1.044,0.977)=1.040$ (EC3 Annex A.1)
Mcr,o=(1.00/2.04)x2425.00=1186.4, C1=1.00
$\bar\lambda o=\sqrt{([10^{-6}]x2194.0x10^3x355/1186.4)}=0.810$
$\bar\lambda o,lim=0.2\sqrt{C1}\,[(1-Ned/Ncr,z)(1-Ned/Ncr,t)]^{0.25}$ (EC3 Annex A.1)
$\bar\lambda o,lim=0.2\sqrt{2.044}\,[(1-92.6/4333)(1-92.6/4851)]^{0.25}=0.283$
$\varepsilon y=(My,ed/Ned)(A/Wel)=([10^3]x171.23/92.62)x(11550.0/1928.0x10^3)=11.07$

```
Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(92.62/3799.0)=0.757, (ψ=-0.14)          (EC3 Annex A, T.A.1)
λ̄o=0.810 > λ̄o,lim=0.283
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
   =0.757+(1-0.757)x(√11.075x0.998)/(1+√11.075x0.998) = 0.944
Cmlt=Cmy^2·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=0.944^2x0.998/√[(1-92.6/4333.0)(1-92.6/4851.0)]=0.908, Cmlt=1.000

Cyy=1+(wy-1)[(2-1.6Cmy^2·λ̄max/wy-1.6Cmy^2·λ̄max^2/wy)npl-blt]>=Wel,y/Wpl,y      (Annex A, T.A.1)
blt=0.5alt·λ̄o^2[My,ed/(χ,lt·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
   =0.5x0.998x0.810^2[0.0/(1.000x684.4)](0.0/76.0) = 0.000
Cyy=1+(1.138-1)[(2-1.6x0.944^2x1.040/1.138-1.6x0.944^2x1.040^2/1.138)x0.023-0.000]=0.998
Cyy>=1928.0x10^3/2194.0x10^3=0.879, Cyy=0.998

Czy=1+(wy-1)[(2-14.0Cmy^2·λ̄max^2/wy^5)npl-dlt]>=0.6√(wy/wz)(Wel,y/Wpl,y)       (Annex A, T.A.1)
dlt=2alt·[λ̄o/(0.1+λ̄z^4)][My,ed/(Cmy·χ,lt·Mpl,y,rd)][Mz,ed/(Cmz·Mpl,z,rd)]=
   =20.998x[0.810/(0.1+0.977^4)][0.0/(0.944x1.000x684.4)][0.0/(0.000x76.0)]=0.000
Czy=1+(1.138-1)[(2-14.0x0.944^2x1.040^2/1.138^5)0.023-0.000]=0.984
Czy>=0.6√(1.138/1.500)(1928.0x10^3/2194.0x10^3)=0.459, Czy=0.984

Cyy=0.998, Czy=0.984                                                            (Annex A, T.A.1)
kyy=0.944x1.000x0.991/(1-92.62/3799.0)x(1/0.998)=0.961
kzy=0.944x1.000x0.992/(1-92.62/3799.0)x(1/0.984)x0.6x√(1.138/1.500)=0.510
```

```
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)=                                     (EC3 Eq.6.61)
92.6/(0.635x4100.2/1.00)+0.961x171.2/(1.000x778.9/1.00)=0.036+0.211=0.247
               0.247< 1.000,  Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)=                                     (EC3 Eq.6.62)
92.6/(0.612x4100.2/1.00)+0.510x171.2/(1.000x778.9/1.00)=0.037+0.112=0.149
               0.149< 1.000,  Is verified
```

13.11. Buckling resistance, Rafter-Uplift conditions (Ultimate Limit State)

```
Maximum design values. Verification for load case: L.C. 210: 1.00Gk+1.50Qw1
 Ned  =    2.4 kN
 Ved  =   13.8 kN
 Myed =   35.5 kNm,  Mzed =0.0 kNm
Rafter length Lr=√[(24000/2)^2+(7000-5000)^2]=12042 mm
Buckling length,In-plane buckling     Lcr,y=16217mm (System length)
Buckling length,Out-of-plane buckling Lcr,z=3201mm (Torsional restrains of rafters)
```

13.12. Flexural Buckling, Rafter-Uplift conditions (Ultimate Limit State) (EN1993-1-1, §6.3.1)

```
Maximum design values. Verification for load case: L.C. 210: 1.00Gk+1.50Qw1
Buckling lengths: Lcr,y=1.347x12042=16217mm, Lcr,z=0.266x12042=3201mm
Non-dimensional slenderness (Cross-section Class: 1 )                          (EC3 §6.3.1.3)
λ̄y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(16217/204.3)x(1/76.06)=1.044
λ̄z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(3201/ 43.1)x(1/76.06)=0.977
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=500/200=2.50>=1.20, tf=16.0mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.635               (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(λ̄y-0.2)+λ̄y^2]=0.5x[1+0.21x(1.044-0.2)+1.044^2]=1.134
χy=1/[Φy+√(Φy^2-λ̄y^2)]=1/[1.134+√(1.134^2-1.044^2)]=0.635 <=1 χy=0.635
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.612
Φz=0.5[1+αz(λ̄z-0.2)+λ̄z^2]=0.5x[1+0.34x(0.977-0.2)+0.977^2]=1.109
χz=1/[Φz+√(Φz^2-λ̄z^2)]=1/[1.109+√(1.109^2-0.977^2)]=0.612 <=1 χz=0.612

Reduction factor χ=1/[Φ+√(Φ^2-λ̄^2)], χ<=1.0, Φ=0.5[1+α(λ̄-0.2)+λ̄^2], χ=0.612     (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.612x[10^-3]x11550x355/1.00=2509.35kN                        (EC3 Eq.6.47)
Nc,ed= 2.44 kN < 2509.35 kN =Nb,rd,  Is verified
```

13.13. Lateral torsional buckling , Rafter-Uplift conditions (EN1993-1-1, §6.3.2)

```
Maximum design values. Verification for load case: L.C. 210: 1.00Gk+1.50Qw1
```

Hogging

$k\cdot L=3201mm$, $zg=-250mm$, $zj=0mm$ (EN1993:2002 T.C.1)

$kz=1.0$, $kw=1.0$, $C1=1.000$, $C2=0.000$, $C3=1.000$ (EN1993:2002 T.C.1)

$Mcr=[10^{-6}]1.000x[\pi^2x2.1x10^5x21.420x10^6/3201^2]$

$x\{\ [1.0x(1249.4x10^9/21.420x10^6)$

$+3201^2x8.1x10^4x0.893x10^6/(\pi^2x2.1x10^5x21.420x10^6)]^{0.5}\ \}=1186.4$ kNm

$\bar{\lambda},lt=\sqrt{}(Wpl,y\cdot fy/Mcr)=\sqrt{}\{[10^{-6}]x2194.0x10^3x355/1186.4\}=0.810$ (EC3 Eq.6.56)

$h/b=500/200=2.50>2.00$ buckling curve:c

imperfection factor:$\alpha,lt=0.49$, $\beta=0.75$, $\chi,lt=0.757$ (T.6.3, T.6.5, Fig.6.4)

$\Phi,lt=0.5[1+\alpha,lt(\bar{\lambda},lt-\bar{\lambda},lto)+\beta\bar{\lambda},lt^2]=0.5x[1+0.49x(0.810-0.40)+0.75x0.810^2]=0.847$

$\chi,lt=1/[\Phi,lt+\sqrt{}(\Phi,lt^2-\beta\bar{\lambda},lt^2)]=1/[0.847+\sqrt{}(0.847^2-0.75x0.847^2)]=0.757$

Reduction factor $\chi,lt=1/[\Phi,lt+\sqrt{}(\Phi,lt^2-\beta\bar{\lambda},lt^2)]$, $\chi,lt<=1.0$, $1/\bar{\lambda},lt^2$, $\chi,lt=0.757$ (Eq.6.57)

$\chi,lt,mod=\chi,lt/f$, $\chi,lt,mod<=1$, $\chi,lt,mod<=1/\bar{\lambda},lt^2=1/0.810^2=1.52$ (EC3 §6.3.2.3(2),Eq.6.58)

$Kc=1/(1.33-0.33\psi)=0.752$, $\psi=0.00$ (EC3 Tab.6.6)

$f=1-0.5(1-kc)[1-2.0(\bar{\lambda},lt-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.810-0.8)^2]=0.876$, $f<=1.0$

$\chi,lt,mod=\chi,lt/f=0.757/0.876=0.864$, $\chi,lt,mod<=1.0$, $\chi,lt,mod<=1.52$, $\chi,lt,mod=0.864$

$Mb,rd=\chi,lt\cdot Wpl,y\cdot fy/\gamma M1=0.864x[10^{-6}]x2194.0x10^3x355/1.00=672.94$kNm (EC3 Eq.6.55)

My,ed= 35.46 kNm < 672.94 kNm =Mb,rd, Is verified

14. Haunch verification (Ultimate Limit State) (EN1993-1-1, §6)

The haunch is fabricated by cutting and welding of an IPE 500 section - S 355

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

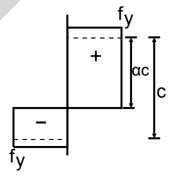
at haunch end	at haunch-middle	at haunch-start
Ned = 92.6 kN	Ned = 91.9 kN	Ned = 89.0 kN
Ved = 96.6 kN	Ved = 84.1 kN	Ved = 75.1 kN
Myed = 348.4 kNm	Myed = 254.5 kNm	Myed = 171.2 kNm

Buckling length,In-plane buckling Lcr,y=2400mm
Buckling length,Out-of-plane buckling Lcr,z=2400mm

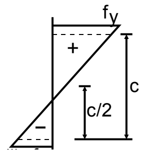
14.1. Classification of cross-sections, at haunch end (EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
$\sigma=[10^{-3}]93/16274 \pm [10^{-6}]348/4640.7\times10^{3} \pm [10^{-6}]0/214.2\times10^{3}$
$\sigma1=81\ N/mm^2$, $\sigma2=-69\ N/mm^2$ (comression positive)

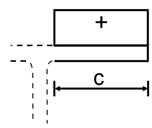
Web
c=1000.0-2x16.0-2x21.0=926.0 mm, t=10.2 mm, c/t=926.0/10.2=90.78
S 355 , t=10.2<= 40 mm, fy=355 N/mm^2, $\varepsilon=(235/355)^{0.5}=0.81$
Position of neutral axis for combined Bending and compression
Ned/(2tw·fy/γM0)=92600/(2x10.2x355/1.00)=12.8 mm
α=(926.0/2+12.8)/926.0=0.514>0.5
c/t=90.78>456x0.81/(13x0.514-1)=65.03
The web is not class 1 or 2



σ=Ned/A±Myed·(0.5d)/Iy, σ1=75 N/mm^2, σ2=-64 N/mm^2
ψ=-64/75=-0.850 > -1
c/t=90.78 > 42x0.81/(0.67+0.33x-0.850)=87.34
The web is not class 3
The web is class 4 (EN1993-1-1, Tab.5.2)



Flange
c=200.0/2-10.2/2-21.0=73.9 mm, t=16.0 mm, c/t=73.9/16.0=4.62
S 355 , t=16.0<= 40 mm, fy=355 N/mm^2, $\varepsilon=(235/355)^{0.5}=0.81$
c/t=4.62<=9ε=9x0.81=7.29
The flange is class 1 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 4, Bending and compression Nc,ed + My,ed

Effective cross-section properties of Class 4 cross-sections (EN1993-1-1, §6.2.2.5)

Web
$\bar\lambda$p=(b/t)/[28.40ε√(Kσ)] (EN1993-1-3, §5.5.2, Eq.5.5, Tabl.5.3)
b=d=926.0mm, t=10.2mm, ε=0.81, ψ=1.00, Kσ=4.00, $\bar\lambda$p=1.973
$\bar\lambda$p=1.973>0.673 ρ=[1-0.055(3+1.00)/1.973]/1.973=0.450 (ρ<1.0), deff=ρ·d=0.450x926=417.0 mm

Effective area Aeff=16274-1x(926.0-417.0)x10.20=11081 mm^2

Web
$\bar\lambda$p=(b/t)/[28.40ε√(Kσ)] (EN1993-1-3, §5.5.2, Eq.5.5, Tabl.5.3)
b=d=926.0mm, t=10.2mm, ε=0.81, ψ=-1.00, Kσ=23.90, $\bar\lambda$p=0.807
$\bar\lambda$p=0.807>0.673 ρ=[1-0.055(3+-1.00)/0.807]/0.807=1.070 (ρ<1.0), heff=ρ·d/2=1.000x463=463.0 mm

Effective area Aeff=16274-1x(463.0-463.0)x10.20=16274 mm^2
emy277.80x(16274/16274-1)=0.00 mm, Iy,eff=2320.3×10^{6} mm^4
Effective section modulus Wy,eff=2320.3×10^{6}/(1000.0/2+0.00)=4640.7×10^{3} mm^3

14.2. Resistance of cross-section, at haunch end (Ultimate Limit State) (EN1993-1-1, §6.2)

Effective cross-section properties of Class 4 cross-sections (EN1993-1-1, §6.2.2.5)

