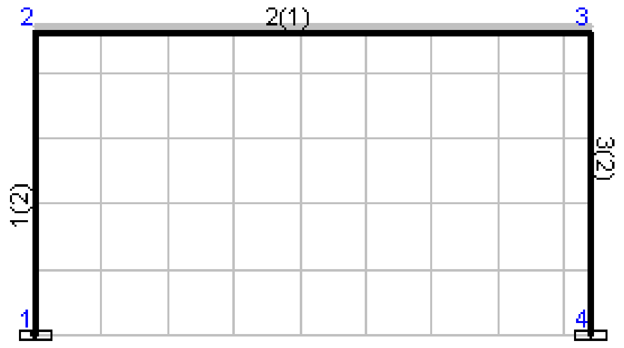
1-Finite element model (FEM)



Nodal points

Node	x [m]	y[m]
1	0.000	0.000
2	0.000	4.600
3	8.400	4.600
4	8.400	0.000

Supports

Node	kind	ux[mm]	uy[mm]	ur[rad]
1	fixed ux=uy=ur=0			
4	fixed ux=uy=ur=0			

Materials

Material : Steel, E= 210.000 [GPa]
Weight density : ρ= 78.500 [kN/m³]
The element self weight is included in loads and masses

Element cross sections

Cr.sec.	b[mm]	h[mm]	Ac[mm²]	Ic[mm4]
1			1.15500E+004	4.82000E+008
2			1.56000E+004	9.20800E+008

Elements

Element	node 1	node 2	material	length(m)	angle(°)
1	1	2	2	4.600	90.000
2	2	3	1	8.400	0.000
3	3	4	2	4.600	270.000

Nodal loads vertical forces Fy, (γg=1.35, γq=1.50)

Node	Fyg[kN]	Fyq[kN]	γgFyg+γqFyq[kN]
2	-95.000	-125.000	-315.750
3	-95.000	-125.000	-315.750

Element distributed loads (γg=1.35, γq=1.50)

element	G[kN/m]	Q[kN/m]	γgG+γqQ[kN/m]	load kind	load direction
2	8.600	12.400	30.210	uniform	perpendicular

Element distributed loads due to self weight (γg=1.35, γq=1.50)

element	G[kN/m]	Q[kN/m]	γgG+γqQ[kN/m]	load kind	load direction
1	1.225	0.000	1.654	uniform	vertical
2	0.907	0.000	1.224	uniform	vertical
3	1.225	0.000	1.654	uniform	vertical

2-Results of static-linear-elastic analysis

Diagrams of internal forces M, V, N, and displacements d, of element 1

n	x/l	x[m]	M[kNm]	V[kN]	N[kN]	dx[mm]	dy[mm]	d[mm]
0	0.000	0.00	78.05	51.98	-455.38	0.000	0.000	0.000
1	0.100	0.46	54.14	51.98	-454.62	-0.038	-0.063	0.074
2	0.200	0.92	30.23	51.98	-453.86	-0.136	-0.127	0.186
3	0.300	1.38	6.32	51.98	-453.10	-0.267	-0.190	0.327
4	0.400	1.84	-17.59	51.98	-452.34	-0.404	-0.254	0.477
5	0.500	2.30	-41.50	51.98	-451.58	-0.523	-0.317	0.611
6	0.600	2.76	-65.41	51.98	-450.82	-0.596	-0.380	0.707
7	0.700	3.22	-89.32	51.98	-450.06	-0.597	-0.444	0.744
8	0.800	3.68	-113.23	51.98	-449.30	-0.501	-0.507	0.713
9	0.900	4.14	-137.14	51.98	-448.54	-0.280	-0.571	0.636
10	1.000	4.60	-161.05	51.98	-447.77	0.090	-0.634	0.641

Maximum values for element 1

maxM= 78.05 kNm, minM= -161.05 kNm
maxV= 51.98 kN, minV= 51.98 kN
maxN= -447.77 kN, minN= -455.38 kN
maxd= 0.744 mm

