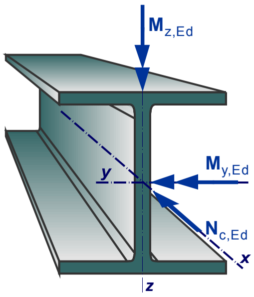
Example

1. STRENGTH-001

Resistance of cross-section, Compression Nc, Bending My and Bending Mz
(EC3 EN1993-1-1:2005, §6.2.9)



Profile : **IPE 400**
Actions : **Bending and compression Nc-My-Mz**
Nc,ed= 650.00kN, My,ed= 40.00kNm, Mz,ed= 15.00kNm
Steel Class : S 355

1.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
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

1.2. 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

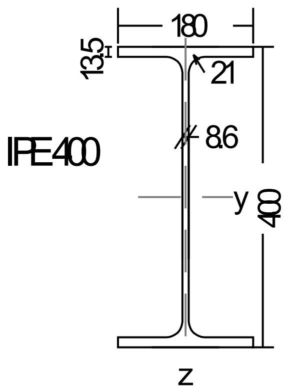
Partial safety factors for actions (EN1990, Annex A1)
γG= 1.35, γQ= 1.50, ψ0= 0.70

Partial factors for materials (EN1993-1-1, §6.1)
γM0= 1.00, γM1= 1.00, γM2= 1.25

Cross-section actions
Bending and compression Nc,ed+My,ed+Mz,ed
Nc,ed=650.00 kN(Compression)
My,ed= 40.00 kNm, Mz,ed= 15.00 kNm

Cross-section properties

Cross-section IPE 400-S 355

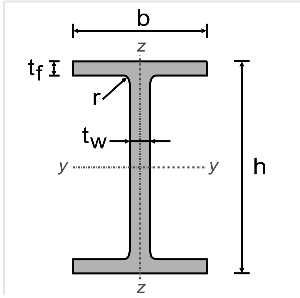


Dimensions of cross section

Depth of cross section	h=	400.00 mm
Width of cross section	b=	180.00 mm
Web depth	hw=	386.50 mm
Depth of straight portion of web	dw=	331.00 mm
Web thickness	tw=	8.60 mm
Flange thickness	tf=	13.50 mm
Radius of root fillet	r=	21.00 mm
Mass	=	66.30 Kg/m

Properties of cross section

Area	$A = 8446$	mm^2		
Second moment of area	$Iy = 231.30 \times 10^6$	mm^4	$Iz = 13.180 \times 10^6$	mm^4
Section modulus	$Wy = 1156.0 \times 10^3$	mm^3	$Wz = 146.40 \times 10^3$	mm^3
Plastic section modulus	$Wpy = 1307.0 \times 10^3$	mm^3	$Wpz = 229.00 \times 10^3$	mm^3
Radius of gyration	$iy = 165.5$	mm	$iz = 39.5$	mm
Shear area	$Avz = 4269$	mm^2	$Avy = 4860$	mm^2
Torsional constant	$It = 0.511 \times 10^6$	mm^4	$ip = 170$	mm
Warping constant	$Iw = 490.05 \times 10^9$	mm^6		



1.3. Classification of cross-sections, Bending and compression

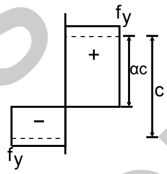
(EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses $\sigma = Ned/Ael \pm Myed/Wel.y \pm Mzed/Wel.z$

$\sigma = [10^3]650.00/8446 \pm [10^6]40.00/1156.0\times10^3 \pm [10^6]15.00/146.4\times10^3$

$\sigma1 = 214$ N/mm², $\sigma2 = -60$ N/mm² $\sigma3 = 9$ N/mm², $\sigma4 = 145$ N/mm² (compression positive)

<u>Web</u>



c=400.0-2x13.5-2x21.0=331.0 mm, t=8.6 mm, c/t=331.0/8.6=38.49

S 355 , t= 8.6<= 40 mm, fy=355 N/mm², $\varepsilon = (235/355)^{0.5} = 0.81$

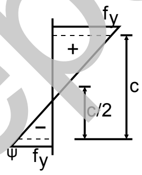
Position of neutral axis for combined Bending and compression

$Ned/(2tw \cdot fy/\gamma M0) = 650000/(2\times8.6\times355/1.00) = 106.5$ mm

$\alpha = (331.0/2+106.5)/331.0 = 0.822 > 0.5$

c/t=38.49>456x0.81/(13x0.822-1)=38.15

The web is not class 1 or 2



$\sigma = Ned/A \pm Myed \cdot (0.5d)/Iy$, $\sigma1 = 77$ N/mm², $\sigma2 = 77$ N/mm²

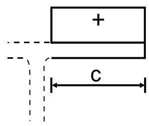
$\psi = 77/77 = 1.000 > -1$

c/t=38.49 > 42x0.81/(0.67+0.33x1.000)=34.02

The web is not class 3

The web is class 4 (EN1993-1-1, Tab.5.2)

<u>Flange</u>



c=180.0/2-8.6/2-21.0=64.7 mm, t=13.5 mm, c/t=64.7/13.5=4.79

S 355 , t=13.5<= 40 mm, fy=355 N/mm², $\varepsilon = (235/355)^{0.5} = 0.81$

$c/t = 4.79 \le 9\varepsilon = 9\times0.81 = 7.29$

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

Overall classification of cross-section is Class 4, Bending and compression

1.4. Effective cross-section properties of Class 4 cross-sections

(EN1993-1-1, §6.2.2.5)

<u>Web</u>

$\bar{\lambda}p = (b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-5, §4.4.2, Eq.4.2, Tabl.4.1)

b=d=331.0mm, t=8.6mm, $\varepsilon = 0.81$, $\psi = 1.00$, $K\sigma = 4.00$, $\bar{\lambda}p = 0.837$

$\bar{\lambda}p = 0.837 > 0.673$ $\rho = [1-0.055(3+1.00)/0.837]/0.837 = 0.881$ ($\rho < 1.0$), $deff = \rho \cdot d = 0.881\times331 = 291.6$ mm

Effective area Aeff=8446-1x(331.0-291.6)x8.60=8107 mm²

1.5. Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

Nc.ed=650.00 kN

Compression Resistance Ncrd= $Aeff \cdot fy/\gamma M0 = [10^{-3}]\times8107\times355/1.00 = 2878.09$kN

Ned= 650.00 kN < 2878.09 kN =Nc,rd, Is verified

Ned/Nc,rd= 650.00/2878.09= 0.226<1

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

Web

$\bar{\lambda}p=(b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-5, §4.4.2, Eq.4.2, Tabl.4.1)

b=d=331.0mm, t=8.6mm, ε=0.81, ψ=-1.00, Kσ=23.90, $\bar{\lambda}p=0.342$

$\bar{\lambda}p=0.342<=0.673$, ρ=1.0, heff=ρ·d/2=1.000x166=165.5 mm

Effective area Aeff=8446-1x(165.5-165.5)x8.60=8446 mm²

emy=99.30x(8446/8446-1)=0.00 mm, Iy,eff=231.30x10^{6} mm^{4}

Effective section modulus Wy,eff=231.30x10^{6}/(400.0/2+0.00)=1156.5x10^{3} mm³

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

My.ed= 40.00 kNm

Bending Resistance Mcy,rd=Weffy·fy/γM0=[10^{-6}]x1156.5x10^{3}x355/1.00= 410.56kNm

My,ed= 40.00 kNm < 410.56 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 40.00/410.56= 0.097<1

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

Web

$\bar{\lambda}p=(b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-5, §4.4.2, Eq.4.2, Tabl.4.1)

b=d=331.0mm, t=8.6mm, ε=0.81, ψ=-1.00, Kσ=23.90, $\bar{\lambda}p=0.342$

$\bar{\lambda}p=0.342<=0.673$, ρ=1.0, heff=ρ·d/2=1.000x166=165.5 mm

Effective area Aeff=8446-1x(165.5-165.5)x8.60=8446 mm²

emz=99.30x(8446/8446-1)=0.00 mm, Iz,eff=13.180x10^{6} mm^{4}

Effective section modulus Wz,eff=13.180x10^{6}/(400.0/2+0.00)=65.900x10^{3} mm³

1.9. Ultimate Limit State, Verification for bending moment z-z (EN1993-1-1, §6.2.5)

Mz.ed= 15.00 kNm

Bending Resistance Mcz,rd=Weffz·fy/γM0=[10^{-6}]x65.900x10^{3}x355/1.00= 23.39kNm

Mz,ed= 15.00 kNm < 23.39 kNm =Mz,rd=Mplz,rd, Is verified

Mz,ed/Mz,rd= 15.00/23.39= 0.641<1

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

N.ed=650.00 kN (Compression), My.ed= 40.00 kNm, Mz.ed= 15.00 kNm

Nplrd=2878.09kN, Mc,y,rd=410.56kNm, Mc,z,rd=23.39kNm

Ned=650.00kN > [10^{-3}]x0.5x386.5x8.6x355/1.00=0.5hw·tw·fy/γM0=589.99 kN

n=Ned/Nplrd=650/2878= 0.226

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

Ved=0 kN, Effect of shear force is neglected (EC3 §6.2.8.2)

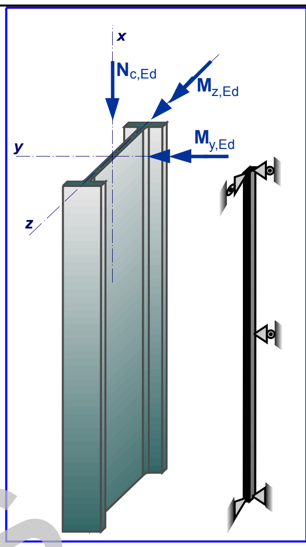
Maximum and minimum cross-section stresses σ=Ned/Aeff ± Myed/Weff.y ± Mzed/Weff.z

σ=[10^{3}]650.00/8107 ± [10^{6}]40.00/1156.5x10³ ± [10^{6}]15.00/65.9x10³

σ1=342 N/mm², σ2=-182 N/mm², σ3=-113 N/mm², σ4=273 N/mm² (compression positive)

σx.ed=342 < 355/1.00=355=fy/γM0 N/mm², Is verified (EC3 Eq.6.43, Eq.6.44)

2. COLUMN-001



Buckling resistance, Members in compression Nc-My-Mz
(EC3 EN1993-1-1:2005, §6.3.1)

Profile : IPE 450
Actions : Bending and compression Nc-My-Mz
Ng= 530.00kN, Nq= 260.00kN
Steel Class : S 355

2.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
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

2.2. Materials

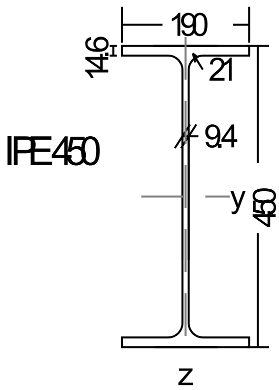
Steel: S 355 (EN1993-1-1, §3.2)
t<= 40 mm, Yield strength fy= 355 N/mm², Ultimate strength fu= 510 N/mm²
40mm<t<= 80 mm, Yield strength fy= 335 N/mm², Ultimate strength fu= 470 N/mm²
Modulus of elasticity E=210000 N/mm², Poisson ratio ν=0.30, Unit mass ρ= 7850 Kg/m³

Partial safety factors for actions (EN1990, Annex A1)
γ_G= 1.35, γ_Q= 1.50, ψ_0= 0.70

Partial factors for materials (EN1993-1-1, §6.1)
γ_{M0}= 1.00, γ_{M1}= 1.00, γ_{M2}= 1.25

Cross-section properties

Cross-section IPE 450-S 355

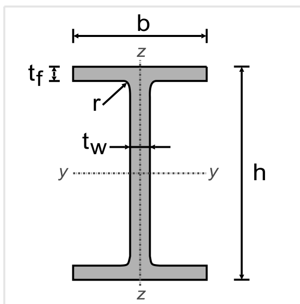


Dimensions of cross section

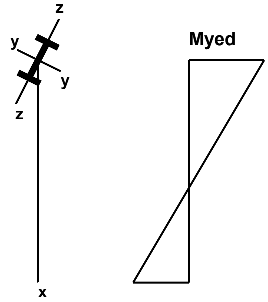
Depth of cross section	h=	450.00 mm
Width of cross section	b=	190.00 mm
Web depth	hw=	435.40 mm
Depth of straight portion of web	dw=	378.80 mm
Web thickness	tw=	9.40 mm
Flange thickness	tf=	14.60 mm
Radius of root fillet	r=	21.00 mm
Mass	=	77.60 Kg/m

Properties of cross section

Area	A= 9882	mm^2		
Second moment of area	Iy=337.40x10^6	mm^4	Iz=16.760x10^6	mm^4
Section modulus	Wy=1500.0x10^3	mm^3	Wz=176.40x10^3	mm^3
Plastic section modulus	Wpy=1702.0x10^3	mm^3	Wpz=276.40x10^3	mm^3
Radius of gyration	iy= 184.8	mm	iz= 41.2	mm
Shear area	Avz= 5084	mm^2	Avy= 5548	mm^2
Torsional constant	It= 0.669x10^6	mm^4	ip= 189	mm
Warping constant	Iw=791.01x10^9	mm^6		



2.3. Dimensions and loads



Column length L=3.400 m
Buckling lengthy-y: Lcr,y=1.000x3.400=3.400m
Buckling lengthz-z: Lcr,z=0.500x3.400=1.700m
Column loads
axial load compression   Ng=530.00 kN   Nq=260.00 k
bending moment at top   MyyAg= 31.00 kNm MyyAq= 19.50 k
bending moment at bottom MyyBg=-25.00 kNm MyyBq=-12.50 k
bending moment at top   MzzAg= 0.00 kNm MzzAq= 0.00 k
bending moment at bottom MzzBg= 0.00 kNm MzzBq= 0.00 kNm

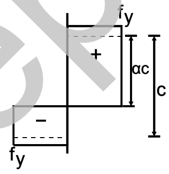
2.4. Design actions

Bending moments, shearing forces, axial forces, load combination 1.35g+1.50q

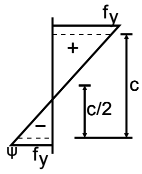
x= 0.00m, Ned=1105.50kN, My,ed= 71.10kNm, Mz,ed=   0.00kNm, Vz,ed= 36.35kN, Vy,ed= 0.00kN
x= 1.70m, Ned=1105.50kN, My,ed=   9.30kNm, Mz,ed=   0.00kNm, Vz,ed= 36.35kN, Vy,ed= 0.00kN
x= 3.40m, Ned=1105.50kN, My,ed= -52.50kNm, Mz,ed=   0.00kNm, Vz,ed= 36.35kN, Vy,ed= 0.00kN

2.5. Classification of cross-sections, Bending and compression (EN1993-1-1, §5.5)

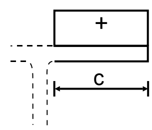
Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
σ=[10^3]1105.50/9882 ± [10^6]71.10/1500.0x10³ ± [10^6]0.00/176.4x10³
σ1=159 N/mm², σ2=64 N/mm² (compression positive)



Web
c=450.0-2x14.6-2x21.0=378.8 mm, t=9.4 mm, c/t=378.8/9.4=40.30
S 355 , t= 9.4<= 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)=1105500/(2x9.4x355/1.00)=165.6 mm
α=(378.8/2+165.6)/378.8=0.937>0.5
c/t=40.30>456x0.81/(13x0.937-1)=33.02
The web is not class 1 or 2



σ=Ned/A±Myed·(0.5d)/Iy, σ1=112 N/mm², σ2=112 N/mm²
ψ=112/112=1.000 > -1
c/t=40.30 > 42x0.81/(0.67+0.33x1.000)=34.02
The web is not class 3
The web is class 4 (EN1993-1-1, Tab.5.2)



Flange
c=190.0/2-9.4/2-21.0=69.3 mm, t=14.6 mm, c/t=69.3/14.6=4.75
S 355 , t=14.6<= 40 mm, fy=355 N/mm², ε=$(235/355)^{0.5}$=0.81
c/t=4.75<=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

2.6. 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)] (EN1993-1-5, §4.4.2, Eq.4.2, Tabl.4.1)
b=d=378.8mm, t=9.4mm, ε=0.81, ψ=1.00, Kσ=4.00, $\bar{\lambda}$p=0.876
$\bar{\lambda}$p=0.876>0.673 ρ=[1-0.055(3+1.00)/0.876]/0.876=0.855 (ρ<1.0), deff=ρ·d=0.855x379=323.8 mm

Effective area Aeff=9882-1x(378.8-323.8)x9.40=9365 mm²

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

Nc.ed=1105.50 kN

Compression Resistance Ncrd= $A_{eff}\cdot f_y/\gamma_{M0}=[10^{-3}]$x9365x355/1.00=3324.74kN

Ned= 1105.50 kN < 3324.74 kN =Nc,rd, Is verified

Ned/Nc,rd= 1105.50/3324.74= 0.333<1

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

Web

$\bar{\lambda}_p=(b/t)/[28.40\varepsilon\sqrt{(K\sigma)}]$ (EN1993-1-5, §4.4.2, Eq.4.2, Tabl.4.1)

b=d=378.8mm, t=9.4mm, ε=0.81, ψ=-1.00, Kσ=23.90, $\bar{\lambda}_p$=0.358

$\bar{\lambda}_p$=0.358<=0.673, ρ=1.0, heff=ρ·d/2=1.000x189=189.4 mm

Effective area Aeff=9882-1x(189.4-189.4)x9.40=9882 mm²

emy=113.64x(9882/9882-1)=0.00 mm, Iy,eff=337.40x10^6 mm^4

Effective section modulus Wy,eff=337.40x10^6/(450.0/2+0.00)=1499.6x10^3 mm³

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

My.ed= 71.10 kNm

Bending Resistance Mcy,rd=Weffy·fy/γM0=[10^{-6}]x1499.6x10^3x355/1.00= 532.34kNm

My,ed= 71.10 kNm < 532.34 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 71.10/532.34= 0.134<1

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

Vz.ed= 36.35 kN

Av=A-2b·tf+(tw+2r)tf=9882-2x190.0x14.6+(9.4+2x21.0)x14.6=5084mm² (EC3 §6.2.6.3)

Av= 5084mm² > η·hw·tw= 1.00x(450.0-2x14.6)x9.4=1.00x435.4x9.4= 4093mm²

Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x5084x(355/1.73)/1.00= 1042.10kN

Vz,ed= 36.35 kN < 1042.10 kN =Vz,rd=Vpl,z,rd, Is verified

Vz,ed/Vz,rd= 36.35/1042.10= 0.035<1

hw/tw=(450.0-2x14.6)/9.4=435.4/9.4=46.32<=72x0.81/1.00=72ε/η=58.32 (η=1.00)

S 355 , t= 9.4<= 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)

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

N.ed= 1105.50kN (Compression), Vz.ed= 36.35kN, My.ed= 71.10kNm

Nplrd=3324.74kN, Mc,y,rd=532.34kNm, Vpl,z,rd=1042.10kN

Ned=1105.50kN > 0.25x3324.74=0.25xNplrd=831.18kN

Ned=1105.50kN > [10^{-3}]x0.5x435.4x9.4x355/1.00=0.5hw·tw·fy/γM0=726.46 kN

n=Ned/Nplrd=1106/3325= 0.333

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

Ved=36.35kN <= 0.50x1042.10=0.50xVpl,rd=521.05kN

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

σ=[10^3]1105.50/9365 ± [10^6]71.10/1499.6x10³ ± [10^6]0.00/176.4x10³

σ1=165 N/mm², σ2=71 N/mm² (compression positive)

σx.ed=165 < 355/1.00=355=fy/γM0 N/mm², Is verified (EC3 Eq.6.43, Eq.6.44)

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

N.ed= 1105.50kN (Compression), Vz.ed= 36.35kN, My.ed= 9.30kNm

Position at x=1.70 m

Nplrd=3324.74kN, Mc,y,rd=532.34kNm, Vpl,z,rd=1042.10kN

Ned=1105.50kN > 0.25x3324.74=0.25xNplrd=831.18kN

Ned=1105.50kN > [10^{-3}]x0.5x435.4x9.4x355/1.00=0.5hw·tw·fy/γM0=726.46 kN

n=Ned/Nplrd=1106/3325= 0.333

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

Ved=36.35kN <= 0.50x1042.10=0.50xVpl,rd=521.05kN

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
σ=[10^3]1105.50/9365 ± [10^6]9.30/1499.6x10^3 ± [10^6]0.00/176.4x10^3
σ1=124 N/mm^2, σ2=112 N/mm^2 (compression positive)
σx.ed=124 < 355/1.00=355=fy/γM0 N/mm^2,  Is verified                  (EC3 Eq.6.43, Eq.6.44)
```

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

N.ed= 1105.50kN (Compression), Vz.ed= 36.35kN, My.ed= 52.50kNm

Position at x=3.40 m

```
Nplrd=3324.74kN, Mc,y,rd=532.34kNm, Vpl,z,rd=1042.10kN
Ned=1105.50kN > 0.25x3324.74=0.25xNplrd=831.18kN
Ned=1105.50kN > [10^-3]x0.5x435.4x9.4x355/1.00=0.5hw·tw·fy/γM0=726.46 kN
n=Ned/Nplrd=1106/3325= 0.333
Effect of axial force is considered               (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=36.35kN <= 0.50x1042.10=0.50xVpl,rd=521.05kN
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
σ=[10^3]1105.50/9365 ± [10^6]52.50/1499.6x10^3 ± [10^6]0.00/176.4x10^3
σ1=153 N/mm^2, σ2=83 N/mm^2 (compression positive)
σx.ed=153 < 355/1.00=355=fy/γM0 N/mm^2,  Is verified                  (EC3 Eq.6.43, Eq.6.44)
```

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

Nc,ed=1105.50 kN, Lcr,y=3.400 m, Lcr,z=1.700 m

```
Buckling lengths: Lcr,y=1.000x3400=3400mm, Lcr,z=0.500x3400=1700mm
Non-dimensional slenderness (Cross-section Class: 4 )                        (EC3 §6.3.1.3)
λ̄y=√(A·fy/Ncr,y)=(Lcr,y/iy)√(Aeff/A)/λ1)=(3400/184.8)x(0.974/76.06)=0.236
λ̄z=√(A·fy/Ncr,z)=(Lcr,z/iz)√(Aeff/A)/λ1)=(1700/ 41.2)x(0.974/76.06)=0.528
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81, √(Aeff/A)=√(9365/9882)=0.974

h/b=450/190=2.37>=1.20, tf=14.6mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.992          (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(λ̄y-0.2)+λ̄y^2]=0.5x[1+0.21x(0.236-0.2)+0.236^2]=0.532
χy=1/[Φy+√(Φy^2-λ̄y^2)]=1/[0.532+√(0.532^2-0.236^2)]=0.992 <=1 χy=0.992
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.872
Φz=0.5[1+αz(λ̄z-0.2)+λ̄z^2]=0.5x[1+0.34x(0.528-0.2)+0.528^2]=0.695
χz=1/[Φz+√(Φz^2-λ̄z^2)]=1/[0.695+√(0.695^2-0.528^2)]=0.872 <=1 χz=0.872

Reduction factor χ=1/[Φ+√(Φ^2-λ̄^2)], χ<=1.0, Φ=0.5[1+α(λ̄-0.2)+λ̄^2], χ=0.872     (EC3 Eq.6.49)
Nb,rd=χ·Aeff·fy/γM1= 0.872x[10^-3]x9365x355/1.00=2899.17kN                      (EC3 Eq.6.48)
Nc,ed= 1105.50 kN < 2899.17 kN =Nb,rd,  Is verified
Nc,ed/Nb,rd= 1105.50/2899.17= 0.381<1
```

2.15. Lateral torsional buckling, (ULS) (EN1993-1-1, §6.3.2)

My,ed=71.10 kN, L=3.400m, Lcr,y=3.400m, Lcr,z=1.700m, Lcr,lt=1.700m

```
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·[π^2EIz/(kL)^2]{√[(kz/kw)^2(Iw/Iz)+(kL)^2GIt/(π^2EIz)+(C2·zg-C3·zj)^2] -(C2·zg-C3·zj)}
```
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
```
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^4 N/mm^2
k·L=1700mm,  zg=h/2=450/2=225mm, zj=0mm                               (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, ψ=-0.738, C1=2.559, C2=0.000, C3=0.000
Mcr=[10^-6]2.559x[π^2x2.1x10^5x16.760x10^6/1700^2]
   x{ [(1.0/1.0)^2x(791.01x10^9/16.760x10^6)
   +1700^2x8.1x10^4x0.669x10^6/(π^2x2.1x10^5x16.760x10^6)]^0.5 }= 6992.7 kNm

