

SANDĖLIS NR. 1 (kompleksas)

Load case 1

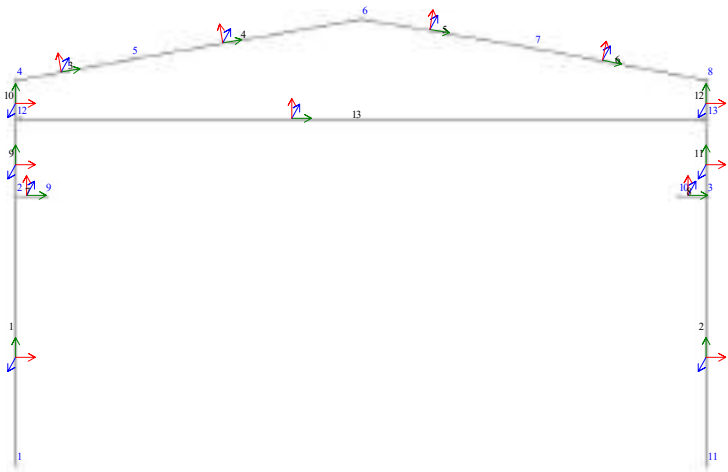


Fig. 1 Nodes, elements, local axes

Load case 1

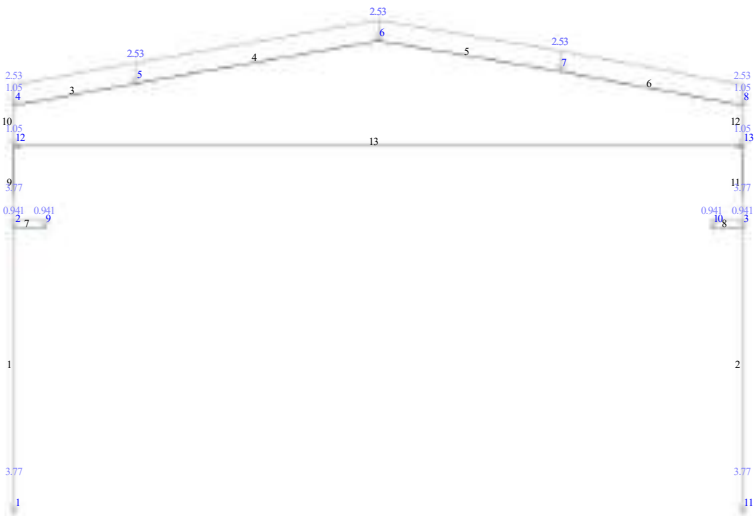


Fig. 2 Load case 1 (dead load)

Load case 2

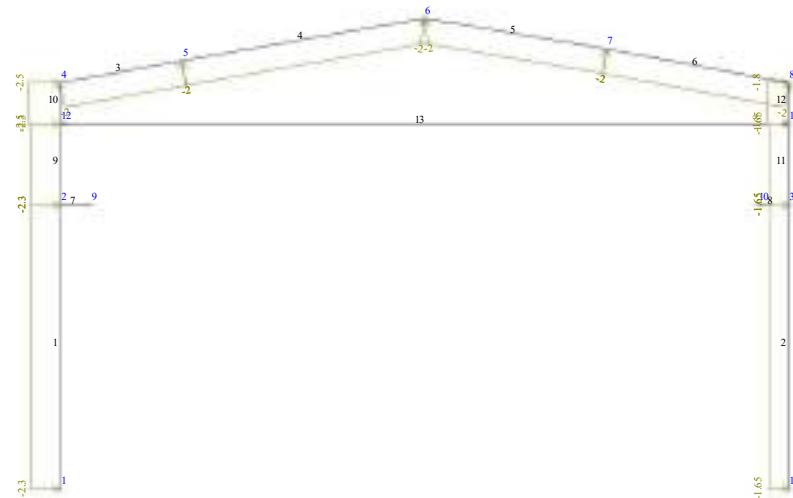


Fig. 3 Load case 2 (wind right)

Load case 3

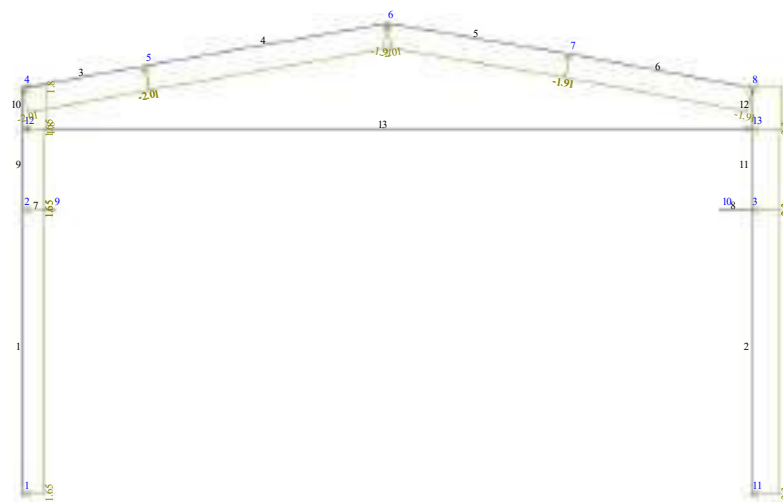


Fig. 4 Load case 3 (wind left)

Load case 4

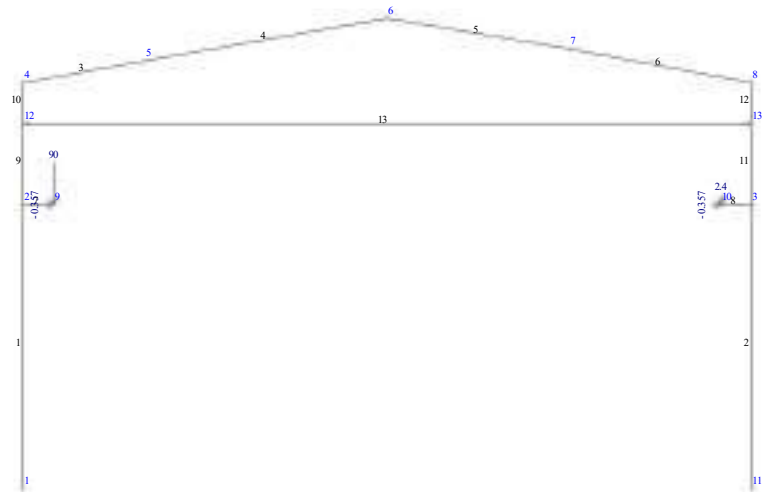


Fig. 5 Load case 4 (crane right)

Load case 5

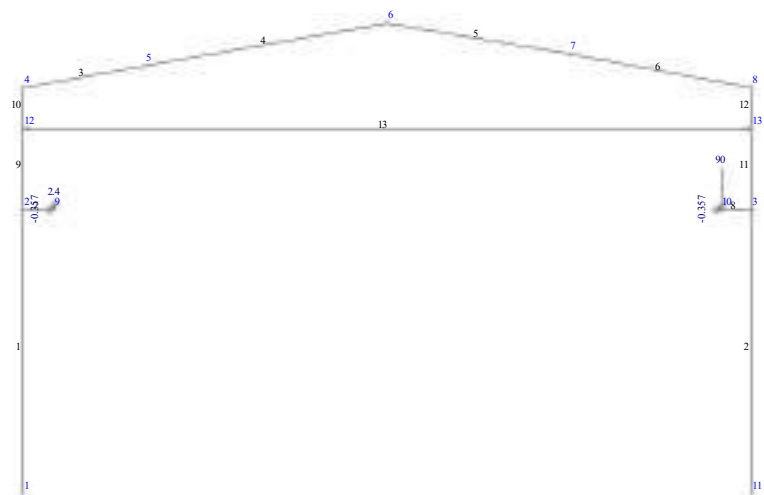


Fig. 6 Load case 5 (crane left)

Load case 6

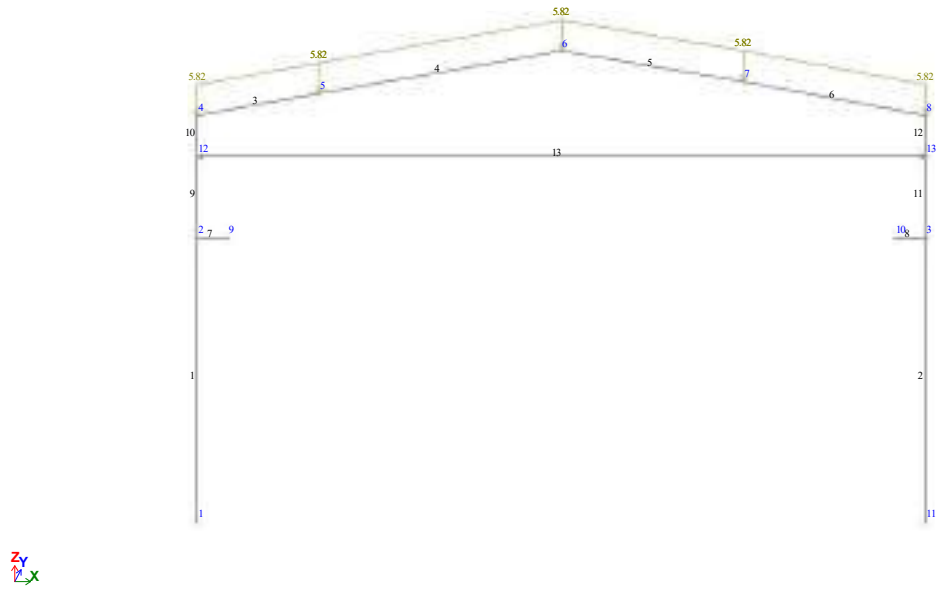


Fig. 7 Load case 6

elem. #	Sec. numb.	FE type	Prop. type	Angle of loc. ax.	Offsets						nodes ##
					AX b	AX e	AY b	AY e	AZ b	AZ e	
1	2	10	1	0	-	-	-	-	-	-	1,2
2	2	10	1	0	-	-	-	-	-	-	11,3
3	3	10	3	0	-	-	-	-	-	-	4,5
4	3	10	3	0	-	-	-	-	-	-	5,6
5	3	10	3	0	-	-	-	-	-	-	6,7
6	3	10	3	0	-	-	-	-	-	-	7,8
7	3	10	2	0	-	-	-	-	-	-	2,9
8	3	10	2	0	-	-	-	-	-	-	10,3
9	2	10	4	0	-	-	-	-	-	-	2,12
10	3	10	3	0	-	-	-	-	-	-	12,4
11	2	10	4	0	-	-	-	-	-	-	3,13
12	3	10	3	0	-	-	-	-	-	-	13,8
13	2	10	5	0	-	-	-	-	-	-	12,13

Fig. 8 Table of elements

node #	Coordinates			Restrains					
	X	Y	Z	X	Y	Z	UX	UY	UZ
1	0.000	0.000	0.000	+	+	+	+	+	+
2	0.000	0.000	7.000	-	-	-	-	-	-
3	18.000	0.000	7.000	-	-	-	-	-	-
4	0.000	0.000	10.000	-	-	-	-	-	-
5	3.000	0.000	10.550	-	-	-	-	-	-
6	9.000	0.000	11.600	-	-	-	-	-	-
7	13.500	0.000	10.850	-	-	-	-	-	-
8	18.000	0.000	10.000	-	-	-	-	-	-
9	0.750	0.000	7.000	-	-	-	-	-	-
10	17.250	0.000	7.000	-	-	-	-	-	-
11	18.000	0.000	0.000	+	+	+	+	+	+
12	0.000	0.000	9.000	-	-	-	-	-	-
13	18.000	0.000	9.000	-	-	-	-	-	-

Fig. 9 Node table

Stif.type	Name	Parameters
1	Rect. bar 40 X 40	Ro=25,E=3.04006e+007,GF=0
		B=40,H=40
2	Rect. bar 40 X 50	Ro=25,E=3.04006e+007,GF=0
		B=40,H=50
3	I-section 400	q=0.56251
		EF=1.50586e+006,EIy=4.18e+004
		EIz=2.41e+003,GIk=27.3
		Y1=1.78,Y2=1.78,Z1=14,Z2=14,RU_Y=0,RU_Z=0
4	Rect. bar 40 X 40	Ro=23.536,E=3.04006e+007,GF=0
		B=40,H=40
5	Pipe 31 x 30	q=0.00725244
		EF=19415,EIy=2.19
		EIz=2.19,GIk=1.66
		Y1=0.727,Y2=0.727,Z1=0.727,Z2=0.727,RU_Y=0,RU_Z=0

Fig. 10 Property table

node #	Type	Direct.	L/G	Value	load.#
9	force	Z	G	90.000	4
9	force	X	G	-0.357	4
10	force	Z	G	2.400	4
10	force	X	G	-0.357	4
9	force	X	G	-0.357	5
9	force	Z	G	2.400	5
10	force	X	G	-0.357	5
10	force	Z	G	90.000	5

Fig. 11 Nodal load table

Coefficients of combinations

Load case #	Type	1	2	3	4	5
1	Dead	1.35	1.35	1.35	1.35	1.35
2	Wind (W1)	0	1.3	1.3	0	0
3	Wind (W2)	0	0	0	1.3	1.3
4	Live	0	1.3	0	1.3	0
5	Live	1.3	0	1.3	0	1.3
6	Snow	0	0	0	0	0
Load case #	Type	6	7	8	9	10
1	Dead	1.35	1.35	1.35	1.35	1.35
2	Wind (W1)	0	1.3	1.3	0	0
3	Wind (W2)	0	0	0	1.3	1.3
4	Live	0	0	0	0	0
5	Live	0	0	0	0	0
6	Snow	1.3	1.3	1.3	1.3	1.3
Load case #	Type	11	12	13	14	15
1	Dead	1.35	1.35	1.35	1.35	1.35
2	Wind (W1)	0	1.3	0	0	1.5
3	Wind (W2)	1.3	0	1.3	1.3	0
4	Live	0	1.3	0	1.3	0
5	Live	1.3	0	1.3	0	0
6	Snow	1.3	1.3	0	1.3	0
Load case #	Type	16	17	18	19	20
1	Dead	1.35	1.35	1.35	1.35	1
2	Wind (W1)	1.3	0	0	0	0
3	Wind (W2)	0	1.3	1.3	0	0
4	Live	0	0	0	0	1
5	Live	0	0	0	0	0
6	Snow	0	0	0	1.3	0
Load case #	Type	21	22	23	24	25
1	Dead	1	1	1	1	1
2	Wind (W1)	0.9	0.9	0	0	0
3	Wind (W2)	0	0	0.9	0.9	0
4	Live	0.9	0	0.9	0	0.9
5	Live	0	0.9	0	0.9	0
6	Snow	0	0	0	0	0.9
Load case #	Type	26	27	28	29	30
1	Dead	1	1	1	1	1
2	Wind (W1)	0.9	0.9	0	0	0.9
3	Wind (W2)	0	0	0.9	0.9	0
4	Live	0	0	0	0	0.9
5	Live	0	0	0	0	0
6	Snow	0.9	0.9	0.9	0.9	0.9
Load case #	Type	31	32	33	34	35

1	Dead	1	1	1	1	1
2	Wind (W1)	0.9	0	0	1	1
3	Wind (W2)	0	0.9	0.9	0	0
4	Live	0	0.9	0	0	0
5	Live	0.9	0	0.9	0	0
6	Snow	0.9	0.9	0.9	0	0
Load case #	Type	36	37	38		
1	Dead	1	1	1		
2	Wind (W1)	0	0	0		
3	Wind (W2)	1	1	0		
4	Live	0	0	0		
5	Live	0	0	0		
6	Snow	0	0	1		

Fig. 12 DCL coefficients

Analysis protocol

Date: 04.10.2023

GenuineIntel Intel(R) Core(TM) i5-7300HQ CPU @ 2.50GHz 4 threads

Microsoft Windows 10 ENU 64-bit. Build 19045

Available RAM = 10219589120

14:27 Reading input data from file C:\Users\Public\Documents\LIRA SAPR\LIRA SAPR 2016
NonCommercial\Data\18mframALL.txt

14:27 Carrying out a check of input data for the main model

Number of nodes = 13 (among them = 13 nodes that are not deleted)

Number of elements = 13 (among them = 13 elements that are not deleted)

MAIN MODEL

14:27 Optimizing order of unknowns

Number of unknowns = 66

STATIC ANALYSIS

14:27 Generating stiffness matrix

14:27 Generating load vectors

14:27 Decomposing stiffness matrix

14:27 Calculating unknowns

14:27 Checking solution

Generating results

14:27 Generating topology

14:27 Generating displacements

14:27 Calculating and generating forces in elements

14:27 Calculating and generating reactions in elements

14:27 Calculating and generating force diagrams in bars

14:27 Calculating and generating deflection diagrams in bars

Total nodal loads on main model:

Load case 1 PX=-2.77556e-017 PY=0 PZ=10.4518 PUX=0 PUY=-1.35308e-015 PUZ=0

Load case 2 PX=-4.06357 PY=0 PZ=-3.67097 PUX=0 PUY=7.56339e-016 PUZ=0

Load case 3 PX=4.07989 PY=0 PZ=-3.59756 PUX=0 PUY=6.8695e-016 PUZ=0

Load case 4 PX=-0.0728078 PY=0 PZ=9.42218 PUX=0 PUY=0 PUZ=0

Load case 5 PX=-0.0728078 PY=0 PZ=9.42218 PUX=0 PUY=0 PUZ=0

Load case 6 PX=0 PY=0 PZ=10.8504 PUX=0 PUY=-2.22045e-016 PUZ=0

Analysis is completed successfully.

Elapsed time = 0 min

Table 1 Displacements (01)

Measurement units for displacements: mm

Measurement units for rotations: RD*1000

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Table 1 Displacements (01)

LC	Node	X, mm	Z, mm	UY, RD*1000
1 - Load case 1				
2 - Load case 2				
3 - Load case 3				
4 - Load case 4				
5 - Load case 5				
6 - Load case 6				
1	1	0	0	0
1	2	-4.2747	-.05478	-.36732
1	3	4.3213	-.05478	.38062
1	4	-3.3240	-.08043	1.4573
1	5	-1.9305	-7.8974	3.1534
1	6	.02420	-19.469	-.01464
1	7	1.1486	-12.413	-2.8640
1	8	3.4195	-.08042	-1.4372
1	9	-4.2747	.22041	-.36680
2	1	0	0	0
2	2	18.647	.02944	2.9254
2	3	11.477	.02236	2.4045
2	4	24.436	.05143	.40017
2	5	23.811	3.6272	-2.2234
2	6	21.822	15.341	-.79130
2	7	21.402	12.568	2.0427
2	8	19.079	.03907	2.8050
2	9	18.647	-2.1646	2.9254
3	1	0	0	0
3	2	-11.787	.02216	-2.4704
3	3	-18.799	.02860	-2.9714
3	4	-19.570	.03872	-2.8508
3	5	-21.183	8.9897	-2.7388
3	6	-22.196	15.094	.85719
3	7	-23.523	6.8661	2.4094
3	8	-24.765	.04998	-.47779
3	9	-11.787	1.8749	-2.4704
4	1	0	0	0
4	2	6.8197	-.12702	2.4426
4	3	7.4488	-.00595	1.5441
4	4	12.192	-.12515	1.1206
4	5	12.475	-1.7080	.02299
4	6	11.984	1.0148	-.57982
4	7	12.213	2.4616	.02716
4	8	11.734	-.00782	1.1541
4	9	6.8197	-2.0589	2.6424
5	1	0	0	0
5	2	-6.6212	-.00577	-1.3978
5	3	-5.9737	-.12720	-2.2906
5	4	-10.552	-.00750	-1.0704
5	5	-10.937	2.0491	-.33447
5	6	-10.779	1.0469	.53193
5	7	-11.178	-1.2858	.32050
5	8	-10.970	-.12546	-1.0283

Table 1 Displacements (01)

LC	Node	X, mm	Z, mm	UY, RD*1000
5	9	-6.6211	1.0399	-1.3925
6	1	0	0	0
6	2	-9.8281	-.07657	-.85289
6	3	9.9350	-.07655	.88343
6	4	-7.6506	-.13377	3.3526
6	5	-4.4439	-18.121	7.2570
6	6	.05549	-44.754	-.03363
6	7	2.6438	-28.515	-6.5916
6	8	7.8699	-.13374	-3.3066
6	9	-9.8281	.56309	-.85289
1	10	4.3213	.23039	.38010
1	11	0	0	0
1	12	-4.2510	-.06473	.46073
1	13	4.3280	-.06472	-.44363
2	10	11.477	1.8258	2.4045
2	11	0	0	0
2	12	23.361	.03785	1.6987
2	13	16.424	.02875	2.5474
3	10	-18.799	-2.1999	-2.9714
3	11	0	0	0
3	12	-16.861	.02849	-2.6069
3	13	-23.620	.03678	-1.7608
4	10	7.4488	1.1495	1.5388
4	11	0	0	0
4	12	10.832	-.12630	1.6145
4	13	10.445	-.00666	1.4082
5	10	-5.9736	-1.9450	-2.4904
5	11	0	0	0
5	12	-9.3604	-.00643	-1.2973
5	13	-9.7162	-.12653	-1.4959
6	10	9.9350	.58602	.88343
6	11	0	0	0
6	12	-9.7825	-.09845	1.0580
6	13	9.9592	-.09842	-1.0187

Table 2 Forces (02)

Measurement units for forces: kN

Measurement units for stresses: kN/m**2

Measurement units for moments: kN*m

Measurement units for distributed moments: (kN*m)/m

Measurement units for distributed shear forces: kN/m

Measurement units for displacements in elements: m

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Table 2 Forces (02). Internal Forces in Bars Elements

LC	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
1 - Load case 1						
2 - Load case 2						
3 - Load case 3						
4 - Load case 4						
5 - Load case 5						
6 - Load case 6						
1	10	1	1	-51.2512	-27.1410	6.78221
1	10	1	2	-24.8909	20.3344	6.78221
1	10	2	1	-51.2454	27.2642	-6.78221

Table 2 Forces (02). Internal Forces in Bars Elements

LC	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
1	10	2	2	-24.8851	-20.2112	-6.78221
1	10	3	1	-19.9449	-49.6708	19.8595
1	10	3	2	-19.2492	-22.2786	16.0645
1	10	3	3	-18.5534	-6.73903	12.2695
1	10	4	1	-18.4537	-6.73903	12.4189
2	10	1	1	20.4569	103.271	-27.1288
2	10	1	2	20.4569	-30.2803	-11.0287
2	10	2	1	15.5429	53.3275	-12.7212
2	10	2	2	15.5429	4.70408	-1.17121
2	10	3	1	14.9137	60.3997	-18.0637
2	10	3	2	14.9137	35.1781	-15.0137
2	10	3	3	14.9137	14.6077	-11.9637
2	10	4	1	14.8166	14.6077	-12.0837
3	10	1	1	15.4006	-54.5715	12.9023
3	10	1	2	15.4006	-4.67979	1.35240
3	10	2	1	19.8793	-103.623	27.1076
3	10	2	2	19.8793	29.7799	11.0075
3	10	3	1	13.6349	15.4137	-13.1575
3	10	3	2	13.6349	-2.31430	-10.0923
3	10	3	3	13.6349	-15.3678	-7.02709
3	10	4	1	13.5778	-15.3678	-7.13695
4	10	1	1	-88.2633	8.89649	3.92411
4	10	1	2	-88.2633	36.3652	3.92411
4	10	2	1	-4.13673	30.5403	-4.63811
4	10	2	2	-4.13673	-1.92646	-4.63811
4	10	3	1	-3.48652	18.7095	-2.40487
4	10	3	2	-3.48652	15.0420	-2.40487
4	10	3	3	-3.48652	11.3746	-2.40487
4	10	4	1	-3.50582	11.3746	-2.37664
5	10	1	1	-4.01264	-26.6798	3.92257
5	10	1	2	-4.01264	.778139	3.92257
5	10	2	1	-88.3873	-4.99450	-4.63657
5	10	2	2	-88.3873	-37.4505	-4.63657
5	10	3	1	-4.12264	-11.4329	.883705
5	10	3	2	-4.12264	-10.0853	.883705
5	10	3	3	-4.12264	-8.73769	.883705
5	10	4	1	-4.11537	-8.73769	.916963
6	10	1	1	-53.2096	-62.2449	15.5265
6	10	1	2	-53.2096	46.4408	15.5265
6	10	2	1	-53.1964	62.5279	-15.5265
6	10	2	2	-53.1964	-46.1578	-15.5265
6	10	3	1	-45.8142	-114.316	45.6971
6	10	3	2	-44.2137	-51.2850	36.9671
6	10	3	3	-42.6132	-1.56671	28.2371
6	10	4	1	-42.3838	-1.56671	28.5803
1	10	4	2	-17.1255	25.5912	4.82898
1	10	4	3	-15.7972	28.7403	-2.76100
1	10	5	1	-15.8179	28.7403	2.63984
1	10	5	2	-16.7667	28.2695	-3.05265
1	10	5	3	-17.7154	14.8138	-8.74515
1	10	6	1	-17.8997	14.8138	-8.36154
1	10	6	2	-18.9749	-10.8495	-14.0540
1	10	6	3	-20.0502	-49.5476	-19.7465
2	10	4	2	14.8166	-12.9189	-5.99262
2	10	4	3	14.8166	-21.8944	.098555

Table 2 Forces (02). Internal Forces in Bars Elements

LC	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
2	10	5	1	14.0090	-21.8944	-4.82575
2	10	5	2	14.0090	-27.6990	-2.63685
2	10	5	3	14.0090	-23.0973	4.29838
2	10	6	1	14.0984	-23.0973	3.99564
2	10	6	2	14.0984	-8.70506	8.57521
2	10	6	3	14.0984	16.1734	13.1547
3	10	4	2	13.5778	-27.7821	-1.01531
3	10	4	3	13.5778	-21.5523	5.10632
3	10	5	1	14.5029	-21.5523	.309274
3	10	5	2	14.5029	-15.8778	4.66605
3	10	5	3	14.5029	-.265451	9.02284
3	10	6	1	14.6939	-.265451	8.70836
3	10	6	2	14.6939	24.6820	13.0818
3	10	6	3	14.6939	59.6439	17.4553
4	10	4	2	-3.50582	4.13633	-2.37664
4	10	4	3	-3.50582	-3.10195	-2.37664
4	10	5	1	-4.09599	-3.10195	-1.07802
4	10	5	2	-4.09599	-5.56097	-1.07802
4	10	5	3	-4.09599	-8.01998	-1.07802
4	10	6	1	-4.11826	-8.01998	-.989552
4	10	6	2	-4.11826	-10.2858	-.989552
4	10	6	3	-4.11826	-12.5517	-.989552
5	10	4	2	-4.11537	-5.94500	.916963
5	10	4	3	-4.11537	-3.15230	.916963
5	10	5	1	-3.57758	-3.15230	2.23115
5	10	5	2	-3.57758	1.93703	2.23115
5	10	5	3	-3.57758	7.02637	2.23115
5	10	6	1	-3.52869	7.02637	2.30769
5	10	6	2	-3.52869	12.3105	2.30769
5	10	6	3	-3.52869	17.5946	2.30769
6	10	4	2	-39.3283	58.8892	11.1203
6	10	4	3	-36.2728	66.1691	-6.33968
6	10	5	1	-36.3203	66.1691	6.06144
6	10	5	2	-38.5028	65.0604	-7.03355
6	10	5	3	-40.6853	34.0815	-20.1285
6	10	6	1	-41.1094	34.0815	-19.2475
6	10	6	2	-43.5829	-24.9836	-32.3425
6	10	6	3	-46.0564	-114.033	-45.4375
1	10	7	1	0	-.264780	.706079
1	10	7	2	0	-.066195	.353039
1	10	7	3	0	0	0
1	10	8	1	0	0	0
1	10	8	2	0	-.066195	-.353039
1	10	8	3	0	-.264780	-.706079
1	10	9	1	-24.1848	20.0696	6.78221
1	10	9	2	-24.1848	33.6341	6.78221
2	10	7	1	0	0	0
2	10	7	2	0	0	0
2	10	7	3	0	0	0
2	10	8	1	0	0	0
2	10	8	2	0	0	0
2	10	8	3	0	0	0
2	10	9	1	20.4569	-30.2803	-11.0287
2	10	9	2	20.4569	-47.7379	-6.42877
3	10	7	1	0	0	0

