Eurocode 3 (EC3)_Important Notes
Sachpazis Lecture Notes

Σημαντικά Σημεία από τον Ευρωκώδικα 3 (EC3)
(Από τις σημειώσεις του Κ. Σαχπάζη)

National Annex parameters EC3 EN1993-1-1:2005

Action coefficients | Load combination factors | Material factors EC3 §6.1 | EN1993-1-1 §6.3 | SLS EC3 §7.2

ULS (EQU) EC0 EN1990:2002 §6.4 Tab.A1.2A

Action coefficient for permanent loads, unfavorable	$\gamma_{G,sup}$= 1.10
Action coefficient for permanent loads, favorable	$\gamma_{G,inf}$= 0.90
Action coefficient for variable loads, unfavorable	γ_{Q}= 1.50

ULS (STR) EC0 EN1990:2002 §6.4 Tab.A1.2B

Action coefficient for permanent loads, unfavorable	$\gamma_{G,sup}$= 1.35
	ξ= 0.925
Action coefficient for permanent loads, favorable	$\gamma_{G,inf}$= 1.00
Action coefficient for variable loads, unfavorable	γ_{Q}= 1.50

National Annex parameters EC3 EN1993-1-1:2005

Action coefficients | Load combination factors | Material factors EC3 §6.1 | EN1993-1-1 §6.3 | SLS EC3 §7.2

Load combination factors EC0 EN1990:2002 Tab.A1.1

Action	ψ0	ψ1	ψ2
Category A (domestic and residential areas)	0.70	0.50	0.30
Category B (office areas)	0.70	0.50	0.30
Category C (common areas)	0.70	0.70	0.60
Category D (shopping areas)	0.70	0.70	0.60
Category E (storage areas)	1.00	0.90	0.80
Category F (parking areas light <=35kN)	0.70	0.70	0.60
Category G (parking areas medium 35kN< ~ <=160kN)	0.70	0.50	0.30
Category H (roofs)	0.00	0.00	0.00
Snow loads on buildings (H>100 m.a.s.l.)	0.70	0.50	0.20
Snow loads on buildings (H<=100 m.a.s.l.)	0.50	0.20	0.00
Wind loads on buildings	0.60	0.20	0.00

National Annex parameters EC3 EN1993-1-1:2005

Action coefficients | Load combination factors | Material factors EC3 §6.1 | EN1993-1-1 §6.3 | SLS EC3 §7.2

Eurocode 3 (Steel structures) EC3 EN1993-1-1:2005 §6.1

γ_{M0}= 1.00

γ_{M1}= 1.00

γ_{M2}= 1.25

National Annex parameters EC3 EN1993-1-1:2005

Action coefficients | Load combination factors | Material factors EC3 §6.1 | EN1993-1-1 §6.3 | SLS EC3 §7.2

Lateral torsional buckling (EN1993-1-1 §6.3.2.3)

Method of computation

○ EC3-Eq.6.56 ◉ EC3-Eq.6.57

$\overline{\lambda}_{LT,0}$= 0.40 β = 0.75

Bending and compression (EN1993-1-1 §6.3.3)

Method of computation

◉ Method 1 ○ Method 2 ○ Method 1 and 2

Elastic critical moment for lateral-torsional buckling

ECCS 119/Galea SN030a-EN-EU Access Steel 2006

National Annex parameters EC3 EN1993-1-1:2005

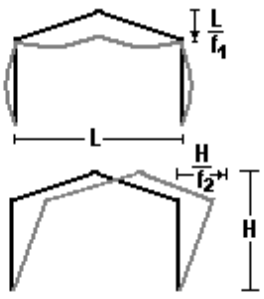
Action coefficients | Load combination factors | Material factors EC3 §6.1 | EN1993-1-1 §6.3 | SLS EC3 §7.2

Limit for vertical deflection w<=L/f1 f1= 200.0

Limit for horizontal deflection u<=H/f2 f2= 150.0

Maximum roof deflection in general Wmax<=L/f3 f3= 200.0

Roof deflection due to variable action w3<=L/f4 f4= 250.0



Limit for vertical deflection, Floors G+Q fo= 200.0

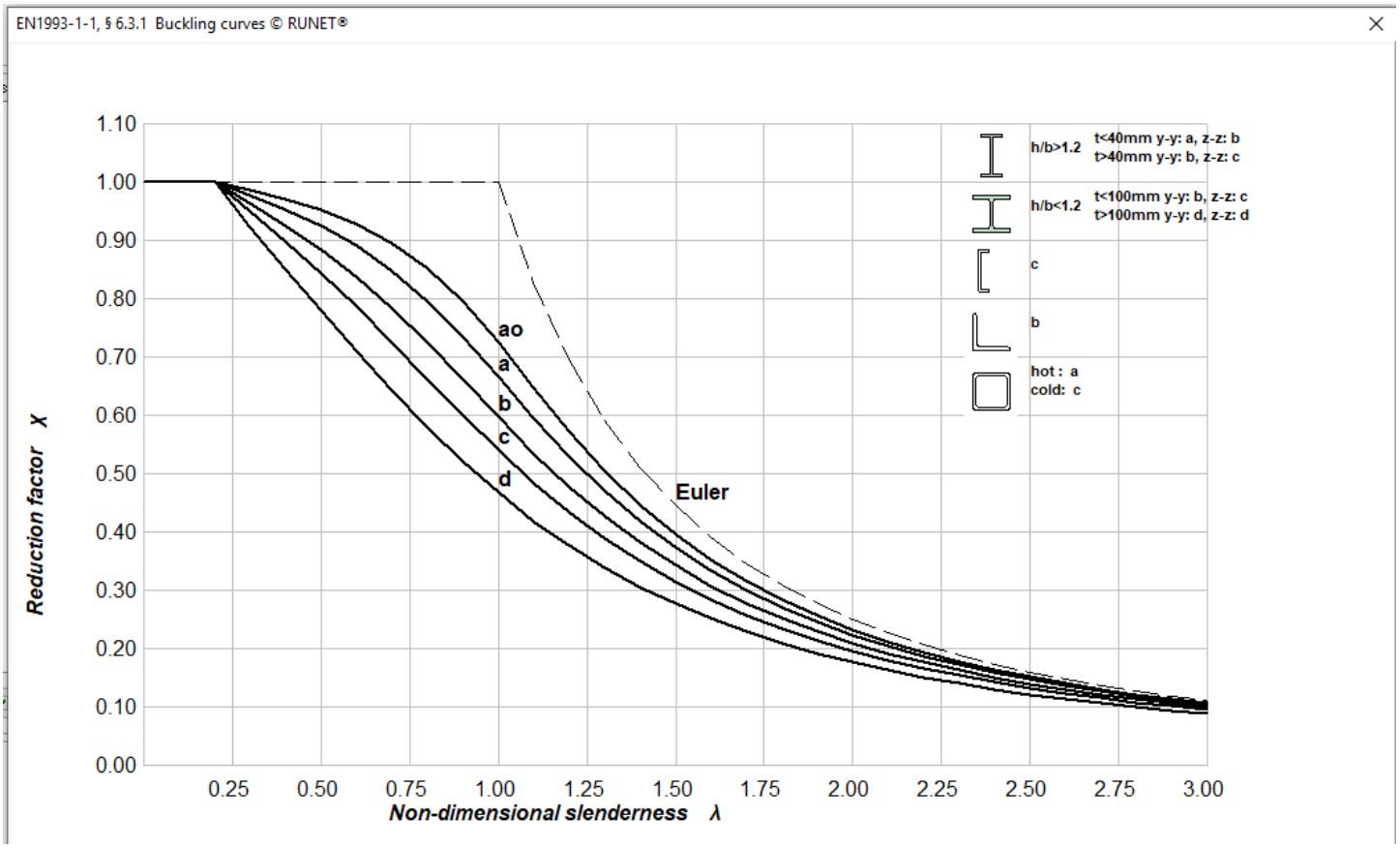
Limit for vertical deflection, Floors Q fo= 360.0

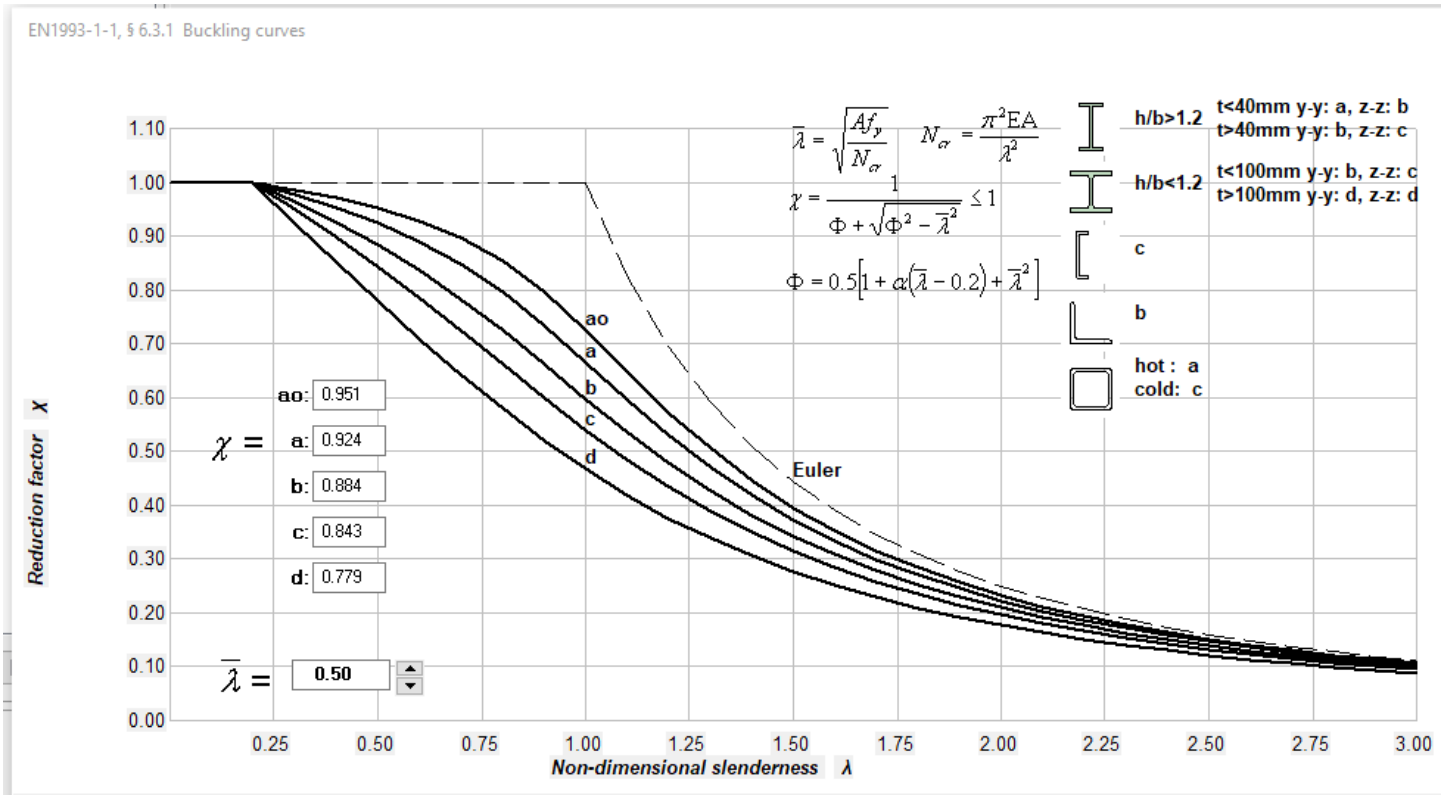
Properties of structural steel

Steel	Grade	fy (MPa) t<=40mm	fu (MPa) t<=40mm	fy (MPa) 40<t<=100mm	fu (MPa) 40<t<=100mm
S 235	EN 10025-2	235	360	215	360
S 275	EN 10025-2	275	430	255	410
S 355	EN 10025-2	355	510	335	470
S 450	EN 10025-2	440	550	410	550
S 275 N/NL	EN 10025-3	275	390	255	370
S 355 N/NL	EN 10025-3	355	490	335	470
S 420 N/NL	EN 10025-3	420	520	390	520
S 460 N/NL	EN 10025-3	460	540	430	540
S 275 M/ML	EN 10025-4	275	370	255	360
S 355 M/ML	EN 10025-4	355	470	335	450
S 420 M/ML	EN 10025-4	420	520	390	500
S 460 M/ML	EN 10025-4	460	540	430	530
S 235 W	EN 10025-5	235	360	215	340
S 355 W	EN 10025-5	355	510	335	490
S460Q/QL	EN 10025-6	460	570	440	550
S 235 H	EN 10210-1	235	360	215	340

Properties of structural steel

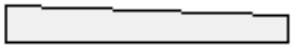
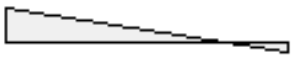
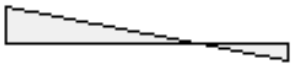
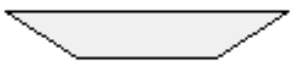
Steel	Grade	fy (MPa) t<=40mm	fu (MPa) t<=40mm	fy (MPa) 40<t<=100mm	fu (MPa) 40<t<=100mm
S 275 H	EN 10210-1	275	430	255	410
S 355 H	EN 10210-1	355	510	335	490
S275NH/NLH	EN 10210-1	275	390	255	370
S355NH/NLH	EN 10210-1	355	490	335	470
S420NH/NLH	EN 10210-1	420	540	390	520
S460NH/NLH	EN 10210-1	460	560	430	550
S460NH/NLH	EN 10210-1	460	560	430	550
S460NH/NLH	EN 10210-1	460	560	430	550
S460NH/NLH	EN 10210-1	460	560	430	550
S220GD+Z	EN 10147	220	300	220	300
S250GD+Z	EN 10147	250	330	250	330
S280GD+Z	EN 10147	280	360	280	360
S320GD+Z	EN 10147	320	390	320	390
S350GD+Z	EN 10147	350	420	350	420
H240LA	EN 10268	240	340	240	340
H280LA	EN 10268	280	370	280	370
H320LA	EN 10268	320	400	320	400
H360LA	EN 10268	360	430	360	430
H400LA	EN 10268	400	460	400	460
H260LAD	EN 10292	240	340	240	340
H300LAD	EN 10292	280	370	280	370
H340LAD	EN 10292	320	400	320	400
H380LAD	EN 10292	360	430	360	430
H420LAD	EN 10292	400	460	400	460
S220GD+ZA	EN 10214	220	300	220	300
S250GD+ZA	EN 10214	250	330	250	330
S280GD+ZA	EN 10214	280	360	280	360
S320GD+ZA	EN 10214	320	390	320	390
S350GD+ZA	EN 10214	350	420	350	420