Diagrams of internal forces M, V, N, and displacements d, of element 2

n	x/l	x[m]	M[kNm]	V[kN]	N[kN]	dx[mm]	dy[mm]	d[mm]
0	0.000	0.00	-161.05	-132.02	-51.98	0.090	-0.634	0.641
1	0.100	0.84	-61.23	-105.62	-51.98	0.072	-1.902	1.903
2	0.200	1.68	16.40	-79.21	-51.98	0.054	-3.610	3.611
3	0.300	2.52	71.85	-52.81	-51.98	0.036	-5.217	5.217
4	0.400	3.36	105.12	-26.40	-51.98	0.018	-6.335	6.335
5	0.500	4.20	116.21	0.00	-51.98	0.000	-6.733	6.733
6	0.600	5.04	105.12	26.40	-51.98	-0.018	-6.335	6.335
7	0.700	5.88	71.85	52.81	-51.98	-0.036	-5.217	5.217
8	0.800	6.72	16.40	79.21	-51.98	-0.054	-3.610	3.611
9	0.900	7.56	-61.23	105.62	-51.98	-0.072	-1.902	1.903
10	1.000	8.40	-161.04	132.02	-51.98	-0.090	-0.634	0.640

Maximum values for element 2

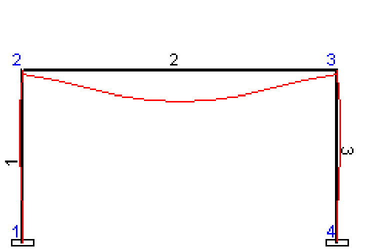
maxM= 116.21 kNm, minM= -161.05 kNm
maxV= 132.02 kN, minV= -132.02 kN
maxN= -51.98 kN, minN= -51.98 kN
maxd= 6.733 mm

Diagrams of internal forces M, V, N, and displacements d, of element 3

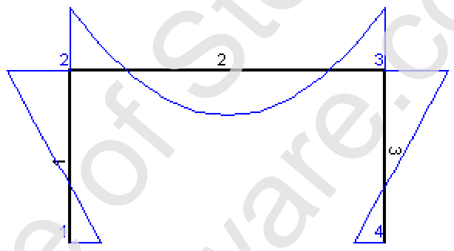
n	x/l	x[m]	M[kNm]	V[kN]	N[kN]	dx[mm]	dy[mm]	d[mm]
0	0.000	0.00	-161.05	-51.98	-447.77	-0.090	-0.634	0.640
1	0.100	0.46	-137.14	-51.98	-448.54	0.280	-0.571	0.636
2	0.200	0.92	-113.23	-51.98	-449.30	0.501	-0.507	0.713
3	0.300	1.38	-89.32	-51.98	-450.06	0.597	-0.444	0.744
4	0.400	1.84	-65.41	-51.98	-450.82	0.595	-0.380	0.706
5	0.500	2.30	-41.50	-51.98	-451.58	0.523	-0.317	0.611
6	0.600	2.76	-17.59	-51.98	-452.34	0.404	-0.254	0.477
7	0.700	3.22	6.32	-51.98	-453.10	0.267	-0.190	0.328
8	0.800	3.68	30.23	-51.98	-453.86	0.136	-0.127	0.186
9	0.900	4.14	54.14	-51.98	-454.62	0.038	-0.063	0.074
10	1.000	4.60	78.05	-51.98	-455.38	0.000	0.000	0.000

Maximum values for element 3

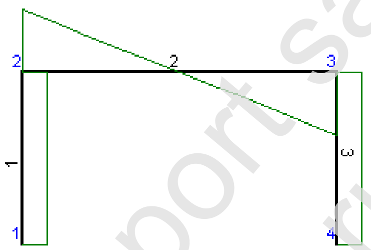
maxM= 78.05 kNm, minM= -161.05 kNm
maxV= -51.98 kN, minV= -51.98 kN
maxN= -447.77 kN, minN= -455.38 kN
maxd= 0.744 mm



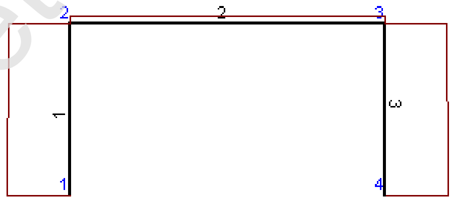
Displacement diagram
maxD=6.73 mm



Bending moment diagram
maxM=116.21 kNm, minM=-161.05 kNm



Shear force diagram
maxV=132.02 kN, minV=-132.02 kN



Axial force diagram
maxN=-51.98 kN, minN=-455.38 kN

3-Dimensioning of Steel (EC3 EN1993-1-1:2005)