Table 2 Forces (02). Internal Forces in Bars Elements

LC	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
3	10	7	2	0	0	0
3	10	7	3	0	0	0
3	10	8	1	0	0	0
3	10	8	2	0	0	0
3	10	8	3	0	0	0
3	10	9	1	15.4006	-4.67979	1.35240
3	10	9	2	15.4006	-5.27498	-1.94759
4	10	7	1	.357000	-67.5000	90.0000
4	10	7	2	.357000	-33.7500	90.0000
4	10	7	3	.357000	0	90.0000
4	10	8	1	-.357000	0	-2.40000
4	10	8	2	-.357000	-.900000	-2.40000
4	10	8	3	-.357000	-1.80000	-2.40000
4	10	9	1	1.73673	-31.1347	4.28111
4	10	9	2	1.73673	-22.5725	4.28111
5	10	7	1	.357000	-1.80000	2.40000
5	10	7	2	.357000	-.900000	2.40000
5	10	7	3	.357000	0	2.40000
5	10	8	1	-.357000	0	-90.0000
5	10	8	2	-.357000	-33.7500	-90.0000
5	10	8	3	-.357000	-67.5000	-90.0000
5	10	9	1	-1.61264	-1.02186	4.27957
5	10	9	2	-1.61264	7.53729	4.27957
6	10	7	1	0	0	0
6	10	7	2	0	0	0
6	10	7	3	0	0	0
6	10	8	1	0	0	0
6	10	8	2	0	0	0
6	10	8	3	0	0	0
6	10	9	1	-53.2096	46.4408	15.5265
6	10	9	2	-53.2096	77.4939	15.5265
1	10	10	1	-24.1848	33.6341	16.0367
1	10	10	2	-23.6577	41.6524	16.0367
1	10	10	3	-23.1306	49.6708	16.0367
1	10	11	1	-24.1791	-19.9464	-6.78221
1	10	11	2	-24.1791	-33.5108	-6.78221
1	10	12	1	-24.1791	-33.5108	-16.0367
1	10	12	2	-23.6519	-41.5292	-16.0367
1	10	12	3	-23.1248	-49.5476	-16.0367
2	10	10	1	20.4569	-47.7379	-13.9118
2	10	10	2	20.4569	-54.3813	-12.6618
2	10	10	3	20.4569	-60.3997	-11.4118
2	10	11	1	15.5429	4.70408	-1.17121
2	10	11	2	15.5429	5.66165	2.12877
2	10	12	1	15.5429	5.66165	9.61181
2	10	12	2	15.5429	10.6925	10.5118
2	10	12	3	15.5429	16.1734	11.4118
3	10	10	1	15.4006	-5.27498	-9.23878
3	10	10	2	15.4006	-10.1193	-10.1387
3	10	10	3	15.4006	-15.4137	-11.0387
3	10	11	1	19.8793	29.7799	11.0075
3	10	11	2	19.8793	47.1951	6.40759
3	10	12	1	19.8793	47.1951	13.6987
3	10	12	2	19.8793	53.7320	12.4487
3	10	12	3	19.8793	59.6439	11.1987

Table 2 Forces (02). Internal Forces in Bars Elements

LC	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
4	10	10	1	1.73673	-22.5725	3.86303
4	10	10	2	1.73673	-20.6410	3.86303
4	10	10	3	1.73673	-18.7095	3.86303
4	10	11	1	-1.73673	-12.6459	-4.28111
4	10	11	2	-1.73673	-8.68868	-4.28111
4	10	12	1	-1.73673	-8.68868	-3.86303
4	10	12	2	-1.73673	-10.6201	-3.86303
4	10	12	3	-1.73673	-12.5517	-3.86303
5	10	10	1	-1.61264	7.53729	3.89570
5	10	10	2	-1.61264	9.48514	3.89570
5	10	10	3	-1.61264	11.4329	3.89570
5	10	11	1	1.61264	30.0494	-4.27957
5	10	11	2	1.61264	21.4903	-4.27957
5	10	12	1	1.61264	21.4903	-3.89570
5	10	12	2	1.61264	19.5424	-3.89570
5	10	12	3	1.61264	17.5946	-3.89570
6	10	10	1	-53.2096	77.4939	36.8226
6	10	10	2	-53.2096	95.9052	36.8226
6	10	10	3	-53.2096	114.316	36.8226
6	10	11	1	-53.1964	-46.1578	-15.5265
6	10	11	2	-53.1964	-77.2109	-15.5265
6	10	12	1	-53.1964	-77.2109	-36.8226
6	10	12	2	-53.1964	-95.6223	-36.8226
6	10	12	3	-53.1964	-114.033	-36.8226
1	10	13	1	9.25453	0	0
1	10	13	2	9.25453	0	0
1	10	13	3	9.25453	0	0
2	10	13	1	-7.48303	0	0
2	10	13	2	-7.48303	0	0
2	10	13	3	-7.48303	0	0
3	10	13	1	-7.29119	0	0
3	10	13	2	-7.29119	0	0
3	10	13	3	-7.29119	0	0
4	10	13	1	-4.18076	0	0
4	10	13	2	-4.18076	0	0
4	10	13	3	-4.18076	0	0
5	10	13	1	-3.83872	0	0
5	10	13	2	-3.83872	0	0
5	10	13	3	-3.83872	0	0
6	10	13	1	21.2961	0	0
6	10	13	2	21.2961	0	0
6	10	13	3	21.2961	0	0

Table 3 Displacements from DCL [EUROCODE 1] (03)

Measurement units for displacements: mm

Measurement units for rotations: RD*1000

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Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
1 - DCL1	1	0	0	0
1 - DCL1	2	-14.4958	-0.79021	-2.36460

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
1 - DCL1	3	-2.44680	-.242934	-2.59752
1 - DCL1	4	-18.5665	-.114707	.449408
1 - DCL1	5	-17.2755	-7.50032	3.64794
1 - DCL1	6	-14.5203	-23.8965	.699067
1 - DCL1	7	-13.5983	-17.8737	-3.29062
1 - DCL1	8	-10.3649	-.273931	-3.25671
2 - DCL2	1	0	0	0
2 - DCL2	2	28.8236	-.202960	6.76933
2 - DCL2	3	31.1681	-.049054	5.82567
2 - DCL2	4	45.1278	-.204095	3.94762
2 - DCL2	5	46.4779	-7.67587	1.12883
2 - DCL2	6	45.6709	-3.22852	-1.87007
2 - DCL2	7	46.8752	4.15263	-.928906
2 - DCL2	8	46.0444	-.062354	3.47656
3 - DCL3	1	0	0	0
3 - DCL3	2	10.6782	-.039277	1.58469
3 - DCL3	3	13.0477	-.212737	.648658
3 - DCL3	4	14.4222	-.045273	.989640
3 - DCL3	5	14.8697	-2.60356	.646261
3 - DCL3	6	14.9394	-3.18522	-.369200
3 - DCL3	7	15.2955	-.906596	-.532898
3 - DCL3	8	15.3928	-.221176	.530133
4 - DCL4	1	0	0	0
4 - DCL4	2	-12.2638	-.212783	-.515018
4 - DCL4	3	-9.70525	-.040630	-1.43192
4 - DCL4	4	-14.2813	-.221256	-.441252
4 - DCL4	5	-14.2651	-.436462	.433116
4 - DCL4	6	-13.7538	-3.56196	.355407
4 - DCL4	7	-13.7747	-3.54515	-.433817
4 - DCL4	8	-13.1462	-.047636	-.955314
5 - DCL5	1	0	0	0
5 - DCL5	2	-30.4091	-.049101	-5.69965
5 - DCL5	3	-27.8257	-.204312	-6.60893
5 - DCL5	4	-44.9869	-.062435	-3.39923
5 - DCL5	5	-45.8733	4.63584	-.049456
5 - DCL5	6	-44.4852	-3.51866	1.85627
5 - DCL5	7	-45.3545	-8.60437	-.037809
5 - DCL5	8	-43.7979	-.206458	-3.90174
6 - DCL6	1	0	0	0
6 - DCL6	2	-18.8251	-.174601	-1.62893
6 - DCL6	3	19.0300	-.174565	1.68745
6 - DCL6	4	-14.6495	-.285171	6.42060
6 - DCL6	5	-8.50896	-34.7306	13.8964
6 - DCL6	6	.106386	-85.7289	-.064451
6 - DCL6	7	5.06244	-54.6335	-12.6220
6 - DCL6	8	15.0698	-.285107	-6.33239
7 - DCL7	1	0	0	0
7 - DCL7	2	6.34895	-.134857	2.32036
7 - DCL7	3	34.5245	-.144368	4.93364
7 - DCL7	4	18.3392	-.215737	6.96083
7 - DCL7	5	23.6362	-29.8338	10.8948
7 - DCL7	6	29.5662	-65.0176	-1.13271
7 - DCL7	7	33.9562	-37.6663	-9.86431
7 - DCL7	8	40.8276	-.232352	-2.54554
8 - DCL8	1	0	0	0

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
8 - DCL8	2	6.34895	-.134857	2.32036
8 - DCL8	3	34.5245	-.144368	4.93364
8 - DCL8	4	18.3392	-.215737	6.96083
8 - DCL8	5	23.6362	-.29.8338	10.8948
8 - DCL8	6	29.5662	-.65.0176	-1.13271
8 - DCL8	7	33.9562	-.37.6663	-9.86431
8 - DCL8	8	40.8276	-.232352	-2.54554
9 - DCL9	1	0	0	0
9 - DCL9	2	-34.7385	-.144681	-4.96398
9 - DCL9	3	-6.34890	-.135943	-2.32395
9 - DCL9	4	-41.0699	-.232899	2.57195
9 - DCL9	5	-37.1068	-.22.5944	10.1990
9 - DCL9	6	-29.8585	-.65.3510	1.09275
9 - DCL9	7	-26.6937	-.45.3641	-9.36923
9 - DCL9	8	-18.3630	-.217634	-6.97741
10 - DCL10	1	0	0	0
10 - DCL10	2	-34.7385	-.144681	-4.96398
10 - DCL10	3	-6.34890	-.135943	-2.32395
10 - DCL10	4	-41.0699	-.232899	2.57195
10 - DCL10	5	-37.1068	-.22.5944	10.1990
10 - DCL10	6	-29.8585	-.65.3510	1.09275
10 - DCL10	7	-26.6937	-.45.3641	-9.36923
10 - DCL10	8	-18.3630	-.217634	-6.97741
11 - DCL11	1	0	0	0
11 - DCL11	2	-43.6771	-.152477	-6.85106
11 - DCL11	3	-14.4134	-.307663	-5.41629
11 - DCL11	4	-55.3152	-.243035	1.12687
11 - DCL11	5	-51.8726	-.19.8280	9.74755
11 - DCL11	6	-44.4103	-.63.9377	1.81086
11 - DCL11	7	-41.7853	-.47.1001	-8.93655
11 - DCL11	8	-33.1734	-.387013	-8.36570
12 - DCL12	1	0	0	0
12 - DCL12	2	15.5556	-.306336	5.61791
12 - DCL12	3	44.5804	-.152405	7.01831
12 - DCL12	4	34.7994	-.384695	8.47373
12 - DCL12	5	40.4786	-.32.1398	10.9258
12 - DCL12	6	45.7458	-.63.6475	-1.91548
12 - DCL12	7	50.4444	-.34.3431	-9.82764
12 - DCL12	8	56.6689	-.242910	-.987396
13 - DCL13	1	0	0	0
13 - DCL13	2	-30.4091	-.049101	-5.69965
13 - DCL13	3	-27.8257	-.204312	-6.60893
13 - DCL13	4	-44.9869	-.062435	-3.39923
13 - DCL13	5	-45.8733	4.63584	-.049456
13 - DCL13	6	-44.4852	-.3.51866	1.85627
13 - DCL13	7	-45.3545	-.8.60437	-.037809
13 - DCL13	8	-43.7979	-.206458	-3.90174
14 - DCL14	1	0	0	0
14 - DCL14	2	-25.5318	-.316159	-1.66642
14 - DCL14	3	3.70703	-.143980	-.239283
14 - DCL14	4	-24.6096	-.401857	4.08485
14 - DCL14	5	-20.2644	-.24.9004	10.2301
14 - DCL14	6	-13.6788	-.63.9810	.309996
14 - DCL14	7	-10.2055	-.42.0408	-9.33256
14 - DCL14	8	-2.52182	-.228192	-5.41927

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
15 - DCL 15	1	0	0	0
15 - DCL 15	2	22.4140	-.027065	3.91058
15 - DCL 15	3	22.8338	-.037662	4.10168
15 - DCL 15	4	32.3330	-.027422	2.49475
15 - DCL 15	5	33.2072	-4.82586	.764276
15 - DCL 15	6	32.7646	-2.29730	-1.20600
15 - DCL 15	7	33.5975	2.71460	-.659162
15 - DCL 15	8	33.0651	-.045935	2.33917
16 - DCL 16	1	0	0	0
16 - DCL 16	2	19.6169	-.031481	3.47177
16 - DCL 16	3	21.1122	-.041018	3.74100
16 - DCL 16	4	28.6675	-.035137	2.43472
16 - DCL 16	5	29.6355	-5.36994	1.09779
16 - DCL 16	6	29.4912	-4.59856	-1.08730
16 - DCL 16	7	30.3870	.829368	-.965575
16 - DCL 16	8	30.2032	-.051797	1.91841
17 - DCL 17	1	0	0	0
17 - DCL 17	2	-21.4705	-.041305	-3.81257
17 - DCL 17	3	-19.7612	-.032593	-3.51659
17 - DCL 17	4	-30.7415	-.052299	-1.95414
17 - DCL 17	5	-31.1075	1.86946	.402079
17 - DCL 17	6	-29.9334	-4.93201	1.13816
17 - DCL 17	7	-30.2629	-6.86841	-.470486
17 - DCL 17	8	-28.9875	-.037079	-2.51345
18 - DCL 18	1	0	0	0
18 - DCL 18	2	-21.4705	-.041305	-3.81257
18 - DCL 18	3	-19.7612	-.032593	-3.51659
18 - DCL 18	4	-30.7415	-.052299	-1.95414
18 - DCL 18	5	-31.1075	1.86946	.402079
18 - DCL 18	6	-29.9334	-4.93201	1.13816
18 - DCL 18	7	-30.2629	-6.86841	-.470486
18 - DCL 18	8	-28.9875	-.037079	-2.51345
19 - DCL 19	1	0	0	0
19 - DCL 19	2	-18.8251	-.174601	-1.62893
19 - DCL 19	3	19.0300	-.174565	1.68745
19 - DCL 19	4	-14.6495	-.285171	6.42060
19 - DCL 19	5	-8.50896	-34.7306	13.8964
19 - DCL 19	6	.106386	-85.7289	-.064451
19 - DCL 19	7	5.06244	-54.6335	-12.6220
19 - DCL 19	8	15.0698	-.285107	-6.33239
20 - DCL 20	1	0	0	0
20 - DCL 20	2	2.54498	-.181810	2.07530
20 - DCL 20	3	11.7701	-.060734	1.92482
20 - DCL 20	4	8.86875	-.205593	2.57796
20 - DCL 20	5	10.5452	-9.60556	3.17643
20 - DCL 20	6	12.0091	-18.4543	-.594470
20 - DCL 20	7	13.3621	-9.95198	-2.83691
20 - DCL 20	8	15.1538	-.088245	-2.83075
21 - DCL 21	1	0	0	0
21 - DCL 21	2	18.6457	-.142612	4.46391
21 - DCL 21	3	21.3549	-.040007	3.93453
21 - DCL 21	4	29.6420	-.146788	2.82605
21 - DCL 21	5	30.7278	-6.17024	1.17301
21 - DCL 21	6	30.4505	-4.74824	-1.24866
21 - DCL 21	7	31.4033	1.11326	-1.00114

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
21 - DCL21	8	31.1522	-.052293	2.12607
22 - DCL22	1	0	0	0
22 - DCL22	2	6.54889	-.033490	1.00748
22 - DCL22	3	9.27466	-.149129	.483189
22 - DCL22	4	9.17163	-.040907	.854067
22 - DCL22	5	9.65576	-2.78870	.851301
22 - DCL22	6	9.96286	-4.71937	-.248086
22 - DCL22	7	10.3501	-2.25955	-.737142
22 - DCL22	8	10.7178	-.158174	.161787
23 - DCL23	1	0	0	0
23 - DCL23	2	-8.74589	-.149161	-.392322
23 - DCL23	3	-5.89399	-.034390	-.903866
23 - DCL23	4	-9.96409	-.158229	-.099861
23 - DCL23	5	-9.76753	-1.34397	.709204
23 - DCL23	6	-9.16597	-4.97053	.234985
23 - DCL23	7	-9.03004	-4.01859	-.671089
23 - DCL23	8	-8.30825	-.042481	-.828511
24 - DCL24	1	0	0	0
24 - DCL24	2	-20.8427	-.040039	-3.84874
24 - DCL24	3	-17.9743	-.143512	-4.35520
24 - DCL24	4	-30.4344	-.052348	-2.07184
24 - DCL24	5	-30.8396	2.03756	.387489
24 - DCL24	6	-29.6536	-4.94166	1.23556
24 - DCL24	7	-30.0831	-7.39140	-.407083
24 - DCL24	8	-28.7426	-.148362	-2.79279
25 - DCL25	1	0	0	0
25 - DCL25	2	-6.98231	-.238025	1.06343
25 - DCL25	3	19.9668	-.129039	2.56550
25 - DCL25	4	.763912	-.313478	5.48330
25 - DCL25	5	5.29818	-25.7440	9.70547
25 - DCL25	6	10.8605	-58.8351	-.566762
25 - DCL25	7	14.5202	-35.8619	-8.77212
25 - DCL25	8	21.0633	-.207833	-3.37446
26 - DCL26	1	0	0	0
26 - DCL26	2	3.66266	-.097210	1.49793
26 - DCL26	3	23.5925	-.103549	3.33984
26 - DCL26	4	11.7829	-.154550	4.83486
26 - DCL26	5	15.5001	-20.9422	7.68366
26 - DCL26	6	19.7140	-45.9409	-.757099
26 - DCL26	7	22.7906	-26.7660	-6.95808
26 - DCL26	8	27.6743	-.165625	-1.88866
27 - DCL27	1	0	0	0
27 - DCL27	2	3.66266	-.097210	1.49793
27 - DCL27	3	23.5925	-.103549	3.33984
27 - DCL27	4	11.7829	-.154550	4.83486
27 - DCL27	5	15.5001	-20.9422	7.68366
27 - DCL27	6	19.7140	-45.9409	-.757099
27 - DCL27	7	22.7906	-26.7660	-6.95808
27 - DCL27	8	27.6743	-.165625	-1.88866
28 - DCL28	1	0	0	0
28 - DCL28	2	-23.7289	-.103759	-3.35830
28 - DCL28	3	-3.65642	-.097933	-1.49855
28 - DCL28	4	-27.8231	-.165991	1.90894
28 - DCL28	5	-24.9952	-16.1159	7.21985
28 - DCL28	6	-19.9024	-46.1632	.726553

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
28 - DCL28	7	-17.6426	-31.8979	-6.62803
28 - DCL28	8	-11.7861	-.155813	-4.84324
29 - DCL29	1	0	0	0
29 - DCL29	2	-23.7289	-.103759	-3.35830
29 - DCL29	3	-3.65642	-.097933	-1.49855
29 - DCL29	4	-27.8231	-.165991	1.90894
29 - DCL29	5	-24.9952	-16.1159	7.21985
29 - DCL29	6	-19.9024	-46.1632	.726553
29 - DCL29	7	-17.6426	-31.8979	-6.62803
29 - DCL29	8	-11.7861	-.155813	-4.84324
30 - DCL30	1	0	0	0
30 - DCL30	2	9.80044	-.211529	3.69630
30 - DCL30	3	30.2964	-.108907	4.72962
30 - DCL30	4	22.7564	-.267188	5.84346
30 - DCL30	5	26.7283	-22.4795	7.70435
30 - DCL30	6	30.5004	-45.0276	-1.27894
30 - DCL30	7	33.7828	-24.5505	-6.93364
30 - DCL30	8	38.2352	-.172663	-.849898
31 - DCL31	1	0	0	0
31 - DCL31	2	-2.29643	-.102407	.239880
31 - DCL31	3	18.2161	-.218029	1.27828
31 - DCL31	4	2.28607	-.161307	3.87147
31 - DCL31	5	5.65625	-19.0979	7.38264
31 - DCL31	6	10.0128	-44.9987	-.278360
31 - DCL31	7	12.7296	-27.9233	-6.66963
31 - DCL31	8	17.8008	-.278544	-2.81418
32 - DCL32	1	0	0	0
32 - DCL32	2	-17.5912	-.218078	-1.15993
32 - DCL32	3	3.04753	-.103291	-.108771
32 - DCL32	4	-16.8496	-.278630	2.91754
32 - DCL32	5	-13.7670	-17.6532	7.24054
32 - DCL32	6	-9.11603	-45.2499	.204711
32 - DCL32	7	-6.65057	-29.6824	-6.60358
32 - DCL32	8	-1.22527	-.162851	-3.80448
33 - DCL33	1	0	0	0
33 - DCL33	2	-29.6880	-.108956	-4.61635
33 - DCL33	3	-9.03277	-.212413	-3.56011
33 - DCL33	4	-37.3200	-.172749	.945555
33 - DCL33	5	-34.8391	-14.2717	6.91882
33 - DCL33	6	-29.6036	-45.2210	1.20529
33 - DCL33	7	-27.7037	-33.0552	-6.33957
33 - DCL33	8	-21.6596	-.268732	-5.76876
34 - DCL34	1	0	0	0
34 - DCL34	2	14.3727	-.025349	2.55808
34 - DCL34	3	15.7987	-.032412	2.78521
34 - DCL34	4	21.1121	-.029006	1.85747
34 - DCL34	5	21.8807	-4.27023	.929977
34 - DCL34	6	21.8462	-4.12742	-.805956
34 - DCL34	7	22.5515	.154581	-.821318
34 - DCL34	8	22.4994	-.041347	1.36781
35 - DCL35	1	0	0	0
35 - DCL35	2	14.3727	-.025349	2.55808
35 - DCL35	3	15.7987	-.032412	2.78521
35 - DCL35	4	21.1121	-.029006	1.85747
35 - DCL35	5	21.8807	-4.27023	.929977

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
35 - DCL35	6	21.8462	-4.12742	-.805956
35 - DCL35	7	22.5515	.154581	-.821318
35 - DCL35	8	22.4994	-.041347	1.36781
36 - DCL36	1	0	0	0
36 - DCL36	2	-16.0624	-.032625	-2.83773
36 - DCL36	3	-14.4778	-.026172	-2.59078
36 - DCL36	4	-22.8946	-.041719	-1.39354
36 - DCL36	5	-23.1141	1.09229	.414631
36 - DCL36	6	-22.1720	-4.37442	.842546
36 - DCL36	7	-22.3744	-5.54748	-.454586
36 - DCL36	8	-21.3456	-.030444	-1.91505
37 - DCL37	1	0	0	0
37 - DCL37	2	-16.0624	-.032625	-2.83773
37 - DCL37	3	-14.4778	-.026172	-2.59078
37 - DCL37	4	-22.8946	-.041719	-1.39354
37 - DCL37	5	-23.1141	1.09229	.414631
37 - DCL37	6	-22.1720	-4.37442	.842546
37 - DCL37	7	-22.3744	-5.54748	-.454586
37 - DCL37	8	-21.3456	-.030444	-1.91505
38 - DCL38	1	0	0	0
38 - DCL38	2	-14.1029	-.131363	-1.22022
38 - DCL38	3	14.2563	-.131336	1.26406
38 - DCL38	4	-10.9746	-.214217	4.80997
38 - DCL38	5	-6.37443	-26.0189	10.4104
38 - DCL38	6	.079701	-64.2240	-.048284
38 - DCL38	7	3.79249	-40.9290	-9.45573
38 - DCL38	8	11.2895	-.214169	-4.74389
1 - DCL1	9	-14.4957	1.69045	-2.35673
1 - DCL1	10	-2.44674	-2.32634	-2.86798
1 - DCL1	11	0	0	0
1 - DCL1	12	-18.1629	-.092844	-1.15249
1 - DCL1	13	-7.49054	-.254963	-2.59623
2 - DCL2	9	28.8236	-5.41522	7.03978
2 - DCL2	10	31.1682	4.31622	5.81779
2 - DCL2	11	0	0	0
2 - DCL2	12	40.6359	-.203568	5.07192
2 - DCL2	13	41.9007	-.054315	4.76350
3 - DCL3	9	10.6783	-1.23177	1.59257
3 - DCL3	10	13.0477	.138488	.378206
3 - DCL3	11	0	0	0
3 - DCL3	12	13.3749	-.041744	1.14087
3 - DCL3	13	14.6826	-.216139	.842830
4 - DCL4	9	-12.2638	.038212	-.244566
4 - DCL4	10	-9.70519	-1.11855	-1.43979
4 - DCL4	11	0	0	0
4 - DCL4	12	-13.6656	-.216198	-.740813
4 - DCL4	13	-12.1607	-.043483	-1.05274
5 - DCL5	9	-30.4091	4.22165	-5.69177
5 - DCL5	10	-27.8256	-5.29628	-6.87938
5 - DCL5	11	0	0	0
5 - DCL5	12	-40.9266	-.054375	-4.67186
5 - DCL5	13	-39.3788	-.205307	-4.97342
6 - DCL6	9	-18.8251	1.04672	-1.62825
6 - DCL6	10	19.0300	1.09064	1.68677
6 - DCL6	11	0	0	0