λ̄,lt=√(Weff,y·fy/Mcr)=√{[10^-6]x1499.6x10^3x355/6992.7}=0.276                  (EC3 Eq.6.56)
λ̄,lt<= 0.40, χ,lt=1.00                                                      (EC3 §6.3.2.2.4)
```

Mb,rd=χ,lt·Weff,y·fy/γM1= 1.000x[10^{-6}]x1499.6x10^{3}x355/1.00=532.34kNm (EC3 Eq.6.55)
My,ed= 71.10 kNm < 532.34 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 71.10/532.34= 0.134<1

2.16. Axial force and bending moment, (ULS) (EN1993-1-1, §6.3.3)

Ned=1105.50 kN, My,ed=71.10 kNm

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=Aeff·fy=[10^{-3}]x9365x355=3324.7 kN (Tab.6.7)
My,rk=Weff,y·fy=[10^{-6}]x1499.6x10^{3}x355=532.3 kNm
χy·Nrk/γM1=χy·Aeff·fy/γM1= 0.992x[10^{-3}]x9365x355/1.00=3298.1kN
χz·Nrk/γM1=χz·Aeff·fy/γM1= 0.872x[10^{-3}]x9365x355/1.00=2899.2kN
χLT·My,rk/γM1=χLT·Weff,y·fy/γM1= 1.000x[10^{-6}]x1499.6x10^{3}x355/1.00=532.3kNm

Interaction factors, Method of computation: Method 2 Annex B (EC3 AnnexB)

kyy=Cmy[1+0.6$\bar{\lambda}$y[Ned/(χy·Nrk/γM1)]], Kyy<=Cmy[1+0.6[Ned/(χy·Nrk/γM1)]] (EC3 Tab.B.1)
kzy=1-[0.05$\bar{\lambda}$z/(Cmlt-0.25)][Ned/(χz·Nrk/γM1)], Kyz>=1-[0.05/(Cmlt-0.25)][Ned/(χz·Nrk/γM1)]
ψ=-0.74, Cmy=0.400, Cmtl=0.40

Kyy=0.40x[1+0.6x0.236x(1105.5/3298.1)], Kyy<=0.40x[1+0.6x(1105.5/3298.1)]
Kyy= 0.419, Kyy<=0.480, Kyy=0.419
Kzy=[1-[0.05x0.528/(0.40-0.25)]x(1105.5/2899.2)], Kzy>=[1-[0.05/(0.40-0.25)]x(1105.5/2899.2)]
Kzy=0.933, Kzy>=0.873, Kzy=0.933

Position at x=0.00 m

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
1105.5/(0.992x3324.7/1.00)+0.419x71.1/(1.000x532.3/1.00)=0.335+0.056=0.391
0.391< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
1105.5/(0.872x3324.7/1.00)+0.933x71.1/(1.000x532.3/1.00)=0.381+0.125=0.506
0.506< 1.000, Is verified

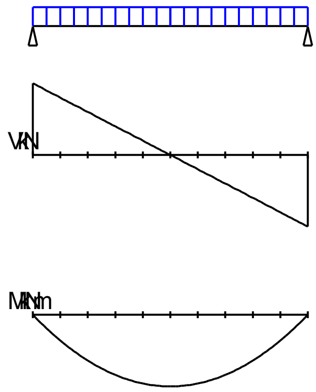
Position at x=1.70 m

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
1105.5/(0.992x3324.7/1.00)+0.419x9.3/(1.000x532.3/1.00)=0.335+0.007=0.343
0.343< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
1105.5/(0.872x3324.7/1.00)+0.933x9.3/(1.000x532.3/1.00)=0.381+0.016=0.398
0.398< 1.000, Is verified

Position at x=3.40 m

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
1105.5/(0.992x3324.7/1.00)+0.419x52.5/(1.000x532.3/1.00)=0.335+0.041=0.377
0.377< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
1105.5/(0.872x3324.7/1.00)+0.933x52.5/(1.000x532.3/1.00)=0.381+0.092=0.473
0.473< 1.000, Is verified

3. BEAM-001



Members in bending, My
(EC3 EN1993-1-1:2005, §6.3.2)

Profile : IPE 240
Actions : Design of beams, Bending and compression
Steel Class : S 355

3.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
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

3.2. Materials

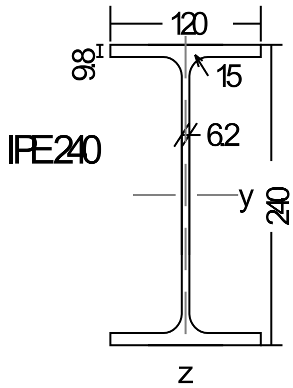
Steel: S 355 (EN1993-1-1, §3.2)
t<= 40 mm, Yield strength fy= 355 N/mm², Ultimate strength fu= 510 N/mm²
40mm<t<= 80 mm, Yield strength fy= 335 N/mm², Ultimate strength fu= 470 N/mm²
Modulus of elasticity E=210000 N/mm², Poisson ratio ν=0.30, Unit mass ρ= 7850 Kg/m³

Partial safety factors for actions (EN1990, Annex A1)
γG= 1.35, γQ= 1.50, ψ0= 0.70

Partial factors for materials (EN1993-1-1, §6.1)
γM0= 1.00, γM1= 1.00, γM2= 1.25

Cross-section properties

Cross-section IPE 240-S 355

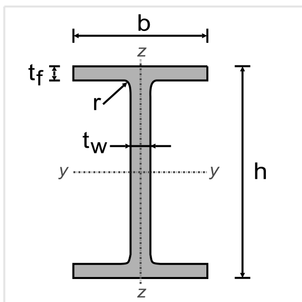


Dimensions of cross section

Depth of cross section	h=	240.00	mm
Width of cross section	b=	120.00	mm
Web depth	hw=	230.20	mm
Depth of straight portion of web	dw=	190.40	mm
Web thickness	tw=	6.20	mm
Flange thickness	tf=	9.80	mm
Radius of root fillet	r=	15.00	mm
Mass	=	30.70	Kg/m

Properties of cross section

Area	A= 3912 mm^2			
Second moment of area	Iy=38.920x10^6	mm^4	Iz= 2.836x10^6	mm^4
Section modulus	Wy=324.30x10^3	mm^3	Wz=47.270x10^3	mm^3
Plastic section modulus	Wpy=366.60x10^3	mm^3	Wpz=73.920x10^3	mm^3
Radius of gyration	iy= 99.7	mm	iz= 26.9	mm
Shear area	Avz= 1915	mm^2	Avy= 2352	mm^2
Torsional constant	It= 0.129x10^6	mm^4	ip= 103	mm
Warping constant	Iw=37.391x10^9	mm^6		



3.3. Dimensions and loads

Beam span L=5.40 m
Buckling lengthy-y: Lcr,y=1.000x5.400=5.400m
Buckling lengthz-z: Lcr,z=0.630x5.400=3.400m

Beam loads

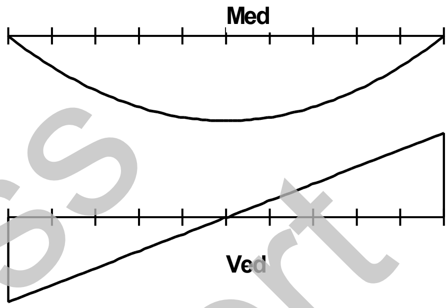
beam self weight	go= 0.31 kN/m	
uniform load	g1= 7.10 kN/m	q1= 4.20 kN/m

3.4. Design actions, shearing forces and bending moments

Bending moments and shears, load combination 1.35g+1.50q

x/L=0.00,	x= 0.00m,	Med= 0.00 kNm,	Ved= 44.01 kN
x/L=0.10,	x= 0.54m,	Med= 21.39 kNm,	Ved= 35.21 kN
x/L=0.20,	x= 1.08m,	Med= 38.02 kNm,	Ved= 26.41 kN
x/L=0.30,	x= 1.62m,	Med= 49.91 kNm,	Ved= 17.60 kN
x/L=0.40,	x= 2.16m,	Med= 57.04 kNm,	Ved= 8.80 kN
x/L=0.50,	x= 2.70m,	Med= 59.41 kNm,	Ved= 0.00 kN
x/L=0.60,	x= 3.24m,	Med= 57.04 kNm,	Ved= -8.80 kN
x/L=0.70,	x= 3.78m,	Med= 49.91 kNm,	Ved= -17.60 kN
x/L=0.80,	x= 4.32m,	Med= 38.02 kNm,	Ved= -26.41 kN
x/L=0.90,	x= 4.86m,	Med= 21.39 kNm,	Ved= -35.21 kN
x/L=1.00,	x= 5.40m,	Med= 0.00 kNm,	Ved= -44.01 kN

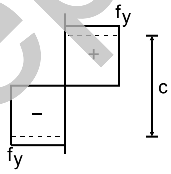
Maximum Med= 59.41 kNm, Maximum Ved= 44.01 kN



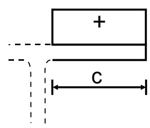
3.5. Classification of cross-sections, Bending My

(EN1993-1-1, §5.5)

Web
c=240.0-2x9.8-2x15.0=190.4 mm, t=6.2 mm, c/t=190.4/6.2=30.71
S 355 , t= 6.2<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
c/t=30.71<=72ε=72x0.81=58.32
The web is class 1 (EN1993-1-1, Tab.5.2)



Flange
c=120.0/2-6.2/2-15.0=41.9 mm, t=9.8 mm, c/t=41.9/9.8=4.28
S 355 , t= 9.8<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
c/t=4.28<=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

3.6. Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

My.ed= 59.41 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x366.60x10^{3}x355/1.00= 130.14kNm
My,ed= 59.41 kNm < 130.14 kNm =My,rd=Mply,rd, Is verified
My,ed/My,rd= 59.41/130.14= 0.457<1

3.7. Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

Vz.ed= 44.01 kN
Av=A-2b·tf+(tw+2r)tf=3912-2x120.0x9.8+(6.2+2x15.0)x9.8=1915mm² (EC3 §6.2.6.3)
Av= 1915mm² > η·hw·tw= 1.00x(240.0-2x9.8)x6.2=1.00x230.2x6.2= 1427mm²
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x1915x(355/1.73)/1.00= 392.45kN
Vz,ed= 44.01 kN < 392.45 kN =Vz,rd=Vpl,z,rd, Is verified
Vz,ed/Vz,rd= 44.01/392.45= 0.112<1

hw/tw=(240.0-2x9.8)/6.2=230.2/6.2=37.13<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t= 6.2<= 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)

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

N.ed= 0.00kN, Vz.ed= 21.78kN, My.ed= 44.86kNm

Position at x=1.35 m

Mpl,y,rd=130.14kNm, Vpl,z,rd=392.45kN
Ned=0 kN, Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=21.78kN <= 0.50x392.45=0.50xVpl,rd=196.23kN
Effect of shear force is neglected (EC3 §6.2.8.2)

3.9. Lateral torsional buckling, (ULS) (x=0.000~2.700m) (EN1993-1-1, §6.3.2)

My,ed=59.41 kN, L=5.400m, Lcr,y=5.400m, Lcr,z=5.400m, Lcr,lt=2.700m

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
$Mcr=C1\cdot[\pi^2EIz/(kL)^2]\{\sqrt{[(kz/kw)^2(Iw/Iz)+(kL)^2GIt/(\pi^2EIz)+(C2\cdot zg-C3\cdot zj)^2]}-(C2\cdot zg-C3\cdot zj)\}$
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
$\mu=Mo/M=qL^2/8M=15.2/59.4=0.26$, $\psi=Mb/Ma=0.0/59.4=0.00$, C1=1.317, C2=0.124
$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^{4}$ N/mm²
k·L=2700mm, zg=h/2=240/2=120mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=1.317, C2=0.124, C3=0.000
$Mcr=[10^{-6}]1.317x[\pi^2x2.1x10^{5}x2.836x10^{6}/2700^2]$
$x\{ [(1.0/1.0)^2x(37.391x10^{9}/2.836x10^{6})$
$+2700^2x8.1x10^{4}x0.129x10^{6}/(\pi^2x2.1x10^{5}x2.836x10^{6})$
$+(0.124x120)^2]^{0.5}-(0.124x120) \}=$ 156.5 kNm

$\bar{\lambda},lt=\sqrt{(Wpl,y\cdot fy/Mcr)}=\sqrt{\{[10^{-6}]x366.60x10^{3}x355/156.5\}}=0.912$ (EC3 Eq.6.56)
h/b=240/120=2.00<=2.00 buckling curve:b
imperfection factor: α,lt=0.34, β=0.75, χ,lt=0.753 (T.6.3, T.6.5, Fig.6.4)
$\Phi,lt=0.5[1+\alpha,lt(\bar{\lambda},lt-\bar{\lambda},lto)+\beta\bar{\lambda},lt^2]=0.5x[1+0.34x(0.912-0.40)+0.75x0.912^2]=0.899$
$\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar{\lambda},lt^2)}]=1/[0.899+\sqrt{(0.899^2-0.75x0.899^2)}]=0.753$
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=0.753 (Eq.6.57)

$\chi,lt,mod=\chi,lt/f$, $\chi,lt,mod<=1$, $\chi,lt,mod<=1/\bar{\lambda},lt^2=1/0.912^2=1.20$ (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)
$f=1-0.5(1-kc)[1-2.0(\bar{\lambda},lt-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.912-0.8)^2]=0.879$, f<=1.0
$\chi,lt,mod=\chi,lt/f=0.753/0.879=0.857$, χ,lt,mod<=1.0, χ,lt,mod<=1.20, χ,lt,mod=0.857

$Mb,rd=\chi,lt\cdot Wpl,y\cdot fy/\gamma M1=0.857x[10^{-6}]x366.60x10^{3}x355/1.00=111.53kNm$ (EC3 Eq.6.55)
My,ed= 59.41 kNm < 111.53 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 59.41/111.53= 0.533<1

3.10. Lateral torsional buckling, (ULS) (x=2.700~5.400m) (EN1993-1-1, §6.3.2)

My,ed=59.41 kN, L=5.400m, Lcr,y=5.400m, Lcr,z=5.400m, Lcr,lt=2.700m

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
$Mcr=C1\cdot[\pi^2EIz/(kL)^2]\{\sqrt{[(kz/kw)^2(Iw/Iz)+(kL)^2GIt/(\pi^2EIz)+(C2\cdot zg-C3\cdot zj)^2]}-(C2\cdot zg-C3\cdot zj)\}$
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
$\mu=Mo/M=qL^2/8M=15.2/59.4=0.26$, $\psi=Mb/Ma=0.0/59.4=0.00$, C1=1.317, C2=0.124
$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^{4}$ N/mm²
k·L=2700mm, zg=h/2=240/2=120mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=1.317, C2=0.124, C3=0.000
$Mcr=[10^{-6}]1.317x[\pi^2x2.1x10^{5}x2.836x10^{6}/2700^2]$
$x\{ [(1.0/1.0)^2x(37.391x10^{9}/2.836x10^{6})$
$+2700^2x8.1x10^{4}x0.129x10^{6}/(\pi^2x2.1x10^{5}x2.836x10^{6})$
$+(0.124x120)^2]^{0.5}-(0.124x120) \}=$ 156.5 kNm

$\bar{\lambda},lt=\sqrt{(Wpl,y\cdot fy/Mcr)}=\sqrt{\{[10^{-6}]x366.60x10^{3}x355/156.5\}}=0.912$ (EC3 Eq.6.56)
h/b=240/120=2.00<=2.00 buckling curve:b
imperfection factor: α,lt=0.34, β=0.75, χ,lt=0.753 (T.6.3, T.6.5, Fig.6.4)
$\Phi,lt=0.5[1+\alpha,lt(\bar{\lambda},lt-\bar{\lambda},lto)+\beta\bar{\lambda},lt^2]=0.5x[1+0.34x(0.912-0.40)+0.75x0.912^2]=0.899$
$\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar{\lambda},lt^2)}]=1/[0.899+\sqrt{(0.899^2-0.75x0.899^2)}]=0.753$
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=0.753 (Eq.6.57)

$\chi_{,lt,mod}=\chi_{,lt}/f$, $\chi_{,lt,mod}<=1$, $\chi_{,lt,mod}<=1/\bar{\lambda}_{,lt}^2=1/0.912^2=1.20$ (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)
$f=1-0.5(1-kc)[1-2.0(\bar{\lambda}_{,lt}-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.912-0.8)^2]=0.879$, $f<=1.0$
$\chi_{,lt,mod}=\chi_{,lt}/f=0.753/0.879=0.857$, $\chi_{,lt,mod}<=1.0$, $\chi_{,lt,mod}<=1.20$, $\chi_{,lt,mod}=0.857$

$Mb,rd=\chi_{,lt}\cdot Wpl,y\cdot fy/\gamma M1=0.857x[10^{-6}]x366.60x10^{3}x355/1.00=111.53kNm$ (EC3 Eq.6.55)
My,ed= 59.41 kNm < 111.53 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 59.41/111.53= 0.533<1

4. JOINTd-001

Chord continuity of I sections
(EC3 EN1993-1-8:2005, §3, §6)

4.1. Materials

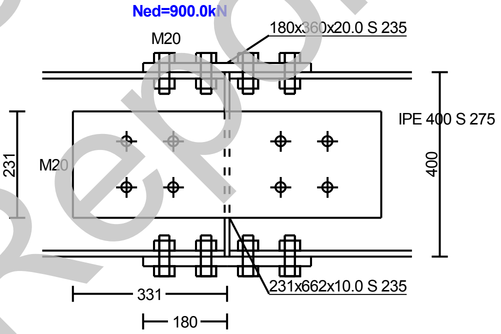
Steel: S 275 (EN1993-1-1, §3.2)
t<= 40 mm, Yield strength fy= 275 N/mm², Ultimate strength fu= 430 N/mm²
40mm<t<= 80 mm, Yield strength fy= 255 N/mm², Ultimate strength fu= 410 N/mm²
Modulus of elasticity E=210000 N/mm², Poisson ratio ν=0.30, Unit mass ρ= 7850 Kg/m³

Partial factors for materials (EN1993-1-1, §6.1)
γ_{M0}= 1.00, γ_{M1}= 1.00, γ_{M2}= 1.25

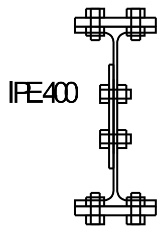
4.2. Actions at beam connection

Ned= 1113.61 kN

4.3. Basic data, Beam-continuation connection



Beam section	IPE 400 h=400mm, b=180mm
	tw=8.6mm, tf=13.5mm, r=21.0m
Flange	tf=13.5mm, S 275, fy=275, fu=430N/mm²
Web	tw= 8.6mm, S 275, fy=275, fu=430N/mm²
Top-bottom plates	360x180x20.0 mm, S 235
	t1=20.0mm, S 235, fy=235, fu=360N/mm²
Side plates	662x231x10.0 mm, S 235
	t2=10.0mm, S 235, fy=235, fu=360N/mm²



Bolts	M20, Grade 8.8 Regular bolts
Bolt diameter	d = 20 mm
Diameter of holes	do = 22 mm
Nominal area	πd²/4 =πx20²/4= 314.2 mm²
Tensile stress area	As = 245.0 mm²
Bolt strength grade	8.8, fyb=640N/mm², fub=800N/mm²

4.4. Edge distances and spacing of bolts

(EN1993-1-8, §3.5, Tab.3.3)

Bolts on top-bottom plates

Minimum edge distances	e1=1.2do=1.2x22=27 mm
	e2=1.2do=1.2x22=27 mm
Maximum edge distances	e1=4t+40=4x13.5+40=95 mm
	e2=4t+40=4x13.5+40=95 mm
Minimum spacing of bolts	p1=2.2do=2.2x22=49 mm
	p2=2.4do=2.4x22=53 mm
Maximum spacing of bolts	p1=min(14t,200)=min(14x13.5,200)=190 mm
	p2=min(14t,200)=min(14x13.5,200)=190 mm

Number of Bolts	2x2 =4
Distance of plate edge to bolt line	e1=ex= 45 mm
Distance of plate edge to bolt line	e2=ey= 45 mm
Pitch between bolt rows	p1=px= 90 mm
Spacing between cross centers	p2=py= 90 mm

Bolts on side plates

Minimum edge distances	e1=1.2do=1.2x22=27 mm
	e2=1.2do=1.2x22=27 mm
Maximum edge distances	e1=4t+40=4x8.6+40=75 mm
	e2=4t+40=4x8.6+40=75 mm
Minimum spacing of bolts	p1=2.2do=2.2x22=49 mm
	p2=2.4do=2.4x22=53 mm
Maximum spacing of bolts	p1=min(14t,200)=min(14x8.6,200)=121 mm
	p2=min(14t,200)=min(14x8.6,200)=121 mm

Number of Bolts	2x2 =4
Distance of plate edge to bolt line	e1=ex= 116 mm
Distance of plate edge to bolt line	e2=ey= 66 mm
Pitch between bolt rows	p1=px= 100 mm
Spacing between cross centers	p2=py= 100 mm

4.5. Design resistance of individual bolts M20-8.8, Top-bottom plates (EC3-1-8 §3.6.1, Tab.3.4)

Bolts d=20mm, Grade 8.8, fu=430N/mm², fub =800N/mm², As=245.0mm², γM2=1.25
Shear resistance of bolts Fv,rd=n·αv·fub·As/γM2, (αv=0.60, n=1)
Shear plane of bolt through the threated portion
Fv,rd=$[10^{-3}]$x0.60x800x245.0/1.25=94.08 kN

Bearing resistance of bolts Fb,rd=k1·αb·fu·d·t/γM2
t=13.5mm, d=20mm, do=22mm, e1=45mm, e2=45mm, p1=90mm, p2=90mm
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[800/430, 1.0, 45/(3x22), 90/(3x22)-0.25]=0.68
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]
=min[2.8x45/22-1.7,1.4x90/22-1.7,2.5]=2.50
Fb,rd= $[10^{-3}]$x2.50x0.68x430x20x13.5/1.25=158.32 kN

4.6. Design resistance of individual bolts M20-8.8, Side plates (EC3-1-8 §3.6.1, Tab.3.4)

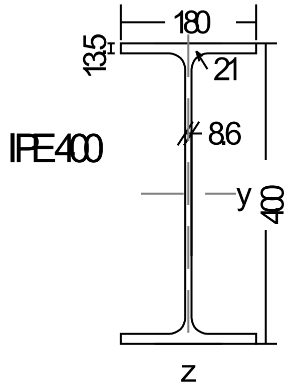
Bolts d=20mm, Grade 8.8, fu=430N/mm², fub =800N/mm², As=245.0mm², γM2=1.25
Shear resistance of bolts Fv,rd=n·αv·fub·As/γM2, (αv=0.60, n=2)
Shear plane of bolt through the threated portion
Fv,rd=$[10^{-3}]$x2x0.60x800x245.0/1.25=188.16 kN

Bearing resistance of bolts Fb,rd=k1·αb·fu·d·t/γM2
t=8.6mm, d=20mm, do=22mm, e1=75mm, e2=66mm, p1=100mm, p2=100mm
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[800/430, 1.0, 75/(3x22), 100/(3x22)-0.25]=1.00
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]
=min[2.8x66/22-1.7,1.4x100/22-1.7,2.5]=2.50
Fb,rd= $[10^{-3}]$x2.50x1.00x430x20x8.6/1.25=144.00 kN

4.7. Beam section

Cross-section properties

Cross-section IPE 400-S 275

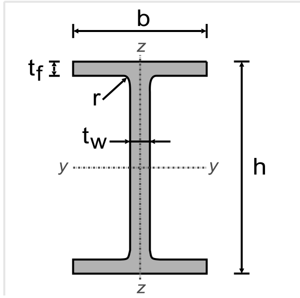


Dimensions of cross section

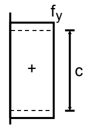
Depth of cross section	h=	400.00 mm
Width of cross section	b=	180.00 mm
Web depth	hw=	386.50 mm
Depth of straight portion of web	dw=	331.00 mm
Web thickness	tw=	8.60 mm
Flange thickness	tf=	13.50 mm
Radius of root fillet	r=	21.00 mm
Mass	=	66.30 Kg/m

Properties of cross section

Area	A= 8446 mm^2	
Second moment of area	Iy=231.30x10^6 mm^4	Iz=13.180x10^6 mm^4
Section modulus	Wy=1156.0x10^3 mm^3	Wz=146.40x10^3 mm^3
Plastic section modulus	Wpy=1307.0x10^3 mm^3	Wpz=229.00x10^3 mm^3
Radius of gyration	iy= 165.5 mm	iz= 39.5 mm
Shear area	Avz= 4269 mm^2	Avy= 4860 mm^2
Torsional constant	It= 0.511x10^6 mm^4	ip= 170 mm
Warping constant	Iw=490.05x10^9 mm^6	

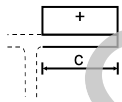


4.8. Classification of cross-sections, Compression Nc (EN1993-1-1, §5.5)



Web
c=400.0-2x13.5-2x21.0=331.0 mm, t=8.6 mm, c/t=331.0/8.6=38.49
S 275 , t= 8.6<= 40 mm, fy=275 N/mm², ε=(235/275)$^{0.5}$=0.92
38ε=38x0.92=34.96<c/t=38.49<=42ε=42x0.92=38.64
The web is class 3 (EN1993-1-1, Tab.5.2)

Flange
c=180.0/2-8.6/2-21.0=64.7 mm, t=13.5 mm, c/t=64.7/13.5=4.79
S 275 , t=13.5<= 40 mm, fy=275 N/mm², ε=(235/275)$^{0.5}$=0.92
c/t=4.79<=9ε=9x0.92=8.28
The flange is class 1 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 3, Compression Nc,ed

4.9. Ultimate Limit State, Verification for tension (EN1993-1-1, §6.2.3)
Nt.ed=1113.61 kN
A=8446mm², Anet=8446-4x22x13.5-2x22x8.6= 6880mm²
Tension Resistance Ntrd=min(A·fy/γM0, 0.9xAnet·fu/γM2)
=min([10^{-3}]x8446x275/1.00, [10^{-3}]x0.9x6880x430/1.25)=min(2322.65, 2129.92)) =2129.92kN
Nt,ed= 1113.61 kN < 2129.92 kN =Nt,rd=Ntrd, Is verified
Nt,ed/Nt,rd= 1113.61/2129.92= 0.523<1

4.10. Resistance of bolt connection, Top-bottom plates (EC3-1-8 §3.6)

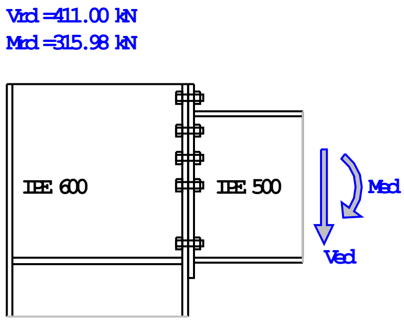
Flange: Cross-section area of plate, A=180x20.0= 3600mm², Anet=(180-2x22)x20.0= 2720mm²
Web : Cross-section area of plate, A=231x10.0= 2310mm², Anet=(231-2x22)x10.0= 1870mm²
Axial forces, Ned=1113.61x3600/(2x3600+2x2310)=274.11 kN
Tension Resistance Ntrd=min(A·fy/γM0, 0.9xAnet·fu/γM2)
=min([10^{-3}]x3600x235/1.00, [10^{-3}]x0.9x2720x360/1.25)=min(846.00, 705.02)) =705.02kN
Shear resistance of bolts, 4 Bolts, Fvrd= 4x94.08= 376.32 kN
Bearing resistance of bolts, 4 Bolts, Fbrd= 4x158.32= 633.28 kN
Resistance of plate in axial load, Nrd= min(705.02, 376.32, 633.28)= 376.32 kN
Ned= 274.11 kN < 376.32 kN =Nrd, Is verified
Ned/Nrd= 274.11/376.32= 0.728<1

4.11. Resistance of bolt connection, Side plates (EC3-1-8 §3.6)

Flange: Cross-section area of plate, A=180x20.0= 3600mm², Anet=(180-2x22)x20.0= 2720mm²
Web : Cross-section area of plate, A=231x10.0= 2310mm², Anet=(231-2x22)x10.0= 1870mm²
Axial forces, Ned=1113.61x2310/(2x3600+2x2310)=175.89 kN
Tension Resistance Ntrd=min(A·fy/γM0, 0.9xAnet·fu/γM2)
=min([10^{-3}]x2310x235/1.00, [10^{-3}]x0.9x1870x360/1.25)=min(542.85, 484.70)) =484.70kN
Shear resistance of bolts, 4 Bolts, Fvrd= 4x188.16= 752.64 kN
Bearing resistance of bolts, 4 Bolts, Fbrd= 4x144.00= 576.00 kN
Resistance of plate in axial load, Nrd= min(484.70, 752.64, 576.00)= 484.70 kN
Ned= 175.89 kN < 484.70 kN =Nrd, Is verified
Ned/Nrd= 175.89/484.70= 0.363<1

5. JOINTd-002

Eave connection
(EC3 EN1993-1-8:2005, §3, §6)



5.1. Basic data, Eave connection

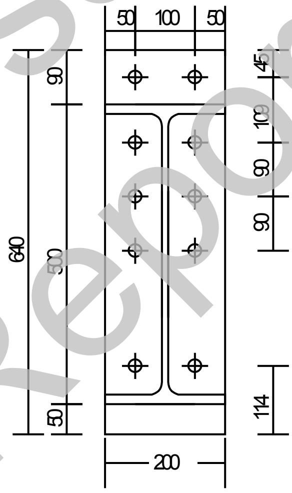
Steel sections

Steel section for column: IPE 600
Steel section for rafter: IPE 500

Design forces of connection, Eave connection

Maximum design values for actions
Ned = -125.0 kN
Ved = 100.0 kN
Med = 240.0 kNm

5.2. Connection data, Eave connection



Bolt connection data

End Plate	200x640x20 mm, S 235
Bolts	M20, Bolt strength grade 8.8
Number of Bolts	top 2x4=8
	bottom 2x1=2
Total number of bolts	=10
Diameter of holes	do = 22 mm
Shear plane of bolt	through the unthreated 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= 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

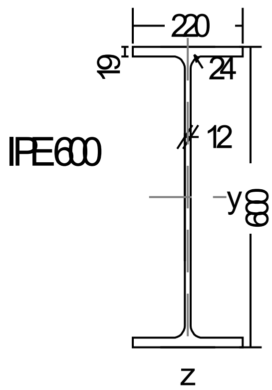
Compression stiffener at the bottom of haunch

Compression stiffener with thickness ts= 20.0 mm

5.3. Column section

Cross-section properties

Cross-section IPE 600-S 355

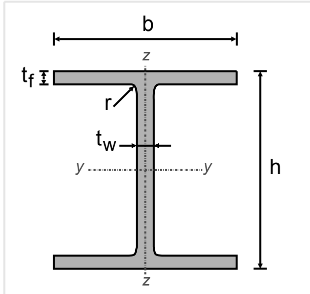


Dimensions of cross section

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

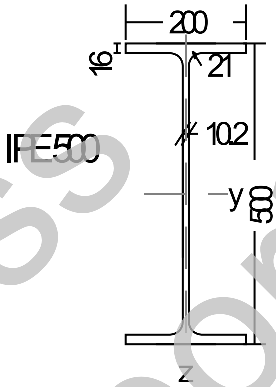
Area	A= 15600 mm^2	
Second moment of area	Iy=920.80x10^6 mm^4	Iz=33.870x10^6 mm^4
Section modulus	Wy=3069.0x10^3 mm^3	Wz=307.90x10^3 mm^3
Plastic section modulus	Wpy=3512.0x10^3 mm^3	Wpz=485.60x10^3 mm^3
Radius of gyration	iy= 243.0 mm	iz= 46.6 mm
Shear area	Avz= 8380 mm^2	Avy= 8360 mm^2
Torsional constant	It= 1.654x10^6 mm^4	ip= 247 mm
Warping constant	Iw=2845.5x10^9 mm^6	



5.4. Rafter section

Cross-section properties

Cross-section IPE 500-S 355

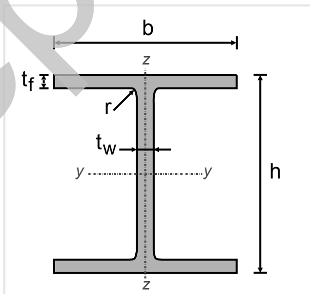


Dimensions of cross section

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=482.00x10^6 mm^4	Iz=21.420x10^6 mm^4
Section modulus	Wy=1928.0x10^3 mm^3	Wz=214.20x10^3 mm^3
Plastic section modulus	Wpy=2194.0x10^3 mm^3	Wpz=335.90x10^3 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= 0.893x10^6 mm^4	ip= 209 mm
Warping constant	Iw=1249.4x10^9 mm^6	



5.5. 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=(100-12.0-2x0.8x7x√2)/2= 36.1 mm
mx,y=45-0.8x11x√2= 32.6 mm
nx,x= emin <=1.25mx,x = min(50.0,1.25x36.1=45.1)= 45.1 mm
nx,y= emin <=1.25mx,y = min(50.0,1.25x32.6=40.7)= 40.8 mm
min(mx,x, mx,y)=min(36.1,32.6)=32.6mm, max(mx,x, mx,y)=max(36.1,32.6)=36.1mm
min(nx,x, nx,y)=min(45.1,40.8)=40.8mm, max(nx,x, nx,y)=max(45.1,40.8)=45.1mm

5.6. 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 =2πx32.6= 204.8 mm
=π·mx+w =πx32.6+100.0= 202.4 mm
=π·mx+2e=πx32.6+2x50.0= 202.4 mm
=4mx+1.25ex = 4x32.6+1.25x50.0=192.9 mm
=e+2mx+0.625ex = 50.0+2x32.6+0.625x50.0=146.4 mm
=0.5bp = 0.5x200= 100.0 mm
=0.5w+2mx+0.625ex=0.5x100.0+2x32.6+0.625x50.0= 146.4 mm
leff,1b=min(204.8, 202.4, 202.4, 192.9, 146.4, 100.0, 146.4)= 100.0 mm
leff,1b= 100.0 mm

Bolt next to tension flange alone

$l_{eff}=2\pi\cdot m_x \;=2\pi x32.6= 204.8$ mm

$=\alpha\cdot m =7.00x32.6=228.2$ mm $(\lambda1=\lambda2=m/(m+e)=0.39,\ \alpha=7.00)$ (EC3-1-8 Fig.6.11)

$l_{eff,2b}=\min(204.8,\ 228.2)= 204.8$ mm

$l_{eff,2b}= \ = 204.8$ mm

Bolt next to tension flange in a group

$l_{eff}=2\pi\cdot m_x \;=2\pi x32.6= 204.8$ mm

$=\alpha\cdot m =7.00x32.6=228.2$ mm $(\lambda1=\lambda2=m/(m+e)=0.39,\ \alpha=7.00)$

$=\pi m+p =\pi x32.6+90.0= 192.4$ mm

$=0.5p+\alpha\cdot m-(2m+0.625e)=0.5x90.0+7.0x32.6-(2x32.6+0.625x50.0)= 176.8$ mm

$l_{eff,3b}=\min(204.8,\ 228.2,\ 192.4,\ 176.8)= 176.8$ mm

$l_{eff,3b}= \ = 176.8$ mm

Inner Bolt-row in a group

$l_{eff}=2\pi\cdot m_x \;=2\pi x36.1= 226.8$ mm

$=4m+1.25e =4x36.1+1.25x50.0= 206.9$ mm

$=2p =2x90.0= 180.0$ mm

$=p = 90.0$ mm

$l_{eff,4b}=\min(226.8,\ 206.9,\ 180.0,\ 90.0)= 90.0$ mm

$l_{eff,4b}= \ = 90.0$ mm

5.7. 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

$M_{pl,1,rd}=M_{pl,2,rd}=0.25l_{eff}\cdot t_f^2\cdot f_y/\gamma_{M0}=[10^{-6}]x0.25x100.0x20.0^2x235/1.00=2.350$kNm

Mode 1 $\quad F_{t,1,rd}= 4M_{pl,1rd}/m =[10^3]x4x2.350/32.6 = 288$ kN

Mode 2 $\quad F_{t,2,rd}= (2M_{pl,2rd}+n\Sigma F_{t,rd})/(m+n)=([10^3]x2x2.350+40.8x2x141)/(32.6+40.8)=221$kN

Mode 3 $\quad F_{t,3,rd}= \Sigma F_{t,rd} =2x141 = 282$ kN

$F_{t,rd}= \min(288,\ 221,\ 282)= \ 221$ kN

Bolt next to tension flange alone

$M_{pl,1,rd}=M_{pl,2,rd}=0.25l_{eff}\cdot t_f^2\cdot f_y/\gamma_{M0}=[10^{-6}]x0.25x204.8x20.0^2x235/1.00=4.813$kNm

Mode 1 $\quad F_{t,1,rd}= 4M_{pl,1rd}/m =[10^3]x4x4.813/32.6 = 591$ kN

Mode 2 $\quad F_{t,2,rd}= (2M_{pl,2rd}+n\Sigma F_{t,rd})/(m+n)=([10^3]x2x4.813+40.8x2x141)/(32.6+40.8)=288$kN

Mode 3 $\quad F_{t,3,rd}= \Sigma F_{t,rd} =2x141 = 282$ kN

$F_{t,rd}= \min(591,\ 288,\ 282)= \ 282$ kN

Bolt next to tension flange in a group

$M_{pl,1,rd}=M_{pl,2,rd}=0.25l_{eff}\cdot t_f^2\cdot f_y/\gamma_{M0}=[10^{-6}]x0.25x176.8x20.0^2x235/1.00=4.155$kNm

Mode 1 $\quad F_{t,1,rd}= 4M_{pl,1rd}/m =[10^3]x4x4.155/32.6 = 510$ kN

Mode 2 $\quad F_{t,2,rd}= (2M_{pl,2rd}+n\Sigma F_{t,rd})/(m+n)=([10^3]x2x4.155+40.8x2x141)/(32.6+40.8)=270$kN

Mode 3 $\quad F_{t,3,rd}= \Sigma F_{t,rd} =2x141 = 282$ kN

$F_{t,rd}= \min(510,\ 270,\ 282)= \ 270$ kN

Inner Bolt-row in a group

$M_{pl,1,rd}=M_{pl,2,rd}=0.25l_{eff}\cdot t_f^2\cdot f_y/\gamma_{M0}=[10^{-6}]x0.25x90.0x20.0^2x235/1.00=2.115$kNm

Mode 1 $\quad F_{t,1,rd}= 4M_{pl,1rd}/m =[10^3]x4x2.115/36.1 = 234$ kN

Mode 2 $\quad F_{t,2,rd}= (2M_{pl,2rd}+n\Sigma F_{t,rd})/(m+n)=([10^3]x2x2.115+45.1x2x141)/(36.1+45.1)=209$kN

Mode 3 $\quad F_{t,3,rd}= \Sigma F_{t,rd} =2x141 = 282$ kN

$F_{t,rd}= \min(234,\ 209,\ 282)= \ 209$ kN

5.8. Rafter flange and web in compression (Eave connection)

(EC3-1-8 §6.2.6.7)

$F_{c,fb,rd}= M_{c,rd}/(h-t_f),\quad M_{c,rd}= W_{el,y}\cdot f_y/\gamma_{M0}$

$W_{el,y}=(200x16.0x484.0^2+10.2x468.0^3/6)/500=1847.7x10^3$ mm³

$M_{c,rd}= [10^{-6}]x1847.7x10^3x355/1.00=656$kNm, $F_{c,fb,rd}=[10^3]x656/484.0= 1355$kN

$F_{c,fb,rd,max}=b\cdot t\cdot f_y/\gamma_{M0}= [10^{-3}]x200.0x16.0x235/1.00= 752$ kN $(h<=600mm)$

$F_{c,fb,rd}= \min(1355,\ 752)= 752$ kN

5.9. 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(176.8, 90.0) =90.0 mm
Ft,wb,rd = [10^{-3}]x90.0x10.2x355/1.00= 326 kN

min Ft,rd=min(221, 282, 270, 209, 326)= 209 kN

5.10. 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=45-0.8x11x√2= 32.6 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(50.0,1.25x32.6=40.7)= 40.8 mm
min(mx,x, mx,y)=min(24.8,32.6)=24.8mm, max(mx,x, mx,y)=max(24.8,32.6)=32.6mm
min(nx,x, nx,y)=min(31.0,40.8)=31.0mm, max(nx,x, nx,y)=max(31.0,40.8)=40.8mm

5.11. 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

5.12. 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·tf²·fy/γM0=[10^{-6}]x0.25x95.0x19.0²x355/1.00=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+31.0x2x141)/(24.8+31.0)=266kN
Mode 3 Ft,3,rd= ΣFt,rd =2x141 = 282 kN
Ft,rd= min(491, 266, 282)= 266 kN

Inner Bolt-row in a group
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x90.0x19.0²x355/1.00=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+31.0x2x141)/(24.8+31.0)=260kN
Mode 3 Ft,3,rd= ΣFt,rd =2x141 = 282 kN
Ft,rd= min(465, 260, 282)= 260 kN

5.13. Column-web in transverse 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

5.14. 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/mm², 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 mm² (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=√(5853x10³/6539) =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=752 kN
Compression stiffener, Is verified

5.15. 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

<u>End-plate in bending</u> (EC3-1-8 §6.2.4.5
Force distribution in bolt rows
Bolt-row 1, hr= 537.0 mm, Ft,rd= 221 kN
Bolt-row 2, hr= 428.0 mm, Ft,rd= 270 kN
Bolt-row 3, hr= 338.0 mm, Ft,rd= 209 kN
Bolt-row 4, hr= 248.0 mm, Ft,rd= 209 kN
Fc,ed=ΣFt,rd=221 + 270 + 209 + 209= 909 kN

<u>End-plate in bending</u> (EC3-1-8 §6.2.4.4
Force distribution in bolt rows
Bolt-row 1, hr= 537.0 mm, Ft,rd= 266 kN
Bolt-row 2, hr= 428.0 mm, Ft,rd= 260 kN
Bolt-row 3, hr= 338.0 mm, Ft,rd= 260 kN
Bolt-row 4, hr= 248.0 mm, Ft,rd= 260 kN
Fc,ed=ΣFt,rd=266 + 260 + 260 + 260= 1046 kN

<u>Rafter web in tension</u> (EC3-1-8 §6.2.6.8)
Ft,wb,rd = 326 kN
<u>Rafter flange and web in compression</u> (EC3-1-8 §6.2.4.7
Fc,fb,rd= 752 kN
Ft,rd<=Ft,wb,rd=326 kN, Fc,ed=ΣFt,rd<=Fc,fb,rd=752 kN
Fc,ed=ΣFt,rd<=Fc,wc,rd=1537 kN

<u>Force distribution in bolt rows</u> (EC3-1-8 §6.2.7.2.(7))
Bolt-row 1, hr= 537.0 mm, Ft,rd= 221 kN
Bolt-row 2, hr= 428.0 mm, Ft,rd= 260 kN
Bolt-row 3, hr= 338.0 mm, Ft,rd= 209 kN
Bolt-row 4, hr= 248.0 mm, Ft,rd= 62 kN
Fc,ed=ΣFt,rd=221 + 260 + 209 + 62= 752 kN

<u>Moment resistance of connection</u> (EN1993-1-8, §6.2.7.2(10)

Mj,rd =[10^{-3}]x[221x537.0+260x428.0+209x338.0+62x248.0]
Mj,rd = 316 kNm
Med= 240.0 kNm < 316.0 kNm =Mj,rd, Is verified

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

<u>Shear resistance of bolts</u>
Fv,rd=αv·fub·(πxd²/4)/γM2=[10^{-3}]x0.60x800x(πx20²/4)/1.25=121 kN
Shear plane of bolt:through the unthreated portion

Bearing resistance of bolts

Fb,rd=k1·αb·fu·d·t/γM2

End-Plate

t=20.0mm, d=20mm, do=22mm, e1=50mm, e2=50mm, p1=90mm, fub=800kN/mm², fu=360kN/mm²,
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[800/360, 1.0, 50/(3x22), 90/(3x22)-0.25]=0.76
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x50/22-1.7,1.4x100/22-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= [10^{-3}]x2.50x0.76x360x20x20.0/1.25=218 kN

Column-Side

t=19.0mm, d=20mm, do=22mm, e1=50mm, e2=50mm, p1=90mm, fub=800kN/mm², fu=510kN/mm²,
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[800/510, 1.0, 50/(3x22), 90/(3x22)-0.25]=0.76
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x50/22-1.7,1.4x100/22-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= [10^{-3}]x2.50x0.76x510x20x19.0/1.25=294 kN

Design resistance of one bolt in shear =min(121, 218, 294)=121 kN

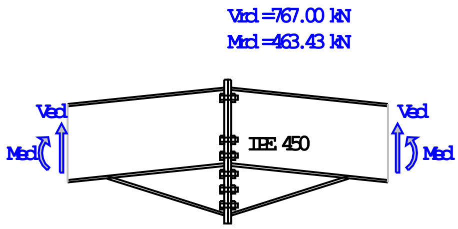
Bending moment and shear (EN1993-1-8, §3.6.1 Tab.3.4)

Maximum tension force in bolts
Ft,ed=260/2=130 kN
Reduction of shear resistance due to bending
ρ=1-Ft,ed/1.40Ft,rd=1-130/(1.40x141)=0.34
Shear acting together with bending moment for all the bolts
Vrd=10x0.34x121 =411 kN

Ved= 100 kN < 411 kN =Vrd, Is verified

6. JOINTd-003

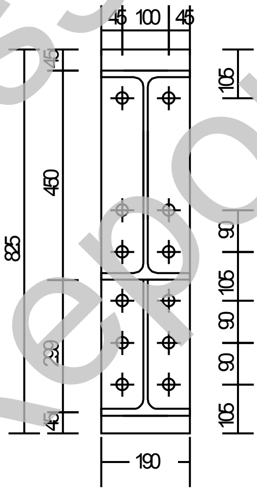
Apex connection
(EC3 EN1993-1-8:2005, §3, §6)



6.1. Basic data, Apex connection

Steel sections
Steel section for rafter: IPE 450
Design forces of connection, Apex connection
Maximum design values for actions
Ned = -125.0 kN
Ved = 40.0 kN
Med = 320.0 kNm

6.2. Connection data, Apex connection



Bolt connection data

End Plate	190x825x20 mm, S 235
Bolts	M24, Bolt strength grade 8.8
Number of Bolts	top 2x1=2
	bottom 2x5=10
Total number of bolts	=12
Diameter of holes	do = 26 mm
Shear plane of bolt	through the threated portion

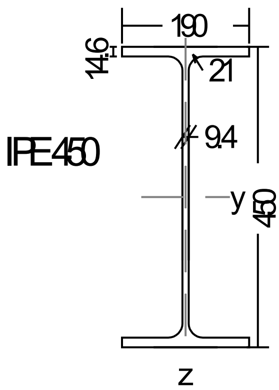
Edge distances and spacing of bolts

Distance of plate edge to bolt line	e1=e2=ex= 45 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=0.55x14.6= 8 mm
Web to end-plate weld	aw>= 0.55tw=0.55x 9.4= 6 mm

6.3. Rafter section

Cross-section properties

Cross-section IPE 450-S 355

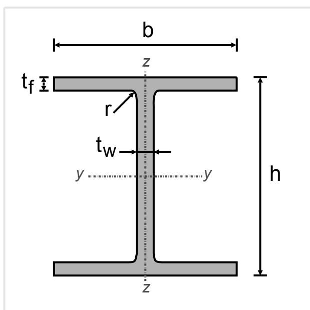


Dimensions of cross section

Depth of cross section	h=	450.00 mm
Width of cross section	b=	190.00 mm
Web depth	hw=	435.40 mm
Depth of straight portion of web	dw=	378.80 mm
Web thickness	tw=	9.40 mm
Flange thickness	tf=	14.60 mm
Radius of root fillet	r=	21.00 mm
Mass	=	77.60 Kg/m

Properties of cross section

Area	A= 9882 mm^2	
Second moment of area	Iy=337.40x10^6 mm^4	Iz=16.760x10^6 mm^4
Section modulus	Wy=1500.0x10^3 mm^3	Wz=176.40x10^3 mm^3
Plastic section modulus	Wpy=1702.0x10^3 mm^3	Wpz=276.40x10^3 mm^3
Radius of gyration	iy= 184.8 mm	iz= 41.2 mm
Shear area	Avz= 5084 mm^2	Avy= 5548 mm^2
Torsional constant	It= 0.669x10^6 mm^4	ip= 189 mm
Warping constant	Iw=791.01x10^9 mm^6	



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

```
e=ex=45 mm, emin=45 mm
mx,x=(100-9.4-2x0.8x6x√2)/2= 38.5 mm
mx,y=45-0.8x8x√2= 35.9 mm
nx,x= emin <=1.25mx,x = min(45.0,1.25x38.5=48.1)= 45.0 mm
nx,y= emin <=1.25mx,y = min(45.0,1.25x35.9=44.9)= 44.9 mm
min(mx,x, mx,y)=min(38.5,35.9)=35.9mm, max(mx,x, mx,y)=max(38.5,35.9)=38.5mm
min(nx,x, nx,y)=min(45.0,44.9)=44.9mm, max(nx,x, nx,y)=max(45.0,44.9)=45.0mm
```

6.5. 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  =2πx35.9= 225.6 mm
      =π·mx+w =πx35.9+100.0= 212.8 mm
      =π·mx+2e=πx35.9+2x45.0= 202.8 mm
      =4mx+1.25ex = 4x35.9+1.25x45.0=199.9 mm
      =e+2mx+0.625ex = 45.0+2x35.9+0.625x45.0=144.9 mm
      =0.5bp = 0.5x190= 95.0 mm
      =0.5w+2mx+0.625ex=0.5x100.0+2x35.9+0.625x45.0= 149.9 mm
 leff,1b=min(225.6, 212.8, 202.8, 199.9, 144.9, 95.0, 149.9)= 95.0 mm
 leff,1b= 95.0 mm
```

Bolt next to tension flange alone

```
  leff=2π·mx  =2πx35.9= 225.6 mm
      =α·m =6.28x35.9=225.6 mm (λ1=λ2=m/(m+e)=0.44, α=6.28)        (EC3-1-8 Fig.6.11)
 leff,2b=min(225.6, 225.6)= 225.6 mm
 leff,2b=  = 225.6 mm
```

Bolt next to tension flange in a group

```
  leff=2π·mx  =2πx35.9= 225.6 mm
      =α·m =6.28x35.9=225.6 mm (λ1=λ2=m/(m+e)=0.44, α=6.28)
      =πm+p =πx35.9+90.0= 202.8 mm
      =0.5p+α·m-(2m+0.625e)=0.5x90.0+6.3x35.9-(2x35.9+0.625x45.0)= 170.6 mm
 leff,3b=min(225.6, 225.6, 202.8, 170.6)= 170.6 mm
 leff,3b=  = 170.6 mm
```

Inner Bolt-row in a group

```
  leff=2π·mx  =2πx38.5= 241.9 mm
      =4m+1.25e =4x38.5+1.25x45.0= 210.3 mm
      =2p =2x90.0= 180.0 mm
      =p = 90.0 mm
 leff,4b=min(241.9, 210.3, 180.0, 90.0)= 90.0 mm
 leff,4b=  = 90.0 mm
```

6.6. 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]x0.25x95.0x20.0²x235/1.00=2.233kNm
Mode 1   Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x2.233/35.9 = 249 kN
Mode 2   Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x2.233+44.9x2x203)/(35.9+44.9)=281kN
Mode 3   Ft,3,rd= ΣFt,rd =2x203 = 406 kN
Ft,rd= min(249, 281, 406)=  249 kN
```

Bolt next to tension flange alone