EN1993-1-1, § 6.3.1 Buckling curves © RUNET®
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
Reduction factor χ
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
Non-dimensional slenderness λ
ao
a
b
c
d
Euler
h/b>1.2
t<40mm y-y: a, z-z: b
t>40mm y-y: b, z-z: c
h/b<1.2
t<100mm y-y: b, z-z: c
t>100mm y-y: d, z-z: d
c
b
hot : a
cold: c


EN1993-1-1, § 6.3.1 Buckling curves
$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$ $N_{cr} = \frac{\pi^2 EA}{\lambda^2}$
$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1$
$\Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$
χ =
ao: 0.951
a: 0.924
b: 0.884
c: 0.843
d: 0.779
$\bar{\lambda}$ = 0.50
h/b>1.2
t<40mm y-y: a, z-z: b
t>40mm y-y: b, z-z: c
h/b<1.2
t<100mm y-y: b, z-z: c
t>100mm y-y: d, z-z: d
c
b
hot : a
cold: c
Euler
Reduction factor χ
Non-dimensional slenderness λ

Elastic critical moment for lateral-torsional bucklingprEN 1993-1-1:2002 Annex C

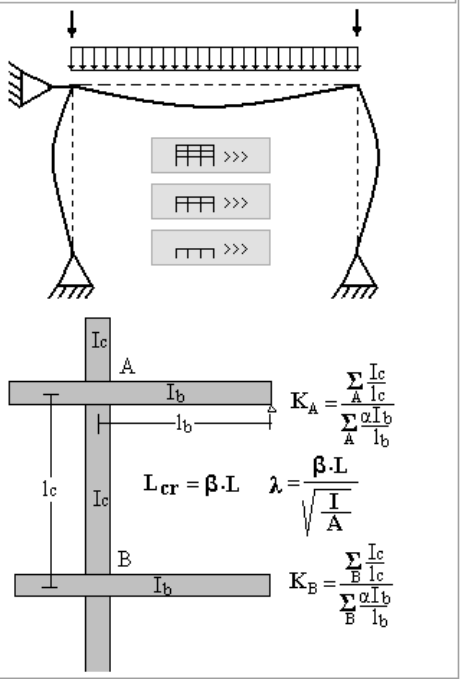
$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 Z_g - C_3 Z_j)^2} - (C_2 Z_g - C_3 Z_j) \right]$$

Bending moment diagram	k	C1	C2	C3
 ψ=+1	1.00 0.50	1.000 1.127		1.000 1.019
 ψ=+3/4	1.00 0.50	1.141 1.285		1.000 1.017
 ψ=+1/2	1.00 0.50	1.320 1.482		1.000 1.000
 ψ=+1/4	1.00 0.50	1.551 1.730		1.000 1.000
 ψ=0	1.00 0.50	1.847 2.027		1.000 1.000
 ψ=-1/4	1.00 0.50	2.207 2.341		1.000 1.000
 ψ=-1/2	1.00 0.50	2.591 2.579		1.000 0.950
 ψ=-3/4	1.00 0.50	2.852 2.606		1.000 0.850
 ψ=-1	1.00 0.50	2.733 2.390		 0.825
	1.00 0.50	1.132 0.997	0.459 0.407	0.525 0.478
	1.00 0.50	1.363 1.087	0.553 0.449	0.411 0.338
	1.00 0.50	1.040 0.960	0.431 0.404	0.562 0.539
	1.00 0.50	2.608 1.515	1.562 0.900	-0.859 -0.516
	1.00 0.50	1.726 0.955	1.388 0.763	-0.716 -0.406

EN1992-1-1, § 5.8.3.2 Effective length © RUNET®

Braced members

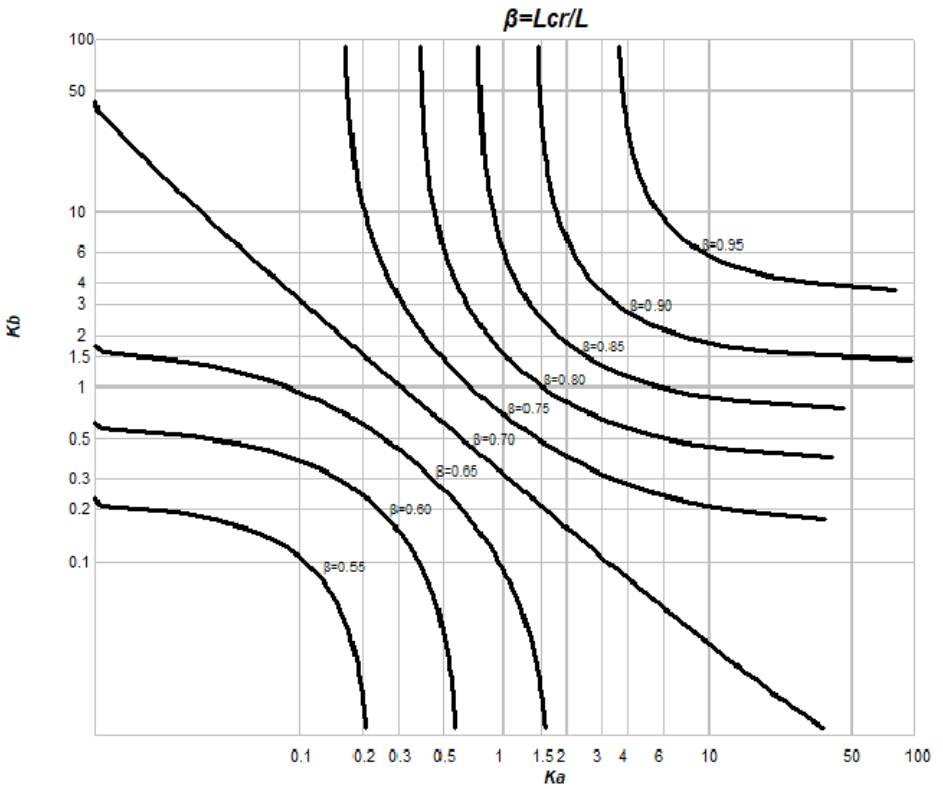
Unbraced members



$$K_A = \frac{\sum_A \frac{Ic}{lc}}{\sum_A \frac{\alpha Ib}{lb}}$$

$$L_{cr} = \beta \cdot L \qquad \lambda = \frac{\beta \cdot L}{\sqrt{\frac{I}{A}}}$$

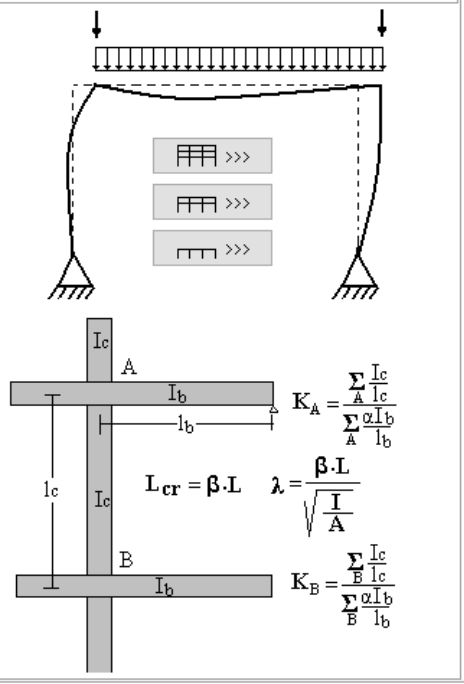
$$K_B = \frac{\sum_B \frac{Ic}{lc}}{\sum_B \frac{\alpha Ib}{lb}}$$



EN1992-1-1, § 5.8.3.2 Effective length © RUNET®

Braced members

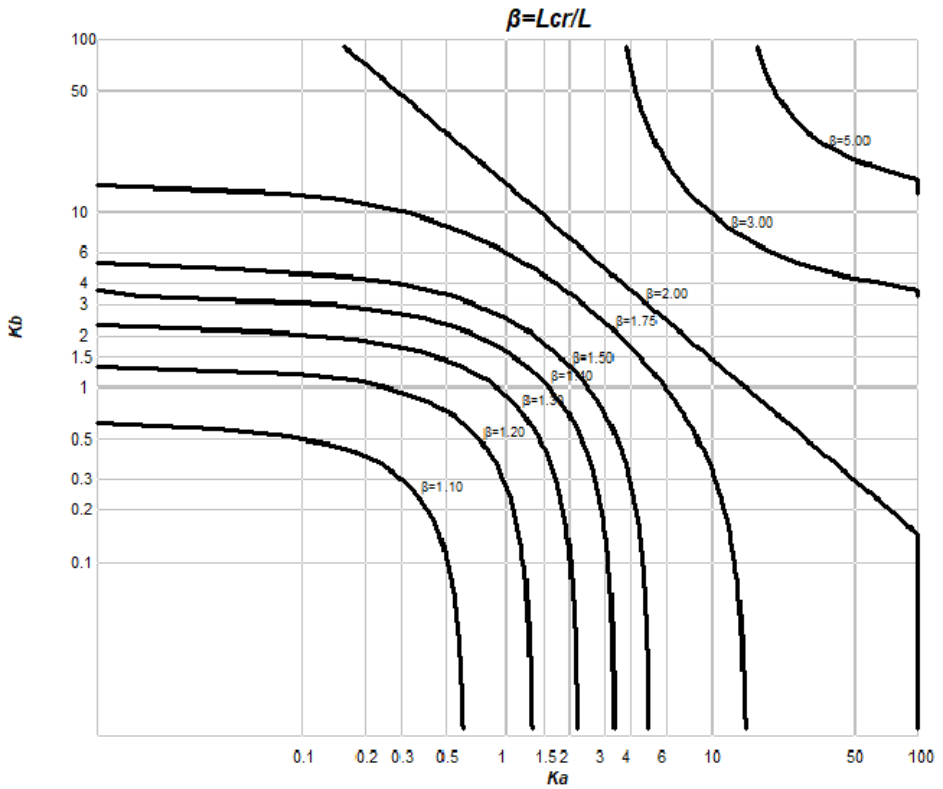
Unbraced members



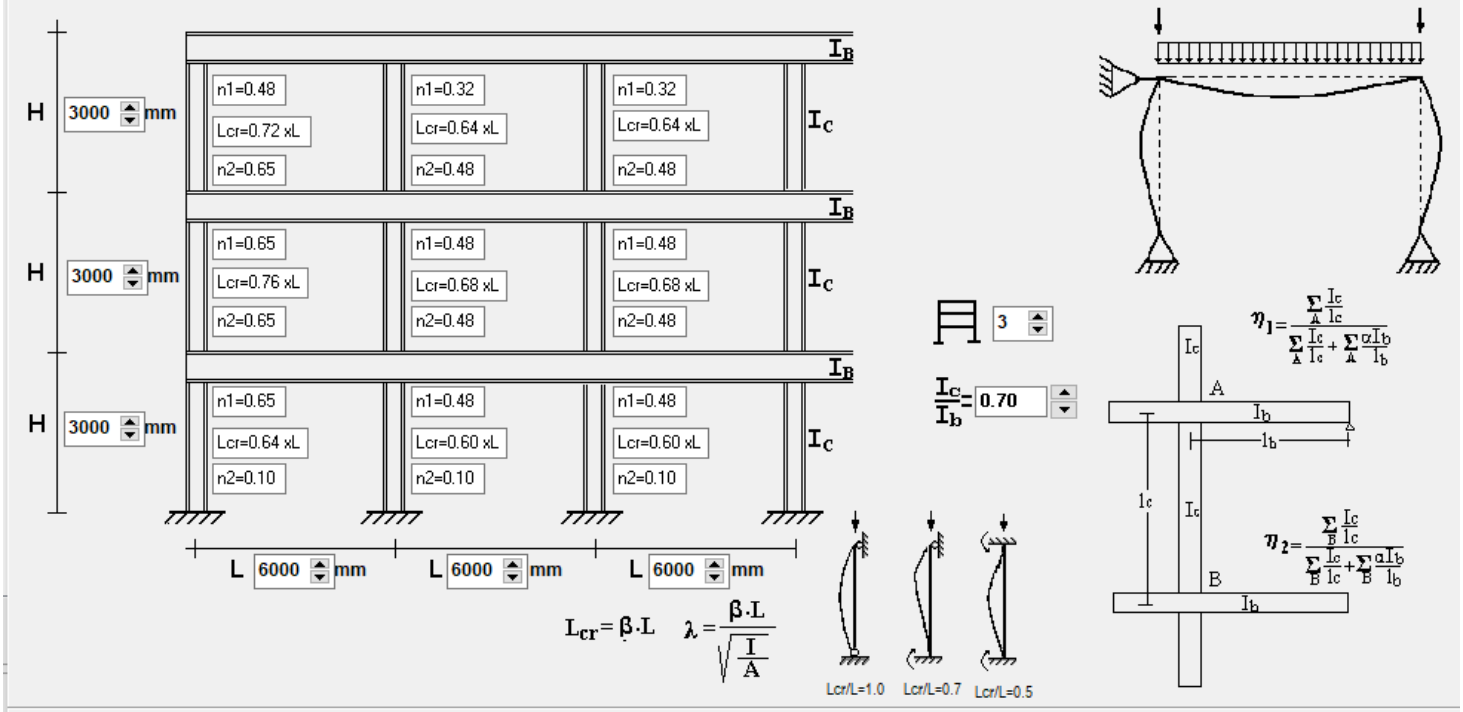
$$K_A = \frac{\sum_A \frac{Ic}{lc}}{\sum_A \frac{\alpha Ib}{lb}}$$

$$L_{cr} = \beta \cdot L \qquad \lambda = \frac{\beta \cdot L}{\sqrt{\frac{I}{A}}}$$