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 Steel structures
EN1997-1-1:2004, Eurocode 7 Geotechnical design
EN1998-1-1:2004, Eurocode 8 Design in earthquake environment
NA - National Annex:

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)

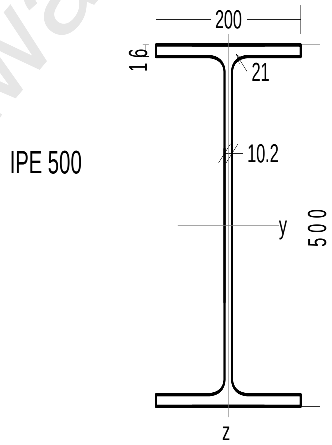
γG= 1.35, γQ= 1.50, ψ= 0.30

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

γM0= 1.00, γM1= 1.00, γM2= 1.25

Cross-section properties

Cross-section : 1, IPE 500-S 355

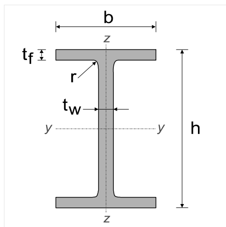


Dimensions of cross section

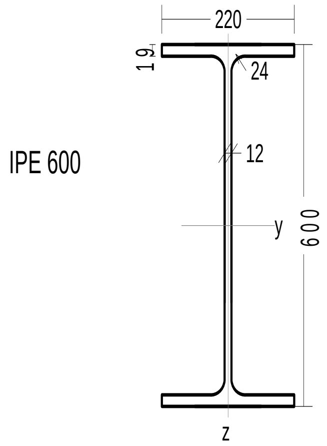
Depth of cross section	h=	500.00	mm
Width of cross section	b=	200.00	mm
Web depth	hw=	468.00	mm
Depth of straight portion of web	dw=	426.00	mm
Web thickness	tw=	10.20	mm
Flange thickness	tf=	16.00	mm
Radius of root fillet	r=	21.00	mm
Mass	=	90.70	Kg/m

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= 6718 mm^2	
Torsional constant	It= 0.893x10^6 mm^4		ip= 209 mm	
Warping constant	Iw=1249.4x10^9 mm^6			



Cross-section : 2, IPE 600-S 355

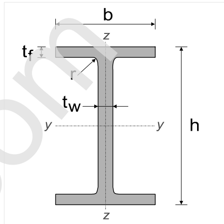


Dimensions of cross section

Depth of cross section	h=	600.00 mm
Width of cross section	b=	220.00 mm
Web depth	hw=	562.00 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= 8792 mm^2
Torsional constant	It= 1.654x10^6 mm^4	ip= 247 mm
Warping constant	Iw=2845.5x10^9 mm^6	



Steel design, element 1 , L= 4.600m, IPE 600

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
σ1=82 N/mm², σ2=-23 N/mm² (comression positive)

Web
c=600.0-2x19.0-2x24.0=514.0 mm, t=12.0 mm, c/t=514.0/12.0=42.83
S 355 , t=12.0<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
Position of neutral axis for combined Bending and compression
Ned/(2tw·fy/γM0)=455382/(2x12.0x355/1.00)=53.4 mm
α=(514.0/2+53.4)/514.0=0.604>0.5
c/t=42.83<=396x0.81/(13x0.604-1)=46.81
The web is class 1 (EN1993-1-1, Tab.5.2)

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

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

Resistance of cross-section (EN1993-1-1 §6.2.3, §6.2.4,§6.2.6,§6.2.5)

Tension Resistance	Nrdt,rd=A·fy/γM0=[10^{-3}]x15600x355/1.00=5538.00kN
Compression Resistance	Nrdc,rd=A·fy/γM0=[10^{-3}]x15600x355/1.00=5538.00kN
Shear resistance	Vrdz,rd=Av(fy/√3)/γM0= [10^{-3}]x8380x(355/1.73)/1.00= 1717.56kN
Bending Resistance	Mrdy,rd=Wely·fy/γM0=[10^{-6}]x3512.0x10^3x355/1.00= 1246.76kNm