```
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^-6]x0.25x225.6x20.0²x235/1.00=5.302kNm
Mode 1   Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x5.302/35.9 = 591 kN
Mode 2   Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x5.302+44.9x2x203)/(35.9+44.9)=357kN
Mode 3   Ft,3,rd= ΣFt,rd =2x203 = 406 kN
Ft,rd= min(591, 357, 406)=  357 kN
```

Bolt next to tension flange in a group

Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x170.6x20.0²x235/1.00=4.009kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x4.009/35.9 = 447 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x4.009+44.9x2x203)/(35.9+44.9)=325kN
Mode 3 Ft,3,rd= ΣFt,rd =2x203 = 406 kN
Ft,rd= min(447, 325, 406)= 325 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}]x4x2.115/38.5 = 220 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x2.115+45.0x2x203)/(38.5+45.0)=269kN
Mode 3 Ft,3,rd= ΣFt,rd =2x203 = 406 kN
Ft,rd= min(220, 269, 406)= 220 kN

6.7. 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=(190x14.6x720.0²+9.4x705.4³/6)/735=2706.2x10^{3} mm³
Mc,rd= [10^{-6}]x2706.2x10^{3}x355/1.00=961kNm, Fc,fb,rd=[10^{3}]x961/720.0= 1334kN
Fc,fb,rd,max=(1/0.8)b·t·fy/γM0= (1/0.8)x[10^{-3}]x190.0x14.6x235/1.00= 815 kN (h>600mm)
Fc,fb,rd= min(1334, 815)= 815 kN

6.8. 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(170.6, 90.0) =90.0 mm
Ft,wb,rd = [10^{-3}]x90.0x9.4x355/1.00= 300 kN

min Ft,rd=min(249, 357, 325, 220, 300)= 220 kN

6.9. 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= 667.7 mm, Ft,rd= 325 kN
Bolt-row 2, hr= 577.7 mm, Ft,rd= 220 kN
Bolt-row 3, hr= 487.7 mm, Ft,rd= 220 kN
Bolt-row 4, hr= 383.1 mm, Ft,rd= 220 kN
Bolt-row 5, hr= 293.1 mm, Ft,rd= 325 kN
Bolt-row 6, hr= 52.3 mm, Ft,rd= 220 kN
Fc,ed=ΣFt,rd=325 + 220 + 220 + 220 + 325 + 220= 1530 kN

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

Ft,wb,rd = 300 kN

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

Fc,fb,rd= 815 kN
Ft,rd<=Ft,wb,rd=300 kN, Fc,ed=ΣFt,rd<=Fc,fb,rd=815 kN

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

Bolt-row 1, hr= 667.7 mm, Ft,rd= 300 kN
Bolt-row 2, hr= 577.7 mm, Ft,rd= 220 kN
Bolt-row 3, hr= 487.7 mm, Ft,rd= 220 kN
Bolt-row 4, hr= 383.1 mm, Ft,rd= 75 kN
Bolt-row 5, hr= 293.1 mm, Ft,rd= 0 kN
Bolt-row 6, hr= 52.3 mm, Ft,rd= 0 kN
Fc,ed=ΣFt,rd=300 + 220 + 220 + 75 + 0 + 0= 815 kN

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

$Mj,rd =[10^{-3}]x[300x667.7+220x577.7+220x487.7+75x383.1+0x293.1+0x52.3]$
Mj,rd = 463 kNm
Med= 320.0 kNm < 463.4 kNm =Mj,rd, Is verified

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

Shear resistance of bolts
$Fv,rd=\alpha\nu \cdot fub \cdot As/\gamma M2=[10^{-3}]x0.60x800x353.0/1.25=136$ kN
Shear plane of bolt:through the threated portion

Bearing resistance of bolts
$Fb,rd=k1 \cdot \alpha b \cdot fu \cdot d \cdot t/\gamma M2$
t=20.0mm, d=24mm, do=26mm, e1=45mm, e2=45mm, p1=90mm, fub=800kN/mm², fu=360kN/mm²,
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[800/360, 1.0, 45/(3x26), 90/(3x26)-0.25]=0.58
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x45/26-1.7,1.4x100/26-1.7,2.5]=2.50
$Fb,rd=k1 \cdot \alpha b \cdot fu \cdot d \cdot t/\gamma M2= [10^{-3}]x2.50x0.58x360x24x20.0/1.25=199$ kN

Design resistance of one bolt in shear =min(136, 199)=136 kN

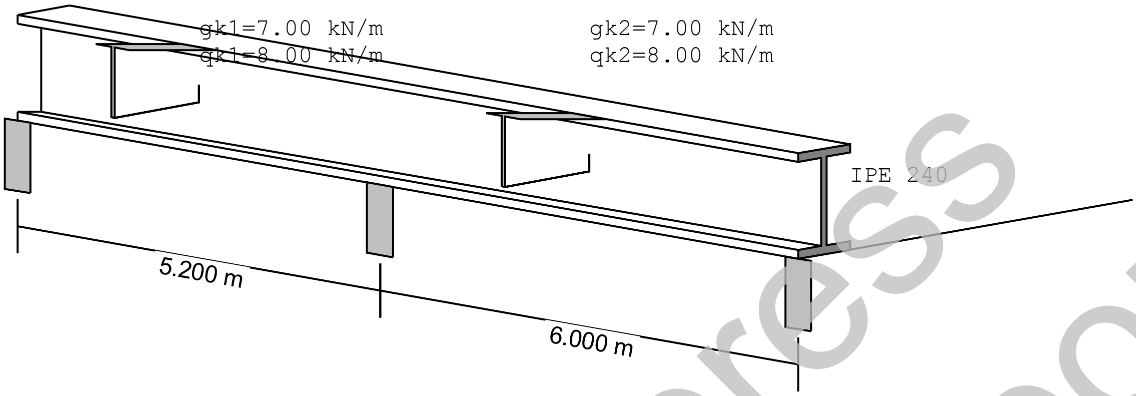
Bending moment and shear (EN1993-1-8, §3.6.1 Tab.3.4)
Maximum tension force in bolts
Ft,ed=300/2=150 kN
Reduction of shear resistance due to bending
ρ=1-Ft,ed/1.40Ft,rd=1-150/(1.40x203)=0.47
Shear acting together with bending moment for all the bolts
Vrd=12x0.47x136 =767 kN

Ved= 40 kN < 767 kN =Vrd, Is verified

7. BEAM-002

Design of Beams,Continuous beam of two spans
(EC3 EN1993-1-1:2005)

Span lengths L1=5.200m, L2=6.000m
Conditions of lateral restraining: Lat. restr. at L/2



7.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
EN1991-1-1:2002, Eurocode 1-1 Actions on structures
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

7.2. 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)
γ_G= 1.35, γ_Q= 1.50

Partial factors for materials (EN1993-1-1, §6.1)
γ_{M0}= 1.00, γ_{M1}= 1.00, γ_{M2}= 1.25

7.3. Loading (EN1991-1-1)

Load on beam

Dead load	Gk1= Span-1 : 7.00 kN/m, Span-2 : 7.00 kN/m
Beam weight	Gk2= Span-1 : 0.30 kN/m, Span-2 : 0.30 kN/m
Permanent load	Gk =Gk1+Gk2= Span-1 : 7.30 kN/m, Span-2 : 7.30 kN/m
Variable load	Qk = Span-1 : 8.00 kN/m, Span-2 : 8.00 kN/m

7.4. Design values of Actions, Load combinations

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

Span-1 γG·Gk+γQ·Qk = 1.35x7.30+1.50x8.00 =21.85kN/m, $ql^2/8$=73.87kNm
Span-2 γG·Gk+γQ·Qk = 1.35x7.30+1.50x8.00 =21.85kN/m, $ql^2/8$=98.35kNm

Design actions, Ultimate Limit State

Myed,1= 50.20kNm, Myed,s=-86.98kNm, Myed,2= 68.89kNm,
Vzed,1A=46.84kN, Vzed,1B=-73.55kN, Vzed,2A=80.06kN, Vzed,2B=-54.87kN

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

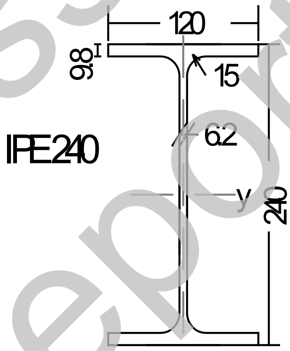
Span-1 Gk+Qk = 7.30+8.00= 15.30kN/m
Span-2 Gk+Qk = 7.30+8.00= 15.30kN/m

Design actions, Serviceability Limit State (SLS)

Myed,1= 33.00kNm, Myed,s=-60.89kNm, Myed,2= 33.00kNm,
Vzed,1A=31.78kN, Vzed,1B=-51.49kN, Vzed,2A=56.05kN, Vzed,2B=-37.84kN

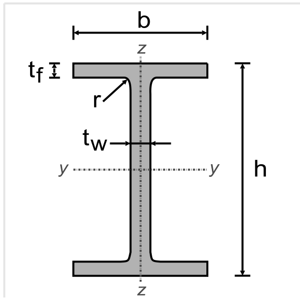
7.5. Cross-section properties

Cross-section IPE 240-S 355



Dimensions of cross section

Depth of cross section	h=	240.00 mm
Width of cross section	b=	120.00 mm
Web depth	hw=	230.20 mm
Depth of straight portion of web	dw=	190.40 mm
Web thickness	tw=	6.20 mm
Flange thickness	tf=	9.80 mm
Radius of root fillet	r=	15.00 mm
Mass	=	30.70 Kg/m



Properties of cross section

Area	A= 3912 mm^2			
Second moment of area	Iy=38.920x10^{6} mm^4		Iz= 2.836x10^{6} mm^4	
Section modulus	Wy=324.30x10^{3} mm^3		Wz=47.270x10^{3} mm^3	
Plastic section modulus	Wpy=366.60x10^{3} mm^3		Wpz=73.920x10^{3} mm^3	
Radius of gyration	iy= 99.7 mm		iz= 26.9 mm	
Shear area	Avz= 1915 mm^2		Avy= 2352 mm^2	
Torsional constant	It= 0.129x10^{6} mm^4		ip= 103 mm	
Warping constant	Iw=37.391x10^{9} mm^6			

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

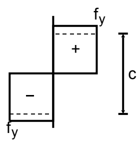
Beam deflections

Loading G+Q: w1=10.1mm=L/515, w2=10.1mm=L/515, L/515<L/200
Loading Q: w1= 6.6mm=L/788, w2= 6.6mm=L/788, L/788<L/360

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

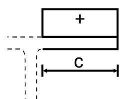
7.7. Classification of cross-sections, Bending My (EN1993-1-1, §5.5)

Web



c=240.0-2x9.8-2x15.0=190.4 mm, t=6.2 mm, c/t=190.4/6.2=30.71
S 355 , t= 6.2<= 40 mm, fy=355 N/mm^2, ε=(235/355)$^{0.5}$=0.81
c/t=30.71<=72ε=72x0.81=58.32
The web is class 1 (EN1993-1-1, Tab.5.2)

Flange



c=120.0/2-6.2/2-15.0=41.9 mm, t=9.8 mm, c/t=41.9/9.8=4.28
S 355 , t= 9.8<= 40 mm, fy=355 N/mm^2, ε=(235/355)$^{0.5}$=0.81
c/t=4.28<=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

7.8. Resistance of cross-section, Beam section (EN1993-1-1, §6.2)

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

My.ed= 86.98 kNm

Bending Resistance $M_{ply,rd}=W_{ply}\cdot f_y/\gamma_{M0}=[10^{-6}]\times366.60\times10^{3}\times355/1.00=130.14\,kNm$

My,ed= 86.98 kNm < 130.14 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 86.98/130.14= 0.668<1

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

Vz.ed= 80.06 kN

$A_v=A-2b\cdot t_f+(t_w+2r)t_f=3912-2\times120.0\times9.8+(6.2+2\times15.0)\times9.8=1915mm^2$ (EC3 §6.2.6.3)

$A_v=1915mm^2>\eta\cdot h_w\cdot t_w=1.00\times(240.0-2\times9.8)\times6.2=1.00\times230.2\times6.2=1427mm^2$

Plastic Shear Resistance $V_{pl,z,rd}=A_v(f_y/\sqrt{3})/\gamma_{M0}=[10^{-3}]\times1915\times(355/1.73)/1.00=392.45kN$

Vz,ed= 80.06 kN < 392.45 kN =Vz,rd=Vpl,z,rd, Is verified

Vz,ed/Vz,rd= 80.06/392.45= 0.204<1

$h_w/t_w=(240.0-2\times9.8)/6.2=230.2/6.2=37.13\le72\times0.81/1.00=72\varepsilon/\eta=58.32\ (\eta=1.00)$

S 355 , t= 6.2<= 40 mm, $f_y=355\,N/mm^2$, $\varepsilon=(235/355)^{0.5}=0.81$

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

Ultimate Limit State, Bending and shear (EN1993-1-1, §6.2.8

Vz,ed=80.06kN, My,ed=86.98kNm

Vz,ed=80.06=kN <= Vplz,rd/2=392.45/2=196.23kN (EC3 §6.2.8(2))

Effect of shear on moment resistance is neglected

7.9. Lateral torsional buckling, (ULS) (Beam span) (EN1993-1-1, §6.3.2)

My,ed=68.89 kN, L=5.200m, Lcr,y=3.640m, Lcr,z=5.200m, Lcr,lt=2.600m

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

$M_{cr}=C1\cdot[\pi^2EI_z/(kL)^2]\{\sqrt{[(k_z/k_w)^2(I_w/I_z)+(kL)^2GI_t/(\pi^2EI_z)+(C2\cdot z_g-C3\cdot z_j)^2]}-(C2\cdot z_g-C3\cdot z_j)\}$

Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006

$\mu=M_o/M=qL^2/8M=18.5/30.4=0.61$, $\psi=M_b/M_a=0.0/30.4=0.00$, C1=1.119, C2=0.222

$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1\times10^{4}\,N/mm^2$

k·L=2600mm, zg=h/2=240/2=120mm, zj=0mm (EN1993:2002 Eq.C.11)

ky=1.0, kz=1.0, kw=1.0, C1=1.119, C2=0.222, C3=0.000

$M_{cr}=[10^{-6}]1.119\times[\pi^2\times2.1\times10^{5}\times2.836\times10^{6}/2600^2]$

$\times\{[(1.0/1.0)^2\times(37.391\times10^{9}/2.836\times10^{6})$

$+2600^2\times8.1\times10^{4}\times0.129\times10^{6}/(\pi^2\times2.1\times10^{5}\times2.836\times10^{6})$

$+(0.222\times120)^2]^{0.5}-(0.222\times120)\}=130.6\,kNm$

$\bar{\lambda}_{,lt}=\sqrt{(W_{pl,y}\cdot f_y/M_{cr})}=\sqrt{\{[10^{-6}]\times366.60\times10^{3}\times355/130.6\}}=0.998$ (EC3 Eq.6.56)

h/b=240/120=2.00<=2.00 buckling curve:b

imperfection factor: $\alpha_{,lt}=0.34$, $\beta=0.75$, $\chi_{,lt}=0.701$ (T.6.3, T.6.5, Fig.6.4)

$\Phi_{,lt}=0.5[1+\alpha_{,lt}(\bar{\lambda}_{,lt}-\bar{\lambda}_{,lto})+\beta\bar{\lambda}_{,lt}^2]=0.5\times[1+0.34\times(0.998-0.40)+0.75\times0.998^2]=0.975$

$\chi_{,lt}=1/[\Phi_{,lt}+\sqrt{(\Phi_{,lt}^2-\beta\bar{\lambda}_{,lt}^2)}]=1/[0.975+\sqrt{(0.975^2-0.75\times0.975^2)}]=0.701$

Reduction factor $\chi_{,lt}=1/[\Phi_{,lt}+\sqrt{(\Phi_{,lt}^2-\beta\bar{\lambda}_{,lt}^2)}]$, $\chi_{,lt}\le1.0$, $1/\bar{\lambda}_{,lt}^2$, $\chi_{,lt}=0.701$ (Eq.6.57)

$\chi_{,lt,mod}=\chi_{,lt}/f$, $\chi_{,lt,mod}\le1$, $\chi_{,lt,mod}\le1/\bar{\lambda}_{,lt}^2=1/0.998^2=1.00$ (EC3 §6.3.2.3(2),Eq.6.58)

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

$f=1-0.5(1-k_c)[1-2.0(\bar{\lambda}_{,lt}-0.8)^2]=1-0.5\times(1-0.752)[1-2.0\times(0.998-0.8)^2]=0.886$, $f\le1.0$

$\chi_{,lt,mod}=\chi_{,lt}/f=0.701/0.886=0.791$, $\chi_{,lt,mod}\le1.0$, $\chi_{,lt,mod}\le1.00$, $\chi_{,lt,mod}=0.791$

$M_{b,rd}=\chi_{,lt}\cdot W_{pl,y}\cdot f_y/\gamma_{M1}=0.791\times[10^{-6}]\times366.60\times10^{3}\times355/1.00=102.94kNm$ (EC3 Eq.6.55)

My,ed= 68.89 kNm < 102.94 kNm =Mb,rd, Is verified

My,ed/Mb,rd= 68.89/102.94= 0.669<1

7.10. Lateral torsional buckling, (ULS) (Beam support) (EN1993-1-1, §6.3.2)

My,ed=86.98 kN, L=5.200m, Lcr,y=3.640m, Lcr,z=5.200m, Lcr,lt=2.600m

```
Check lateral-torsional buckling at support region, Lc= 2.600m

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)}
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
μ=Mo/M=qL²/8M=-18.5/87.0=-0.21, ψ=Mb/Ma=30.4/-87.0=-0.35, C1=2.864, C2=0.168
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^4 N/mm²
k·L=2600mm,  zg=h/2=240/2=120mm, zj=0mm                              (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=2.864, C2=0.168, C3=0.000
Mcr=[10^-6]2.864x[π²x2.1x10^5x2.836x10^6/2600²]
   x{ [(1.0/1.0)²x(37.391x10^9/2.836x10^6)
   +2600²x8.1x10^4x0.129x10^6/(π²x2.1x10^5x2.836x10^6)
   +(0.168x120)²]^0.5-(0.168x120) }= 348.0 kNm

λ̄,lt=√(Wpl,y·fy/Mcr)=√{[10^-6]x366.60x10^3x355/348.0}=0.612                   (EC3 Eq.6.56)
h/b=240/120=2.00<=2.00 buckling curve:b
imperfection factor:α,lt=0.34, β=0.75, χ,lt=0.912                   (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt(λ̄,lt-λ̄,lto)+βλ̄,lt²]=0.5x[1+0.34x(0.612-0.40)+0.75x0.612²]=0.676
χ,lt=1/[Φ,lt+√(Φ,lt²-βλ̄,lt²)]=1/[0.676+√(0.676²-0.75x0.676²)]=0.912
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt²-βλ̄,lt²)], χ,lt<=1.0, 1/λ̄,lt², χ,lt=0.912         (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/λ̄,lt²=1/0.612²=2.67         (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(0.612-0.8)²]=0.885, f<=1.0
χ,lt,mod=χ,lt/f=0.912/0.885=1.031, χ,lt,mod<=1.0, χ,lt,mod<=2.67,  χ,lt,mod=1.000

Mb,rd=χ,lt·Wpl,y·fy/γM1= 1.000x[10^-6]x366.60x10^3x355/1.00=130.14kNm          (EC3 Eq.6.55)
My,ed= 86.98 kNm < 130.14 kNm =Mb,rd,  Is verified
My,ed/Mb,rd= 86.98/130.14= 0.668<1
```

7.11. Lateral torsional buckling, (ULS) (Beam span) (EN1993-1-1, §6.3.2)

My,ed=68.89 kN, L=6.000m, Lcr,y=4.200m, Lcr,z=6.000m, Lcr,lt=3.000m

```
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)}
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
μ=Mo/M=qL²/8M=24.6/54.9=0.45, ψ=Mb/Ma=0.0/54.9=0.00, C1=1.166, C2=0.192
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^4 N/mm²
k·L=3000mm,  zg=h/2=240/2=120mm, zj=0mm                              (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=1.166, C2=0.192, C3=0.000
Mcr=[10^-6]1.166x[π²x2.1x10^5x2.836x10^6/3000²]
   x{ [(1.0/1.0)²x(37.391x10^9/2.836x10^6)
   +3000²x8.1x10^4x0.129x10^6/(π²x2.1x10^5x2.836x10^6)
   +(0.192x120)²]^0.5-(0.192x120) }= 113.5 kNm