Web

$\bar{\lambda}_p=(b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-3, §5.5.2, Eq.5.5, Tabl.5.3)

b=d=926.0mm, t=10.2mm, ε=0.81, ψ=1.00, $K\sigma$=4.00, $\bar{\lambda}_p$=1.973

$\bar{\lambda}_p$=1.973>0.673 ρ=[1-0.055(3+1.00)/1.973]/1.973=0.450 (ρ<1.0), deff=$\rho\cdot$d=0.450x926=417.0 mm

Effective area Aeff=16274-1x(926.0-417.0)x10.20=11081 mm²

Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)

Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2

Nc.ed= 93.70 kN

Compression Resistance Ncrd= Aeff$\cdot$fy/γM0=[10^{-3}]x11081x355/1.00=3933.90kN

Ned= 93.70 kN < 3933.90 kN =Nc,rd, Is verified

Effective cross-section properties of Class 4 cross-sections (EN1993-1-1, §6.2.2.5)

Web

$\bar{\lambda}_p=(b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-3, §5.5.2, Eq.5.5, Tabl.5.3)

b=d=926.0mm, t=10.2mm, ε=0.81, ψ=-1.00, $K\sigma$=23.90, $\bar{\lambda}_p$=0.807

$\bar{\lambda}_p$=0.807>0.673 ρ=[1-0.055(3+-1.00)/0.807]/0.807=1.070 (ρ<1.0), heff=$\rho\cdot$d/2=1.000x463=463.0 mm

Effective area Aeff=16274-1x(463.0-463.0)x10.20=16274 mm²

emy277.80x(16274/16274-1)=0.00 mm, Iy,eff=2320.3x10^{6} mm^{4}

Effective section modulus Wy,eff=2320.3x10^{6}/(1000.0/2+0.00)=4640.7x10^{3} mm³

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

My.ed=348.40 kNm

Bending Resistance Mcy,rd=Weffy$\cdot$fy/γM0=[10^{-6}]x4640.7x10^{3}x355/1.00= 1647.43kNm

My,ed= 348.40 kNm < 1647.43 kNm =My,rd=Mply,rd, Is verified

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

Vz.ed= 96.60 kNm

Av=A-2b$\cdot$tf+(tw+2r)tf=16274-2x200.0x16.0+(10.2+2x21.0)x16.0=10709mm² (EC3 §6.2.6.3)

Av= 10709mm² > $\eta\cdot$hw$\cdot$tw= 1.00x(1000.0-2x16.0)x10.2=1.00x984.0x10.2= 10037mm²

Plastic Shear Resistance Vpl,z,rd=Av(fy/$\sqrt{3}$)/γM0= [10^{-3}]x10709x(355/1.73)/1.00= 2194.87kN

Vz,ed= 96.60 kN < 2194.87 kN =Vz,rd=Vpl,z,rd, Is verified

hw/tw=(1000.0-2x16.0)/10.2=984.0/10.2=96.47>72x0.81/1.00=72ε/η=58.32 (η=1.00)

S 355 , t=10.2<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81

Shear buckling resistance must be verified (EC3 §6.2.6.6)

Shear buckling resistance (EC3 EN1993-1-5:2006, §5)

$\bar{\lambda}_w$=(926.0/10.2)/(37.4x0.81x$\sqrt{(5.34)}$)=1.297, $K\tau$=5.34 (EC3-1-5 §5, Eq.5.6, A.3)

$\bar{\lambda}_w$=1.297 >= 1.08, χv=0.83/1.297=0.640 (EC3-1-5 Tab.5.1)

Vb,rd=χv$\cdot$fyw$\cdot$hw$\cdot$t/($\sqrt{3}\gamma$M1)=0.001x355x0.640x926.0x10.2/(1.73x1.00)=1239.04kN (EC3-1-5 Tab.5.1)

Ved=97 kN < 1239=Vb,rd kN, Is verified

Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

N.ed= 92.60kN (Compression), Vz.ed= 96.60kNm, My.ed= 348.40kNm

Nplrd=3933.90kN, Mc,y,rd=1647.43kNm, Vpl,z,rd=1239.04kN

Ned=92.60kN <= 0.25x3933.90=0.25xNplrd=983.47kN

Ned=92.60kN <= [10^{-3}]x0.5x984.0x10.2x355/1.00=0.5hw$\cdot$tw$\cdot$fy/γM0=1781.53 kN

n=Ned/Nplrd=93/3934= 0.024

Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

Ved=96.60kN <= 0.50x1239.04=0.50xVpl,rd=619.52kN

Effect of shear force is neglected (EC3 §6.2.8.2)

Maximum and minimum cross-section stresses σ=Ned/Aeff ± Myed/Weff.y ± Mzed/Weff.z
$\sigma=[10^{-3}]0/11081 \pm [10^{-6}]348/4640.7\text{x}10^3 \pm [10^{-6}]0/214.2\text{x}10^3$
σ1=75 N/mm², σ2=-75 N/mm² (comression positive)
σx.ed=75 < 355/1.00=355=fy/γM0 N/mm², Is verified (EC3 Eq.6.43, Eq.6.44)

14.3. Out-of-plane buckling, at haunch end (Ultimate Limit State) (EN1993-1-1, §6.3.2.4)

We check an equivalent T-section for the compresive part of the haunch section
The equivalent T-section is made of the bottom flange and 1/3 of the compressed part of the web

Properties of equivalent T-section

Depth of cross section	hf =	167 mm
Width of cross section	bf =	200 mm
Web thickness	tw =	10.20 mm
Flange thickness	tf =	16.00 mm
Area	Af =	4737 mm^2
Second moment of area	If,z =	$10.667\text{x}10^6$ mm^4
Radius of gyration	if,z =	$\sqrt{(10.667\text{x}10^6/4737)}$ =47.5 mm

Compression in the T-section

Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2
$Ned,f=Ned\cdot Af/A+Med\cdot Af/Wel,y=92.6\text{x}4737/16274+348.4\text{x}4737\text{x}10^3/4640.7\text{x}10^3=382.6kN$

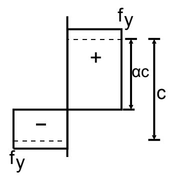
$\bar{\lambda}z=\sqrt{(A\cdot fy/Ncr,z)}=(Lcr,z/iz)\sqrt{(Aeff/A)/\lambda1)}=(2400/\ 47.5)\text{x}(1.000/76.06)=0.665$
f,z-f,z buckling curve:c, imperfection factor:αf,z=0.49, χf,z=0.746 (T.6.2,T.6.1,Fig.6.4)
$\Phi f,z=0.5[1+\alpha f,z(\bar{\lambda}f,z-0.2)+\bar{\lambda}f,z^2]=0.5\text{x}[1+0.49\text{x}(0.665-0.2)+0.665^2]=0.835$
$\chi f,z=1/[\Phi f,z+\sqrt{(\Phi f,z^2-\bar{\lambda}f,z^2)}]=1/[0.835+\sqrt{(0.835^2-0.665^2)}]=0.746 <=1\ \chi f,z=0.746$
Nb,rd=χz·A·fy/γM1= 0.746x16274x355/1.00=1254.45kN (EC3 Eq.6.47)
Nc,ed= 382.57 kN < 1254.45 kN =Nb,rd, Is verified

14.4. Classification of cross-sections, at haunch-middle (EN1993-1-1, §5.5)

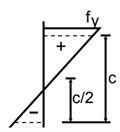
Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
$\sigma=[10^{-3}]92/13724 \pm [10^{-6}]255/3138.1\text{x}10^3 \pm [10^{-6}]0/214.0\text{x}10^3$
σ1=88 N/mm², σ2=-74 N/mm² (comression positive)

Web

c=750.0-2x16.0-2x21.0=676.0 mm, t=10.2 mm, c/t=676.0/10.2=66.27
S 355 , t=10.2<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}$=0.81
Position of neutral axis for combined Bending and compression
Ned/(2tw·fy/γM0)=91900/(2x10.2x355/1.00)=12.7 mm
α=(676.0/2+12.7)/676.0=0.519>0.5
c/t=66.27>456x0.81/(13x0.519-1)=64.30
The web is not class 1 or 2

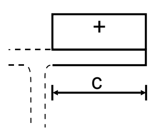


σ=Ned/A±Myed·(0.5d)/Iy, σ1=80 N/mm², σ2=-66 N/mm²
ψ=-66/80=-0.830 > -1
c/t=66.27 <= 42x0.81/(0.67+0.33x-0.830)=85.89
The web is class 3 (EN1993-1-1, Tab.5.2)



Flange

c=200.0/2-10.2/2-21.0=73.9 mm, t=16.0 mm, c/t=73.9/16.0=4.62
S 355 , t=16.0<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}$=0.81
c/t=4.62<=9ε=9x0.81=7.29
The flange is class 1 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 3, Bending and compression Nc,ed + My,ed

14.5. Resistance of cross-section, at haunch-middle (Ultimate Limit State) (EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)

Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2

Nc.ed= 91.90 kN

Compression Resistance Nplrd= A·fy/γM0=[10^{-3}]x13724x355/1.00=4871.88kN

Ned= 91.90 kN < 4871.88 kN =Nc,rd=Nplrd, Is verified

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

My.ed=267.90 kNm

Bending Resistance Mely,rd=Wely·fy/γM0=[10^{-6}]x3138.1x10^3x355/1.00= 1114.01kNm

My,ed= 267.90 kNm < 1114.01 kNm =My,rd=Mply,rd, Is verified

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1

Vz.ed= 85.90 kNm

Av=A-2b·tf+(tw+2r)tf=13724-2x200.0x16.0+(10.2+2x21.0)x16.0=8159mm^2 (EC3 §6.2.6.3)

Av= 8159mm^2 > η·hw·tw= 1.00x(750.0-2x16.0)x10.2=1.00x734.0x10.2= 7487mm^2

Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x8159x(355/1.73)/1.00= 1672.22kN

Vz,ed= 85.90 kN < 1672.22 kN =Vz,rd=Vpl,z,rd, Is verified

hw/tw=(750.0-2x16.0)/10.2=734.0/10.2=71.96>72x0.81/1.00=72ε/η=58.32 (η=1.00)

S 355 , t=10.2<= 40 mm, fy=355 N/mm^2, ε=$(235/355)^{0.5}$=0.81

Shear buckling resistance must be verified (EC3 §6.2.6.6)

Shear buckling resistance (EC3 EN1993-1-5:2006, §5)

$\bar{\lambda}$w=(676.0/10.2)/(37.4x0.81x√(5.34))=0.947, Kτ=5.34 (EC3-1-5 §5, Eq.5.6, A.3)

0.83/η <= $\bar{\lambda}$w=0.947 < 1.08, χv=0.83/0.947=0.877 (η=1.00) (EC3-1-5 Tab.5.1)

Vb,rd=χv·fyw·hw·t/(√3γM1)=0.001x355x0.877x676.0x10.2/(1.73x1.00)=1239.04kN (EC3-1-5 Tab.5.1)

Ved=86 kN < 1239=Vb,rd kN, Is verified

Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)

Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2

N.ed= 91.90kN (Compression), Vz.ed= 84.10kNm, My.ed= 254.50kN

Nplrd=4871.88kN, Mel,y,rd=1114.01kNm, Vpl,z,rd=1239.04kN

Ned=91.90kN <= 0.25x4871.88=0.25xNplrd=1217.97kN

Ned=91.90kN <= [10^{-3}]x0.5x734.0x10.2x355/1.00=0.5hw·tw·fy/γM0=1328.91 kN

n=Ned/Nplrd=92/4872= 0.019

Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

Ved=84.10kN <= 0.50x1239.04=0.50xVpl,rd=619.52kN

Effect of shear force is neglected (EC3 §6.2.8.2)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z

σ=[10^{-3}]0/13724 ± [10^{-6}]255/3138.1x10^3 ± [10^{-6}]0/214.0x10^3

σ1=81 N/mm^2, σ2=-81 N/mm^2 (comression positive)

σx.ed=81 < 355/1.00=355=fy/γM0 N/mm^2, Is verified (EC3 Eq.6.42)

14.6. Out-of-plane buckling, at haunch-middle (Ultimate Limit State) (EN1993-1-1, §6.3.2.4)

We check an equivalent T-section for the compresive part of the haunch section

The equivalent T-section is made of the bottom flange and 1/3 of the compressed part of the web

Properties of equivalent T-section

Depth of cross section	hf =	125 mm
Width of cross section	bf =	200 mm
Web thickness	tw =	10.20 mm
Flange thickness	tf =	16.00 mm
Area	Af =	4312 mm^2
Second moment of area	If,z =	10.667x10^6 mm^4
Radius of gyration	if,z =	√(10.667x10^6/4312) =49.7 mm

Compression in the T-section

Maximum design values. Verification for load case: L.C. 212: 1.35xGk+1.50Qs1+0.90Qw2

$N_{ed,f}=N_{ed}\cdot A_f/A+M_{ed}\cdot A_f/W_{el,y}=91.9\text{x}4312/13724+235.5\text{x}4312\text{x}10^3/3138.1\text{x}10^3=352.4\text{kN}$