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
6 - DCL 6	12	-18.7328	-.217065	2.02734
6 - DCL 6	13	19.0714	-.217018	-1.95210
7 - DCL 7	9	6.34895	-1.87551	2.32104
7 - DCL 7	10	34.5245	3.55548	4.93296
7 - DCL 7	11	0	0	0
7 - DCL 7	12	12.8050	-.165965	4.32071
7 - DCL 7	13	41.2445	-.178193	1.48695
8 - DCL 8	9	6.34895	-1.87551	2.32104
8 - DCL 8	10	34.5245	3.55548	4.93296
8 - DCL 8	11	0	0	0
8 - DCL 8	12	12.8050	-.165965	4.32071
8 - DCL 8	13	41.2445	-.178193	1.48695
9 - DCL 9	9	-34.7385	3.57792	-4.96330
9 - DCL 9	10	-6.34890	-1.87928	-2.32463
9 - DCL 9	11	0	0	0
9 - DCL 9	12	-41.4965	-.178596	-1.49202
9 - DCL 9	13	-12.8168	-.167362	-4.32929
10 - DCL 10	9	-34.7385	3.57792	-4.96330
10 - DCL 10	10	-6.34890	-1.87928	-2.32463
10 - DCL 10	11	0	0	0
10 - DCL 10	12	-41.4965	-.178596	-1.49202
10 - DCL 10	13	-12.8168	-.167362	-4.32929
11 - DCL 11	9	-43.6771	4.98184	-6.84319
11 - DCL 11	10	-14.4133	-4.50515	-5.68674
11 - DCL 11	11	0	0	0
11 - DCL 11	12	-54.1330	-.187287	-3.24347
11 - DCL 11	13	-25.9338	-.338186	-6.34880
12 - DCL 12	9	15.5556	-4.65504	5.88837
12 - DCL 12	10	44.5805	5.10735	7.01043
12 - DCL 12	11	0	0	0
12 - DCL 12	12	27.4294	-.336480	6.50031
12 - DCL 12	13	55.3457	-.187194	3.38813
13 - DCL 13	9	-30.4091	4.22165	-5.69177
13 - DCL 13	10	-27.8256	-5.29628	-6.87938
13 - DCL 13	11	0	0	0
13 - DCL 13	12	-40.9266	-.054375	-4.67186
13 - DCL 13	13	-39.3788	-.205307	-4.97342
14 - DCL 14	9	-25.5318	.798394	-1.39597
14 - DCL 14	10	3.70709	-.327421	-.247156
14 - DCL 14	11	0	0	0
14 - DCL 14	12	-26.8721	-.349110	.687574
14 - DCL 14	13	1.28430	-.176362	-2.42812
15 - DCL 15	9	22.4140	-2.96038	3.91126
15 - DCL 15	10	22.8338	3.03822	4.10101
15 - DCL 15	11	0	0	0
15 - DCL 15	12	29.5157	-.027376	3.14714
15 - DCL 15	13	30.2632	-.041000	3.24445
16 - DCL 16	9	19.6169	-2.63569	3.47245
16 - DCL 16	10	21.1122	2.76435	3.74032
16 - DCL 16	11	0	0	0
16 - DCL 16	12	26.0115	-.033053	2.89232
16 - DCL 16	13	27.7995	-.045314	2.86233
17 - DCL 17	9	-21.4705	2.81774	-3.81189
17 - DCL 17	10	-19.7612	-2.67042	-3.51727
17 - DCL 17	11	0	0	0

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
17 - DCL 17	12	-28.2900	-.045684	-2.92041
17 - DCL 17	13	-26.2619	-.034482	-2.95392
18 - DCL 18	9	-21.4705	2.81774	-3.81189
18 - DCL 18	10	-19.7612	-2.67042	-3.51727
18 - DCL 18	11	0	0	0
18 - DCL 18	12	-28.2900	-.045684	-2.92041
18 - DCL 18	13	-26.2619	-.034482	-2.95392
19 - DCL 19	9	-18.8251	1.04672	-1.62825
19 - DCL 19	10	19.0300	1.09064	1.68677
19 - DCL 19	11	0	0	0
19 - DCL 19	12	-18.7328	-.217065	2.02734
19 - DCL 19	13	19.0714	-.217018	-1.95210
20 - DCL 20	9	2.54502	-1.83849	2.27566
20 - DCL 20	10	11.7702	1.37992	1.91897
20 - DCL 20	11	0	0	0
20 - DCL 20	12	6.58181	-.191040	2.07525
20 - DCL 20	13	14.7733	-.071390	.964641
21 - DCL 21	9	18.6458	-3.58076	4.64428
21 - DCL 21	10	21.3550	2.90819	3.92921
21 - DCL 21	11	0	0	0
21 - DCL 21	12	26.5238	-.144343	3.44271
21 - DCL 21	13	28.5109	-.044840	3.11652
22 - DCL 22	9	6.54893	-.791797	1.01280
22 - DCL 22	10	9.27470	.123045	.302818
22 - DCL 22	11	0	0	0
22 - DCL 22	12	8.34984	-.036461	.822015
22 - DCL 22	13	10.3654	-.152722	.502735
23 - DCL 23	9	-8.74585	.054863	-.211951
23 - DCL 23	10	-5.89395	-.714982	-.909184
23 - DCL 23	11	0	0	0
23 - DCL 23	12	-9.67723	-.152763	-.432444
23 - DCL 23	13	-7.53007	-.037619	-.760983
24 - DCL 24	9	-20.8427	2.84382	-3.84342
24 - DCL 24	10	-17.9742	-3.50013	-4.53557
24 - DCL 24	11	0	0	0
24 - DCL 24	12	-27.8512	-.044881	-3.05314
24 - DCL 24	13	-25.6755	-.145501	-3.37476
25 - DCL 25	9	-6.98227	-1.12582	1.24380
25 - DCL 25	10	19.9668	1.79239	2.56018
25 - DCL 25	11	0	0	0
25 - DCL 25	12	-3.30578	-.267017	2.86605
25 - DCL 25	13	22.6921	-.159309	-.093105
26 - DCL 26	9	3.66266	-1.22095	1.49845
26 - DCL 26	10	23.5925	2.40104	3.33932
26 - DCL 26	11	0	0	0
26 - DCL 26	12	7.96990	-.119275	2.94190
26 - DCL 26	13	28.0734	-.127425	.932152
27 - DCL 27	9	3.66266	-1.22095	1.49845
27 - DCL 27	10	23.5925	2.40104	3.33932
27 - DCL 27	11	0	0	0
27 - DCL 27	12	7.96990	-.119275	2.94190
27 - DCL 27	13	28.0734	-.127425	.932152
28 - DCL 28	9	-23.7289	2.41467	-3.35777
28 - DCL 28	10	-3.65642	-1.22214	-1.49907
28 - DCL 28	11	0	0	0

Table 3 Displacements from DCL [EUROCODE 1] (03)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
28 - DCL28	12	-28.2311	-.127695	-.933250
28 - DCL28	13	-7.96752	-.120204	-2.94535
29 - DCL29	9	-23.7289	2.41467	-3.35777
29 - DCL29	10	-3.65642	-1.22214	-1.49907
29 - DCL29	11	0	0	0
29 - DCL29	12	-28.2311	-.127695	-.933250
29 - DCL29	13	-7.96752	-.120204	-2.94535
30 - DCL30	9	9.80048	-3.07397	3.87667
30 - DCL30	10	30.2965	3.43562	4.72430
30 - DCL30	11	0	0	0
30 - DCL30	12	17.7195	-.232951	4.39497
30 - DCL30	13	37.4742	-.133426	2.19960
31 - DCL31	9	-2.29639	-.285009	.245198
31 - DCL31	10	18.2162	.650465	1.09791
31 - DCL31	11	0	0	0
31 - DCL31	12	-4.54476	-.125069	1.77427
31 - DCL31	13	19.3288	-.241308	-.414183
32 - DCL32	9	-17.5911	.561651	-.979559
32 - DCL32	10	3.04757	-.187561	-.114090
32 - DCL32	11	0	0	0
32 - DCL32	12	-18.4815	-.241371	.519814
32 - DCL32	13	1.43327	-.126205	-1.67790
33 - DCL33	9	-29.6880	3.35061	-4.61103
33 - DCL33	10	-9.03273	-2.97271	-3.74048
33 - DCL33	11	0	0	0
33 - DCL33	12	-36.6555	-.133489	-2.10088
33 - DCL33	13	-16.7121	-.234087	-4.29168
34 - DCL34	9	14.3727	-1.94420	2.55860
34 - DCL34	10	15.7987	2.05620	2.78468
34 - DCL34	11	0	0	0
34 - DCL34	12	19.1103	-.026882	2.15953
34 - DCL34	13	20.7525	-.035963	2.10381
35 - DCL35	9	14.3727	-1.94420	2.55860
35 - DCL35	10	15.7987	2.05620	2.78468
35 - DCL35	11	0	0	0
35 - DCL35	12	19.1103	-.026882	2.15953
35 - DCL35	13	20.7525	-.035963	2.10381
36 - DCL36	9	-16.0624	2.09538	-2.83721
36 - DCL36	10	-14.4778	-1.96955	-2.59130
36 - DCL36	11	0	0	0
36 - DCL36	12	-21.1130	-.036237	-2.14620
36 - DCL36	13	-19.2929	-.027940	-2.20452
37 - DCL37	9	-16.0624	2.09538	-2.83721
37 - DCL37	10	-14.4778	-1.96955	-2.59130
37 - DCL37	11	0	0	0
37 - DCL37	12	-21.1130	-.036237	-2.14620
37 - DCL37	13	-19.2929	-.027940	-2.20452
38 - DCL38	9	-14.1029	.783511	-1.21970
38 - DCL38	10	14.2563	.816419	1.26354
38 - DCL38	11	0	0	0
38 - DCL38	12	-14.0336	-.163186	1.51880
38 - DCL38	13	14.2872	-.163151	-1.46243

Table 4 Displacements from DCL [EUROCODE 2] (04)

Measurement units for displacements: mm

Measurement units for rotations: RD*1000

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Table 4 Displacements from DCL [EUROCODE 2] (04)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
1 - DCL1	1	0	0	0
1 - DCL1	2	-10.9247	-.387577	1.83361
1 - DCL1	3	32.5577	-.205934	4.21239
1 - DCL1	4	1.82719	-.509056	8.89595
1 - DCL1	5	9.15205	-.415904	15.6502
1 - DCL1	6	18.0969	-.94.8137	-.94.2162
1 - DCL1	7	24.0089	-.57.7010	-14.1428
1 - DCL1	8	34.5356	-.332984	-5.38456
2 - DCL2	1	0	0	0
2 - DCL2	2	4.45869	-.264496	3.16805
2 - DCL2	3	17.0070	-.082883	2.83014
2 - DCL2	4	13.8017	-.296323	3.64835
2 - DCL2	5	16.1074	-13.2237	4.29163
2 - DCL2	6	18.0100	-24.7611	-.889509
2 - DCL2	7	19.8708	-13.0659	-3.82575
2 - DCL2	8	22.2177	-.120303	-.209025
3 - DCL3	1	0	0	0
3 - DCL3	2	10.4645	-.042017	1.56633
3 - DCL3	3	13.2638	-.215476	.667690
3 - DCL3	4	14.2560	-.049295	1.06250
3 - DCL3	5	14.7732	-2.99843	.803934
3 - DCL3	6	14.9406	-4.15868	-.369933
3 - DCL3	7	15.3529	-1.52727	-.676102
3 - DCL3	8	15.5638	-.225197	.458270
4 - DCL4	1	0	0	0
4 - DCL4	2	-12.4776	-.215523	-.533384
4 - DCL4	3	-9.48919	-.043369	-1.41289
4 - DCL4	4	-14.4475	-.225278	-.368387
4 - DCL4	5	-14.3617	-.831336	.590789
4 - DCL4	6	-13.7525	-4.53542	.354674
4 - DCL4	7	-13.7173	-4.16583	-.577021
4 - DCL4	8	-12.9753	-.051657	-1.02717
5 - DCL5	1	0	0	0
5 - DCL5	2	-30.6229	-.051840	-5.71801
5 - DCL5	3	-27.6096	-.207051	-6.58990
5 - DCL5	4	-45.1531	-.066457	-3.32637
5 - DCL5	5	-45.9698	4.24097	.108217
5 - DCL5	6	-44.4840	-4.49212	1.85554
5 - DCL5	7	-45.2970	-9.22506	-.181013
5 - DCL5	8	-43.6269	-.210479	-3.97360
6 - DCL6	1	0	0	0
6 - DCL6	2	-30.6229	-.051840	-5.71801
6 - DCL6	3	-27.6096	-.207051	-6.58990
6 - DCL6	4	-45.1531	-.066457	-3.32637
6 - DCL6	5	-45.9698	4.24097	.108217
6 - DCL6	6	-44.4840	-4.49212	1.85554
6 - DCL6	7	-45.2970	-9.22506	-.181013
6 - DCL6	8	-43.6269	-.210479	-3.97360
7 - DCL7	1	0	0	0
7 - DCL7	2	-18.7709	-.356615	-.236826
7 - DCL7	3	21.2374	-.357060	.698813
7 - DCL7	4	-12.6008	-.468287	6.56127

Table 4 Displacements from DCL [EUROCODE 2] (04)

DCL No.	Node	X, mm	Z, mm	UY, RD*1000
7 - DCL 7	5	-6.52895	-34.6650	13.6336
7 - DCL 7	6	1.73538	-83.9190	-.129839
7 - DCL 7	7	6.51650	-53.6668	-12.2958
7 - DCL 7	8	16.2717	-4.69065	-6.23439
1 - DCL 1	9	-10.9246	-1.91310	2.13414
1 - DCL 1	10	32.5578	2.94892	4.20362
1 - DCL 1	11	0	0	0
1 - DCL 1	12	-4.80113	-4.34240	4.69997
1 - DCL 1	13	37.0989	-.254728	-.081236
2 - DCL 2	9	4.45875	-2.79081	3.46851
2 - DCL 2	10	17.0071	2.03533	2.82144
2 - DCL 2	11	0	0	0
2 - DCL 2	12	10.5103	-.276850	3.04376
2 - DCL 2	13	21.5108	-.097376	1.51350
3 - DCL 3	9	10.4646	-1.22075	1.57422
3 - DCL 3	10	13.2638	.150008	.397212
3 - DCL 3	11	0	0	0
3 - DCL 3	12	13.1624	-.044981	1.16391
3 - DCL 3	13	14.8990	-.219375	.820648
4 - DCL 4	9	-12.4775	.049232	-.262906
4 - DCL 4	10	-9.48913	-1.10703	-1.42079
4 - DCL 4	11	0	0	0
4 - DCL 4	12	-13.8781	-.219435	-.717776
4 - DCL 4	13	-11.9443	-.046719	-1.07492
5 - DCL 5	9	-30.6228	4.23268	-5.71011
5 - DCL 5	10	-27.6095	-5.28476	-6.86038
5 - DCL 5	11	0	0	0
5 - DCL 5	12	-4.11391	-.057611	-4.64882
5 - DCL 5	13	-39.1624	-.208543	-4.99560
6 - DCL 6	9	-30.6228	4.23268	-5.71011
6 - DCL 6	10	-27.6095	-5.28476	-6.86038
6 - DCL 6	11	0	0	0
6 - DCL 6	12	-4.11391	-.057611	-4.64882
6 - DCL 6	13	-39.1624	-.208543	-4.99560
7 - DCL 7	9	-18.7707	-.317876	.040847
7 - DCL 7	10	21.2376	.028170	.421141
7 - DCL 7	11	0	0	0
7 - DCL 7	12	-16.9575	-.399507	2.47852
7 - DCL 7	13	20.2720	-4.00079	-2.09261

Table 5 Forces from DCL [EUROCODE 1] (05)

Measurement units for forces: kN

Measurement units for stresses: kN/m**2

Measurement units for moments: kN*m

Measurement units for distributed moments: (kN*m)/m

Measurement units for distributed shear forces: kN/m

Measurement units for displacements in elements: m

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Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
1 - DCL 1	10	1	1	-72.043	-71.301	14.112
1 - DCL 1	10	1	2	-37.775	27.485	14.112
1 - DCL 1	10	2	1	-185.94	28.700	-15.076
1 - DCL 1	10	2	2	-151.67	-76.832	-15.076

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
1 - DCL1	10	3	1	-31.494	-80.006	27.010
1 - DCL1	10	3	2	-30.589	-42.577	22.076
1 - DCL1	10	3	3	-29.685	-12.671	17.143
1 - DCL1	10	4	1	-29.545	-12.671	17.382
1 - DCL1	10	4	2	-27.818	25.242	7.5155
2 - DCL2	10	1	1	-158.16	116.14	-22.509
2 - DCL2	10	1	2	-123.89	34.649	-77443
2 - DCL2	10	2	1	-51.220	148.66	-32.251
2 - DCL2	10	2	2	-16.952	-22.524	-16.659
2 - DCL2	10	3	1	-10.501	42.225	-1.8152
2 - DCL2	10	3	2	-9.5972	38.834	-2.6312
2 - DCL2	10	3	3	-8.6927	34.200	-3.4472
2 - DCL2	10	4	1	-8.7203	34.200	-3.3769
2 - DCL2	10	4	2	-6.9936	21.412	-5.0208
3 - DCL3	10	1	1	-44.426	68.114	-22.511
3 - DCL3	10	1	2	-10.158	-13.393	-77650
3 - DCL3	10	2	1	-164.95	100.69	-32.249
3 - DCL3	10	2	2	-130.69	-70.482	-16.657
3 - DCL3	10	3	1	-11.360	1.5329	2.6243
3 - DCL3	10	3	2	-10.456	4.9129	1.8083
3 - DCL3	10	3	3	-9.5515	7.0484	.99236
3 - DCL3	10	4	1	-9.5432	7.0484	1.0694
3 - DCL3	10	4	2	-7.8165	7.8022	-57445
4 - DCL4	10	1	1	-164.99	-96.944	31.532
4 - DCL4	10	1	2	-130.72	69.210	15.940
4 - DCL4	10	2	1	-45.366	-63.218	21.516
4 - DCL4	10	2	2	-11.098	11.327	-21806
4 - DCL4	10	3	1	-12.228	-18.505	4.8081
4 - DCL4	10	3	2	-11.323	-11.779	4.0127
4 - DCL4	10	3	3	-10.419	-6.2669	3.2172
4 - DCL4	10	4	1	-10.392	-6.2669	3.3013
4 - DCL4	10	4	2	-8.6660	1.3467	1.6985
5 - DCL5	10	1	1	-51.252	-144.97	31.530
5 - DCL5	10	1	2	-16.984	21.167	15.938
5 - DCL5	10	2	1	-159.10	-111.19	21.519
5 - DCL5	10	2	2	-124.83	-36.629	-21599
5 - DCL5	10	3	1	-13.086	-59.198	9.2476
5 - DCL5	10	3	2	-12.182	-45.701	8.4522
5 - DCL5	10	3	3	-11.277	-33.418	7.6568
5 - DCL5	10	4	1	-11.215	-33.418	7.7476
5 - DCL5	10	4	2	-9.4889	-12.263	6.1449
6 - DCL6	10	1	1	-138.45	-119.31	29.777
6 - DCL6	10	1	2	-104.19	89.129	29.777
6 - DCL6	10	2	1	-138.43	119.85	-29.777
6 - DCL6	10	2	2	-104.16	-88.587	-29.777
6 - DCL6	10	3	1	-87.777	-218.89	87.508
6 - DCL6	10	3	2	-84.712	-98.197	70.789
6 - DCL6	10	3	3	-81.647	-2.9911	54.070
6 - DCL6	10	4	1	-81.208	-2.9911	54.728
6 - DCL6	10	4	2	-75.356	112.76	21.290
7 - DCL7	10	1	1	-110.84	20.102	-6.8461
7 - DCL7	10	1	2	-76.574	48.251	14.888
7 - DCL7	10	2	1	-117.45	191.84	-46.951
7 - DCL7	10	2	2	-83.182	-82.237	-31.358
7 - DCL7	10	3	1	-67.644	-137.35	63.122

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
7 - DCL 7	10	3	2	-64.579	-50.706	50.521
7 - DCL 7	10	3	3	-61.513	16.729	37.919
7 - DCL 7	10	4	1	-61.205	16.729	38.414
7 - DCL 7	10	4	2	-55.354	95.328	13.200
8 - DCL 8	10	1	1	-110.84	20.102	-6.8461
8 - DCL 8	10	1	2	-76.574	48.251	14.888
8 - DCL 8	10	2	1	-117.45	191.84	-46.951
8 - DCL 8	10	2	2	-83.182	-82.237	-31.358
8 - DCL 8	10	3	1	-67.644	-137.35	63.122
8 - DCL 8	10	3	2	-64.579	-50.706	50.521
8 - DCL 8	10	3	3	-61.513	16.729	37.919
8 - DCL 8	10	4	1	-61.205	16.729	38.414
8 - DCL 8	10	4	2	-55.354	95.328	13.200
9 - DCL 9	10	1	1	-117.66	-192.98	47.195
9 - DCL 9	10	1	2	-83.400	82.812	31.603
9 - DCL 9	10	2	1	-111.59	-20.035	6.8175
9 - DCL 9	10	2	2	-77.328	-48.384	-14.917
9 - DCL 9	10	3	1	-69.370	-198.09	69.745
9 - DCL 9	10	3	2	-66.305	-101.32	57.164
9 - DCL 9	10	3	3	-63.240	-23.737	44.584
9 - DCL 9	10	4	1	-62.878	-23.737	45.093
9 - DCL 9	10	4	2	-57.026	75.263	19.919
10 - DCL 10	10	1	1	-117.66	-192.98	47.195
10 - DCL 10	10	1	2	-83.400	82.812	31.603
10 - DCL 10	10	2	1	-111.59	-20.035	6.8175
10 - DCL 10	10	2	2	-77.328	-48.384	-14.917
10 - DCL 10	10	3	1	-69.370	-198.09	69.745
10 - DCL 10	10	3	2	-66.305	-101.32	57.164
10 - DCL 10	10	3	3	-63.240	-23.737	44.584
10 - DCL 10	10	4	1	-62.878	-23.737	45.093
10 - DCL 10	10	4	2	-57.026	75.263	19.919
11 - DCL 11	10	1	1	-123.08	-229.00	52.491
11 - DCL 11	10	1	2	-88.817	83.862	36.898
11 - DCL 11	10	2	1	-230.92	-26.777	55820
11 - DCL 11	10	2	2	-196.65	-98.943	-21.176
11 - DCL 11	10	3	1	-74.935	-213.52	70.938
11 - DCL 11	10	3	2	-71.870	-114.93	58.357
11 - DCL 11	10	3	3	-68.805	-35.533	45.777
11 - DCL 11	10	4	1	-68.433	-35.533	46.331
11 - DCL 11	10	4	2	-62.582	67.237	21.157
12 - DCL 12	10	1	1	-229.99	32.112	-15486
12 - DCL 12	10	1	2	-195.72	97.344	20.186
12 - DCL 12	10	2	1	-123.03	233.07	-53.212
12 - DCL 12	10	2	2	-88.767	-84.837	-37.620
12 - DCL 12	10	3	1	-72.350	-112.10	59.875
12 - DCL 12	10	3	2	-69.285	-30.399	47.274
12 - DCL 12	10	3	3	-66.220	32.085	34.672
12 - DCL 12	10	4	1	-65.938	32.085	35.206
12 - DCL 12	10	4	2	-60.086	100.91	9.9915
13 - DCL 13	10	1	1	-51.252	-144.97	31.530
13 - DCL 13	10	1	2	-16.984	21.167	15.938
13 - DCL 13	10	2	1	-159.10	-111.19	21.519
13 - DCL 13	10	2	2	-124.83	-36.629	-21599
13 - DCL 13	10	3	1	-13.086	-59.198	9.2476
13 - DCL 13	10	3	2	-12.182	-45.701	8.4522

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
13 - DCL 13	10	3	3	-11.277	-33.418	7.6568
13 - DCL 13	10	4	1	-11.215	-33.418	7.7476
13 - DCL 13	10	4	2	-9.4889	-12.263	6.1449
14 - DCL 14	10	1	1	-236.82	-180.97	52.493
14 - DCL 14	10	1	2	-202.55	131.90	36.900
14 - DCL 14	10	2	1	-117.18	21.194	.55612
14 - DCL 14	10	2	2	-82.913	-50.985	-21.178
14 - DCL 14	10	3	1	-74.077	-172.83	66.499
14 - DCL 14	10	3	2	-71.012	-81.014	53.918
14 - DCL 14	10	3	3	-67.946	-8.3820	41.337
14 - DCL 14	10	4	1	-67.610	-8.3820	41.884
14 - DCL 14	10	4	2	-61.759	80.847	16.710
15 - DCL 15	10	1	1	-35.941	119.62	-31.876
15 - DCL 15	10	1	2	-1.6727	-18.985	-7.7263
15 - DCL 15	10	2	1	-43.304	115.43	-27.898
15 - DCL 15	10	2	2	-9.0362	-19.218	-10.573
15 - DCL 15	10	3	1	-3.5578	26.027	-1.2782
15 - DCL 15	10	3	2	-2.6534	23.804	-1.6367
15 - DCL 15	10	3	3	-1.7489	21.035	-1.9952
15 - DCL 15	10	4	1	-1.7649	21.035	-1.9810
15 - DCL 15	10	4	2	-.03826	13.890	-2.7112
16 - DCL 16	10	1	1	-39.009	104.13	-27.807
16 - DCL 16	10	1	2	-4.7412	-14.443	-6.0719
16 - DCL 16	10	2	1	-45.636	107.43	-25.990
16 - DCL 16	10	2	2	-11.367	-19.924	-10.398
16 - DCL 16	10	3	1	-5.7949	16.967	1.4313
16 - DCL 16	10	3	2	-4.8904	18.528	.61536
16 - DCL 16	10	3	3	-3.9859	18.844	-.20063
16 - DCL 16	10	4	1	-3.9874	18.844	-.16845
16 - DCL 16	10	4	2	-2.2607	15.828	-1.8123
17 - DCL 17	10	1	1	-45.835	-108.95	26.235
17 - DCL 17	10	1	2	-11.567	20.117	10.642
17 - DCL 17	10	2	1	-39.781	-104.44	27.778
17 - DCL 17	10	2	2	-5.5136	13.928	6.0433
17 - DCL 17	10	3	1	-7.5211	-43.763	8.0546
17 - DCL 17	10	3	2	-6.6167	-32.086	7.2592
17 - DCL 17	10	3	3	-5.7122	-21.622	6.4638
17 - DCL 17	10	4	1	-5.6598	-21.622	6.5097
17 - DCL 17	10	4	2	-3.9331	-4.2372	4.9070
18 - DCL 18	10	1	1	-45.835	-108.95	26.235
18 - DCL 18	10	1	2	-11.567	20.117	10.642
18 - DCL 18	10	2	1	-39.781	-104.44	27.778
18 - DCL 18	10	2	2	-5.5136	13.928	6.0433
18 - DCL 18	10	3	1	-7.5211	-43.763	8.0546
18 - DCL 18	10	3	2	-6.6167	-32.086	7.2592
18 - DCL 18	10	3	3	-5.7122	-21.622	6.4638
18 - DCL 18	10	4	1	-5.6598	-21.622	6.5097
18 - DCL 18	10	4	2	-3.9331	-4.2372	4.9070
19 - DCL 19	10	1	1	-138.45	-119.31	29.777
19 - DCL 19	10	1	2	-104.19	89.129	29.777
19 - DCL 19	10	2	1	-138.43	119.85	-29.777
19 - DCL 19	10	2	2	-104.16	-88.587	-29.777
19 - DCL 19	10	3	1	-87.777	-218.89	87.508
19 - DCL 19	10	3	2	-84.712	-98.197	70.789
19 - DCL 19	10	3	3	-81.647	-2.9911	54.070