λ̄,lt=√(Wpl,y·fy/Mcr)=√{[10^-6]x366.60x10^3x355/113.5}=1.071                   (EC3 Eq.6.56)
h/b=240/120=2.00<=2.00 buckling curve:b
imperfection factor:α,lt=0.34, β=0.75, χ,lt=0.656                   (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt(λ̄,lt-λ̄,lto)+βλ̄,lt²]=0.5x[1+0.34x(1.071-0.40)+0.75x1.071²]=1.044
χ,lt=1/[Φ,lt+√(Φ,lt²-βλ̄,lt²)]=1/[1.044+√(1.044²-0.75x1.044²)]=0.656
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt²-βλ̄,lt²)], χ,lt<=1.0, 1/λ̄,lt², χ,lt=0.656         (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/λ̄,lt²=1/1.071²=0.87         (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(1.071-0.8)²]=0.894, f<=1.0
χ,lt,mod=χ,lt/f=0.656/0.894=0.734, χ,lt,mod<=1.0, χ,lt,mod<=0.87,  χ,lt,mod=0.734
```

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.734x[10^{-6}]x366.60x10^{3}x355/1.00=95.52kNm (EC3 Eq.6.55)
My,ed= 68.89 kNm < 95.52 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 68.89/95.52= 0.721<1

7.12. Lateral torsional buckling, (ULS) (Beam support) (EN1993-1-1, §6.3.2)
My,ed=86.98 kN, L=6.000m, Lcr,y=4.200m, Lcr,z=6.000m, Lcr,lt=3.000m

Check lateral-torsional buckling at support region, Lc= 3.000m

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·[π^{2}EIz/(kL)^2]{√[(kz/kw)^2(Iw/Iz)+(kL)^{2}GIt/(π^{2}EIz)+(C2·zg-C3·zj)^2] -(C2·zg-C3·zj)}
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
μ=Mo/M=qL^2/8M=-24.6/87.0=-0.28, ψ=Mb/Ma=54.9/-87.0=-0.63, C1=3.001, C2=0.311
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^{4} N/mm^2
k·L=3000mm, zg=h/2=240/2=120mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=3.001, C2=0.311, C3=0.000
Mcr=[10^{-6}]3.001x[π^{2}x2.1x10^{5}x2.836x10^{6}/3000^2]
x{ [(1.0/1.0)^{2}x(37.391x10^{9}/2.836x10^{6})
+3000^{2}x8.1x10^{4}x0.129x10^{6}/(π^{2}x2.1x10^{5}x2.836x10^{6})
+(0.311x120)^2]$^{0.5}$-(0.311x120) }= 269.1 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{[10^{-6}]x366.60x10^{3}x355/269.1}=0.695 (EC3 Eq.6.56)
h/b=240/120=2.00<=2.00 buckling curve:b
imperfection factor:α,lt=0.34, β=0.75, χ,lt=0.872 (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt($\bar{\lambda}$,lt-$\bar{\lambda}$,lto)+β$\bar{\lambda}$,lt^2]=0.5x[1+0.34x(0.695-0.40)+0.75x0.695^2]=0.732
χ,lt=1/[Φ,lt+√(Φ,lt^2-β$\bar{\lambda}$,lt^2)]=1/[0.732+√(0.732^2-0.75x0.732^2)]=0.872
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt^2-β$\bar{\lambda}$,lt^2)], χ,lt<=1.0, 1/$\bar{\lambda}$,lt^2, χ,lt=0.872 (Eq.6.57)

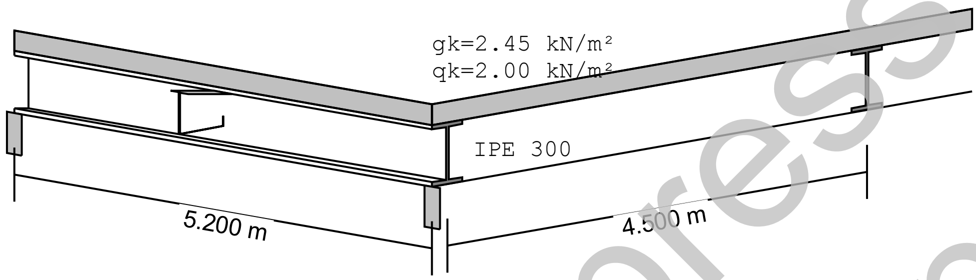
χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/$\bar{\lambda}$,lt^2=1/0.695^2=2.07 (EC3 §6.3.2.3(2),Eq.6.58)
Kc=1/(1.33-0.33ψ)=0.752, ψ=0.00 (EC3 Tab.6.6)
f=1-0.5(1-kc)[1-2.0($\bar{\lambda}$,lt-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.695-0.8)^2]=0.879, f<=1.0
χ,lt,mod=χ,lt/f=0.872/0.879=0.992, χ,lt,mod<=1.0, χ,lt,mod<=2.07, χ,lt,mod=0.992

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.992x[10^{-6}]x366.60x10^{3}x355/1.00=129.10kNm (EC3 Eq.6.55)
My,ed= 86.98 kNm < 129.10 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 86.98/129.10= 0.674<1

8. BEAM-003

Design of Floor Beams,Floor of one span
(EC3 EN1993-1-1:2005)

Span of floor beam L=5.200 m, Spacing of floor beams s=4.500 m, Simply supported beam
Conditions of lateral restraining: Lat. restr. at L/2



8.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
EN1991-1-1:2002, Eurocode 1-1 Actions on structures
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

8.2. 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)
γ_G= 1.35, γ_Q= 1.50

Partial factors for materials (EN1993-1-1, §6.1)
γ_{M0}= 1.00, γ_{M1}= 1.00, γ_{M2}= 1.25

8.3. Loading (EN1991-1-1)

Floor loads

Weight of floor finishing	gk1 = 0.500 kN/m²	(EN1991-1-1 §5.2)
Weight of floor structure	gk0 = 1.750 kN/m²	
Weight of ceiling under floor	gk2 = 0.200 kN/m²	
Self weight of floor	gk = 0.500+1.750+0.200= 2.450 kN/m²	
Live load on floor	qk = 2.000 kN/m²	(EN1991-1-1 §6.3)

Load on beam

Spacing of floor beams s= 4.500 m
Self weight of floor Gk1= 4.500x2.450= 11.03kN/m
Beam weight Gk2= 0.41 kN/m
Permanent load Gk =Gk1+Gk2= 11.44 kN/m
Variable load Qk = 4.500x2.000= 9.00kN/m

8.4. Design values of Actions, Load combinations

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

$\gamma G \cdot Gk + \gamma Q \cdot Qk$ = 1.35x11.44+1.50x9.00 =28.94kN/m, $ql^2/8$=97.83kNm

Design actions, Ultimate Limit State

Myed= $28.94 \times 5.200^2/8$= 97.83 kNm, Vzed= 28.94x5.200/2= 75.25 kN

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

Gk+Qk = 11.44+9.00= 20.44kN/m

Design actions, Serviceability Limit State (SLS)

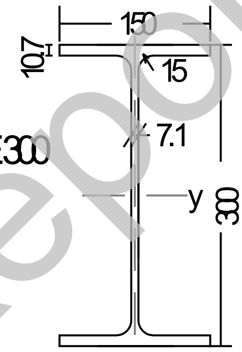
Myed= $20.44 \times 5.200^2/8$= 69.09 kNm, Vzed= 20.44x5.200/2= 53.14 kN

8.5. Cross-section properties

Cross-section IPE 300-S 355

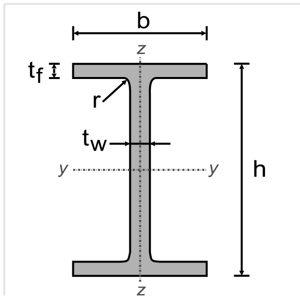
Dimensions of cross section

Depth of cross section	h=	300.00 mm
Width of cross section	b=	150.00 mm
Web depth	hw=	289.30 mm
Depth of straight portion of web	dw=	248.60 mm
Web thickness	tw=	7.10 mm
Flange thickness	tf=	10.70 mm
Radius of root fillet	r=	15.00 mm
Mass	=	42.20 Kg/m



Properties of cross section

Area	A= 5381 mm^2	
Second moment of area	Iy=83.560×10^6 mm^4	Iz= 6.038×10^6 mm^4
Section modulus	Wy=557.10×10^3 mm^3	Wz=80.500×10^3 mm^3
Plastic section modulus	Wpy=628.40×10^3 mm^3	Wpz=125.20×10^3 mm^3
Radius of gyration	iy= 124.6 mm	iz= 33.5 mm
Shear area	Avz= 2568 mm^2	Avy= 3210 mm^2
Torsional constant	It= 0.201×10^6 mm^4	ip= 129 mm
Warping constant	Iw=125.93×10^9 mm^6	



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

Beam deflections

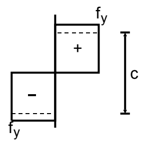
Loading G+Q: w=$5 \times 20.44 \times 5200^4/(384 \times 2.1 \times 10^5 \times 83.560 \times 10^6)$= 11.09mm = L/469<L/200
Loading Q: w=$5 \times 9.00 \times 5200^4/(384 \times 2.1 \times 10^5 \times 83.560 \times 10^6)$= 4.88mm = L/1065<L/360

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

8.7. Classification of cross-sections, Bending My (EN1993-1-1, §5.5)

Web

c=300.0-2x10.7-2x15.0=248.6 mm, t=7.1 mm, c/t=248.6/7.1=35.01
S 355 , t= 7.1<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}$=0.81
c/t=35.01<=72ε=72x0.81=58.32
The web is class 1 (EN1993-1-1, Tab.5.2)



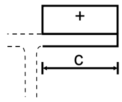
Flange

c=150.0/2-7.1/2-15.0=56.5 mm, t=10.7 mm, c/t=56.5/10.7=5.28

S 355 , t=10.7<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}=0.81$

c/t=5.28<=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

8.8. Resistance of cross-section, Beam section (EN1993-1-1, §6.2)

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

My.ed= 97.83 kNm

Bending Resistance Mply,rd=Wply·fy/γM0=$[10^{-6}]$x628.40x10^{3}x355/1.00= 223.08kNm

My,ed= 97.83 kNm < 223.08 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 97.83/223.08= 0.439<1

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

Vz.ed= 75.25 kN

Av=A-2b·tf+(tw+2r)tf=5381-2x150.0x10.7+(7.1+2x15.0)x10.7=2568mm² (EC3 §6.2.6.3)

Av= 2568mm² > η·hw·tw= 1.00x(300.0-2x10.7)x7.1=1.00x289.3x7.1= 2054mm²

Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= $[10^{-3}]$x2568x(355/1.73)/1.00= 526.33kN

Vz,ed= 75.25 kN < 526.33 kN =Vz,rd=Vpl,z,rd, Is verified

Vz,ed/Vz,rd= 75.25/526.33= 0.143<1

hw/tw=(300.0-2x10.7)/7.1=289.3/7.1=40.75<=72x0.81/1.00=72ε/η=58.32 (η=1.00)

S 355 , t= 7.1<= 40 mm, fy=355 N/mm², $\varepsilon=(235/355)^{0.5}=0.81$

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

Ultimate Limit State, Bending and shear (EN1993-1-1, §6.2.8

Vz,ed=37.63kN, My,ed=73.37kNm , Position at x=1.300m

Vz,ed=37.63=kN <= Vplz,rd/2=526.33/2=263.17kN (EC3 §6.2.8(2))

Effect of shear on moment resistance is neglected

8.9. Lateral torsional buckling, (ULS) (Beam span) (EN1993-1-1, §6.3.2)

My,ed=97.83 kN, L=5.200m, Lcr,y=5.200m, Lcr,z=5.200m, Lcr,lt=2.600m

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)}

Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006

μ=Mo/M=qL²/8M=0.25, ψ=Mb/Ma=0.00, C1=1.325, C2=0.122

G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^{4} N/mm²

k·L=2600mm, zg=0mm, zj=0mm (EN1993:2002 Eq.C.11)

ky=1.0, kz=1.0, kw=1.0, C1=1.325, C2=0.122, C3=0.000

Mcr=$[10^{-6}]$1.325x[π²x2.1x10^{5}x6.038x10^{6}/2600²]

x{ [(1.0/1.0)²x(125.93x10^{9}/6.038x10^{6})

+2600²x8.1x10^{4}x0.201x10^{6}/(π²x2.1x10^{5}x6.038x10^{6})]$^{0.5}$ }= 422.3 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{$[10^{-6}]$x628.40x10^{3}x355/422.3}=0.727 (EC3 Eq.6.56)

h/b=300/150=2.00<=2.00 buckling curve:b

imperfection factor:α,lt=0.34, β=0.75, χ,lt=0.856 (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+0.34x(0.727-0.40)+0.75x0.727²]=0.754

χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)]=1/[0.754+√(0.754²-0.75x0.754²)]=0.856

Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)], χ,lt<=1.0, 1/$\bar{\lambda}$,lt², χ,lt=0.856 (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/$\bar{\lambda}$,lt²=1/0.727²=1.89 (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($\bar{\lambda}$,lt-0.8)²]=1-0.5x(1-0.752)[1-2.0x(0.727-0.8)²]=0.877, f<=1.0

χ,lt,mod=χ,lt/f=0.856/0.877=0.976, χ,lt,mod<=1.0, χ,lt,mod<=1.89, χ,lt,mod=0.976

$M_{b,rd}=\chi_{,lt}\cdot W_{pl,y}\cdot f_y/\gamma_{M1}= 0.976\times[10^{-6}]\times628.40\times10^{3}\times355/1.00=217.73$ kNm (EC3 Eq.6.55)

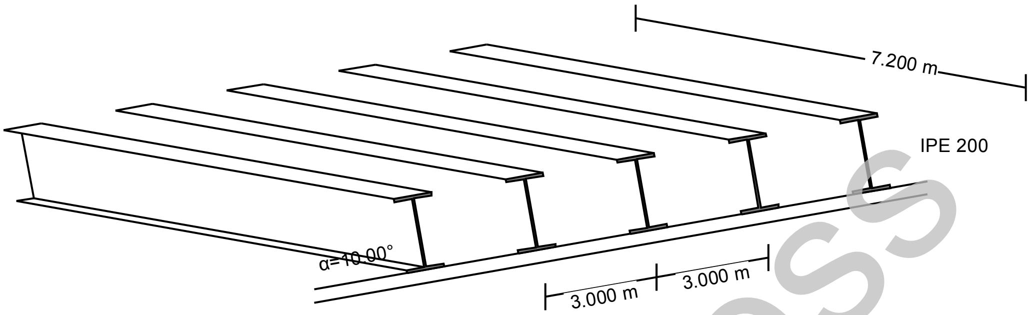
$M_{y,ed}= 97.83$ kNm < 217.73 kNm $=M_{b,rd}$, Is verified

$M_{y,ed}/M_{b,rd}= 97.83/217.73= 0.449<1$

9. STR01-001

Design of purlins
(EC3 EN1993-1-1:2005)

Purlin laterally restrained, IPE 200 S 355
Continuous purlin, L= 7.200 m, s= 3.000 m



9.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
EN1991-1-1:2002, Eurocode 1-1 Actions on structures
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

9.2. 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, γ_Q= 1.50, $\gamma_{G,inf}$= 1.00, ψ_0= 0.70

Partial factors for materials (EN1993-1-1, §6.1)
γ_{M0}= 1.00, γ_{M1}= 1.00, γ_{M2}= 1.25

9.3. Loading (EN1991-1-1)

Roof loads

Roof slope	α = 10.00°	
Load of roof covering	gk1 = 0.200 kN/m²	(EN1991-1-1 §5)
Imposed load (category H)	qk = 0.500 kN/m²	(EN1991-1-1 §6.3.4.2)
Snow load	qsk = 1.500 kN/m²	(EN1991-1-3 §5.3)
Wind pressure	wk = 0.000 kN/m²	(EN1991-1-4 §7.2)
Wind uplift	wk =-0.900 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= 0.22 kN/m
Permanent load	Gk =Gk1+Gk2=0.60+0.22=0.82 kN/m
Imposed load (category H)	Qkk= 3.000x0.500= 1.50kN/m
Snow load	Qsk= 3.000x1.500= 4.50kN/m
Wind uplift	Qwk=-3.000x0.900=-2.70kN/m

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

Permanent load	Gk,z = 0.82xcos(10.00)= 0.81kN/m,	Gk,y = 0.82xsin(10.00)= 0.14kN/m
Imposed load (category H)	Qkk,z= 1.50xcos(10.00)= 1.48kN/m,	Qkk,y= 1.50xsin(10.00)= 0.26kN/m
Snow load	Qsk,z= 4.50xcos(10.00)= 4.43kN/m,	Qsk,y= 4.50xsin(10.00)= 0.78kN/m
Wind pressure	Qwk,z= 0.00kN/m,	Qwk,y= 0.00kN/m
Wind uplift	Qwk,z= - 2.70kN/m,	Qwk,y= 0.00kN/m

9.4. Design values of Actions, Load combinations

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

Sagging	γG,sup·Gk,z+γQ·Qk,z+γQ·ψo·Qwk,z	= 1.35x0.81 +1.50x4.43 +1.50x0.70x0.00	= 7.74kN/m
Hogging	γG,inf·Gk,z-γQ·Qwk,z	= 1.00x0.81 -1.50x2.70	= -3.24kN/m
Transverse	γG,sup·Gk,y+γQ·Qk,y	= 1.35x0.14 +1.50x0.78	= 1.36kN/m

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

Sagging	Gk,z+Qk,z+ψo·Qwk,z	= 0.81+4.43+0.70x0.00	= 5.24kN/m
Hogging	Gk,z+Qwk,z	= 0.81-2.70	= -1.89kN/m

9.5. Design actions

Design actions, Ultimate Limit State

Sagging Myed,o= 0.078x 7.74x7.200^2= 31.29kNm, Myed,s=-0.105x 7.74x7.200^2= -42.12kNm
Vzed=0.605x7.74x7.200= 33.71kN

Hogging Myed,o=-0.078x 3.24x7.200^2= -13.11kNm, Myed,s= 0.105x 3.24x7.200^2= 17.65kNm
Vzed=0.605x3.24x7.200= 14.12kN

Transverse Laterally restrained

Design actions, Serviceability Limit State (SLS)

Sagging Myed,o= 0.078x 5.24x7.200^2= 21.18kNm, Myed,s=-0.105x 5.24x7.200^2= -28.52kNm

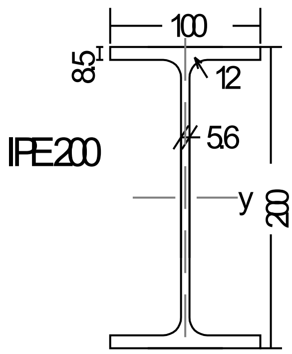
Hogging Myed,o=-0.078x 1.89x7.200^2= -7.65kNm, Myed,s= 0.105x 1.89x7.200^2= 10.30kNm

9.6. Cross-section properties, Purlins

Cross-section IPE 200-S 355

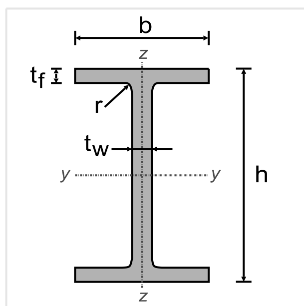
Dimensions of cross section

Depth of cross section	h=	200.00 mm
Width of cross section	b=	100.00 mm
Web depth	hw=	191.50 mm
Depth of straight portion of web	dw=	159.00 mm
Web thickness	tw=	5.60 mm
Flange thickness	tf=	8.50 mm
Radius of root fillet	r=	12.00 mm
Mass	=	22.40 Kg/m



Properties of cross section

Area	A= 2848 mm^2	
Second moment of area	Iy=19.430x10^6 mm^4	Iz= 1.424x10^6 mm^4
Section modulus	Wy=194.30x10^3 mm^3	Wz=28.470x10^3 mm^3
Plastic section modulus	Wpy=220.60x10^3 mm^3	Wpz=44.610x10^3 mm^3
Radius of gyration	iy= 82.6 mm	iz= 22.4 mm
Shear area	Avz= 1400 mm^2	Avy= 1700 mm^2
Torsional constant	It= 0.070x10^6 mm^4	ip= 86 mm
Warping constant	Iw=12.988x10^9 mm^6	



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

Purlin deflections, Sagging

Loading G+Q: w= 5.24x7200^4/(153.6x2.1x10^{5}x19.430x10^6)= 22.47mm = L/321<L/200

Loading Q: w= 4.43x7200^4/(153.6x2.1x10^{5}x19.430x10^6)= 19.00mm = L/379<L/250

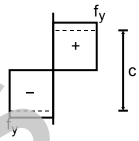
Purlin deflections, Hogging

Loading G+Q: w=-1.89x7200^4/(153.6x2.1x10^{5}x19.430x10^6)= -8.11mm = L/886<L/200

Loading Q: w=-2.70x7200^4/(153.6x2.1x10^{5}x19.430x10^6)=-11.58mm = L/620<L/250

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

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



Web

c=200.0-2x8.5-2x12.0=159.0 mm, t=5.6 mm, c/t=159.0/5.6=28.39

S 355 , t= 5.6<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81

c/t=28.39<=72ε=72x0.81=58.32

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

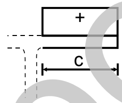
Flange

c=100.0/2-5.6/2-12.0=35.2 mm, t=8.5 mm, c/t=35.2/8.5=4.14

S 355 , t= 8.5<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81

c/t=4.14<=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

9.9. 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= 42.12 kNm

Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x220.60x10^{3}x355/1.00= 78.31kNm

My,ed= 42.12 kNm < 78.31 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 42.12/78.31= 0.538<1

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

Vz.ed= 33.71 kN

Av=A-2b·tf+(tw+2r)tf=2848-2x100.0x8.5+(5.6+2x12.0)x8.5=1400mm² (EC3 §6.2.6.3)

Av= 1400mm² > η·hw·tw= 1.00x(200.0-2x8.5)x5.6=1.00x191.5x5.6= 1072mm²

Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x1400x(355/1.73)/1.00= 286.86kN

Vz,ed= 33.71 kN < 286.86 kN =Vz,rd=Vpl,z,rd, Is verified

Vz,ed/Vz,rd= 33.71/286.86= 0.118<1

hw/tw=(200.0-2x8.5)/5.6=191.5/5.6=34.20<=72x0.81/1.00=72ε/η=58.32 (η=1.00)

S 355 , t= 5.6<= 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= 33.71kN, My.ed= 42.12kNm

Mpl,y,rd=78.31kNm, Vpl,z,rd=286.86kN

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

Ved=33.71kN <= 0.50x286.86=0.50xVpl,rd=143.43kN

Effect of shear force is neglected (EC3 §6.2.8.2)

9.10. 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)

$S=t^{1.5}(50+10b^{0.33})s/hw=0.750^{1.5}x(50+10x7200^{0.33}x3000/40.0=11842$ kNm/m

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

$Smin=[\pi^2E\cdot Iw/L^2+G\cdot It+\pi^2E\cdot Iz(h/2)^2/L^2]\cdot 70/h= [\pi^2x2.1x10^5x12.988x10^9/7200^2$
$+8.1x10^4x0.070x10^6+\pi^2x2.1x10^5x1.424x10^6x100^2/7200^2]x70/200^2]x[10^{-3}]=11771$kNm/m

s=11842 kNm/m > 11771 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)

$Cd,c=k\cdot E\cdot Ieff/s$, k=2, $Ieff=0.3x0.75x39.25^2=347mm^4/m$, s=3000mm (Eq.10.16)

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

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

C100=2.0, kba=1.25x200/100=2.50, $kt=(0.75/0.75)^{1.5}=1.00$, kbr=1.0, ka=1.0, kbt=1.0

Cd,a=2.0x2.50x1.00x1.0x1.0x1.0=5.0kNm/m

Cd=Cd,a= 4.5kNm/m

9.11. 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

$Mcr=C1\cdot[\pi^2EIz/(kL)^2]\{\sqrt{[(kz/kw)^2(Iw/Iz)+(kL)^2GIt,eq/(\pi^2EIz)+(C2\cdot zg-C3\cdot zj)^2]}-(C2\cdot zg-C3\cdot zj)\}$

Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006

$\mu=Mo/M=qL^2/8M=-1.19$, $\psi=Mb/Ma=0.00$, C1=1.629, C2=0.805

$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4$ N/mm², $It,eq=It+Cd\cdot(kL)^2/(\pi^2G)$

Hogging

$k\cdot L$=7200mm, zg=-100mm, zj=0mm (EN1993:2002 Eq.C.11)

ky=1.0, kz=1.0, kw=1.0, C1=1.629, C2=0.805, C3=0.000

$Mcr=[10^{-6}]1.629x[\pi^2x2.1x10^5x1.424x10^6/7200^2]$
$x\{ [(1.0/1.0)^2x(12.988x10^9/1.424x10^6)$
$+7200^2x8.1x10^4x0.365x10^6/(\pi^2x2.1x10^5x1.424x10^6)$
$+(-0.805x100)^2]^{0.5}-(-0.805x100) \}= 75.1$ kNm

$It,eq=(0.070x10^6+10^3x4.5x7200^2/(\pi^2 8.1x10^4))= 0.365x10^6$ mm^4

$\bar{\lambda},lt=\sqrt{(Wpl,y\cdot fy/Mcr)}=\sqrt{\{[10^{-6}]x220.60x10^3x355/75.1\}}=1.021$ (EC3 Eq.6.56)

h/b=200/100=2.00<=2.00 buckling curve:b

imperfection factor:α,lt=0.34, β=0.75, χ,lt=0.687 (T.6.3, T.6.5, Fig.6.4)

$\Phi,lt=0.5[1+\alpha,lt(\bar{\lambda},lt-\bar{\lambda},lto)+\beta\bar{\lambda},lt^2]=0.5x[1+0.34x(1.021-0.40)+0.75x1.021^2]=0.997$

$\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar{\lambda},lt^2)}]=1/[0.997+\sqrt{(0.997^2-0.75x0.997^2)}]=0.687$

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=0.687 (Eq.6.57)

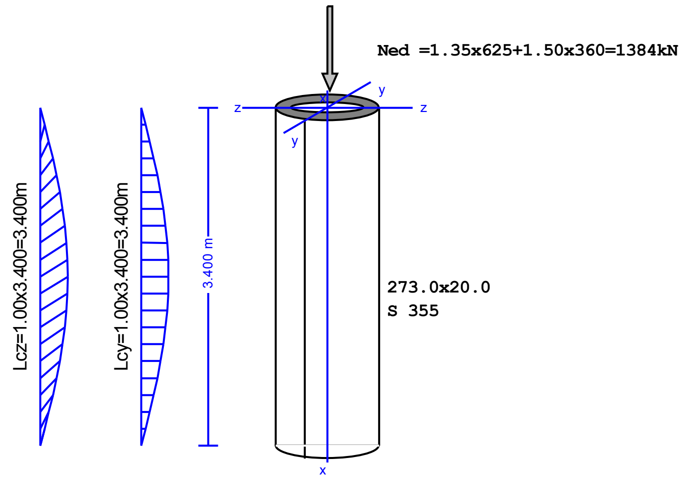
$Mb,rd=\chi,lt\cdot Wpl,y\cdot fy/\gamma M1= 0.687x[10^{-6}]x220.60x10^3x355/1.00=53.80$kNm (EC3 Eq.6.55)

My,ed= 17.65 kNm < 53.80 kNm =Mb,rd, Is verified

My,ed/Mb,rd= 17.65/53.80= 0.328<1

10. COLUMN-002

Design of Build-up Columns,Build-up laced column
(EC3 EN1993-1-1:2005)



10.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
EN1991-1-1:2002, Eurocode 1-1 Actions on structures
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

10.2. 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)
γ_G= 1.35, γ_Q= 1.50

Partial factors for materials (EN1993-1-1, §6.1)
γ_{M0}= 1.00, γ_{M1}= 1.00, γ_{M2}= 1.25

10.3. Loading (EN1991-1-1)

Permanent load Ngk= 625.00kN
Variable load Nqk= 360.00kN

10.4. Dimensions

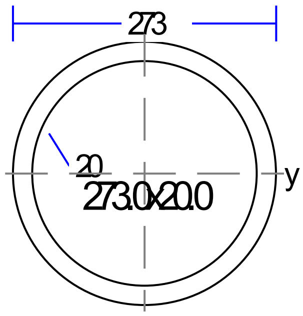
Column length L=3.400 m
Buckling lengthy-y: Lcr,y=1.000x3.400=3.400m
Buckling lengthz-z: Lcr,z=1.000x3.400=3.400m

10.5. Design values of Actions, Load combinations

Ultimate Limit State, Load combinations (EN1990 §6.4.3.2, T.A1.2A, T.A1.2B)
$N_{ed} = \gamma_G \cdot N_{gk} + \gamma_Q \cdot N_{qk}$ = 1.35x625.00+1.50x360.00= 1383.75kN

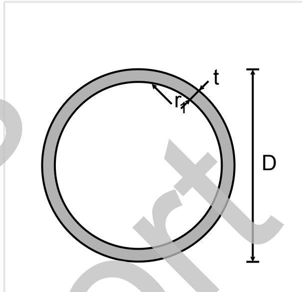
10.6. Cross-section properties

Cross-section O273.0x20.0-S 355



Dimensions of cross section

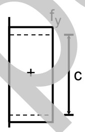
Depth of cross section	h=	273.00 mm
Width of cross section	b=	273.00 mm
Web depth	hw=	273.00 mm
Depth of straight portion of web	dw=	273.00 mm
Web thickness	tw=	20.00 mm
Flange thickness	tf=	20.00 mm
Mass	=	124.87 Kg/m



Properties of cross section

Area	A= 15896 mm^2	
Second moment of area	Iy=127.98x10^6 mm^4	Iz=127.98x10^6 mm^4
Section modulus	Wy=937.61x10^3 mm^3	Wz=937.61x10^3 mm^3
Plastic section modulus	Wpy=1282.8x10^3 mm^3	Wpz=1282.8x10^3 mm^3
Radius of gyration	iy= 89.7 mm	iz= 89.7 mm
Shear area	Avz= 10120 mm^2	Avy= 10120 mm^2
Torsional constant	It=255.97x10^6 mm^4	ip= 127 mm
Torsional modulus	Wt=1875.2x10^3 mm^3	

10.7. Classification of cross-sections, Compression Nc (EN1993-1-1, §5.5)



d=273.0 mm, t=20.0 mm, d/t=273.0/20.0=13.65
S 355 , t=20.0<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
d/t=13.65<=50ε²=50x0.81=32.80

Overall classification of cross-section is Class 1, Compression Nc,ed

10.8. Resistance of cross-section, Column section (EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)
Nc.ed=1383.75 kN
Compression Resistance Nplrd= A·fy/γM0=[10^{-3}]x15896x355/1.00=5643.24kN
Ned= 1383.75 kN < 5643.24 kN =Nc,rd=Nplrd, Is verified
Ned/Nc,rd= 1383.75/5643.24= 0.245<1

Ultimate Limit State, Verification for bending and axial force (EN1993-1-1, §6.2.9)
N.ed=1383.75 kN (Compression)
Nplrd=5643.24kN,
Ned=1383.75kN <= 0.25x5643.24=0.25xNplrd=1410.81kN
Ned=1383.75kN > [10^{-3}]x0.5x273.0x20.0x355/1.00=0.5hw·tw·fy/γM0=969.15 kN
n=Ned/Nplrd=1384/5643= 0.245
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=0 kN, Effect of shear force is neglected (EC3 §6.2.8.2)

n=Ned/Npl,rd=1384/5643=0.245
aw=(A-2b·t)/A, aw<=0.5, aw=(15896-2x273x20.0)/15896=0.31 (§6.2.9.1.5)
af=(A-2h·t)/A, af<=0.5, af=(15896-2x273x20.0)/15896=0.31
Mny,rd=Mply,rd(1-n)/(1-0.50aw)=0.00x0.895, Mny,rd<=Mply,rd, Mny,rd=0.00kNm (EC3 Eq.6.39)
Mnz,rd=Mplz,rd(1-n)/(1-0.50af)=0.00x0.895, Mnz,rd<=Mplz,rd, Mnz,rd=0.00kNm (EC3 Eq.6.40)
My,ed= 0.00 kNm <= 0.00 kNm =Mny,rd, Is verified
My,ed/Mny,rd= 0.00/0.00= 1.000<=1

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

Nc,ed=1383.75 kN, Lcr,y=3.400 m, Lcr,z=3.400 m

Buckling lengths: Lcr,y=1.000x3400=3400mm, Lcr,z=1.000x3400=3400mm

Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)

$\bar{\lambda}y=\sqrt{(A\cdot fy/Ncr,y)}=(Lcr,y/iy)\cdot(1/\lambda1)=(3400/\ 89.7)x(1/76.06)=0.498$

$\bar{\lambda}z=\sqrt{(A\cdot fy/Ncr,z)}=(Lcr,z/iz)\cdot(1/\lambda1)=(3400/\ 89.7)x(1/76.06)=0.498$

$\lambda1=\pi\sqrt{(E/fy)}=93.9\varepsilon=76.06$, $\varepsilon=\sqrt{(235/fy)}=0.81$

y-y buckling curve:a, imperfection factor:$\alpha y=0.21$, $\chi y=0.925$ (T.6.2,T.6.1,Fig.6.4)

$\Phi y=0.5[1+\alpha y(\bar{\lambda}y-0.2)+\bar{\lambda}y^2]=0.5x[1+0.21x(0.498-0.2)+0.498^2]=0.655$

$\chi y=1/[\Phi y+\sqrt{(\Phi y^2-\bar{\lambda}y^2)}]=1/[0.655+\sqrt{(0.655^2-0.498^2)}]=0.925 <=1$ $\chi y=0.925$

z-z buckling curve:a, imperfection factor:$\alpha z=0.21$, $\chi z=0.925$

$\Phi z=0.5[1+\alpha z(\bar{\lambda}z-0.2)+\bar{\lambda}z^2]=0.5x[1+0.21x(0.498-0.2)+0.498^2]=0.655$

$\chi z=1/[\Phi z+\sqrt{(\Phi z^2-\bar{\lambda}z^2)}]=1/[0.655+\sqrt{(0.655^2-0.498^2)}]=0.925 <=1$ $\chi z=0.925$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi<=1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.925$ (EC3 Eq.6.49)

$Nb,rd=\chi\cdot A\cdot fy/\gamma M1=\ 0.925x[10^{-3}]x15896x355/1.00=5220.00kN$ (EC3 Eq.6.47)

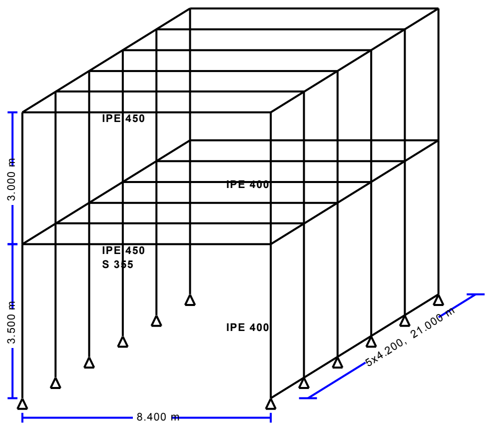
Nc,ed= 1383.75 kN < 5220.00 kN =Nb,rd, Is verified

Nc,ed/Nb,rd= 1383.75/5220.00= 0.265<1

11. FRAME-001

Two floor frame
(EC3 EN1993-1-1:2005)

FRAME-001



11.1. 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

11.2. Basic data

11.2.1. Geometry of frame structure

Bay width	L =	8.400 m
Column height	H1 =	3.500 m
	H2 =	3.000 m
Total length	B =	21.000 m (5x4.200m)
Spacing of frames	s =	4.200 m
Roof slope	α =	0.00°
Crossbeam spacing	=	3.000 m

11.2.2. Steel sections

Column section		IPE 400 - S 355
Beam section		IPE 450 - S 355
Lateral bracing of columns	Lm1=	3.275 m
Torsional restrains of beams	Lm2=	3.901 m

Compression stiffener at the bottom of corner joint between rafter and column

11.2.3. 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

11.3. Materials and Code parameters

11.3.1. 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

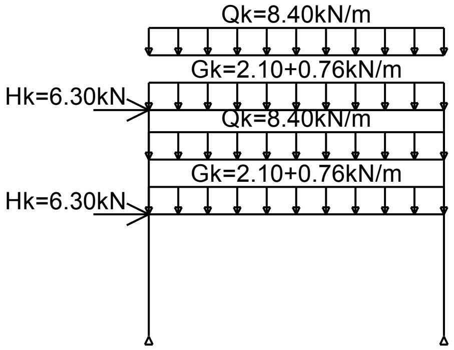
Partial factors for materials (EN1993-1-1, §6.1)

$\gamma M0$= 1.00, $\gamma M1$= 1.00, $\gamma M2$= 1.25

11.4. Loads

Permanent load	gk1 = 2.100 kN/m	gk2 = 2.100 kN/m
Self weight of beams	G(IPE 450)= 0.76kN/m	G(IPE 450)= 0.76kN/m
Permanent load on frame	Gk1 = 2.10+0.76= 2.86kN/m	Gk2 = 2.10+0.76= 2.86kN/m
Self weight of columns	G(IPE 400)= 0.65kN/m	G(IPE 400)= 0.65kN/m
Variable load-1	Qk1 = 8.400 kN/m	Qk2 = 8.400 kN/m
Variable load-2	Hk1 = 6.300 kN	Hk2 = 6.300 kN

Load Gk,Qk,Hk



11.4.1. Ultimate Limit State (ULS) (STR)

Ed = γG·Gk + γQ·Qk1 + γQ·ψo·Qk2 (Eq.6.10)
γG,sup=1.35 (Unfavorable)
γG,inf=1.00 (Favorable)
γQ =1.50 (Unfavorable)
γQ =0.00 (Favorable)
ψo =0.70 (Load combination)

Load combinations (ULS)(STR),
Permanent load Gk, Variable load-1 Qk Variable load-2 Hk

L.C. 201: 1.35Gk+1.50Qk (Eq.6.10)
L.C. 202: 1.35Gk+1.50Hk (Eq.6.10)
L.C. 221: 1.35Gk+1.50Qk+0.70x1.50Hk= 1.35xGk+1.50Qk+1.05Hk (Eq.6.10)
L.C. 222: 1.35Gk+1.50Hk+0.70x1.50Qk= 1.35xGk+1.50Hk+1.05Qk (Eq.6.10)

11.4.2. Serviceability Limit State (SLS)

Ed = Gk + Qk1 + ψo·Qk2 + ψo·Qk3 (Characteristic combination) (Eq.6.14b)

Load combinations (SLS)
Permanent load Gk, Variable load-1 Qk Variable load-2 Hk

L.C. 301: Gk+Qk (Eq.6.14a)
L.C. 302: Gk+Hk (Eq.6.14a)
L.C. 311: G + Qk + 0.70Hk (Eq.6.14a)
L.C. 312: G + Hk + 0.70Qk (Eq.6.14a)

11.4.3. Summary of load combination

Permanent load Gk, Variable load-1 Qk Variable load-2 Hk

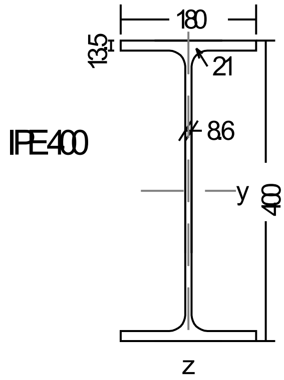
1	L.C. 201	(ULS)(STR)	1.35Gk+1.50Qk+0.00Hk
2	L.C. 202	(ULS)(STR)	1.35Gk+0.00Qk+1.50Hk
3	L.C. 221	(ULS)(STR)	1.35Gk+1.50Qk+1.05Hk
4	L.C. 222	(ULS)(STR)	1.35Gk+1.05Qk+1.50Hk
5	L.C. 301	(SLS)	1.00Gk+1.00Qk+0.00Hk
6	L.C. 302	(SLS)	1.00Gk+0.00Qk+1.00Hk
7	L.C. 311	(SLS)	1.00Gk+1.00Qk+0.70Hk
8	L.C. 312	(SLS)	1.00Gk+0.70Qk+1.00Hk

11.5. Steel sections

11.5.1. Column section

Cross-section properties

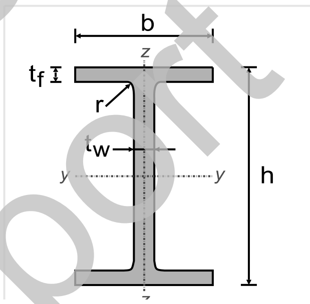
Cross-section IPE 400-S 355



Dimensions of cross section

Depth of cross section	h=	400.00 mm
Width of cross section	b=	180.00 mm
Web depth	hw=	386.50 mm
Depth of straight portion of web	dw=	331.00 mm
Web thickness	tw=	8.60 mm
Flange thickness	tf=	13.50 mm
Radius of root fillet	r=	21.00 mm
Mass	=	66.30 Kg/m

Properties of cross section

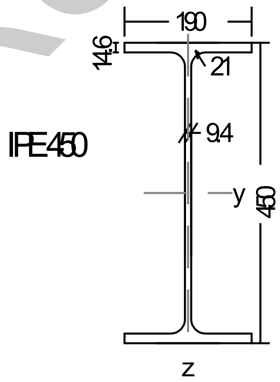


Area	A= 8446	mm^2		
Second moment of area	Iy=231.30x10^6	mm^4	Iz=13.180x10^6	mm^4
Section modulus	Wy=1156.0x10^3	mm^3	Wz=146.40x10^3	mm^3
Plastic section modulus	Wpy=1307.0x10^3	mm^3	Wpz=229.00x10^3	mm^3
Radius of gyration	iy= 165.5	mm	iz= 39.5	mm
Shear area	Avz= 4269	mm^2	Avy= 4860	mm^2
Torsional constant	It= 0.511x10^6	mm^4	ip= 170	mm
Warping constant	Iw=490.05x10^9	mm^6		

11.5.2. Beam section

Cross-section properties

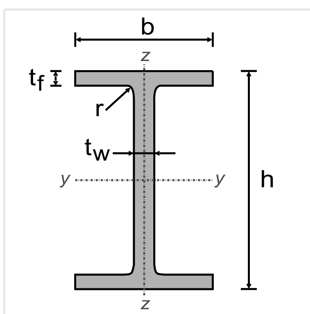
Cross-section IPE 450-S 355



Dimensions of cross section

Depth of cross section	h=	450.00 mm
Width of cross section	b=	190.00 mm
Web depth	hw=	435.40 mm
Depth of straight portion of web	dw=	378.80 mm
Web thickness	tw=	9.40 mm
Flange thickness	tf=	14.60 mm
Radius of root fillet	r=	21.00 mm
Mass	=	77.60 Kg/m

Properties of cross section



Area	A= 9882	mm^2		
Second moment of area	Iy=337.40x10^6	mm^4	Iz=16.760x10^6	mm^4
Section modulus	Wy=1500.0x10^3	mm^3	Wz=176.40x10^3	mm^3
Plastic section modulus	Wpy=1702.0x10^3	mm^3	Wpz=276.40x10^3	mm^3
Radius of gyration	iy= 184.8	mm	iz= 41.2	mm
Shear area	Avz= 5084	mm^2	Avy= 5548	mm^2
Torsional constant	It= 0.669x10^6	mm^4	ip= 189	mm
Warping constant	Iw=791.01x10^9	mm^6		

11.6. 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 global or local imperfections are taken into account by equivalent loads.

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

11.6.1. Data used for elastic analysis

Nodal points

Node	x [mm]	y[mm]
1	0	0
2	0	3500
3	0	6500
4	8400	6500
5	8400	3500
6	8400	0

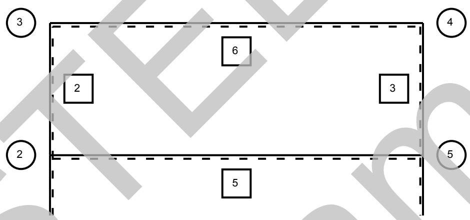
Supports

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

Elements

Element	node 1	node 2	length(mm)	angle(°)	E(GPa)	A(mm^2)	I(mm^4)
1	1	2	3500	90.00	210	8446	231300x10^3
2	2	3	3000	90.00	210	8446	231300x10^3
3	2	5	8400	0.00	210	9882	337400x10^3
4	3	4	8400	0.00	210	9882	337400x10^3
5	4	5	3000	270.00	210	8446	231300x10^3
6	5	6	3500	270.00	210	8446	231300x10^3

Finite element model (FEM) Linear elastic analysis



11.6.2. Element uniform loads, q perpendicular to element, qy vertical, qx horisontal [kN/m]

L.C.		Column(1)			Column(2)			Beam(1)			Beam(2)		
		q	qy	qx	q	qy	qx	q	qy	qx	q	qy	qx
201	ULS-STR	0.00	0.88	0	0.00	0.88	0	0.00	16.46	0	0.00	16.46	0
202	ULS-STR	0.00	0.88	0	0.00	0.88	0	0.00	3.86	0	0.00	3.86	0
221	ULS-STR	0.00	0.88	0	0.00	0.88	0	0.00	16.46	0	0.00	16.46	0
222	ULS-STR	0.00	0.88	0	0.00	0.88	0	0.00	12.68	0	0.00	12.68	0
301	SLS	0.00	0.65	0	0.00	0.65	0	0.00	11.26	0	0.00	11.26	0
302	SLS	0.00	0.65	0	0.00	0.65	0	0.00	2.86	0	0.00	2.86	0
311	SLS	0.00	0.65	0	0.00	0.65	0	0.00	11.26	0	0.00	11.26	0
312	SLS	0.00	0.65	0	0.00	0.65	0	0.00	8.74	0	0.00	8.74	0

11.7. Results of static-linear-elastic analysis

11.7.1. Displacements [mm]

L.C.	Hor. defl. Column Dx mm	Bending Defl. Beam(1) wy mm	Bending Defl. Beam(2) wy mm
201 ULS-STR	0.096	3.012	4.961
202 ULS-STR	2.836	0.707	1.149
221 ULS-STR	2.066	3.012	4.951
222 ULS-STR	2.888	2.320	3.808
301 SLS	0.066	2.060	3.394
302 SLS	1.893	0.523	0.852
311 SLS	1.379	2.060	3.387
312 SLS	1.927	1.599	2.625

11.7.2. 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
201 ULS-STR	6.8	144.0	0.0	-6.8	144.0	0.0
202 ULS-STR	-3.2	34.2	0.0	-6.3	42.1	0.0
221 ULS-STR	3.5	141.2	0.0	-10.1	146.7	0.0
222 ULS-STR	0.5	108.3	0.0	-10.0	116.2	0.0
301 SLS	4.7	98.8	0.0	-4.7	98.8	0.0
302 SLS	-2.0	25.6	0.0	-4.3	30.9	0.0
311 SLS	2.5	97.0	0.0	-6.9	100.6	0.0
312 SLS	0.5	75.0	0.0	-6.8	80.3	0.0

11.7.3. Axial forces Ned [kN]

L.C.	Column(1) Ned,6	Column(2) Ned,5	Beam(1) Ned,3	Beam(2) Ned,4
201 ULS-STR	-142.4	-70.5	40.8	-47.6
202 ULS-STR	-40.5	-18.3	4.9	-11.2
221 ULS-STR	-145.2	-71.0	37.5	-47.7
222 ULS-STR	-114.6	-55.3	26.8	-36.7
301 SLS	-97.7	-48.3	27.9	-32.6
302 SLS	-29.7	-13.5	4.0	-8.3
311 SLS	-99.5	-48.6	25.7	-32.6
312 SLS	-79.1	-38.2	18.6	-25.3

11.7.4. Shearing forces Ved [kN]

	Column(1)		Column(2)		Beam(1)		Beam(2)	
L.C.	VedA,6	VedB,6	VedA,5	VedB,5	VedA,3	VedB,3	VedA,4	VedB,4
201 ULS-STR	6.8	6.8	47.6	47.6	69.1	-69.1	69.1	-69.1
202 ULS-STR	6.3	6.3	11.2	11.2	13.0	-19.4	15.4	-17.0
221 ULS-STR	10.1	10.1	47.7	47.7	66.9	-71.4	68.6	-69.7
222 ULS-STR	10.0	10.0	36.7	36.7	50.1	-56.4	52.5	-54.0
301 SLS	4.7	4.7	32.6	32.6	47.3	-47.3	47.3	-47.3
302 SLS	4.3	4.3	8.3	8.3	9.9	-14.1	11.5	-12.5
311 SLS	6.9	6.9	32.6	32.6	45.8	-48.8	46.9	-47.7
312 SLS	6.8	6.8	25.3	25.3	34.6	-38.8	36.2	-37.2

A:left end, B: right end

11.7.5. Bending moments Med [kNm]

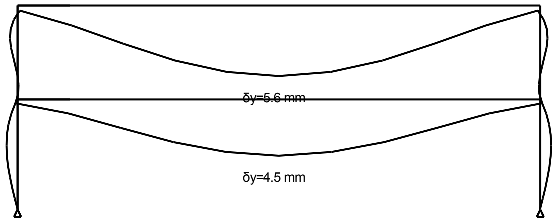
	Column(1)			Column(2)		
L.C.	MedA,6	MedM,6	MedB,6	MedA,5	MedM,5	MedB,5
201 ULS-STR	-23.9	-12.0	0.0	-79.3	-7.9	63.5
202 ULS-STR	-22.0	-11.0	0.0	-21.9	-5.0	11.8
221 ULS-STR	-35.4	-17.7	0.0	-81.6	-10.1	61.4
222 ULS-STR	-34.8	-17.4	0.0	-64.4	-9.2	45.9
301 SLS	-16.4	-8.2	0.0	-54.2	-5.4	43.5
302 SLS	-15.1	-7.5	0.0	-16.0	-3.5	9.0
311 SLS	-24.0	-12.0	0.0	-55.8	-6.9	42.0
312 SLS	-23.6	-11.8	0.0	-44.3	-6.3	31.7

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

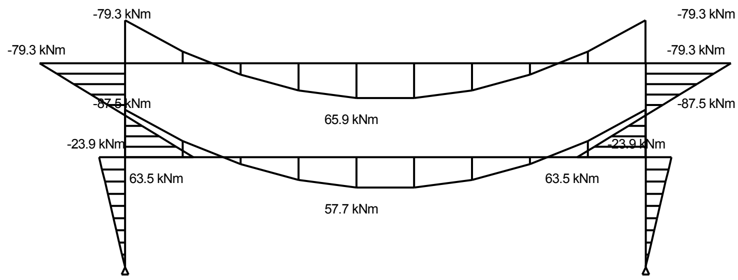
L.C.	Beam(1) MedA,3	MedM,3	MedB,3	Beam(2) MedA,4	MedM,4	MedB,4
201 ULS-STR	-87.5	57.7	-87.5	-79.3	65.9	-79.3
202 ULS-STR	-7.2	14.8	-33.8	-15.4	15.5	-21.9
221 ULS-STR	-78.2	57.8	-96.8	-77.1	65.9	-81.6
222 ULS-STR	-54.1	44.8	-80.7	-57.9	50.7	-64.4
301 SLS	-59.8	39.5	-59.8	-54.2	45.1	-54.2
302 SLS	-6.3	10.8	-24.1	-11.7	11.5	-16.0
311 SLS	-53.6	39.6	-66.1	-52.8	45.1	-55.8
312 SLS	-37.6	30.9	-55.3	-40.0	35.0	-44.3

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

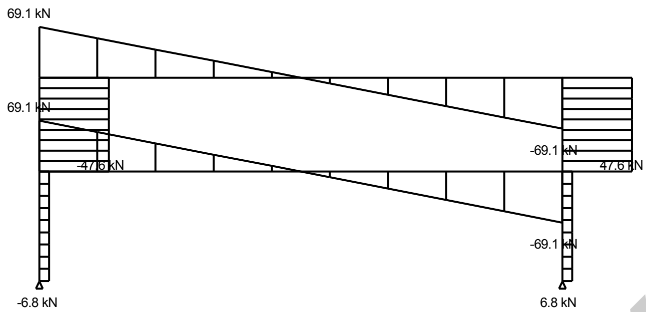
L.C. 201 Displacements mm



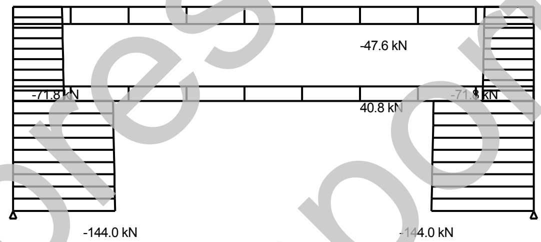
L.C. 201 Bend. moments kNm



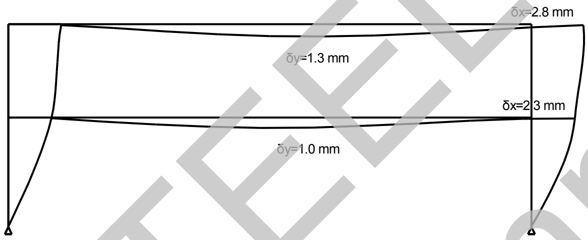
L.C. 201 Shear forces kN



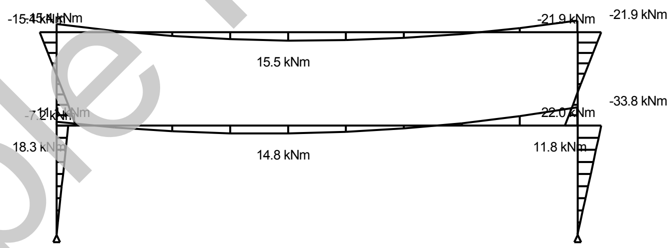
L.C. 201 Axial forces kN



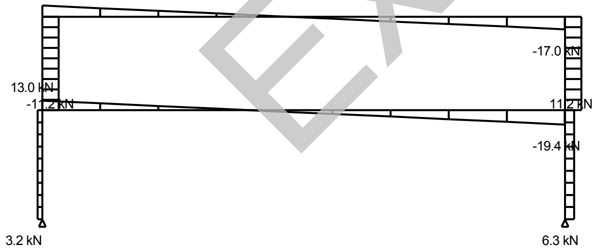
L.C. 202 Displacements mm



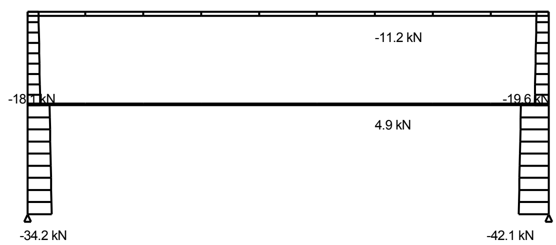
L.C. 202 Bend. moments kNm



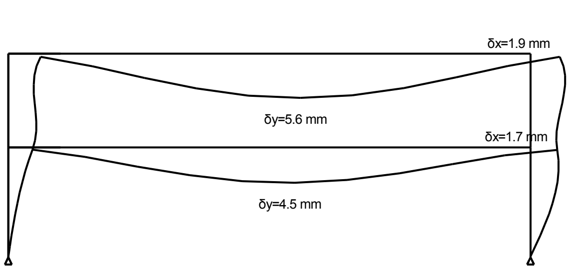
L.C. 202 Shear forces kN



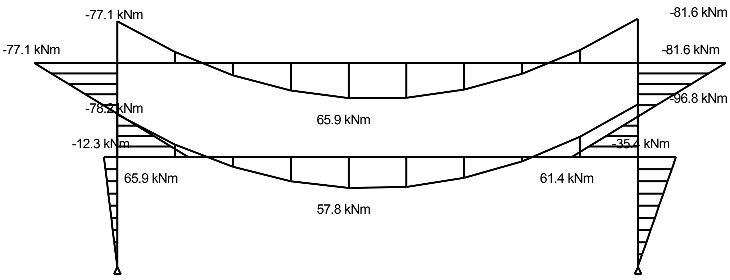
L.C. 202 Axial forces kN



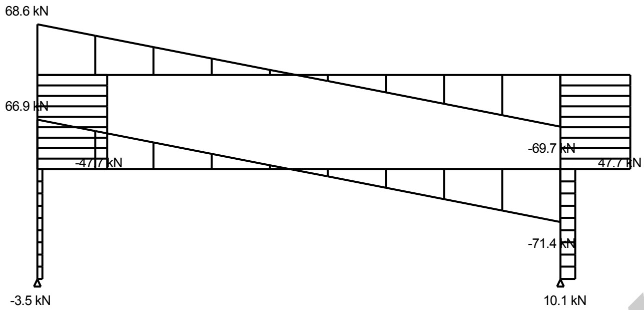
L.C. 221 Displacements mm



L.C. 221 Bend. moments kNm

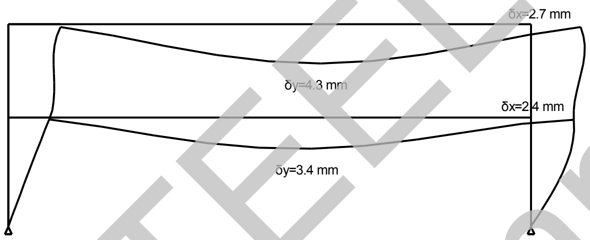


L.C. 221 Shear forces kN

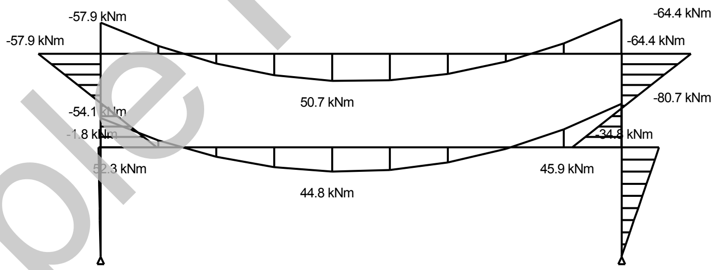


L.C. 221 Axial forces kN

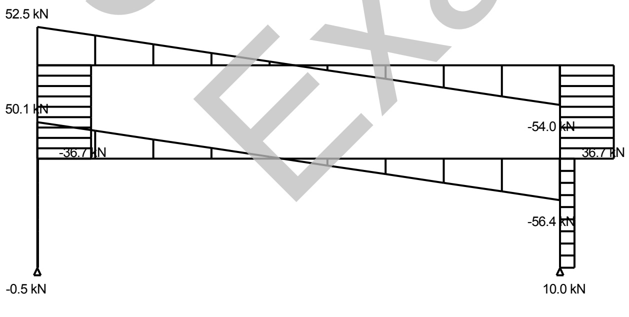
L.C. 222 Displacements mm



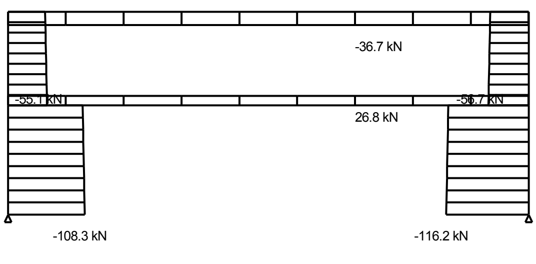
L.C. 222 Bend. moments kNm



L.C. 222 Shear forces kN



L.C. 222 Axial forces kN



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

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

$\alpha cr=(Hnhf/Ved)(h/\delta h,ed)$ (Eq.5.2)

From elastic analysis we obtain, L.C. 221: 1.35Gk+1.50Qk+0.70x1.50Hk= 1.35xGk+1.50Qk+1.05Hk

Vertical reaction at the base of column	Ved=	146.7 kN
Horizontal reaction at the base of column	Hed=	10.1 kN
Axial force at rafters	Nred=	47.7 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=	1.09 mm

$\alpha cr=(1.0/146.7)(3000/1.09)=18.79$ (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=8400/cos0.00° =8400mm

$Ncr=\pi^2EI/L^2=\pi^2 x210x337.40x10^6/(8400)^2=9910.7kN$

Maximum axial force in the rafters Ned=47.7 kN, L.C. 221: 1.35Gk+1.50Qk+0.70x1.50Hk

$\lambda= \sqrt{(9882x355/9910728)} =0.59 <= 0.3 \sqrt{(9882x355/47655)}=2.57$

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

αcr=18.79 > 10 (Eq.5.1)

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

Amplification factor for design moments $\delta=1/(1-1/\alpha cr)=1/(1-1/18.79)=1.06$ (Eq.5.4)

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

$\varphi=\varphi o\cdot \alpha h\cdot \alpha m\cdot \delta=(1/200)x1.000x0.866x1.056=4.574x10^{-3}=1/219$ (Eq.5.5)

$\varphi o=1/200$, $\alpha h=2/\sqrt{h}=2/\sqrt{3.000}=1.000$ $2/3<=\alpha h<=1.0$, $\alpha m=\sqrt{(0.5(1+1/2))}=0.866$

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

Effect of initial sway imperfection $Heq=4.574x10^{-3}xVed$ (§5.3.2 (5))

11.8.3. Sway imperfections for columns (EN1993-1-1, §5.3.2)

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

	Left support 1		Right support 2		Hed1+Hed2	Ved1+Ved2		φ·Ved
	Hed,1	Ved,1	Hed,2	Ved,2	Hed	Ved	Hed/Vhe	Heq kN
201 ULS-STR	6.8	144.0	69.1	47.6	76.0	191.6	0.40	0.658
202 ULS-STR	-3.2	34.2	17.0	11.2	13.8	45.4	0.30	0.192
221 ULS-STR	3.5	141.2	69.7	47.7	73.2	188.9	0.39	0.671
222 ULS-STR	0.5	108.3	54.0	36.7	54.5	145.0	0.38	0.531
301 SLS	4.7	98.8	47.3	32.6	52.0	131.4	0.40	0.452
302 SLS	-2.0	25.6	12.5	8.3	10.5	33.9	0.31	0.141
311 SLS	2.5	97.0	47.7	32.6	50.1	129.6	0.39	0.460
312 SLS	0.5	75.0	37.2	25.3	37.7	100.3	0.38	0.367

11.8.4. Internal forces and bending moments with imperfection effect

11.8.5. Axial forces Ned [kN]

L.C.	Column(1) Ned,6	Column(2) Ned,5	Beam(1) Ned,3	Beam(2) Ned,4
201 ULS-STR	-143.2	-70.7	40.4	-47.9
202 ULS-STR	-40.8	-18.4	4.8	-11.3
221 ULS-STR	-146.0	-71.2	37.2	-48.0
222 ULS-STR	-115.3	-55.5	26.5	-37.0
301 SLS	-98.2	-48.4	27.7	-32.8
302 SLS	-29.9	-13.5	3.9	-8.4
311 SLS	-100.1	-48.8	25.5	-32.8
312 SLS	-79.6	-38.3	18.4	-25.5

11.8.6. Shearing forces Ved [kN]

L.C.		Column(1) VedA,6	VedB,6	Column(2) VedA,5	VedB,5	Beam(1) VedA,3	VedB,3	Beam(2) VedA,4	VedB,4
201	ULS-STR	7.5	7.5	47.9	47.9	68.6	-69.7	68.9	-69.4
202	ULS-STR	6.5	6.5	11.3	11.3	12.9	-19.5	15.4	-17.0
221	ULS-STR	10.8	10.8	48.0	48.0	66.3	-71.9	68.4	-69.9
222	ULS-STR	10.5	10.5	37.0	37.0	49.6	-56.9	52.3	-54.2
301	SLS	5.1	5.1	32.8	32.8	46.9	-47.7	47.1	-47.4
302	SLS	4.5	4.5	8.4	8.4	9.8	-14.2	11.5	-12.6
311	SLS	7.3	7.3	32.8	32.8	45.4	-49.2	46.8	-47.8
312	SLS	7.1	7.1	25.5	25.5	34.3	-39.1	36.1	-37.3

A:left end, B: right end

11.8.7. Bending moments Med [kNm]

L.C.		Column(1) MedA,6	MedM,6	MedB,6	Column(2) MedA,5	MedM,5	MedB,5
201	ULS-STR	-26.2	-13.1	0.0	-80.2	-8.3	63.6
202	ULS-STR	-22.7	-11.3	0.0	-22.2	-5.2	11.8
221	ULS-STR	-37.8	-18.9	0.0	-82.5	-10.6	61.4
222	ULS-STR	-36.7	-18.4	0.0	-65.1	-9.6	45.9
301	SLS	-18.0	-9.0	0.0	-54.9	-5.7	43.5
302	SLS	-15.6	-7.8	0.0	-16.2	-3.6	9.0
311	SLS	-25.6	-12.8	0.0	-56.4	-7.2	42.1
312	SLS	-24.9	-12.5	0.0	-44.8	-6.5	31.7

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

L.C.		Beam(1) MedA,3	MedM,3	MedB,3	Beam(2) MedA,4	MedM,4	MedB,4
201	ULS-STR	-85.1	57.7	-89.8	-78.3	65.9	-80.2
202	ULS-STR	-6.5	15.0	-34.5	-15.2	15.5	-22.2
221	ULS-STR	-75.8	57.9	-99.2	-76.1	65.9	-82.5
222	ULS-STR	-52.2	45.0	-82.6	-57.2	50.7	-65.1
301	SLS	-58.2	39.5	-61.5	-53.6	45.1	-54.9
302	SLS	-5.8	10.9	-24.6	-11.5	11.5	-16.2
311	SLS	-52.0	39.6	-67.7	-52.1	45.1	-56.4
312	SLS	-36.3	31.0	-56.6	-39.5	35.0	-44.8

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

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

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

Maximum vertical deflection, L.C. 301: Gk+Qk Dy= 2.1 mm= 8400/4000=L/4000
Limit for vertical deflection L/200, Is verified

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

Maximum horizontal deflection, L.C. 302: Gk+Hk Dx= 1.5 mm= 3500/2000=h/2000
Limit for horizontal deflection H/150, Is verified

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

Eigenfrequencies and Eigenperiods of the structure

Mass of building, for loading: L.C. 301: Gk+Qk

1	f=	1.995 Hz	T=	0.501 sec
2	f=	10.562 Hz	T=	0.095 sec
3	f=	11.592 Hz	T=	0.086 sec
4	f=	15.916 Hz	T=	0.063 sec
5	f=	33.114 Hz	T=	0.030 sec
6	f=	40.156 Hz	T=	0.025 sec
7	f=	79.995 Hz	T=	0.013 sec
8	f=	84.829 Hz	T=	0.012 sec
9	f=	91.773 Hz	T=	0.011 sec
10	f=	132.348 Hz	T=	0.008 sec

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

Profile : IPE 400-S 355

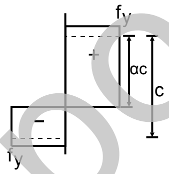
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

Ned = 147.5 kN
Ved = 10.8 kN
Myed = 37.8 kNm, Mzed =0.0 kNm
Myed = 35.3 kNm (Column top under the beam)
Buckling length,In-plane buckling Lcr,y =3078mm (Braced members) (EC3 §5.5.2.(7))
Buckling length,Out-of-plane buckling Lcr,z =3275mm (System length)
Buckling length,Torsional buckling Lcr,t =3275mm
Buckling length,Lateral torsional buckling Lcr,lt=3275mm

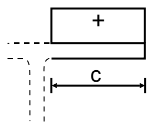
11.10.1. Classification of cross-sections, Column(1) (EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
σ=[10^3]147.50/8446 ± [10^6]37.80/1156.0x10^3 ± [10^6]0.00/146.4x10^3
σ1=50 N/mm², σ2=-15 N/mm² (compression positive)

Web
c=400.0-2x13.5-2x21.0=331.0 mm, t=8.6 mm, c/t=331.0/8.6=38.49
S 355 , t= 8.6<= 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)=147500/(2x8.6x355/1.00)=24.2 mm
α=(331.0/2+24.2)/331.0=0.573>0.5
c/t=38.49<=396x0.81/(13x0.573-1)=49.74
The web is class 1 (EN1993-1-1, Tab.5.2)



Flange
c=180.0/2-8.6/2-21.0=64.7 mm, t=13.5 mm, c/t=64.7/13.5=4.79
S 355 , t=13.5<= 40 mm, fy=355 N/mm², ε=$(235/355)^{0.5}$=0.81
c/t=4.79<=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

11.10.2. Resistance of cross-section, Column(1) (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. 221: 1.35xGk+1.50Qk+1.05Hk
Nc.ed=147.50 kN
Compression Resistance Nplrd= A·fy/γM0=[10^{-3}]x8446x355/1.00=2998.33kN
Ned= 147.50 kN < 2998.33 kN =Nc,rd=Nplrd, Is verified
Ned/Nc,rd= 147.50/2998.33= 0.049<1

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
My.ed= 37.80 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x1307.0x10^3x355/1.00= 463.98kNm
My,ed= 37.80 kNm < 463.98 kNm =My,rd=Mply,rd, Is verified
My,ed/My,rd= 37.80/463.98= 0.081<1

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Vz.ed= 10.80 kN
Av=A-2b·tf+(tw+2r)tf=8446-2x180.0x13.5+(8.6+2x21.0)x13.5=4269mm² (EC3 §6.2.6.3)
Av= 4269mm² > η·hw·tw= 1.00x(400.0-2x13.5)x8.6=1.00x386.5x8.6= 3324mm²
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x4269x(355/1.73)/1.00= 874.99kN
Vz,ed= 10.80 kN < 874.99 kN =Vz,rd=Vpl,z,rd, Is verified
Vz,ed/Vz,rd= 10.80/874.99= 0.012<1

hw/tw=(400.0-2x13.5)/8.6=386.5/8.6=44.94<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t= 8.6<= 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. 221: 1.35xGk+1.50Qk+1.05Hk
N.ed= 147.50kN (Compression), Vz.ed= 10.80kN, My.ed= 37.80kNm
Nplrd=2998.33kN, Mpl,y,rd=463.98kNm, Vpl,z,rd=874.99kN
Ned=147.50kN <= 0.25x2998.33=0.25xNplrd=749.58kN
Ned=147.50kN <= [10^{-3}]x0.5x386.5x8.6x355/1.00=0.5hw·tw·fy/γM0=589.99 kN
n=Ned/Nplrd=148/2998= 0.049
Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=10.80kN <= 0.50x874.99=0.50xVpl,rd=437.49kN
Effect of shear force is neglected (EC3 §6.2.8.2)

My,ed= 37.80 kNm < 463.98 kNm =Mply,rd, Is verified
My,ed/Mply,rd= 37.80/463.98= 0.081<1

11.10.3. Buckling length, In-plane buckling (EN1993-1-1, §5.2.2.8)

Buckling lengths ENV 1993-1-1:1992 Annex E
kc=Ic1/H1=231x10^6/3500= 66085.71, k1=Ic2/H2=231x10^6/3000= 77100.00
k12= 1.50Ib1/L= 1.50x337x10^6/8400= 60250.00
η2=1.00, η1= (Kc+k1)/(kc+k1+k12)=(66085.71+77100.00)/(66085.71+77100.00+60250.00)= 0.704
Column in braced frame, ky=0.879, Lcr,y=0.879x3500=3078mm, ENV 1993-1-1:1992 Annex E.5

11.10.4. Flexural Buckling, Column(1) (Ultimate Limit State) (EN1993-1-1, §6.3.1)
Nc,ed=147.50 kN, Lcr,y=3.078 m, Lcr,z=3.275 m
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Buckling lengths: Lcr,y=0.879x3500=3078mm, Lcr,z=0.936x3500=3275mm
Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)
$\bar{\lambda}$y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(3078/165.5)x(1/76.06)=0.245
$\bar{\lambda}$z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(3275/ 39.5)x(1/76.06)=1.090
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=400/180=2.22>=1.20, tf=13.5mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.990 (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy($\bar{\lambda}$y-0.2)+$\bar{\lambda}$y²]=0.5x[1+0.21x(0.245-0.2)+0.245²]=0.535
χy=1/[Φy+√(Φy²-$\bar{\lambda}$y²)]=1/[0.535+√(0.535²-0.245²)]=0.990 <=1 χy=0.990
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.541
Φz=0.5[1+αz($\bar{\lambda}$z-0.2)+$\bar{\lambda}$z²]=0.5x[1+0.34x(1.090-0.2)+1.090²]=1.245
χz=1/[Φz+√(Φz²-$\bar{\lambda}$z²)]=1/[1.245+√(1.245²-1.090²)]=0.541 <=1 χz=0.541

Reduction factor χ=1/[Φ+√(Φ²-$\bar{\lambda}$²)], χ<=1.0, Φ=0.5[1+α($\bar{\lambda}$-0.2)+$\bar{\lambda}$²], χ=0.541 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.541x[10^{-3}]x8446x355/1.00=1622.10kN (EC3 Eq.6.47)
Nc,ed= 147.50 kN < 1622.10 kN =Nb,rd, Is verified
Nc,ed/Nb,rd= 147.50/1622.10= 0.091<1

11.10.5. Lateral torsional buckling, Column(1) (ULS) (EN1993-1-1, §6.3.2)
My,ed=35.34 kN, L=3.500m, Lcr,y=3.078m, Lcr,z=3.275m, Lcr,lt=3.275m
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
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)}
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
ψ=Mb/Ma=0.0/-35.3=0.00, C1=1.770, C2=0.000, C3=1.000,
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^4 N/mm²
k·L=3275mm, zg=h/2=400/2=200mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=0.9, kz=1.0, kw=1.0, C1=1.770, C2=0.000, C3=1.000
Mcr=[10^{-6}]1.770x[π²x2.1x10^5x13.180x10^6/3275²]
x{ [(1.0/1.0)²x(490.05x10^9/13.180x10^6)
+3275²x8.1x10^4x0.511x10^6/(π²x2.1x10^5x13.180x10^6)]$^{0.5}$ }= 1041.5 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{[10^{-6}]x1307.0x10^{3}x355/1041.5}=0.667 (EC3 Eq.6.56)
h/b=400/180=2.22>2.00 buckling curve:c
imperfection factor:α,lt=0.49, β=0.75, χ,lt=0.846 (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt($\bar{\lambda}$,lt-$\bar{\lambda}$,lto)+β$\bar{\lambda}$,lt^2]=0.5x[1+0.49x(0.667-0.40)+0.75x0.667^2]=0.733
χ,lt=1/[Φ,lt+√(Φ,lt^2-β$\bar{\lambda}$,lt^2)]=1/[0.733+√(0.733^2-0.75x0.733^2)]=0.846
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt^2-β$\bar{\lambda}$,lt^2)], χ,lt<=1.0, 1/$\bar{\lambda}$,lt^2, χ,lt=0.846 (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/$\bar{\lambda}$,lt^2=1/0.667^2=2.24 (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($\bar{\lambda}$,lt-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.667-0.8)^2]=0.880, f<=1.0
χ,lt,mod=χ,lt/f=0.846/0.880=0.961, χ,lt,mod<=1.0, χ,lt,mod<=2.24, χ,lt,mod=0.961

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.961x[10^{-6}]x1307.0x10^{3}x355/1.00=445.89kNm (EC3 Eq.6.55)
My,ed= 35.34 kNm < 445.89 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 35.34/445.89= 0.079<1

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

Ned=147.50 kN, My,ed=35.34 kNm

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}]x8446x355=2998.3 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x1307.0x10^{3}x355=464.0 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.990x[10^{-3}]x8446x355/1.00=2968.3kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.541x[10^{-3}]x8446x355/1.00=1622.1kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.961x[10^{-6}]x1307.0x10^{3}x355/1.00=445.9kNm

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·CmLT(μz/(1-Ned/Ncr,y)(1/Czy)0.60√(wy/wz), μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)

Ncr,y=π^{2}EIy/lcr,y^2=3.14^{2}x[10^{-3}]x210000x231.30x10^{6}/ 3078^2= 50601 kN
Ncr,z=π^{2}EIz/lcr,z^2=3.14^{2}x[10^{-3}]x210000x13.180x10^{6}/ 3275^2= 2547 kN
Ncr,t=(1/ip^2)x(G·It + π^{2}EIw/Lcr,t^2) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10^{-3}]x(1/170^2)[80769x 0.511x10^{6}+π^{2}x210000x490.05x10^{9}/2880^2]=5655 kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 147.5/ 50601)/(1-0.990x 147.5/ 50601)=1.000
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 147.5/ 2547)/(1-0.541x 147.5/ 2547)=0.973
alt=1-It/Iy>=0=1- 0.511x10^{6}/231.30x10^{6}=0.998 (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 1.307x10^{6}/ 1.156x10^{6}=1.131 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.229x10^{6}/ 0.146x10^{6}=1.564 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=147.50/(2998.30/1.00)=0.049

$\bar{\lambda}$max=max(0.245,1.090)=1.090 (EC3 Annex A.1)
Mcr,o=(1.00/1.77)x1041.50=588.4, C1=1.00
$\bar{\lambda}$o=√([10^{-6}]x1307.0x10^{3}x355/588.4)=0.890
$\bar{\lambda}$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√1.770 [(1-147.5/2547)(1-147.5/5655)]$^{0.25}$=0.260
εy=(My,ed/Ned)(A/Wel)=([10^{3}]x35.34/147.50)x(8446.0/1156.0x10^{3})=1.75

Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(147.50/50601.0)=1.001, (ψ=1.00) (EC3 Annex A, T.A.1)
$\bar{\lambda}$o=0.890 > $\bar{\lambda}$o,lim=0.260
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
=1.001+(1-1.001)x(√1.751x0.998)/(1+√1.751x0.998) = 1.000
Cmlt=Cmy^2·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=1.000^{2}x0.998/√[(1-147.5/2547.0)(1-147.5/5655.0)]=1.042, Cmlt=1.042

$Cyy=1+(wy-1)[(2-1.6Cmy^2\cdot\bar{\lambda}max/wy-1.6Cmy^2\cdot\bar{\lambda}max^2/wy)npl-blt]>=Wel,y/Wpl,y$ (Annex A, T.A.1)
$blt=0.5alt\cdot\bar{\lambda}o^2[My,ed/(\chi,lt\cdot Mpl,y,rd)](Mz,ed/Mpl,z,rd) =$
$=0.5x0.998x0.890^2[35.3/(0.961x410.4)](0.0/52.0) = 0.000$
$Cyy=1+(1.131-1)[(2-1.6x1.000^2x1.090/1.131-1.6x1.000^2x1.090^2/1.131)x0.049-0.000]=0.992$
$Cyy>=1156.0x10^3/1307.0x10^3=0.884$, $Cyy=0.992$

$Czy=1+(wy-1)[(2-14.0Cmy^2\cdot\bar{\lambda}max^2/wy^5)npl-dlt]>=0.6\sqrt{}(wy/wz)(Wel,y/Wpl,y)$ (Annex A, T.A.1)
$dlt=2alt\cdot[\bar{\lambda}o/(0.1+\bar{\lambda}z^4)][My,ed/(Cmy\cdot\chi,lt\cdot Mpl,y,rd)][Mz,ed/(Cmz\cdot Mpl,z,rd)]=$
$=20.998x[0.890/(0.1+1.090^4)][35.3/(1.000x0.961x410.4)][0.0/(0.000x52.0)]=0.000$
$Czy=1+(1.131-1)[(2-14.0x1.000^2x1.090^2/1.131^5)0.049-0.000]=0.955$
$Czy>=0.6\sqrt{}(1.131/1.500)(1156.0x10^3/1307.0x10^3)=0.461$, $Czy=0.955$

Cyy=0.992, Czy=0.955 (Annex A, T.A.1)
kyy=1.000x1.042x1.000/(1-147.50/50601.0)x(1/0.992)=1.053
$kzy=1.000x1.042x0.973/(1-147.50/50601.0)x(1/0.955)x0.6x\sqrt{}(1.131/1.500)=0.555$

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
$Ned/(\chi y\cdot Nrk/\gamma M1)+kyy\cdot My,ed/(\chi LT\cdot My,rk/\gamma M1)=$ (EC3 Eq.6.61)
147.5/(0.990x2998.3/1.00)+1.053x35.3/(0.961x464.0/1.00)=0.050+0.083=0.133
0.133< 1.000, Is verified
$Ned/(\chi z\cdot Nrk/\gamma M1)+kzy\cdot My,ed/(\chi LT\cdot My,rk/\gamma M1)=$ (EC3 Eq.6.62)
147.5/(0.541x2998.3/1.00)+0.555x35.3/(0.961x464.0/1.00)=0.091+0.044=0.135
0.135< 1.000, Is verified

11.11. Column verification, Column(2), (Ultimate Limit State) (EN1993-1-1, §6)

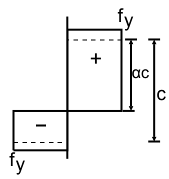
Profile : IPE 400-S 355

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned = 72.5 kN
Ved = 48.0 kN
Myed = 82.5 kNm, Mzed =0.0 kNm
Myed = 71.7 kNm (Column top under the beam)
Buckling length,In-plane buckling Lcr,y =2244mm (Braced members) (EC3 §5.5.2.(7))
Buckling length,Out-of-plane buckling Lcr,z =2775mm (System length)
Buckling length,Torsional buckling Lcr,t =2775mm
Buckling length,Lateral torsional buckling Lcr,lt=2775mm

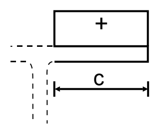
11.11.1. Classification of cross-sections, Column(2) (EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
$\sigma=[10^3]72.50/8446 \pm [10^6]82.50/1156.0x10^3 \pm [10^6]0.00/146.4x10^3$
$\sigma1=80\ N/mm^2$, $\sigma2=-63\ N/mm^2$ (compression positive)

Web
c=400.0-2x13.5-2x21.0=331.0 mm, t=8.6 mm, c/t=331.0/8.6=38.49
$S\ 355$, t= 8.6<= 40 mm, $fy=355\ N/mm^2$, $\varepsilon=(235/355)^{0.5}=0.81$
Position of neutral axis for combined Bending and compression
$Ned/(2tw\cdot fy/\gamma M0)=72500/(2x8.6x355/1.00)=11.9$ mm
α=(331.0/2+11.9)/331.0=0.536>0.5
c/t=38.49<=396x0.81/(13x0.536-1)=53.76
The web is class 1 (EN1993-1-1, Tab.5.2)



Flange
c=180.0/2-8.6/2-21.0=64.7 mm, t=13.5 mm, c/t=64.7/13.5=4.79
$S\ 355$, t=13.5<= 40 mm, $fy=355\ N/mm^2$, $\varepsilon=(235/355)^{0.5}=0.81$
c/t=4.79<=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

11.11.2. Resistance of cross-section, Column(2) (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. 221: 1.35xGk+1.50Qk+1.05Hk

Nc.ed= 72.50 kN

Compression Resistance Nplrd= A·fy/γM0=$[10^{-3}]$x8446x355/1.00=2998.33kN

Ned= 72.50 kN < 2998.33 kN =Nc,rd=Nplrd, Is verified

Ned/Nc,rd= 72.50/2998.33= 0.024<1

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

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

My.ed= 82.50 kNm

Bending Resistance Mply,rd=Wply·fy/γM0=$[10^{-6}]$x1307.0x10^{3}x355/1.00= 463.98kNm

My,ed= 82.50 kNm < 463.98 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 82.50/463.98= 0.178<1

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

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

Vz.ed= 48.00 kN

Av=A-2b·tf+(tw+2r)tf=8446-2x180.0x13.5+(8.6+2x21.0)x13.5=4269mm² (EC3 §6.2.6.3)

Av= 4269mm² > η·hw·tw= 1.00x(400.0-2x13.5)x8.6=1.00x386.5x8.6= 3324mm²

Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= $[10^{-3}]$x4269x(355/1.73)/1.00= 874.99kN

Vz,ed= 48.00 kN < 874.99 kN =Vz,rd=Vpl,z,rd, Is verified

Vz,ed/Vz,rd= 48.00/874.99= 0.055<1

hw/tw=(400.0-2x13.5)/8.6=386.5/8.6=44.94<=72x0.81/1.00=72ε/η=58.32 (η=1.00)

S 355 , t= 8.6<= 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. 221: 1.35xGk+1.50Qk+1.05Hk

N.ed= 72.50kN (Compression), Vz.ed= 48.00kN, My.ed= 82.50kNm

Nplrd=2998.33kN, Mpl,y,rd=463.98kNm, Vpl,z,rd=874.99kN

Ned=72.50kN <= 0.25x2998.33=0.25xNplrd=749.58kN

Ned=72.50kN <= $[10^{-3}]$x0.5x386.5x8.6x355/1.00=0.5hw·tw·fy/γM0=589.99 kN

n=Ned/Nplrd=73/2998= 0.024

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

Ved=48.00kN <= 0.50x874.99=0.50xVpl,rd=437.49kN

Effect of shear force is neglected (EC3 §6.2.8.2)

My,ed= 82.50 kNm < 463.98 kNm =Mply,rd, Is verified

My,ed/Mply,rd= 82.50/463.98= 0.178<1

11.11.3. Buckling length, In-plane buckling (EN1993-1-1, §5.2.2.8)

Buckling lengths ENV 1993-1-1:1992 Annex E

kc=Ic2/H2=231x10^{6}/3000=77100.00, k2=Ic1x10^{6}/H1=231/3500=66085.71

k12=1.50Ib2/L=1.50x337x10^{6}/8400=60250.00, k22=1.50Ib1/L=1.50x337x10^{6}/8400= 60250.00

η2= (Kc+k1)/(kc+k1+k12)=(66085.71+77100.00)/(66085.71+77100.00+60250.00)= 0.704

η1= Kc/(kc+k22)=77100.00/(77100.00+60250.00)= 0.561

Column in braced frame, ky=0.748, Lcr,y=0.748x3000=2244mm, ENV 1993-1-1:1992 Annex E.5

11.11.4. Flexural Buckling, Column(2) (Ultimate Limit State) (EN1993-1-1, §6.3.1)

Nc,ed=72.50 kN, Lcr,y=2.244 m, Lcr,z=2.775 m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

Buckling lengths: Lcr,y=0.748x3000=2244mm, Lcr,z=0.925x3000=2775mm

Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)

$\bar{\lambda}$y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(2244/165.5)x(1/76.06)=0.178

$\bar{\lambda}$z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(2775/ 39.5)x(1/76.06)=0.924

λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

$h/b=400/180=2.22>=1.20$, $tf=13.5mm<=40$ mm
$\bar{\lambda}<= 0.20$, $\chi y=1.00$ (EC3 §6.3.1.2.4)
z-z buckling curve:b, imperfection factor: $\alpha z=0.34$, $\chi z=0.646$
$\Phi z=0.5[1+\alpha z(\bar{\lambda}z-0.2)+\bar{\lambda}z^2]=0.5x[1+0.34x(0.924-0.2)+0.924^2]=1.050$
$\chi z=1/[\Phi z+\sqrt{(\Phi z^2-\bar{\lambda}z^2)}]=1/[1.050+\sqrt{(1.050^2-0.924^2)}]=0.646 <=1$ $\chi z=0.646$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi<=1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.646$ (EC3 Eq.6.49)
$Nb,rd=\chi\cdot A\cdot fy/\gamma M1= 0.646x[10^{-3}]x8446x355/1.00=1936.92kN$ (EC3 Eq.6.47)
Nc,ed= 72.50 kN < 1936.92 kN =Nb,rd, Is verified
Nc,ed/Nb,rd= 72.50/1936.92= 0.037<1

11.11.5. Lateral torsional buckling, Column(2) (ULS) (EN1993-1-1, §6.3.2)

My,ed=71.74 kN, L=3.000m, Lcr,y=2.244m, Lcr,z=2.775m, Lcr,lt=2.775m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
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
$Mcr=C1\cdot[\pi^2EIz/(kL)^2]\{\sqrt{[(kz/kw)^2(Iw/Iz)+(kL)^2GIt/(\pi^2EIz)+(C2\cdot zg-C3\cdot zj)^2]}-(C2\cdot zg-C3\cdot zj)\}$
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
$\psi=Mb/Ma=61.4/-71.7=-0.86$, C1=2.561, C2=0.000, C3=1.000,
$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4$ N/mm²
$k\cdot L=2775mm$, $zg=h/2=400/2=200mm$, zj=0mm (EN1993:2002 Eq.C.11)
ky=0.7, kz=1.0, kw=1.0, C1=2.561, C2=0.000, C3=1.000
$Mcr=[10^{-6}]2.561x[\pi^2x2.1x10^5x13.180x10^6/2775^2]$
$x\{ [(1.0/1.0)^2x(490.05x10^9/13.180x10^6)$
$+2775^2x8.1x10^4x0.511x10^6/(\pi^2x2.1x10^5x13.180x10^6)]^{0.5}\}= 2007.5$ kNm

$\bar{\lambda},lt=\sqrt{(Wpl,y\cdot fy/Mcr)}=\sqrt{\{[10^{-6}]x1307.0x10^3x355/2007.5\}}=0.481$ (EC3 Eq.6.56)
h/b=400/180=2.22>2.00 buckling curve:c
imperfection factor: $\alpha,lt=0.49$, $\beta=0.75$, $\chi,lt=0.955$ (T.6.3, T.6.5, Fig.6.4)
$\Phi,lt=0.5[1+\alpha,lt(\bar{\lambda},lt-\bar{\lambda},lto)+\beta\bar{\lambda},lt^2]=0.5x[1+0.49x(0.481-0.40)+0.75x0.481^2]=0.606$
$\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar{\lambda},lt^2)}]=1/[0.606+\sqrt{(0.606^2-0.75x0.606^2)}]=0.955$
Reduction factor $\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\beta\bar{\lambda},lt^2)}]$, $\chi,lt<=1.0$, $1/\bar{\lambda},lt^2$, $\chi,lt=0.955$ (Eq.6.57)

$\chi,lt,mod=\chi,lt/f$, $\chi,lt,mod<=1$, $\chi,lt,mod<=1/\bar{\lambda},lt^2=1/0.481^2=4.33$ (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)
$f=1-0.5(1-kc)[1-2.0(\bar{\lambda},lt-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.481-0.8)^2]=0.901$, $f<=1.0$
$\chi,lt,mod=\chi,lt/f=0.955/0.901=1.060$, $\chi,lt,mod<=1.0$, $\chi,lt,mod<=4.33$, $\chi,lt,mod=1.000$

$Mb,rd=\chi,lt\cdot Wpl,y\cdot fy/\gamma M1= 1.000x[10^{-6}]x1307.0x10^3x355/1.00=463.98kNm$ (EC3 Eq.6.55)
My,ed= 71.74 kNm < 463.98 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 71.74/463.98= 0.155<1

11.11.6. Axial force and bending moment, Column(2) (ULS) (EN1993-1-1, §6.3.3)

Ned=72.50 kN, My,ed=71.74 kNm

$Ned/(\chi y\cdot Nrk/\gamma M1)+kyy\cdot My,ed/(\chi LT\cdot My,rk/\gamma M1)<=1$ (EC3 Eq.6.61)
$Ned/(\chi z\cdot Nrk/\gamma M1)+kzy\cdot My,ed/(\chi LT\cdot My,rk/\gamma M1)<=1$ (EC3 Eq.6.62)
$Nrk=A\cdot fy=[10^{-3}]x8446x355=2998.3$ kN (Tab.6.7)
$My,rk=Wpl,y\cdot fy=[10^{-6}]x1307.0x10^3x355=464.0$ kNm
$\chi y\cdot Nrk/\gamma M1=\chi y\cdot A\cdot fy/\gamma M1= 1.000x[10^{-3}]x8446x355/1.00=2998.3kN$
$\chi z\cdot Nrk/\gamma M1=\chi z\cdot A\cdot fy/\gamma M1= 0.646x[10^{-3}]x8446x355/1.00=1936.9kN$
$\chi LT\cdot My,rk/\gamma M1=\chi LT\cdot Wpl,y\cdot fy/\gamma M1= 1.000x[10^{-6}]x1307.0x10^3x355/1.00=464.0kNm$

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

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

$Ncr,y=\pi^2EIy/lcr,y^2=3.14^2x[10^{-3}]x210000x231.30x10^6/ 2244^2= 95203$ kN
$Ncr,z=\pi^2EIz/lcr,z^2=3.14^2x[10^{-3}]x210000x13.180x10^6/ 2775^2= 3547$ kN
$Ncr,t=(1/ip^2)x(G\cdot It + \pi^2EIw/Lcr,t^2)$ (EC3 NCCI SN003b-EN-EU)
$Ncr,t=[10^{-3}]x(1/170^2)[80769x 0.511x10^6+\pi^2x210000x490.05x10^9/2076^2]=9569$ kN