$\bar{\lambda}_z=\sqrt{(A\cdot f_y/N_{cr,z})}=(L_{cr,z}/i_z)\cdot(1/\lambda_1)=(2400/\ 49.7)\text{x}(1/76.06)=0.634$

f,z-f,z buckling curve:c, imperfection factor:$\alpha_{f,z}=0.49$, $\chi_{f,z}=0.765$ (T.6.2,T.6.1,Fig.6.4)

$\Phi_{f,z}=0.5[1+\alpha_{f,z}(\bar{\lambda}_{f,z}-0.2)+\bar{\lambda}_{f,z}^2]=0.5\text{x}[1+0.49\text{x}(0.634-0.2)+0.634^2]=0.807$

$\chi_{f,z}=1/[\Phi_{f,z}+\sqrt{(\Phi_{f,z}^2-\bar{\lambda}_{f,z}^2)}]=1/[0.807+\sqrt{(0.807^2-0.634^2)}]=0.765 <=1$ $\chi_{f,z}=0.765$

$N_{b,rd}=\chi_z\cdot A\cdot f_y/\gamma_{M1}=\ 0.765\text{x}13724\text{x}355/1.00=1170.98\text{kN}$ (EC3 Eq.6.47)

$N_{c,ed}=\ 352.40$ kN < 1170.98 kN $=N_{b,rd}$, Is verified

Connections

15. Connection data (EN1993-1-8)

15.1. Bolt connection data (eave, apex) (EN1993-1-8)

```
Type of connection          End-plate connection, non-preloaded bolts
Category of connection      Category A: Bearing type                    (EC3-1-8  §3.4.1)
                            Category D: Non-preloded                    (EC3-1-8  §3.4.2)
End Plate                   Thickness tp=20 mm, S 235
Plate of Apex connecion     200x706x20 mm, S 235
Plate of Eave connecion     200x1140x20 mm, S 235
Bolts                      M24, Strength grade 10.9
 Bolt diameterer           d  = 24 mm
 Diameter of holes         do = 26 mm
 Nominal area              πd²/4 =πx24²/4= 452.4 mm²
 Tensile stress area       As = 353.0 mm²
 Bolt strength grade       10.9, fyb=900N/mm², fub=1000N/mm²            (EC3-1-8  §3.1.1)
```

15.2. Edge distances and spacing of bolts (eave, apex) (EN1993-1-8, §3.5, Tab.3.3)

```
Minimum edge distances                    e1=1.2do=1.2x26=32 mm
                                          e2=1.2do=1.2x26=32 mm
Maximum edge distances                    e1=4t+40=4x19.0+40=117 mm
                                          e2=4t+40=4x19.0+40=117 mm
Minimum spacing of bolts                  p1=2.2do=2.2x26=58 mm
                                          p2=2.4do=2.4x26=63 mm
Maximum spacing of bolts                  p1=min(14t,200)=min(14x19.0,200)=200 mm
                                          p2=min(14t,200)=min(14x19.0,200)=200 mm
Distance of plate edge to bolt line       e1=e2=ex= 50 mm
Distance of section edge to bolt line           ec= 45 mm
Distance of flange enge to bolt line            ef= 45 mm
Pitch between bolt rows                    p1=p3=p= 90 mm
Spacing between cross centers              p2=g =w= 100 mm
Flange to end-plate weld    atf>= 0.55tf=0.55x16.0= 9 mm
Web to end-plate weld        aw>= 0.55tw=0.55x10.2= 6 mm
```

15.3. Design resistance of individual bolts (eave, apex) (EC3-1-8 §3.6.1, Tab.3.4)

```
Bolt strength grade=10.9,  fub =1000N/mm², As=353.0mm², γM2=1.25
Tension resistance of bolts   Ft,rd=k2·fub·As/γM2, (k2=0.90 )
                              Ft,rd=[10^-3]x0.90x1000x353.0/1.25=254 kN
Shear resistance of bolts     Fv,rd=αν·fub·As/γM2, (αν=0.50)
                              Fv,rd=[10^-3]x0.50x1000x353.0/1.25=141 kN
```

16. Apex connection

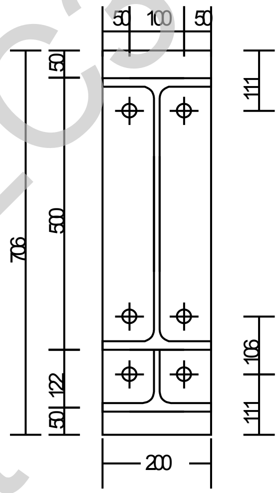
16.1. Basic data (Apex connection)

Design forces of connection ((Apex connection))

Maximum design values for actions (L.C. 202: 1.35Gk+1.50Qs1)

Ned = -71.4 kN
Ved = 24.6 kN
Med = 135.6 kNm

16.2. Connection data (Apex connection)



Bolt connection data

End Plate	200x706x20 mm, S 235
Bolts	M24, Bolt strength grade 10.9
Number of Bolts	top 2x1=2
	bottom 2x2=4
Total number of bolts	=6
Diameter of holes	do = 26 mm
Shear plane of bolt	through the threated portion

Edge distances and spacing of bolts

Distance of plate edge to bolt line	e1=e2=ex= 50 mm
Distance of section edge to bolt line	ec= 45 mm
Distance of flange enge to bolt line	ef= 45 mm
Pitch between bolt rows	p1=p3=p= 90 mm
Spacing between cross centers	p2=g =w= 100 mm
Flange to end-plate weld	atf>= 0.55tf=0.55x16.0= 9 mm
Web to end-plate weld	aw>= 0.55tw=0.55x10.2= 6 mm

16.3. Connection geometry of end-plate (Apex connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=50 mm, emin=50 mm
mx,x=(100-10.2-2x0.8x6x√2)/2= 38.1 mm
mx,y=45-0.8x9x√2= 34.8 mm
nx,x= emin <=1.25mx,x = min(50.0,1.25x38.1=47.6)= 47.6 mm
nx,y= emin <=1.25mx,y = min(50.0,1.25x34.8=43.5)= 43.5 mm
min(mx,x, mx,y)=min(38.1,34.8)=34.8mm, max(mx,x, mx,y)=max(38.1,34.8)=38.1mm
min(nx,x, nx,y)=min(47.6,43.5)=43.5mm, max(nx,x, nx,y)=max(47.6,43.5)=47.6mm

16.4. Effective lengths of end-plate (Apex connection)

(EC3-1-8 §6.2.6.5 Tab.6.6)

Bolt-row outside tension flange of beam

leff=2π·mx =2πx34.8= 218.7 mm
=π·mx+w =πx34.8+100.0= 209.3 mm
=π·mx+2e=πx34.8+2x50.0= 209.3 mm
=4mx+1.25ex = 4x34.8+1.25x50.0=201.7 mm
=e+2mx+0.625ex = 50.0+2x34.8+0.625x50.0=150.8 mm
=0.5bp = 0.5x200= 100.0 mm
=0.5w+2mx+0.625ex=0.5x100.0+2x34.8+0.625x50.0= 150.8 mm
leff,1b=min(218.7, 209.3, 209.3, 201.7, 150.8, 100.0, 150.8)= 100.0 mm
leff,1b= 100.0 mm

Bolt next to tension flange alone

leff=2π·mx =2πx34.8= 218.7 mm
=α·m =6.28x34.8=218.7 mm (λ1=λ2=m/(m+e)=0.41, α=6.28) (EC3-1-8 Fig.6.11)
leff,2b=min(218.7, 218.7)= 218.7 mm
leff,2b= = 218.7 mm

Bolt next to tension flange in a group
leff=2π·mx =2πx34.8= 218.7 mm
=α·m =6.28x34.8=218.7 mm (λ1=λ2=m/(m+e)=0.41, α=6.28)
=πm+p =πx34.8+90.0= 199.3 mm
=0.5p+α·m-(2m+0.625e)=0.5x90.0+6.3x34.8-(2x34.8+0.625x50.0)= 162.8 mm
leff,3b=min(218.7, 218.7, 199.3, 162.8)= 162.8 mm
leff,3b= = 162.8 mm

Inner Bolt-row in a group
leff=2π·mx =2πx38.1= 239.4 mm
=4m+1.25e =4x38.1+1.25x50.0= 214.9 mm
=2p =2x90.0= 180.0 mm
=p = 90.0 mm
leff,4b=min(239.4, 214.9, 180.0, 90.0)= 90.0 mm
leff,4b= = 90.0 mm

16.5. End-Plate, Resistance of T-stub flange (Apex connection) (EC3-1-8 §6.2.4.1, Tab.6.2)

Bolt-row outside tension flange of beam
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x100.0x20.0²x235/1.00=2.350kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x2.350/34.8 = 270 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x2.350+43.5x2x254)/(34.8+43.5)=342kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(270, 342, 508)= 270 kN

Bolt next to tension flange alone
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x218.7x20.0²x235/1.00=5.139kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x5.139/34.8 = 591 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x5.139+43.5x2x254)/(34.8+43.5)=413kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(591, 413, 508)= 413 kN

Bolt next to tension flange in a group
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x162.8x20.0²x235/1.00=3.826kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x3.826/34.8 = 440 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x3.826+43.5x2x254)/(34.8+43.5)=380kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(440, 380, 508)= 380 kN

Inner Bolt-row in a group
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x90.0x20.0²x235/1.00=2.115kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x2.115/38.1 = 222 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x2.115+47.6x2x254)/(38.1+47.6)=332kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(222, 332, 508)= 222 kN

16.6. Rafter flange and web in compression (Apex connection) (EC3-1-8 §6.2.6.7)
Fc,fb,rd= Mc,rd/(h-tf), Mc,rd= Wel,y·fy/γM0
Wel,y=(200x16.0x590.0²+10.2x574.0³/6)/606=2368.7x10^{3} mm³
Mc,rd= [10^{-6}]x2368.7x10^{3}x355/1.00=841kNm, Fc,fb,rd=[10^{3}]x841/590.0= 1425kN
Fc,fb,rd,max=(1/0.8)b·t·fy/γM0= (1/0.8)x[10^{-3}]x200.0x16.0x235/1.00= 940 kN (h>600mm)
Fc,fb,rd= min(1425, 940)= 940 kN

16.7. Rafter web in tension (Apex connection) (EC3-1-8 §6.2.6.8)

Ft,wb,rd = beff,t,wb·twb·fy,wb/γM0
beff,t,wb=Leff=min(leff,3b, leff,4b)=min(162.8, 90.0) =90.0 mm
Ft,wb,rd = [10^{-3}]x90.0x10.2x355/1.00= 326 kN

min Ft,rd=min(270, 413, 380, 222, 326)= 222 kN

16.8. Moment resistance of connection (Apex connection) (EN1993-1-8, §6.2.7.2)

Mj,rd = Σ hr·Ftr,rd (EN1993-1-8, §6.2.7.2Eq.6.25

hr: row numbering from bottom, distances from center of top (compression) flange

End-plate in bending (EC3-1-8 §6.2.4.5

Force distribution in bolt rows

Bolt-row 1, hr= 537.0 mm, Ft,rd= 380 kN

Bolt-row 2, hr= 431.0 mm, Ft,rd= 222 kN

Bolt-row 3, hr= 53.0 mm, Ft,rd= 413 kN

Fc,ed=ΣFt,rd=380 + 222 + 413= 1015 kN

Rafter web in tension (EC3-1-8 §6.2.6.8)

Ft,wb,rd = 326 kN

Rafter flange and web in compression (EC3-1-8 §6.2.4.7

Fc,fb,rd= 940 kN

Ft,rd<=Ft,wb,rd=326 kN, Fc,ed=ΣFt,rd<=Fc,fb,rd=940 kN

Force distribution in bolt rows (EC3-1-8 §6.2.7.2.(7))

Bolt-row 1, hr= 537.0 mm, Ft,rd= 326 kN

Bolt-row 2, hr= 431.0 mm, Ft,rd= 222 kN

Bolt-row 3, hr= 53.0 mm, Ft,rd= 326 kN

Fc,ed=ΣFt,rd=326 + 222 + 326= 874 kN

Moment resistance of connection (EN1993-1-8, §6.2.7.2(10)

Mj,rd =[10^{-3}]x[326x537.0+222x431.0+326x53.0]

Mj,rd = 288 kNm

Med= 135.6 kNm < 288.0 kNm =Mj,rd, Is verified

16.9. Shear resistance (Apex connection) (EN1993-1-8, §3.6.1 Tab.3.4)

Shear resistance of bolts

Fv,rd=αv·fub·As/γM2=[10^{-3}]x0.50x1000x353.0/1.25=141 kN

Shear plane of bolt:through the threated portion

Bearing resistance of bolts

Fb,rd=k1·αb·fu·d·t/γM2

t=20.0mm, d=24mm, do=26mm, e1=50mm, e2=50mm, p1=90mm, fub=1000kN/mm², fu=360kN/mm²,

αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=

=min[1000/360, 1.0, 50/(3x26), 90/(3x26)-0.25]=0.64

k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x50/26-1.7,1.4x100/26-1.7,2.5]=2.50