Steel design, element 1, [Span], L= 4.600m, IPE 600

Med = -89.32 kNm, Ved = 51.98 kN, Ned = -450.06 kN

Ultimate Limit State , Verification for compression (EN1993-1-1, §6.2.4)
Nc.ed=450.06 kN
Compression Resistance Nplrd=A·fy/γM0=[10^{-3}]x15600x355/1.00=5538.00kN
Ned= 450.06 kN < 5538.00 kN =Nc,rd=Nplrd, Check is verified

Ultimate Limit State , Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed= 89.32 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x3512.0x10^{3}x355/1.00= 1246.76kNm
My,ed= 89.32 kNm < 1246.76 kNm =My,rd=Mply,rd, Check is verified

Ultimate Limit State , Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 450.06kN (Compression), Vz.ed= 51.98kNm, My.ed= 89.32kN
Nplrd=5538.00kN, Mpl,y,rd=1246.76kNm, Vpl,z,rd=0.00kN
Ned=450.06kN <= 0.25x5538.00=0.25xNplrd=1384.50kN
Ned=450.06kN <= [10^{-3}]x0.5x562.0x12.0x355/1.00=0.5hw·tw·fy/γM0=1197.06 kN
Effect of axial force is neglected (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)
My,ed= 89.32 kNm < 1246.76 kNm =Mply,rd, Check is verified

Steel design, element 1, [Left end], L= 4.600m, IPE 600

MedA= 78.05 kNm, VedA= 51.98 kN, NedA= -455.38 kN

Ultimate Limit State , Verification for compression (EN1993-1-1, §6.2.4)
Nc.ed=455.38 kN
Compression Resistance Nplrd=A·fy/γM0=[10^{-3}]x15600x355/1.00=5538.00kN
Ned= 455.38 kN < 5538.00 kN =Nc,rd=Nplrd, Check is verified

Ultimate Limit State , Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed= 78.05 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x3512.0x10^{3}x355/1.00= 1246.76kNm
My,ed= 78.05 kNm < 1246.76 kNm =My,rd=Mply,rd, Check is verified

Ultimate Limit State , Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 455.38kN (Compression), Vz.ed= 51.98kNm, My.ed= 78.05kN
Nplrd=5538.00kN, Mpl,y,rd=1246.76kNm, Vpl,z,rd=0.00kN
Ned=455.38kN <= 0.25x5538.00=0.25xNplrd=1384.50kN
Ned=455.38kN <= [10^{-3}]x0.5x562.0x12.0x355/1.00=0.5hw·tw·fy/γM0=1197.06 kN
Effect of axial force is neglected (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)
My,ed= 78.05 kNm < 1246.76 kNm =Mply,rd, Check is verified

Steel design, element 1, [Right end], L= 4.600m, IPE 600

MedB= -161.05 kNm, VedB= 51.98 kN, NedB= -447.77 kN

Ultimate Limit State , Verification for compression (EN1993-1-1, §6.2.4)
Nc.ed=447.77 kN
Compression Resistance Nplrd=A·fy/γM0=[10^{-3}]x15600x355/1.00=5538.00kN
Ned= 447.77 kN < 5538.00 kN =Nc,rd=Nplrd, Check is verified

Ultimate Limit State , Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed=161.05 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x3512.0x10^{3}x355/1.00= 1246.76kNm
My,ed= 161.05 kNm < 1246.76 kNm =My,rd=Mply,rd, Check is verified

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

N.ed= 447.77kN (Compression), Vz.ed= 51.98kNm, My.ed= 161.05kN

Nplrd=5538.00kN, Mpl,y,rd=1246.76kNm, Vpl,z,rd=0.00kN
Ned=447.77kN <= 0.25x5538.00=0.25xNplrd=1384.50kN
Ned=447.77kN <= $[10^{-3}]$x0.5x562.0x12.0x355/1.00=0.5hw·tw·fy/γM0=1197.06 kN
Effect of axial force is neglected (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)
My,ed= 161.05 kNm < 1246.76 kNm =Mply,rd, Check is verified