```
μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1-  72.5/ 95203)/(1-1.000x  72.5/ 95203)=1.000
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1-  72.5/  3547)/(1-0.646x  72.5/  3547)=0.993
alt=1-It/Iy>=0=1- 0.511x10^6/231.30x10^6=0.998                           (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 1.307x10^6/ 1.156x10^6=1.131 <= 1.50            (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.229x10^6/ 0.146x10^6=1.564 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=72.50/(2998.30/1.00)=0.024

λ̄max=max(0.178,0.924)=0.920                                              (EC3 Annex A.1)
Mcr,o=(1.00/2.56)x2007.50=783.7,  C1=1.00
λ̄o=√([10^-6]x1307.0x10^3x355/783.7)=0.770
λ̄o,lim=0.2√C1 [(1-Ned/Ncr,z)(1-Ned/Ncr,t)]^0.25                         (EC3 Annex A.1)
λ̄o,lim=0.2√2.561 [(1-72.5/3547)(1-72.5/9569)]^0.25=0.318
εy=(My,ed/Ned)(A/Wel)=([10^3]x71.74/72.50)x(8446.0/1156.0x10^3)=7.23

Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(72.50/95203.0)=1.000, (ψ=1.00)       (EC3 Annex A, T.A.1)
λ̄o=0.770 > λ̄o,lim=0.318
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
   =1.000+(1-1.000)x(√7.230x0.998)/(1+√7.230x0.998) = 1.000
Cmlt=Cmy²·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=1.000²x0.998/√[(1-72.5/3547.0)(1-72.5/9569.0)]=1.012, Cmlt=1.012

Cyy=1+(wy-1)[(2-1.6Cmy²·λ̄max/wy-1.6Cmy²·λ̄max²/wy)npl-blt]>=Wel,y/Wpl,y     (Annex A, T.A.1)
blt=0.5alt·λ̄o²[My,ed/(χ,lt·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
   =0.5x0.998x0.770²[71.7/(1.000x410.4)](0.0/52.0) = 0.000
Cyy=1+(1.131-1)[(2-1.6x1.000²x0.920/1.131-1.6x1.000²x0.920²/1.131)x0.024-0.000]=0.998
Cyy>=1156.0x10^3/1307.0x10^3=0.884, Cyy=0.998

Czy=1+(wy-1)[(2-14.0Cmy²·λ̄max²/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[0.770/(0.1+0.924^4)][71.7/(1.000x1.000x410.4)][0.0/(0.000x52.0)]=0.000
Czy=1+(1.131-1)[(2-14.0x1.000²x0.920²/1.131^5)0.024-0.000]=0.986
Czy>=0.6√(1.131/1.500)(1156.0x10^3/1307.0x10^3)=0.461, Czy=0.986

Cyy=0.998, Czy=0.986                                                   (Annex A, T.A.1)
kyy=1.000x1.012x1.000/(1-72.50/95203.0)x(1/0.998)=1.015
kzy=1.000x1.012x0.993/(1-72.50/95203.0)x(1/0.986)x0.6x√(1.131/1.500)=0.531