Fb,rd=k1·αb·fu·d·t/γM2= [10^{-3}]x2.50x0.64x360x24x20.0/1.25=222 kN

Design resistance of one bolt in shear =min(141, 222)=141 kN

Bending moment and shear (EN1993-1-8, §3.6.1 Tab.3.4)

Maximum tension force in bolts

Ft,ed=326/2=163 kN

Reduction of shear resistance due to bending

ρ=1-Ft,ed/1.40Ft,rd=1-163/(1.40x254)=0.54

Shear acting together with bending moment for all the bolts

Vrd=6x0.54x141 =457 kN

Ved= 25 kN < 457 kN =Vrd, Is verified

17. Eave connection

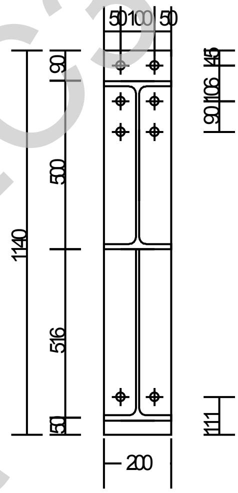
17.1. Basic data (Eave connection)

Design forces of connection ((Eave connection))

Maximum design values for actions (L.C. 202: 1.35Gk+1.50Qs1)

Ned = -75.5 kN
Ved = 110.4 kN
Med = -348.4 kNm

17.2. Connection data (Eave connection)



Bolt connection data

End Plate	200x1140x20 mm, S 235
Bolts	M24, Bolt strength grade 10.9
Number of Bolts	top 2x3=6
	bottom 2x1=2
Total number of bolts	=8
Diameter of holes	do = 26 mm
Shear plane of bolt	through the threated portion

Edge distances and spacing of bolts

Distance of plate edge to bolt line	e1=e2=ex= 50 mm
Distance of section edge to bolt line	ec= 45 mm
Distance of flange enge to bolt line	ef= 45 mm
Pitch between bolt rows	p1=p3=p= 90 mm
Spacing between cross centers	p2=g =w= 100 mm
Flange to end-plate weld	atf>= 0.55tf=0.55x16.0= 9 mm
Web to end-plate weld	aw>= 0.55tw=0.55x10.2= 6 mm

Compression stiffener at the bottom of haunch

Compression stiffener with thickness ts= 20.0 mm

17.3. Connection geometry of end-plate (Eave connection) (EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=50 mm, emin=50 mm
mx,x=(100-10.2-2x0.8x6x√2)/2= 38.1 mm
mx,y=45-0.8x9x√2= 34.8 mm
nx,x= emin <=1.25mx,x = min(50.0,1.25x38.1=47.6)= 47.6 mm
nx,y= emin <=1.25mx,y = min(50.0,1.25x34.8=43.5)= 43.5 mm
min(mx,x, mx,y)=min(38.1,34.8)=34.8mm, max(mx,x, mx,y)=max(38.1,34.8)=38.1mm
min(nx,x, nx,y)=min(47.6,43.5)=43.5mm, max(nx,x, nx,y)=max(47.6,43.5)=47.6mm

17.4. Effective lengths of end-plate (Eave connection) (EC3-1-8 §6.2.6.5 Tab.6.6)

Bolt-row outside tension flange of beam

leff=2π·mx =2πx34.8= 218.7 mm
=π·mx+w =πx34.8+100.0= 209.3 mm
=π·mx+2e=πx34.8+2x50.0= 209.3 mm
=4mx+1.25ex = 4x34.8+1.25x50.0=201.7 mm
=e+2mx+0.625ex = 50.0+2x34.8+0.625x50.0=150.8 mm
=0.5bp = 0.5x200= 100.0 mm
=0.5w+2mx+0.625ex=0.5x100.0+2x34.8+0.625x50.0= 150.8 mm
leff,1b=min(218.7, 209.3, 209.3, 201.7, 150.8, 100.0, 150.8)= 100.0 mm
leff,1b= 100.0 mm

Bolt next to tension flange alone

leff=2π·mx =2πx34.8= 218.7 mm
=α·m =6.28x34.8=218.7 mm (λ1=λ2=m/(m+e)=0.41, α=6.28) (EC3-1-8 Fig.6.11)
leff,2b=min(218.7, 218.7)= 218.7 mm
leff,2b= = 218.7 mm

Bolt next to tension flange in a group

leff=2π·mx =2πx34.8= 218.7 mm
=α·m =6.28x34.8=218.7 mm (λ1=λ2=m/(m+e)=0.41, α=6.28)
=πm+p =πx34.8+90.0= 199.3 mm
=0.5p+α·m-(2m+0.625e)=0.5x90.0+6.3x34.8-(2x34.8+0.625x50.0)= 162.8 mm
leff,3b=min(218.7, 218.7, 199.3, 162.8)= 162.8 mm
leff,3b= = 162.8 mm

Inner Bolt-row in a group

leff=2π·mx =2πx38.1= 239.4 mm
=4m+1.25e =4x38.1+1.25x50.0= 214.9 mm
=2p =2x90.0= 180.0 mm
=p = 90.0 mm
leff,4b=min(239.4, 214.9, 180.0, 90.0)= 90.0 mm
leff,4b= = 90.0 mm

17.5. End-Plate, Resistance of T-stub flange (Eave connection) (EC3-1-8 §6.2.4.1, Tab.6.2)

Bolt-row outside tension flange of beam

Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=[10^{-6}]x0.25x100.0x20.0^{2}x235/1.00=2.350kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x2.350/34.8 = 270 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x2.350+43.5x2x254)/(34.8+43.5)=342kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(270, 342, 508)= 270 kN

Bolt next to tension flange alone

Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=[10^{-6}]x0.25x218.7x20.0^{2}x235/1.00=5.139kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x5.139/34.8 = 591 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x5.139+43.5x2x254)/(34.8+43.5)=413kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(591, 413, 508)= 413 kN

Bolt next to tension flange in a group

Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=[10^{-6}]x0.25x162.8x20.0^{2}x235/1.00=3.826kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x3.826/34.8 = 440 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x3.826+43.5x2x254)/(34.8+43.5)=380kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(440, 380, 508)= 380 kN

Inner Bolt-row in a group

Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=[10^{-6}]x0.25x90.0x20.0^{2}x235/1.00=2.115kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x2.115/38.1 = 222 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x2.115+47.6x2x254)/(38.1+47.6)=332kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(222, 332, 508)= 222 kN

17.6. Rafter flange and web in compression (Eave connection) (EC3-1-8 §6.2.6.7)

Fc,fb,rd= Mc,rd/(h-tf), Mc,rd= Wel,y·fy/γM0
Wel,y=(200x16.0x984.0^2+10.2x968.0^3/6)/1000=4640.4x10^3 mm^3
Mc,rd= [10^{-6}]x4640.4x10^3x355/1.00=1647kNm, Fc,fb,rd=[10^3]x1647/984.0= 1674kN
Fc,fb,rd,max=(1/0.8)b·t·fy/γM0= (1/0.8)x[10^{-3}]x200.0x16.0x235/1.00= 940 kN (h>600mm)
Fc,fb,rd= min(1674, 940)= 940 kN

17.7. Rafter web in tension (Eave connection) (EC3-1-8 §6.2.6.8)

Ft,wb,rd = beff,t,wb·twb·fy,wb/γM0
beff,t,wb=Leff=min(leff,3b, leff,4b)=min(162.8, 90.0) =90.0 mm
Ft,wb,rd = [10^{-3}]x90.0x10.2x355/1.00= 326 kN

min Ft,rd=min(270, 413, 380, 222, 326)= 222 kN

17.8. Connection geometry of column-side (Eave connection) (EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=50 mm, emin=50 mm
mx,x=(100-12.0-2x0.8x24)/2= 24.8 mm
mx,y=45-0.8x9x√2= 34.8 mm
nx,x= emin <=1.25mx,x = min(50.0,1.25x24.8=31.0)= 31.0 mm
nx,y= emin <=1.25mx,y = min(50.0,1.25x34.8=43.5)= 43.5 mm
min(mx,x, mx,y)=min(24.8,34.8)=24.8mm, max(mx,x, mx,y)=max(24.8,34.8)=34.8mm
min(nx,x, nx,y)=min(31.0,43.5)=31.0mm, max(nx,x, nx,y)=max(31.0,43.5)=43.5mm

17.9. Effective lengths of column-side (Eave connection) (EC3-1-8 §6.2.6.4 Tab.6.4)

End Bolt-row in a group
leff=2π·m =2πx24.8= 155.8 mm
=π·m+2e1 =πx24.8+2x50.0= 177.9 mm
=4m+1.25e =4x24.8+1.25x50.0= 161.7 mm
=2m+0.63e+e1 =2x24.8+0.63x50.0+50.0= 130.8 mm
=π·m+p =πx24.8+90.0= 167.9 mm
=2e1+p =2x50.0+90.0= 190.0 mm
=2m+0.63e+0.5p =2x24.8+0.63x50.0+0.5x90.0= 125.8 mm
=e1+0.5p =50.0+0.5x90.0= 95.0 mm
leff,1c=min(155.8, 177.9, 161.7, 130.8, 167.9, 190.0, 125.8, 95.0)= 95.0 mm
leff,1c= = 95.0 mm

Inner Bolt-row in a group
leff=2π·m =2πx24.8= 155.8 mm
=4m+1.25e =4x24.8+1.25x50.0= 161.7 mm
=2p =2x90.0= 180.0 mm
=p = 90.0 mm
leff,2c=min(155.8, 161.7, 180.0, 90.0)= 90.0 mm
leff,2c= = 90.0 mm

17.10. Column-Side, Resistance of T-stub flange (Eave connection) (EC3-1-8 §6.2.4.1, Tab.6.2)

End Bolt-row in a group
Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=[10^{-6}]x0.25x95.0x19.0^{2}x355/1.00=3.044kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x3.044/24.8 = 491 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x3.044+31.0x2x254)/(24.8+31.0)=391kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(491, 391, 508)= 391 kN

Inner Bolt-row in a group
Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=[10^{-6}]x0.25x90.0x19.0^{2}x355/1.00=2.883kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x2.883/24.8 = 465 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x2.883+31.0x2x254)/(24.8+31.0)=386kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(465, 386, 508)= 386 kN

17.11. Column-web in tranverse tension (Eave connection) (EC3-1-8 §6.2.6.3)

Ft,wc,rd = ω·beff,t,wc·twc·fy,c/γM0
β=1, ω=ω1=1/√[1+1.3(beff,c·twc/Avc)^2], beff,c=90.0 mm (EC3-1-8 §6.2.6.2, Tab.6.3)
ω=1/√[1+1.3x(90.0x12.0/8380)^2]]= 0.99
Ft,wc,rd = [10^{-3}]x0.99x90.0x12.0x355/1.00= 380 kN

17.12. Design resistance of compression stiffener (Eave connection) (EC3-1-5 §9.1)

Compression stiffener at the bottom of haunch ts= 20.0 mm

fy=235N/mm^2, bs=(200-12.0-2x24.0)/2=70.0mm, ts=20.0mm, tw=12.0mm, ε=√(235/fy)=0.81
Aeff,s=2x70.0x20.0+(2x15x0.81x12.0+20.0)x12.0=6539 mm^2 (EC3-1-5 §9.1(2)
Leff,s=min(70.0, 14x0.81x20.0)=min(70.0, 226.80)=70.0 mm, (EC3 Tab.5.2)
Ieff,s=$(2x70.0+12.0)^3$x20.0/12= 5853.0x10^3 mm^4
ieff,s=√(5853x10^3/6539) =29.9mm, λ1=π√(E/fy)=93.9ε=76.06

Lcr=0.75x(600-2x19.0)= 421.5 mm (EC3-1-5 §9.4(2)
$\bar{\lambda}$=Lcr/(ieff,s·λ1)=421.5/(29.9x76.06)= 0.19 (EC3 §6.3.1.3(1)
$\bar{\lambda}$<= 0.20, χ=1.00 (EC3 §6.3.1.2.4)
Fc,wc,rd = χ·Aeff,s·fy,c/γM1= 1.000x6539x235/1.00=1537 kN > Fc,fb,rd=940 kN
Compression stiffener, Is verified

17.13. Moment resistance of connection (Eave connection) (EN1993-1-8, §6.2.7.2)

Mj,rd = Σ hr·Ftr,rd (EN1993-1-8, §6.2.7.2Eq.6.25
hr: row numbering from top, distances from center of bottom (compression) flange

End-plate in bending (EC3-1-8 §6.2.4.5
Force distribution in bolt rows
Bolt-row 1, hr=1037.0 mm, Ft,rd= 270 kN
Bolt-row 2, hr= 931.0 mm, Ft,rd= 380 kN
Bolt-row 3, hr= 841.0 mm, Ft,rd= 222 kN
Fc,ed=ΣFt,rd=270 + 380 + 222= 872 kN

End-plate in bending (EC3-1-8 §6.2.4.4
Force distribution in bolt rows
Bolt-row 1, hr=1037.0 mm, Ft,rd= 391 kN
Bolt-row 2, hr= 931.0 mm, Ft,rd= 386 kN
Bolt-row 3, hr= 841.0 mm, Ft,rd= 386 kN
Fc,ed=ΣFt,rd=391 + 386 + 386= 1163 kN