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
19 - DCL19	10	4	1	-81.208	-2.9911	54.728
19 - DCL19	10	4	2	-75.356	112.76	21.290
20 - DCL20	10	1	1	-139.51	-18.244	10.706
20 - DCL20	10	1	2	-113.15	56.699	10.706
20 - DCL20	10	2	1	-55.382	57.804	-11.420
20 - DCL20	10	2	2	-29.021	-22.137	-11.420
20 - DCL20	10	3	1	-23.431	-30.961	17.454
20 - DCL20	10	3	2	-22.735	-7.2366	13.659
20 - DCL20	10	3	3	-22.039	10.700	9.8647
20 - DCL20	10	4	1	-21.959	10.700	10.042
20 - DCL20	10	4	2	-20.631	29.727	2.4523
21 - DCL21	10	1	1	-112.27	73.809	-14.102
21 - DCL21	10	1	2	-85.916	25.810	.38800
21 - DCL21	10	2	1	-40.979	102.74	-22.405
21 - DCL21	10	2	2	-14.619	-17.711	-12.010
21 - DCL21	10	3	1	-9.6604	21.527	1.4377
21 - DCL21	10	3	2	-8.9647	22.919	.38779
21 - DCL21	10	3	3	-8.2689	22.710	-.66220
21 - DCL21	10	4	1	-8.2740	22.710	-.59541
21 - DCL21	10	4	2	-6.9458	17.686	-2.7033
22 - DCL22	10	1	1	-36.451	41.791	-14.103
22 - DCL22	10	1	2	-10.091	-6.2175	.38662
22 - DCL22	10	2	1	-116.80	70.763	-22.404
22 - DCL22	10	2	2	-90.445	-49.683	-12.009
22 - DCL22	10	3	1	-10.232	-5.6008	4.3975
22 - DCL22	10	3	2	-9.5372	.30479	3.3475
22 - DCL22	10	3	3	-8.8414	4.6091	2.2975
22 - DCL22	10	4	1	-8.8226	4.6091	2.3688
22 - DCL22	10	4	2	-7.4944	8.6136	.26089
23 - DCL23	10	1	1	-116.82	-68.248	21.926
23 - DCL23	10	1	2	-90.467	48.851	11.531
23 - DCL23	10	2	1	-37.077	-38.510	13.440
23 - DCL23	10	2	2	-10.716	4.8568	-1.0496
23 - DCL23	10	3	1	-10.811	-18.959	5.8533
23 - DCL23	10	3	2	-10.115	-10.823	4.8170
23 - DCL23	10	3	3	-9.4198	-4.2678	3.7808
23 - DCL23	10	4	1	-9.3889	-4.2678	3.8567
23 - DCL23	10	4	2	-8.0607	4.3100	1.7762
24 - DCL24	10	1	1	-41.001	-100.26	21.924
24 - DCL24	10	1	2	-14.641	16.822	11.529
24 - DCL24	10	2	1	-112.90	-70.491	13.441
24 - DCL24	10	2	2	-86.542	-27.114	-1.0482
24 - DCL24	10	3	1	-11.383	-46.088	8.8130
24 - DCL24	10	3	2	-10.688	-33.438	7.7768
24 - DCL24	10	3	3	-9.9923	-22.368	6.7405
24 - DCL24	10	4	1	-9.9375	-22.368	6.8209
24 - DCL24	10	4	2	-8.6093	-4.7631	4.7404
25 - DCL25	10	1	1	-178.57	-75.154	24.287
25 - DCL25	10	1	2	-152.21	94.859	24.287
25 - DCL25	10	2	1	-102.84	111.02	-24.930
25 - DCL25	10	2	2	-76.485	-63.487	-24.930
25 - DCL25	10	3	1	-64.315	-135.71	58.822
25 - DCL25	10	3	2	-62.179	-54.897	47.170
25 - DCL25	10	3	3	-60.043	8.1532	35.518
25 - DCL25	10	4	1	-59.754	8.1532	36.002

Table 5 Forces from DCL [EUROCODE 1] (Q5). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
25 - DCL25	10	4	2	-55.676	82.314	12.698
26 - DCL26	10	1	1	-80.728	9.7826	-3.6598
26 - DCL26	10	1	2	-54.368	34.878	10.830
26 - DCL26	10	2	1	-85.133	131.53	-32.205
26 - DCL26	10	2	2	-58.773	-57.519	-21.810
26 - DCL26	10	3	1	-47.755	-98.196	44.729
26 - DCL26	10	3	2	-45.619	-36.774	35.822
26 - DCL26	10	3	3	-43.483	11.063	26.915
26 - DCL26	10	4	1	-43.264	11.063	27.265
26 - DCL26	10	4	2	-39.186	66.964	9.4439
27 - DCL27	10	1	1	-80.728	9.7826	-3.6598
27 - DCL27	10	1	2	-54.368	34.878	10.830
27 - DCL27	10	2	1	-85.133	131.53	-32.205
27 - DCL27	10	2	2	-58.773	-57.519	-21.810
27 - DCL27	10	3	1	-47.755	-98.196	44.729
27 - DCL27	10	3	2	-45.619	-36.774	35.822
27 - DCL27	10	3	3	-43.483	11.063	26.915
27 - DCL27	10	4	1	-43.264	11.063	27.265
27 - DCL27	10	4	2	-39.186	66.964	9.4439
28 - DCL28	10	1	1	-85.279	-132.27	32.368
28 - DCL28	10	1	2	-58.919	57.919	21.973
28 - DCL28	10	2	1	-81.230	-9.7216	3.6407
28 - DCL28	10	2	2	-54.870	-34.951	-10.849
28 - DCL28	10	3	1	-48.906	-138.68	49.145
28 - DCL28	10	3	2	-46.770	-70.518	40.251
28 - DCL28	10	3	3	-44.633	-15.915	31.358
28 - DCL28	10	4	1	-44.379	-15.915	31.718
28 - DCL28	10	4	2	-40.301	53.587	13.923
29 - DCL29	10	1	1	-85.279	-132.27	32.368
29 - DCL29	10	1	2	-58.919	57.919	21.973
29 - DCL29	10	2	1	-81.230	-9.7216	3.6407
29 - DCL29	10	2	2	-54.870	-34.951	-10.849
29 - DCL29	10	3	1	-48.906	-138.68	49.145
29 - DCL29	10	3	2	-46.770	-70.518	40.251
29 - DCL29	10	3	3	-44.633	-15.915	31.358
29 - DCL29	10	4	1	-44.379	-15.915	31.718
29 - DCL29	10	4	2	-40.301	53.587	13.923
30 - DCL30	10	1	1	-160.16	17.789	-12812
30 - DCL30	10	1	2	-133.80	67.607	14.361
30 - DCL30	10	2	1	-88.856	159.02	-36.379
30 - DCL30	10	2	2	-62.496	-59.253	-25.984
30 - DCL30	10	3	1	-50.893	-81.357	42.565
30 - DCL30	10	3	2	-48.757	-23.237	33.658
30 - DCL30	10	3	3	-46.620	21.300	24.751
30 - DCL30	10	4	1	-46.419	21.300	25.126
30 - DCL30	10	4	2	-42.341	70.687	7.3049
31 - DCL31	10	1	1	-84.339	-14.229	-12950
31 - DCL31	10	1	2	-57.979	35.579	14.360
31 - DCL31	10	2	1	-164.68	127.03	-36.378
31 - DCL31	10	2	2	-138.32	-91.225	-25.983
31 - DCL31	10	3	1	-51.465	-108.48	45.524
31 - DCL31	10	3	2	-49.329	-45.851	36.617
31 - DCL31	10	3	3	-47.193	3.1990	27.710
31 - DCL31	10	4	1	-46.968	3.1990	28.091
31 - DCL31	10	4	2	-42.889	61.613	10.269

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
32 - DCL32	10	1	1	-164.71	-124.26	35.899
32 - DCL32	10	1	2	-138.35	90.648	25.504
32 - DCL32	10	2	1	-84.953	17.764	-.53354
32 - DCL32	10	2	2	-58.593	-36.685	-15.023
32 - DCL32	10	3	1	-52.044	-121.84	46.980
32 - DCL32	10	3	2	-49.907	-56.980	38.087
32 - DCL32	10	3	3	-47.771	-5.6778	29.194
32 - DCL32	10	4	1	-47.534	-5.6778	29.579
32 - DCL32	10	4	2	-43.456	57.310	11.784
33 - DCL33	10	1	1	-88.890	-156.28	35.898
33 - DCL33	10	1	2	-62.530	58.619	25.503
33 - DCL33	10	2	1	-160.77	-14.216	-.53216
33 - DCL33	10	2	2	-134.41	-68.656	-15.022
33 - DCL33	10	3	1	-52.616	-148.97	49.940
33 - DCL33	10	3	2	-50.480	-79.594	41.047
33 - DCL33	10	3	3	-48.344	-23.778	32.153
33 - DCL33	10	4	1	-48.083	-23.778	32.543
33 - DCL33	10	4	2	-44.004	48.237	14.748
34 - DCL34	10	1	1	-30.794	76.130	-20.346
34 - DCL34	10	1	2	-4.4339	-9.9458	-4.2465
34 - DCL34	10	2	1	-35.702	80.591	-19.503
34 - DCL34	10	2	2	-9.3422	-15.507	-7.9534
34 - DCL34	10	3	1	-5.0312	10.728	1.7958
34 - DCL34	10	3	2	-4.3354	12.899	1.0508
34 - DCL34	10	3	3	-3.6397	13.933	.30580
34 - DCL34	10	4	1	-3.6371	13.933	.33518
34 - DCL34	10	4	2	-2.3089	12.672	-1.1636
35 - DCL35	10	1	1	-30.794	76.130	-20.346
35 - DCL35	10	1	2	-4.4339	-9.9458	-4.2465
35 - DCL35	10	2	1	-35.702	80.591	-19.503
35 - DCL35	10	2	2	-9.3422	-15.507	-7.9534
35 - DCL35	10	3	1	-5.0312	10.728	1.7958
35 - DCL35	10	3	2	-4.3354	12.899	1.0508
35 - DCL35	10	3	3	-3.6397	13.933	.30580
35 - DCL35	10	4	1	-3.6371	13.933	.33518
35 - DCL35	10	4	2	-2.3089	12.672	-1.1636
36 - DCL36	10	1	1	-35.850	-81.712	19.684
36 - DCL36	10	1	2	-9.4902	15.654	8.1346
36 - DCL36	10	2	1	-31.366	-76.359	20.325
36 - DCL36	10	2	2	-5.0058	9.5687	4.2253
36 - DCL36	10	3	1	-6.3099	-34.257	6.7019
36 - DCL36	10	3	2	-5.6142	-24.592	5.9722
36 - DCL36	10	3	3	-4.9184	-16.041	5.2424
36 - DCL36	10	4	1	-4.8759	-16.041	5.2820
36 - DCL36	10	4	2	-3.5477	-2.1908	3.8136
37 - DCL37	10	1	1	-35.850	-81.712	19.684
37 - DCL37	10	1	2	-9.4902	15.654	8.1346
37 - DCL37	10	2	1	-31.366	-76.359	20.325
37 - DCL37	10	2	2	-5.0058	9.5687	4.2253
37 - DCL37	10	3	1	-6.3099	-34.257	6.7019
37 - DCL37	10	3	2	-5.6142	-24.592	5.9722
37 - DCL37	10	3	3	-4.9184	-16.041	5.2424
37 - DCL37	10	4	1	-4.8759	-16.041	5.2820
37 - DCL37	10	4	2	-3.5477	-2.1908	3.8136
38 - DCL38	10	1	1	-104.46	-89.385	22.308

Table 5 Forces from DCL [EUROCODE 1] (Q5). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
38 - DCL38	10	1	2	-78.100	66.775	22.308
38 - DCL38	10	2	1	-104.44	89.792	-22.308
38 - DCL38	10	2	2	-78.081	-66.369	-22.308
38 - DCL38	10	3	1	-65.759	-163.98	65.556
38 - DCL38	10	3	2	-63.462	-73.563	53.031
38 - DCL38	10	3	3	-61.166	-2.2406	40.506
38 - DCL38	10	4	1	-60.837	-2.2406	40.999
38 - DCL38	10	4	2	-56.453	84.480	15.949
1 - DCL1	10	4	3	-26.092	33.106	-2.3514
1 - DCL1	10	5	1	-25.393	33.106	6.4438
1 - DCL1	10	5	2	-26.626	39.365	-95639
1 - DCL1	10	5	3	-27.859	28.743	-8.3566
1 - DCL1	10	6	1	-28.033	28.743	-7.7546
1 - DCL1	10	6	2	-29.431	2.5147	-15.154
1 - DCL1	10	6	3	-30.829	-40.659	-22.555
1 - DCL1	10	7	1	.48195	-2.7742	4.1579
1 - DCL1	10	7	2	.48195	-1.3010	3.6989
2 - DCL2	10	4	3	-5.2668	3.6173	-6.6647
2 - DCL2	10	5	1	-7.1807	3.6173	-4.5383
2 - DCL2	10	5	2	-8.4140	-8.1506	-5.7797
2 - DCL2	10	5	3	-9.6474	-22.750	-7.0212
2 - DCL2	10	6	1	-9.7964	-22.750	-6.8117
2 - DCL2	10	6	2	-11.194	-39.742	-8.0296
2 - DCL2	10	6	3	-12.592	-59.522	-9.2474
2 - DCL2	10	7	1	.48195	-91.469	122.41
2 - DCL2	10	7	2	.48195	-45.648	121.95
3 - DCL3	10	4	3	-6.0897	3.5493	-2.2183
3 - DCL3	10	5	1	-6.4808	3.5493	-0.7091
3 - DCL3	10	5	2	-7.7142	1.9717	-1.3123
3 - DCL3	10	5	3	-8.9476	-2.4377	-2.5538
3 - DCL3	10	6	1	-9.0005	-2.4377	-2.3605
3 - DCL3	10	6	2	-10.398	-9.2370	-3.5783
3 - DCL3	10	6	3	-11.796	-18.824	-4.7961
3 - DCL3	10	7	1	.48195	-2.7742	4.1579
3 - DCL3	10	7	2	.48195	-1.3010	3.6989
4 - DCL4	10	4	3	-6.9392	4.0791	.09575
4 - DCL4	10	5	1	-6.5139	4.0791	2.3939
4 - DCL4	10	5	2	-7.7473	7.8078	.87539
4 - DCL4	10	5	3	-8.9807	8.0727	-.64319
4 - DCL4	10	6	1	-8.9924	8.0727	-.44960
4 - DCL4	10	6	2	-10.390	5.3304	-1.9456
4 - DCL4	10	6	3	-11.788	-.83744	-3.4416
4 - DCL4	10	7	1	.48195	-91.469	122.41
4 - DCL4	10	7	2	.48195	-45.648	121.95
5 - DCL5	10	4	3	-7.7621	4.0111	4.5421
5 - DCL5	10	5	1	-5.8141	4.0111	6.8613
5 - DCL5	10	5	2	-7.0474	17.930	5.3427
5 - DCL5	10	5	3	-8.2808	28.385	3.8241
5 - DCL5	10	6	1	-8.1965	28.385	4.0016
5 - DCL5	10	6	2	-9.5943	35.835	2.5056
5 - DCL5	10	6	3	-10.992	39.860	1.0096
5 - DCL5	10	7	1	.48195	-2.7742	4.1579
5 - DCL5	10	7	2	.48195	-1.3010	3.6989
6 - DCL6	10	4	3	-69.504	126.69	-12.147
6 - DCL6	10	5	1	-69.595	126.69	11.614

Table 5 Forces from DCL [EUROCODE 1] (Q5). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
6 - DCL6	10	5	2	-73.775	124.58	-13.463
6 - DCL6	10	5	3	-77.955	65.268	-38.542
6 - DCL6	10	6	1	-78.767	65.268	-36.854
6 - DCL6	10	6	2	-83.504	-47.832	-61.932
6 - DCL6	10	6	3	-88.241	-218.35	-87.011
6 - DCL6	10	7	1	0	-34.421	.91790
6 - DCL6	10	7	2	0	-.08605	.45895
7 - DCL7	10	4	3	-49.502	97.133	-12.014
7 - DCL7	10	5	1	-50.683	97.133	5.0999
7 - DCL7	10	5	2	-54.863	87.188	-13.819
7 - DCL7	10	5	3	-59.043	34.086	-32.739
7 - DCL7	10	6	1	-59.734	34.086	-31.460
7 - DCL7	10	6	2	-64.471	-59.584	-50.356
7 - DCL7	10	6	3	-69.208	-196.52	-69.252
7 - DCL7	10	7	1	0	-34.421	.91790
7 - DCL7	10	7	2	0	-.08605	.45895
8 - DCL8	10	4	3	-49.502	97.133	-12.014
8 - DCL8	10	5	1	-50.683	97.133	5.0999
8 - DCL8	10	5	2	-54.863	87.188	-13.819
8 - DCL8	10	5	3	-59.043	34.086	-32.739
8 - DCL8	10	6	1	-59.734	34.086	-31.460
8 - DCL8	10	6	2	-64.471	-59.584	-50.356
8 - DCL8	10	6	3	-69.208	-196.52	-69.252
8 - DCL8	10	7	1	0	-34.421	.91790
8 - DCL8	10	7	2	0	-.08605	.45895
9 - DCL9	10	4	3	-51.174	97.595	-5.2543
9 - DCL9	10	5	1	-50.016	97.595	12.032
9 - DCL9	10	5	2	-54.196	103.14	-7.1645
9 - DCL9	10	5	3	-58.376	64.909	-26.361
9 - DCL9	10	6	1	-58.930	64.909	-25.097
9 - DCL9	10	6	2	-63.667	-14.511	-44.272
9 - DCL9	10	6	3	-68.404	-137.83	-63.446
9 - DCL9	10	7	1	0	-34.421	.91790
9 - DCL9	10	7	2	0	-.08605	.45895
10 - DCL10	10	4	3	-51.174	97.595	-5.2543
10 - DCL10	10	5	1	-50.016	97.595	12.032
10 - DCL10	10	5	2	-54.196	103.14	-7.1645
10 - DCL10	10	5	3	-58.376	64.909	-26.361
10 - DCL10	10	6	1	-58.930	64.909	-25.097
10 - DCL10	10	6	2	-63.667	-14.511	-44.272
10 - DCL10	10	6	3	-68.404	-137.83	-63.446
10 - DCL10	10	7	1	0	-34.421	.91790
10 - DCL10	10	7	2	0	-.08605	.45895
11 - DCL11	10	4	3	-56.730	93.339	-4.0164
11 - DCL11	10	5	1	-54.846	93.339	15.044
11 - DCL11	10	5	2	-59.026	105.76	-4.1525
11 - DCL11	10	5	3	-63.206	74.395	-23.349
11 - DCL11	10	6	1	-63.694	74.395	-21.982
11 - DCL11	10	6	2	-68.431	2.1075	-41.156
11 - DCL11	10	6	3	-73.168	-114.08	-60.331
11 - DCL11	10	7	1	.48195	-2.7742	4.1579
11 - DCL11	10	7	2	.48195	-1.3010	3.6989
12 - DCL12	10	4	3	-54.235	92.945	-15.223
12 - DCL12	10	5	1	-56.213	92.945	3.6446
12 - DCL12	10	5	2	-60.392	79.680	-15.275

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
12 - DCL 12	10	5	3	-64.572	23.259	-34.194
12 - DCL 12	10	6	1	-65.294	23.259	-32.796
12 - DCL 12	10	6	2	-70.031	-73.470	-51.692
12 - DCL 12	10	6	3	-74.768	-213.46	-70.588
12 - DCL 12	10	7	1	.48195	-91.469	122.41
12 - DCL 12	10	7	2	.48195	-45.648	121.95
13 - DCL 13	10	4	3	-7.7621	4.0111	4.5421
13 - DCL 13	10	5	1	-5.8141	4.0111	6.8613
13 - DCL 13	10	5	2	-7.0474	17.930	5.3427
13 - DCL 13	10	5	3	-8.2808	28.385	3.8241
13 - DCL 13	10	6	1	-8.1965	28.385	4.0016
13 - DCL 13	10	6	2	-9.5943	35.835	2.5056
13 - DCL 13	10	6	3	-10.992	39.860	1.0096
13 - DCL 13	10	7	1	.48195	-2.7742	4.1579
13 - DCL 13	10	7	2	.48195	-1.3010	3.6989
14 - DCL 14	10	4	3	-55.907	93.407	-8.4628
14 - DCL 14	10	5	1	-55.546	93.407	10.576
14 - DCL 14	10	5	2	-59.726	95.639	-8.6199
14 - DCL 14	10	5	3	-63.905	54.082	-27.816
14 - DCL 14	10	6	1	-64.490	54.082	-26.433
14 - DCL 14	10	6	2	-69.227	-28.397	-45.608
14 - DCL 14	10	6	3	-73.964	-154.78	-64.782
14 - DCL 14	10	7	1	.48195	-91.469	122.41
14 - DCL 14	10	7	2	.48195	-45.648	121.95
15 - DCL 15	10	4	3	1.6884	4.5208	-3.4414
15 - DCL 15	10	5	1	.45022	4.5208	-3.8068
15 - DCL 15	10	5	2	-.78314	-4.7981	-4.3639
15 - DCL 15	10	5	3	-2.0165	-15.387	-4.9211
15 - DCL 15	10	6	1	-2.1220	-15.387	-4.8765
15 - DCL 15	10	6	2	-3.5198	-27.162	-5.4074
15 - DCL 15	10	6	3	-4.9176	-40.151	-5.9383
15 - DCL 15	10	7	1	0	-.34421	.91790
15 - DCL 15	10	7	2	0	-.08605	.45895
16 - DCL 16	10	4	3	-.53403	7.8049	-3.4562
16 - DCL 16	10	5	1	-1.6511	7.8049	-3.0829
16 - DCL 16	10	5	2	-2.8845	-64.329	-4.3244
16 - DCL 16	10	5	3	-4.1178	-11.923	-5.5658
16 - DCL 16	10	6	1	-4.2368	-11.923	-5.4758
16 - DCL 16	10	6	2	-5.6346	-25.856	-6.6937
16 - DCL 16	10	6	3	-7.0324	-42.577	-7.9115
16 - DCL 16	10	7	1	0	-.34421	.91790
16 - DCL 16	10	7	2	0	-.08605	.45895
17 - DCL 17	10	4	3	-2.2064	8.2667	3.3042
17 - DCL 17	10	5	1	-.98438	8.2667	3.8493
17 - DCL 17	10	5	2	-2.2177	15.315	2.3307
17 - DCL 17	10	5	3	-3.4511	18.899	.81214
17 - DCL 17	10	6	1	-3.4328	18.899	.88628
17 - DCL 17	10	6	2	-4.8306	19.216	-.60973
17 - DCL 17	10	6	3	-6.2284	16.107	-2.1057
17 - DCL 17	10	7	1	0	-.34421	.91790
17 - DCL 17	10	7	2	0	-.08605	.45895
18 - DCL 18	10	4	3	-2.2064	8.2667	3.3042
18 - DCL 18	10	5	1	-.98438	8.2667	3.8493
18 - DCL 18	10	5	2	-2.2177	15.315	2.3307
18 - DCL 18	10	5	3	-3.4511	18.899	.81214

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
18 - DCL 18	10	6	1	-3.4328	18.899	.88628
18 - DCL 18	10	6	2	-4.8306	19.216	-.60973
18 - DCL 18	10	6	3	-6.2284	16.107	-2.1057
18 - DCL 18	10	7	1	0	-.34421	.91790
18 - DCL 18	10	7	2	0	-.08605	.45895
19 - DCL 19	10	4	3	-69.504	126.69	-12.147
19 - DCL 19	10	5	1	-69.595	126.69	11.614
19 - DCL 19	10	5	2	-73.775	124.58	-13.463
19 - DCL 19	10	5	3	-77.955	65.268	-38.542
19 - DCL 19	10	6	1	-78.767	65.268	-36.854
19 - DCL 19	10	6	2	-83.504	-47.832	-61.932
19 - DCL 19	10	6	3	-88.241	-218.35	-87.011
19 - DCL 19	10	7	1	0	-.34421	.91790
19 - DCL 19	10	7	2	0	-.08605	.45895
20 - DCL 20	10	4	3	-19.303	25.638	-5.1376
20 - DCL 20	10	5	1	-19.913	25.638	1.5618
20 - DCL 20	10	5	2	-20.862	22.708	-4.1306
20 - DCL 20	10	5	3	-21.811	6.7939	-9.8231
20 - DCL 20	10	6	1	-22.017	6.7939	-9.3511
20 - DCL 20	10	6	2	-23.093	-21.135	-15.043
20 - DCL 20	10	6	3	-24.168	-62.099	-20.736
20 - DCL 20	10	7	1	.35700	-67.764	90.706
20 - DCL 20	10	7	2	.35700	-33.816	90.353
21 - DCL 21	10	4	3	-5.6175	6.2435	-4.8112
21 - DCL 21	10	5	1	-6.8962	6.2435	-2.6735
21 - DCL 21	10	5	2	-7.8449	-1.6644	-4.2601
21 - DCL 21	10	5	3	-8.7937	-13.191	-5.8468
21 - DCL 21	10	6	1	-8.9176	-13.191	-5.6560
21 - DCL 21	10	6	2	-9.9928	-27.941	-7.2269
21 - DCL 21	10	6	3	-11.068	-46.288	-8.7978
21 - DCL 21	10	7	1	.32130	-61.014	81.706
21 - DCL 21	10	7	2	.32130	-30.441	81.353
22 - DCL 22	10	4	3	-6.1661	6.1982	-1.8470
22 - DCL 22	10	5	1	-6.4296	6.1982	.30470
22 - DCL 22	10	5	2	-7.3783	5.0837	-1.2819
22 - DCL 22	10	5	3	-8.3271	.35001	-2.8685
22 - DCL 22	10	6	1	-8.3869	.35001	-2.6885
22 - DCL 22	10	6	2	-9.4622	-7.6046	-4.2594
22 - DCL 22	10	6	3	-10.537	-19.156	-5.8303
22 - DCL 22	10	7	1	.32130	-1.8847	2.8660
22 - DCL 22	10	7	2	.32130	-.87619	2.5130
23 - DCL 23	10	4	3	-6.7325	6.5514	-.30430
23 - DCL 23	10	5	1	-6.4517	6.5514	1.9479
23 - DCL 23	10	5	2	-7.4004	8.9745	.17657
23 - DCL 23	10	5	3	-8.3492	7.3569	-1.5948
23 - DCL 23	10	6	1	-8.3816	7.3569	-1.4146
23 - DCL 23	10	6	2	-9.4568	2.1070	-3.1709
23 - DCL 23	10	6	3	-10.532	-7.1646	-4.9273
23 - DCL 23	10	7	1	.32130	-61.014	81.706
23 - DCL 23	10	7	2	.32130	-30.441	81.353
24 - DCL 24	10	4	3	-7.2810	6.5061	2.6599
24 - DCL 24	10	5	1	-5.9851	6.5061	4.9262
24 - DCL 24	10	5	2	-6.9338	15.722	3.1548
24 - DCL 24	10	5	3	-7.8826	20.898	1.3834
24 - DCL 24	10	6	1	-7.8510	20.898	1.5529