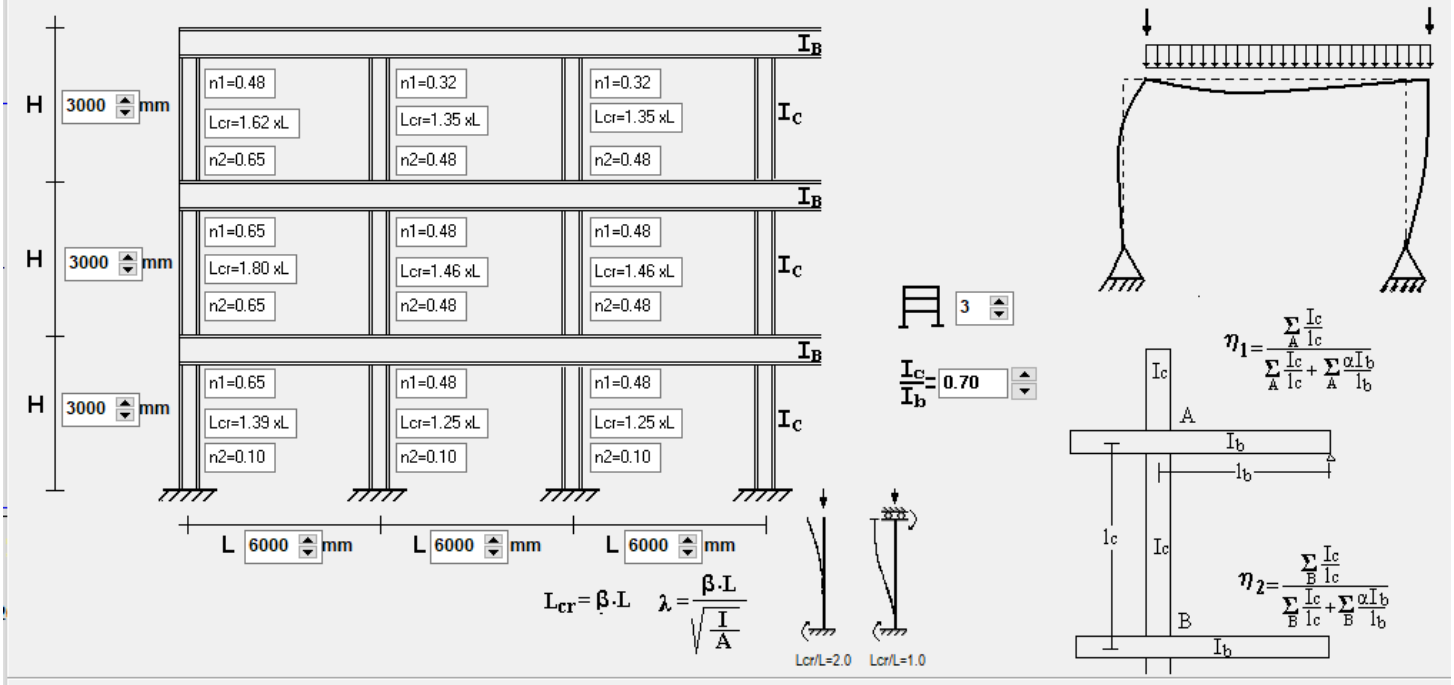
$$K_B = \frac{\sum_B \frac{Ic}{lc}}{\sum_B \frac{\alpha Ib}{lb}}$$



Effective length of columns-Braced members ENV 1993-1-1:1992 Annex E



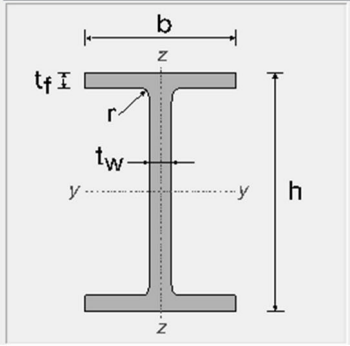
Effective length of columns-Unbraced members ENV 1993-1-1:1992 Annex E



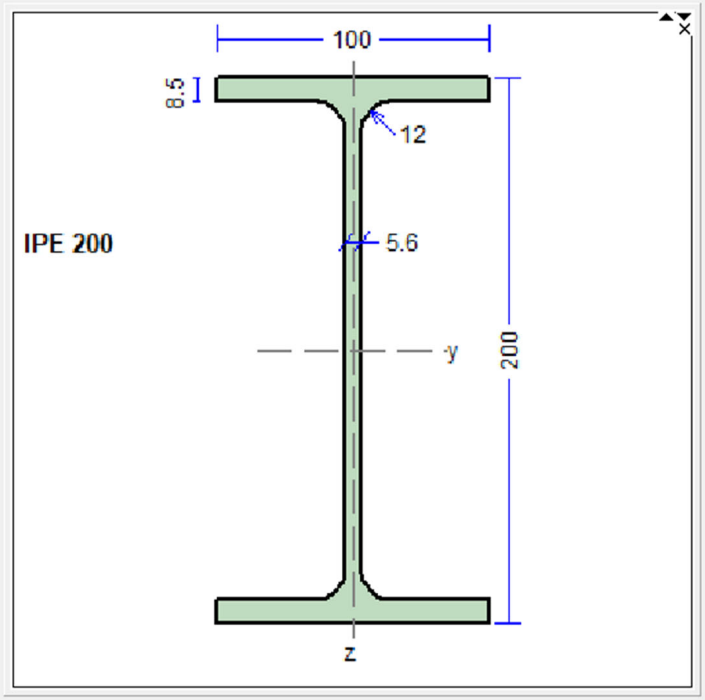
Steel Section Properties

Steel sections

- I
- IPE
  - IPE Euronorm 19-57
  - IPE A
  - IPE O
  - IPE V
  - IPE 750
- HE
- HD
- HP
- IPN
- W (ASTM)
- UB, UC (BS)
- ISMB, SC, HB
- I GOST
- U
- L
- La
- hot rolled
- hot rolled
- cold formed
- cold formed
- D= 10.2- 33.7 mm
- D= 38.0- 63.5 mm
- D= 70.0- 88.9 mm
- D=101.6-159.0 mm
- D=168.3-193.7 mm
- D=219.1-355.6 mm
- D=406.4-508.0 mm
- D=559.0-660.0 mm
- D=771.0-1016 mm
- C
- Z



IPE	h	b	tw	tf	r	A	G	Iy	Wy	Wply	iy	Av,z	Iz	Wz	Wplz	iz	Av,y	It	Iw
Euronorm 19-57	mm	mm	mm	mm	mm	cm²	Kg/m	cm⁴	cm³	cm³	cm	cm²	cm⁴	cm³	cm³	cm	cm²	cm⁴	cm⁶
IPE 80	80	46	3.8	5.2	5.0	7.64	6.00	80.14	20.03	23.22	3.24	3.57	8.49	3.69	5.82	1.05	4.78	0.698	118.0
IPE 100	100	55	4.1	5.7	7.0	10.32	8.10	171.0	34.20	39.41	4.07	5.08	15.92	5.79	9.15	1.24	6.27	1.20	351.4
IPE 120	120	64	4.4	6.3	7.0	13.21	10.4	317.8	52.96	60.73	4.90	6.31	27.67	8.65	13.58	1.45	8.06	1.74	889.6
IPE 140	140	73	4.7	6.9	7.0	16.43	12.9	541.2	77.32	88.34	5.74	7.65	44.92	12.31	19.25	1.65	10.07	2.45	1 981
IPE 160	160	82	5.0	7.4	9.0	20.09	15.8	869.3	108.7	123.9	6.58	9.66	68.31	16.66	26.10	1.84	12.14	3.60	3 959
IPE 180	180	91	5.3	8.0	9.0	23.95	18.8	1 317	146.3	166.4	7.42	11.25	100.9	22.16	34.60	2.05	14.56	4.79	7 431
IPE 200	200	100	5.6	8.5	12.0	28.48	22.4	1 943	194.3	220.6	8.26	14.00	142.4	28.47	44.61	2.24	17.00	6.98	12 988
IPE 220	220	110	5.9	9.2	12.0	33.37	26.2	2 772	252.0	285.4	9.11	15.88	204.9	37.25	58.11	2.48	20.24	9.07	22 672
IPE 240	240	120	6.2	9.8	15.0	39.12	30.7	3 892	324.3	366.6	9.97	19.15	283.6	47.27	73.92	2.69	23.52	12.88	37 391
IPE 270	270	135	6.6	10.2	15.0	45.94	36.1	5 790	428.9	484.0	11.23	22.13	419.9	62.20	96.95	3.02	27.54	15.94	70 578
IPE 300	300	150	7.1	10.7	15.0	53.81	42.2	8 356	557.1	628.4	12.46	25.68	603.8	80.50	125.2	3.35	32.10	20.12	125 934
IPE 330	330	160	7.5	11.5	18.0	62.61	49.1	11 770	713.1	804.3	13.71	30.81	788.1	98.52	153.7	3.55	36.80	28.15	199 097
IPE 360	360	170	8.0	12.7	18.0	72.73	57.1	16 270	903.6	1 019	14.96	35.14	1 043	122.8	191.1	3.79	43.18	37.32	313 580
IPE 400	400	180	8.6	13.5	21.0	84.46	66.3	23 130	1 156	1 307	16.55	42.69	1 318	146.4	229.0	3.95	48.60	51.08	490 048
IPE 450	450	190	9.4	14.6	21.0	98.82	77.6	33 740	1 500	1 702	18.48	50.84	1 676	176.4	276.4	4.12	55.48	66.87	791 005
IPE 500	500	200	10.2	16.0	21.0	115.5	90.7	48 200	1 928	2 194	20.43	59.85	2 142	214.2	335.9	4.31	64.00	89.29	1 249 365
IPE 550	550	210	11.1	17.2	24.0	134.4	106.0	67 120	2 441	2 787	22.35	72.33	2 668	254.1	400.5	4.46	72.24	123.2	1 884 098
IPE 600	600	220	12.0	19.0	24.0	156.0	122.0	92 080	3 069	3 512	24.30	83.80	3 387	307.9	485.6	4.66	83.60	165.4	2 845 527

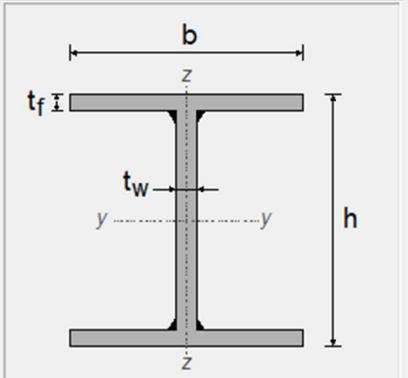


Welded (fabricated) sections

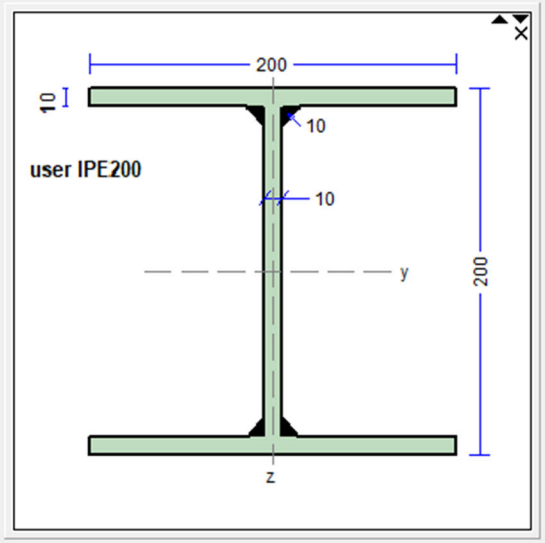
File

Steel sections

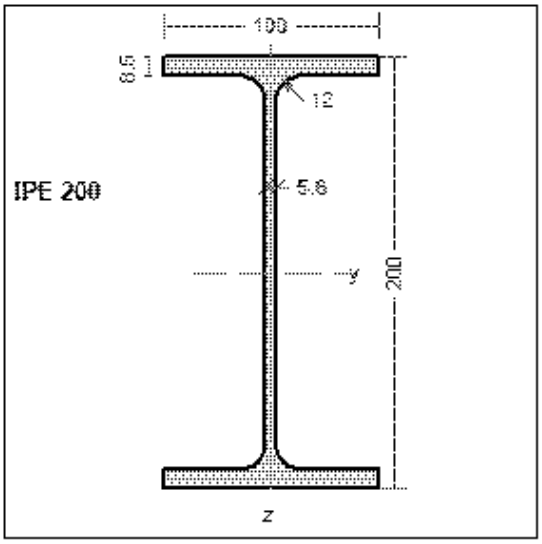
- I
- IPE
- BOX



	h	b	tw	tf	a	A	G	Iy	Wy	Wply	iy	Av,z	Iz	Wz	Wplz	iz	Av,y	It	Iw
	mm	mm	mm	mm	mm	cm²	Kg/m	cm⁴	cm³	cm³	cm	cm²	cm⁴	cm³	cm³	cm	cm²	cm⁴	cm⁶
user IPE200	200	200	10.0	10.0	10.0	58.00	45.6	4 099	409.9	461.0	8.41	19.00	1 335	133.5	204.5	4.80	40.00	19.62	120 333
user IPE250	250	250	15.0	15.0	10.0	108.0	84.8	11 700	936.0	1 063	10.41	35.25	3 912	313.0	481.1	6.02	75.00	82.46	539 307
user IPE300	300	300	20.0	20.0	10.0	172.0	135.1	26 489	1 766	2 018	12.41	56.00	9 017	601.2	926.0	7.24	120.0	233.9	1 764 000
user IPE400	400	400	24.0	24.0	10.0	276.5	217.2	76 675	3 834	4 353	16.65	90.24	25 641	1 282	1 971	9.63	192.0	540.4	9 048 064
user IPE500	500	500	28.0	28.0	10.0	404.3	317.6	176 555	7 062	7 988	20.90	132.2	58 415	2 337	3 587	12.02	280.0	1 074	32 489 334
user IPE600	600	600	30.0	30.0	10.0	522.0	410.0	332 046	11 068	12 447	25.22	171.0	108 122	3 604	5 522	14.39	360.0	1 589	87 723 000
user IPE700	700	700	32.0	32.0	10.0	651.5	511.8	568 756	16 250	18 199	29.55	213.8	183 107	5 232	8 003	16.76	448.0	2 254	204 073 104
user IPE800	800	800	35.0	35.0	15.0	815.5	640.6	933 350	23 334	26 083	33.83	267.8	298 928	7 473	11 424	19.15	560.0	3 373	436 968 000
user IPE900	900	900	50.0	50.0	15.0	1300.0	1021.2	1 840 833	40 907	46 250	37.63	425.0	608 333	13 519	20 750	21.63	900.0	11 013	1 097 296 896
user IPE1000	1000	1000	60.0	60.0	15.0	1728.0	1357.3	2 995 136	59 903	68 016	41.63	564.0	1 001 584	20 032	30 792	24.08	1 200	21 109	2 208 999 936
user IPE1200	1200	1200	60.0	60.0	15.0	2088.0	1640.1	5 312 736	88 546	99 576	50.44	684.0	1 729 944	28 832	44 172	28.78	1 440	25 429	5 614 272 000