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)=                                (EC3 Eq.6.61)
72.5/(1.000x2998.3/1.00)+1.015x71.7/(1.000x464.0/1.00)=0.024+0.157=0.181
                0.181< 1.000,  Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)=                                (EC3 Eq.6.62)
72.5/(0.646x2998.3/1.00)+0.531x71.7/(1.000x464.0/1.00)=0.037+0.082=0.120
                0.120< 1.000,  Is verified
```

11.12. Beam verification, Beam(1), (Ultimate Limit State) (EN1993-1-1, §6)

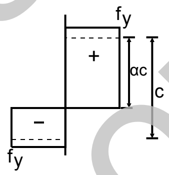
Profile : IPE 450-S 355

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned = 37.2 kN
Ved = 71.9 kN
Myed = 99.2 kNm, Mzed =0.0 kNm
Myed = 57.9 kNm (at mid-span)
Myed = -84.8 kNm (at column face)

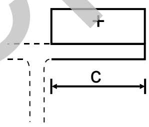
11.12.1. Classification of cross-sections, Beam(1) (EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
σ=[10^3]37.20/9882 ± [10^6]99.20/1500.0x10^3 ± [10^6]0.00/176.4x10^3
σ1=70 N/mm², σ2=-62 N/mm² (compression positive)

Web
c=450.0-2x14.6-2x21.0=378.8 mm, t=9.4 mm, c/t=378.8/9.4=40.30
S 355 , t= 9.4<= 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)=37200/(2x9.4x355/1.00)=5.6 mm
α=(378.8/2+5.6)/378.8=0.515>0.5
c/t=40.30<=396x0.81/(13x0.515-1)=56.36
The web is class 1 (EN1993-1-1, Tab.5.2)



Flange
c=190.0/2-9.4/2-21.0=69.3 mm, t=14.6 mm, c/t=69.3/14.6=4.75
S 355 , t=14.6<= 40 mm, fy=355 N/mm², ε=$(235/355)^{0.5}$=0.81
c/t=4.75<=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

11.12.2. Resistance of cross-section, Beam(1) (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. 201: 1.35Gk+1.50Qk
Nc.ed= 40.40 kN
Compression Resistance Nplrd= A·fy/γM0=[10^{-3}]x9882x355/1.00=3508.11kN
Ned= 40.40 kN < 3508.11 kN =Nc,rd=Nplrd, Is verified
Ned/Nc,rd= 40.40/3508.11= 0.012<1

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
My.ed= 99.20 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x1702.0x10^3x355/1.00= 604.21kNm
My,ed= 99.20 kNm < 604.21 kNm =My,rd=Mply,rd, Is verified
My,ed/My,rd= 99.20/604.21= 0.164<1

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Vz.ed= 71.90 kN
Av=A-2b·tf+(tw+2r)tf=9882-2x190.0x14.6+(9.4+2x21.0)x14.6=5084mm² (EC3 §6.2.6.3)
Av= 5084mm² > η·hw·tw= 1.00x(450.0-2x14.6)x9.4=1.00x435.4x9.4= 4093mm²
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x5084x(355/1.73)/1.00= 1042.10kN
Vz,ed= 71.90 kN < 1042.10 kN =Vz,rd=Vpl,z,rd, Is verified
Vz,ed/Vz,rd= 71.90/1042.10= 0.069<1

hw/tw=(450.0-2x14.6)/9.4=435.4/9.4=46.32<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t= 9.4<= 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. 221: 1.35xGk+1.50Qk+1.05Hk

N.ed= 37.20kN (Compression), Vz.ed= 71.90kN, My.ed= 99.20kNm

Nplrd=3508.11kN, Mpl,y,rd=604.21kNm, Vpl,z,rd=1042.10kN

Ned=37.20kN <= 0.25x3508.11=0.25xNplrd=877.03kN

Ned=37.20kN <= [10^{-3}]x0.5x435.4x9.4x355/1.00=0.5hw·tw·fy/γM0=726.46 kN

n=Ned/Nplrd=37/3508= 0.011

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

Ved=71.90kN <= 0.50x1042.10=0.50xVpl,rd=521.05kN

Effect of shear force is neglected (EC3 §6.2.8.2)

My,ed= 99.20 kNm < 604.21 kNm =Mply,rd, Is verified

My,ed/Mply,rd= 99.20/604.21= 0.164<1

11.12.3. Buckling resistance, Beam(1) mid-span region (Ultimate Limit State)

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

Ned = 37.2 kN

Ved = 66.3 kN

Myed = 57.9 kNm, Mzed =0.0 kNm

Rafter length Lr=8400 mm

Buckling length,In-plane buckling

αcr=18.79, Ned=37.2kN, Lcr,y=π√[EI/αcr·Ned] <= Lr=8400 mm

Lcr,y=π√[210000x337.40x10^{6}/(18.79x37.2x10^{3})=31630mm], Lcr,y=8400mm

Buckling length,In-plane buckling Lcr,y=8400mm (System length)

Buckling length,Out-of-plane buckling Lcr,z=3000mm (Crossbeam spacing)

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

Nc,ed=37.20 kN, Lcr,y=8.400 m, Lcr,z=3.000 m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

Buckling lengths: Lcr,y=1.000x8400=8400mm, Lcr,z=0.357x8400=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)=(8400/184.8)x(1/76.06)=0.598

$\bar{\lambda}$z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(3000/ 41.2)x(1/76.06)=0.958

λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=450/190=2.37>=1.20, tf=14.6mm<=40 mm

y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.891 (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(0.598-0.2)+0.598^2]=0.721

χy=1/[Φy+√(Φy^2-$\bar{\lambda}$y^2)]=1/[0.721+√(0.721^2-0.598^2)]=0.891 <=1 χy=0.891

z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.624

Φz=0.5[1+αz($\bar{\lambda}$z-0.2)+$\bar{\lambda}$z^2]=0.5x[1+0.34x(0.958-0.2)+0.958^2]=1.088

χz=1/[Φz+√(Φz^2-$\bar{\lambda}$z^2)]=1/[1.088+√(1.088^2-0.958^2)]=0.624 <=1 χz=0.624

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=0.624 (EC3 Eq.6.49)

Nb,rd=χ·A·fy/γM1= 0.624x[10^{-3}]x9882x355/1.00=2189.06kN (EC3 Eq.6.47)

Nc,ed= 37.20 kN < 2189.06 kN =Nb,rd, Is verified

Nc,ed/Nb,rd= 37.20/2189.06= 0.017<1

11.12.5. Lateral torsional buckling, Beam(1) mid-span region (ULS) (EN1993-1-1, §6.3.2)

My,ed=57.93 kN, L=8.400m, Lcr,y=8.400m, Lcr,z=3.000m, Lcr,lt=3.000m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

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·[π^{2}EIz/(kL)^2]{√[(kz/kw)^2(Iw/Iz)+(kL)^{2}GIt/(π^{2}EIz)+(C2·zg-C3·zj)^2] -(C2·zg-C3·zj)}

Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006

μ=Mo/M=qL^2/8M=18.5/57.9=0.32, ψ=Mb/Ma=-56.7/57.9=-0.98, C1=1.943, C2=0.197

G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^{4} N/mm^2

k·L=3000mm, zg=h/2=450/2=225mm, zj=0mm (EN1993:2002 Eq.C.11)

ky=1.0, kz=1.0, kw=1.0, C1=1.943, C2=0.197, C3=0.000

Mcr=[10^{-6}]1.943x[π^{2}x2.1x10^{5}x16.760x10^{6}/3000^2]

x{ [(1.0/1.0)^{2}x(791.01x10^{9}/16.760x10^{6})

+3000^{2}x8.1x10^{4}x0.669x10^{6}/(π^{2}x2.1x10^{5}x16.760x10^{6})

+(0.197x225)^2]$^{0.5}$-(0.197x225) }= 1552.2 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{[10^{-6}]x1702.0x10^{3}x355/1552.2}=0.624 (EC3 Eq.6.56)
h/b=450/190=2.37>2.00 buckling curve:c
imperfection factor:α,lt=0.49, β=0.75, χ,lt=0.872 (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt($\bar{\lambda}$,lt-$\bar{\lambda}$,lto)+β$\bar{\lambda}$,lt^2]=0.5x[1+0.49x(0.624-0.40)+0.75x0.624^2]=0.701
χ,lt=1/[Φ,lt+√(Φ,lt^2-β$\bar{\lambda}$,lt^2)]=1/[0.701+√(0.701^2-0.75x0.701^2)]=0.872
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt^2-β$\bar{\lambda}$,lt^2)], χ,lt<=1.0, 1/$\bar{\lambda}$,lt^2, χ,lt=0.872 (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/$\bar{\lambda}$,lt^2=1/0.624^2=2.57 (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(0.624-0.8)^2]=1.000, f<=1.0
χ,lt,mod=χ,lt/f=0.872/1.000=0.872, χ,lt,mod<=1.0, χ,lt,mod<=2.57, χ,lt,mod=0.872

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.872x[10^{-6}]x1702.0x10^{3}x355/1.00=526.87kNm (EC3 Eq.6.55)
My,ed= 57.93 kNm < 526.87 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 57.93/526.87= 0.110<1

11.12.6. Axial force and bending moment, Beam(1) mid-span region (ULS) (EN1993-1-1, §6.3.3)
Ned=37.20 kN, My,ed=57.93 kNm

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}]x9882x355=3508.1 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x1702.0x10^{3}x355=604.2 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.891x[10^{-3}]x9882x355/1.00=3125.7kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.624x[10^{-3}]x9882x355/1.00=2189.1kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.872x[10^{-6}]x1702.0x10^{3}x355/1.00=526.9kNm

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·CmLT(μz/(1-Ned/Ncr,y)(1/Czy)0.60√(wy/wz), μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)

Ncr,y=π^{2}EIy/lcr,y^2=3.14^{2}x[10^{-3}]x210000x337.40x10^{6}/ 8400^2= 9911 kN
Ncr,z=π^{2}EIz/lcr,z^2=3.14^{2}x[10^{-3}]x210000x16.760x10^{6}/ 3000^2= 3860 kN
Ncr,t=(1/ip^2)x(G·It + π^{2}EIw/Lcr,t^2) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10^{-3}]x(1/189^2)[80769x 0.669x10^{6}+π^{2}x210000x791.01x10^{9}/3000^2]=6590 kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 37.2/ 9911)/(1-0.891x 37.2/ 9911)=1.000
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 37.2/ 3860)/(1-0.624x 37.2/ 3860)=0.996
alt=1-It/Iy>=0=1- 0.669x10^{6}/337.40x10^{6}=0.998 (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 1.702x10^{6}/ 1.500x10^{6}=1.135 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.276x10^{6}/ 0.176x10^{6}=1.567 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=37.20/(3508.10/1.00)=0.011

$\bar{\lambda}$max=max(0.598,0.958)=0.960 (EC3 Annex A.1)
Mcr,o=(1.00/1.94)x1552.20=799.0, C1=1.00
$\bar{\lambda}$o=√([10^{-6}]x1702.0x10^{3}x355/799.0)=0.870
$\bar{\lambda}$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√1.943 [(1-37.2/3860)(1-37.2/6590)]$^{0.25}$=0.278
εy=(My,ed/Ned)(A/Wel)=([10^{3}]x57.93/37.20)x(9882.0/1500.0x10^{3})=10.26

Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(37.20/9911.0)=1.001, (ψ=1.00) (EC3 Annex A, T.A.1)
$\bar{\lambda}$o=0.870 > $\bar{\lambda}$o,lim=0.278
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
=1.001+(1-1.001)x(√10.259x0.998)/(1+√10.259x0.998) = 1.000
Cmlt=Cmy^2·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=1.000^{2}x0.998/√[(1-37.2/3860.0)(1-37.2/6590.0)]=1.006, Cmlt=1.006

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) =
=0.5x0.998x0.870^2[0.0/(0.872x532.5)](0.0/62.6) = 0.000
Cyy=1+(1.135-1)[(2-1.6x1.000^{2}x0.960/1.135-1.6x1.000^{2}x0.960^2/1.135)x0.011-0.000]=0.999
Cyy>=1500.0x10^3/1702.0x10^3=0.881, Cyy=0.999

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[0.870/(0.1+0.958^4)][0.0/(1.000x0.872x532.5)][0.0/(0.000x62.6)]=0.000
Czy=1+(1.135-1)[(2-14.0x1.000^{2}x0.960^2/1.135^5)0.011-0.000]=0.993
Czy>=0.6√(1.135/1.500)(1500.0x10^3/1702.0x10^3)=0.460, Czy=0.993

Cyy=0.999, Czy=0.993 (Annex A, T.A.1)
kyy=1.000x1.006x1.000/(1-37.20/9911.0)x(1/0.999)=1.011
kzy=1.000x1.006x0.996/(1-37.20/9911.0)x(1/0.993)x0.6x√(1.135/1.500)=0.529

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
37.2/(0.891x3508.1/1.00)+1.011x57.9/(0.872x604.2/1.00)=0.012+0.111=0.123
0.123< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
37.2/(0.624x3508.1/1.00)+0.529x57.9/(0.872x604.2/1.00)=0.017+0.058=0.075
0.075< 1.000, Is verified

11.12.7. Buckling resistance, Beam(1) end-span region (Ultimate Limit State)

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned = 37.2 kN
Ved = 66.3 kN
Myed = 62.8 kNm, Mzed =0.0 kNm
Rafter length Lr=8400 mm
Buckling length,In-plane buckling Lcr,y=8400mm (System length)
Buckling length,Out-of-plane buckling Lcr,z=3901mm (Torsional restrains of beams)

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

Nc,ed=37.20 kN, Lcr,y=8.400 m, Lcr,z=3.901 m
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Buckling lengths: Lcr,y=1.000x8400=8400mm, Lcr,z=0.464x8400=3901mm
Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)
λ̄y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(8400/184.8)x(1/76.06)=0.598
λ̄z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(3901/ 41.2)x(1/76.06)=1.245
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=450/190=2.37>=1.20, tf=14.6mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.891 (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(λ̄y-0.2)+λ̄y^2]=0.5x[1+0.21x(0.598-0.2)+0.598^2]=0.721
χy=1/[Φy+√(Φy^2-λ̄y^2)]=1/[0.721+√(0.721^2-0.598^2)]=0.891 <=1 χy=0.891
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.454
Φz=0.5[1+αz(λ̄z-0.2)+λ̄z^2]=0.5x[1+0.34x(1.245-0.2)+1.245^2]=1.453
χz=1/[Φz+√(Φz^2-λ̄z^2)]=1/[1.453+√(1.453^2-1.245^2)]=0.454 <=1 χz=0.454

Reduction factor χ=1/[Φ+√(Φ^2-λ̄^{2})], χ<=1.0, Φ=0.5[1+α(λ̄-0.2)+λ̄^{2}], χ=0.454 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.454x[10^{-3}]x9882x355/1.00=1592.68kN (EC3 Eq.6.47)
Nc,ed= 37.20 kN < 1592.68 kN =Nb,rd, Is verified
Nc,ed/Nb,rd= 37.20/1592.68= 0.023<1

11.12.9. Lateral torsional buckling, Beam(1) end-span region (ULS) (EN1993-1-1, §6.3.2)

My,ed=62.84 kN, L=8.400m, Lcr,y=8.400m, Lcr,z=3.901m, Lcr,lt=3.901m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
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)}
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
μ=Mo/M=qL²/8M=-31.3/62.8=-0.50, ψ=Mb/Ma=57.9/-62.8=-0.92, C1=1.741, C2=0.388
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^{4} N/mm²
k·L=3901mm, zg=h/2=450/2=225mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=1.741, C2=0.388, C3=0.000
Mcr=[10^{-6}]1.741x[π²x2.1x10^{5}x16.760x10^{6}/3901²]
x{ [(1.0/1.0)²x(791.01x10^{9}/16.760x10^{6})
+3901²x8.1x10^{4}x0.669x10^{6}/(π²x2.1x10^{5}x16.760x10^{6})
+(0.388x225)²]$^{0.5}$-(0.388x225) }= 766.4 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{[10^{-6}]x1702.0x10^{3}x355/766.4}=0.888 (EC3 Eq.6.56)
h/b=450/190=2.37>2.00 buckling curve:c
imperfection factor:α,lt=0.49, β=0.75, χ,lt=0.709 (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(0.888-0.40)+0.75x0.888²]=0.915
χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)]=1/[0.915+√(0.915²-0.75x0.915²)]=0.709
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)], χ,lt<=1.0, 1/$\bar{\lambda}$,lt², χ,lt=0.709 (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/$\bar{\lambda}$,lt²=1/0.888²=1.27 (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($\bar{\lambda}$,lt-0.8)²]=1-0.5x(1-0.752)[1-2.0x(0.888-0.8)²]=0.878, f<=1.0
χ,lt,mod=χ,lt/f=0.709/0.878=0.808, χ,lt,mod<=1.0, χ,lt,mod<=1.27, χ,lt,mod=0.808

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.808x[10^{-6}]x1702.0x10^{3}x355/1.00=488.20kNm (EC3 Eq.6.55)
My,ed= 62.84 kNm < 488.20 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 62.84/488.20= 0.129<1

11.12.10. Axial force and bending moment, Beam(1) end-span region (ULS) (EN1993-1-1, §6.3.3)

Ned=37.20 kN, My,ed=62.84 kNm

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}]x9882x355=3508.1 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x1702.0x10^{3}x355=604.2 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.891x[10^{-3}]x9882x355/1.00=3125.7kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.454x[10^{-3}]x9882x355/1.00=1592.7kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.808x[10^{-6}]x1702.0x10^{3}x355/1.00=488.2kNm

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}]x210000x337.40x10^{6}/ 8400²= 9911 kN
Ncr,z=π²EIz/lcr,z²=3.14²x[10^{-3}]x210000x16.760x10^{6}/ 3901²= 2283 kN
Ncr,t=(1/ip²)x(G·It + π²EIw/Lcr,t²) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10^{-3}]x(1/189²)[80769x 0.669x10^{6}+π²x210000x791.01x10^{9}/3901²]=4513 kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 37.2/ 9911)/(1-0.891x 37.2/ 9911)=1.000
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 37.2/ 2283)/(1-0.454x 37.2/ 2283)=0.991
alt=1-It/Iy>=0=1- 0.669x10^{6}/337.40x10^{6}=0.998 (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 1.702x10^{6}/ 1.500x10^{6}=1.135 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.276x10^{6}/ 0.176x10^{6}=1.567 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=37.20/(3508.10/1.00)=0.011

λ̄max=max(0.598,1.245)=1.250 (EC3 Annex A.1)
Mcr,o=(1.00/1.74)x766.40=440.1, C1=1.00
λ̄o=√([10^{-6}]x1702.0x10^{3}x355/440.1)=1.170
λ̄o,lim=0.2√C1 [(1-Ned/Ncr,z)(1-Ned/Ncr,t)]$^{0.25}$ (EC3 Annex A.1)
λ̄o,lim=0.2√1.741 [(1-37.2/2283)(1-37.2/4513)]$^{0.25}$=0.262
εy=(My,ed/Ned)(A/Wel)=([10^{3}]x62.84/37.20)x(9882.0/1500.0x10^{3})=11.13

Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(37.20/9911.0)=1.001, (ψ=1.00) (EC3 Annex A, T.A.1)
λ̄o=1.170 > λ̄o,lim=0.262
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
=1.001+(1-1.001)x(√11.128x0.998)/(1+√11.128x0.998) = 1.000
Cmlt=Cmy²·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=1.000²x0.998/√[(1-37.2/2283.0)(1-37.2/4513.0)]=1.010, Cmlt=1.010

Cyy=1+(wy-1)[(2-1.6Cmy²·λ̄max/wy-1.6Cmy²·λ̄max²/wy)npl-blt]>=Wel,y/Wpl,y (Annex A, T.A.1)
blt=0.5alt·λ̄o²[My,ed/(χ,lt·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
=0.5x0.998x1.170²[0.0/(0.808x532.5)](0.0/62.6) = 0.000
Cyy=1+(1.135-1)[(2-1.6x1.000²x1.250/1.135-1.6x1.000²x1.250²/1.135)x0.011-0.000]=0.997
Cyy>=1500.0x10^{3}/1702.0x10^{3}=0.881, Cyy=0.997

Czy=1+(wy-1)[(2-14.0Cmy²·λ̄max²/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[1.170/(0.1+1.245^{4})][0.0/(1.000x0.808x532.5)][0.0/(0.000x62.6)]=0.000
Czy=1+(1.135-1)[(2-14.0x1.000²x1.250²/1.135^{5})0.011-0.000]=0.986
Czy>=0.6√(1.135/1.500)(1500.0x10^{3}/1702.0x10^{3})=0.460, Czy=0.986

Cyy=0.997, Czy=0.986 (Annex A, T.A.1)
kyy=1.000x1.010x1.000/(1-37.20/9911.0)x(1/0.997)=1.017
kzy=1.000x1.010x0.991/(1-37.20/9911.0)x(1/0.986)x0.6x√(1.135/1.500)=0.532

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
37.2/(0.891x3508.1/1.00)+1.017x62.8/(0.808x604.2/1.00)=0.012+0.131=0.143
0.143< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
37.2/(0.454x3508.1/1.00)+0.532x62.8/(0.808x604.2/1.00)=0.023+0.068=0.092
0.092< 1.000, Is verified

11.13. Beam verification, Beam(2), (Ultimate Limit State) (EN1993-1-1, §6)

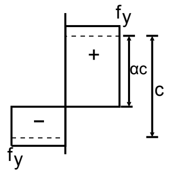
Profile : IPE 450-S 355

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned = 48.0 kN
Ved = 69.9 kN
Myed = 82.5 kNm, Mzed =0.0 kNm
Myed = 65.9 kNm (at mid-span)
Myed = -68.5 kNm (at column face)

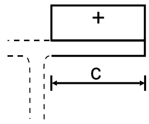
11.13.1. Classification of cross-sections, Beam(2) (EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses σ=Ned/Ael ± Myed/Wel.y ± Mzed/Wel.z
σ=[10^{3}]48.00/9882 ± [10^{6}]82.50/1500.0x10³ ± [10^{6}]0.00/176.4x10³
σ1=60 N/mm², σ2=-50 N/mm² (compression positive)

Web
c=450.0-2x14.6-2x21.0=378.8 mm, t=9.4 mm, c/t=378.8/9.4=40.30
S 355 , t= 9.4<= 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)=48000/(2x9.4x355/1.00)=7.2 mm
α=(378.8/2+7.2)/378.8=0.519>0.5
c/t=40.30<=396x0.81/(13x0.519-1)=55.82
The web is class 1 (EN1993-1-1, Tab.5.2)



Flange
c=190.0/2-9.4/2-21.0=69.3 mm, t=14.6 mm, c/t=69.3/14.6=4.75
S 355 , t=14.6<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
c/t=4.75<=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

11.13.2. Resistance of cross-section, Beam(2) (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. 221: 1.35xGk+1.50Qk+1.05Hk
Nc.ed= 48.00 kN
Compression Resistance Nplrd= A·fy/γM0=[10^{-3}]x9882x355/1.00=3508.11kN
Ned= 48.00 kN < 3508.11 kN =Nc,rd=Nplrd, Is verified
Ned/Nc,rd= 48.00/3508.11= 0.014<1

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
My.ed= 82.50 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x1702.0x10^{3}x355/1.00= 604.21kNm
My,ed= 82.50 kNm < 604.21 kNm =My,rd=Mply,rd, Is verified
My,ed/My,rd= 82.50/604.21= 0.137<1

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)
Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Vz.ed= 69.90 kN
Av=A-2b·tf+(tw+2r)tf=9882-2x190.0x14.6+(9.4+2x21.0)x14.6=5084mm² (EC3 §6.2.6.3)
Av= 5084mm² > η·hw·tw= 1.00x(450.0-2x14.6)x9.4=1.00x435.4x9.4= 4093mm²
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x5084x(355/1.73)/1.00= 1042.10kN
Vz,ed= 69.90 kN < 1042.10 kN =Vz,rd=Vpl,z,rd, Is verified
Vz,ed/Vz,rd= 69.90/1042.10= 0.067<1

hw/tw=(450.0-2x14.6)/9.4=435.4/9.4=46.32<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t= 9.4<= 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. 221: 1.35xGk+1.50Qk+1.05Hk
N.ed= 48.00kN (Compression), Vz.ed= 69.90kN, My.ed= 82.50kNm
Nplrd=3508.11kN, Mpl,y,rd=604.21kNm, Vpl,z,rd=1042.10kN
Ned=48.00kN <= 0.25x3508.11=0.25xNplrd=877.03kN
Ned=48.00kN <= [10^{-3}]x0.5x435.4x9.4x355/1.00=0.5hw·tw·fy/γM0=726.46 kN
n=Ned/Nplrd=48/3508= 0.014
Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=69.90kN <= 0.50x1042.10=0.50xVpl,rd=521.05kN
Effect of shear force is neglected (EC3 §6.2.8.2)

My,ed= 82.50 kNm < 604.21 kNm =Mply,rd, Is verified
My,ed/Mply,rd= 82.50/604.21= 0.137<1

11.13.3. Buckling resistance, Beam(2) mid-span region (Ultimate Limit State)

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned = 48.0 kN
Ved = 68.4 kN
Myed = 65.9 kNm, Mzed =0.0 kNm
Rafter length Lr=8400 mm
Buckling length,In-plane buckling
αcr=18.79, Ned=48.0kN, Lcr,y=π√[EI/αcr·Ned] <= Lr=8400 mm
Lcr,y=π√[210000x337.40x10^{6}/(18.79x48.0x10^{3})=27847mm], Lcr,y=8400mm
Buckling length,In-plane buckling Lcr,y=8400mm (System length)
Buckling length,Out-of-plane buckling Lcr,z=3000mm (Crossbeam spacing)

11.13.4. Flexural Buckling, Beam(2) mid-span region (Ultimate Limit State) (EN1993-1-1, §6.3.1)

Nc,ed=47.99 kN, Lcr,y=8.400 m, Lcr,z=3.000 m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Buckling lengths: Lcr,y=1.000x8400=8400mm, Lcr,z=0.357x8400=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)=(8400/184.8)x(1/76.06)=0.598
$\bar{\lambda}$z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(3000/ 41.2)x(1/76.06)=0.958
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=450/190=2.37>=1.20, tf=14.6mm<=40 mm
y-y buckling curve:a, imperfection factor:αy=0.21, χy=0.891 (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy($\bar{\lambda}$y-0.2)+$\bar{\lambda}$y²]=0.5x[1+0.21x(0.598-0.2)+0.598²]=0.721
χy=1/[Φy+√(Φy²-$\bar{\lambda}$y²)]=1/[0.721+√(0.721²-0.598²)]=0.891 <=1 χy=0.891
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.624
Φz=0.5[1+αz($\bar{\lambda}$z-0.2)+$\bar{\lambda}$z²]=0.5x[1+0.34x(0.958-0.2)+0.958²]=1.088
χz=1/[Φz+√(Φz²-$\bar{\lambda}$z²)]=1/[1.088+√(1.088²-0.958²)]=0.624 <=1 χz=0.624

Reduction factor χ=1/[Φ+√(Φ²-$\bar{\lambda}$²)], χ<=1.0, Φ=0.5[1+α($\bar{\lambda}$-0.2)+$\bar{\lambda}$²], χ=0.624 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.624x[10^{-3}]x9882x355/1.00=2189.06kN (EC3 Eq.6.47)
Nc,ed= 47.99 kN < 2189.06 kN =Nb,rd, Is verified
Nc,ed/Nb,rd= 47.99/2189.06= 0.022<1

11.13.5. Lateral torsional buckling, Beam(2) mid-span region (ULS) (EN1993-1-1, §6.3.2)

My,ed=65.88 kN, L=8.400m, Lcr,y=8.400m, Lcr,z=3.000m, Lcr,lt=3.000m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
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)}
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
μ=Mo/M=qL²/8M=18.5/65.9=0.28, ψ=Mb/Ma=-62.4/65.9=-0.95, C1=1.996, C2=0.178
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^{4} N/mm²
k·L=3000mm, zg=h/2=450/2=225mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=1.996, C2=0.178, C3=0.000
Mcr=[10^{-6}]1.996x[π²x2.1x10^{5}x16.760x10^{6}/3000²]
x{ [(1.0/1.0)²x(791.01x10^{9}/16.760x10^{6})
+3000²x8.1x10^{4}x0.669x10^{6}/(π²x2.1x10^{5}x16.760x10^{6})
+(0.178x225)²]$^{0.5}$-(0.178x225) }= 1622.2 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{[10^{-6}]x1702.0x10^{3}x355/1622.2}=0.610 (EC3 Eq.6.56)
h/b=450/190=2.37>2.00 buckling curve:c
imperfection factor:α,lt=0.49, β=0.75, χ,lt=0.880 (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(0.610-0.40)+0.75x0.610²]=0.691
χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)]=1/[0.691+√(0.691²-0.75x0.691²)]=0.880
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt²-β$\bar{\lambda}$,lt²)], χ,lt<=1.0, 1/$\bar{\lambda}$,lt², χ,lt=0.880 (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/$\bar{\lambda}$,lt²=1/0.610²=2.68 (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(0.610-0.8)²]=1.000, f<=1.0
χ,lt,mod=χ,lt/f=0.880/1.000=0.880, χ,lt,mod<=1.0, χ,lt,mod<=2.68, χ,lt,mod=0.880

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.880x[10^{-6}]x1702.0x10^{3}x355/1.00=531.70kNm (EC3 Eq.6.55)
My,ed= 65.88 kNm < 531.70 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 65.88/531.70= 0.124<1

11.13.6. Axial force and bending moment, Beam(2) mid-span region (ULS) (EN1993-1-1, §6.3.3)
Ned=47.99 kN, My,ed=65.88 kNm

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}]x9882x355=3508.1 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x1702.0x10^{3}x355=604.2 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.891x[10^{-3}]x9882x355/1.00=3125.7kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.624x[10^{-3}]x9882x355/1.00=2189.1kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.880x[10^{-6}]x1702.0x10^{3}x355/1.00=531.7kNm

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=π^{2}EIy/lcr,y^2=3.14^{2}x[10^{-3}]x210000x337.40x10^{6}/ 8400^2= 9911 kN
Ncr,z=π^{2}EIz/lcr,z^2=3.14^{2}x[10^{-3}]x210000x16.760x10^{6}/ 3000^2= 3860 kN
Ncr,t=(1/ip^2)x(G·It + π^{2}EIw/Lcr,t^2) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10^{-3}]x(1/189^2)[80769x 0.669x10^{6}+π^{2}x210000x791.01x10^{9}/3000^2]=6590 kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 48.0/ 9911)/(1-0.891x 48.0/ 9911)=0.999
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 48.0/ 3860)/(1-0.624x 48.0/ 3860)=0.995
alt=1-It/Iy>=0=1- 0.669x10^{6}/337.40x10^{6}=0.998 (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 1.702x10^{6}/ 1.500x10^{6}=1.135 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.276x10^{6}/ 0.176x10^{6}=1.567 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=47.99/(3508.10/1.00)=0.014

$\bar{\lambda}$max=max(0.598,0.958)=0.960 (EC3 Annex A.1)
Mcr,o=(1.00/2.00)x1622.20=812.7, C1=1.00
$\bar{\lambda}$o=√([10^{-6}]x1702.0x10^{3}x355/812.7)=0.860
$\bar{\lambda}$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√1.996 [(1-48.0/3860)(1-48.0/6590)]$^{0.25}$=0.281
εy=(My,ed/Ned)(A/Wel)=([10^{3}]x65.88/47.99)x(9882.0/1500.0x10^{3})=9.04

Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(47.99/9911.0)=1.001, (ψ=1.00) (EC3 Annex A, T.A.1)
$\bar{\lambda}$o=0.860 > $\bar{\lambda}$o,lim=0.281
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
=1.001+(1-1.001)x(√9.044x0.998)/(1+√9.044x0.998) = 1.000
Cmlt=Cmy^2·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=1.000^{2}x0.998/√[(1-48.0/3860.0)(1-48.0/6590.0)]=1.008, Cmlt=1.008

Cyy=1+(wy-1)[(2-1.6Cmy^2·$\bar{\lambda}$max/wy-1.6Cmy^2·$\bar{\lambda}$max^2/wy)npl-blt]>=Wel,y/Wpl,y (Annex A, T.A.1)
blt=0.5alt·$\bar{\lambda}$o^2[My,ed/(χ,lt·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
=0.5x0.998x0.860^2[0.0/(0.880x532.5)](0.0/62.6) = 0.000
Cyy=1+(1.135-1)[(2-1.6x1.000^{2}x0.960/1.135-1.6x1.000^{2}x0.960^2/1.135)x0.014-0.000]=0.999
Cyy>=1500.0x10^{3}/1702.0x10^{3}=0.881, Cyy=0.999

Czy=1+(wy-1)[(2-14.0Cmy^2·$\bar{\lambda}$max^2/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[0.860/(0.1+0.958^{4})][0.0/(1.000x0.880x532.5)][0.0/(0.000x62.6)]=0.000
Czy=1+(1.135-1)[(2-14.0x1.000^{2}x0.960^2/1.135^{5})0.014-0.000]=0.991
Czy>=0.6√(1.135/1.500)(1500.0x10^{3}/1702.0x10^{3})=0.460, Czy=0.991

Cyy=0.999, Czy=0.991 (Annex A, T.A.1)
kyy=1.000x1.008x0.999/(1-47.99/9911.0)x(1/0.999)=1.013
kzy=1.000x1.008x0.995/(1-47.99/9911.0)x(1/0.991)x0.6x√(1.135/1.500)=0.531

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

$N_{ed}/(\chi_y \cdot N_{rk}/\gamma_{M1})+k_{yy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1})=$ (EC3 Eq.6.61)

48.0/(0.891x3508.1/1.00)+1.013x65.9/(0.880x604.2/1.00)=0.015+0.126=0.141

0.141< 1.000, Is verified

$N_{ed}/(\chi_z \cdot N_{rk}/\gamma_{M1})+k_{zy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1})=$ (EC3 Eq.6.62)

48.0/(0.624x3508.1/1.00)+0.531x65.9/(0.880x604.2/1.00)=0.022+0.066=0.088

0.088< 1.000, Is verified

11.13.7. Buckling resistance, Beam(2) end-span region (Ultimate Limit State)

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

Ned = 48.0 kN

Ved = 68.4 kN

Myed = 68.9 kNm, Mzed =0.0 kNm

Rafter length Lr=8400 mm

Buckling length,In-plane buckling Lcr,y=8400mm (System length)

Buckling length,Out-of-plane buckling Lcr,z=3901mm (Torsional restrains of beams)

11.13.8. Flexural Buckling, Beam(2) end-span region (Ultimate Limit State) (EN1993-1-1, §6.3.1)

Nc,ed=47.99 kN, Lcr,y=8.400 m, Lcr,z=3.901 m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

Buckling lengths: Lcr,y=1.000x8400=8400mm, Lcr,z=0.464x8400=3901mm

Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)

$\bar{\lambda}_y=\sqrt{(A \cdot f_y/N_{cr,y})}=(L_{cr,y}/i_y)\cdot(1/\lambda_1)=(8400/184.8)x(1/76.06)=0.598$

$\bar{\lambda}_z=\sqrt{(A \cdot f_y/N_{cr,z})}=(L_{cr,z}/i_z)\cdot(1/\lambda_1)=(3901/ 41.2)x(1/76.06)=1.245$

$\lambda_1=\pi\sqrt{(E/f_y)}=93.9\varepsilon=76.06$, $\varepsilon=\sqrt{(235/f_y)}=0.81$

h/b=450/190=2.37>=1.20, tf=14.6mm<=40 mm

y-y buckling curve:a, imperfection factor: $\alpha_y=0.21$, $\chi_y=0.891$ (T.6.2,T.6.1,Fig.6.4)

$\Phi_y=0.5[1+\alpha_y(\bar{\lambda}_y-0.2)+\bar{\lambda}_y^2]=0.5x[1+0.21x(0.598-0.2)+0.598^2]=0.721$

$\chi_y=1/[\Phi_y+\sqrt{(\Phi_y^2-\bar{\lambda}_y^2)}]=1/[0.721+\sqrt{(0.721^2-0.598^2)}]=0.891 <=1$ $\chi_y=0.891$

z-z buckling curve:b, imperfection factor: $\alpha_z=0.34$, $\chi_z=0.454$

$\Phi_z=0.5[1+\alpha_z(\bar{\lambda}_z-0.2)+\bar{\lambda}_z^2]=0.5x[1+0.34x(1.245-0.2)+1.245^2]=1.453$

$\chi_z=1/[\Phi_z+\sqrt{(\Phi_z^2-\bar{\lambda}_z^2)}]=1/[1.453+\sqrt{(1.453^2-1.245^2)}]=0.454 <=1$ $\chi_z=0.454$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi<=1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.454$ (EC3 Eq.6.49)

$N_{b,rd}=\chi \cdot A \cdot f_y/\gamma_{M1}=$ 0.454x[10^{-3}]x9882x355/1.00=1592.68kN (EC3 Eq.6.47)

Nc,ed= 47.99 kN < 1592.68 kN =Nb,rd, Is verified

Nc,ed/Nb,rd= 47.99/1592.68= 0.030<1

11.13.9. Lateral torsional buckling, Beam(2) end-span region (ULS) (EN1993-1-1, §6.3.2)

My,ed=68.89 kN, L=8.400m, Lcr,y=8.400m, Lcr,z=3.901m, Lcr,lt=3.901m

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk

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

$M_{cr}=C_1 \cdot [\pi^2 E I_z/(kL)^2]\{\sqrt{[(k_z/k_w)^2(I_w/I_z)+(kL)^2 G I_t/(\pi^2 E I_z)+(C_2 \cdot z_g-C_3 \cdot z_j)^2]} -(C_2 \cdot z_g-C_3 \cdot z_j)\}$

Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006

$\mu=M_o/M=qL^2/8M=-31.3/68.9=-0.45$, $\psi=M_b/M_a=65.7/-68.9=-0.95$, C1=1.777, C2=0.345

$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4$ N/mm²

k·L=3901mm, zg=h/2=450/2=225mm, zj=0mm (EN1993:2002 Eq.C.11)

ky=1.0, kz=1.0, kw=1.0, C1=1.777, C2=0.345, C3=0.000

$M_{cr}=[10^{-6}]1.777x[\pi^2 x2.1x10^5 x16.760x10^6/3901^2]$

$x\{ [(1.0/1.0)^2 x(791.01x10^9/16.760x10^6)$

$+3901^2 x8.1x10^4 x0.669x10^6/(\pi^2 x2.1x10^5 x16.760x10^6)$

$+(0.345x225)^2]^{0.5}-(0.345x225) \}=$ 809.8 kNm

$\bar{\lambda}_{,lt}=\sqrt{(W_{pl,y} \cdot f_y/M_{cr})}=\sqrt{\{[10^{-6}]x1702.0x10^3 x355/809.8\}}=0.864$ (EC3 Eq.6.56)

h/b=450/190=2.37>2.00 buckling curve:c

imperfection factor: $\alpha_{,lt}=0.49$, $\beta=0.75$, $\chi_{,lt}=0.724$ (T.6.3, T.6.5, Fig.6.4)

$\Phi_{,lt}=0.5[1+\alpha_{,lt}(\bar{\lambda}_{,lt}-\bar{\lambda}_{,lto})+\beta\bar{\lambda}_{,lt}^2]=0.5x[1+0.49x(0.864-0.40)+0.75x0.864^2]=0.893$

$\chi_{,lt}=1/[\Phi_{,lt}+\sqrt{(\Phi_{,lt}^2-\beta\bar{\lambda}_{,lt}^2)}]=1/[0.893+\sqrt{(0.893^2-0.75x0.893^2)}]=0.724$

Reduction factor $\chi_{,lt}=1/[\Phi_{,lt}+\sqrt{(\Phi_{,lt}^2-\beta\bar{\lambda}_{,lt}^2)}]$, $\chi_{,lt}<=1.0$, $1/\bar{\lambda}_{,lt}^2$, $\chi_{,lt}=0.724$ (Eq.6.57)

χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/ $\bar{\lambda}$,lt²=1/0.864²=1.34 (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($\bar{\lambda}$,lt-0.8)²]=1-0.5x(1-0.752)[1-2.0x(0.864-0.8)²]=0.877, f<=1.0
χ,lt,mod=χ,lt/f=0.724/0.877=0.826, χ,lt,mod<=1.0, χ,lt,mod<=1.34, χ,lt,mod=0.826

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.826x[10^{-6}]x1702.0x10^{3}x355/1.00=499.08kNm (EC3 Eq.6.55)
My,ed= 68.89 kNm < 499.08 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 68.89/499.08= 0.138<1

11.13.10. Axial force and bending moment, Beam(2) end-span region (ULS) (EN1993-1-1, §6.3.3)
Ned=47.99 kN, My,ed=68.89 kNm

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}]x9882x355=3508.1 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x1702.0x10^{3}x355=604.2 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.891x[10^{-3}]x9882x355/1.00=3125.7kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.454x[10^{-3}]x9882x355/1.00=1592.7kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.826x[10^{-6}]x1702.0x10^{3}x355/1.00=499.1kNm

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}]x210000x337.40x10^{6}/ 8400²= 9911 kN
Ncr,z=π²EIz/lcr,z²=3.14²x[10^{-3}]x210000x16.760x10^{6}/ 3901²= 2283 kN
Ncr,t=(1/ip²)x(G·It + π²EIw/Lcr,t²) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10^{-3}]x(1/189²)[80769x 0.669x10^{6}+π²x210000x791.01x10^{9}/3901²]=4513 kN

μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 48.0/ 9911)/(1-0.891x 48.0/ 9911)=0.999
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 48.0/ 2283)/(1-0.454x 48.0/ 2283)=0.988
alt=1-It/Iy>=0=1- 0.669x10^{6}/337.40x10^{6}=0.998 (EC3 Annex A.1)

wy=Wpl,y/Wel,y<=1.50, wy= 1.702x10^{6}/ 1.500x10^{6}=1.135 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.276x10^{6}/ 0.176x10^{6}=1.567 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=47.99/(3508.10/1.00)=0.014

$\bar{\lambda}$max=max(0.598,1.245)=1.250 (EC3 Annex A.1)
Mcr,o=(1.00/1.78)x809.80=455.7, C1=1.00
$\bar{\lambda}$o=√([10^{-6}]x1702.0x10^{3}x355/455.7)=1.150
$\bar{\lambda}$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√1.777 [(1-48.0/2283)(1-48.0/4513)]$^{0.25}$=0.264
εy=(My,ed/Ned)(A/Wel)=([10^{3}]x68.89/47.99)x(9882.0/1500.0x10^{3})=9.46

Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(47.99/9911.0)=1.001, (ψ=1.00) (EC3 Annex A, T.A.1)
$\bar{\lambda}$o=1.150 > $\bar{\lambda}$o,lim=0.264
Cmy=Cmy,o+(1-Cmy,o)(√εy·alt)/(1+√εy·alt)=
=1.001+(1-1.001)x(√9.457x0.998)/(1+√9.457x0.998) = 1.000
Cmlt=Cmy²·alt/√[(1-Ned/Ncr,z)(1-Ned/Ncr,t)] >=1
Cmlt=1.000²x0.998/√[(1-48.0/2283.0)(1-48.0/4513.0)]=1.014, Cmlt=1.014

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) =
=0.5x0.998x1.150²[0.0/(0.826x532.5)](0.0/62.6) = 0.000
Cyy=1+(1.135-1)[(2-1.6x1.000²x1.250/1.135-1.6x1.000²x1.250²/1.135)x0.014-0.000]=0.996
Cyy>=1500.0x10^{3}/1702.0x10^{3}=0.881, Cyy=0.996

```
Czy=1+(wy-1)[(2-14.0Cmy²·λ̄max²/wy⁵)npl-dlt]>=0.6√(wy/wz)(Wel,y/Wpl,y)          (Annex A, T.A.1)
dlt=2alt·[λ̄o/(0.1+λ̄z⁴)][My,ed/(Cmy·χ,lt·Mpl,y,rd)][Mz,ed/(Cmz·Mpl,z,rd)]=
   =20.998x[1.150/(0.1+1.245⁴)][0.0/(1.000x0.826x532.5)][0.0/(0.000x62.6)]=0.000
Czy=1+(1.135-1)[(2-14.0x1.000²x1.250²/1.135⁵)0.014-0.000]=0.982
Czy>=0.6√(1.135/1.500)(1500.0x10³/1702.0x10³)=0.460, Czy=0.982

Cyy=0.996, Czy=0.982                                                              (Annex A, T.A.1)
kyy=1.000x1.014x0.999/(1-47.99/9911.0)x(1/0.996)=1.022
kzy=1.000x1.014x0.988/(1-47.99/9911.0)x(1/0.982)x0.6x√(1.135/1.500)=0.535

Maximum design values. Verification for load case: L.C. 221: 1.35xGk+1.50Qk+1.05Hk
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)=                                          (EC3 Eq.6.61)
48.0/(0.891x3508.1/1.00)+1.022x68.9/(0.826x604.2/1.00)=0.015+0.141=0.156
               0.156< 1.000,  Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)=                                          (EC3 Eq.6.62)
48.0/(0.454x3508.1/1.00)+0.535x68.9/(0.826x604.2/1.00)=0.030+0.074=0.104
               0.104< 1.000,  Is verified
```

Connections

11.14. Connection data (EN1993-1-8)

11.14.1. Bolt connection data (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 Eave connecion     190x585x20 mm, S 235
                            190x585x20 mm, S 235
Bolts                      M24, Strength grade 10.9
 Bolt diameter             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)
```

11.14.2. Edge distances and spacing of bolts (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=4x13.5+40=95 mm
                                          e2=4t+40=4x13.5+40=95 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(14x13.5,200)=190 mm
                                          p2=min(14t,200)=min(14x13.5,200)=190 mm
Distance of plate edge to bolt line       e1=e2=ex= 45 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=0.55x14.6= 8 mm
Web to end-plate weld        aw>= 0.55tw=0.55x 9.4= 6 mm
```

11.14.3. Design resistance of individual bolts (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 $F_{t,rd}=k_2 \cdot f_{ub} \cdot A_s/\gamma_{M2}$, ($k_2$=0.90)

$F_{t,rd}=[10^{-3}]$x0.90x1000x353.0/1.25=254 kN

Shear resistance of bolts $F_{v,rd}=\alpha_v \cdot f_{ub} \cdot A_s/\gamma_{M2}$, ($\alpha_v$=0.50)

$F_{v,rd}=[10^{-3}]$x0.50x1000x353.0/1.25=141 kN

11.14.4. Edge distances and spacing of bolts (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=4x13.5+40=95 mm
                                          e2=4t+40=4x13.5+40=95 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(14x13.5,200)=190 mm
                                          p2=min(14t,200)=min(14x13.5,200)=190 mm
Distance of plate edge to bolt line       e1=e2=ex= 45 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=0.55x14.6= 8 mm
Web to end-plate weld        aw>= 0.55tw=0.55x 9.4= 6 mm
```