Steel design, element 1 , L= 4.600m, IPE 600, Buckling resistance

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

Buckling lengths: Lcr,y=1.870x4600=8600mm, Lcr,z=1.000x4600=4600mm
Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)
$\bar{\lambda}y=\sqrt{(A\cdot fy/Ncr,y)}=(Lcr,y/iy)\cdot(1/\lambda1)=(8600/243.0)x(1/76.06)=0.465$
$\bar{\lambda}z=\sqrt{(A\cdot fy/Ncr,z)}=(Lcr,z/iz)\cdot(1/\lambda1)=(4600/\ 46.6)x(1/76.06)=1.298$
$\lambda1=\pi\sqrt{(E/fy)}=93.9\varepsilon=76.06,\ \varepsilon=\sqrt{(235/fy)}=0.81$

h/b=600/220=2.73>=1.20, tf=19.0mm<=40 mm
y-y bucling curve:a, imperfection factor:αy=0.21, χy=0.935 (EC3 T.6.2,T.6.1,Fig.6.4)
$\Phi y=0.5[1+\alpha y(\bar{\lambda}y-0.2)+\bar{\lambda}y^2]=0.5x[1+0.21x(0.465-0.2)+0.465^2]=0.636$
$\chi y=1/[\Phi y+\sqrt{(\Phi y^2-\bar{\lambda}y^2)}]=1/[0.636+\sqrt{(0.636^2-0.465^2)}]=0.935 <=1\ \chi y=0.935$
z-z bucling curve:b, imperfection factor:αz=0.34, χz=0.428
$\Phi z=0.5[1+\alpha z(\bar{\lambda}z-0.2)+\bar{\lambda}z^2]=0.5x[1+0.34x(1.298-0.2)+1.298^2]=1.529$
$\chi z=1/[\Phi z+\sqrt{(\Phi z^2-\bar{\lambda}z^2)}]=1/[1.529+\sqrt{(1.529^2-1.298^2)}]=0.428 <=1\ \chi z=0.428$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}],\ \chi<=1.0,\ \Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2],\ \chi=0.428$ (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.428x$[10^{-3}]$x15600x355/1.00=2370.26kN (EC3 Eq.6.47)
Nc,ed= 455.38 kN < 2370.26 kN =Nb,rd, Check is verified

Lateral torsional buckling (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
$Mcr=C1\cdot[\pi^2EIz/(kL)^2]\{\sqrt{[(kz/kw)^2(Iw/Iz)+(kL)^2GIt/(\pi^2EIz)+(C2\cdot zg-C3\cdot zj)^2]}-(C2\cdot zg-C3\cdot zj)\}$
$G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4\ N/mm^2$
k·L=4600mm, zg=0mm, zj=0mm (EN1993:2002 T.C.1)
kz=1.0, kw=1.0, C1=2.567, C2=0.553, C3=0.000 (EN1993:2002 T.C.1)
$Mcr=[10^{-6}]2.567x[\pi^2x2.1x10^5x33.870x10^6/4600^2]$
$x\{\ [1.0x(2845.5x10^9/33.870x10^6)$
$+4600^2x8.1x10^4x1.654x10^6/(\pi^2x2.1x10^5x33.870x10^6)]^{0.5}\ \}=\ 3002.8\ kNm$

$\bar{\lambda},lt=\sqrt{(Wpl,y\cdot fy/Mcr)}=\sqrt{\{[10^{-6}]x3512.0x10^3x355/3002.8\}}=0.644$ (EC3 Eq.6.56)
h/b=600/220=2.73>2.00 bucling curve:b
imperfection factor:α,lt=0.34, χ,lt=0.814 (EC3 T.6.2,T.6.1,Fig.6.4)
$\Phi,lt=0.5[1+\alpha,lt(\bar{\lambda},lt-0.2)+\bar{\lambda},lt^2]=0.5x[1+0.34x(0.644-0.2)+0.644^2]=0.783$
$\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\bar{\lambda},lt^2)}]=1/[0.783+\sqrt{(0.783^2-0.644^2)}]=0.814 <=1\ \chi,lt=0.814$
Reduction factor $\chi,lt=1/[\Phi,lt+\sqrt{(\Phi,lt^2-\bar{\lambda},lt^2)}],\ \chi,lt<=1.0,\ \chi,lt=0.814$ (Eq.6.56)

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.814x$[10^{-6}]$x3512.0x10^3x355/1.00=1014.86kNm (EC3 Eq.6.55)
My,ed= 161.05 kNm < 1014.86 kNm =Mb,rd, Check is verified