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
24 - DCL24	10	6	2	-8.9262	22.443	-20343
24 - DCL24	10	6	3	-10.001	19.967	-19597
24 - DCL24	10	7	1	.32130	-1.8847	2.8660
24 - DCL24	10	7	2	.32130	-87619	2.5130
25 - DCL25	10	4	3	-51.598	85.500	-10.605
25 - DCL25	10	5	1	-52.192	85.500	7.1249
25 - DCL25	10	5	2	-55.105	81.819	-10.353
25 - DCL25	10	5	3	-58.018	38.269	-27.831
25 - DCL25	10	6	1	-58.604	38.269	-26.574
25 - DCL25	10	6	2	-61.906	-42.592	-44.052
25 - DCL25	10	6	3	-65.207	-163.47	-61.530
25 - DCL25	10	7	1	.32130	-61.014	81.706
25 - DCL25	10	7	2	.32130	-30.441	81.353
26 - DCL26	10	4	3	-35.107	68.587	-8.3780
26 - DCL26	10	5	1	-35.898	68.587	3.7519
26 - DCL26	10	5	2	-38.811	61.894	-9.6201
26 - DCL26	10	5	3	-41.724	24.699	-22.992
26 - DCL26	10	6	1	-42.209	24.699	-22.088
26 - DCL26	10	6	2	-45.511	-41.169	-35.444
26 - DCL26	10	6	3	-48.812	-137.62	-48.801
26 - DCL26	10	7	1	0	-264.78	.70607
26 - DCL26	10	7	2	0	-.06619	.35303
27 - DCL27	10	4	3	-35.107	68.587	-8.3780
27 - DCL27	10	5	1	-35.898	68.587	3.7519
27 - DCL27	10	5	2	-38.811	61.894	-9.6201
27 - DCL27	10	5	3	-41.724	24.699	-22.992
27 - DCL27	10	6	1	-42.209	24.699	-22.088
27 - DCL27	10	6	2	-45.511	-41.169	-35.444
27 - DCL27	10	6	3	-48.812	-137.62	-48.801
27 - DCL27	10	7	1	0	-264.78	.70607
27 - DCL27	10	7	2	0	-.06619	.35303
28 - DCL28	10	4	3	-36.222	68.895	-3.8710
28 - DCL28	10	5	1	-35.453	68.895	8.3734
28 - DCL28	10	5	2	-38.366	72.533	-5.1834
28 - DCL28	10	5	3	-41.279	45.248	-18.740
28 - DCL28	10	6	1	-41.673	45.248	-17.846
28 - DCL28	10	6	2	-44.975	-11.121	-31.388
28 - DCL28	10	6	3	-48.276	-98.498	-44.930
28 - DCL28	10	7	1	0	-264.78	.70607
28 - DCL28	10	7	2	0	-.06619	.35303
29 - DCL29	10	4	3	-36.222	68.895	-3.8710
29 - DCL29	10	5	1	-35.453	68.895	8.3734
29 - DCL29	10	5	2	-38.366	72.533	-5.1834
29 - DCL29	10	5	3	-41.279	45.248	-18.740
29 - DCL29	10	6	1	-41.673	45.248	-17.846
29 - DCL29	10	6	2	-44.975	-11.121	-31.388
29 - DCL29	10	6	3	-48.276	-98.498	-44.930
29 - DCL29	10	7	1	0	-264.78	.70607
29 - DCL29	10	7	2	0	-.06619	.35303
30 - DCL30	10	4	3	-38.263	65.795	-10.517
30 - DCL30	10	5	1	-39.584	65.795	2.7817
30 - DCL30	10	5	2	-42.497	56.889	-10.590
30 - DCL30	10	5	3	-45.410	17.481	-23.962
30 - DCL30	10	6	1	-45.916	17.481	-22.978
30 - DCL30	10	6	2	-49.217	-50.426	-36.335

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
30 - DCL30	10	6	3	-52.518	-148.91	-49.691
30 - DCL30	10	7	1	.32130	-61.014	81.706
30 - DCL30	10	7	2	.32130	-30.441	81.353
31 - DCL31	10	4	3	-38.811	65.750	-7.5527
31 - DCL31	10	5	1	-39.117	65.750	5.7600
31 - DCL31	10	5	2	-42.030	63.638	-7.6121
31 - DCL31	10	5	3	-44.943	31.023	-20.984
31 - DCL31	10	6	1	-45.385	31.023	-20.011
31 - DCL31	10	6	2	-48.686	-30.089	-33.367
31 - DCL31	10	6	3	-51.988	-121.78	-46.724
31 - DCL31	10	7	1	.32130	-1.8847	2.8660
31 - DCL31	10	7	2	.32130	-87619	2.5130
32 - DCL32	10	4	3	-39.378	66.103	-6.0100
32 - DCL32	10	5	1	-39.140	66.103	7.4032
32 - DCL32	10	5	2	-42.053	67.528	-6.1536
32 - DCL32	10	5	3	-44.966	38.030	-19.710
32 - DCL32	10	6	1	-45.380	38.030	-18.737
32 - DCL32	10	6	2	-48.681	-20.378	-32.279
32 - DCL32	10	6	3	-51.982	-109.79	-45.821
32 - DCL32	10	7	1	.32130	-61.014	81.706
32 - DCL32	10	7	2	.32130	-30.441	81.353
33 - DCL33	10	4	3	-39.926	66.058	-3.0457
33 - DCL33	10	5	1	-38.673	66.058	10.381
33 - DCL33	10	5	2	-41.586	74.277	-3.1753
33 - DCL33	10	5	3	-44.499	51.572	-16.732
33 - DCL33	10	6	1	-44.849	51.572	-15.769
33 - DCL33	10	6	2	-48.150	-04.156	-29.311
33 - DCL33	10	6	3	-51.452	-82.663	-42.853
33 - DCL33	10	7	1	.32130	-1.8847	2.8660
33 - DCL33	10	7	2	.32130	-87619	2.5130
34 - DCL34	10	4	3	-.98066	6.8459	-2.6624
34 - DCL34	10	5	1	-1.8089	6.8459	-2.1859
34 - DCL34	10	5	2	-2.7576	.57050	-3.3163
34 - DCL34	10	5	3	-3.7064	-8.2834	-4.4467
34 - DCL34	10	6	1	-3.8013	-8.2834	-4.3659
34 - DCL34	10	6	2	-4.8765	-19.554	-5.4788
34 - DCL34	10	6	3	-5.9518	-33.374	-6.5917
34 - DCL34	10	7	1	0	-.26478	.70607
34 - DCL34	10	7	2	0	-.06619	.35303
35 - DCL35	10	4	3	-.98066	6.8459	-2.6624
35 - DCL35	10	5	1	-1.8089	6.8459	-2.1859
35 - DCL35	10	5	2	-2.7576	.57050	-3.3163
35 - DCL35	10	5	3	-3.7064	-8.2834	-4.4467
35 - DCL35	10	6	1	-3.8013	-8.2834	-4.3659
35 - DCL35	10	6	2	-4.8765	-19.554	-5.4788
35 - DCL35	10	6	3	-5.9518	-33.374	-6.5917
35 - DCL35	10	7	1	0	-.26478	.70607
35 - DCL35	10	7	2	0	-.06619	.35303
36 - DCL36	10	4	3	-2.2194	7.1879	2.3453
36 - DCL36	10	5	1	-1.3150	7.1879	2.9491
36 - DCL36	10	5	2	-2.2637	12.391	1.6134
36 - DCL36	10	5	3	-3.2125	14.548	.27769
36 - DCL36	10	6	1	-3.2057	14.548	.34682
36 - DCL36	10	6	2	-4.2810	13.832	-.97217
36 - DCL36	10	6	3	-5.3562	10.096	-2.2911

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
36 - DCL36	10	7	1	0	-264.78	70607
36 - DCL36	10	7	2	0	-06619	35303
37 - DCL37	10	4	3	-2.2194	7.1879	2.3453
37 - DCL37	10	5	1	-1.3150	7.1879	2.9491
37 - DCL37	10	5	2	-2.2637	12.391	1.6134
37 - DCL37	10	5	3	-3.2125	14.548	2.7769
37 - DCL37	10	6	1	-3.2057	14.548	3.4682
37 - DCL37	10	6	2	-4.2810	13.832	-97217
37 - DCL37	10	6	3	-5.3562	10.096	-2.2911
37 - DCL37	10	7	1	0	-264.78	70607
37 - DCL37	10	7	2	0	-06619	35303
38 - DCL38	10	4	3	-52.070	94.909	-9.1006
38 - DCL38	10	5	1	-52.138	94.909	8.7012
38 - DCL38	10	5	2	-55.269	93.329	-10.086
38 - DCL38	10	5	3	-58.400	48.895	-28.873
38 - DCL38	10	6	1	-59.009	48.895	-27.609
38 - DCL38	10	6	2	-62.557	-35.833	-46.396
38 - DCL38	10	6	3	-66.106	-163.58	-65.184
38 - DCL38	10	7	1	0	-264.78	70607
38 - DCL38	10	7	2	0	-06619	35303
1 - DCL1	10	7	3	.48195	0	3.2400
1 - DCL1	10	8	1	-.48195	0	-121.50
1 - DCL1	10	8	2	-.48195	-45.648	-121.95
1 - DCL1	10	8	3	-.48195	-91.469	-122.41
1 - DCL1	10	9	1	-33.617	24.711	14.594
1 - DCL1	10	9	2	-33.617	53.899	14.594
1 - DCL1	10	10	1	-33.617	53.899	26.106
1 - DCL1	10	10	2	-32.932	66.953	26.106
1 - DCL1	10	10	3	-32.246	80.006	26.106
2 - DCL2	10	7	3	.48195	0	121.50
2 - DCL2	10	8	1	-.48195	0	-3.2400
2 - DCL2	10	8	2	-.48195	-1.3010	-3.6989
2 - DCL2	10	8	3	-.48195	-2.7742	-4.1579
2 - DCL2	10	9	1	-1.4787	-56.819	-29248
2 - DCL2	10	9	2	-1.4787	-51.194	5.9175
2 - DCL2	10	10	1	-1.4787	-51.194	7.2819
2 - DCL2	10	10	2	-.79354	-47.131	8.9694
2 - DCL2	10	10	3	-.10829	-42.225	10.656
3 - DCL3	10	7	3	.48195	0	3.2400
3 - DCL3	10	8	1	-.48195	0	-121.50
3 - DCL3	10	8	2	-.48195	-45.648	-121.95
3 - DCL3	10	8	3	-.48195	-91.469	-122.41
3 - DCL3	10	9	1	-6.0004	-16.167	-29455
3 - DCL3	10	9	2	-6.0004	-10.546	5.9154
3 - DCL3	10	10	1	-6.0004	-10.546	7.3260
3 - DCL3	10	10	2	-5.3152	-6.4616	9.0135
3 - DCL3	10	10	3	-4.6299	-1.5329	10.701
4 - DCL4	10	7	3	.48195	0	121.50
4 - DCL4	10	8	1	-.48195	0	-3.2400
4 - DCL4	10	8	2	-.48195	-1.3010	-3.6989
4 - DCL4	10	8	3	-.48195	-2.7742	-4.1579
4 - DCL4	10	9	1	-8.3048	-22.259	16.422
4 - DCL4	10	9	2	-8.3048	6.1301	11.967
4 - DCL4	10	10	1	-8.3048	6.1301	13.590
4 - DCL4	10	10	2	-7.6195	12.621	12.375

Table 5 Forces from DCL [EUROCODE 1] (Q5). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
4 - DCL4	10	10	3	-6.9343	18.505	11.160
5 - DCL5	10	7	3	.48195	0	3.2400
5 - DCL5	10	8	1	-.48195	0	-121.50
5 - DCL5	10	8	2	-.48195	-45.648	-121.95
5 - DCL5	10	8	3	-.48195	-91.469	-122.41
5 - DCL5	10	9	1	-12.826	18.393	16.420
5 - DCL5	10	9	2	-12.826	46.778	11.965
5 - DCL5	10	10	1	-12.826	46.778	13.634
5 - DCL5	10	10	2	-12.141	53.292	12.419
5 - DCL5	10	10	3	-11.456	59.198	11.204
6 - DCL6	10	7	3	0	0	0
6 - DCL6	10	8	1	0	0	0
6 - DCL6	10	8	2	0	-.08605	-.45895
6 - DCL6	10	8	3	0	-.34421	-.91790
6 - DCL6	10	9	1	-103.27	88.785	29.777
6 - DCL6	10	9	2	-103.27	148.34	29.777
6 - DCL6	10	10	1	-103.27	148.34	70.558
6 - DCL6	10	10	2	-102.58	183.62	70.558
6 - DCL6	10	10	3	-101.90	218.89	70.558
7 - DCL7	10	7	3	0	0	0
7 - DCL7	10	8	1	0	0	0
7 - DCL7	10	8	2	0	-.08605	-.45895
7 - DCL7	10	8	3	0	-.34421	-.91790
7 - DCL7	10	9	1	-75.656	47.907	14.888
7 - DCL7	10	9	2	-75.656	83.894	21.098
7 - DCL7	10	10	1	-75.656	83.894	51.777
7 - DCL7	10	10	2	-74.971	110.20	53.464
7 - DCL7	10	10	3	-74.285	137.35	55.152
8 - DCL8	10	7	3	0	0	0
8 - DCL8	10	8	1	0	0	0
8 - DCL8	10	8	2	0	-.08605	-.45895
8 - DCL8	10	8	3	0	-.34421	-.91790
8 - DCL8	10	9	1	-75.656	47.907	14.888
8 - DCL8	10	9	2	-75.656	83.894	21.098
8 - DCL8	10	10	1	-75.656	83.894	51.777
8 - DCL8	10	10	2	-74.971	110.20	53.464
8 - DCL8	10	10	3	-74.285	137.35	55.152
9 - DCL9	10	7	3	0	0	0
9 - DCL9	10	8	1	0	0	0
9 - DCL9	10	8	2	0	-.08605	-.45895
9 - DCL9	10	8	3	0	-.34421	-.91790
9 - DCL9	10	9	1	-82.482	82.468	31.603
9 - DCL9	10	9	2	-82.482	141.21	27.148
9 - DCL9	10	10	1	-82.482	141.21	58.086
9 - DCL9	10	10	2	-81.797	169.95	56.871
9 - DCL9	10	10	3	-81.111	198.09	55.656
10 - DCL10	10	7	3	0	0	0
10 - DCL10	10	8	1	0	0	0
10 - DCL10	10	8	2	0	-.08605	-.45895
10 - DCL10	10	8	3	0	-.34421	-.91790
10 - DCL10	10	9	1	-82.482	82.468	31.603
10 - DCL10	10	9	2	-82.482	141.21	27.148
10 - DCL10	10	10	1	-82.482	141.21	58.086
10 - DCL10	10	10	2	-81.797	169.95	56.871
10 - DCL10	10	10	3	-81.111	198.09	55.656

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
11 - DCL11	10	7	3	.48195	0	3.2400
11 - DCL11	10	8	1	-.48195	0	-121.50
11 - DCL11	10	8	2	-.48195	-45.648	-121.95
11 - DCL11	10	8	3	-.48195	-91.469	-122.41
11 - DCL11	10	9	1	-84.659	81.088	37.380
11 - DCL11	10	9	2	-84.659	151.39	32.925
11 - DCL11	10	10	1	-84.659	151.39	63.345
11 - DCL11	10	10	2	-83.974	182.76	62.130
11 - DCL11	10	10	3	-83.289	213.52	60.915
12 - DCL12	10	7	3	.48195	0	121.50
12 - DCL12	10	8	1	-.48195	0	-3.2400
12 - DCL12	10	8	2	-.48195	-1.3010	-3.6989
12 - DCL12	10	8	3	-.48195	-2.7742	-4.1579
12 - DCL12	10	9	1	-73.311	5.8753	20.668
12 - DCL12	10	9	2	-73.311	53.422	26.878
12 - DCL12	10	10	1	-73.311	53.422	56.992
12 - DCL12	10	10	2	-72.626	82.340	58.680
12 - DCL12	10	10	3	-71.941	112.10	60.367
13 - DCL13	10	7	3	.48195	0	3.2400
13 - DCL13	10	8	1	-.48195	0	-121.50
13 - DCL13	10	8	2	-.48195	-45.648	-121.95
13 - DCL13	10	8	3	-.48195	-91.469	-122.41
13 - DCL13	10	9	1	-12.826	18.393	16.420
13 - DCL13	10	9	2	-12.826	46.778	11.965
13 - DCL13	10	10	1	-12.826	46.778	13.634
13 - DCL13	10	10	2	-12.141	53.292	12.419
13 - DCL13	10	10	3	-11.456	59.198	11.204
14 - DCL14	10	7	3	.48195	0	121.50
14 - DCL14	10	8	1	-.48195	0	-3.2400
14 - DCL14	10	8	2	-.48195	-1.3010	-3.6989
14 - DCL14	10	8	3	-.48195	-2.7742	-4.1579
14 - DCL14	10	9	1	-80.137	40.436	37.382
14 - DCL14	10	9	2	-80.137	110.74	32.927
14 - DCL14	10	10	1	-80.137	110.74	63.301
14 - DCL14	10	10	2	-79.452	142.09	62.086
14 - DCL14	10	10	3	-78.767	172.83	60.871
15 - DCL15	10	7	3	0	0	0
15 - DCL15	10	8	1	0	0	0
15 - DCL15	10	8	2	0	-.08605	-.45895
15 - DCL15	10	8	3	0	-.34421	-.91790
15 - DCL15	10	9	1	-.75483	-19.329	-7.7263
15 - DCL15	10	9	2	-.75483	-27.882	-.82629
15 - DCL15	10	10	1	-.75483	-27.882	-.01994
15 - DCL15	10	10	2	-.06958	-27.423	1.8550
15 - DCL15	10	10	3	.61566	-26.027	3.7300
16 - DCL16	10	7	3	0	0	0
16 - DCL16	10	8	1	0	0	0
16 - DCL16	10	8	2	0	-.08605	-.45895
16 - DCL16	10	8	3	0	-.34421	-.91790
16 - DCL16	10	9	1	-3.8233	-14.787	-6.0719
16 - DCL16	10	9	2	-3.8233	-20.721	.13802
16 - DCL16	10	10	1	-3.8233	-20.721	2.0668
16 - DCL16	10	10	2	-3.1381	-19.266	3.7543
16 - DCL16	10	10	3	-2.4528	-16.967	5.4418
17 - DCL17	10	7	3	0	0	0

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
17 - DCL17	10	8	1	0	0	0
17 - DCL17	10	8	2	0	-.08605	-.45895
17 - DCL17	10	8	3	0	-.34421	-.91790
17 - DCL17	10	9	1	-10.649	19.772	10.642
17 - DCL17	10	9	2	-10.649	36.603	6.1876
17 - DCL17	10	10	1	-10.649	36.603	8.3754
17 - DCL17	10	10	2	-9.9641	40.487	7.1604
17 - DCL17	10	10	3	-9.2789	43.763	5.9454
18 - DCL18	10	7	3	0	0	0
18 - DCL18	10	8	1	0	0	0
18 - DCL18	10	8	2	0	-.08605	-.45895
18 - DCL18	10	8	3	0	-.34421	-.91790
18 - DCL18	10	9	1	-10.649	19.772	10.642
18 - DCL18	10	9	2	-10.649	36.603	6.1876
18 - DCL18	10	10	1	-10.649	36.603	8.3754
18 - DCL18	10	10	2	-9.9641	40.487	7.1604
18 - DCL18	10	10	3	-9.2789	43.763	5.9454
19 - DCL19	10	7	3	0	0	0
19 - DCL19	10	8	1	0	0	0
19 - DCL19	10	8	2	0	-.08605	-.45895
19 - DCL19	10	8	3	0	-.34421	-.91790
19 - DCL19	10	9	1	-103.27	88.785	29.777
19 - DCL19	10	9	2	-103.27	148.34	29.777
19 - DCL19	10	10	1	-103.27	148.34	70.558
19 - DCL19	10	10	2	-102.58	183.62	70.558
19 - DCL19	10	10	3	-101.90	218.89	70.558
20 - DCL20	10	7	3	.35700	0	90.000
20 - DCL20	10	8	1	-.35700	0	-2.4000
20 - DCL20	10	8	2	-.35700	-.96619	-2.7530
20 - DCL20	10	8	3	-.35700	-2.0647	-3.1060
20 - DCL20	10	9	1	-22.448	-11.065	11.063
20 - DCL20	10	9	2	-22.448	11.061	11.063
20 - DCL20	10	10	1	-22.448	11.061	19.899
20 - DCL20	10	10	2	-21.921	21.011	19.899
20 - DCL20	10	10	3	-21.393	30.961	19.899
21 - DCL21	10	7	3	.32130	0	81.000
21 - DCL21	10	8	1	-.32130	0	-2.1600
21 - DCL21	10	8	2	-.32130	-.87619	-2.5130
21 - DCL21	10	8	3	-.32130	-1.8847	-2.8660
21 - DCL21	10	9	1	-4.2105	-35.203	.70930
21 - DCL21	10	9	2	-4.2105	-29.645	4.8493
21 - DCL21	10	10	1	-4.2105	-29.645	6.9928
21 - DCL21	10	10	2	-3.6833	-25.867	8.1178
21 - DCL21	10	10	3	-3.1562	-21.527	9.2428
22 - DCL22	10	7	3	.32130	0	2.1600
22 - DCL22	10	8	1	-.32130	0	-81.000
22 - DCL22	10	8	2	-.32130	-30.441	-81.353
22 - DCL22	10	8	3	-.32130	-61.014	-81.706
22 - DCL22	10	9	1	-7.2249	-8.1022	.70792
22 - DCL22	10	9	2	-7.2249	-2.5464	4.8479
22 - DCL22	10	10	1	-7.2249	-2.5464	7.0222
22 - DCL22	10	10	2	-6.6978	1.2459	8.1472
22 - DCL22	10	10	3	-6.1707	5.6008	9.2722
23 - DCL23	10	7	3	.32130	0	81.000
23 - DCL23	10	8	1	-.32130	0	-2.1600

Table 5 Forces from DCL [EUROCODE 1] (Q5). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
23 - DCL23	10	8	2	-.32130	-.87619	-2.5130
23 - DCL23	10	8	3	-.32130	-1.8847	-2.8660
23 - DCL23	10	9	1	-8.7611	-12.163	11.852
23 - DCL23	10	9	2	-8.7611	8.5713	8.8823
23 - DCL23	10	10	1	-8.7611	8.5713	11.198
23 - DCL23	10	10	2	-8.2340	13.968	10.388
23 - DCL23	10	10	3	-7.7069	18.959	9.5785
24 - DCL24	10	7	3	.32130	0	2.1600
24 - DCL24	10	8	1	-.32130	0	-81.000
24 - DCL24	10	8	2	-.32130	-30.441	-81.353
24 - DCL24	10	8	3	-.32130	-61.014	-81.706
24 - DCL24	10	9	1	-11.775	14.938	11.850
24 - DCL24	10	9	2	-11.775	35.670	8.8809
24 - DCL24	10	10	1	-11.775	35.670	11.227
24 - DCL24	10	10	2	-11.248	41.081	10.417
24 - DCL24	10	10	3	-10.721	46.088	9.6079
25 - DCL25	10	7	3	.32130	0	81.000
25 - DCL25	10	8	1	-.32130	0	-2.1600
25 - DCL25	10	8	2	-.32130	-.87619	-2.5130
25 - DCL25	10	8	3	-.32130	-1.8847	-2.8660
25 - DCL25	10	9	1	-70.510	33.845	24.609
25 - DCL25	10	9	2	-70.510	83.063	24.609
25 - DCL25	10	10	1	-70.510	83.063	52.653
25 - DCL25	10	10	2	-69.983	109.39	52.653
25 - DCL25	10	10	3	-69.456	135.71	52.653
26 - DCL26	10	7	3	0	0	0
26 - DCL26	10	8	1	0	0	0
26 - DCL26	10	8	2	0	-.06619	-.35303
26 - DCL26	10	8	3	0	-.26478	-.70607
26 - DCL26	10	9	1	-53.662	34.614	10.830
26 - DCL26	10	9	2	-53.662	60.414	14.970
26 - DCL26	10	10	1	-53.662	60.414	36.656
26 - DCL26	10	10	2	-53.135	79.024	37.781
26 - DCL26	10	10	3	-52.608	98.196	38.906
27 - DCL27	10	7	3	0	0	0
27 - DCL27	10	8	1	0	0	0
27 - DCL27	10	8	2	0	-.06619	-.35303
27 - DCL27	10	8	3	0	-.26478	-.70607
27 - DCL27	10	9	1	-53.662	34.614	10.830
27 - DCL27	10	9	2	-53.662	60.414	14.970
27 - DCL27	10	10	1	-53.662	60.414	36.656
27 - DCL27	10	10	2	-53.135	79.024	37.781
27 - DCL27	10	10	3	-52.608	98.196	38.906
28 - DCL28	10	7	3	0	0	0
28 - DCL28	10	8	1	0	0	0
28 - DCL28	10	8	2	0	-.06619	-.35303
28 - DCL28	10	8	3	0	-.26478	-.70607
28 - DCL28	10	9	1	-58.212	57.654	21.973
28 - DCL28	10	9	2	-58.212	98.631	19.003
28 - DCL28	10	10	1	-58.212	98.631	40.862
28 - DCL28	10	10	2	-57.685	118.85	40.052
28 - DCL28	10	10	3	-57.158	138.68	39.242
29 - DCL29	10	7	3	0	0	0
29 - DCL29	10	8	1	0	0	0
29 - DCL29	10	8	2	0	-.06619	-.35303