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

Bolt strength grade=10.9, f_{ub} =1000N/mm², A_s=353.0mm², γ_{M2}=1.25

Tension resistance of bolts $F_{t,rd}=k_2 \cdot f_{ub} \cdot A_s/\gamma_{M2}$, ($k_2$=0.90)

$F_{t,rd}=[10^{-3}]\times0.90\times1000\times353.0/1.25=254$ kN

Shear resistance of bolts $F_{v,rd}=\alpha_\nu \cdot f_{ub} \cdot A_s/\gamma_{M2}$, ($\alpha_\nu$=0.50)

$F_{v,rd}=[10^{-3}]\times0.50\times1000\times353.0/1.25=141$ kN

11.15. Eave connection(1)

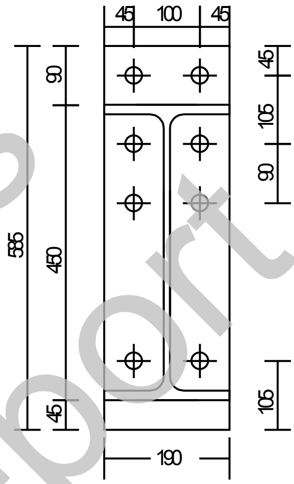
11.15.1. Basic data (Eave connection(1))

Design forces of connection (Eave connection(1))

Maximum design values for actions (L.C. 221: 1.35Gk+1.50Qk+0.70x1.50Hk= 1.35xGk+1.50Qk+1.05Hk)

Ned = -37.2 kN
Ved = 71.9 kN
Med = -84.8 kNm

11.15.2. Connection data (Eave connection(1))



Bolt connection data

End Plate	190x585x20 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= 45 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=0.55x14.6= 8 mm
Web to end-plate weld	aw>= 0.55tw=0.55x 9.4= 6 mm

Compression stiffener at the bottom of haunch

Compression stiffener with thickness ts= 20.0 mm

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

e=ex=45 mm, emin=45 mm
mx,x=(100-9.4-2x0.8x6x√2)/2= 38.5 mm
mx,y=45-0.8x8x√2= 35.9 mm
nx,x= emin <=1.25mx,x = min(45.0,1.25x38.5=48.1)= 45.0 mm
nx,y= emin <=1.25mx,y = min(45.0,1.25x35.9=44.9)= 44.9 mm
min(mx,x, mx,y)=min(38.5,35.9)=35.9mm, max(mx,x, mx,y)=max(38.5,35.9)=38.5mm
min(nx,x, nx,y)=min(45.0,44.9)=44.9mm, max(nx,x, nx,y)=max(45.0,44.9)=45.0mm

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

Bolt-row outside tension flange of beam

leff=2π·mx =2πx35.9= 225.6 mm
=π·mx+w =πx35.9+100.0= 212.8 mm
=π·mx+2e=πx35.9+2x45.0= 202.8 mm
=4mx+1.25ex = 4x35.9+1.25x45.0=199.9 mm
=e+2mx+0.625ex = 45.0+2x35.9+0.625x45.0=144.9 mm
=0.5bp = 0.5x190= 95.0 mm
=0.5w+2mx+0.625ex=0.5x100.0+2x35.9+0.625x45.0= 149.9 mm
leff,1b=min(225.6, 212.8, 202.8, 199.9, 144.9, 95.0, 149.9)= 95.0 mm
leff,1b= 95.0 mm

Bolt next to tension flange alone

leff=2π·mx =2πx35.9= 225.6 mm
=α·m =6.28x35.9=225.6 mm (λ1=λ2=m/(m+e)=0.44, α=6.28) (EC3-1-8 Fig.6.11)
leff,2b=min(225.6, 225.6)= 225.6 mm
leff,2b= = 225.6 mm

Bolt next to tension flange in a group

leff=2π·mx =2πx35.9= 225.6 mm
=α·m =6.28x35.9=225.6 mm (λ1=λ2=m/(m+e)=0.44, α=6.28)
=πm+p =πx35.9+90.0= 202.8 mm
=0.5p+α·m-(2m+0.625e)=0.5x90.0+6.3x35.9-(2x35.9+0.625x45.0)= 170.6 mm
leff,3b=min(225.6, 225.6, 202.8, 170.6)= 170.6 mm
leff,3b= = 170.6 mm

Inner Bolt-row in a group

leff=2π·mx =2πx38.5= 241.9 mm
=4m+1.25e =4x38.5+1.25x45.0= 210.3 mm
=2p =2x90.0= 180.0 mm
=p = 90.0 mm
leff,4b=min(241.9, 210.3, 180.0, 90.0)= 90.0 mm
leff,4b= = 90.0 mm

11.15.5. End-Plate, Resistance of T-stub flange (Eave connection(1)) (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}]x0.25x95.0x20.0²x235/1.00=2.233kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x2.233/35.9 = 249 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x2.233+44.9x2x254)/(35.9+44.9)=338kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(249, 338, 508)= 249 kN

Bolt next to tension flange alone

Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x225.6x20.0²x235/1.00=5.302kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x5.302/35.9 = 591 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x5.302+44.9x2x254)/(35.9+44.9)=414kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(591, 414, 508)= 414 kN

Bolt next to tension flange in a group

Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x170.6x20.0²x235/1.00=4.009kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x4.009/35.9 = 447 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x4.009+44.9x2x254)/(35.9+44.9)=382kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(447, 382, 508)= 382 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]x4x2.115/38.5 = 220 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x2.115+45.0x2x254)/(38.5+45.0)=324kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(220, 324, 508)= 220 kN

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

Fc,fb,rd= Mc,rd/(h-tf), Mc,rd= Wel,y·fy/γM0
Wel,y=(190x14.6x435.4²+9.4x420.8³/6)/450=1428.0x10^3 mm³
Mc,rd= [10^{-6}]x1428.0x10^3x355/1.00=507kNm, Fc,fb,rd=[10^3]x507/435.4= 1164kN
Fc,fb,rd,max=b·t·fy/γM0= [10^{-3}]x190.0x14.6x235/1.00= 652 kN (h<=600mm)
Fc,fb,rd= min(1164, 652)= 652 kN

11.15.7. Rafter web in tension (Eave connection(1)) (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(170.6, 90.0) =90.0 mm
Ft,wb,rd = [10^{-3}]x90.0x9.4x355/1.00= 300 kN

min Ft,rd=min(249, 414, 382, 220, 300)= 220 kN

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

e=ex=40 mm, emin=40 mm
mx,x=(100-8.6-2x0.8x21)/2= 28.9 mm
mx,y=45-0.8x8x√2= 35.9 mm
nx,x= emin <=1.25mx,x = min(40.0,1.25x28.9=36.1)= 36.1 mm
nx,y= emin <=1.25mx,y = min(40.0,1.25x35.9=44.9)= 40.0 mm
min(mx,x, mx,y)=min(28.9,35.9)=28.9mm, max(mx,x, mx,y)=max(28.9,35.9)=35.9mm
min(nx,x, nx,y)=min(36.1,40.0)=36.1mm, max(nx,x, nx,y)=max(36.1,40.0)=40.0mm

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

End Bolt-row in a group
leff=2π·m =2πx28.9= 181.6 mm
=π·m+2e1 =πx28.9+2x45.0= 180.8 mm
=4m+1.25e =4x28.9+1.25x40.0= 165.6 mm
=2m+0.63e+e1 =2x28.9+0.63x40.0+45.0= 127.8 mm
=π·m+p =πx28.9+90.0= 180.8 mm
=2e1+p =2x45.0+90.0= 180.0 mm
=2m+0.63e+0.5p =2x28.9+0.63x40.0+0.5x90.0= 127.8 mm
=e1+0.5p =45.0+0.5x90.0= 90.0 mm
leff,1c=min(181.6, 180.8, 165.6, 127.8, 180.8, 180.0, 127.8, 90.0)= 90.0 mm
leff,1c= = 90.0 mm

Inner Bolt-row in a group
leff=2π·m =2πx28.9= 181.6 mm
=4m+1.25e =4x28.9+1.25x40.0= 165.6 mm
=2p =2x90.0= 180.0 mm
=p = 90.0 mm
leff,2c=min(181.6, 165.6, 180.0, 90.0)= 90.0 mm
leff,2c= = 90.0 mm

11.15.10. Column-Side, Resistance of T-stub flange (Eave connection(1)) (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=$[10^{-6}]$x0.25x90.0x13.5^{2}x355/1.00=1.456kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =$[10^{3}]$x4x1.456/28.9 = 202 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=($[10^{3}]$x2x1.456+36.1x2x254)/(28.9+36.1)=327kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(202, 327, 508)= 202 kN

Inner Bolt-row in a group
Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=$[10^{-6}]$x0.25x90.0x13.5^{2}x355/1.00=1.456kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =$[10^{3}]$x4x1.456/28.9 = 202 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=($[10^{3}]$x2x1.456+36.1x2x254)/(28.9+36.1)=327kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(202, 327, 508)= 202 kN

11.15.11. Column-web in transverse tension (Eave connection(1)) (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(90.0x8.6/4269)^2]]= 0.98
Ft,wc,rd = $[10^{-3}]$x0.98x90.0x8.6x355/1.00= 269 kN

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

Compression stiffener at the bottom of haunch ts= 20.0 mm

fy=235N/mm², bs=(190-8.6-2x21.0)/2=69.7mm, ts=20.0mm, tw=8.6mm, ε=√(235/fy)=0.81
Aeff,s=2x69.7x20.0+(2x15x0.81x8.6+20.0)x8.6=4757 mm² (EC3-1-5 §9.1(2)
Leff,s=min(69.7, 14x0.81x20.0)=min(69.7, 226.80)=69.7 mm, (EC3 Tab.5.2)
Ieff,s=(2x69.7+8.6)^{3}x20.0/12= 5403.0x10^3 mm^4
ieff,s=√(5403x10³/4757) =33.7mm, λ1=π√(E/fy)=93.9ε=76.06

Lcr=0.75x(400-2x13.5)= 279.8 mm (EC3-1-5 §9.4(2)
$\bar{\lambda}$=Lcr/(ieff,s·λ1)=279.8/(33.7x76.06)= 0.11 (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= 1.000x4757x235/1.00=1118 kN > Fc,fb,rd=652 kN
Compression stiffener, Is verified

11.15.13. Moment resistance of connection (Eave connection(1)) (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= 487.7 mm, Ft,rd= 249 kN
Bolt-row 2, hr= 383.1 mm, Ft,rd= 382 kN
Bolt-row 3, hr= 293.1 mm, Ft,rd= 220 kN
Fc,ed=ΣFt,rd=249 + 382 + 220= 851 kN

End-plate in bending (EC3-1-8 §6.2.4.4
Force distribution in bolt rows
Bolt-row 1, hr= 487.7 mm, Ft,rd= 202 kN
Bolt-row 2, hr= 383.1 mm, Ft,rd= 202 kN
Bolt-row 3, hr= 293.1 mm, Ft,rd= 202 kN
Fc,ed=ΣFt,rd=202 + 202 + 202= 606 kN

Rafter web in tension (EC3-1-8 §6.2.6.8)
Ft,wb,rd = 300 kN
Rafter flange and web in compression (EC3-1-8 §6.2.4.7
Fc,fb,rd= 652 kN
Ft,rd<=Ft,wb,rd=300 kN, Fc,ed=ΣFt,rd<=Fc,fb,rd=652 kN
Fc,ed=ΣFt,rd<=Fc,wc,rd=1118 kN

Force distribution in bolt rows (EC3-1-8 §6.2.7.2.(7))
Bolt-row 1, hr= 487.7 mm, Ft,rd= 202 kN
Bolt-row 2, hr= 383.1 mm, Ft,rd= 202 kN
Bolt-row 3, hr= 293.1 mm, Ft,rd= 202 kN
Fc,ed=ΣFt,rd=202 + 202 + 202= 606 kN

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

Mj,rd =[10^{-3}]x[202x487.7+202x383.1+202x293.1]
Mj,rd = 235 kNm
Med= 84.8 kNm < 235.1 kNm =Mj,rd, Is verified

11.15.14. Shear resistance (Eave connection(1)) (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
$F_{b,rd}=k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t/\gamma_{M2}$

End-Plate
t=20.0mm, d=24mm, do=26mm, e1=45mm, e2=45mm, 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, 45/(3x26), 90/(3x26)-0.25]=0.58
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x45/26-1.7,1.4x100/26-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= $[10^{-3}]$x2.50x0.58x360x24x20.0/1.25=199 kN

Column-Side
t=13.5mm, d=24mm, do=26mm, e1=45mm, e2=45mm, 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, 45/(3x26), 90/(3x26)-0.25]=0.58
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x45/26-1.7,1.4x100/26-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= $[10^{-3}]$x2.50x0.58x510x24x13.5/1.25=191 kN

Design resistance of one bolt in shear =min(141, 199, 191)=141 kN

Bending moment and shear (EN1993-1-8, §3.6.1 Tab.3.4)
Maximum tension force in bolts
Ft,ed=202/2=101 kN
Reduction of shear resistance due to bending
ρ=1-Ft,ed/1.40Ft,rd=1-101/(1.40x254)=0.72
Shear acting together with bending moment for all the bolts
Vrd=8x0.72x141 =812 kN

Ved= 72 kN < 812 kN =Vrd, Is verified

11.16. Eave connection(2)

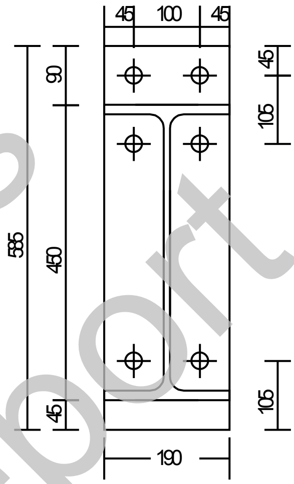
11.16.1. Basic data (Eave connection(2))

Design forces of connection (Eave connection(2))

Maximum design values for actions (L.C. 221: 1.35Gk+1.50Qk+0.70x1.50Hk= 1.35xGk+1.50Qk+1.05Hk)

Ned = -48.0 kN
Ved = 69.9 kN
Med = -68.6 kNm

11.16.2. Connection data (Eave connection(2))



Bolt connection data

End Plate	190x585x20 mm, S 235
Bolts	M24, Bolt strength grade 10.9
Number of Bolts	top 2x2=4
	bottom 2x1=2
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= 45 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=0.55x14.6= 8 mm
Web to end-plate weld	aw>= 0.55tw=0.55x 9.4= 6 mm

Compression stiffener at the bottom of haunch

Compression stiffener with thickness ts= 20.0 mm

11.16.3. Connection geometry of end-plate (Eave connection(2)) (EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=45 mm, emin=45 mm
mx,x=(100-9.4-2x0.8x6x√2)/2= 38.5 mm
mx,y=45-0.8x8x√2= 35.9 mm
nx,x= emin <=1.25mx,x = min(45.0,1.25x38.5=48.1)= 45.0 mm
nx,y= emin <=1.25mx,y = min(45.0,1.25x35.9=44.9)= 44.9 mm
min(mx,x, mx,y)=min(38.5,35.9)=35.9mm, max(mx,x, mx,y)=max(38.5,35.9)=38.5mm
min(nx,x, nx,y)=min(45.0,44.9)=44.9mm, max(nx,x, nx,y)=max(45.0,44.9)=45.0mm

11.16.4. Effective lengths of end-plate (Eave connection(2)) (EC3-1-8 §6.2.6.5 Tab.6.6)

Bolt-row outside tension flange of beam

leff=2π·mx =2πx35.9= 225.6 mm
=π·mx+w =πx35.9+100.0= 212.8 mm
=π·mx+2e=πx35.9+2x45.0= 202.8 mm
=4mx+1.25ex = 4x35.9+1.25x45.0=199.9 mm
=e+2mx+0.625ex = 45.0+2x35.9+0.625x45.0=144.9 mm
=0.5bp = 0.5x190= 95.0 mm
=0.5w+2mx+0.625ex=0.5x100.0+2x35.9+0.625x45.0= 149.9 mm
leff,1b=min(225.6, 212.8, 202.8, 199.9, 144.9, 95.0, 149.9)= 95.0 mm
leff,1b= 95.0 mm

Bolt next to tension flange alone

leff=2π·mx =2πx35.9= 225.6 mm
=α·m =6.28x35.9=225.6 mm (λ1=λ2=m/(m+e)=0.44, α=6.28) (EC3-1-8 Fig.6.11)
leff,2b=min(225.6, 225.6)= 225.6 mm
leff,2b= = 225.6 mm

Bolt next to tension flange in a group
leff=2π·mx =2πx35.9= 225.6 mm
=α·m =6.28x35.9=225.6 mm (λ1=λ2=m/(m+e)=0.44, α=6.28)
=πm+p =πx35.9+90.0= 202.8 mm
=0.5p+α·m-(2m+0.625e)=0.5x90.0+6.3x35.9-(2x35.9+0.625x45.0)= 170.6 mm
leff,3b=min(225.6, 225.6, 202.8, 170.6)= 170.6 mm
leff,3b= = 170.6 mm

Inner Bolt-row in a group
leff=2π·mx =2πx38.5= 241.9 mm
=4m+1.25e =4x38.5+1.25x45.0= 210.3 mm
=2p =2x90.0= 180.0 mm
=p = 90.0 mm
leff,4b=min(241.9, 210.3, 180.0, 90.0)= 90.0 mm
leff,4b= = 90.0 mm

11.16.5. End-Plate, Resistance of T-stub flange (Eave connection(2)) (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}]x0.25x95.0x20.0²x235/1.00=2.233kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x2.233/35.9 = 249 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x2.233+44.9x2x254)/(35.9+44.9)=338kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(249, 338, 508)= 249 kN

Bolt next to tension flange alone
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x225.6x20.0²x235/1.00=5.302kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x5.302/35.9 = 591 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x5.302+44.9x2x254)/(35.9+44.9)=414kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(591, 414, 508)= 414 kN

Bolt next to tension flange in a group
Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x170.6x20.0²x235/1.00=4.009kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x4.009/35.9 = 447 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x4.009+44.9x2x254)/(35.9+44.9)=382kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(447, 382, 508)= 382 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}]x4x2.115/38.5 = 220 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x2.115+45.0x2x254)/(38.5+45.0)=324kN
Mode 3 Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(220, 324, 508)= 220 kN

11.16.6. Rafter flange and web in compression (Eave connection(2)) (EC3-1-8 §6.2.6.7)
Fc,fb,rd= Mc,rd/(h-tf), Mc,rd= Wel,y·fy/γM0
Wel,y=(190x14.6x435.4²+9.4x420.8³/6)/450=1428.0x10^{3} mm³
Mc,rd= [10^{-6}]x1428.0x10^{3}x355/1.00=507kNm, Fc,fb,rd=[10^{3}]x507/435.4= 1164kN
Fc,fb,rd,max=b·t·fy/γM0= [10^{-3}]x190.0x14.6x235/1.00= 652 kN (h<=600mm)
Fc,fb,rd= min(1164, 652)= 652 kN

11.16.7. Rafter web in tension (Eave connection(2)) (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(170.6, 90.0) =90.0 mm
Ft,wb,rd = [10^{-3}]x90.0x9.4x355/1.00= 300 kN

min Ft,rd=min(249, 414, 382, 220, 300)= 220 kN

11.16.8. Connection geometry of column-side (Eave connection(2)) (EC3-1-8 §6.2.4.1, Fig.6.2)

```
e=ex=40 mm, emin=40 mm
mx,x=(100-8.6-2x0.8x21)/2= 28.9 mm
mx,y=45-0.8x8x√2= 35.9 mm
nx,x= emin <=1.25mx,x = min(40.0,1.25x28.9=36.1)= 36.1 mm
nx,y= emin <=1.25mx,y = min(40.0,1.25x35.9=44.9)= 40.0 mm
min(mx,x, mx,y)=min(28.9,35.9)=28.9mm, max(mx,x, mx,y)=max(28.9,35.9)=35.9mm
min(nx,x, nx,y)=min(36.1,40.0)=36.1mm, max(nx,x, nx,y)=max(36.1,40.0)=40.0mm
```

11.16.9. Effective lengths of column-side (Eave connection(2)) (EC3-1-8 §6.2.6.4 Tab.6.4)

End Bolt-row in a group

```
 leff=2π·m  =2πx28.9= 181.6 mm
     =π·m+2e1 =πx28.9+2x45.0= 180.8 mm
     =4m+1.25e =4x28.9+1.25x40.0= 165.6 mm
     =2m+0.63e+e1 =2x28.9+0.63x40.0+45.0= 127.8 mm
     =π·m+p =πx28.9+90.0= 180.8 mm
     =2e1+p =2x45.0+90.0= 180.0 mm
     =2m+0.63e+0.5p =2x28.9+0.63x40.0+0.5x90.0= 127.8 mm
     =e1+0.5p =45.0+0.5x90.0= 90.0 mm
leff,1c=min(181.6, 180.8, 165.6, 127.8, 180.8, 180.0, 127.8, 90.0)= 90.0 mm
leff,1c=  = 90.0 mm
```

Inner Bolt-row in a group

```
 leff=2π·m  =2πx28.9= 181.6 mm
     =4m+1.25e =4x28.9+1.25x40.0= 165.6 mm
     =2p =2x90.0= 180.0 mm
     =p = 90.0 mm
leff,2c=min(181.6, 165.6, 180.0, 90.0)= 90.0 mm
leff,2c=  = 90.0 mm
```

11.16.10. Column-Side, Resistance of T-stub flange (Eave connection(2)) (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]x0.25x90.0x13.5^2x355/1.00=1.456kNm
Mode 1   Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x1.456/28.9 = 202 kN
Mode 2   Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x1.456+36.1x2x254)/(28.9+36.1)=327kN
Mode 3   Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(202, 327, 508)=  202 kN
```

Inner Bolt-row in a group

```
Mpl,1,rd=Mpl,2,rd=0.25leff·tf^2·fy/γM0=[10^-6]x0.25x90.0x13.5^2x355/1.00=1.456kNm
Mode 1   Ft,1,rd= 4Mpl,1rd/m =[10^3]x4x1.456/28.9 = 202 kN
Mode 2   Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^3]x2x1.456+36.1x2x254)/(28.9+36.1)=327kN
Mode 3   Ft,3,rd= ΣFt,rd =2x254 = 508 kN
Ft,rd= min(202, 327, 508)=  202 kN
```

11.16.11. Column-web in transverse tension (Eave connection(2)) (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(90.0x8.6/4269)^2]]= 0.98
Ft,wc,rd = [10^-3]x0.98x90.0x8.6x355/1.00= 269 kN
```

11.16.12. Design resistance of compression stiffener (Eave connection(2)) (EC3-1-5 §9.1)

Compression stiffener at the bottom of haunch ts= 20.0 mm

fy=235N/mm², bs=(190-8.6-2x21.0)/2=69.7mm, ts=20.0mm, tw=8.6mm, ε=√(235/fy)=0.81
Aeff,s=2x69.7x20.0+(2x15x0.81x8.6+20.0)x8.6=4757 mm² (EC3-1-5 §9.1(2)
Leff,s=min(69.7, 14x0.81x20.0)=min(69.7, 226.80)=69.7 mm, (EC3 Tab.5.2)
Ieff,s=(2x69.7+8.6)^{3}x20.0/12= 5403.0x10^3 mm^4
ieff,s=√(5403x10³/4757) =33.7mm, λ1=π√(E/fy)=93.9ε=76.06

Lcr=0.75x(400-2x13.5)= 279.8 mm (EC3-1-5 §9.4(2)
$\bar{\lambda}$=Lcr/(ieff,s·λ1)=279.8/(33.7x76.06)= 0.11 (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= 1.000x4757x235/1.00=1118 kN > Fc,fb,rd=652 kN
Compression stiffener, Is verified

11.16.13. Moment resistance of connection (Eave connection(2)) (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

<u>End-plate in bending</u> (EC3-1-8 §6.2.4.5
Force distribution in bolt rows
Bolt-row 1, hr= 487.7 mm, Ft,rd= 249 kN
Bolt-row 2, hr= 383.1 mm, Ft,rd= 414 kN
Fc,ed=ΣFt,rd=249 + 414= 663 kN

<u>End-plate in bending</u> (EC3-1-8 §6.2.4.4
Force distribution in bolt rows
Bolt-row 1, hr= 487.7 mm, Ft,rd= 202 kN
Bolt-row 2, hr= 383.1 mm, Ft,rd= 202 kN
Fc,ed=ΣFt,rd=202 + 202= 404 kN

<u>Rafter web in tension</u> (EC3-1-8 §6.2.6.8)
Ft,wb,rd = 300 kN
<u>Rafter flange and web in compression</u> (EC3-1-8 §6.2.4.7
Fc,fb,rd= 652 kN
Ft,rd<=Ft,wb,rd=300 kN, Fc,ed=ΣFt,rd<=Fc,fb,rd=652 kN
Fc,ed=ΣFt,rd<=Fc,wc,rd=1118 kN

<u>Force distribution in bolt rows</u> (EC3-1-8 §6.2.7.2.(7))
Bolt-row 1, hr= 487.7 mm, Ft,rd= 202 kN
Bolt-row 2, hr= 383.1 mm, Ft,rd= 202 kN
Fc,ed=ΣFt,rd=202 + 202= 404 kN

<u>Moment resistance of connection</u> (EN1993-1-8, §6.2.7.2(10)

Mj,rd =[10^{-3}]x[202x487.7+202x383.1]
Mj,rd = 176 kNm
Med= 68.6 kNm < 175.9 kNm =Mj,rd, Is verified

11.16.14. Shear resistance (Eave connection(2)) (EN1993-1-8, §3.6.1 Tab.3.4)

<u>Shear resistance of bolts</u>
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

End-Plate

t=20.0mm, d=24mm, do=26mm, e1=45mm, e2=45mm, 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, 45/(3x26), 90/(3x26)-0.25]=0.58
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x45/26-1.7,1.4x100/26-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= $[10^{-3}]$x2.50x0.58x360x24x20.0/1.25=199 kN

Column-Side

t=13.5mm, d=24mm, do=26mm, e1=45mm, e2=45mm, 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, 45/(3x26), 90/(3x26)-0.25]=0.58
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x45/26-1.7,1.4x100/26-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= $[10^{-3}]$x2.50x0.58x510x24x13.5/1.25=191 kN

Design resistance of one bolt in shear =min(141, 199, 191)=141 kN

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

Maximum tension force in bolts
Ft,ed=202/2=101 kN
Reduction of shear resistance due to bending
ρ=1-Ft,ed/1.40Ft,rd=1-101/(1.40x254)=0.72
Shear acting together with bending moment for all the bolts
Vrd=6x0.72x141 =609 kN

Ved= 70 kN < 609 kN =Vrd, Is verified

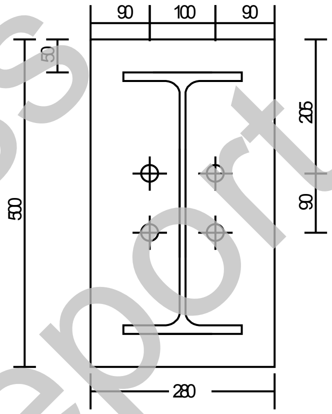
11.17. Column base Connection

11.17.1. Basic data (Base connection)

Design forces of connection (Base connection)

Axial force (compression)   Ned=-148 kN, L.C. 221: 1.35Gk+1.50Qk+0.70x1.50Hk= 1.35xGk+1.50Qk+1.05Hk
Axial force (tension)   Ned= 0 kN,
Shear force   Ved= 11 kN, L.C. 221: 1.35Gk+1.50Qk+0.70x1.50Hk= 1.35xGk+1.50Qk+1.05Hk
Moment   Med= 0 kNm,

Connection data (Base connection)



Base plate steel grade   500x280x30 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 400, S 355
Spacing between cross centers   100 mm
Flange to end-plate weld   8 mm
Web to end-plate weld   6 mm

Edge distances and spacing of bolts

Distance of plate edge to bolt line   e1=e2=ex= 90 mm
Distance of section edge to bolt line   ec= 46 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=0.55x13.5= 8 mm
Web to end-plate weld   aw>= 0.55tw=0.55x 8.6= 6 mm

Concrete of foundation

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)

11.17.2. 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

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

e=ex=90 mm, emin=90 mm
mx,x=(100-8.6-2x0.8x6x√2)/2= 38.9 mm
mx,y= 38.9 mm
nx,x= emin <=1.25mx,x = min(90.0,1.25x38.9=48.6)= 48.6 mm
nx,y= emin <=1.25mx,y = min(90.0,1.25x38.9=48.6)= 48.6 mm
min(mx,x, mx,y)=min(38.9,38.9)=38.9mm, max(mx,x, mx,y)=max(38.9,38.9)=38.9mm
min(nx,x, nx,y)=min(48.6,48.6)=48.6mm, max(nx,x, nx,y)=max(48.6,48.6)=48.6mm

11.17.4. 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 =2πx38.9= 244.4 mm

=4m+1.25e =4x38.9+1.25x90.0= 268.1 mm

=2p =2x90.0= 180.0 mm

=p = 90.0 mm

leff,4b=min(244.4, 268.1, 180.0, 90.0)= 90.0 mm

leff,4b= = 90.0 mm

11.17.5. 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}]x4x4.759/38.9 = 489 kN

Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x4.759+48.6x2x127)/(38.9+48.6)=250kN

Mode 3 Ft,3,rd= ΣFt,rd =2x127 = 254 kN

Ft,rd= min(489, 250, 254)= 250 kN

11.17.6. 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}]x90.0x8.6x355/1.00= 275 kN

min Ft,rd=min(250, 275)= 250 kN

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

Uplift force of connection Ft,ed= 0 kN

Tension resistance of connection Ft,rd= 2x250 = 500 kN

Ned= 0 kN < 500 kN =Nrd, Is verified

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

Shear resistance of bolts

Fv,rd=αν·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=90mm, e2=90mm, 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, 90/(3x26), 90/(3x26)-0.25]=0.90

k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x90/26-1.7,1.4x100/26-1.7,2.5]=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=250/2=125 kN

Reduction of shear resistance due to tension

ρ=1-Ft,ed/1.40Ft,rd=1-125/(1.40x127)=0.30

Shear acting together with tension for all the bolts

Vrd=4x0.30x85 =102 kN

Ved= 11 kN < 102 kN =Vrd, Is verified

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

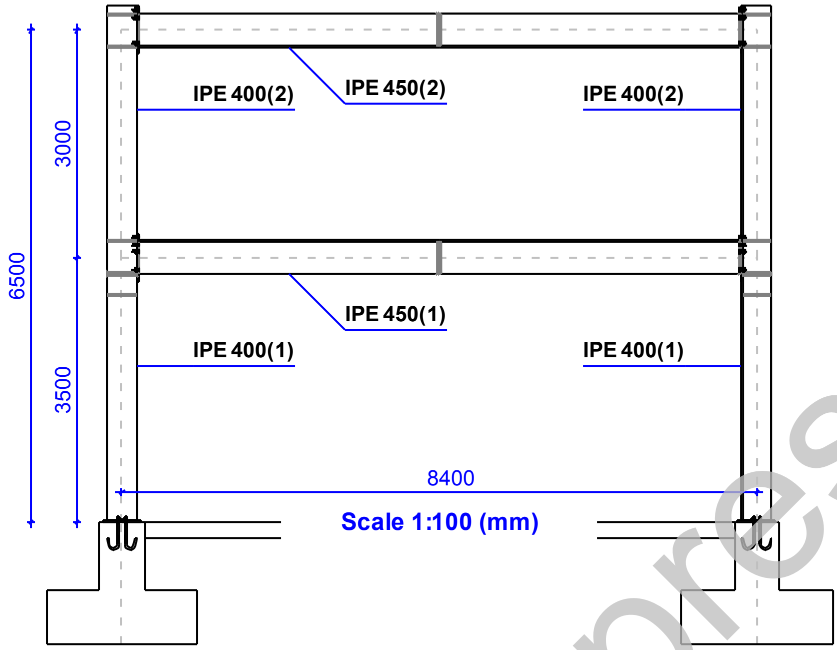
Compression resistance of T-stub flange Fc,rd= fjd·beff·leff (§6.2.5(3)Eq.6.4), §6.2.5(7)
fjd=β·√(Ac1/Aco)·fcd=(2/3)x√(2.25)x16.67=16.67N/mm² (EC2 EN1992-1-1:2004, §6.7,Eq.6.63)
h=400.0mm, b=180.0mm, tf=13.5mm, tw=8.6mm, tp=30.0mm
c=tp·(fy/(3fjd·γM0)$^{0.5}$=30x(235.00/(3x16.67x1.00)$^{0.5}$=65.0, <50.0, c=50.0mm (Eq.6.5)
2c+bf=2x50.0+180=280.0mm <= bp=280mm, leff=280.0mm
Aco,f=leff·(2c+tf)=280.0x(2x50.0+13.5) = 31780 mm² (EC3-1-8 ,Fig.6.4)
Aco,w=(h-2tf-2c)·(tw+2c)= (400.0-2x13.5-2x50.0)x(8.6+2x50.0)= 29648 mm²
Nj,rd= [10^{-3}]x16.7x(2x31780+29648)=[10^{-3}]x16.7x93208= 1557kN
Nj,ed= 148 kN < 1557 kN =Nj,rd, Is verified

Bending resistance of base plate (EN1993-1-8, §6.2.6.10)

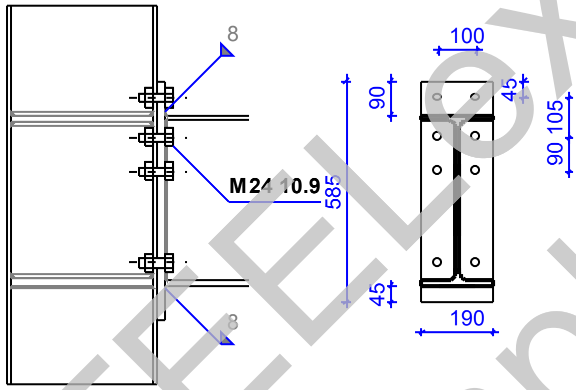
Mp,rd=Wel·fy/γM0=[10^{-6}](280x30.0²/6)x235/1.0= 10 kNm (§6.2.5)
Mp,ed=bp·qed·c²/2=[10^{-6}][280x147531/(2x31780+29648.0)]x50.0²/2= 1 kNm
Mp,ed= 1.0 kNm < 10.0 kNm =Mp,rd, Is verified

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

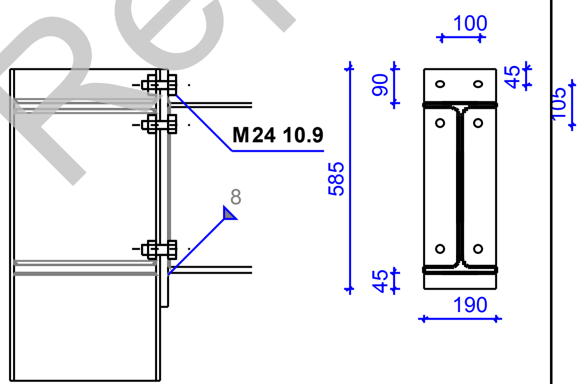
Anchoring hook (EN1992-1-1 §8.4)
Number of fasteners: 4, of diameter 24mm, As= 353mm²
Basic required anchorage length lb,rqd, Design anchorage length lbd=0.70lb,rqd
lb,rqd=(∅/4)(σsd/fbd)=(24/4)x(0.0/1.20)=0 mm
σsd=[10^{3}]x0/(4x353)=0.0 N/mm², fbd=fctd= 1.20 N/mm²
Design anchorage length lbd=0.70x0 >(10x24,100) lbd=250 mm



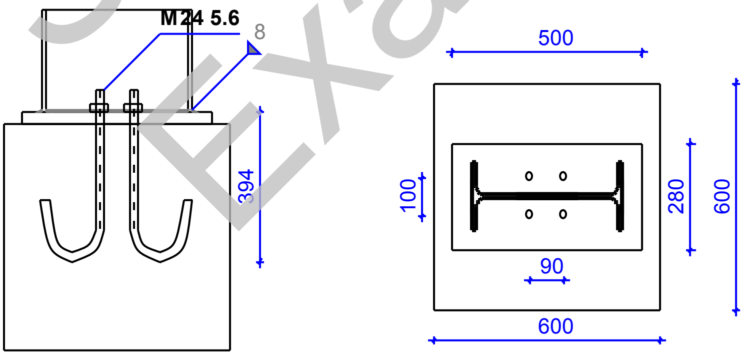
Scale 1:100 (mm)



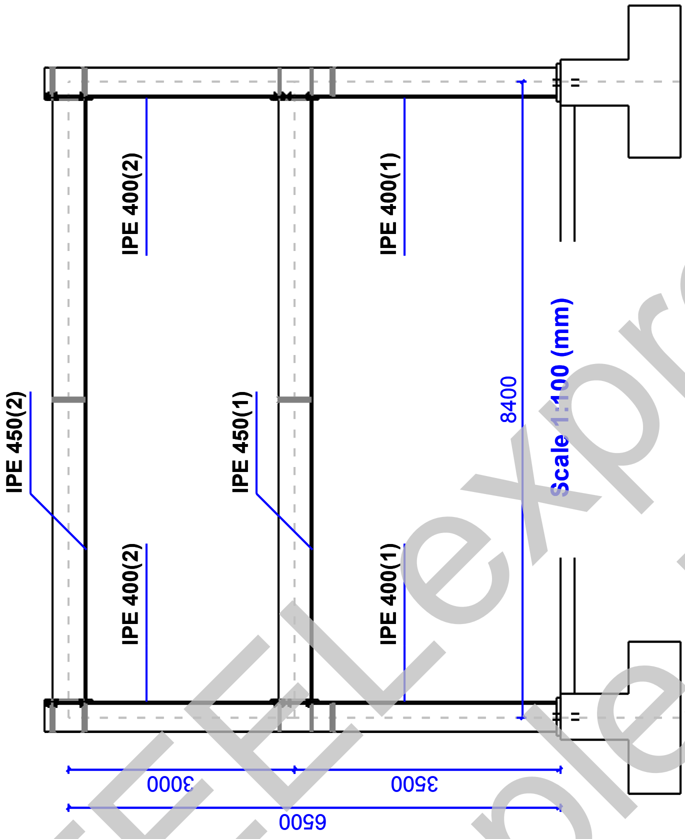
Eave connection(1) Scale 1:20 (mm)

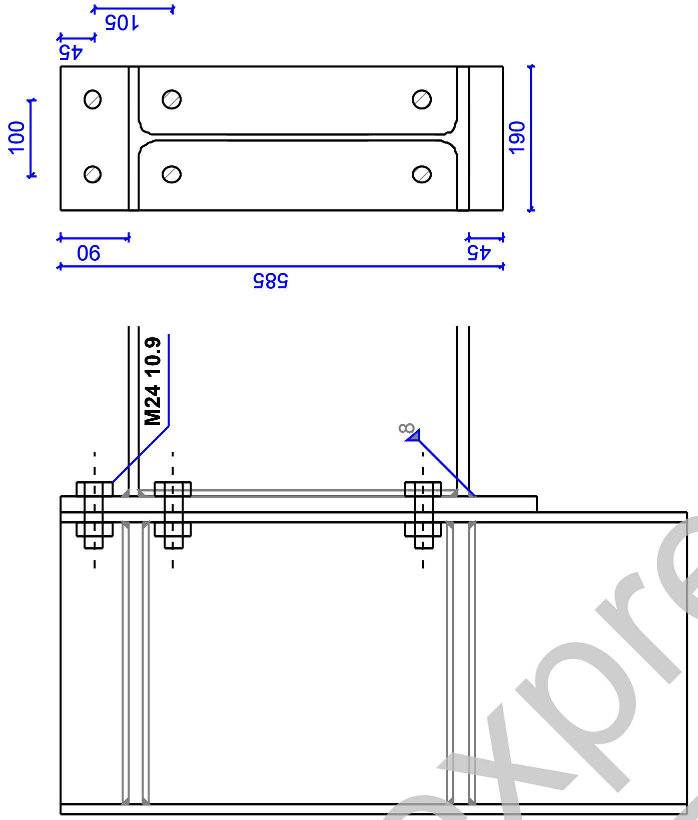


Eave connection(2) Scale 1:20 (mm)