Rafter web in tension (EC3-1-8 §6.2.6.8)
Ft,wb,rd = 326 kN
Rafter flange and web in compression (EC3-1-8 §6.2.4.7
Fc,fb,rd= 940 kN
Ft,rd<=Ft,wb,rd=326 kN, Fc,ed=ΣFt,rd<=Fc,fb,rd=940 kN
Fc,ed=ΣFt,rd<=Fc,wc,rd=1537 kN

Force distribution in bolt rows (EC3-1-8 §6.2.7.2.(7))
Bolt-row 1, hr=1037.0 mm, Ft,rd= 270 kN
Bolt-row 2, hr= 931.0 mm, Ft,rd= 326 kN
Bolt-row 3, hr= 841.0 mm, Ft,rd= 222 kN
Fc,ed=ΣFt,rd=270 + 326 + 222= 818 kN

Moment resistance of connection (EN1993-1-8, §6.2.7.2(10)

Mj,rd =[10^{-3}]x[270x1037.0+326x931.0+222x841.0]
Mj,rd = 770 kNm
Med= 348.4 kNm < 770.2 kNm =Mj,rd, Is verified

17.14. Shear resistance (Eave connection) (EN1993-1-8, §3.6.1 Tab.3.4)

Shear resistance of bolts
Fv,rd=αν·fub·As/γM2=[10^{-3}]x0.50x1000x353.0/1.25=141 kN
Shear plane of bolt:through the threated portion

Bearing resistance of bolts
Fb,rd=k1·αb·fu·d·t/γM2

End-Plate
t=20.0mm, d=24mm, do=26mm, e1=50mm, e2=50mm, p1=90mm, fub=1000kN/mm², fu=360kN/mm²,
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[1000/360, 1.0, 50/(3x26), 90/(3x26)-0.25]=0.64
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x50/26-1.7,1.4x100/26-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= [10^{-3}]x2.50x0.64x360x24x20.0/1.25=222 kN

```
Column-Side
t=19.0mm, d=24mm, do=26mm, e1=50mm, e2=50mm, p1=90mm, fub=1000kN/mm², fu=510kN/mm²,
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
  =min[1000/510, 1.0, 50/(3x26), 90/(3x26)-0.25]=0.64
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x50/26-1.7,1.4x100/26-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= [10^-3]x2.50x0.64x510x24x19.0/1.25=298 kN

Design resistance of one bolt in shear =min(141, 222, 298)=141 kN
```

Bending moment and shear (EN1993-1-8, §3.6.1 Tab.3.4)

```
Maximum tension force in bolts
 Ft,ed=326/2=163 kN
Reduction of shear resistance due to bending
 ρ=1-Ft,ed/1.40Ft,rd=1-163/(1.40x254)=0.54
Shear acting together with bending moment for all the bolts
 Vrd=8x0.54x141 =609 kN

Ved= 110 kN < 609 kN =Vrd,  Is verified
```

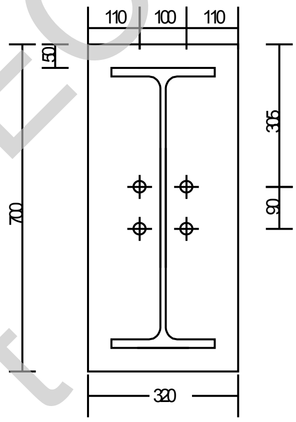
18. Column base Connection

18.1. Basic data (Base connection)

Design forces of connection (Base connection)

Axial force (compression)	Ned=-119 kN, L.C. 202: 1.35Gk+1.50Qs1
Axial force (tension)	Ned= 16 kN, L.C. 111: 0.90Gk+1.50Qw1
Shear force	Ved= 75 kN, L.C. 202: 1.35Gk+1.50Qs1
Moment	Med= 0 kNm,

Connection data (Base connection)



Base plate steel grade	700x320x30 mm, S 235
Anchor bolts	M24, Grade 5.6
Shear plane of bolt	through the threated portion
	middle 2x2=4
Total number of bolts	=4
Diameter of holes	do = 26 mm
Steel section for column	IPE 600, S 355
Spacing between cross centers	100 mm
Flange to end-plate weld	11 mm
Web to end-plate weld	7 mm

Edge distances and spacing of bolts

Distance of plate edge to bolt line	e1=e2=ex= 110 mm
Distance of section edge to bolt line	ec= 44 mm
Distance of flange enge to bolt line	ef= 45 mm
Pitch between bolt rows	p1=p3=p= 90 mm
Spacing between cross centers	p2=g =w= 100 mm
Flange to end-plate weld	atf>= 0.55tf=0.55x19.0= 11 mm
Web to end-plate weld	aw>= 0.55tw=0.55x12.0= 7 mm

Concrete of foundament

Concrete-Steel class	C25/30-B500C	(EC2 §3.1, §3.2)
Partial factors for materials	γc=1.50, γs=1.15	(EC2 §2.4.2.4)
Design compressive strength	fcd=αcc·fck/γc=1.00x25/1.50=16.67 N/mm²	(EC2 §3.1.6)
Design tensile strength	fctd=αct·fctk05/γc=1.00x2/1.50=1.20 N/mm²	
Bearing strength	fjd=β·√Ac1/Aco·fcd=(2/3)x1.5x16.67=16.67N/mm²	(EC2 §6.7)

18.2. Design resistance of individual bolts (Base connection) (EC3-1-8 §3.6.1, Tab.3.4)

Bolt strength grade=5.6, fub =500N/mm², As=353.0mm², γM2=1.25

Tension resistance of bolts	Ft,rd=k2·fub·As/γM2, (k2=0.90)
	Ft,rd=[10^{-3}]x0.90x500x353.0/1.25=127 kN
Shear resistance of bolts	Fv,rd=αν·fub·As/γM2, (αν=0.60)
	Fv,rd=[10^{-3}]x0.60x500x353.0/1.25=85 kN

18.3. Connection geometry of end-plate (Base connection) (EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=110 mm, emin=110 mm
mx,x=(100-12.0-2x0.8x7x√2)/2= 36.1 mm
mx,y= 36.1 mm
nx,x= emin <=1.25mx,x = min(110.0,1.25x36.1=45.1)= 45.1 mm
nx,y= emin <=1.25mx,y = min(110.0,1.25x36.1=45.1)= 45.1 mm
min(mx,x, mx,y)=min(36.1,36.1)=36.1mm, max(mx,x, mx,y)=max(36.1,36.1)=36.1mm
min(nx,x, nx,y)=min(45.1,45.1)=45.1mm, max(nx,x, nx,y)=max(45.1,45.1)=45.1mm

18.4. Effective lengths of end-plate (Base connection) (EC3-1-8 §6.2.6.5 Tab.6.6)

Inner Bolt-row in a group

leff=2π·mx =2πx36.1= 226.8 mm
=4m+1.25e =4x36.1+1.25x110.0= 281.9 mm
=2p =2x90.0= 180.0 mm
=p = 90.0 mm
leff,4b=min(226.8, 281.9, 180.0, 90.0)= 90.0 mm
leff,4b= = 90.0 mm

18.5. End-Plate, Resistance of T-stub flange (Base connection) (EC3-1-8 §6.2.4.1, Tab.6.2)

Inner Bolt-row in a group

Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x90.0x30.0²x235/1.00=4.759kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x4.759/36.1 = 527 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x4.759+45.1x2x127)/(36.1+45.1)=258kN
Mode 3 Ft,3,rd= ΣFt,rd =2x127 = 254 kN
Ft,rd= min(527, 258, 254)= 254 kN

18.6. Column web in tension (Base connection) (EC3-1-8 §6.2.6.8)

Ft,wb,rd = beff,t,wb·twb·fy,wb/γM0
beff,t,wb=Leff= leff,4b = =90.0 mm
Ft,wb,rd = [10^{-3}]x90.0x12.0x355/1.00= 383 kN

min Ft,rd=min(254, 383)= 254 kN

18.7. Tension resistance of connection (EN1993-1-8, §6.2.4

Uplift force of connection Ft,ed= 16 kN
Tension resistance of connection Ft,rd= 2x254 = 508 kN
Ned= 16 kN < 508 kN =Nrd, Is verified

18.8. Shear resistance (Base connection) (EN1993-1-8, §3.6.1 Tab.3.4)

Shear resistance of bolts

Fv,rd=αv·fub·As/γM2=[10^{-3}]x0.60x500x353.0/1.25=85 kN
Shear plane of bolt:through the threated portion

Bearing resistance of bolts

Fb,rd=k1·αb·fu·d·t/γM2

t=30.0mm, d=24mm, do=26mm, e1=110mm, e2=110mm, p1=90mm, fub=500kN/mm², fu=360kN/mm²,
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[500/360, 1.0, 110/(3x26), 90/(3x26)-0.25]=0.90
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x110/26-1.7,1.4x100/26-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= [10^{-3}]x2.50x0.90x360x24x30.0/1.25=469 kN

Design resistance of one bolt in shear =min(85, 469)=85 kN

Tension and shear (EN1993-1-8, §3.6.1 Tab.3.4)

Maximum tension force in bolts
Ft,ed=254/2=127 kN
Reduction of shear resistance due to tension
ρ=1-Ft,ed/1.40Ft,rd=1-127/(1.40x127)=0.29
Shear acting together with tension for all the bolts
Vrd=4x0.29x85 =99 kN

Ved= 75 kN < 99 kN =Vrd, Is verified

18.9. Bearing resistance (Base connection) (EN1993-1-8, §6.2.5)

Compression resistance of T-stub flange $F_{c,rd} = f_{jd} \cdot b_{eff} \cdot l_{eff}$ (§6.2.5(3)Eq. 6.4)

$f_{jd} = \beta \cdot \sqrt{(A_{c1}/A_{co})} \cdot f_{cd} = (2/3) \times 1.5 \times 16.67 = 16.67$ N/mm² (EC3-1-8 §6.2.5(7))

h=600.0mm, b=220.0mm, tf=19.0mm, tw=12.0mm, tp=30.0mm

$c = t_p \cdot (f_y/(3 f_{jd} \cdot \gamma_{M0}))^{0.5} = 30 \times (235.00/(3 \times 16.67 \times 1.00))^{0.5} = 65.0$, <50.0, c=50.0mm (Eq. 6.5)

$2c + b_f = 2 \times 50.0 + 220 = 320.0$mm <= bp=320mm, leff=320.0mm

$A_{co,f} = l_{eff} \cdot (2c + t_f) = 320.0 \times (2 \times 50.0 + 19.0) = 38080$ mm² (EC3-1-8 ,Fig.6.4)

$A_{co,w} = (h - 2t_f - 2c) \cdot (t_w + 2c) = (600.0 - 2 \times 19.0 - 2 \times 50.0) \times (12.0 + 2 \times 50.0) = 51744$ mm²

$N_{j,rd} = [10^{-3}] \times 16.7 \times (2 \times 38080 + 51744) = [10^{-3}] \times 16.7 \times 127904 = 2136$kN

$N_{j,ed} = 119$ kN < 2136 kN $= N_{j,rd}$, Is verified

18.10. Anchoring resistance (Base connection) (EN1993-1-8, §6.2.6.12)

Anchoring hook (EN1992-1-1 §8.4)

Number of fasteners: 4, of diameter 24mm, As= 353mm²

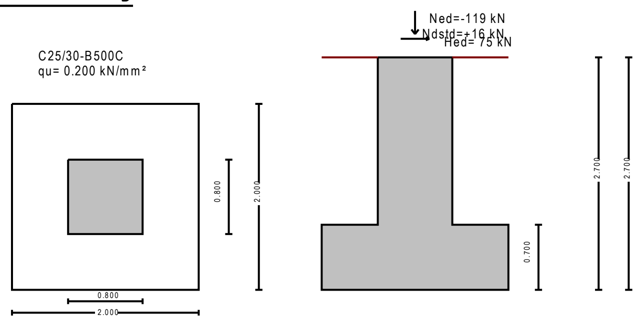
Basic required anchorage length lb,rqd, Design anchorage length lbd=0.70lb,rqd

$l_{b,rqd} = (\varnothing/4)(\sigma_{sd}/f_{bd}) = (24/4) \times (11.5/1.20) = 57$ mm

$\sigma_{sd} = [10^{3}] \times 16/(4 \times 353) = 11.5$ N/mm², $f_{bd} = f_{ctd} = 1.20$ N/mm²

Design anchorage length lbd=0.70x57 >(10x24,100) lbd=250 mm

19. Concrete footing



19.1. Design loads on concrete footing

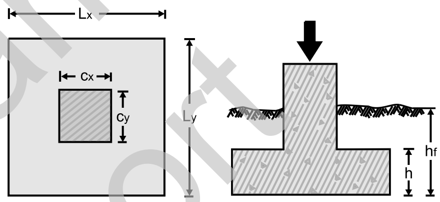
```
Axial force (downwards)   Ned= 119 kN, L.C. 202: 1.35Gk+1.50Qs1
Axial force (upwards)     Ned=  16 kN, L.C. 111: 0.90Gk+1.50Qw1
Shear force               Hed=  75 kN, L.C. 202: 1.35Gk+1.50Qs1
Moment                    Med=   0 kNm,
```