Table 5 Forces from DCL [EUROCODE 1] (Q5). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
29 - DCL29	10	8	3	0	-264.78	-70607
29 - DCL29	10	9	1	-58.212	57.654	21.973
29 - DCL29	10	9	2	-58.212	98.631	19.003
29 - DCL29	10	10	1	-58.212	98.631	40.862
29 - DCL29	10	10	2	-57.685	118.85	40.052
29 - DCL29	10	10	3	-57.158	138.68	39.242
30 - DCL30	10	7	3	.32130	0	81.000
30 - DCL30	10	8	1	-.32130	0	-2.1600
30 - DCL30	10	8	2	-.32130	-.87619	-2.5130
30 - DCL30	10	8	3	-.32130	-1.8847	-2.8660
30 - DCL30	10	9	1	-52.099	6.5928	14.683
30 - DCL30	10	9	2	-52.099	40.099	18.823
30 - DCL30	10	10	1	-52.099	40.099	40.133
30 - DCL30	10	10	2	-51.572	60.447	41.258
30 - DCL30	10	10	3	-51.044	81.357	42.383
31 - DCL31	10	7	3	.32130	0	2.1600
31 - DCL31	10	8	1	-.32130	0	-81.000
31 - DCL31	10	8	2	-.32130	-30.441	-81.353
31 - DCL31	10	8	3	-.32130	-61.014	-81.706
31 - DCL31	10	9	1	-55.113	33.694	14.681
31 - DCL31	10	9	2	-55.113	67.198	18.821
31 - DCL31	10	10	1	-55.113	67.198	40.162
31 - DCL31	10	10	2	-54.586	87.560	41.287
31 - DCL31	10	10	3	-54.059	108.48	42.412
32 - DCL32	10	7	3	.32130	0	81.000
32 - DCL32	10	8	1	-.32130	0	-2.1600
32 - DCL32	10	8	2	-.32130	-.87619	-2.5130
32 - DCL32	10	8	3	-.32130	-1.8847	-2.8660
32 - DCL32	10	9	1	-56.649	29.633	25.826
32 - DCL32	10	9	2	-56.649	78.315	22.856
32 - DCL32	10	10	1	-56.649	78.315	44.339
32 - DCL32	10	10	2	-56.122	100.28	43.529
32 - DCL32	10	10	3	-55.595	121.84	42.719
33 - DCL33	10	7	3	.32130	0	2.1600
33 - DCL33	10	8	1	-.32130	0	-81.000
33 - DCL33	10	8	2	-.32130	-30.441	-81.353
33 - DCL33	10	8	3	-.32130	-61.014	-81.706
33 - DCL33	10	9	1	-59.664	56.734	25.824
33 - DCL33	10	9	2	-59.664	105.41	22.854
33 - DCL33	10	10	1	-59.664	105.41	44.368
33 - DCL33	10	10	2	-59.137	127.39	43.558
33 - DCL33	10	10	3	-58.610	148.97	42.748
34 - DCL34	10	7	3	0	0	0
34 - DCL34	10	8	1	0	0	0
34 - DCL34	10	8	2	0	-.06619	-.35303
34 - DCL34	10	8	3	0	-.26478	-.70607
34 - DCL34	10	9	1	-3.7278	-10.210	-4.2465
34 - DCL34	10	9	2	-3.7278	-14.103	.35343
34 - DCL34	10	10	1	-3.7278	-14.103	2.1249
34 - DCL34	10	10	2	-3.2007	-12.728	3.3749
34 - DCL34	10	10	3	-2.6736	-10.728	4.6249
35 - DCL35	10	7	3	0	0	0
35 - DCL35	10	8	1	0	0	0
35 - DCL35	10	8	2	0	-.06619	-.35303
35 - DCL35	10	8	3	0	-.26478	-.70607

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
35 - DCL35	10	9	1	-3.7278	-10.210	-4.2465
35 - DCL35	10	9	2	-3.7278	-14.103	.35343
35 - DCL35	10	10	1	-3.7278	-14.103	2.1249
35 - DCL35	10	10	2	-3.2007	-12.728	3.3749
35 - DCL35	10	10	3	-2.6736	-10.728	4.6249
36 - DCL36	10	7	3	0	0	0
36 - DCL36	10	8	1	0	0	0
36 - DCL36	10	8	2	0	-.06619	-.35303
36 - DCL36	10	8	3	0	-.26478	-.70607
36 - DCL36	10	9	1	-8.7841	15.389	8.1346
36 - DCL36	10	9	2	-8.7841	28.359	4.8346
36 - DCL36	10	10	1	-8.7841	28.359	6.7979
36 - DCL36	10	10	2	-8.2570	31.533	5.8979
36 - DCL36	10	10	3	-7.7299	34.257	4.9979
37 - DCL37	10	7	3	0	0	0
37 - DCL37	10	8	1	0	0	0
37 - DCL37	10	8	2	0	-.06619	-.35303
37 - DCL37	10	8	3	0	-.26478	-.70607
37 - DCL37	10	9	1	-8.7841	15.389	8.1346
37 - DCL37	10	9	2	-8.7841	28.359	4.8346
37 - DCL37	10	10	1	-8.7841	28.359	6.7979
37 - DCL37	10	10	2	-8.2570	31.533	5.8979
37 - DCL37	10	10	3	-7.7299	34.257	4.9979
38 - DCL38	10	7	3	0	0	0
38 - DCL38	10	8	1	0	0	0
38 - DCL38	10	8	2	0	-.06619	-.35303
38 - DCL38	10	8	3	0	-.26478	-.70607
38 - DCL38	10	9	1	-77.394	66.510	22.308
38 - DCL38	10	9	2	-77.394	111.12	22.308
38 - DCL38	10	10	1	-77.394	111.12	52.859
38 - DCL38	10	10	2	-76.867	137.55	52.859
38 - DCL38	10	10	3	-76.340	163.98	52.859
1 - DCL1	10	11	1	-29.255	14.636	-14.594
1 - DCL1	10	11	2	-29.255	-14.552	-14.594
1 - DCL1	10	12	1	-29.255	-14.552	-26.106
1 - DCL1	10	12	2	-28.570	-27.605	-26.106
1 - DCL1	10	12	3	-27.885	-40.659	-26.106
1 - DCL1	10	13	1	11.512	0	0
1 - DCL1	10	13	2	11.512	0	0
1 - DCL1	10	13	3	11.512	0	0
2 - DCL2	10	11	1	-12.794	-19.750	-16.177
2 - DCL2	10	11	2	-12.794	-47.650	-11.722
2 - DCL2	10	12	1	-12.794	-47.650	-13.086
2 - DCL2	10	12	2	-12.109	-53.890	-11.871
2 - DCL2	10	12	3	-11.423	-59.522	-10.656
2 - DCL2	10	13	1	1.3644	0	0
2 - DCL2	10	13	2	1.3644	0	0
2 - DCL2	10	13	3	1.3644	0	0
3 - DCL3	10	11	1	-8.2727	20.986	-16.175
3 - DCL3	10	11	2	-8.2727	-6.9089	-11.720
3 - DCL3	10	12	1	-8.2727	-6.9089	-13.131
3 - DCL3	10	12	2	-7.5875	-13.170	-11.916
3 - DCL3	10	12	3	-6.9022	-18.824	-10.701
3 - DCL3	10	13	1	1.4105	0	0
3 - DCL3	10	13	2	1.4105	0	0

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
3 - DCL3	10	13	3	1.4105	0	0
4 - DCL4	10	11	1	-6.9403	14.101	.26388
4 - DCL4	10	11	2	-6.9403	8.4195	-5.9461
4 - DCL4	10	12	1	-6.9403	8.4195	-7.5695
4 - DCL4	10	12	2	-6.2550	4.2129	-9.2570
4 - DCL4	10	12	3	-5.5698	-.83744	-10.944
4 - DCL4	10	13	1	1.6233	0	0
4 - DCL4	10	13	2	1.6233	0	0
4 - DCL4	10	13	3	1.6233	0	0
5 - DCL5	10	11	1	-2.4186	54.839	.26595
5 - DCL5	10	11	2	-2.4186	49.161	-5.9440
5 - DCL5	10	12	1	-2.4186	49.161	-7.6136
5 - DCL5	10	12	2	-1.7333	44.932	-9.3011
5 - DCL5	10	12	3	-1.0481	39.860	-10.988
5 - DCL5	10	13	1	1.6695	0	0
5 - DCL5	10	13	2	1.6695	0	0
5 - DCL5	10	13	3	1.6695	0	0
6 - DCL6	10	11	1	-103.24	-88.243	-29.777
6 - DCL6	10	11	2	-103.24	-147.79	-29.777
6 - DCL6	10	12	1	-103.24	-147.79	-70.558
6 - DCL6	10	12	2	-102.56	-183.07	-70.558
6 - DCL6	10	12	3	-101.87	-218.35	-70.558
6 - DCL6	10	13	1	40.780	0	0
6 - DCL6	10	13	2	40.780	0	0
6 - DCL6	10	13	3	40.780	0	0
7 - DCL7	10	11	1	-82.265	-81.893	-31.358
7 - DCL7	10	11	2	-82.265	-140.15	-26.903
7 - DCL7	10	12	1	-82.265	-140.15	-57.582
7 - DCL7	10	12	2	-81.579	-168.64	-56.367
7 - DCL7	10	12	3	-80.894	-196.52	-55.152
7 - DCL7	10	13	1	30.678	0	0
7 - DCL7	10	13	2	30.678	0	0
7 - DCL7	10	13	3	30.678	0	0
8 - DCL8	10	11	1	-82.265	-81.893	-31.358
8 - DCL8	10	11	2	-82.265	-140.15	-26.903
8 - DCL8	10	12	1	-82.265	-140.15	-57.582
8 - DCL8	10	12	2	-81.579	-168.64	-56.367
8 - DCL8	10	12	3	-80.894	-196.52	-55.152
8 - DCL8	10	13	1	30.678	0	0
8 - DCL8	10	13	2	30.678	0	0
8 - DCL8	10	13	3	30.678	0	0
9 - DCL9	10	11	1	-76.410	-48.040	-14.917
9 - DCL9	10	11	2	-76.410	-84.085	-21.127
9 - DCL9	10	12	1	-76.410	-84.085	-52.065
9 - DCL9	10	12	2	-75.725	-110.53	-53.752
9 - DCL9	10	12	3	-75.040	-137.83	-55.440
9 - DCL9	10	13	1	30.937	0	0
9 - DCL9	10	13	2	30.937	0	0
9 - DCL9	10	13	3	30.937	0	0
10 - DCL10	10	11	1	-76.410	-48.040	-14.917
10 - DCL10	10	11	2	-76.410	-84.085	-21.127
10 - DCL10	10	12	1	-76.410	-84.085	-52.065
10 - DCL10	10	12	2	-75.725	-110.53	-53.752
10 - DCL10	10	12	3	-75.040	-137.83	-55.440
10 - DCL10	10	13	1	30.937	0	0

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
10 - DCL 10	10	13	2	30.937	0	0
10 - DCL 10	10	13	3	30.937	0	0
11 - DCL 11	10	11	1	-74.233	-74.738	-20.694
11 - DCL 11	10	11	2	-74.233	-55.073	-26.904
11 - DCL 11	10	12	1	-74.233	-55.073	-57.324
11 - DCL 11	10	12	2	-73.548	-84.157	-59.011
11 - DCL 11	10	12	3	-72.863	-114.08	-60.699
11 - DCL 11	10	13	1	30.419	0	0
11 - DCL 11	10	13	2	30.419	0	0
11 - DCL 11	10	13	3	30.419	0	0
12 - DCL 12	10	11	1	-84.609	-82.063	-37.138
12 - DCL 12	10	11	2	-84.609	-151.88	-32.683
12 - DCL 12	10	12	1	-84.609	-151.88	-62.797
12 - DCL 12	10	12	2	-83.924	-182.98	-61.582
12 - DCL 12	10	12	3	-83.239	-213.46	-60.367
12 - DCL 12	10	13	1	30.114	0	0
12 - DCL 12	10	13	2	30.114	0	0
12 - DCL 12	10	13	3	30.114	0	0
13 - DCL 13	10	11	1	-2.4186	54.839	.26595
13 - DCL 13	10	11	2	-2.4186	49.161	-5.9440
13 - DCL 13	10	12	1	-2.4186	49.161	-7.6136
13 - DCL 13	10	12	2	-1.7333	44.932	-9.3011
13 - DCL 13	10	12	3	-1.0481	39.860	-10.988
13 - DCL 13	10	13	1	1.6695	0	0
13 - DCL 13	10	13	2	1.6695	0	0
13 - DCL 13	10	13	3	1.6695	0	0
14 - DCL 14	10	11	1	-78.755	-48.211	-20.696
14 - DCL 14	10	11	2	-78.755	-95.815	-26.906
14 - DCL 14	10	12	1	-78.755	-95.815	-57.280
14 - DCL 14	10	12	2	-78.070	-124.87	-58.967
14 - DCL 14	10	12	3	-77.385	-154.78	-60.655
14 - DCL 14	10	13	1	30.373	0	0
14 - DCL 14	10	13	2	30.373	0	0
14 - DCL 14	10	13	3	30.373	0	0
15 - DCL 15	10	11	1	-8.1183	-18.874	-10.573
15 - DCL 15	10	11	2	-8.1183	-35.071	-5.6237
15 - DCL 15	10	12	1	-8.1183	-35.071	-6.4300
15 - DCL 15	10	12	2	-7.4331	-37.949	-5.0800
15 - DCL 15	10	12	3	-6.7479	-40.151	-3.7300
15 - DCL 15	10	13	1	.80634	0	0
15 - DCL 15	10	13	2	.80634	0	0
15 - DCL 15	10	13	3	.80634	0	0
16 - DCL 16	10	11	1	-10.449	-19.579	-10.398
16 - DCL 16	10	11	2	-10.449	-35.920	-5.9430
16 - DCL 16	10	12	1	-10.449	-35.920	-7.8718
16 - DCL 16	10	12	2	-9.7645	-39.553	-6.6568
16 - DCL 16	10	12	3	-9.0793	-42.577	-5.4418
16 - DCL 16	10	13	1	1.9288	0	0
16 - DCL 16	10	13	2	1.9288	0	0
16 - DCL 16	10	13	3	1.9288	0	0
17 - DCL 17	10	11	1	-4.5957	14.272	6.0433
17 - DCL 17	10	11	2	-4.5957	20.149	-1.6662
17 - DCL 17	10	12	1	-4.5957	20.149	-2.3544
17 - DCL 17	10	12	2	-3.9104	18.550	-4.0419
17 - DCL 17	10	12	3	-3.2252	16.107	-5.7294

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
17 - DCL17	10	13	1	2.1877	0	0
17 - DCL17	10	13	2	2.1877	0	0
17 - DCL17	10	13	3	2.1877	0	0
18 - DCL18	10	11	1	-4.5957	14.272	6.0433
18 - DCL18	10	11	2	-4.5957	20.149	-16662
18 - DCL18	10	12	1	-4.5957	20.149	-2.3544
18 - DCL18	10	12	2	-3.9104	18.550	-4.0419
18 - DCL18	10	12	3	-3.2252	16.107	-5.7294
18 - DCL18	10	13	1	2.1877	0	0
18 - DCL18	10	13	2	2.1877	0	0
18 - DCL18	10	13	3	2.1877	0	0
19 - DCL19	10	11	1	-103.24	-88.243	-29.777
19 - DCL19	10	11	2	-103.24	-147.79	-29.777
19 - DCL19	10	12	1	-103.24	-147.79	-70.558
19 - DCL19	10	12	2	-102.56	-183.07	-70.558
19 - DCL19	10	12	3	-101.87	-218.35	-70.558
19 - DCL19	10	13	1	40.780	0	0
19 - DCL19	10	13	2	40.780	0	0
19 - DCL19	10	13	3	40.780	0	0
20 - DCL20	10	11	1	-25.915	-20.072	-11.063
20 - DCL20	10	11	2	-25.915	-42.199	-11.063
20 - DCL20	10	12	1	-25.915	-42.199	-19.899
20 - DCL20	10	12	2	-25.388	-52.149	-19.899
20 - DCL20	10	12	3	-24.861	-62.099	-19.899
20 - DCL20	10	13	1	8.8364	0	0
20 - DCL20	10	13	2	8.8364	0	0
20 - DCL20	10	13	3	8.8364	0	0
21 - DCL21	10	11	1	-11.753	-15.826	-11.689
21 - DCL21	10	11	2	-11.753	-36.235	-8.7193
21 - DCL21	10	12	1	-11.753	-36.235	-10.862
21 - DCL21	10	12	2	-11.226	-41.464	-10.052
21 - DCL21	10	12	3	-10.699	-46.288	-9.2428
21 - DCL21	10	13	1	2.1435	0	0
21 - DCL21	10	13	2	2.1435	0	0
21 - DCL21	10	13	3	2.1435	0	0
22 - DCL22	10	11	1	-8.7390	11.331	-11.687
22 - DCL22	10	11	2	-8.7390	-9.0741	-8.7179
22 - DCL22	10	12	1	-8.7390	-9.0741	-10.892
22 - DCL22	10	12	2	-8.2119	-14.317	-10.082
22 - DCL22	10	12	3	-7.6848	-19.156	-9.2722
22 - DCL22	10	13	1	2.1743	0	0
22 - DCL22	10	13	2	2.1743	0	0
22 - DCL22	10	13	3	2.1743	0	0
23 - DCL23	10	11	1	-7.8507	6.7416	-72837
23 - DCL23	10	11	2	-7.8507	1.1449	-4.8683
23 - DCL23	10	12	1	-7.8507	1.1449	-7.1845
23 - DCL23	10	12	2	-7.3236	-2.7286	-8.3095
23 - DCL23	10	12	3	-6.7965	-7.1646	-9.4345
23 - DCL23	10	13	1	2.3162	0	0
23 - DCL23	10	13	2	2.3162	0	0
23 - DCL23	10	13	3	2.3162	0	0
24 - DCL24	10	11	1	-4.8363	33.900	-72699
24 - DCL24	10	11	2	-4.8363	28.306	-4.8669
24 - DCL24	10	12	1	-4.8363	28.306	-7.2139
24 - DCL24	10	12	2	-4.3091	24.417	-8.3389

Table 5 Forces from DCL [EUROCODE 1] (Q5). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
24 - DCL24	10	12	3	-3.7820	19.967	-9.4639
24 - DCL24	10	13	1	2.3469	0	0
24 - DCL24	10	13	2	2.3469	0	0
24 - DCL24	10	13	3	2.3469	0	0
25 - DCL25	10	11	1	-73.618	-61.602	-24.609
25 - DCL25	10	11	2	-73.618	-110.82	-24.609
25 - DCL25	10	12	1	-73.618	-110.82	-52.653
25 - DCL25	10	12	2	-73.091	-137.14	-52.653
25 - DCL25	10	12	3	-72.564	-163.47	-52.653
25 - DCL25	10	13	1	28.044	0	0
25 - DCL25	10	13	2	28.044	0	0
25 - DCL25	10	13	3	28.044	0	0
26 - DCL26	10	11	1	-58.067	-57.254	-21.810
26 - DCL26	10	11	2	-58.067	-97.905	-18.840
26 - DCL26	10	12	1	-58.067	-97.905	-40.526
26 - DCL26	10	12	2	-57.540	-117.96	-39.716
26 - DCL26	10	12	3	-57.013	-137.62	-38.906
26 - DCL26	10	13	1	21.686	0	0
26 - DCL26	10	13	2	21.686	0	0
26 - DCL26	10	13	3	21.686	0	0
27 - DCL27	10	11	1	-58.067	-57.254	-21.810
27 - DCL27	10	11	2	-58.067	-97.905	-18.840
27 - DCL27	10	12	1	-58.067	-97.905	-40.526
27 - DCL27	10	12	2	-57.540	-117.96	-39.716
27 - DCL27	10	12	3	-57.013	-137.62	-38.906
27 - DCL27	10	13	1	21.686	0	0
27 - DCL27	10	13	2	21.686	0	0
27 - DCL27	10	13	3	21.686	0	0
28 - DCL28	10	11	1	-54.164	-34.686	-10.849
28 - DCL28	10	11	2	-54.164	-60.525	-14.989
28 - DCL28	10	12	1	-54.164	-60.525	-36.848
28 - DCL28	10	12	2	-53.637	-79.230	-37.973
28 - DCL28	10	12	3	-53.110	-98.498	-39.098
28 - DCL28	10	13	1	21.859	0	0
28 - DCL28	10	13	2	21.859	0	0
28 - DCL28	10	13	3	21.859	0	0
29 - DCL29	10	11	1	-54.164	-34.686	-10.849
29 - DCL29	10	11	2	-54.164	-60.525	-14.989
29 - DCL29	10	12	1	-54.164	-60.525	-36.848
29 - DCL29	10	12	2	-53.637	-79.230	-37.973
29 - DCL29	10	12	3	-53.110	-98.498	-39.098
29 - DCL29	10	13	1	21.859	0	0
29 - DCL29	10	13	2	21.859	0	0
29 - DCL29	10	13	3	21.859	0	0
30 - DCL30	10	11	1	-59.630	-57.368	-25.663
30 - DCL30	10	11	2	-59.630	-105.72	-22.693
30 - DCL30	10	12	1	-59.630	-105.72	-44.003
30 - DCL30	10	12	2	-59.103	-127.52	-43.193
30 - DCL30	10	12	3	-58.576	-148.91	-42.383
30 - DCL30	10	13	1	21.310	0	0
30 - DCL30	10	13	2	21.310	0	0
30 - DCL30	10	13	3	21.310	0	0
31 - DCL31	10	11	1	-56.615	-30.210	-25.661
31 - DCL31	10	11	2	-56.615	-78.563	-22.691
31 - DCL31	10	12	1	-56.615	-78.563	-44.032

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
31 - DCL31	10	12	2	-56.088	-100.37	-43.222
31 - DCL31	10	12	3	-55.561	-121.78	-42.412
31 - DCL31	10	13	1	21.340	0	0
31 - DCL31	10	13	2	21.340	0	0
31 - DCL31	10	13	3	21.340	0	0
32 - DCL32	10	11	1	-55.727	-34.800	-14.702
32 - DCL32	10	11	2	-55.727	-68.344	-18.842
32 - DCL32	10	12	1	-55.727	-68.344	-40.325
32 - DCL32	10	12	2	-55.200	-88.788	-41.450
32 - DCL32	10	12	3	-54.673	-109.79	-42.575
32 - DCL32	10	13	1	21.482	0	0
32 - DCL32	10	13	2	21.482	0	0
32 - DCL32	10	13	3	21.482	0	0
33 - DCL33	10	11	1	-52.713	-7.6420	-14.700
33 - DCL33	10	11	2	-52.713	-41.183	-18.840
33 - DCL33	10	12	1	-52.713	-41.183	-40.354
33 - DCL33	10	12	2	-52.186	-61.642	-41.479
33 - DCL33	10	12	3	-51.658	-82.663	-42.604
33 - DCL33	10	13	1	21.513	0	0
33 - DCL33	10	13	2	21.513	0	0
33 - DCL33	10	13	3	21.513	0	0
34 - DCL34	10	11	1	-8.6361	-15.242	-7.9534
34 - DCL34	10	11	2	-8.6361	-27.849	-4.6534
34 - DCL34	10	12	1	-8.6361	-27.849	-6.4249
34 - DCL34	10	12	2	-8.1090	-30.836	-5.5249
34 - DCL34	10	12	3	-7.5819	-33.374	-4.6249
34 - DCL34	10	13	1	1.7715	0	0
34 - DCL34	10	13	2	1.7715	0	0
34 - DCL34	10	13	3	1.7715	0	0
35 - DCL35	10	11	1	-8.6361	-15.242	-7.9534
35 - DCL35	10	11	2	-8.6361	-27.849	-4.6534
35 - DCL35	10	12	1	-8.6361	-27.849	-6.4249
35 - DCL35	10	12	2	-8.1090	-30.836	-5.5249
35 - DCL35	10	12	3	-7.5819	-33.374	-4.6249
35 - DCL35	10	13	1	1.7715	0	0
35 - DCL35	10	13	2	1.7715	0	0
35 - DCL35	10	13	3	1.7715	0	0
36 - DCL36	10	11	1	-4.2997	9.8334	4.2253
36 - DCL36	10	11	2	-4.2997	13.684	-3.7461
36 - DCL36	10	12	1	-4.2997	13.684	-2.3379
36 - DCL36	10	12	2	-3.7726	12.202	-3.5879
36 - DCL36	10	12	3	-3.2455	10.096	-4.8379
36 - DCL36	10	13	1	1.9633	0	0
36 - DCL36	10	13	2	1.9633	0	0
36 - DCL36	10	13	3	1.9633	0	0
37 - DCL37	10	11	1	-4.2997	9.8334	4.2253
37 - DCL37	10	11	2	-4.2997	13.684	-3.7461
37 - DCL37	10	12	1	-4.2997	13.684	-2.3379
37 - DCL37	10	12	2	-3.7726	12.202	-3.5879
37 - DCL37	10	12	3	-3.2455	10.096	-4.8379
37 - DCL37	10	13	1	1.9633	0	0
37 - DCL37	10	13	2	1.9633	0	0
37 - DCL37	10	13	3	1.9633	0	0
38 - DCL38	10	11	1	-77.375	-66.104	-22.308
38 - DCL38	10	11	2	-77.375	-110.72	-22.308

Table 5 Forces from DCL [EUROCODE 1] (05). Internal Forces for DCL in Bars Elements

DCL No.	Typ	Elem	SN	N, kN	MY, kN*m	QZ, kN
38 - DCL38	10	12	1	-77.375	-110.72	-52.859
38 - DCL38	10	12	2	-76.848	-137.15	-52.859
38 - DCL38	10	12	3	-76.321	-163.58	-52.859
38 - DCL38	10	13	1	30.550	0	0
38 - DCL38	10	13	2	30.550	0	0
38 - DCL38	10	13	3	30.550	0	0

Table 6 Steel elements. Check. [Variant 1] (08)

Table 6 Steel elements. Check. [Variant 1] (08). Trusses

Gr	Elem	SN	Group	Class %	Strength %	Buckling %	Rupture %	Res.%	Length
1 - Main model:									
1	13	1		1	213.1	0.0	213.1	NG	18.00
1	13	1		0	[5.4.3]	[5.6.3]	[5.4.3]		
1	13	2		1	213.1	0.0	213.1	NG	18.00
1	13	2		0	[5.4.3]	[5.6.3]	[5.4.3]		
1	13	3		1	213.1	0.0	213.1	NG	18.00
1	13	3		0	[5.4.3]	[5.6.3]	[5.4.3]		