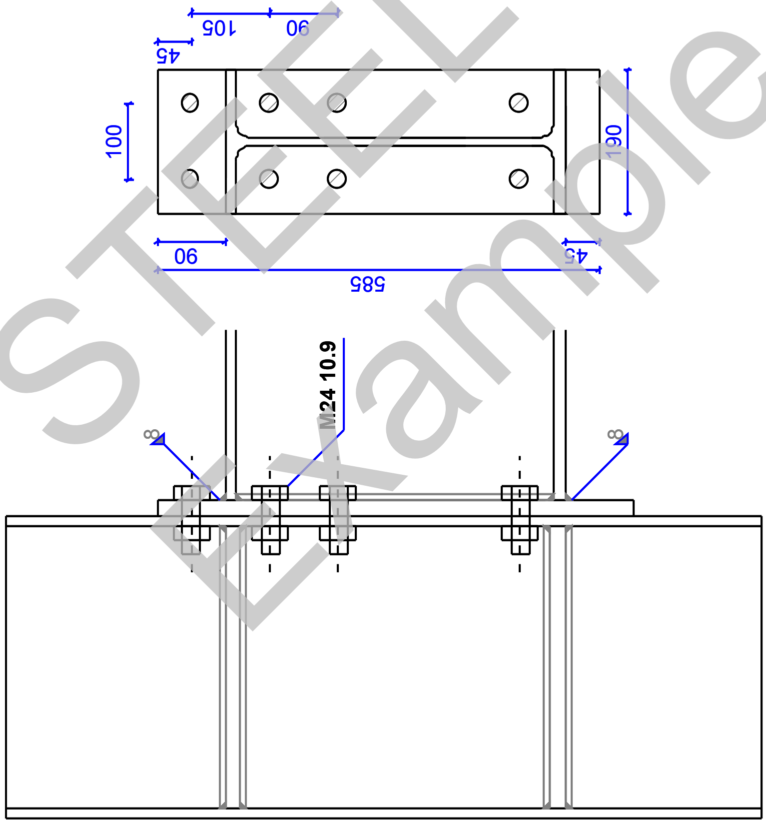


Base connection Scale 1:20 (mm)

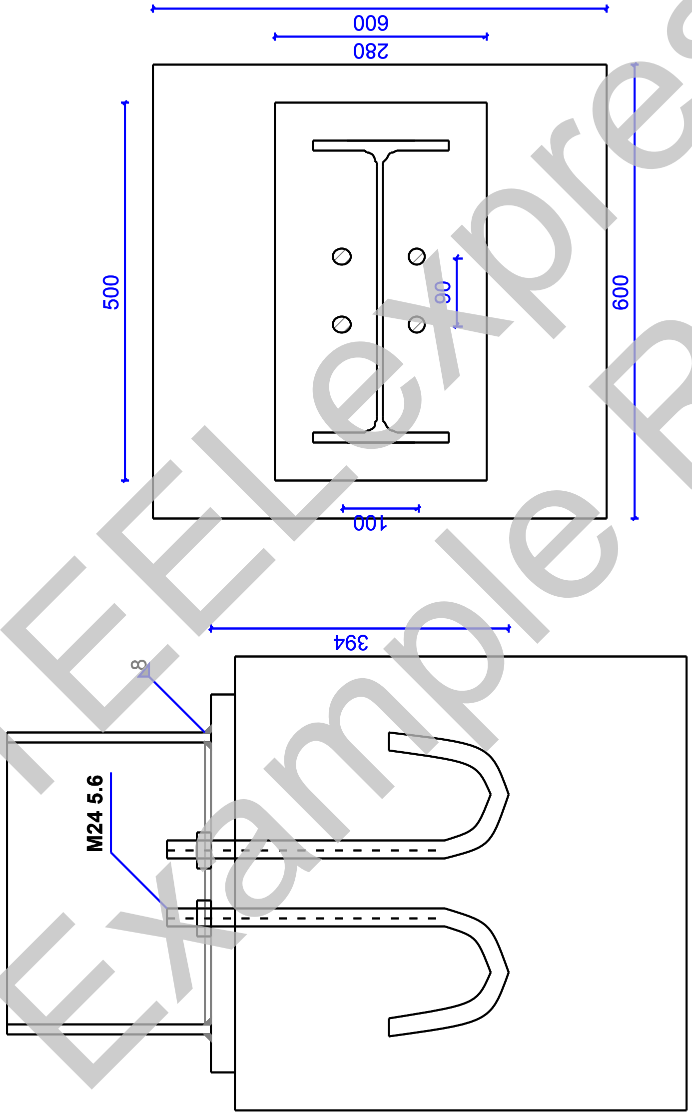

IPE 450(2)
IPE 400(2)
IPE 400(2)
IPE 450(1)
IPE 400(1)
IPE 400(1)
8400
Scale 1:100 (mm)
3000
3500
6500



Eave connection(2) Scale 1:10 (mm)



Eave connection(1) Scale 1:10 (mm)

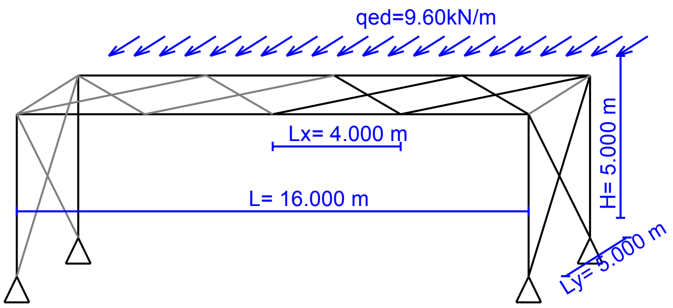


Base connection Scale 1:10 (mm)

12. BRACE-001

Design of bracing systems
(EC3 EN1993-1-1:2005)

Design of lateral bracing system L=16.000m, H=5.000m, Lx=4.000m, Ly=5.000m, qed=9.60kN/m



12.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
EN1991-1-1:2002, Eurocode 1-1 Actions on structures
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

12.2. 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)
γ_G= 1.35, γ_Q= 1.50

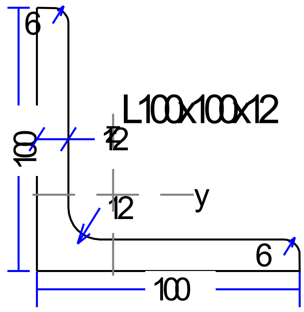
Partial factors for materials (EN1993-1-1, §6.1)
γ_{M0}= 1.00, γ_{M1}= 1.00, γ_{M2}= 1.25

12.3. Dimensions and loads (EN1991-1-1)

Dimensions	L = 16.000 m
	H = 5.000 m
	Lx = 4.000 m
	Ly = 5.000 m
Transverse load/m on roof level	qed = 9.600 kN/m

12.4. Cross-section properties

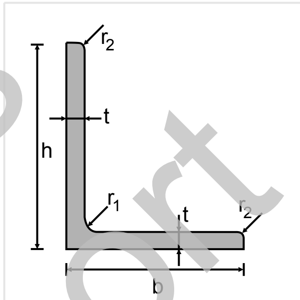
Cross-section L100x100x12-S 355



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=	12.00 mm
Flange thickness	tf=	12.00 mm
Radius of root fillet	r=	12.00 mm
Mass	=	17.80 Kg/m

Properties of cross section



Area	A= 2271 mm^2	
Second moment of area	Iy= $2.067x10^6$ mm^4	Iz= $2.067x10^6$ mm^4
Second moment of area	Iu= $3.280x10^6$ mm^4	Iv= $0.854x10^6$ mm^4
Section modulus	Wy=$29.110x10^3$ mm^3	Wz=$29.110x10^3$ mm^3
Plastic section modulus	Wpy=$122.93x10^3$ mm^3	Wpz=$62.736x10^3$ mm^3
Radius of gyration	iy= 30.2 mm	iz= 30.2 mm
Radius of gyration	iu= 38.0 mm	iv= 19.4 mm
Shear area	Avz= 1215 mm^2	Avy= 1200 mm^2
Torsional constant	It= $0.145x10^6$ mm^4	ip= 43 mm
Warping constant	Iw= $1.936x10^9$ mm^6	

12.5. Horizontal loadings

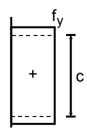
Horizontal (roof) braced girder

Load on bracing system, roof level Qed1= 9.60x4.000 =38.40 kN
The braced girder at roof is loaded with point horizontal loads Qed1=9.60x4.000=38.40kN
at the nodes of the bracing system (at spacing 4.000 m).
The braced girder is supported horizontally at the columns
Length of braced girder members 6.403 m, inclination φ=51.34°, tanφ=5.000/4.000=1.250
Forces in members of braced girder
Compression Nced1= 1.00x38.4/sin51.34= 49.2 kN
Tension Nted1= 1.00x38.4/sin51.34= 49.2 kN

Vertical (wall) braced girder

Load on bracing system, at top of column Qed2= 9.60x16.000/2= 76.80 kN
The vertical brace system is loaded with point horizontal load Qed2=76.80kN
at the top of the column h= 5.000m.
Length of braced girder members 7.071 m, inclination φ=45.00°, tanφ=5.000/5.000=1.000
Forces in bracing members
Tension Nted2= 1.00x76.8/cos45.00= 108.6 kN
Compression on columns Nced2=108.6xsin45.00=76.8kN

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



h/t=100.0/12.0=8.33, (b+h)/2t=(100.0+100.0)/(2x12.0)=8.33
S 355 , t=12.0<= 40 mm, fy=355 N/mm², ε=$(235/355)^{0.5}$=0.81
h/t=8.33<=15ε=12.15, (b+h)/2t=8.33<=11.5ε=9.32

Overall classification of cross-section is Class 3, Compression Nc,ed

12.7. 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=108.60 kN

Tension Resistance Nplrd= A·fy/γM0=[10^{-3}]x2271x355/1.00=806.20kN
Nt,ed= 108.60 kN < 806.20 kN =Nt,rd=Nplrd, Is verified
Nt,ed/Nt,rd= 108.60/806.20= 0.135<1

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

Nc.ed= 49.20 kN

Compression Resistance Nplrd= $A \cdot fy/\gamma M0=[10^{-3}]x2271x355/1.00=806.20kN$

Ned= 49.20 kN < 806.20 kN =Nc,rd=Nplrd, Is verified

Ned/Nc,rd= 49.20/806.20= 0.061<1

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

Nc,ed=49.20 kN, Lcr,y=6.403 m, Lcr,z=6.403 m

Buckling lengths: Lcr,y=1.000x6403=6403mm, Lcr,z=1.000x6403=6403mm

Non-dimensional slenderness (Cross-section Class: 3) (EC3 §6.3.1.3)

$\bar{\lambda}y=\sqrt{(A \cdot fy/Ncr,y)}=(Lcr,y/iy)\cdot(1/\lambda 1)=(6403/\ 30.2)x(1/76.06)=2.790$

$\bar{\lambda}z=\sqrt{(A \cdot fy/Ncr,z)}=(Lcr,z/iz)\cdot(1/\lambda 1)=(6403/\ 30.2)x(1/76.06)=2.790$

$\bar{\lambda}v=\sqrt{(A \cdot fy/Ncr,v)}=(Lcr,v/iv)\cdot(1/\lambda 1)=(6403/\ 19.4)x(1/76.06)=4.341$

$\lambda 1=\pi\sqrt{(E/fy)}=93.9\varepsilon=76.06,\ \varepsilon=\sqrt{(235/fy)}=0.81$

$\bar{\lambda}eff,y=0.50+0.7x2.790=2.453,\ \bar{\lambda}eff,z=0.50+0.7x2.790=2.453$ (EC3 Annex BB.1.2)

$\bar{\lambda}eff,v=0.35+0.7x4.341=3.389$

y-y buckling curve:b, imperfection factor:$\alpha y=0.34,\ \chi y=0.145$ (T.6.2,T.6.1,Fig.6.4)

$\Phi y=0.5[1+\alpha y(\bar{\lambda}y-0.2)+\bar{\lambda}y^2]=0.5x[1+0.34x(2.453-0.2)+2.453^2]=3.892$

$\chi y=1/[\Phi y+\sqrt{(\Phi y^2-\bar{\lambda}y^2)}]=1/[3.892+\sqrt{(3.892^2-2.453^2)}]=0.145 <=1\ \chi y=0.145$

z-z buckling curve:b, imperfection factor:$\alpha z=0.34,\ \chi z=0.145$

$\Phi z=0.5[1+\alpha z(\bar{\lambda}z-0.2)+\bar{\lambda}z^2]=0.5x[1+0.34x(2.453-0.2)+2.453^2]=3.892$

$\chi z=1/[\Phi z+\sqrt{(\Phi z^2-\bar{\lambda}z^2)}]=1/[3.892+\sqrt{(3.892^2-2.453^2)}]=0.145 <=1\ \chi z=0.145$

v-v buckling curve:b, imperfection factor:$\alpha v=0.34,\ \chi v=0.079$

$\Phi v=0.5[1+\alpha v(\bar{\lambda}v-0.2)+\bar{\lambda}v^2]=0.5x[1+0.34x(3.389-0.2)+3.389^2]=6.784$

$\chi v=1/[\Phi v+\sqrt{(\Phi v^2-\bar{\lambda}v^2)}]=1/[6.784+\sqrt{(6.784^2-3.389^2)}]=0.079 <=1\ \chi v=0.079$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}],\ \chi<=1.0,\ \Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2],\ \chi=0.079$ (EC3 Eq.6.49)

$Nb,rd=\chi \cdot A \cdot fy/\gamma M1=\ 0.079x[10^{-3}]x2271x355/1.00=63.69kN$ (EC3 Eq.6.47)

Nc,ed= 49.20 kN < 63.69 kN =Nb,rd, Is verified

Nc,ed/Nb,rd= 49.20/63.69= 0.772<1

12.9. 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=12 mm	
Bolts	M22, Strength grade 8.8	
Bolt diameter	d = 22 mm	
Diameter of holes	do = 24 mm	
Nominal area	$\pi d^2/4=\pi x22^2/4=$ 380.1 mm^2	
Tensile stress area	As = 380.1 mm^2	
Bolt strength grade	8.8, fyb=640N/mm^2, fub=800N/mm^2	(EC3-1-8 §3.1.1)

Shear resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)

$Fv,rd=\alpha v \cdot fub \cdot As/\gamma M2=[10^{-3}]x0.60x800x380.1/1.25=146.0$ kN

Bearing resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)

$Fb,rd=k1 \cdot \alpha b \cdot fu \cdot d \cdot t/\gamma M2$

t=12.0mm, d=22mm, do=24mm, e1=50mm, e2=50mm, p1=100mm, fub=800kN/mm^2, fu=360kN/mm^2,

αb=min[fub/fu, 1, e1/3do, p1/3do-1/4]=min[800/360, 1, 50/(3x24), 100/(3x24)-0.25]=0.69

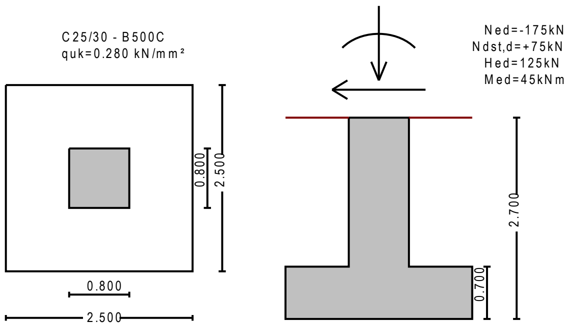
k1=min[2.8e2/do-1.7, 2.5]=min[2.8x50/24-1.7, 2.5]=2.50

$Fb,rd=k1 \cdot \alpha b \cdot fu \cdot d \cdot t/\gamma M2=\ [10^{-3}]x2.50x0.69x360x22x12.0/1.25=132.0$ kN

Necessary bolts per brace 108.6/132.0= 1 M22, Grade 8.8

13. FOOT-001

Footing of steel column, Footing (fixed) N-H-M
(EC3 EN1993-1-1:2005)



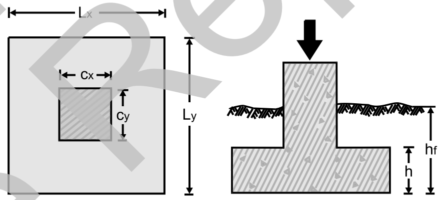
13.1. Design loads on concrete footing

Axial force (downwards)	Ned= 175 kN,
Axial force (upwards)	Ned= 75 kN,
Shear force	Hed= 125 kN,
Moment	Med= 45 kNm,

13.2. Dimensions, materials, loads

Dimensions

Footing	Lx= 2.500 m	Ly= 2.500 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= 6.25 m²
Volume of footing		Vf= 5.66 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

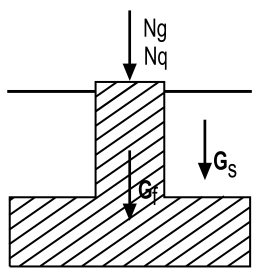
Soil bearing pressure quk= 0.280 N/mm² (MPa)
Unit weight of soil γ=20.000 kN/m³

Loads

Self weight of footing	(1.28+4.38)x25.00	Gf= 141.50 kN
Soil weight on footing	(6.25x2.70-5.66)x20.00	Gs= 224.30 kN

<u>Design Loads</u>

Vertical load downwards	Ned= 175.00 kN
Vertical load upwards	Ndst,d= 75.00 kN
Horizontal load	Hed= 125.00 kN
Moment	Med= 45.00 kNm



Eurocode parameters

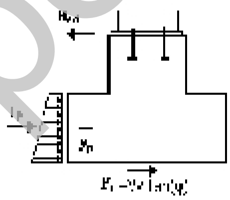
Check of soil bearing capacity (EC7 EN1997-1-1:2004, §6)
Partial factors for actions and soil properties (EC7 Tab. A.1-A.4, EC8-5 §3.1)
Equilibrium limit state (EQU), Structural limit state (STR), Geotechnical limit state (GEO)

			(EQU)	(STR)	(GEO)
Actions	Permanent Unfavorable	γGdst:	1.10	1.35	1.00
	Permanent Favorable	γGstb:	1.00	1.00	1.00
	Variable Unfavorable	γQdst:	1.50	1.50	1.30
	Variable Favorable	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)

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



Angle of shearing resistance of ground φd=φk/γM=35.00/1.25=28.00°
Unit weight of soil γk =20.00 kN/m³
Footing depth hf =2.700 m
Footing height h =0.700 m
Footing width By =2.500 m
Coefficient of passive earth pressure Kp =2.770
Earth pressure at the top p1 =20.00x2.000x2.770=110.79kN/m²
Earth pressure at the bottom p2 =20.00x2.700x2.770=149.57kN/m²
Earth force Fprd =0.5x(110.79+149.57)x2.500x0.700=227.82 kN
Point of application of earth force yp=0.433 m

13.4. Sliding resistance forces at footing base (EC7 EN1997-1-1:2004, §6.5.3(8))

Angle of shearing resistance of ground δk=φcv,k=(2/3)φ=(2/3)x35.00=23.33°
Vertical load Vd= 175.00+1.00x(141.50+224.30)=540.80kN
Resisting forces due to soil friction Rd= Vd·tanδk/γM=540.80xtan(23.33°)/1.25=186.62kkNm

13.5. Failure check against sliding (EC7 EN1997-1-1:2004, §6.5.3)

The horizontal force acting outwards, is resisted by the passive earth pressure acting on the side of the footing, and the friction force at the footing base

Sum of driving forces Hed= 125.00 kN
Sum of resisting forces Hrd= 186.62 + 1.00x227.82= 414.44 kN

Sliding resistance check Hd=125.00kN < Rd=414.44kN, Is verified

13.6. Check stability for forces upwards

Loading (EQU), 1.00xPermanent + 1.50xVariable (EC7 §2.4.7.2)
Vertical forces upwards Ndst,d = 75 kN
Vertical forces downwards Gk =141.50+224.30= 365.80 kN
Holding down forces Nstb,d = γGxGk= 1.00x365.80= 366 kN

Ndst,d=75 kN < 366 kN =Nstb,d, Is verified

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

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

Design Loads

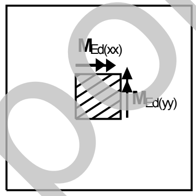
Vertical load at footing bottom   Ned= 175.00+1.35x(141.50+224.30) = 668.83kN
Vertical load at footing top   Ned1= 175.00+1.35x32.00 = 218.20kN
Moment at footing bottom   Med= 45.00+125.00x2.700-1.00x227.82x0.433 = 283.85kNm

relative eccentricity ex/Lx=Myy/(N·Lx)=283.85/(668.83x2.500)=(1/5.891)= 0.170
Eccentricity ec=283.85/668.83=0.424m, ec>2.500/6=0.417m
Soil pressure q=0.216 N/mm^2 Bq=2.477 m
pressure from self weight   q= 10^{-3}fx(668.83-218.20)/(2.50x2.500)= 0.072 N/mm^2
Effective footing L'=2.500-2x0.424= 1.651 m (EC7 Annex D)
Design effective foundation area A'=1.651x2.500=4.13 m^2 (EC7 Annex D)
Soil pressure   q=Ned/A'=10^{-3}fx668.83/(1.65x2.500)=0.162N/mm^2
Soil bearing capacity   Rd=A'·quk/γM=4.128x(10^{3}x0.28)/1.40= 825.50 kN

Ned=668.83kN < 825.50kN =Nrd, Is verified

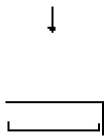
13.8. Design for bending (EC2 EN1992-1-1:2004, §6.1)

Bending at bottom surface



Med(yy)=1000x(0.162-0.072)x2.500x0.850^2/2= 81.28kNm
Med(xx)=0.125x218x2.500x(1-0.800/2.500)^2 = 31.53kNm

Med=81.28kNm, b=2500mm, d=641mm, Kd=11.24, x/d=0.02
εc/εs=0.5/20.0, Ks=2.32, As= 294mm^2
Minimum reinforcement As>=0.26bd·fctm/fyk (As= 607mm^2/m)(EC2 §9.3.1)
Minimum reinforcement ∅14/25.0 (616mm^2/m)



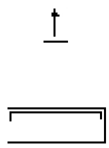
Med=31.53kNm, b=2500mm, d=641mm, Kd=18.05, x/d=0.01
εc/εs=0.3/20.0, Ks=2.31, As= 114mm^2
Minimum reinforcement As>=0.26bd·fctm/fyk (As= 607mm^2/m)
Minimum reinforcement ∅14/25.0 (616mm^2/m)

Reinforcement of footing at bottom surface
Reinforcement in x-x direction: ∅16/32.5 (618mm^2/m), 9∅16 (1809mm^2)
Reinforcement in y-y direction: ∅16/32.5 (618mm^2/m), 9∅16 (1809mm^2)

Bending at top surface

Med(yy)=0.125x75x2.500x(1-0.800/2.500)^2 = 10.84kNm
Med(xx)=0.125x75x2.500x(1-0.800/2.500)^2 = 10.84kNm

Med=10.84kNm, b=2500mm, d=641mm, Kd=30.79, x/d=0.01
εc/εs=0.2/20.0, Ks=2.31, As= 39mm^2
Minimum reinforcement As>=0.26bd·fctm/fyk (As= 607mm^2/m)(EC2 §9.3.1)
Minimum reinforcement ∅14/25.0 (616mm^2/m)



Med=10.84kNm, b=2500mm, d=641mm, Kd=30.79, x/d=0.01
εc/εs=0.2/20.0, Ks=2.31, As= 39mm^2
Minimum reinforcement As>=0.26bd·fctm/fyk (As= 607mm^2/m)
Minimum reinforcement ∅14/25.0 (616mm^2/m)

Reinforcement of footing at top surface
Reinforcement in x-x direction: ∅16/32.5 (618mm^2/m), 9∅16 (1809mm^2)
Reinforcement in y-y direction: ∅16/32.5 (618mm^2/m), 9∅16 (1809mm^2)

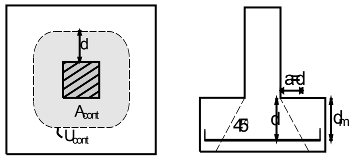
13.9. Design for shear (EC2 EN1992-1-1:2004, §6.2)

The design for shear is covered by the design in punching shear,
because the critical rupture surface is considered at angle θ=45°, tan(θ)=1

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

Footing cantilevers in x-x, L1=0.850>d=0.641m, L2=0.850>d=0.641m
Footing cantilevers in y-y, L1=0.850>d=0.641m, L2=0.850>d=0.641m

Control perimeter at 1.0d=0.641m<2.0d (EC2 §6.4.2.2)
We consider rupture surface at angle θ=45°, tan(θ)=1
Ucont=(0.800+0.800+0.800+0.800)+3.14x(0.641+0.641)=7.225m
Base area within the control perimeter
Acont=0.800x0.800+0.800x1.282+0.800x1.282+3.14x0.641x0.641=3.98m^2
Minimum effective height of footing at control section dm= 641mm



Applied shear force at control perimeter Ved=Ned-σo·Acont, ved=Vedxβ/Ucont
σo=218.20/(2.500x2.500)=34.91 kN/m^2, β=1.15 (EC2 §6.4.3 Fig.6.21N)
ved=(218.20-34.91x3.98)x1.15/7.225=12.61 kN/m

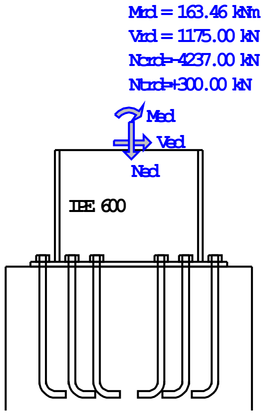
Tension reinforcement at control section Asxx= 6.18cm^2/m, Asyy= 6.18cm^2/m
$As1^2$=(Asxx)(Asyy)=6.18x6.18, As1=6.18 cm^2

<u>Punching shear capacity without shear reinforcement Vrdc</u> (EC2 §6.4.4)
Vrdc=[Crdc·k·(100ρ1·fck)$^{0.33}$·(2d/a)]·bw·d (EC2 Eq.6.50)
Vrdc>=[vmin·2d/a]·bw·d, d=dm=641mm, a=641mm
Crdc=0.18/γc=0.18/1.50=0.120, fck=25MPa ,bw=1000mm, d=641mm
k=1+√(200/d)<=2, k=1.56
ρ1=As1/(bw·d)=618/(1000x641)=0.0010
vmin=0.0350·k$^{1.50}$·√fck = 0.34N/mm^2, (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.34x2x641/641)x1000x641=435.88kN/m
Vrdc=0.001x[0.120x1.56x(0.10x25)$^{0.33}$x2x641/641]x1000x641=325.72, Vrdc=435.88kN/m
Ved=12.61 kN/m <= Vrdc=435.88 kN/m, shear and punching shear OK

14. JOINTd-004

Fixed column base connection

(EC3 EN1993-1-8:2005, §3, §6)

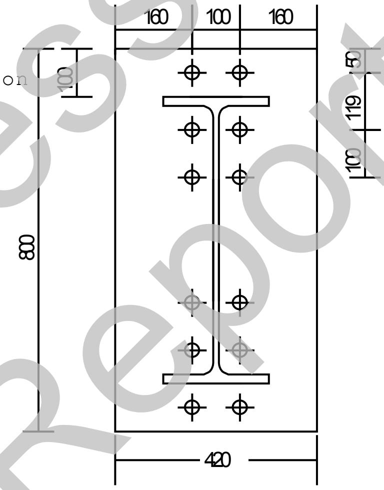


14.1. Basic data (Base connection)

Design forces of connection (Base connection)

Axial force (compression)	Ned=-275 kN,
Axial force (tension)	Ned= 0 kN,
Shear force	Ved= 175 kN,
Moment	Med= 90 kNm,

Connection data (Base connection)



Base plate steel grade	800x420x45 mm, S 235
Anchor bolts	M27, Grade 5.6
Shear plane of bolt	through the threated portion
Number of Bolts	top 2x3=6
	bottom 2x3=6
Total number of bolts	=12
Diameter of holes	do = 30 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= 160 mm
Distance of section edge to bolt line	ec= 44 mm
Distance of flange enge to bolt line	ef= 50 mm
Pitch between bolt rows	p1=p3=p= 100 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

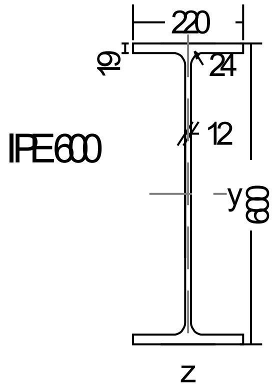
Concrete of foundation

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)

14.2. Column section

Cross-section properties

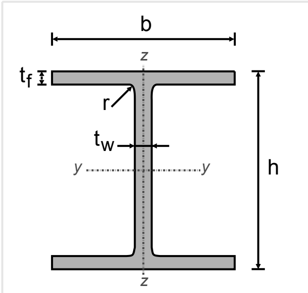
Cross-section IPE 600-S 355



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=920.80x10^6 mm^4	Iz=33.870x10^6 mm^4
Section modulus	Wy=3069.0x10^3 mm^3	Wz=307.90x10^3 mm^3
Plastic section modulus	Wpy=3512.0x10^3 mm^3	Wpz=485.60x10^3 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= 1.654x10^6 mm^4	ip= 247 mm
Warping constant	Iw=2845.5x10^9 mm^6	

14.3. 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=459.0mm², γM2=1.25

Tension resistance of bolts Ft,rd=k2·fub·As/γM2, (k2=0.90)
Ft,rd=[10^{-3}]x0.90x500x459.0/1.25=165 kN

Shear resistance of bolts Fv,rd=αν·fub·As/γM2, (αν=0.60)
Fv,rd=[10^{-3}]x0.60x500x459.0/1.25=110 kN

14.4. Connection geometry of end-plate (Base connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=160 mm, emin=160 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(160.0,1.25x36.1=45.1)= 45.1 mm
nx,y= emin <=1.25mx,y = min(160.0,1.25x36.1=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

14.5. Effective lengths of end-plate (Base connection)

(EC3-1-8 §6.2.6.5 Tab.6.6)

Bolt-row outside tension flange of beam

leff=2π·mx =2πx36.1= 226.8 mm
=π·mx+w =πx36.1+100.0= 213.4 mm
=π·mx+2e=πx36.1+2x160.0= 433.4 mm
=4mx+1.25ex = 4x36.1+1.25x160.0=344.4 mm
=e+2mx+0.625ex = 160.0+2x36.1+0.625x160.0=332.2 mm
=0.5bp = 0.5x420= 210.0 mm
=0.5w+2mx+0.625ex=0.5x100.0+2x36.1+0.625x160.0= 222.2 mm
leff,1b=min(226.8, 213.4, 433.4, 344.4, 332.2, 210.0, 222.2)= 210.0 mm
leff,1b= 210.0 mm

Bolt next to tension flange alone

leff=2π·mx =2πx36.1= 226.8 mm
=α·m =8.00x36.1=288.8 mm (λ1=λ2=m/(m+e)=0.18, α=8.00) (EC3-1-8 Fig.6.11)
leff,2b=min(226.8, 288.8)= 226.8 mm
leff,2b= = 226.8 mm

Bolt next to tension flange in a group

leff=2π·mx =2πx36.1= 226.8 mm
=α·m =8.00x36.1=288.8 mm (λ1=λ2=m/(m+e)=0.18, α=8.00)
=πm+p =πx36.1+100.0= 213.4 mm
=0.5p+α·m-(2m+0.625e)=0.5x100.0+8.0x36.1-(2x36.1+0.625x160.0)= 166.6 mm
leff,3b=min(226.8, 288.8, 213.4, 166.6)= 166.6 mm
leff,3b= = 166.6 mm

Inner Bolt-row in a group

leff=2π·mx =2πx36.1= 226.8 mm
=4m+1.25e =4x36.1+1.25x160.0= 344.4 mm
=2p =2x100.0= 200.0 mm
=p = 100.0 mm
leff,4b=min(226.8, 344.4, 200.0, 100.0)= 100.0 mm
leff,4b= = 100.0 mm

14.6. End-Plate, Resistance of T-stub flange (Base 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}]x0.25x210.0x45.0²x215/1.00=22.857kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x22.857/36.1 = 2533 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x22.857+45.1x2x165)/(36.1+45.1)=746kN
Mode 3 Ft,3,rd= ΣFt,rd =2x165 = 330 kN
Ft,rd= min(2533, 746, 330)= 330 kN

Bolt next to tension flange alone

Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x226.8x45.0²x215/1.00=24.686kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x24.686/36.1 = 2735 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x24.686+45.1x2x165)/(36.1+45.1)=791kN
Mode 3 Ft,3,rd= ΣFt,rd =2x165 = 330 kN
Ft,rd= min(2735, 791, 330)= 330 kN

Bolt next to tension flange in a group

Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x166.6x45.0²x215/1.00=18.133kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x18.133/36.1 = 2009 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x18.133+45.1x2x165)/(36.1+45.1)=630kN
Mode 3 Ft,3,rd= ΣFt,rd =2x165 = 330 kN
Ft,rd= min(2009, 630, 330)= 330 kN

Inner Bolt-row in a group

Mpl,1,rd=Mpl,2,rd=0.25leff·tf²·fy/γM0=[10^{-6}]x0.25x100.0x45.0²x215/1.00=10.884kNm
Mode 1 Ft,1,rd= 4Mpl,1rd/m =[10^{3}]x4x10.884/36.1 = 1206 kN
Mode 2 Ft,2,rd= (2Mpl,2rd+nΣFt,rd)/(m+n)=([10^{3}]x2x10.884+45.1x2x165)/(36.1+45.1)=451kN
Mode 3 Ft,3,rd= ΣFt,rd =2x165 = 330 kN
Ft,rd= min(1206, 451, 330)= 330 kN

14.7. 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 = =100.0 mm
Ft,wb,rd = [10^{-3}]x100.0x12.0x355/1.00= 426 kN

min Ft,rd=min(330, 330, 330, 426)= 330 kN

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

Anchoring hook (EN1992-1-1 §8.4)

d=27mm, As= 459mm², lbd=435mm
lb,rqd= (∅/4)(σsd/fbd), lbd=0.70lb,rqd, fbd=fctd=1.20N/mm², σsd=0.70lbd·fbd/(∅/4)
Ft,anc,rd =σsd·As =[10^{-3}]0.70x435x1.20/(27/4)x459 =25kN

min Ft,rd=min(330, 2x25)= 50 kN

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

Uplift force of connection Ft,ed= 0 kN
Tension resistance of connection Ft,rd= 6x50 = 300 kN
Ned= 0 kN < 300 kN =Nrd, Is verified

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

Compression resistance of T-stub flange Fc,rd= fjd·beff·leff (§6.2.5(3)Eq.6.4), §6.2.5(7)
fjd=β·√(Ac1/Aco)·fcd=(2/3)x√(2.25)x16.67=16.67N/mm² (EC2 EN1992-1-1:2004, §6.7,Eq.6.63)
h=600.0mm, b=220.0mm, tf=19.0mm, tw=12.0mm, tp=45.0mm
c=tp·(fy/(3fjd·γM0)$^{0.5}$=45x(235.00/(3x16.67x1.00)$^{0.5}$=97.5, <100.0, c=97.5mm (Eq.6.5)
2c+bf=2x97.5+220=415.1mm <= bp=420mm, leff=415.1mm
Aco,f=leff·(2c+tf)=415.1x(2x97.5+19.0) = 88871 mm² (EC3-1-8 ,Fig.6.4)
Aco,w=(h-2tf-2c)·(tw+2c)= (600.0-2x19.0-2x97.5)x(12.0+2x97.5)= 75984 mm²
Nj,rd= [10^{-3}]x16.7x(2x88871+75984)=[10^{-3}]x16.7x253726= 4237kN
Nj,ed= 275 kN < 4237 kN =Nj,rd, Is verified

Bending resistance of base plate (EN1993-1-8, §6.2.6.10)

Mp,rd=Wel·fy/γM0=[10^{-6}](420x45.0²/6)x235/1.0= 33 kNm (§6.2.5)
Mp,ed=bp·qed·c²/2=[10^{-6}][420x275000/(2x88871+75984.0)]x97.5²/2= 2 kNm
Mp,ed= 2.0 kNm < 33.0 kNm =Mp,rd, Is verified

14.11. Bending Resistance (Base connection) (EN1993-1-8, §6.2.5

Nj,rd = Aeff·fjd-∑(Ft,rd)
Mj,rd = ∑(Ft,rd·rb) + Aeff·fjd·rc
rb,1=350.0mm, rb,2=231.0mm, rb,3=131.0mm, rc=300.0mm
fjd=β·√(Ac1/Aco)·fcd=(2/3)x1.5x16.67=16.67 N/mm² (EC3-1-8 §6.2.5(7))
∑Ft,rd = (50 +50+50)= 150kN
∑Ft,rd·rb = 0.001x(50x350+50x231+50x131)= 36kNm
Aeff = beff·leff = [10^{3}]x(275+150)/16.67= 25495mm²
h=600.0mm, b=220.0mm, tf=19.0mm, tw=12.0mm, tp=45.0mm
c=tp·(fy/(3fjd·γM0)$^{0.5}$=45x(235.00/(3x16.67x1.00)$^{0.5}$=97.5, <100.0, c=97.5mm (Eq.6.5)
2c+bf=2x97.5+220=415.1mm <= bp=420mm, leff=415.1mm
beff= Aeff/leff= 25495/415.1=61.4mm < tf+2c=19.0+2x97.5=214.1mm, beff=61.4mm
Mj,rd = 36 + [10^{-6}]x415.1x61.4x16.67x300 = 163.5kNm
Mj,ed= 90.0 kNm < 163.5 kNm =Mj,rd, Is verified

Bending resistance of base plate (EN1993-1-8, §6.2.6.10)
Mp,rd=Wel·fy/γM0=[10^{-6}](420x45.0²/6)x235/1.0= 33 kNm (§6.2.5)
Mp,ed=bp·qed·c²/2=[10^{-6}][420x16.67x97.5²/2= 33 kNm
Mp,ed= 33.0 kNm <= 33.0 kNm =Mp,rd, Is verified

14.12. 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}]x0.60x500x459.0/1.25=110 kN
Shear plane of bolt:through the threated portion

Bearing resistance of bolts
Fb,rd=k1·αb·fu·d·t/γM2
t=45.0mm, d=27mm, do=30mm, e1=160mm, e2=160mm, p1=100mm, fub=500kN/mm², fu=360kN/mm²,
αb=min[fub/fu, 1.0, e1/3do, p1/3do-1/4]=
=min[500/360, 1.0, 160/(3x30), 100/(3x30)-0.25]=0.86
k1=min[2.8e2/do-1.7,1.4p2/do-1.7,2.5]=min[2.8x160/30-1.7,1.4x100/30-1.7,2.5]=2.50
Fb,rd=k1·αb·fu·d·t/γM2= [10^{-3}]x2.50x0.86x360x27x45.0/1.25=753 kN

Design resistance of one bolt in shear =min(110, 753)=110 kN

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

Maximum tension force in bolts

$F_{t,ed}=50/2=25$ kN

Reduction of shear resistance due to bending

$\rho=1-F_{t,ed}/1.40F_{t,rd}=1-25/(1.40x165)=0.89$

Shear acting together with bending moment for all the bolts

$V_{rd}=12x0.89x110=1175$ kN

$V_{ed}=175$ kN $<$ 1175 kN $=V_{rd}$, Is verified