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=	19.430×10^6	mm^4
	Iz=	1.424×10^6	mm^4
Section modulus	Wy=	194.30×10^3	mm^3
	Wz=	28.470×10^3	mm^3
Plastic section modulus	Wpy=	220.60×10^3	mm^3
	Wpz=	44.610×10^3	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=	0.070×10^6	mm^4
	ip=	86	mm
Torsional modulus	Wt=	8.212×10^3	mm^3
Warping constant	Iw=	12.988×10^9	mm^6

Cross-section IPE 200-S 450

Classification and Resistance of cross section (EN1993-1-1:2005 §5.6, §6.2)

Steel	Class			Tension	Compression	Moment resistance		Shear resistance	
	Nx	Myy	Mzz	NtRd[kN]	NcRd[kN]	McRdy[kNm]	McRdz[kNm]	VcRdz [kN]	VcRdy [kN]
S 235	1	1	1	669	669	51.8	10.5	190	231
S 275	1	1	1	783	783	60.7	12.3	222	270
S 355	2	1	2	1011	1011	78.3	15.8	287	348
S 450	3	1	3	1253	1253	97.1	12.5	356	432

(γM0= 1.00, γM1= 1.00, γM2= 1.00)

Buckling resistance, Buckling length Lc[m], (EC3 §6.2)

	Lc[m]	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0	12.0
S 235	Nbrdy	669	669	661	651	640	628	615	584	542	490	431	372	319	236
	Nbrdz	599	519	420	323	248	192	153	102	73	54	42	33	27	19
S 275	Nbrdy	783	782	769	757	742	726	708	663	604	532	456	386	327	239
	Nbrdz	686	579	450	337	253	195	154	103	73	55	42	34	27	20
S 355	Nbrdy	1011	1002	984	964	941	916	885	807	705	592	491	406	340	246
	Nbrdz	852	680	496	358	264	201	158	104	74	55	42	34	27	19
S 450	Nbrdy	1253	1236	1209	1179	1147	1107	1059	934	782	637	516	422	351	251
	Nbrdz	1015	764	531	373	273	207	162	107	75	56	43	34	28	20

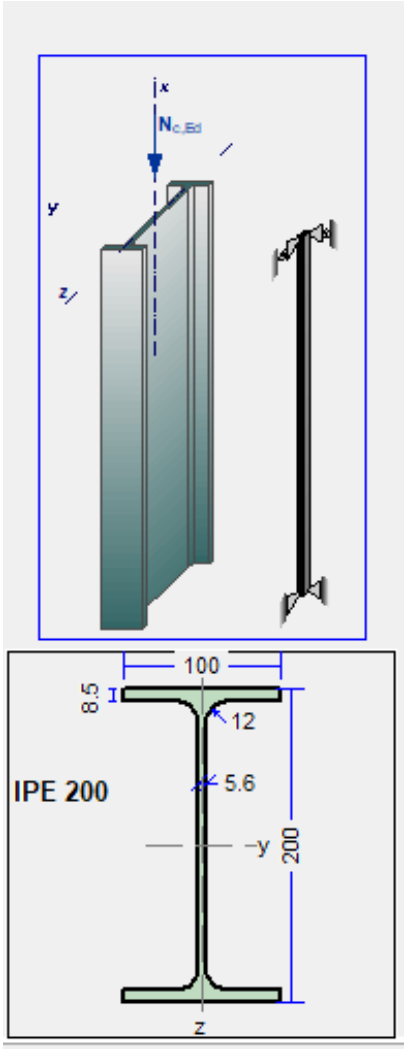
Nbyrd: buckling resistance about y-y axis, Nbzrd: buckling resistance about z-z axis

Lateral torsional buckling resistance, Buckling length Llt[m], (EC3 §6.3)

	Llt[m]	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0	12.0
S 235	Mbrd1	49.2	46.4	43.0	39.0	35.0	31.2	27.8	22.8	19.2	16.5	14.6	13.0	11.8	9.85
	Mbrd2	48.3	44.7	40.5	36.1	32.0	28.7	25.8	21.6	18.6	16.3	14.6	13.1	12.0	10.2
S 275	Mbrd1	57.1	53.3	48.4	42.8	37.5	32.8	29.0	23.5	19.6	16.9	14.8	13.2	11.9	9.95
	Mbrd2	55.8	50.8	44.9	38.9	33.9	29.9	26.8	22.1	18.9	16.6	14.8	13.3	12.1	10.3
S 355	Mbrd1	72.3	65.7	57.2	48.3	40.9	35.0	30.5	24.3	20.1	17.2	15.0	13.4	12.1	10.0
	Mbrd2	70.2	61.5	51.5	42.8	36.3	31.6	28.0	22.9	19.4	16.9	15.0	13.5	12.3	10.4
S 450	Mbrd1	87.8	77.3	64.1	52.0	43.0	36.3	31.4	24.8	20.5	17.5	15.2	13.5	12.2	10.2
	Mbrd2	84.4	70.6	56.1	45.2	37.9	32.5	28.7	23.3	19.8	17.2	15.2	13.7	12.4	10.5

Mb1rd: uniform shape moment diagram, Mb2rd: parabolic shape moment diagram

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



Name of design object		EC3-COLUMN-001					
Structural steel grade (EN1993-1-1 §3.2)		S 355	fy=355N/mm² fu=510N/mm²				
Partial safety factors for actions		γG=	1.35		γQ=	1.50	
Partial factors for materials	EC3	γM0=	1.00	γM1=	1.00	γM2=	1.25
Compression		Nc,ed=	100.00	kN			
Column height		L=	5.200	m			
Buckling length y-y		Lcr,y=	1.00	X	5.20	5.20	m
Buckling length z-z		Lcr,z=	1.00	X	5.20	5.20	m
Selected section	IPE 200						
		Standard sections					

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	mm²
Second moment of area	Iy=	1.943E007	mm^4
	Iz=	1.424E006	mm^4
Section modulus	Wy=	194300	mm³
	Wz=	28470	mm³
Plastic section modulus	Wpy=	220600	mm³
	Wpz=	44610	mm³
Radius of gyration	iy=	83	mm
	iz=	22	mm
Shear area	Avz=	1400	mm²
	Avy=	1700	mm²
Torsional constant	It=	69801	mm^4
	ip=	86	mm
Torsional modulus	Wt=	8212	mm³
Warping constant	Iw=	1.2988E010	mm^6

EC3-COLUMN-001

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

Profile : IPE 200
Actions : Compression Nc,ed
Nc,ed= 100.00kN
Steel Class : S 355

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

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
Compression Nc,ed
Nc,ed=100.00 kN(Compression)

Steel cross-section properties

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
Torsional modulus	Wt= 8.212x10 ^3 mm ^3			
Warping constant	Iw=12.988x10 ^9 mm ^6			

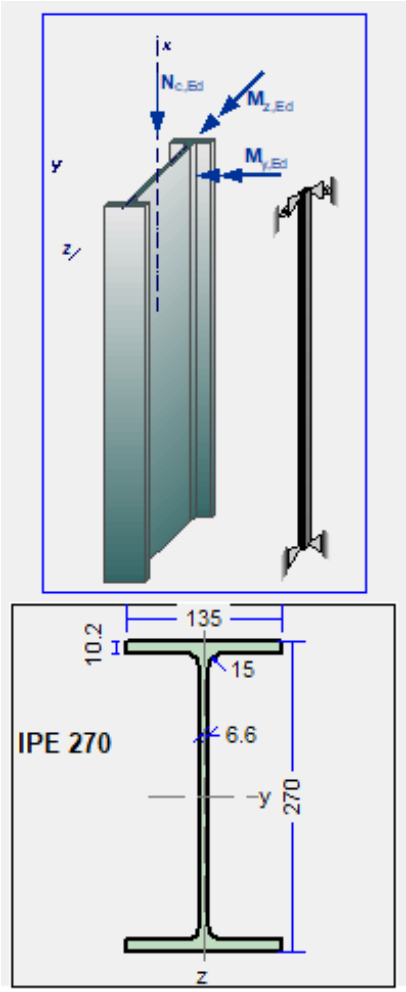
Dimensions and loads

Column length L=5.200 m
Buckling lengthy-y: Lcr,y=1.000x5.200=5.200m
Buckling lengthz-z: Lcr,z=1.000x5.200=5.200m
Beam loads
axial load compression Nc,ed= 100.00 kN
Classification of steel cross-section, Compression Nc (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
33ε=33x0.81=26.73<c/t=28.39<=38ε=38x0.81=30.78
The web is class 2 (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 2, Compression Nc,ed
Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)
Nc.ed=100.00 kN
Compression Resistance Nplrd=A·fy/γM0=[10 ^-3]x2848x355/1.00=1011.04kN
Ned= 100.00 kN < 1011.04 kN =Nc,rd=Nplrd, Is verified
Ned/Nc,rd= 100.00/1011.04= 0.099<1
Flexural Buckling, (Ultimate Limit State) (EN1993-1-1, §6.3.1)
Nc,ed=100.00 kN, Lcr,y=5.200 m, Lcr,z=5.200 m

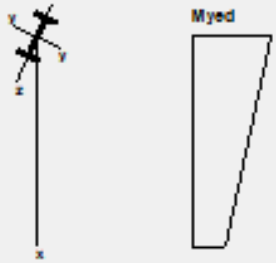
Buckling lengths: Lcr,y=1.000x5200=5200mm, Lcr,z=1.000x5200=5200mm
Non-dimensional slenderness (Cross-section Class: 2) (EC3 §6.3.1.3)
¬λy=(A·fy/Ncr,y)^^½=(Lcr,y/iy)·(1/λ1)=(5200/ 82.6)x(1/76.06)=0.828
¬λz=(A·fy/Ncr,z)^^½=(Lcr,z/iz)·(1/λ1)=(5200/ 22.4)x(1/76.06)=3.057
λ1=π(E/fy)^^½=93.9ε=76.06, ε= (235/fy)^½=0.81
h/b=200/100=2.00>=1.20, tf=8.5mm<=40 mm
y-y buckling curve:a, imperfection factor: áy=0.21, ÷y=0.779 (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(¬λy-0.2)+¬λy²]=0.5x[1+0.21x(0.828-0.2)+0.828²]=0.909
χy=1/[Φy+(Φy²-¬λy²)^^½]=1/[0.909+ (0.909²-0.828²)^½]=0.779 <=1 χy=0.779
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.096
Φz=0.5[1+αz(¬λz-0.2)+¬λz²]=0.5x[1+0.34x(3.057-0.2)+3.057²]=5.658
χz=1/[Φz+(Φz²-¬λz²)^^½]=1/[5.658+ (5.658²-3.057²)^½]=0.096 <=1 χz=0.096
Reduction factor ÷=1/[Ö+(Ö²-¬ë²)^^½], ÷<=1.0, Ö=0.5[1+á(¬ë-0.2)+¬ë²], ÷=0.096 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.096x[10 ^-3]x2848x355/1.00=97.06kN (EC3 Eq.6.47)
Nc,ed= 100.00 kN > 97.06 kN =Nb,rd, Is NOT verified
Nc,ed/Nb,rd= 100.00/97.06= 1.030>1

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



Name of design object	EC3-COLUMN-001					
Structural steel grade (EN1993-1-1 §3.2)	S 355	fy=355N/mm² fu=510N/mm²				
Partial safety factors for actions		γG=	1.35	γQ=	1.50	
Partial factors for materials	EC3	γ_{M0}=	1.00	γ_{M1}=	1.00	γ_{M2}= 1.25

Loading



Nc,ed=	395.00	kN
MyyA,ed=	60.00	kNm
MzzA,ed=	0.00	kNm
MyyB,ed=	25.00	kNm
MzzB,ed=	0.00	kNm

Column height	L=	3.400	m				
Buckling length y-y	Lcr,y=	1.00	X	3.40	3.40	m	
Buckling length z-z	Lcr,z=	1.00	X	3.40	3.40	m	

EC3 (§6.3.2.2)
- ○ EC3-Eq.6.56
- ◉ EC3-Eq.6.57

EC3 (§6.3.3)
- ◉ Method 1
- ○ Method 2
- ○ Method 1 and 2

Cross-section IPE 270-S 355

Dimensions of cross section

Depth of cross section	h=	270.00	mm
Width of cross section	b=	135.00	mm
Web depth	hw=	259.80	mm
Depth of straight portion of web	dw=	219.60	mm
Web thickness	tw=	6.60	mm
Flange thickness	tf=	10.20	mm
Radius of root fillet	r=	15.00	mm
Mass	=	36.10	Kg/m