19.2. Dimensions, materials, loads (Concrete footing)

Dimensions

```
Footing         Lx= 2.000 m  Ly= 2.000 m
Column          cx= 0.800 m  cy= 0.800 m
Height of footing             h= 0.700 m
Depth of footing             hf= 2.700 m

Base area of footing         Af=  4.00 m²
Volume of footing            Vf=  4.08 m³
```



Materials of footing

Concrete-Steel class: C25/30-B500C (EN1992-1-1, §3)

Concrete cover: Cnom=35 mm (EC2 §4.4.1)

Effective depth of cross section d=h-d1, d1=Cnom+∅(3/2)=35+3x16/2=59mm, d=700-59=641mm

Concrete weight: 25.0 kN/m³

γc=1.50, γs=1.15 (EC2 Table 2.1N)

fcd=αcc·fck/γc=1.00x25/1.50=16.67 MPa (EC2 §3.1.6)

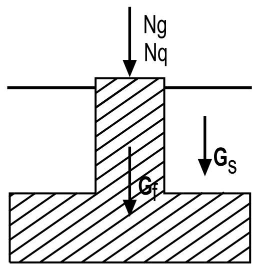
fyd=fyk/γs=500/1.15=435 MPa (EC2 §3.2.7)

Soil

```
Soil bearing pressure  qu= 0.200 N/mm²(MPa)
Unit weight of soil      γ=18.000 kN/m³
```

Loads

```
Self weight of footing          (1.28+2.80)x25.00  Gf= 102.00 kN
Soil weight on footing      (4.00x2.70-4.08)x18.00  Gs= 120.96 kN
```
Design Loads
```
Vertical load downwards        Ned= 118.65 kN
Vertical load upwards     Ndst,d=  16.22 kN
Horizontal load                Hed=  75.48 kN
Moment                         Med=   0.00 kNm
```



Eurocode parameters

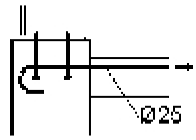
Check of soil bearing capacity (EC7 EN1997-1-1:2004, §6)
Partial factors for actions and soil properties (EC7 Tab. A.1-A.4, EC8-5 §3.1)
Equilibrium limit state (EQU), Structural limit state (STR), Geotechnical limit state (GEO)

			(EQU)	(STR)	(GEO)
Actions	Permanent Unfavourable	γGdst:	1.10	1.35	1.00
	Permanent Favourable	γGstb:	0.90	1.00	1.00
	Variable Unfavourable	γQdst:	1.50	1.50	1.30
	Variable Favourable	γQstb:	0.00	0.00	0.00
Soil parameters	Angle of shearing resistance	γφ:	1.25	1.00	1.25
	Effective cohesion	γc:	1.25	1.00	1.25
	Undrained shear strength	γcu:	1.40	1.00	1.40
	Unconfined strength	γqu:	1.40	1.00	1.40
	Weight density	γw:	1.00	1.00	1.00

Partial safety factors for actions : γG=1.35, γQ=1.50 (EC0 Annex A1)
Combination of accidental actions :(EC7) ψ2 =0.30
Combination of accidental actions :(EC2) ψ2 =0.30

Design of reinforced concrete (EC2 EN1992-1-1:2004)

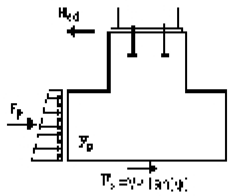
19.3. Tie for horizontal forces (Concrete footing)



The horizontal force acting outwards, is resisted by a steel tie
cast into the floor slab connected to the base of the columns
Maximum horizontal force force acting outwards Hed= 75 kN
Steel B500C, (fyk=500N/mm²)
Required steel area As=Hed/(fy/γs)=10^3x75/(500/1.15)=174mm²
Provide 2 steel bars Ø25 mm, (As=982mm²)

Necessary anchor length (bars with standard hook) (EN1992-1-1 §8.4)
Basic required anchorage lengthlb,rqd=(Ø/4)(σsd/fbd)=(25/4)x(77/2.70)=178 mm
σsd=10^3x75/(2x491)=77 N/mm², fbd=2.25xfctd=2.25x1.20= 2.70 N/mm²
Design anchorage length lbd=0.70x178, >(10x25,100) =260 mm (EC2 §8.4.4)
Design lap length lo=1.5x178, >(15x25,200) =380 mm (EC2 §8.7.3)

19.4. Passive earth pressure on the side of the foundament (EC7 EN1997-1-1:2004, §9.5)



Angle of shearing resistance of ground	φ/γM=30.00/1.25=24.00°
Unit weight of soil	γ =18.00 kN/m³
Foundament depth	hf =2.700 m
Foundament height	h =0.700 m
Foundament width	By =2.000 m
Coefficient of passive earth pressure	Kp =2.371
Earth pressure at the top	p1 =18.00x2.000x2.371=85.36kN/m²
Earth pressure at the bottom	p2 =18.00x2.700x2.371=115.24kN/m²
Earth force	Fprd =0.5x(85.36+115.24)x2.000x0.700=140.42 kN
Point of application of earth force	yp=0.433 m

19.5. Check stability for forces upwards (Concrete footing)

Loading (EQU), 0.90xPermanent + 1.50xVariable (EC7 §2.4.7.2)
Vertical forces upwards Ndst,d = 16 kN
Vertical forces downwards G =102.00+120.96= 222.96 kN
Holding down forces Nstb,d = γGxG= 0.90x222.96= 201 kN

Ndst,d=16 kN < 201 kN =Nstb,d, Is verified

19.6. Check of soil bearing capacity (Concrete footing) (EC7 EN1997-1-1:2004, §6)

Loading (STR,GEO), 1.35xPermanent + 1.50xVariable (EC7 §2.4.7.3)

Design Loads

Vertical load at foundament bottom Ned= 118.65+1.35x(102.00+120.96) = 419.65kN

Vertical load at foundament top Ned1= 118.65+1.35x32.00 = 161.85kN

Soil pressure q=10^{-3}x419.65/(2.000x2.000)=0.105 N/mm^2(Mpa)

Check bearing resistance failure Rd>=Vd (EC7 EN1997-1-1:2004, §6.5.2)

Design effective foundation area A'=2.000x2.000=4.00 m^2 (EC7 Annex D)

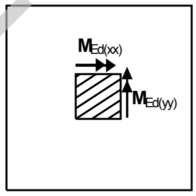
Design bearing resistance of footing Rd=A·qu/γqu, qu=0.20N/mm^2, γqu(EQU,GEO)=1.40

Rd=1000x4.00x0.200/1.40=571.43 kN > Vd=419.65 kN

Ned=419.65kN < 571.43kN =Nrd, Is verified

19.7. Design for bending (Concrete footing) (EC2 EN1992-1-1:2004, §6.1)

Bending at bottom surface



Med(yy)=0.125x162x2.000x(1-0.800/2.000)^2 = 14.57kNm

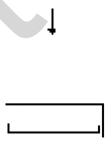
Med(xx)=0.125x162x2.000x(1-0.800/2.000)^2 = 14.57kNm

Med=14.57kNm, b=2000mm, d=641mm, Kd=23.75, x/d=0.01

εc/εs=0.2/20.0, Ks=2.31, As= 0.52cm^2

Minimum reinforcement s<=400mm (O16/30.0, As= 6.70cm^2/m) (EC2 §9.3.1)

Minimum reinforcement ∅16/30.0 (6.70cm^2/m)



Med=14.57kNm, b=2000mm, d=641mm, Kd=23.75, x/d=0.01

εc/εs=0.2/20.0, Ks=2.31, As= 0.52cm^2

Minimum reinforcement s<=400mm (O16/30.0, As= 6.70cm^2/m)

Minimum reinforcement ∅16/30.0 (6.70cm^2/m)

Reinforcement of footing at bottom surface

Reinforcement in x-x direction: ∅16/30.0 (6.70cm^2/m), 8∅16 (16.08cm^2)

Reinforcement in y-y direction: ∅16/30.0 (6.70cm^2/m), 8∅16 (16.08cm^2)

Bending at top surface

Med(yy)=0.125x16x2.000x(1-0.800/2.000)^2 = 1.46kNm

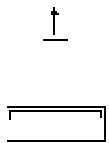
Med(xx)=0.125x16x2.000x(1-0.800/2.000)^2 = 1.46kNm

Med=1.46kNm, b=2000mm, d=641mm, Kd=75.03, x/d=0.00

εc/εs=0.1/20.0, Ks=2.30, As= 0.05cm^2

Minimum reinforcement s<=400mm (O16/30.0, As= 6.70cm^2/m) (EC2 §9.3.1)

Minimum reinforcement ∅16/30.0 (6.70cm^2/m)



Med=1.46kNm, b=2000mm, d=641mm, Kd=75.03, x/d=0.00

εc/εs=0.1/20.0, Ks=2.30, As= 0.05cm^2

Minimum reinforcement s<=400mm (O16/30.0, As= 6.70cm^2/m)

Minimum reinforcement ∅16/30.0 (6.70cm^2/m)

Reinforcement of footing at top surface

Reinforcement in x-x direction: ∅16/30.0 (6.70cm^2/m), 8∅16 (16.08cm^2)

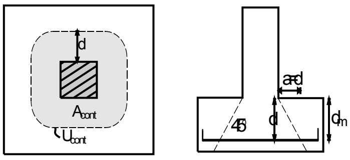
Reinforcement in y-y direction: ∅16/30.0 (6.70cm^2/m), 8∅16 (16.08cm^2)

19.8. Design for shear (Concrete footing) (EC2 EN1992-1-1:2004, §6.2)

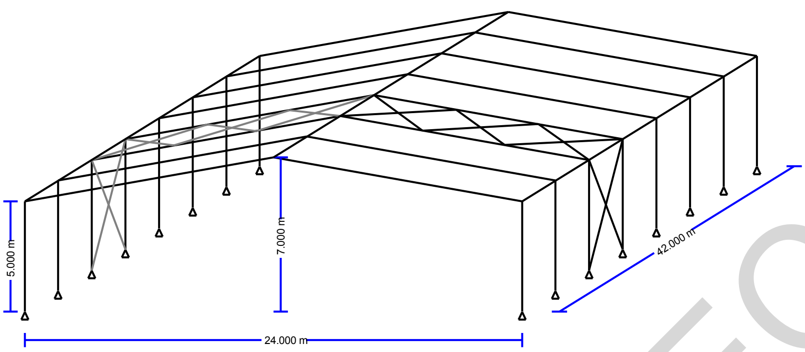
The design for shear is covered by the design in punching shear,
because the critical rupture surface is considered at angle 45°

19.9. Design for punching shear (Concrete footing) (EC2 EN1992-1-1:2004, §6.4)

```
Footing cantilevers in x-x, L1=0.600<d=0.641m, L2=0.600<d=0.641m
Footing cantilevers in y-y, L1=0.600<d=0.641m, L2=0.600<d=0.641m
```



the width of footing cantilevers is < footing height d.
he critical rupture surface at angle 45°,
is outside the area of the footing.
The check for punching shear is satisfied



20. Transverse restraint system

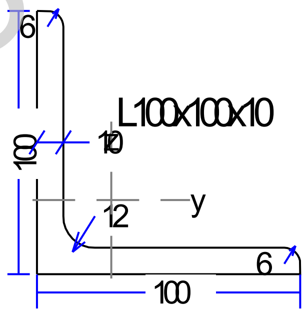
Number of frames in transverse direction 8, at 6.000m appart. Totaly 1 bracing systems are provided. Each bracing system carries 1/1=1.00 of the total horizontal load.
The roof bracing system consists of wind girders diagonally between 24/6=4.000m points.
The vertical bracing system at the columns, consists of 2 diagonal girders from the top to the bottom of the next column, acting as strut/tie.