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

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.61)
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.62)
Nrk=A·fy=$[10^{-3}]$x15600x355=5538.0 kN (Tab.6.7)
My,rk=Wpl,y·fy=$[10^{-6}]$x3512.0x10^3x355=1246.8 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.935x$[10^{-3}]$x15600x355/1.00=5178.0kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.428x$[10^{-3}]$x15600x355/1.00=2370.3kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.814x$[10^{-6}]$x3512.0x10^3x355/1.00=1014.9kNm

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

kyy=Cmy[1+($\bar{\lambda}$y-0.2)[Ned/(χy·Nrk/γM1)]], Kyy<=Cmy[1+0.8[Ned/(χy·Nrk/γM1)]] (EC3 Tab.B.1)
kzy=1-[0.10$\bar{\lambda}$z/(Cmlt-0.25)][Ned/(χz·Nrk/γM1)], Kyz>=1-[0.10/(Cmlt-0.25)][Ned/(χz·Nrk/γM1)]
ψ=-0.48, Cmy=0.410, Cmtl=0.41,

Kyy=0.41x[1+(0.465-0.2)x(455.4/5178.0)], Kyy<=0.41x[1+0.8x(455.4/5178.0)]
Kyy= 0.420, Kyy<=0.439, Kyy=0.420
Kzy=[1-[0.10x1.298/(0.41-0.25)]x(455.4/2370.3)], Kzy>=[1-[0.10/(0.41-0.25)]x(455.4/2370.3)]
Kzy=0.844, Kzy>=0.880, Kzy=0.880

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
455.4/(0.935x5538.0/1.00)+0.420x161.0/(0.814x1246.8/1.00)=0.155
0.155< 1.000, Check is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
455.4/(0.428x5538.0/1.00)+0.880x161.0/(0.814x1246.8/1.00)=0.332
0.332< 1.000, Check is verified

Steel design, element 2 , L= 8.400m, IPE 500

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
σ1=88 N/mm², σ2=-79 N/mm² (comression positive)

Web
c=500.0-2x16.0-2x21.0=426.0 mm, t=10.2 mm, c/t=426.0/10.2=41.76
S 355 , t=10.2<= 40 mm, fy=355 N/mm², ε=(235/355)$^{0.5}$=0.81
Position of neutral axis for combined Bending and compression
Ned/(2tw·fy/γM0)=51978/(2x10.2x355/1.00)=7.2 mm
α=(426.0/2+7.2)/426.0=0.517>0.5
c/t=41.76<=396x0.81/(13x0.517-1)=56.09
The web is class 1 (EN1993-1-1, Tab.5.2)

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

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

Resistance of cross-section (EN1993-1-1 §6.2.3, §6.2.4,§6.2.6,§6.2.5)

Tension Resistance Nrdt,rd=A·fy/γM0=[10^{-3}]x11550x355/1.00=4100.25kN
Compression Resistance Nrdc,rd=A·fy/γM0=[10^{-3}]x11550x355/1.00=4100.25kN
Shear resistance Vrdz,rd=Av(fy/√3)/γM0= [10^{-3}]x5985x(355/1.73)/1.00= 1226.72kN
Bending Resistance Mrdy,rd=Wely·fy/γM0=[10^{-6}]x2194.0x10^{3}x355/1.00= 778.87kNm

Steel design, element 2, [Span], L= 8.400m, IPE 500

Med = 116.21 kNm, Ved = 0.00 kN, Ned = -51.98 kN

Ultimate Limit State , Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed=116.21 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x2194.0x10^{3}x355/1.00= 778.87kNm
My,ed= 116.21 kNm < 778.87 kNm =My,rd=Mply,rd, Check is verified

Steel design, element 2, [Left end], L= 8.400m, IPE 500

MedA= -161.05 kNm, VedA= 132.02 kN, NedA= -51.98 kN

Ultimate Limit State , Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed=161.05 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x2194.0x10^{3}x355/1.00= 778.87kNm
My,ed= 161.05 kNm < 778.87 kNm =My,rd=Mply,rd, Check is verified