Properties of cross section

Area	A=	4594	mm²
Second moment of area	Iy=	5.79E007	mm^4
	Iz=	4.199E006	mm^4
Section modulus	Wy=	428900	mm³
	Wz=	62200	mm³
Plastic section modulus	Wpy=	484000	mm³
	Wpz=	96950	mm³
Radius of gyration	iy=	112	mm
	iz=	30	mm
Shear area	Avz=	2213	mm²
	Avy=	2754	mm²
Torsional constant	It=	159448	mm^4
	ip=	116	mm
Torsional modulus	Wt=	15632	mm³
Warping constant	Iw=	7.0578E010	mm^6

EC3-COLUMN-001

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

Profile : IPE 270
Actions : Bending and compression Nc,ed - My,ed - Mz,ed
Nc,ed= 395.00kN
Steel Class : S 355

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

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

Steel cross-section properties

Cross-section IPE 270-S 355

Dimensions of cross section

Depth of cross section	h=	270.00	mm
Width of cross section	b=	135.00	mm
Web depth	hw=	259.80	mm
Depth of straight portion of web	dw=	219.60	mm

Web thickness tw= 6.60 mm
Flange thickness tf= 10.20 mm
Radius of root fillet r= 15.00 mm
Mass = 36.10 Kg/m

Properties of cross section

Area A= 4594 mm ^2
Second moment of area Iy=57.900x10 ^6 mm ^4 Iz= 4.199x10 ^6 mm ^4
Section modulus Wy=428.90x10 ^3 mm ^3 Wz=62.200x10 ^3 mm ^3
Plastic section modulus Wpy=484.00x10 ^3 mm ^3 Wpz=96.950x10 ^3 mm ^3
Radius of gyration iy= 112.3 mm iz= 30.2 mm
Shear area Avz= 2213 mm ^2 Avy= 2754 mm ^2
Torsional constant It= 0.159x10 ^6 mm ^4 ip= 116 mm
Torsional modulus Wt=15.632x10 ^3 mm ^3
Warping constant Iw=70.578x10 ^9 mm ^6

Dimensions and loads

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

Column loads

axial load compression Nc,ed=395.00 kN
bending moment at top MyyA,ed= 60.00 kNm
bending moment at bottom MyyB,ed= 25.00 kNm
bending moment at top MzzA,ed= 0.00 kNm
bending moment at bottom MzzB,ed= 0.00 kNm

Design actions

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

x= 0.00m, Ned=395.00kN, My,ed= 60.00kNm, Mz,ed= 0.00kNm, Vz,ed= 10.29kN, Vy,ed= 0.00kN
x= 1.70m, Ned=395.00kN, My,ed= 42.50kNm, Mz,ed= 0.00kNm, Vz,ed= 10.29kN, Vy,ed= 0.00kN
x= 3.40m, Ned=395.00kN, My,ed= 25.00kNm, Mz,ed= 0.00kNm, Vz,ed= 10.29kN, Vy,ed= 0.00kN

Classification of steel cross-section, 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]395.00/4594 ± [10 ^6]60.00/428.9x10³ ± [10 ^6]0.00/62.2x10³
σ1=226 N/mm², σ2=-54 N/mm² (compression positive)

Web

c=270.0-2x10.2-2x15.0=219.6 mm, t=6.6 mm, c/t=219.6/6.6=33.27
S 355 , t= 6.6<= 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)=395000/(2x6.6x355/1.00)=84.3 mm
α=(219.6/2+84.3)/219.6=0.884>0.5
396x0.810/(13x0.884-1)=30.58<c/t=33.27<=456x0.810/(13x0.884-1)=35.21
The web is class 2 (EN1993-1-1, Tab.5.2)

Flange

c=135.0/2-6.6/2-15.0=49.2 mm, t=10.2 mm, c/t=49.2/10.2=4.82
S 355 , t=10.2<= 40 mm, fy=355 N/mm², ε=(235/355) ^0.5=0.81
c/t=4.82<=9ε=9x0.81=7.29
The flange is class 1 (EN1993-1-1, Tab.5.2)

Overall classification of cross-section is Class 2, Bending and compression

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

Nc.ed=395.00 kN

Compression Resistance Nplrd=A·fy/γM0=[10 ^-3]x4594x355/1.00=1630.87kN
Ned= 395.00 kN < 1630.87 kN =Nc,rd=Nplrd, Is verified
Ned/Nc,rd= 395.00/1630.87= 0.242<1

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

My.ed= 60.00 kNm

Bending Resistance Mply,rd=Wply·fy/γM0=[10 ^-6]x484.00x10 ^3x355/1.00= 171.82kNm
My,ed= 60.00 kNm < 171.82 kNm =My,rd=Mply,rd, Is verified
My,ed/My,rd= 60.00/171.82= 0.349<1

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

Vz.ed= 10.29 kN

Av=A-2b·tf+(tw+2r)tf=4594-2x135.0x10.2+(6.6+2x15.0)x10.2=2213mm² (EC3 §6.2.6.3)
Av= 2213mm² > η·hw·tw= 1.00x(270.0-2x10.2)x6.6=1.00x259.8x6.6= 1715mm²
Plastic Shear Resistance Vpl,z,rd=Av(fy/3^^½)/γM0= [10 ^-3]x2213x(355/1.73)/1.00= 453.64kN
Vz,ed= 10.29 kN < 453.64 kN =Vz,rd=Vpl,z,rd, Is verified
Vz,ed/Vz,rd= 10.29/453.64= 0.023<1
hw/tw=(270.0-2x10.2)/6.6=259.8/6.6=39.36<=72x0.81/1.00=72ε/η=58.32 (η=1.00)
S 355 , t= 6.6<= 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= 395.00kN (Compression), Vz.ed= 10.29kN, My.ed= 60.00kNm

Nplrd=1630.87kN, Mpl,y,rd=171.82kNm, Vpl,z,rd=453.64kN
Ned=395.00kN <= 0.25x1630.87=0.25xNplrd=407.72kN
Ned=395.00kN > [10 ^-3]x0.5x259.8x6.6x355/1.00=0.5hw·tw·fy/γM0=304.36 kN
n=Ned/Nplrd=395/1631= 0.242
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=10.29kN <= 0.50x453.64=0.50xVpl,rd=226.82kN
Effect of shear force is neglected (EC3 §6.2.8.2)

Mny,rd=Mply,rd(1-n)/(1-0.50a)=171.82x0.948, Mny,rd<=Mply,rd, Mny,rd=162.81kNm (EC3 Eq.6.36)
n=Ned/Npl,rd=395/1631=0.242, a=(A-2b·t)/A, a<=0.5, a=(4594-2x135x10.2)/4594=0.40 (§6.2.9.1.5)
My,ed= 60.00 kNm < 162.81 kNm =Mny,rd, Is verified
My,ed/Mny,rd= 60.00/162.81= 0.369<1
Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 395.00kN (Compression), Vz.ed= 10.29kN, My.ed= 42.50kNm
Position at x=1.70 m
Nplrd=1630.87kN, Mpl,y,rd=171.82kNm, Vpl,z,rd=453.64kN
Ned=395.00kN <= 0.25x1630.87=0.25xNplrd=407.72kN
Ned=395.00kN > [10 ^-3]x0.5x259.8x6.6x355/1.00=0.5hw·tw·fy/γM0=304.36 kN
n=Ned/Nplrd=395/1631= 0.242
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=10.29kN <= 0.50x453.64=0.50xVpl,rd=226.82kN
Effect of shear force is neglected (EC3 §6.2.8.2)
Mny,rd=Mply,rd(1-n)/(1-0.50a)=171.82x0.948, Mny,rd<=Mply,rd, Mny,rd=162.81kNm (EC3 Eq.6.36)
n=Ned/Npl,rd=395/1631=0.242, a=(A-2b·t)/A, a<=0.5, a=(4594-2x135x10.2)/4594=0.40 (§6.2.9.1.5)
My,ed= 42.50 kNm < 162.81 kNm =Mny,rd, Is verified
My,ed/Mny,rd= 42.50/162.81= 0.261<1
Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 395.00kN (Compression), Vz.ed= 10.29kN, My.ed= 25.00kNm
Position at x=3.40 m
Nplrd=1630.87kN, Mpl,y,rd=171.82kNm, Vpl,z,rd=453.64kN
Ned=395.00kN <= 0.25x1630.87=0.25xNplrd=407.72kN
Ned=395.00kN > [10 ^-3]x0.5x259.8x6.6x355/1.00=0.5hw·tw·fy/γM0=304.36 kN
n=Ned/Nplrd=395/1631= 0.242
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=10.29kN <= 0.50x453.64=0.50xVpl,rd=226.82kN
Effect of shear force is neglected (EC3 §6.2.8.2)
Mny,rd=Mply,rd(1-n)/(1-0.50a)=171.82x0.948, Mny,rd<=Mply,rd, Mny,rd=162.81kNm (EC3 Eq.6.36)
n=Ned/Npl,rd=395/1631=0.242, a=(A-2b·t)/A, a<=0.5, a=(4594-2x135x10.2)/4594=0.40 (§6.2.9.1.5)
My,ed= 25.00 kNm < 162.81 kNm =Mny,rd, Is verified
My,ed/Mny,rd= 25.00/162.81= 0.154<1
Flexural Buckling, (Ultimate Limit State) (EN1993-1-1, §6.3.1)
Nc,ed=395.00 kN, Lcr,y=3.400 m, Lcr,z=3.400 m

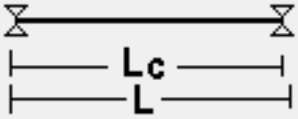
Buckling lengths: Lcr,y=1.000x3400=3400mm, Lcr,z=1.000x3400=3400mm
Non-dimensional slenderness (Cross-section Class: 2) (EC3 §6.3.1.3)
¬λy=(A·fy/Ncr,y)^^½=(Lcr,y/iy)·(1/λ1)=(3400/112.3)x(1/76.06)=0.398
¬λz=(A·fy/Ncr,z)^^½=(Lcr,z/iz)·(1/λ1)=(3400/ 30.2)x(1/76.06)=1.479
λ1=π(E/fy)^^½=93.9ε=76.06, ε= (235/fy)^½=0.81
h/b=270/135=2.00>=1.20, tf=10.2mm<=40 mm
y-y buckling curve:a, imperfection factor: áy=0.21, ÷y=0.953 (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(¬λy-0.2)+¬λy²]=0.5x[1+0.21x(0.398-0.2)+0.398²]=0.600
χy=1/[Φy+(Φy²-¬λy²)^^½]=1/[0.600+ (0.600²-0.398²)^½]=0.953 <=1 χy=0.953
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.350
Φz=0.5[1+αz(¬λz-0.2)+¬λz²]=0.5x[1+0.34x(1.479-0.2)+1.479²]=1.811
χz=1/[Φz+(Φz²-¬λz²)^^½]=1/[1.811+ (1.811²-1.479²)^½]=0.350 <=1 χz=0.350
Reduction factor ÷=1/[Ö+(Ö²-¬ë²)^^½], ÷<=1.0, Ö=0.5[1+á(¬ë-0.2)+¬ë²], ÷=0.350 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.350x[10 ^-3]x4594x355/1.00=570.80kN (EC3 Eq.6.47)
Nc,ed= 395.00 kN < 570.80 kN =Nb,rd, Is verified
Nc,ed/Nb,rd= 395.00/570.80= 0.692<1
Lateral torsional buckling, (ULS) (EN1993-1-1, §6.3.2)
My,ed=60.00 kN, L=3.400m, Lcr,y=3.400m, Lcr,z=3.400m, Lcr,lt=3.400m

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]^^½ -(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/mm²
k·L=3400mm, zg=h/2=270/2=135mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, ψ=0.417, C1=1.380, C2=0.000, C3=0.000
Mcr=[10 ^-6]1.380x[π²x2.1x10 ^5x4.199x10 ^6/3400²]
x{ [(1.0/1.0)²x(70.578x10 ^9/4.199x10 ^6)
+3400²x8.1x10 ^4x0.159x10 ^6/(π²x2.1x10 ^5x4.199x10 ^6)] ^0.5 }= 191.3 kNm
¬λ,lt=(Wpl,y·fy/Mcr)^^½= {[10 ^-6]x484.00x10 ^3x355/191.3}^½=0.948 (EC3 Eq.6.56)
h/b=270/135=2.00<=2.00 buckling curve:b
imperfection factor: á,lt=0.34, â=0.75, ÷,lt=0.731 (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt(¬λ,lt-¬λ,lto)+β¬λ,lt²]=0.5x[1+0.34x(0.948-0.40)+0.75x0.948²]=0.930
χ,lt=1/[Φ,lt+(Φ,lt²-β¬λ,lt²)^^½]=1/[0.930+ (0.930²-0.75x0.930²)^½]=0.731
Reduction factor ÷,lt=1/[Ö,lt+(Ö,lt²-â¬ë,lt²)^^½], ÷,lt<=1.0, 1/¬ë,lt², ÷,lt=0.731 (Eq.6.57)
χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/¬λ,lt²=1/0.948²=1.11 (EC3 §6.3.2.3(2),Eq.6.58)
Kc=1/(1.33-0.33ψ)=0.839, ψ=0.42 (EC3 Tab.6.6)
f=1-0.5(1-kc)[1-2.0(¬λ,lt-0.8)²]=1-0.5x(1-0.839)[1-2.0x(0.948-0.8)²]=0.923, f<=1.0
χ,lt,mod=χ,lt/f=0.731/0.923=0.792, χ,lt,mod<=1.0, χ,lt,mod<=1.11, χ,lt,mod=0.792
Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.792x[10 ^-6]x484.00x10 ^3x355/1.00=136.08kNm (EC3 Eq.6.55)
My,ed= 60.00 kNm < 136.08 kNm =Mb,rd, Is verified
My,ed/Mb,rd= 60.00/136.08= 0.441<1
Axial force and bending moment, (ULS) (EN1993-1-1, §6.3.3)
Ned=395.00 kN, My,ed=60.00 kNm