20.1. Cross-section properties, Bracing member

Cross-section L100x100x10-S 355

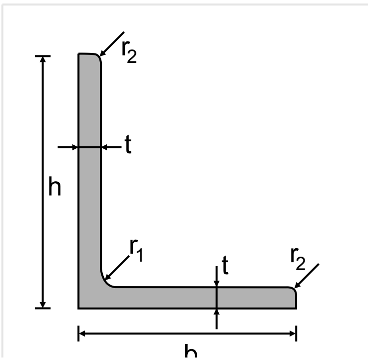
Dimensions of cross section

Depth of cross section	h=	100.00 mm
Width of cross section	b=	100.00 mm
Web depth	hw=	100.00 mm
Depth of straight portion of web	dw=	100.00 mm
Web thickness	tw=	10.00 mm
Flange thickness	tf=	10.00 mm
Radius of root fillet	r=	12.00 mm
Mass	=	15.00 Kg/m



Properties of cross section

Area	A=	1915 mm^2		
Second moment of area	Iy=	$1.767x10^6$ mm^4	Iz=	$1.767x10^6$ mm^4
Second moment of area	Iu=	$2.807x10^6$ mm^4	Iv=	$0.726x10^6$ mm^4
Section modulus	Wy=	$24.610x10^3$ mm^3	Wz=	$24.610x10^3$ mm^3
Plastic section modulus	Wpy=	$106.00x10^3$ mm^3	Wpz=	$52.000x10^3$ mm^3
Radius of gyration	iy=	30.4 mm	iz=	30.4 mm
Radius of gyration	iu=	38.3 mm	iv=	19.5 mm
Shear area	Avz=	1025 mm^2	Avy=	1000 mm^2
Torsional constant	It=	$0.089x10^6$ mm^4	ip=	43 mm
Warping constant	Iw=	$1.688x10^9$ mm^6		



20.2. Horizontal loadings

Wind load on wall surface ,	wk=0.90kN/m², Cpe,D=0.8, Cpe,E=-0.5	(EN1991-1-4,Tab.7.1)
Transverse wind load/m on roof level	qed,w= 1.50x0.90x(0.8+0.5)x7.000/2= 6.14kN/m	
Load/m on bracing system, roof level	qed= (1/1)x6.14 =6.14 kN/m	
Load on bracing system, roof level	Qed1= 6.14x4.000 =24.6 kN	
Load on bracing system, at top of column	Qed2= 6.14x24.000/2= 73.7 kN	

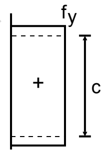
Horizontal (roof) braced girder

The braced girder at roof is loaded with point horizontal loads Qed1= 6.1x4.000= 24.6kN
at the nodes of the bracing system (at spacing L/6=4.000 m).
The braced girder is supported horizontally at the columns
Length of braced girder members 7.211 m, inclination φ=56.31°, tanφ=6.000/4.000=1.500
Forces in members of braced girder
Compression Nced1= 1.00x24.6/sin56.31= 29.6 kN
Tension Nted1= 0.50x24.6/sin56.31= 14.8 kN

Vertical (wall) braced girder

The vertical brace system is loaded with point horizontal load Qed2= 73.7kN
at the top of the column h= 5.000m.
Length of braced girder members 7.810 m, inclination φ=39.81°, tanφ=5.000/6.000=0.833
Forces in bracing members
Tension Nted2= 1.00x73.7/cos39.81= 95.9 kN

20.3. Classification of cross-sections, Compression Nc (Bracing member) (EN1993-1-1, §5.5)



h/t=100.0/10.0=10.00, (b+h)/2t=(100.0+100.0)/(2x10.0)=10.00
S 355 , t=10.0<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}$=0.81
h/t=10.00<=15ε=12.15, (b+h)/2t=10.00>11.5ε=9.32

Overall classification of cross-section is Class 4, Compression Nc,ed

Effective cross-section properties of Class 4 cross-sections (EN1993-1-1, §6.2.2.5)

Web
$\bar{\lambda}p=(b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-3, §5.5.2, Eq.5.5, Tabl.5.3)
b=d=100.0mm, t=10.0mm, ε=0.81, ψ=1.00, Kσ=4.00, $\bar{\lambda}$p=0.217
$\bar{\lambda}$p=0.217<=0.673, ρ=1.0, deff=$\rho\cdot$d=1.000x100=100.0 mm

Flange
$\bar{\lambda}p=(b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-3, §5.5.2, Eq.5.5, Tabl.5.3)
b=c=100.0mm, t=10.0mm, ε=0.81, ψ=1.00, Kσ=0.43, $\bar{\lambda}$p=0.663
$\bar{\lambda}$p=0.663<=0.673, ρ=1.0, ceff=$\rho\cdot$c=1.000x100=100.0 mm

Effective area Aeff=1915-1x(100.0-100.0)x10.00-1x(100.0-100.0)x10.00=1915 mm²

Effective area Aeff=0 mm²
emy0.00 mm, Iy,eff= 0.000×10^{6} mm^4
Effective section modulus Wy,eff= 0.000×10^{6}/(100.0/2+0.00)= 0.000×10^{3} mm³

20.4. Resistance of cross-section, Bracing member (EN1993-1-1, §6.2)

Ultimate Limit State, Verification for tension (EN1993-1-1, §6.2.3)

Nt.ed= 95.94 kN
Tension Resistance Nplrd= A$\cdot$fy/γM0=[10^{-3}]x1915x355/1.00=679.82kN
Nt,ed= 95.94 kN < 679.82 kN =Nt,rd=Nplrd, Is verified

Effective cross-section properties of Class 4 cross-sections (EN1993-1-1, §6.2.2.5)

Effective area Aeff=1915 mm²

Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)

Nc.ed= 29.60 kN
Compression Resistance Ncrd= Aeff$\cdot$fy/γM0=[10^{-3}]x1915x355/1.00=679.82kN
Ned= 29.60 kN < 679.82 kN =Nc,rd, Is verified

20.5. Flexural Buckling, Bracing member (Ultimate Limit State) (EN1993-1-1, §6.3.1)

Buckling lengths: $L_{cr,y}=1.000\text{x}7211=7211$mm, $L_{cr,z}=1.000\text{x}7211=7211$mm
Non-dimensional slenderness (Cross-section Class: 4) (EC3 §6.3.1.3)
$\bar{\lambda}_y=\sqrt{(A\cdot f_y/N_{cr,y})}=(L_{cr,y}/i_y)\sqrt{(A_{eff}/A)}/\lambda_1)=(7211/\ 30.4)\text{x}(1.000/76.06)=3.121$
$\bar{\lambda}_z=\sqrt{(A\cdot f_y/N_{cr,z})}=(L_{cr,z}/i_z)\sqrt{(A_{eff}/A)}/\lambda_1)=(7211/\ 30.4)\text{x}(1.000/76.06)=3.121$
$\bar{\lambda}_v=\sqrt{(A\cdot f_y/N_{cr,v})}=(L_{cr,v}/i_v)\sqrt{(A_{eff}/A)}/\lambda_1)=(7211/\ 19.5)\text{x}(1.000/76.06)=4.868$
$\lambda_1=\pi\sqrt{(E/f_y)}=93.9\varepsilon=76.06$, $\varepsilon=\sqrt{(235/f_y)}=0.81$, $\sqrt{(A_{eff}/A)}=\sqrt{(1915/1915)}=1.000$
$\bar{\lambda}_{eff,y}=0.50+0.7\text{x}3.121=2.685$, $\bar{\lambda}_{eff,z}=0.50+0.7\text{x}3.121=2.685$ (EC3 Annex BB.1.2)
$\bar{\lambda}_{eff,v}=0.35+0.7\text{x}4.868=3.758$

y-y buckling curve:b, imperfection factor:$\alpha_y=0.34$, $\chi_y=0.122$ (T.6.2,T.6.1,Fig.6.4)
$\Phi_y=0.5[1+\alpha_y(\bar{\lambda}_y-0.2)+\bar{\lambda}_y^2]=0.5\text{x}[1+0.34\text{x}(2.685-0.2)+2.685^2]=4.526$
$\chi_y=1/[\Phi_y+\sqrt{(\Phi_y^2-\bar{\lambda}_y^2)}]=1/[4.526+\sqrt{(4.526^2-2.685^2)}]=0.122 <=1$ $\chi_y=0.122$
z-z buckling curve:b, imperfection factor:$\alpha_z=0.34$, $\chi_z=0.122$
$\Phi_z=0.5[1+\alpha_z(\bar{\lambda}_z-0.2)+\bar{\lambda}_z^2]=0.5\text{x}[1+0.34\text{x}(2.685-0.2)+2.685^2]=4.526$
$\chi_z=1/[\Phi_z+\sqrt{(\Phi_z^2-\bar{\lambda}_z^2)}]=1/[4.526+\sqrt{(4.526^2-2.685^2)}]=0.122 <=1$ $\chi_z=0.122$

v-v buckling curve:b, imperfection factor:$\alpha_v=0.34$, $\chi_v=0.065$
$\Phi_v=0.5[1+\alpha_v(\bar{\lambda}_v-0.2)+\bar{\lambda}_v^2]=0.5\text{x}[1+0.34\text{x}(3.758-0.2)+3.758^2]=8.165$
$\chi_v=1/[\Phi_v+\sqrt{(\Phi_v^2-\bar{\lambda}_v^2)}]=1/[8.165+\sqrt{(8.165^2-3.758^2)}]=0.065 <=1$ $\chi_v=0.065$
Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi<=1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.065$ (EC3 Eq.6.49)
$N_{b,rd}=\chi\cdot A_{eff}\cdot f_y/\gamma_{M1}=0.065\text{x}[10^{-3}]\text{x}1915\text{x}355/1.00=44.19$kN (EC3 Eq.6.48)
$N_{c,ed}=29.60$ kN < 44.19 kN $=N_{b,rd}$, Is verified

20.6. Bolts connecting braces

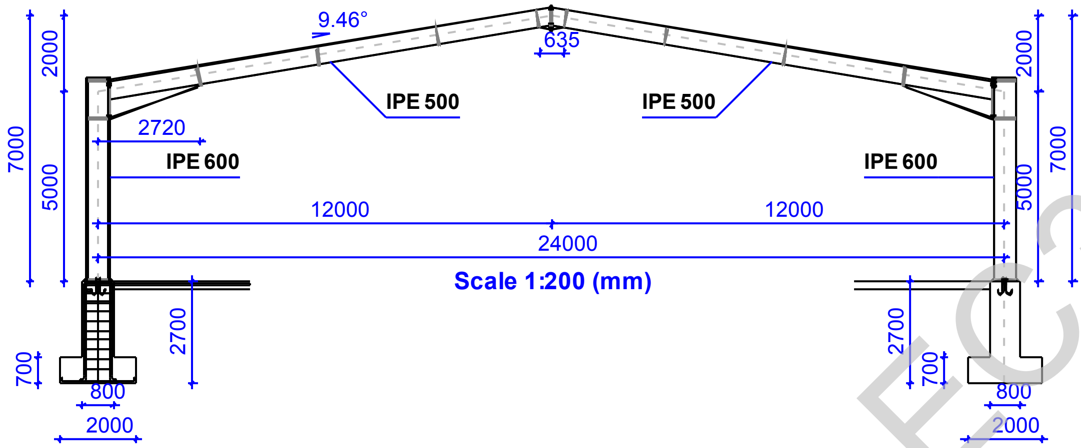
Bolt connection data, Bracing member (EN1993-1-8)

Type of connection	End-plate connection, non-preloaded bolts	
Category of connection	Category A: Bearing type	(EC3-1-8 §3.4.1)
Connected members	Thickness t=10 mm	
Bolts	M20, Strength grade 10.9	
Bolt diameterer	d = 20 mm	
Diameter of holes	do = 22 mm	
Nominal area	$\pi d^2/4 = \pi\text{x}20^2/4 = 314.2$ mm²	
Tensile stress area	As = 314.2 mm²	
Bolt strength grade	10.9, fyb=900N/mm², fub=1000N/mm²	(EC3-1-8 §3.1.1)

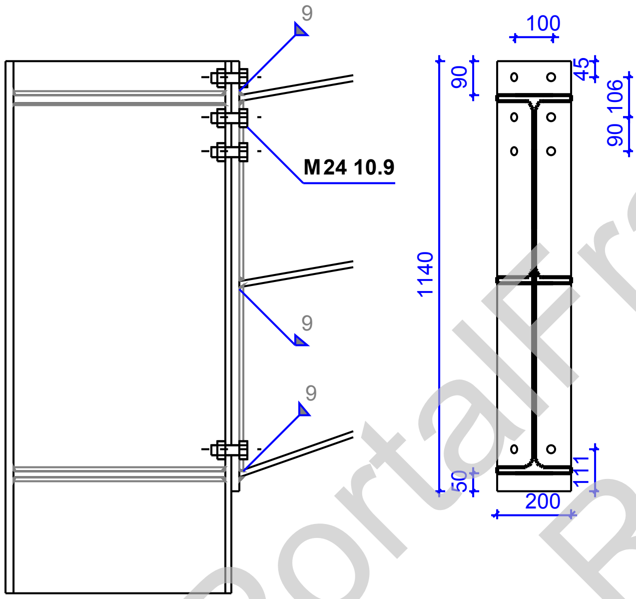
Shear resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)
$F_{v,rd}=\alpha_v\cdot f_{ub}\cdot A_s/\gamma_{M2}=[10^{-3}]\text{x}0.50\text{x}1000\text{x}314.2/1.25=125.7$ kN

Bearing resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)
$F_{b,rd}=k_1\cdot\alpha_b\cdot f_u\cdot d\cdot t/\gamma_{M2}$
t=10.0mm, d=20mm, do=22mm, e1=50mm, e2=50mm, p1=100mm, fub=1000kN/mm², fu=360kN/mm²,
$\alpha_b=\min[f_{ub}/f_u, 1, e_1/3d_o, p_1/3d_o-1/4]=\min[1000/360, 1, 50/(3\text{x}22), 100/(3\text{x}22)-0.25]=0.76$
$k_1=\min[2.8e_2/d_o-1.7, 2.5]=\min[2.8\text{x}50/22-1.7, 2.5]=2.50$
$F_{b,rd}=k_1\cdot\alpha_b\cdot f_u\cdot d\cdot t/\gamma_{M2}=[10^{-3}]\text{x}2.50\text{x}0.76\text{x}360\text{x}20\text{x}10.0/1.25=109.1$ kN