Ultimate Limit State , Verification for shear z (EN1993-1-1, §6.2.6)
Vz.ed=132.02 kNm
Av=A-2b·tf+(tw+2r)tf=11550-2x200.0x16.0+(10.2+2x21.0)x16.0=5985mm^2 (EC3 §6.2.6.3)
Av= 5985mm^2 > η·hw·tw= 1.00x(500.0-2x16.0)x10.2=1.00x468.0x10.2= 4774mm^2
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x5985x(355/1.73)/1.00= 1226.72kN
Vz,ed= 132.02 kN < 1226.72 kN =Vz,rd=Vpl,z,rd, Check is verified

hw/tw=(500.0-2x16.0)/10.2=468.0/10.2=45.88<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t=10.2<= 40 mm, fy=355 N/mm^2, ε=(235/355)$^{0.5}$=0.81
Shear buckling resistance is not necessary to be verified (EC3 §6.2.6.6)

Ultimate Limit State , Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 51.98kN (Compression), Vz.ed= 132.02kNm, My.ed= 161.05kN
Nplrd=0.00kNMpl,y,rd=778.87kNm, Vpl,z,rd=1226.72kN
Ned=0 kN, Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=132.02kN <= 0.50x1226.72=0.50xVpl,rd=613.36kN
Effect of shear force is neglected (EC3 §6.2.8.2)
My,ed= 161.05 kNm < 778.87 kNm =Mply,rd, Check is verified

Steel design, element 2, [Right end], L= 8.400m, IPE 500

MedB= -161.04 kNm, VedB= 132.02 kN, NedB= -51.98 kN

Ultimate Limit State , Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed=161.04 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10^{-6}]x2194.0x10^{3}x355/1.00= 778.87kNm
My,ed= 161.04 kNm < 778.87 kNm =My,rd=Mply,rd, Check is verified

Ultimate Limit State , Verification for shear z (EN1993-1-1, §6.2.6)
Vz.ed=132.02 kNm
Av=A-2b·tf+(tw+2r)tf=11550-2x200.0x16.0+(10.2+2x21.0)x16.0=5985mm^2 (EC3 §6.2.6.3)
Av= 5985mm^2 > η·hw·tw= 1.00x(500.0-2x16.0)x10.2=1.00x468.0x10.2= 4774mm^2
Plastic Shear Resistance Vpl,z,rd=Av(fy/√3)/γM0= [10^{-3}]x5985x(355/1.73)/1.00= 1226.72kN
Vz,ed= 132.02 kN < 1226.72 kN =Vz,rd=Vpl,z,rd, Check is verified

hw/tw=(500.0-2x16.0)/10.2=468.0/10.2=45.88<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t=10.2<= 40 mm, fy=355 N/mm^2, ε=(235/355)$^{0.5}$=0.81
Shear buckling resistance is not necessary to be verified (EC3 §6.2.6.6)

Ultimate Limit State , Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 51.98kN (Compression), Vz.ed= 132.02kNm, My.ed= 161.04kN
Nplrd=0.00kNMpl,y,rd=778.87kNm, Vpl,z,rd=1226.72kN
Ned=0 kN, Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=132.02kN <= 0.50x1226.72=0.50xVpl,rd=613.36kN
Effect of shear force is neglected (EC3 §6.2.8.2)
My,ed= 161.04 kNm < 778.87 kNm =Mply,rd, Check is verified

Steel design, element 2 , L= 8.400m, IPE 500, Buckling resistance

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

Buckling lengths: Lcr,y=1.000x8400=8400mm, Lcr,z=0.250x8400=2100mm
Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)
$\bar{\lambda}$y=√(A·fy/Ncr,y)=(Lcr,y/iy)·(1/λ1)=(8400/204.3)x(1/76.06)=0.541
$\bar{\lambda}$z=√(A·fy/Ncr,z)=(Lcr,z/iz)·(1/λ1)=(2100/ 43.1)x(1/76.06)=0.641
λ1=π√(E/fy)=93.9ε=76.06, ε=√(235/fy)=0.81

h/b=500/200=2.50>=1.20, tf=16.0mm<=40 mm
y-y bucling curve:a, imperfection factor:αy=0.21, χy=0.911 (EC3 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(0.541-0.2)+0.541^2]=0.682
χy=1/[Φy+√(Φy^2-$\bar{\lambda}$y^2)]=1/[0.682+√(0.682^2-0.541^2)]=0.911 <=1 χy=0.911
z-z bucling curve:b, imperfection factor:αz=0.34, χz=0.816
Φz=0.5[1+αz($\bar{\lambda}$z-0.2)+$\bar{\lambda}$z^2]=0.5x[1+0.34x(0.641-0.2)+0.641^2]=0.780
χz=1/[Φz+√(Φz^2-$\bar{\lambda}$z^2)]=1/[0.780+√(0.780^2-0.641^2)]=0.816 <=1 χz=0.816