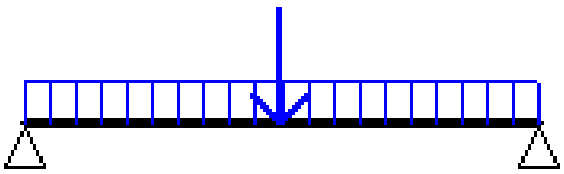
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.61)
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)<=1 (EC3 Eq.6.62)
Nrk=A·fy=[10 ^-3]x4594x355=1630.9 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10 ^-6]x484.00x10 ^3x355=171.8 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.953x[10 ^-3]x4594x355/1.00=1554.2kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.350x[10 ^-3]x4594x355/1.00=570.8kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.792x[10 ^-6]x484.00x10 ^3x355/1.00=136.1kNm
Interaction factors, Method of computation: Method 1 Annex A (EC3 AnnexA)

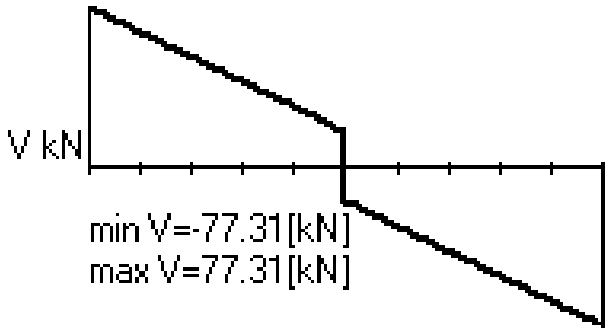
kyy=Cmy·CmLT(μy/(1-Ned/Ncr,y)(1/Cyy), μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y) (EC3 Tab.A.1)
kzy=Cmy·CmLT(μz/(1-Ned/Ncr,y)(1/Czy)0.60(wy/wz)^^½, μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)
Ncr,y=π²EIy/lcr,y²=3.14²x[10 ^-3]x210000x57.900x10 ^6/ 3400²= 10381 kN
Ncr,z=π²EIz/lcr,z²=3.14²x[10 ^-3]x210000x 4.199x10 ^6/ 3400²= 753 kN
Ncr,t=(1/ip²)x(G·It + π²EIw/Lcr,t²) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10 ^-3]x(1/116²)[80769x 0.159x10 ^6+π²x210000x70.578x10 ^9/3400²]=1889 kN
μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1- 395.0/ 10381)/(1-0.953x 395.0/ 10381)=0.998
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1- 395.0/ 753)/(1-0.350x 395.0/ 753)=0.582
alt=1-It/Iy>=0=1- 0.159x10 ^6/57.900x10 ^6=0.997 (EC3 Annex A.1)
wy=Wpl,y/Wel,y<=1.50, wy= 0.484x10 ^6/ 0.429x10 ^6=1.128 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.097x10 ^6/ 0.062x10 ^6=1.559 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=395.00/(1630.90/1.00)=0.242
¬λmax=max(0.398,1.479)=1.480 (EC3 Annex A.1)
Mcr,o=(1.00/1.38)x191.30=138.6, C1=1.00
¬λo=([10 ^-6]x484.00x10 ^3x355/138.6)^^½=1.110
¬λo,lim=0.2C1^^½ [(1-Ned/Ncr,z)(1-Ned/Ncr,t)] ^0.25 (EC3 Annex A.1)
¬λo,lim=0.21.380^^½ [(1-395.0/753)(1-395.0/1889)] ^0.25=0.184
εy=(My,ed/Ned)(A/Wel)=([10 ^3]x60.00/395.00)x(4594.0/428.90x10 ^3)=1.63
Cmy,o=0.79+0.21ψ+0.36(ψ-0.33)x(395.00/10381.0)=0.879, (ψ=0.42) (EC3 Annex A, T.A.1)
¬λo=1.110 > ¬λo,lim=0.184
Cmy=Cmy,o+(1-Cmy,o)(εy^^½·alt)/(1+ εy^½·alt)=
=0.879+(1-0.879)x(1.627^^½x0.997)/(1+ 1.627^½x0.997) = 0.947
Cmlt=Cmy²·alt/[(1-Ned/Ncr,z)(1-Ned/Ncr,t)]^^½ >=1
Cmlt=0.947²x0.997/[(1-395.0/753.0)(1-395.0/1889.0)]^^½=1.458, Cmlt=1.458
Cyy=1+(wy-1)[(2-1.6Cmy²·¬λmax/wy-1.6Cmy²·¬λmax²/wy)npl-blt]>=Wel,y/Wpl,y (Annex A, T.A.1)
blt=0.5alt·¬λo²[My,ed/(χ,lt·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
=0.5x0.997x1.110²[60.0/(0.792x152.3)](0.0/22.1) = 0.000
Cyy=1+(1.128-1)[(2-1.6x0.947²x1.480/1.128-1.6x0.947²x1.480²/1.128)x0.242-0.000]=0.917
Cyy>=428.90x10 ^3/484.00x10 ^3=0.886, Cyy=0.917
Czy=1+(wy-1)[(2-14.0Cmy²·¬λmax²/wy ^5)npl-dlt]>=0.6(wy/wz)^^½(Wel,y/Wpl,y) (Annex A, T.A.1)
dlt=2alt·[¬λo/(0.1+¬λz ^4)][My,ed/(Cmy·χ,lt·Mpl,y,rd)][Mz,ed/(Cmz·Mpl,z,rd)]=
=20.997x[1.110/(0.1+1.479 ^4)][60.0/(0.947x0.792x152.3)][0.0/(0.000x22.1)]=0.000
Czy=1+(1.128-1)[(2-14.0x0.947²x1.480²/1.128 ^5)0.242-0.000]=0.595
Czy>=0.6(1.128/1.500)^^½(428.90x10 ^3/484.00x10 ^3)=0.461, Czy=0.595
Cyy=0.917, Czy=0.595 (Annex A, T.A.1)
kyy=0.947x1.458x0.998/(1-395.00/10381.0)x(1/0.917)=1.562
kzy=0.947x1.458x0.582/(1-395.00/10381.0)x(1/0.595)x0.6x(1.128/1.500)^^½=0.731
Position at x=0.00 m
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
395.0/(0.953x1630.9/1.00)+1.562x60.0/(0.792x171.8/1.00)=0.254+0.689=0.943
0.943< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
395.0/(0.350x1630.9/1.00)+0.731x60.0/(0.792x171.8/1.00)=0.692+0.322=1.014
1.014> 1.000, Is NOT verified
Position at x=1.70 m
Ned/(χy·Nrk/γM1)+kyy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.61)
395.0/(0.953x1630.9/1.00)+1.562x42.5/(0.792x171.8/1.00)=0.254+0.488=0.742
0.742< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
395.0/(0.350x1630.9/1.00)+0.731x42.5/(0.792x171.8/1.00)=0.692+0.228=0.920
0.920< 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)
395.0/(0.953x1630.9/1.00)+1.562x25.0/(0.792x171.8/1.00)=0.254+0.287=0.541
0.541< 1.000, Is verified
Ned/(χz·Nrk/γM1)+kzy·My,ed/(χLT·My,rk/γM1)= (EC3 Eq.6.62)
395.0/(0.350x1630.9/1.00)+0.731x25.0/(0.792x171.8/1.00)=0.692+0.134=0.826
0.826< 1.000, Is verified

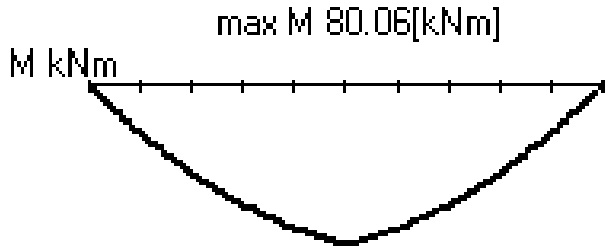
Members in bending, My,ed (EC3 EN1993-1-1:2005, §6.3.2)

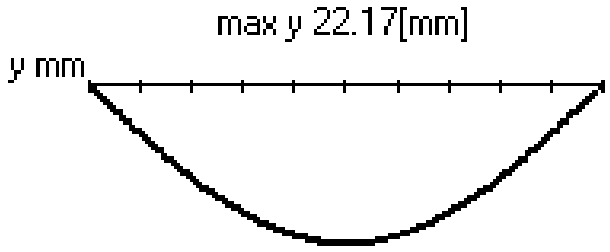
Name of design object	EC3-BEAM-001					
Structural steel grade (EN1993-1-1 §3.2)	S 355	fy=355N/mm² fu=510N/mm²				
Partial safety factors for actions	γG=	1.35	γQ=	1.50		
Partial factors for materials	EC3	γM0=	1.00	γM1=	1.00	γM2= 1.25
Support conditions						
Span length	L=	3.400	m			EC3
Lateral bracing length Lc	Lc=	3.400	m			
Loading	q,ed=	15.00	kN/m			
	Q1,ed=	25.00	kN	x1=	1.700	m
	Q2,ed=	0.00	kN	x2=	0.000	m
	q2,ed=	0.00	kN/m			
	q3,ed=	0.00	kN/m			
	q4,ed=	0.00	kN/m			

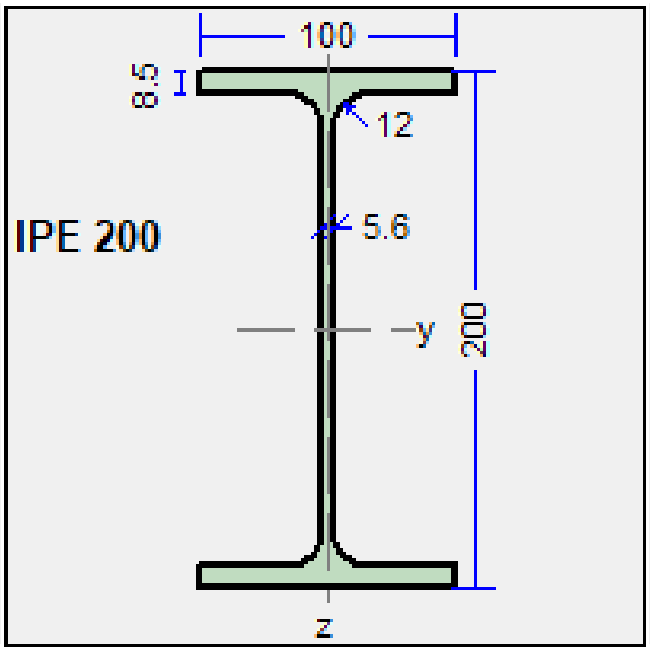
Error, Inadequate design











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	mm²
Second moment of area	Iy=	1.943E007	mm^4
	Iz=	1.424E006	mm^4
Section modulus	Wy=	194300	mm³
	Wz=	28470	mm³
Plastic section modulus	Wpy=	220600	mm³
	Wpz=	44610	mm³
Radius of gyration	iy=	83	mm
	iz=	22	mm
Shear area	Avz=	1400	mm²
	Avy=	1700	mm²
Torsional constant	It=	69801	mm^4
	ip=	86	mm
Torsional modulus	Wt=	8212	mm³
Warping constant	Iw=	1.2988E010	mm^6

EC3-BEAM-001

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

Profile : IPE 200
Actions : Design of beams, Bending
Steel Class : S 355

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

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

Steel cross-section properties

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
Torsional modulus Wt= 8.212x10 ^3 mm ^3
Warping constant Iw=12.988x10 ^9 mm ^6

Dimensions and loads

Beam span L=3.40 m
Buckling lengthy-y: Lcr,y=1.000x3.400=3.400m
Buckling lengthz-z: Lcr,z=1.000x3.400=3.400m

Beam loads

uniform load q1,ed= 15.00 kN/m
concentrated load Q1,ed= 25.00 kN x1= 1.700m

Design actions, shearing forces and bending moments

Shearing forces and bending moments

x/L=0.00, x= 0.00m, Med= 0.00 kNm, Ved= 38.00 kN
x/L=0.10, x= 0.34m, Med= 12.05 kNm, Ved= 32.90 kN
x/L=0.20, x= 0.68m, Med= 22.37 kNm, Ved= 27.80 kN
x/L=0.30, x= 1.02m, Med= 30.96 kNm, Ved= 22.70 kN
x/L=0.40, x= 1.36m, Med= 37.81 kNm, Ved= 17.60 kN
x/L=0.50, x= 1.70m, Med= 42.92 kNm, Ved= -12.50 kN
x/L=0.60, x= 2.04m, Med= 37.81 kNm, Ved= -17.60 kN
x/L=0.70, x= 2.38m, Med= 30.96 kNm, Ved= -22.70 kN
x/L=0.80, x= 2.72m, Med= 22.37 kNm, Ved= -27.80 kN
x/L=0.90, x= 3.06m, Med= 12.05 kNm, Ved= -32.90 kN
x/L=1.00, x= 3.40m, Med= 0.00 kNm, Ved= -38.00 kN
Maximum Med= 42.92 kNm, Maximum Ved= 38.00 kN

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

Maximum vertical deflection
Loading G : wg = 11.54 mm = L/295 < L/200
Loading G+Q : w = 11.54 mm = L/295 < L/200
Beam deflections, Serviceability Limit State (SLS), Is verified

Classification of steel cross-section, Bending My (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

Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed= 42.92 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10 ^-6]x220.60x10 ^3x355/1.00= 78.31kNm
My,ed= 42.92 kNm < 78.31 kNm =My,rd=Mply,rd, Is verified
My,ed/My,rd= 42.92/78.31= 0.548<1

Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)
Vz.ed= 38.00 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^^½)/γM0= [10 ^-3]x1400x(355/1.73)/1.00= 286.86kN
Vz,ed= 38.00 kN < 286.86 kN =Vz,rd=Vpl,z,rd, Is verified
Vz,ed/Vz,rd= 38.00/286.86= 0.132<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)

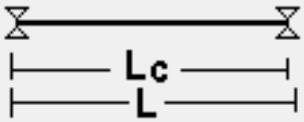
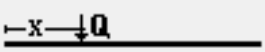
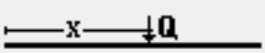
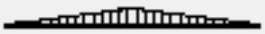
Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 0.00kN, Vz.ed= 25.12kN, My.ed= 27.10kNm
Position at x=0.85 m
Mpl,y,rd=78.31kNm, Vpl,z,rd=286.86kN
Ned=0 kN, Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=25.12kN <= 0.50x286.86=0.50xVpl,rd=143.43kN
Effect of shear force is neglected (EC3 §6.2.8.2)

Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 0.00kN, Vz.ed= 13.79kN, My.ed= 42.92kNm
Position at x=1.70 m
Mpl,y,rd=78.31kNm, Vpl,z,rd=286.86kN

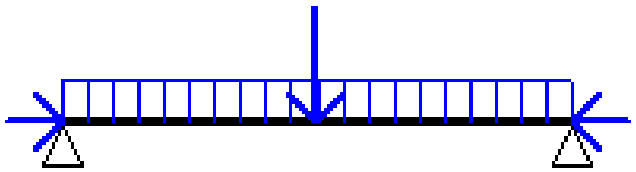
Ned=0 kN, Effect of axial force is neglected (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=13.79kN <= 0.50x286.86=0.50xVpl,rd=143.43kN
Effect of shear force is neglected (EC3 §6.2.8.2)
Lateral torsional buckling, (ULS) (EN1993-1-1, §6.3.2)
My,ed=42.92 kN, L=3.400m, Lcr,y=3.400m, Lcr,z=3.400m, Lcr,lt=3.400m

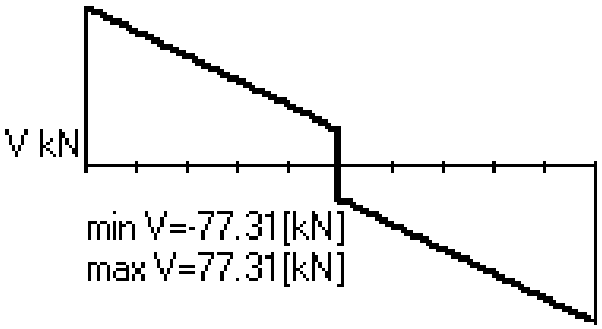
Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko,S.P, Gere,J.M, Theory of elastic stability, McGraw-Hill, 1961
Mcr=C1·[π²EIz/(kL)²]{[(kz/kw)²(Iw/Iz)+(kL)²GIt/(π²EIz)+(C2·zg-C3·zj)²]^^½ -(C2·zg-C3·zj)}
Method of computation C1,C2,C3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
G=E/(2(1+ν))=210000/(2(1+0.30))=80769=8.1x10 ^4 N/mm²
k·L=3400mm, zg=h/2=200/2=100mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=1.348, C2=0.630, C3=0.000
Mcr=[10 ^-6]1.348x[π²x2.1x10 ^5x1.424x10 ^6/3400²]
x{ [(1.0/1.0)²x(12.988x10 ^9/1.424x10 ^6)
+3400²x8.1x10 ^4x0.070x10 ^6/(π²x2.1x10 ^5x1.424x10 ^6)
+(0.630x100)²] ^0.5-(0.630x100) }= 42.9 kNm
¬λ,lt=(Wpl,y·fy/Mcr)^^½= {[10 ^-6]x220.60x10 ^3x355/42.9}^½=1.351 (EC3 Eq.6.56)
h/b=200/100=2.00<=2.00 buckling curve:b
imperfection factor: á,lt=0.34, â=0.75, ÷,lt=0.497 (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt(¬λ,lt-¬λ,lto)+β¬λ,lt²]=0.5x[1+0.34x(1.351-0.40)+0.75x1.351²]=1.346
χ,lt=1/[Φ,lt+(Φ,lt²-β¬λ,lt²)^^½]=1/[1.346+ (1.346²-0.75x1.346²)^½]=0.497
Reduction factor ÷,lt=1/[Ö,lt+(Ö,lt²-â¬ë,lt²)^^½], ÷,lt<=1.0, 1/¬ë,lt², ÷,lt=0.497 (Eq.6.57)
χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/¬λ,lt²=1/1.351²=0.55 (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(¬λ,lt-0.8)²]=1-0.5x(1-0.752)[1-2.0x(1.351-0.8)²]=0.951, f<=1.0
χ,lt,mod=χ,lt/f=0.497/0.951=0.522, χ,lt,mod<=1.0, χ,lt,mod<=0.55, χ,lt,mod=0.522
Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.522x[10 ^-6]x220.60x10 ^3x355/1.00=40.88kNm (EC3 Eq.6.55)
My,ed= 42.92 kNm > 40.88 kNm =Mb,rd, Is NOT verified
My,ed/Mb,rd= 42.92/40.88= 1.050>1

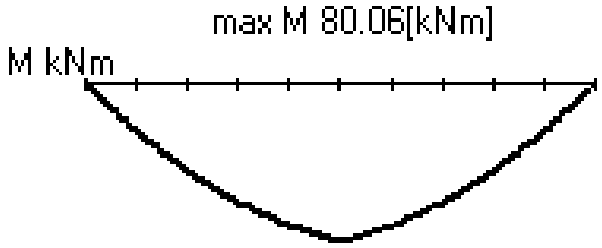
Members in compression and bending, Nc,ed - My,ed (EC3 EN1993-1-1:2005, §6.3.2)

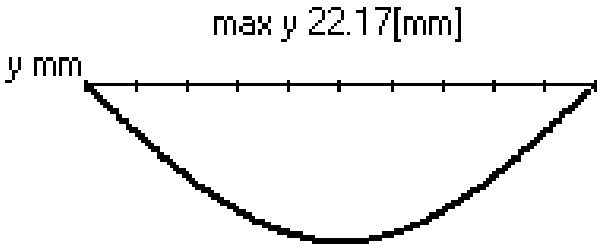
Name of design object	EC3-BEAM-001					
Structural steel grade (EN1993-1-1 §3.2)	S 355	fy=355N/mm² fu=510N/mm²				
Partial safety factors for actions	γG=	1.35	γQ=	1.50		
Partial factors for materials	EC3	γM0=	1.00	γM1=	1.00	γM2= 1.25
Support conditions						
Span length	L=	3.400	m			EC3
Lateral bracing length Lc	Lc=	3.400	m			
Compression	Nc,ed=	1000	kN			
Loading		q,ed=	15.00	kN/m		
		Q1,ed=	25.00	kN	x1=	1.700 m
		Q2,ed=	0.00	kN	x2=	0.000 m
		q2,ed=	0.00	kN/m		
		q3,ed=	0.00	kN/m		
		q4,ed=	0.00	kN/m		
Selected section	IPE 200	Standard sections				

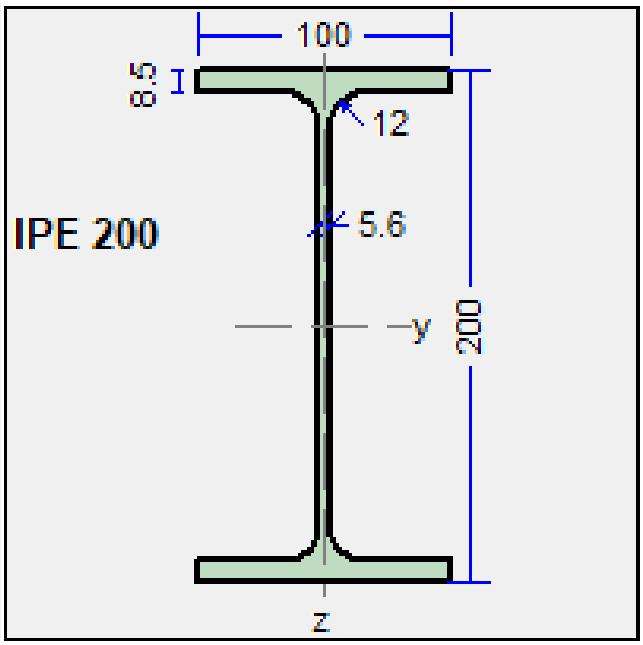
Error, Inadequate design











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	mm²
Second moment of area	Iy=	1.943E007	mm^4
	Iz=	1.424E006	mm^4
Section modulus	Wy=	194300	mm³
	Wz=	28470	mm³
Plastic section modulus	Wpy=	220600	mm³
	Wpz=	44610	mm³
Radius of gyration	iy=	83	mm
	iz=	22	mm
Shear area	Avz=	1400	mm²
	Avy=	1700	mm²
Torsional constant	It=	69801	mm^4
	ip=	86	mm
Torsional modulus	Wt=	8212	mm³
Warping constant	Iw=	1.2988E010	mm^6

EC3-BEAM-001

Members in compression and bending, Nc,ed - My,ed
(EC3 EN1993-1-1:2005, §6.3.2)

Profile : IPE 200
Actions : Design of beams, Bending and compression
Steel Class : S 355

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

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

Steel cross-section properties

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
Torsional modulus Wt= 8.212x10 ^3 mm ^3
Warping constant Iw=12.988x10 ^9 mm ^6

Dimensions and loads

Beam span L=3.40 m
Buckling lengthy-y: Lcr,y=1.000x3.400=3.400m
Buckling lengthz-z: Lcr,z=1.000x3.400=3.400m
Beam loads
uniform load q1,ed= 15.00 kN/m
concentrated load Q1,ed= 25.00 kN x1= 1.700m
axial load compression N,ed=1000.00 kN

Design actions, shearing forces and bending moments

Shearing forces and bending moments

x/L=0.00, x= 0.00m, Med= 0.00 kNm, Ved= 38.00 kN
x/L=0.10, x= 0.34m, Med= 12.05 kNm, Ved= 32.90 kN
x/L=0.20, x= 0.68m, Med= 22.37 kNm, Ved= 27.80 kN
x/L=0.30, x= 1.02m, Med= 30.96 kNm, Ved= 22.70 kN
x/L=0.40, x= 1.36m, Med= 37.81 kNm, Ved= 17.60 kN
x/L=0.50, x= 1.70m, Med= 42.92 kNm, Ved= -12.50 kN
x/L=0.60, x= 2.04m, Med= 37.81 kNm, Ved= -17.60 kN
x/L=0.70, x= 2.38m, Med= 30.96 kNm, Ved= -22.70 kN
x/L=0.80, x= 2.72m, Med= 22.37 kNm, Ved= -27.80 kN
x/L=0.90, x= 3.06m, Med= 12.05 kNm, Ved= -32.90 kN
x/L=1.00, x= 3.40m, Med= 0.00 kNm, Ved= -38.00 kN
Axial load Ned= 1000.00 kN, (compression)
Maximum Med= 42.92 kNm, Maximum Ved= 38.00 kN

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

Maximum vertical deflection
Loading G : wg = 11.54 mm = L/295 < L/200
Loading G+Q : w = 11.54 mm = L/295 < L/200
Beam deflections, Serviceability Limit State (SLS), Is verified

Classification of steel cross-section (EN1993-1-1, §5.5)

Classification of steel cross-section, Compression Nc (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
33ε=33x0.81=26.73<c/t=28.39<=38ε=38x0.81=30.78
The web is class 2 (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 2, Compression Nc,ed
Classification of steel cross-section, Bending My (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
Classification of steel cross-section, 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]1000.00/2848 ± [10 ^6]42.92/194.3x10³ ± [10 ^6]0.00/28.5x10³
σ1=572 N/mm², σ2=130 N/mm² (compression positive)
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
Position of neutral axis for combined Bending and compression
Ned/(2tw·fy/γM0)=1000000/(2x5.6x355/1.00)=251.5 mm
α=(159.0/2+251.5)/159.0=2.082>0.5, α=1>0.5
396x0.810/(13x1.000-1)=26.73<c/t=28.39<=456x0.810/(13x1.000-1)=30.78
The web is class 2 (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 2, Bending and compression Nc,ed+My,ed
Ultimate Limit State, Verification for compression (EN1993-1-1, §6.2.4)
Nc.ed=1000.00 kN
Compression Resistance Nplrd=A·fy/γM0=[10 ^-3]x2848x355/1.00=1011.04kN
Ned= 1000.00 kN < 1011.04 kN =Nc,rd=Nplrd, Is verified
Ned/Nc,rd= 1000.00/1011.04= 0.989<1
Ultimate Limit State, Verification for bending moment y-y (EN1993-1-1, §6.2.5)
My.ed= 42.92 kNm
Bending Resistance Mply,rd=Wply·fy/γM0=[10 ^-6]x220.60x10 ^3x355/1.00= 78.31kNm
My,ed= 42.92 kNm < 78.31 kNm =My,rd=Mply,rd, Is verified
My,ed/My,rd= 42.92/78.31= 0.548<1
Ultimate Limit State, Verification for shear z (EN1993-1-1, §6.2.6)
Vz.ed= 38.00 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^^½)/γM0= [10 ^-3]x1400x(355/1.73)/1.00= 286.86kN
Vz,ed= 38.00 kN < 286.86 kN =Vz,rd=Vpl,z,rd, Is verified
Vz,ed/Vz,rd= 38.00/286.86= 0.132<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= 1000.00kN (Compression), Vz.ed= 38.00kN
Nplrd=1011.04kN, Vpl,z,rd=286.86kN
Ned=1000.00kN > 0.25x1011.04=0.25xNplrd=252.76kN
Ned=1000.00kN > [10 ^-3]x0.5x191.5x5.6x355/1.00=0.5hw·tw·fy/γM0=190.35 kN
n=Ned/Nplrd=1000/1011= 0.989
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=38.00kN <= 0.50x286.86=0.50xVpl,rd=143.43kN
Effect of shear force is neglected (EC3 §6.2.8.2)
n=Ned/Npl,rd=1000/1011=0.989, a=(A-2b·t)/A, a<=0.5, a=(2848-2x100x8.5)/2848=0.40 (§6.2.9.1.5)
My,ed= 0.00 kNm < 78.31 kNm =Mny,rd, Is verified
My,ed/Mny,rd= 0.00/78.31= 0.000<1
Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 1000.00kN (Compression), Vz.ed= 25.12kN, My.ed= 27.10kNm
Position at x=0.85 m
Nplrd=1011.04kN, Mpl,y,rd=78.31kNm, Vpl,z,rd=286.86kN
Ned=1000.00kN > 0.25x1011.04=0.25xNplrd=252.76kN
Ned=1000.00kN > [10 ^-3]x0.5x191.5x5.6x355/1.00=0.5hw·tw·fy/γM0=190.35 kN
n=Ned/Nplrd=1000/1011= 0.989
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=25.12kN <= 0.50x286.86=0.50xVpl,rd=143.43kN
Effect of shear force is neglected (EC3 §6.2.8.2)
Mny,rd=Mply,rd(1-n)/(1-0.50a)=78.31x0.014, Mny,rd<=Mply,rd, Mny,rd=1.07kNm (EC3 Eq.6.36)
n=Ned/Npl,rd=1000/1011=0.989, a=(A-2b·t)/A, a<=0.5, a=(2848-2x100x8.5)/2848=0.40 (§6.2.9.1.5)
My,ed= 27.10 kNm > 1.07 kNm =Mny,rd, Is NOT verified
My,ed/Mny,rd= 27.10/1.07= 25.302>1
Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 1000.00kN (Compression), Vz.ed= 13.79kN, My.ed= 42.92kNm
Position at x=1.70 m
Nplrd=1011.04kN, Mpl,y,rd=78.31kNm, Vpl,z,rd=286.86kN
Ned=1000.00kN > 0.25x1011.04=0.25xNplrd=252.76kN
Ned=1000.00kN > [10 ^-3]x0.5x191.5x5.6x355/1.00=0.5hw·tw·fy/γM0=190.35 kN
n=Ned/Nplrd=1000/1011= 0.989
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=13.79kN <= 0.50x286.86=0.50xVpl,rd=143.43kN
Effect of shear force is neglected (EC3 §6.2.8.2)
Mny,rd=Mply,rd(1-n)/(1-0.50a)=78.31x0.014, Mny,rd<=Mply,rd, Mny,rd=1.07kNm (EC3 Eq.6.36)
n=Ned/Npl,rd=1000/1011=0.989, a=(A-2b·t)/A, a<=0.5, a=(2848-2x100x8.5)/2848=0.40 (§6.2.9.1.5)
My,ed= 42.92 kNm > 1.07 kNm =Mny,rd, Is NOT verified
My,ed/Mny,rd= 42.92/1.07= 40.081>1
Ultimate Limit State, Verification for axial force, shear and bending (EN1993-1-1, §6.2.9)
N.ed= 1000.00kN (Compression), Vz.ed= 12.50kN, My.ed= 42.92kNm
Position at x=1.70 m
Nplrd=1011.04kN, Mpl,y,rd=78.31kNm, Vpl,z,rd=286.86kN
Ned=1000.00kN > 0.25x1011.04=0.25xNplrd=252.76kN
Ned=1000.00kN > [10 ^-3]x0.5x191.5x5.6x355/1.00=0.5hw·tw·fy/γM0=190.35 kN
n=Ned/Nplrd=1000/1011= 0.989
Effect of axial force is considered (EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)
Ved=12.50kN <= 0.50x286.86=0.50xVpl,rd=143.43kN
Effect of shear force is neglected (EC3 §6.2.8.2)
Mny,rd=Mply,rd(1-n)/(1-0.50a)=78.31x0.014, Mny,rd<=Mply,rd, Mny,rd=1.07kNm (EC3 Eq.6.36)
n=Ned/Npl,rd=1000/1011=0.989, a=(A-2b·t)/A, a<=0.5, a=(2848-2x100x8.5)/2848=0.40 (§6.2.9.1.5)
My,ed= 42.92 kNm > 1.07 kNm =Mny,rd, Is NOT verified
My,ed/Mny,rd= 42.92/1.07= 40.081>1