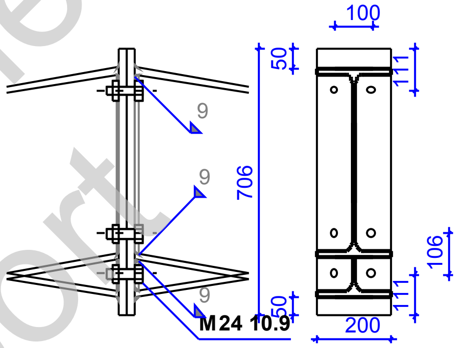
Necessary bolts per brace 95.9/109.1= 1 M20, Grade 10.9



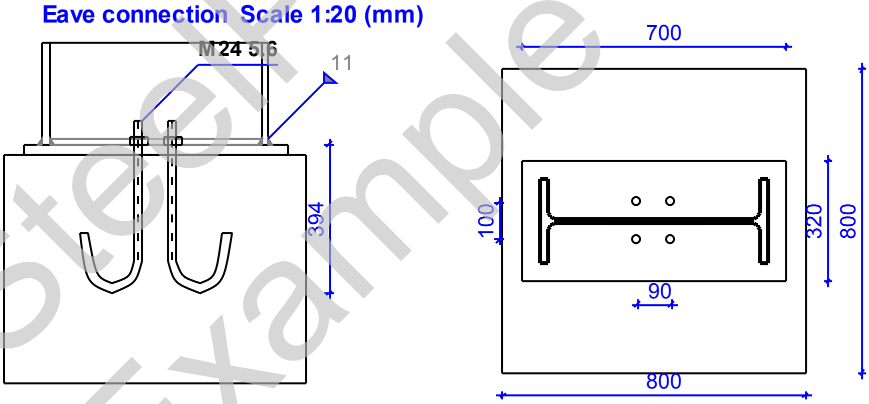
Scale 1:200 (mm)



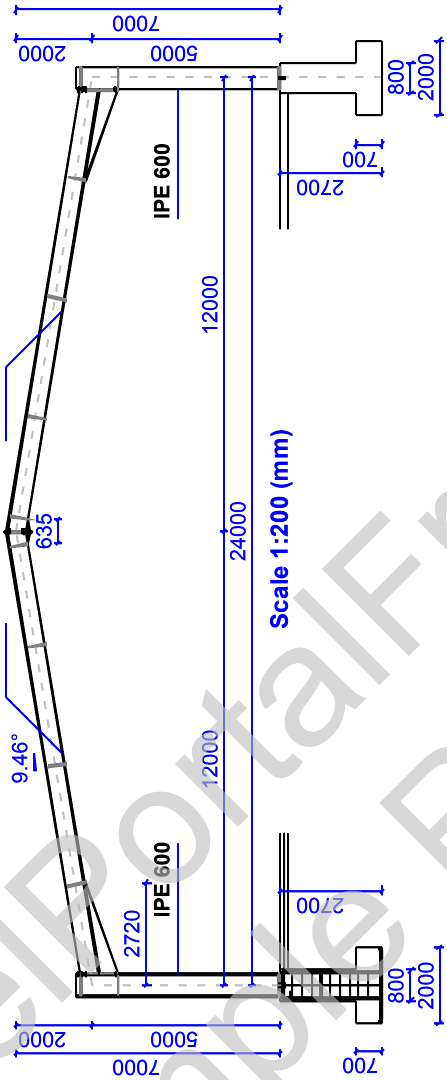
Eave connection Scale 1:20 (mm)

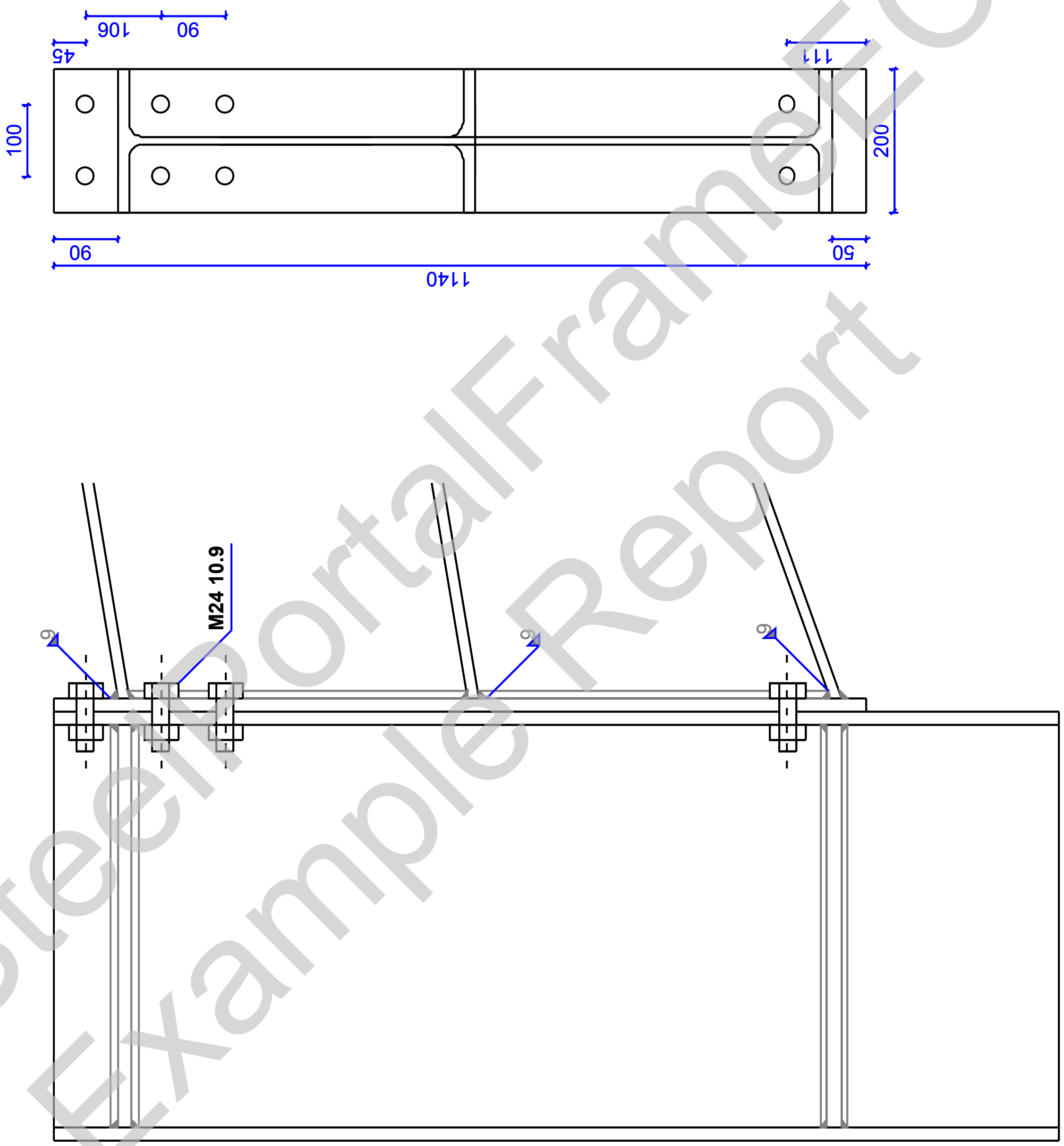


Apex connection Scale 1:20 (mm)

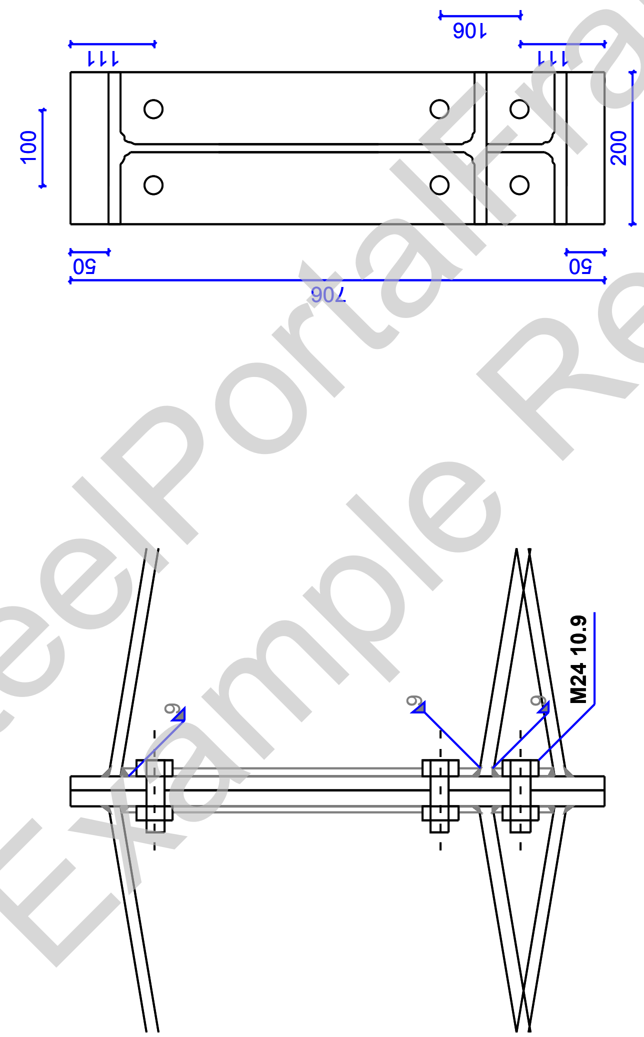


Base connection Scale 1:20 (mm)

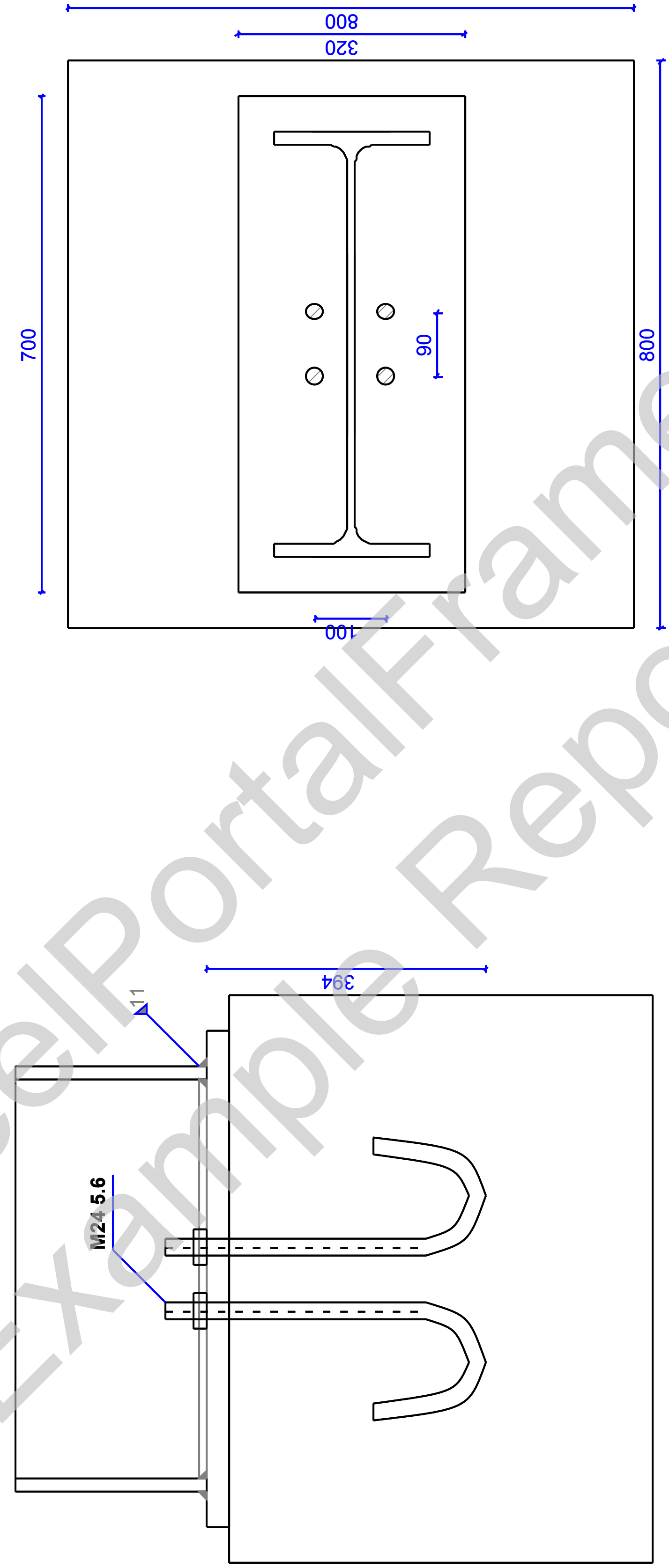

IPE 600
IPE 500
IPE 500
IPE 600
Scale 1:200 (mm)
12000
12000
24000
635
9.46°
2720
7000
5000
2000
2700
700
800
2000



Eave connection Scale 1:10 (mm)



Apex connection Scale 1:10 (mm)



Base connection Scale 1:10 (mm)

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