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=0.816 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.816x[10^{-3}]x11550x355/1.00=3345.80kN (EC3 Eq.6.47)
Nc,ed= 51.98 kN < 3345.80 kN =Nb,rd, Check is verified

Lateral torsional buckling (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·[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)}
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10^4 N/mm^2
k·L=2100mm, zg=h/2=500/2=250mm, zj=0mm (EN1993:2002 T.C.1)
kz=1.0, kw=1.0, C1=1.623, C2=0.083, C3=-2.587 (EN1993:2002 T.C.1)
Mcr=[10^{-6}]1.623x[π^{2}x2.1x10^{5}x21.420x10^6/2100^2]
x{ [1.0x(1249.4x10^9/21.420x10^6)
+2100^{2}x8.1x10^{4}x0.893x10^6/(π^{2}x2.1x10^{5}x21.420x10^6)
+(0.083x250)^2]$^{0.5}$-(0.083x250) }= 3856.0 kNm

$\bar{\lambda}$,lt=√(Wpl,y·fy/Mcr)=√{[10^{-6}]x2194.0x10^{3}x355/3856.0}=0.449 (EC3 Eq.6.56)
h/b=500/200=2.50>2.00 bucling curve:b
imperfection factor:α,lt=0.34, χ,lt=0.906 (EC3 T.6.2,T.6.1,Fig.6.4)
Φ,lt=0.5[1+α,lt($\bar{\lambda}$,lt-0.2)+$\bar{\lambda}$,lt^2]=0.5x[1+0.34x(0.449-0.2)+0.449^2]=0.643
χ,lt=1/[Φ,lt+√(Φ,lt^2-$\bar{\lambda}$,lt^2)]=1/[0.643+√(0.643^2-0.449^2)]=0.906 <=1 χ,lt=0.906
Reduction factor χ,lt=1/[Φ,lt+√(Φ,lt^2-$\bar{\lambda}$,lt^2)], χ,lt<=1.0, χ,lt=0.906 (Eq.6.56)

Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.906x[10^{-6}]x2194.0x10^{3}x355/1.00=705.66kNm (EC3 Eq.6.55)
My,ed= 161.05 kNm < 705.66 kNm =Mb,rd, Check is verified

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

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.61)
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.62)
Nrk=A·fy=[10^{-3}]x11550x355=4100.2 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10^{-6}]x2194.0x10^{3}x355=778.9 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.911x[10^{-3}]x11550x355/1.00=3735.3kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.816x[10^{-3}]x11550x355/1.00=3345.8kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.906x[10^{-6}]x2194.0x10^{3}x355/1.00=705.7kNm

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

kyy=Cmy[1+($\bar{\lambda}$y-0.2)[Ned/(χy·Nrk/γM1)]], Kyy<=Cmy[1+0.8[Ned/(χy·Nrk/γM1)]] (EC3 Tab.B.1)
kzy=1-[0.10$\bar{\lambda}$z/(Cmlt-0.25)][Ned/(χz·Nrk/γM1)], Kyz>=1-[0.10/(Cmlt-0.25)][Ned/(χz·Nrk/γM1)]
ψ=1.00, as=0.00, Cmy=0.400, Cmtl=0.40,

```
Kyy=0.40x[1+(0.541-0.2)x(52.0/3735.3)], Kyy<=0.40x[1+0.8x(52.0/3735.3)]
Kyy= 0.402, Kyy<=0.404,  Kyy=0.402
Kzy=[1-[0.10x0.641/(0.40-0.25)]x(52.0/3345.8)], Kzy>=[1-[0.10/(0.40-0.25)]x(52.0/3345.8)]
Kzy=0.993, Kzy>=0.990, Kzy=0.993

Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)=                                        (EC3 Eq.6.61)
52.0/(0.911x4100.2/1.00)+0.402x161.0/(0.906x778.9/1.00)=0.106
               0.106< 1.000,  Check is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)=                                        (EC3 Eq.6.62)
52.0/(0.816x4100.2/1.00)+0.993x161.0/(0.906x778.9/1.00)=0.242
               0.242< 1.000,  Check is verified
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