Flexural Buckling, (Ultimate Limit State) (EN1993-1-1, §6.3.1)
Nc,ed=1000.00 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: 2) (EC3 §6.3.1.3)
¬λy=(A·fy/Ncr,y)^^½=(Lcr,y/iy)·(1/λ1)=(3400/ 82.6)x(1/76.06)=0.541
¬λz=(A·fy/Ncr,z)^^½=(Lcr,z/iz)·(1/λ1)=(3400/ 22.4)x(1/76.06)=1.999
λ1=π(E/fy)^^½=93.9ε=76.06, ε= (235/fy)^½=0.81
h/b=200/100=2.00>=1.20, tf=8.5mm<=40 mm
y-y buckling curve:a, imperfection factor: áy=0.21, ÷y=0.911 (T.6.2,T.6.1,Fig.6.4)
Φy=0.5[1+αy(¬λy-0.2)+¬λy²]=0.5x[1+0.21x(0.541-0.2)+0.541²]=0.682
χy=1/[Φy+(Φy²-¬λy²)^^½]=1/[0.682+ (0.682²-0.541²)^½]=0.911 <=1 χy=0.911
z-z buckling curve:b, imperfection factor:αz=0.34, χz=0.210
Φz=0.5[1+αz(¬λz-0.2)+¬λz²]=0.5x[1+0.34x(1.999-0.2)+1.999²]=2.804
χz=1/[Φz+(Φz²-¬λz²)^^½]=1/[2.804+ (2.804²-1.999²)^½]=0.210 <=1 χz=0.210
Reduction factor ÷=1/[Ö+(Ö²-¬ë²)^^½], ÷<=1.0, Ö=0.5[1+á(¬ë-0.2)+¬ë²], ÷=0.210 (EC3 Eq.6.49)
Nb,rd=χ·A·fy/γM1= 0.210x[10 ^-3]x2848x355/1.00=212.32kN (EC3 Eq.6.47)
Nc,ed= 1000.00 kN > 212.32 kN =Nb,rd, Is NOT verified
Nc,ed/Nb,rd= 1000.00/212.32= 4.710>1
Lateral torsional buckling, (ULS) (EN1993-1-1, §6.3.2)
My,ed=42.92 kN, L=3.400m, Lcr,y=3.400m, Lcr,z=3.400m, Lcr,lt=3.400m

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]^^½ -(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/mm²
k·L=3400mm, zg=h/2=200/2=100mm, zj=0mm (EN1993:2002 Eq.C.11)
ky=1.0, kz=1.0, kw=1.0, C1=1.348, C2=0.630, C3=0.000
Mcr=[10 ^-6]1.348x[π²x2.1x10 ^5x1.424x10 ^6/3400²]
x{ [(1.0/1.0)²x(12.988x10 ^9/1.424x10 ^6)
+3400²x8.1x10 ^4x0.070x10 ^6/(π²x2.1x10 ^5x1.424x10 ^6)
+(0.630x100)²] ^0.5-(0.630x100) }= 42.9 kNm
¬λ,lt=(Wpl,y·fy/Mcr)^^½= {[10 ^-6]x194.30x10 ^3x355/42.9}^½=1.268 (EC3 Eq.6.56)
h/b=200/100=2.00<=2.00 buckling curve:b
imperfection factor: á,lt=0.34, â=0.75, ÷,lt=0.541 (T.6.3, T.6.5, Fig.6.4)
Φ,lt=0.5[1+α,lt(¬λ,lt-¬λ,lto)+β¬λ,lt²]=0.5x[1+0.34x(1.268-0.40)+0.75x1.268²]=1.251
χ,lt=1/[Φ,lt+(Φ,lt²-β¬λ,lt²)^^½]=1/[1.251+ (1.251²-0.75x1.251²)^½]=0.541
Reduction factor ÷,lt=1/[Ö,lt+(Ö,lt²-â¬ë,lt²)^^½], ÷,lt<=1.0, 1/¬ë,lt², ÷,lt=0.541 (Eq.6.57)
χ,lt,mod=χ,lt/f, χ,lt,mod<=1, χ,lt,mod<=1/¬λ,lt²=1/1.268²=0.62 (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(¬λ,lt-0.8)²]=1-0.5x(1-0.752)[1-2.0x(1.268-0.8)²]=0.930, f<=1.0
χ,lt,mod=χ,lt/f=0.541/0.930=0.582, χ,lt,mod<=1.0, χ,lt,mod<=0.62, χ,lt,mod=0.582
Mb,rd=χ,lt·Wpl,y·fy/γM1= 0.582x[10 ^-6]x194.30x10 ^3x355/1.00=40.14kNm (EC3 Eq.6.55)
My,ed= 42.92 kNm > 40.14 kNm =Mb,rd, Is NOT verified
My,ed/Mb,rd= 42.92/40.14= 1.069>1
Axial force and bending moment, (ULS) (EN1993-1-1, §6.3.3)
Ned=1000.00 kN, My,ed=42.92 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]x2848x355=1011.0 kN (Tab.6.7)
My,rk=Wpl,y·fy=[10 ^-6]x194.30x10 ^3x355=69.0 kNm
χy·Nrk/γM1=χy·A·fy/γM1= 0.911x[10 ^-3]x2848x355/1.00=921.1kN
χz·Nrk/γM1=χz·A·fy/γM1= 0.210x[10 ^-3]x2848x355/1.00=212.3kN
χLT·My,rk/γM1=χLT·Wpl,y·fy/γM1= 0.000x[10 ^-6]x194.30x10 ^3x355/1.00=0.0kNm
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)^^½, μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)
Ncr,y=π²EIy/lcr,y²=3.14²x[10 ^-3]x210000x19.430x10 ^6/ 3400²= 3484 kN
Ncr,z=π²EIz/lcr,z²=3.14²x[10 ^-3]x210000x 1.424x10 ^6/ 3400²= 255 kN
Ncr,t=(1/ip²)x(G·It + π²EIw/Lcr,t²) (EC3 NCCI SN003b-EN-EU)
Ncr,t=[10 ^-3]x(1/86²)[80769x 0.070x10 ^6+π²x210000x12.988x10 ^9/3400²]=1088 kN
μy=(1-Ned/Ncr,y)/(1-χy·Ned/Ncr,y)=(1-1000.0/ 3484)/(1-0.911x1000.0/ 3484)=0.965
μz=(1-Ned/Ncr,z)/(1-χz·Ned/Ncr,z)=(1-1000.0/ 255)/(1-0.210x1000.0/ 255)=0.013
alt=1-It/Iy>=0=1- 0.070x10 ^6/19.430x10 ^6=0.996 (EC3 Annex A.1)
wy=Wpl,y/Wel,y<=1.50, wy= 0.221x10 ^6/ 0.194x10 ^6=1.135 <= 1.50 (EC3 Annex A.1)
wz=Wpl,z/Wel,z<=1.50, wz= 0.045x10 ^6/ 0.028x10 ^6=1.567 > 1.50 , wz=1.50
npl=Ned/(Nrk/γM1)=1000.00/(1011.00/1.00)=0.989
¬λmax=max(0.541,1.999)=2.000 (EC3 Annex A.1)
Mcr,o=(1.00/1.35)x42.90=31.8, C1=1.00
¬λo=([10 ^-6]x220.60x10 ^3x355/31.8)^^½=1.570
¬λo,lim=0.2C1^^½ [(1-Ned/Ncr,z)(1-Ned/Ncr,t)] ^0.25 (EC3 Annex A.1)
¬λo,lim=0.21.348^^½ [(1-1000.0/255)(1-1000.0/1088)] ^0.25=0.039
εy=(My,ed/Ned)(A/Wel)=([10 ^3]x42.92/1000.00)x(2848.0/194.30x10 ^3)=0.63
Cmy,o=1-0.18(Ned/Ncr,y)=1-0.18x(1000.00/3484.0)=0.948 (EC3 Annex A, T.A.1)
¬λo=1.570 > ¬λo,lim=0.039
Cmy=Cmy,o+(1-Cmy,o)(εy^^½·alt)/(1+ εy^½·alt)=
=0.948+(1-0.948)x(0.629^^½x0.996)/(1+ 0.629^½x0.996) = 0.971

Cmlt=Cmy² ·alt/[(1-Ned/Ncr,z)(1-Ned/Ncr,t)]^^½ >=1
Cmlt=0.971²x0.996/[(1-1000.0/255.0)(1-1000.0/1088.0)]^^½=33.020, Cmlt=33.020
Cyy=1+(wy-1)[(2-1.6Cmy² ·¬λmax/wy-1.6Cmy² ·¬λmax²/wy)npl-blt]>=Wel,y/Wpl,y (Annex A, T.A.1)
blt=0.5alt ·¬λo²[My,ed/(χ,lt ·Mpl,y,rd)](Mz,ed/Mpl,z,rd) =
=0.5x0.996x1.570²[42.9/(0.000x69.0)](0.0/10.1) = 0.000
Cyy=1+(1.135-1)[(2-1.6x0.971²x2.000/1.135-1.6x0.971²x2.000²/1.135)x0.989-0.000]=0.202
Cyy>=194.30x10 ^3/220.60x10 ^3=0.881, Cyy=0.881
Czy=1+(wy-1)[(2-14.0Cmy² ·¬λmax²/wy ^5)npl-dlt]>=0.6(wy/wz)^^½(Wel,y/Wpl,y) (Annex A, T.A.1)
dlt=2alt ·[¬λo/(0.1+¬λz ^4)][My,ed/(Cmy ·χ,lt ·Mpl,y,rd)][Mz,ed/(Cmz ·Mpl,z,rd)]=
=20.996x[1.570/(0.1+1.999 ^4)][42.9/(0.971x0.000x69.0)][0.0/(0.000x10.1)]=0.000
Czy=1+(1.135-1)[(2-14.0x0.971²x2.000²/1.135 ^5)0.989-0.000]=-2.476
Czy>=0.6(1.135/1.500)^^½(194.30x10 ^3/220.60x10 ^3)=0.460, Czy=0.460
Cyy=0.881, Czy=0.460 (Annex A, T.A.1)
kyy=0.971x33.020x0.965/(1-1000.00/3484.0)x(1/0.881)=49.258
kzy=0.971x33.020x0.013/(1-1000.00/3484.0)x(1/0.460)x0.6x(1.135/1.500)^^½=0.663
Ned/(χy ·Nrk/γM1)+kyy ·My,ed/(χLT ·My,rk/γM1)= (EC3 Eq.6.61)
1000.0/(0.911x1011.0/1.00)+49.258x42.9/(0.000x69.0/1.00)=1.086+0.000=1.086
1.086> 1.000, Is NOT verified
Ned/(χz ·Nrk/γM1)+kzy ·My,ed/(χLT ·My,rk/γM1)= (EC3 Eq.6.62)
1000.0/(0.210x1011.0/1.00)+0.663x42.9/(0.000x69.0/1.00)=4.710+0.000=4.710
4.710> 1.000, Is NOT verified

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