Table 6 Steel elements. Check. [Variant 1] (08). Beams

Gr	Elem	SN	Group	Class %	Strengt h %	Buckling %	Shear %	Loc.Bkl %	Deflec. %	Res.%	Length
1 - Section: 3.1.1.1 I-section 400											
Shape: 400/ tolerances: NF A 45-206 (novembre 1983) Euronorm 44-63											
Steel: Fe E 275/											
Steel file: Poutrelles IPE-A											
1	3	1		1	69.3	0.0	16.7	0.0	24.3	NG	3.05
1	3	1		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	3	2		1	29.4	0.0	13.4	0.0	137.0	NG	3.05
1	3	2		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	3	3		1	10.6	0.0	10.1	0.0	301.3	NG	3.05
1	3	3		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	4	1		1	10.6	0.0	10.2	0.0	150.8	NG	6.09
1	4	1		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	4	2		1	42.3	0.0	3.6	0.0	293.4	NG	6.09
1	4	2		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	4	3		1	43.8	0.0	3.0	0.0	339.5	NG	6.09
1	4	3		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	5	1		1	43.8	0.0	2.3	0.0	423.6	NG	4.56
1	5	1		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	5	2		1	41.9	0.0	2.9	0.0	371.8	NG	4.56
1	5	2		0	[5.4.5.2]	[5.5.2]	[5.4.6(1) J]	[5.6.3]	[4.2]		
1	5	3		1	20.6	0.0	7.9	0.0	246.6	NG	4.56

Table 6 Steel elements. Check. [Variant 1] (08). Beams

Gr	Elem	SN	Group	Class %	Strengt h %	Buckling %	Shear %	Loc.Bkl %	Deflec. %	Res.%	Length
1	5	3		0	[5.4.5.2]	[5.5.2]	[5.4.6(1)] J	[5.6.3]	[4.2]		
1	6	1		1	20.6	0.0	7.6	0.0	242.2	NG	4.58
1	6	1		0	[5.4.5.2]	[5.5.2]	[5.4.6(1)] J	[5.6.3]	[4.2]		
1	6	2		1	22.0	0.0	12.5	0.0	99.4	NG	4.58
1	6	2		0	[5.4.5.2]	[5.5.2]	[5.4.6(1)] J	[5.6.3]	[4.2]		
1	6	3		1	84.0	0.0	17.5	0.0	29.4	NG	4.58
1	6	3		0	[5.4.5.2]	[5.5.2]	[5.4.6(1)] J	[5.6.3]	[4.2]		

Table 6 Steel elements. Check. [Variant 1] (08). Columns

ГР	Элем	Сеч	Группа	Класс %	Прочн %	Устойч %	Срез %	Мст.уст. %	Рез.%	Длина
1 - Section: 3.1.2.1 I-section 400										
1	10	1		2	47.9	52.3	15.0	0.0	OK	1.00
1	10	1		0	[5.4.8.1]	[5.5.4(1)]	[5.4.6(1)]	[5.6.3]		
1	10	2		1	61.4	67.1	15.0	0.0	OK	1.00
1	10	2		0	[5.4.8.1]	[5.5.4(1)]	[5.4.6(1)]	[5.6.3]		
1	10	3		1	74.8	81.8	15.0	0.0	OK	1.00
1	10	3		0	[5.4.8.1]	[5.5.4(1)]	[5.4.6(1)]	[5.6.3]		
1	12	1		1	62.8	76.6	15.0	0.0	NG	1.00
1	12	1		0	[5.4.8.1]	[5.5.4(1)]	[5.4.6(1)]	[5.6.3]		
1	12	2		3	76.3	93.2	15.0	0.0	NG	1.00
1	12	2		0	[5.4.8.1]	[5.5.4(1)]	[5.4.6(1)]	[5.6.3]		
1	12	3		1	89.8	109.8	15.0	0.0	NG	1.00
1	12	3		0	[5.4.8.1]	[5.5.4(1)]	[5.4.6(1)]	[5.6.3]		

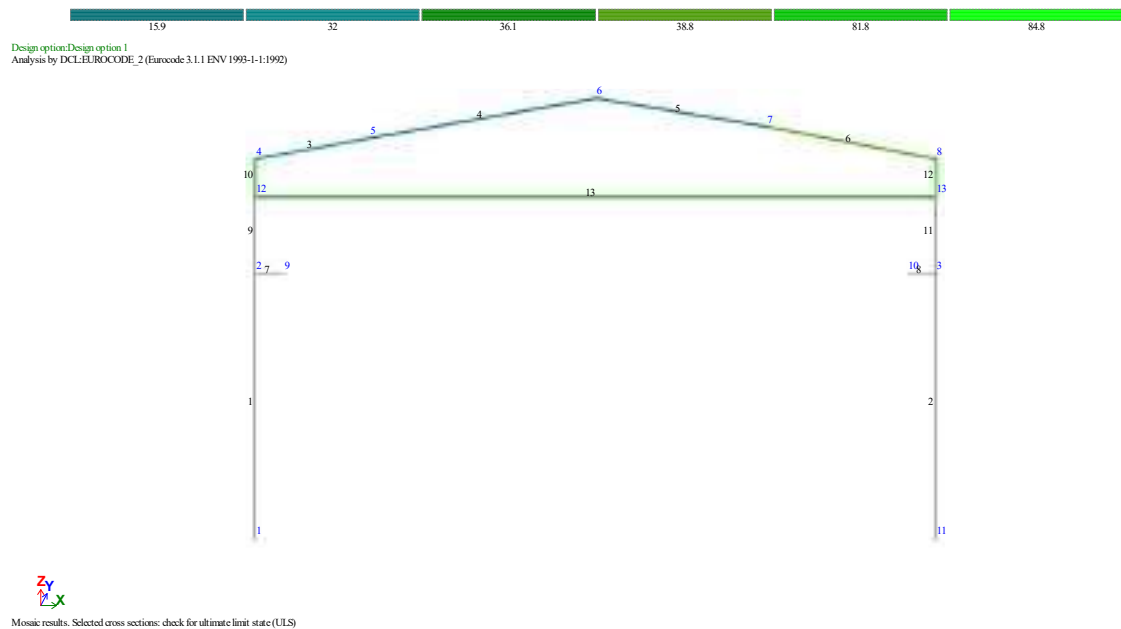
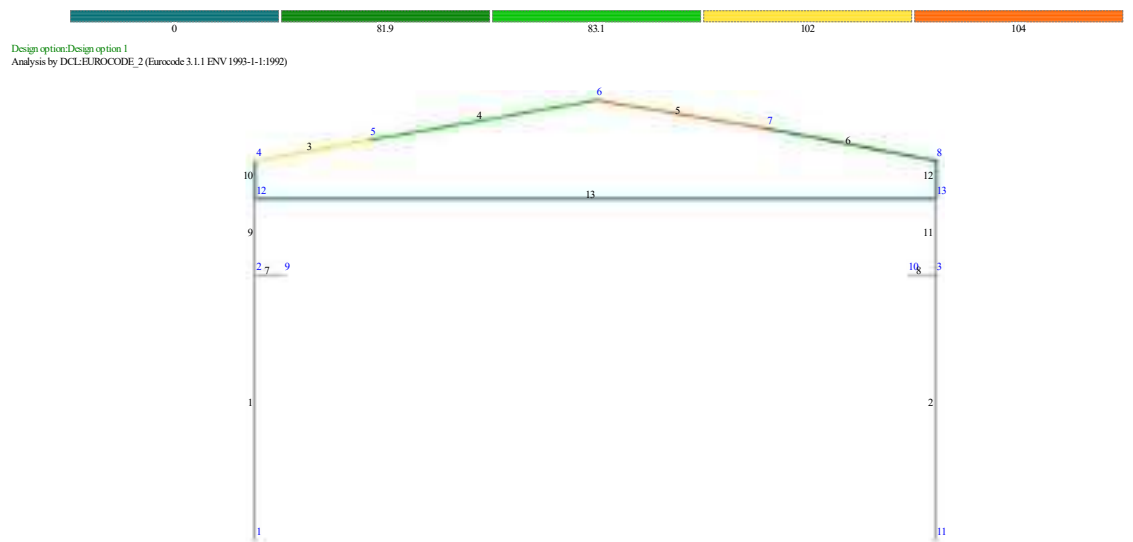


Fig. 13 Mosaic results. Selected cross sections: check for ultimate limit state (ULS)




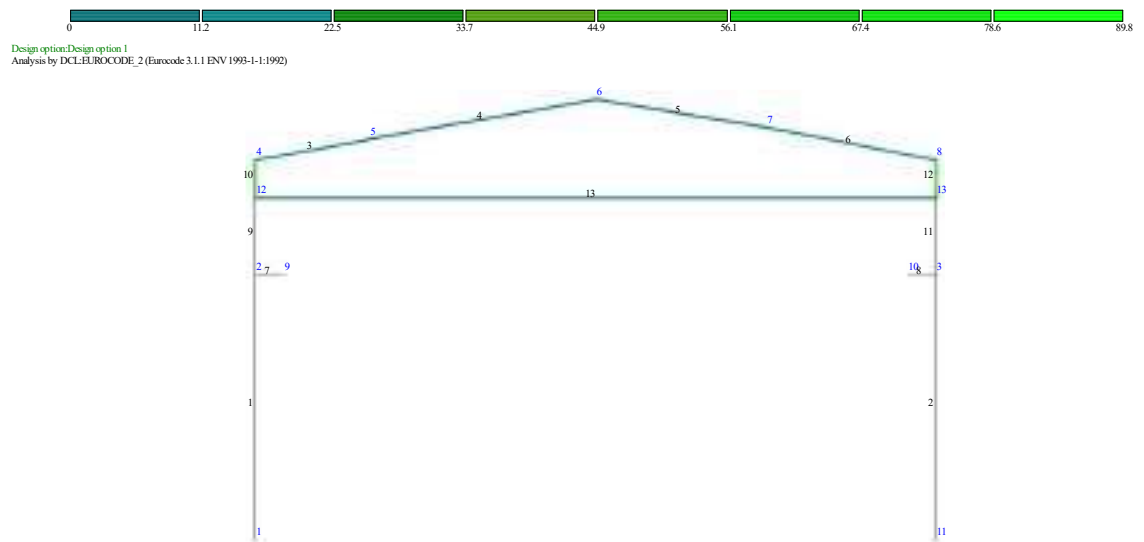

Mosaic results. Selected cross sections: check for serviceability limit state (SLS)

Fig. 14 Mosaic results. Selected cross sections: check for serviceability limit state (SLS)




Mosaic results. Assigned cross sections: check for normal stress

Fig. 15 Mosaic results. Assigned cross sections: check for normal stress

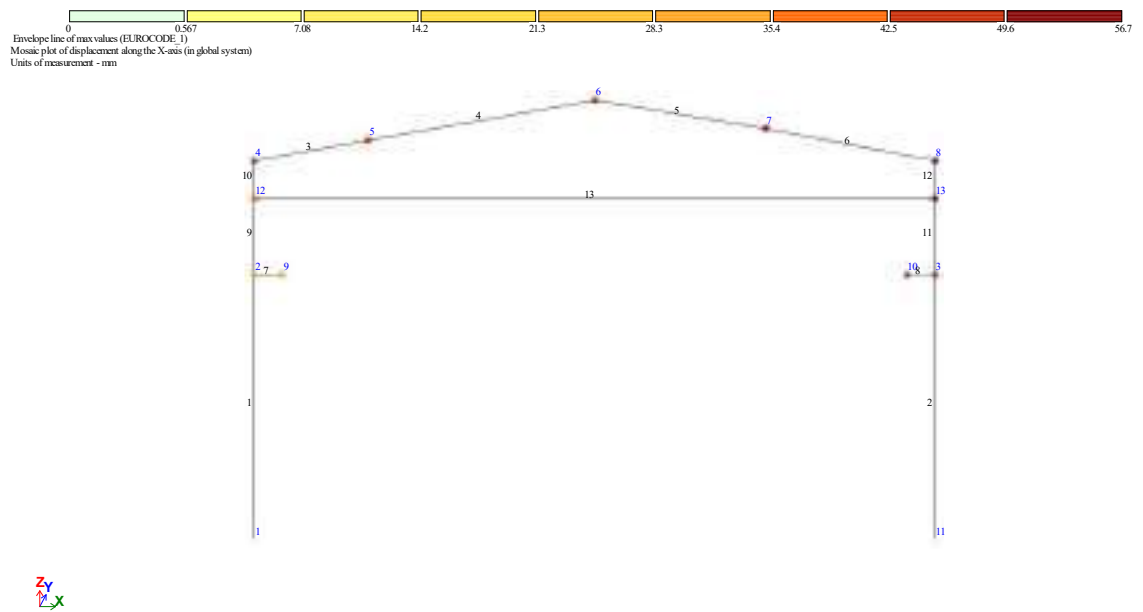


Fig. 16 Mosaic plot of displacement along the X-axis (in global system)

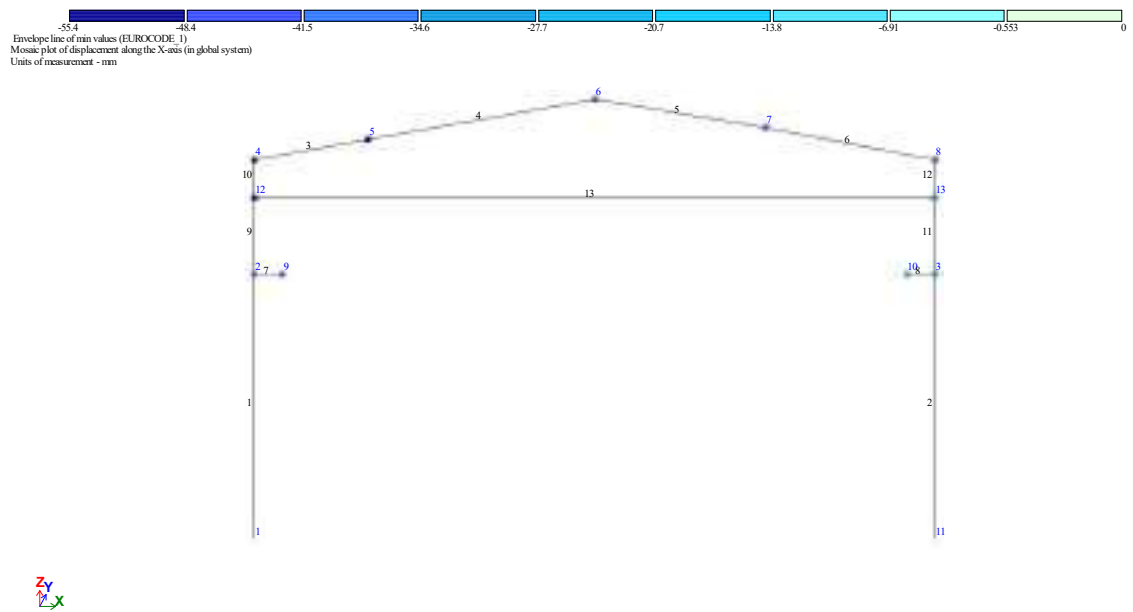


Fig. 17 Mosaic plot of displacement along the X-axis (in global system)(2)

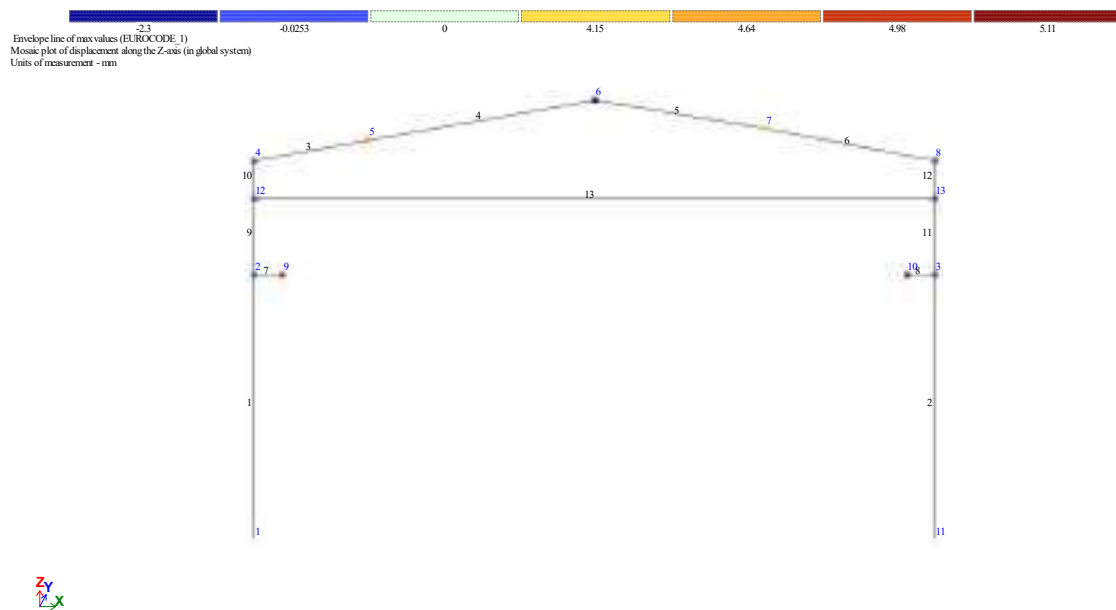


Fig. 18 Mosaic plot of displacement along the Z-axis (in global system)

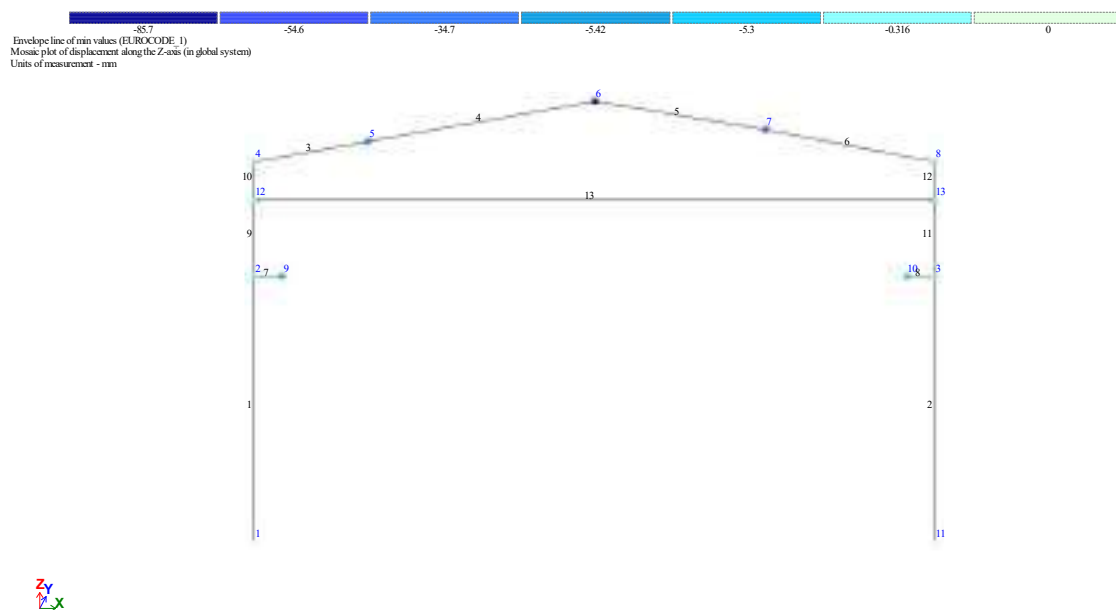


Fig. 19 Mosaic plot of displacement along the Z-axis (in global system)(2)

Envelope line of max values (EUROCODE 1)
Diagram N
Units of measurement - kN

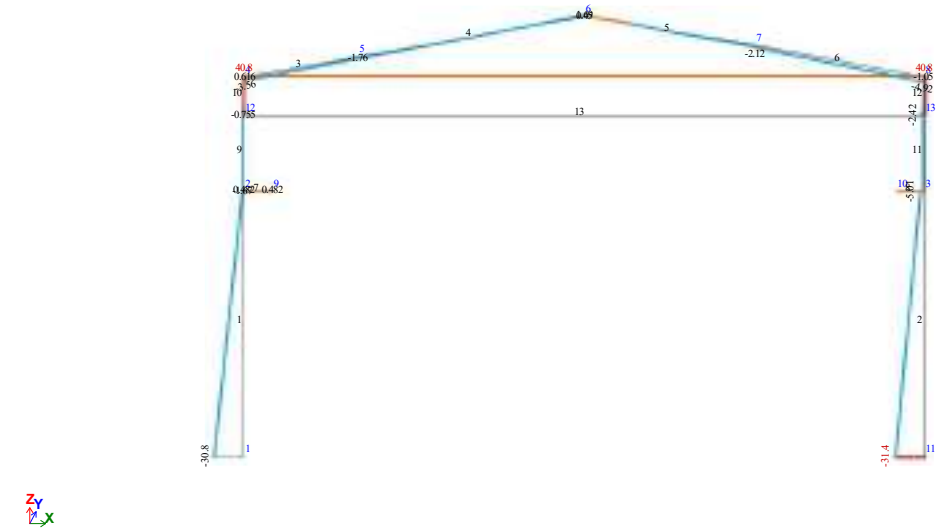


Fig. 20 Diagram N

Envelope line of min values (EUROCODE 1)
Diagram N
Units of measurement - kN

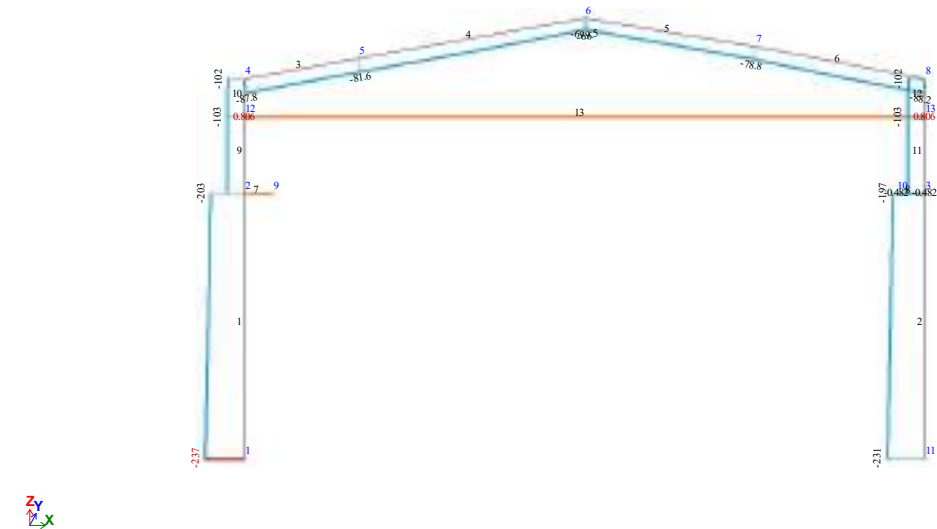
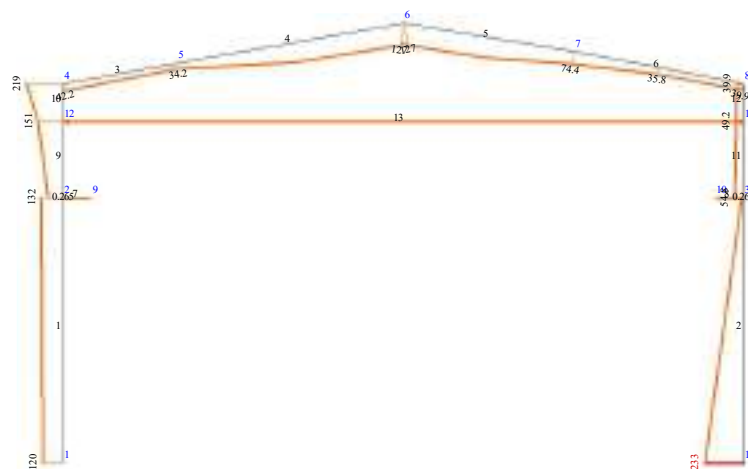


Fig. 21 Diagram N(2)

Envelope line of max values (EUROCODE 1)
Shows or hides diagram of moments M_y+M_z
Units of measurement - kN*m




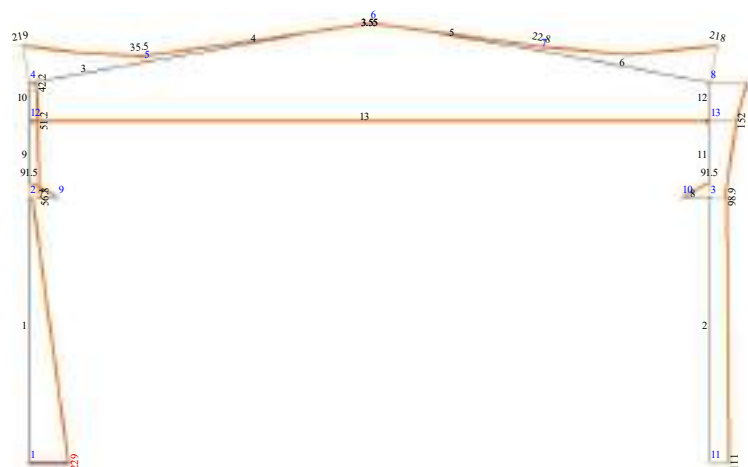

Maximum force: 233.078

Fig. 22 Shows or hides diagram of moments M_y+M_z

Envelope line of min values (EUROCODE 1)
Shows or hides diagram of moments M_y+M_z
Units of measurement - kN*m




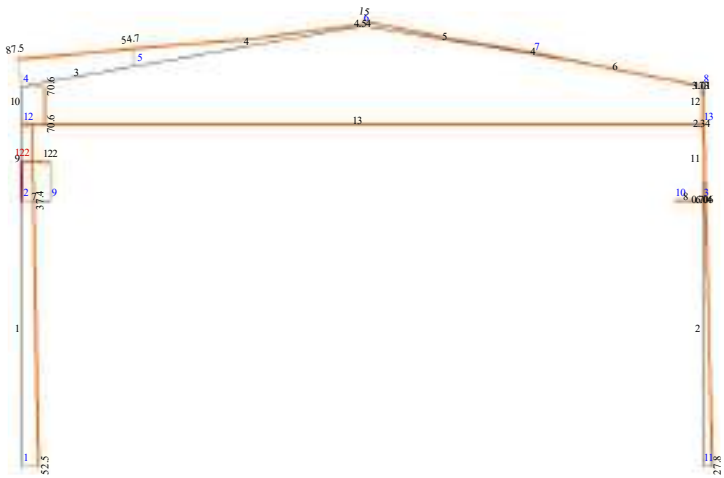

Maximum force: 229.003

Fig. 23 Shows or hides diagram of moments $M_y+M_z(2)$

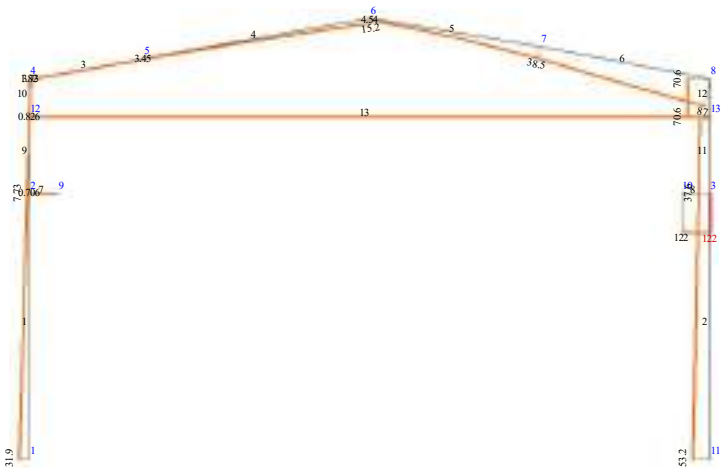
Envelope line of max values (EUROCODE 1)
Shows or hides diagram of forces Q_y+Q_z
Units of measurement - kN



Maximum force: 122.418

Fig. 24 Shows or hides diagram of forces Q_y+Q_z

Envelope line of min values (EUROCODE 1)
Shows or hides diagram of forces Q_y+Q_z
Units of measurement - kN



Maximum force: 122.418

Fig. 25 Shows or hides diagram of forces $Q_y+Q_z(2)$

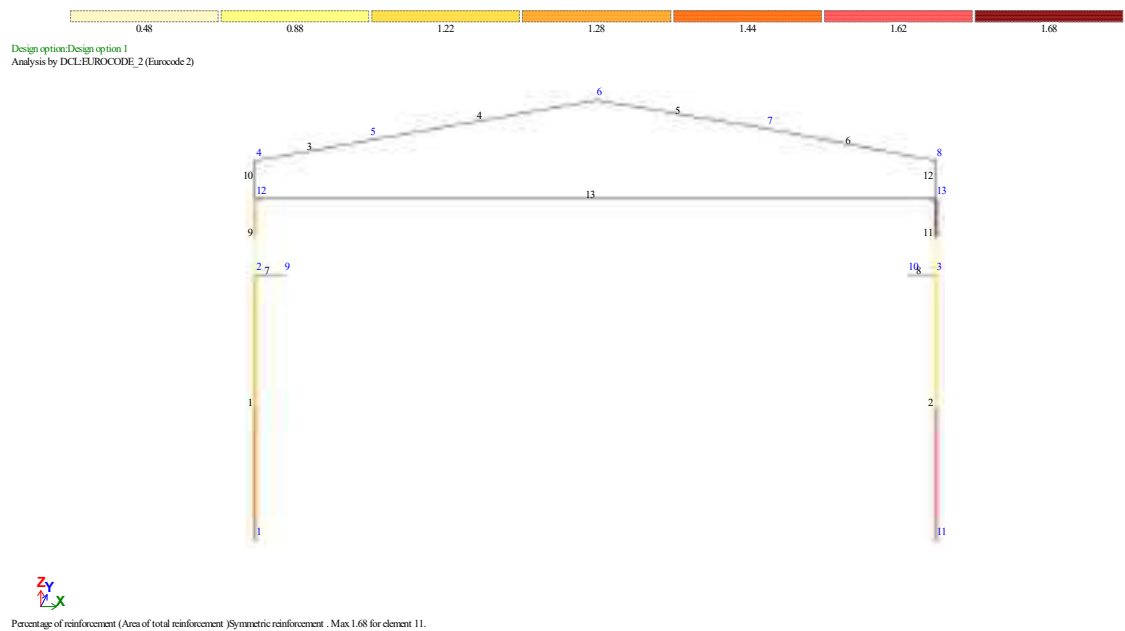


Fig. 26 Percentage of reinforcement (Area of total reinforcement / Symmetric reinforcement) . Max 1.68 for element 11.

CraneGirder EC

Crane supporting Structures according to Eurocode 1993-6
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Page: 1

Date: 10/3/2023

Time: 16:36

Project: TestProjekt
Identification: TestTravers

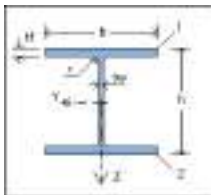
File: c:\users\viktoras\documents\sh01.cgec

General:

Material: S275 $f_y = 275 \text{ MPa}$ $E = 210000.0 \text{ MPa}$ $g_{m0}/g_{m1} = 1.00/1.00$
 $g_{mFf}/g_{mMf} = 1.00/1.15$

Crane & Loadcombinations:

Cranetype: Overhead Crane Craneclass Crane 1: S3
ULS: $1.35 \cdot (q_k + 1.10 \cdot V_k + 1.10 \cdot H_k)$ SLS: $1.00 \cdot (q_k + V_k + H_k)$ FLS: $1.00 \cdot 1.05 \cdot V_k$
Load on gangway $q_k = 0.2 \text{ kN/m}$

Profile:

Profile
1 - HEA 240

$M_y R_d / V_z R_d \text{ (kNm/kN)}$
204.9/399.8

Section Class M+/-
1/1

Comment

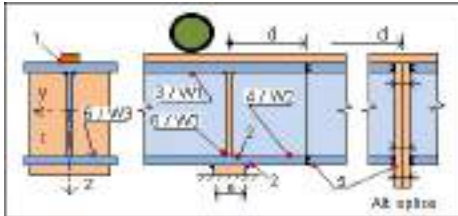
Deadload (q_k) of profile included, non rigid end-post (one stiffener/support)
Rail: PI 50x30, deadload (q_k) not included, not fixed mounted

Lateral Torsional Buckling:

	Positiv Moment	Negativ Moment
C1 = (Endspan/Midspan)	1.850/1.850	1.770/1.310
C2 = (Endspan/Midspan)	1.010/1.010	
C3 = (Endspan/Midspan)	0.640/0.640	1.000/1.000

Lateral torsional buckling curves: Rolled or welded I-sections (ch 6.3.2.3/EN 1993-1-1)

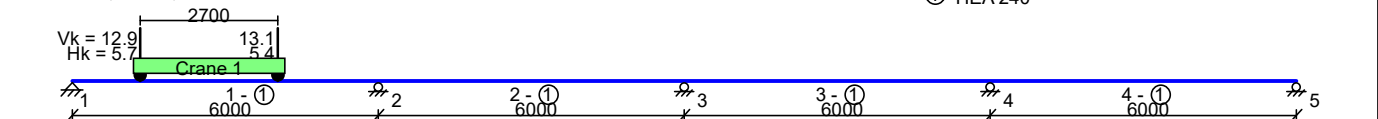
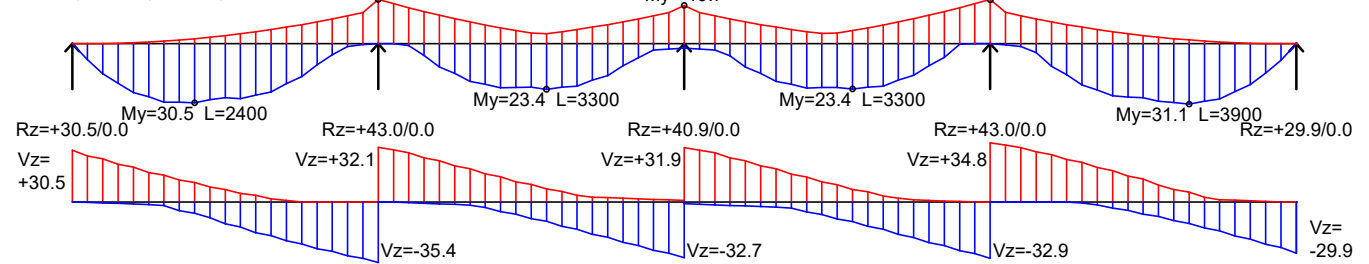
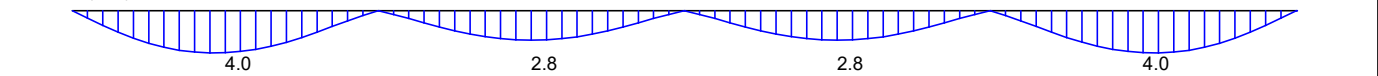
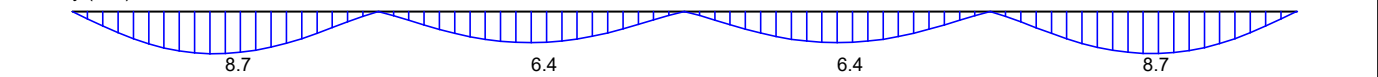
Load Level: Top of rail

Fatigue Details:

- 1 Rail to top flange
- 2 Stiffener to bottom flange
- 3 Top flange/web
- 6 Support Point

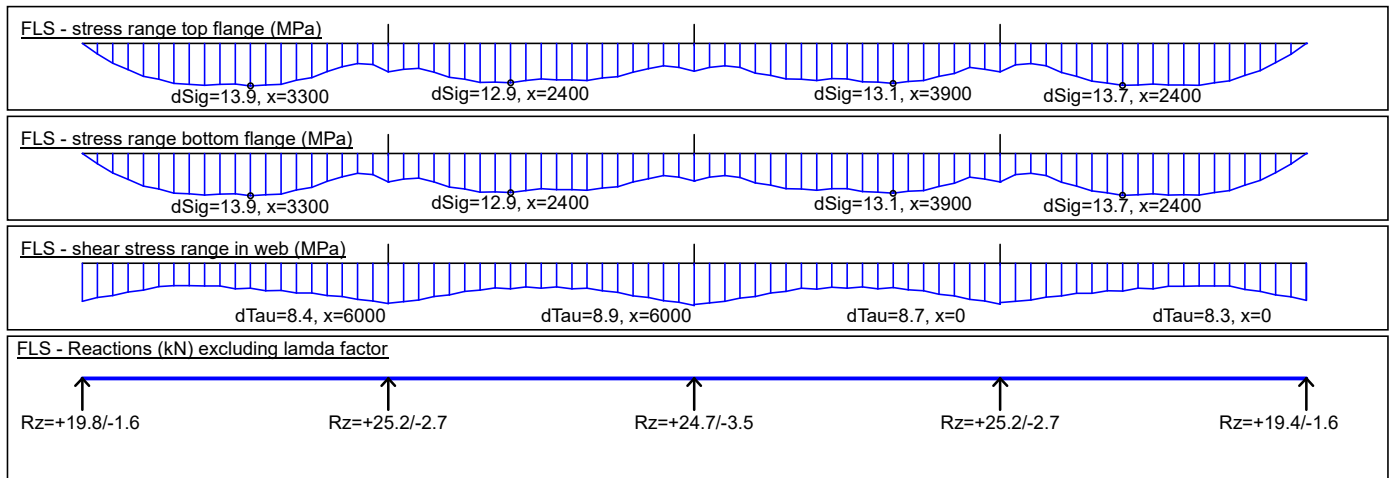
$C_x = 90$
 $C_x = 80$
 $C_z/C_t = 160/100$
 $C_z = 80$

Full Pen Weld
Fillet Weld - a4.0
e = 100, t = 15

Beam Data (mm/kN)**ULS - $M_y/V_z/R_z$ (kN/kNm)****SLS - U_z (mm)****SLS - U_y (mm)**

Project: TestProjekt
Identification: TestTravers

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Results:

ULS

$$IR = V_z, Ed/V_z, Rd = 35.4/399.8$$

$$IR = [M_y, Ed/M_y, Rd]^a + M_z, Ed/M_z, Rd = [30.5/204.9]^2 + 12.0/47.5 =$$

$$IR = M_y, Ed/(x_{LT} \cdot M_y, Rd) + M_z, Ed/M_z, Rd = 30.5/(0.88 \cdot 204.9) + 12.0/47.5 =$$

$$IR = F_z, Ed/F_z, Rd = 19.5/438.5 =$$

SLS - Deflection Control

$$IR = dz/(L/600) = 4.0/(6000/600) =$$

$$IR = dy/(L/600) = 8.7/(6000/600) =$$

FLS - Fatigue Control

$$1: IR = dSigE2/dSigc = 1.00 \cdot 13.9/(90/1.15) =$$

$$2: IR = dSigE2/dSigc = 1.00 \cdot 9.3/(80/1.15) =$$

$$3: IR = [dSigE2/dSigc]^3 + [dTauE2/dTauc]^5 = [1.00 \cdot 11.2/(160/1.15)]^3 + [1.00 \cdot (8.4+5.9)/(100/1.15)]^5 =$$

$$6: IR = dSigE2/dSigc = 1.00 \cdot 4.4/(80/1.15) =$$

$$0.088 < 1.00 \text{ (1/6000 mm; Ch 6.2.6)}$$

$$0.275 < 1.00 \text{ (1/2400 mm; Ch 6.2.5/6.2.9)}$$

$$0.423 < 1.00 \text{ Beam: 1 (Ch 6.3.3)}$$

$$0.044 < 1.00 \text{ (Ch 5.7.1/EN 1993-6)}$$

$$0.402 < 1.00$$

$$0.869 < 1.00$$

$$0.177 < 1.00 \text{ (1/3300 mm)}$$

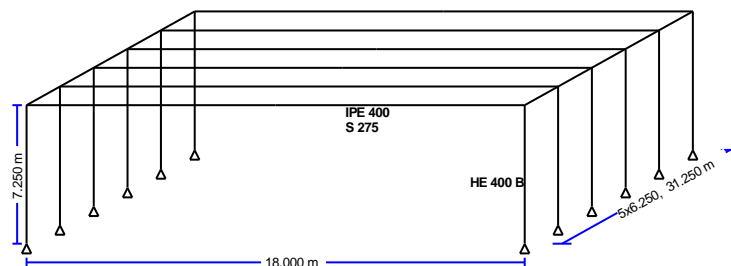
$$0.134 < 1.00 \text{ (Support: 2)}$$

$$0.001 < 1.00 \text{ (1/6000 mm)}$$

$$0.063 < 1.00 \text{ (Support: 3)}$$

PFR 30/07/2022

STOGINĖ NR. 2 (karkasas)

**1. Design codes**

EN1990:2002, Eurocode 0 Basis of Structural Design
 EN1991-1-1:2002, Eurocode 1-1 Actions on structures
 EN1991-1-3:2003, Eurocode 1-3 Snow loads
 EN1991-1-4:2005, Eurocode 1-4 Wind actions
 EN1993-1-1:2005, Eurocode 3 1-1 Design of steel structures
 EN1993-1-3:2005, Eurocode 3 1-3 Cold-formed members
 EN1993-1-5:2006, Eurocode 3 1-5 Plated structural elements
 EN1993-1-8:2005, Eurocode 3 1-8 Design of Joints
 CEN/TS 1992-4-1:2009, Design of fastenings in concrete, General
 CEN/TS 1992-4-2:2009, Design of fastenings, Headed Fasteners
 EN1998-1-1:2004, Eurocode 8 Design in earthquake environment

2. Basic data**2.1. Geometry of frame structure**

Bay width	L = 18.000 m
Total height(max)	H = 7.250 m
Column height	H1 = 7.250 m
Total length	B = 31.250 m (5x6.250m)
Spacing of frames	s = 6.250 m
Roof slope	$\alpha = 0.00^\circ$
Haunch size	L1 = L/10.0 = 1.800 m
Cladding	Sheeting thickness $t_w = 0.100$ mm, Profile depth $h_w = 5.0$ mm
Purlin spacing	= 1.400 m
	Purlin laterally restrained, Simply supported purlin

2.2. Steel sections

Column section	HE 400 B - S 275
Rafter section	IPE 400 - S 275
Purlin section	Z30025 - S 275
Transverse restraint system	L90x90x8 - S 275
Lateral bracing of columns	Lm1 = 6.650 m
Torsional restrains of rafters	Lm2 = 6.000 m
Compression stiffener at the bottom of haunch	

2.3. Steel joints

Type of connection	End-plate connection, non-preloaded bolts
Category of connection	Category A: Bearing type Category D: Non-preloaded
End Plate	Thickness $t_p=25$ mm, S 275
Bolts	M24, Grade 10.9

3. Materials and Code parameters

3.1. Materials

Steel: S 275 (EN1993-1-1, §3.2)

$t \leq 40$ mm, Yield strength $f_y = 275$ N/mm², Ultimate strength $f_u = 430$ N/mm²

$40\text{mm} < t \leq 80$ mm, Yield strength $f_y = 255$ N/mm², Ultimate strength $f_u = 410$ N/mm²

Modulus of elasticity $E = 210000$ N/mm², Poisson ratio $\nu = 0.30$, Unit mass $\rho = 7850$ Kg/m³

Partial factors for materials

(EN1993-1-1, §6.1)

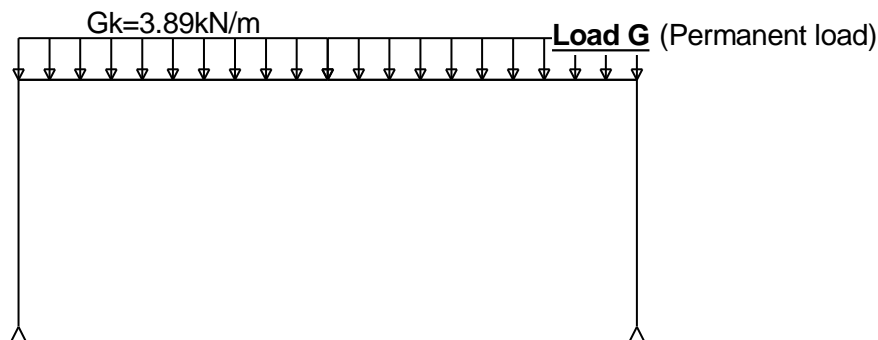
$\gamma_{M0} = 1.00$, $\gamma_{M1} = 1.00$, $\gamma_{M2} = 1.25$

4. Loads

4.1. Permanent loads

(EN1991-1-1)

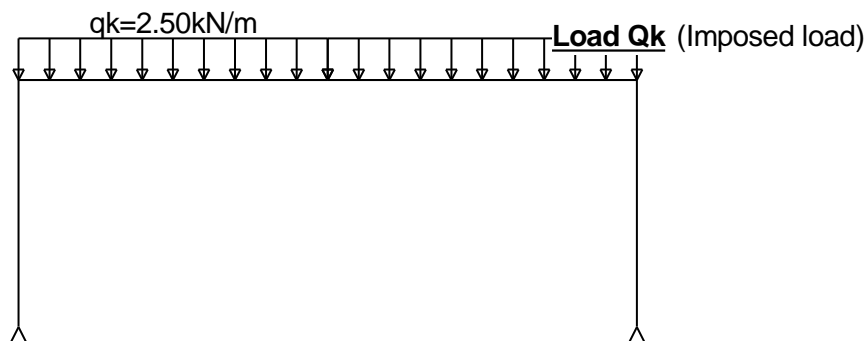
Self weight of purlins and finishing	$g_{k1} = 0.200 + 0.097 / 1.400 = 0.269$ kN/m ²
Self weight of ceiling under the roof	$g_{k2} = 0.250$ kN/m ² $g_k = g_{k1} + g_{k2} = 0.519$ kN/m ²
Spacing of frames	$s = 6.250$ m
Roof load on frame	$(g_{k1} + g_{k2}) \cdot s = 0.519 \times 6.250 = 3.24$ kN/m
Self weight of rafters	$G(\text{IPE } 400) = 0.65$ kN/m
Permanent load on frame	$G_k = 3.24 + 0.65 = 3.89$ kN/m
Self weight of columns	$G(\text{HE } 400 \text{ B}) = 1.52$ kN/m



4.2. Imposed loads

(EC1 EN1991-1-1:2002 Tab.6.10)

Roof slope	$\alpha = 0.00^\circ$
Imposed load (category H)	$q_k = 0.40$ kN/m ²
Roof load on frame	$q_k \cdot s = 0.40 \times 6.250 = 2.50$ kN/m



4.3. Snow load

(EC1 EN1991-1-3:2003)

Snow load on the ground

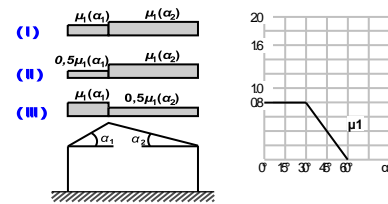
(EN1991-1-3 §4, Annex C)

Characteristic value of snow load on the ground: $s_k = 1.200$ kN/m²

Snow load on the roof

(EC1 EN1991-1-3:2003 §5)

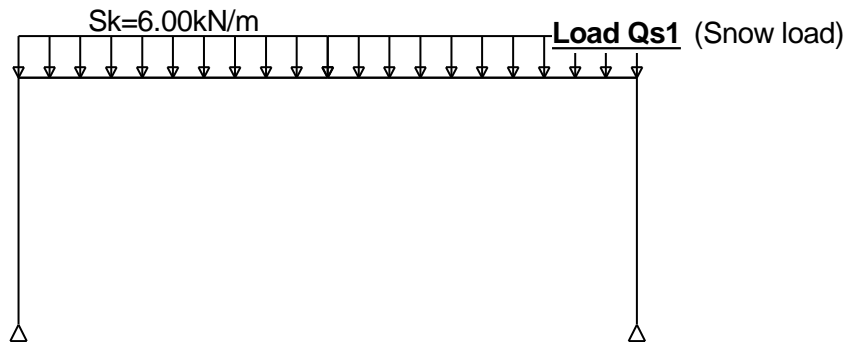
pitched roof (EC1-1-3 §5.3.3))

Angle of pitch of roof : $\alpha_1=0.000^\circ$ Angle of pitch of roof : $\alpha_2=0.000^\circ$ Exposure coefficient : $C_e=1.000$ (EC1-1-3 §5.2(7))Thermal coefficient : $C_t=1.000$ (EC1-1-3 §5.2(8))Shape coefficients $\mu_1(\alpha_1)=\mu_1(\alpha_2)=0.800$ (EC1-1-3 T.5.2)) $S(\alpha_1)=\mu_1(\alpha_1) \cdot C_e \cdot C_t \cdot S_k = 0.800 \times 1.000 \times 1.000 \times 1.200 = 0.960 \text{ kN/m}^2$ $S(\alpha_2)=\mu_1(\alpha_2) \cdot C_e \cdot C_t \cdot S_k = 0.800 \times 1.000 \times 1.000 \times 1.200 = 0.960 \text{ kN/m}^2$ **Snow load**

(EC1 EN1991-1-3:2003, §5.2, §5.3.3)

Load case (I) , $S(\text{Left})=S(\alpha_1) = 0.960 \text{ kN/m}^2$, $S(\text{Right})=S(\alpha_2) = 0.960 \text{ kN/m}^2$ **4.4. Snow load on frame**

(EC1 EN1991-1-3:2003)

Snow load on the ground $s_k = 1.200 \text{ kN/m}^2$ Snow load on the roof $S_k = 0.8 \times 1.200 \times 1.00 \times 1.00 = 0.960 \text{ kN/m}^2$ Spacing of frames $s = 6.250 \text{ m}$ Snow load on frame $S_{k1} = 0.960 \times 6.250 / \cos 0.00^\circ = 6.00 \text{ kN/m}$ $S_{k2} = 0.5 \times 0.960 \times 6.250 / \cos 0.00^\circ = 3.00 \text{ kN/m}$ Load case(I) $S_{k1} = 6.00 \text{ kN/m}$, $S_{k2} = 6.00 \text{ kN/m}$ **4.5. Wind load**

(EC1 EN1991-1-4:2005)

Reference velocity

(EN1991-1-4, §4.2)

 $v_{bo} = 0.00 \text{ m/s}$, Zone: 2 $v_b = C_{dir} \cdot C_{season} \cdot V_{bo} = 24.01 \text{ m/s}$ **Terrain effects**

(EN1991-1-4, §4.3.2, Annex A)

Terrain category : III

(EN1991-1-4, Tab.4.1)

Area with regular cover of vegetation or buildings (villages, suburban terrain, forest)

Roughness factor $C_r(z)$

(EN1991-1-4, §4.3.2)

Terrain category: III, $z = 7.250 \text{ m}$, $z_o = 0.300 \text{ m}$, $z_{min} = 5 \text{ m}$, $z_{max} = 200 \text{ m}$, $z_{oII} = 0.050 \text{ m}$ $kr = 0.19 \cdot (0.300/0.05)^{0.07} = 0.215$ $C_r(z) = kr \cdot \ln(z/z_o) = 0.215 \times \ln(7.250/0.300) = 0.686$ Orography factor $C_o(z)$

(EN1991-1-4, §4.3.3)

 $C_o(z) = 1.000$

(EN1991-1-4, §4.3.3)

Turbulence factor K_t

(EN1991-1-4, §4.4)

 $K_t = 1.000$ Exposure factor $C_e(z)$

(EN1991-1-4, §4.5)

Terrain category: III

(EN1991-1-4, Tab.4.1)

 $z = 7.25 \text{ m}$, $kr = 0.215$, $lv(z) = 0.314$, $C_e(z) = 1.505$ (EC1 EN1991-1-4:2005, eq.A. 4.8,4.7,4.4,4.3) $q(z) = C_e(z) \cdot (\frac{1}{2}\rho) \cdot V_b^2 = [0.001] \times 1.505 \times 0.625 \times 24.01^2 = 0.542 \text{ kN/m}^2$

Wind peak velocity pressure $q(z)=C_e(z) \cdot q_b = C_e(z) \cdot (0.625) \cdot V_b^2$

(EN1991-1-4, §4.5)

$V_b=24.01\text{m/sec}$

$z=7.250\text{m}$

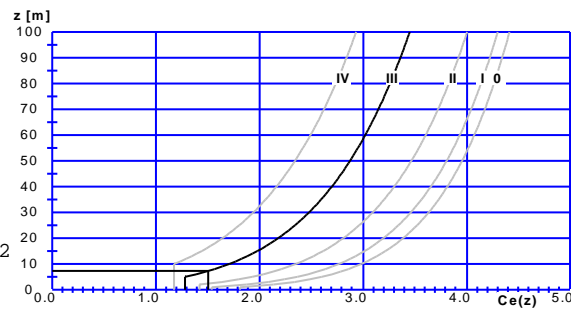
$C_r(z)=0.686$

$C_o(z)=1.000$

$K_t=1.000$

$C_e(z)=1.505$

$$\begin{aligned} q(z) &= C_e(z) \cdot \left(\frac{1}{2}\rho\right) \cdot V_b^2 \\ &= [0.001] \times 1.505 \times 0.625 \times 2 \\ &= 0.542 \text{ kN/m}^2 \end{aligned}$$



Wind forces on flat roof, wind direction: 0.00

(EN1991-1-4, §7.2.3)

Wind pressure coefficients C_{pe}

(EN1991-1-4, Tab. 7.2)

wind direction: $\theta=0.00$

$b=31.25\text{m}$, $d=18.00\text{m}$, $h=7.25\text{m}$, $e=\min(b, 2h)=14.50\text{m}$

$e/4=3.63\text{m}$, $e/10=1.45\text{m}$, $e/2=7.25\text{m}$

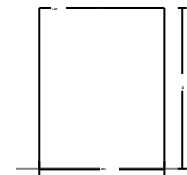
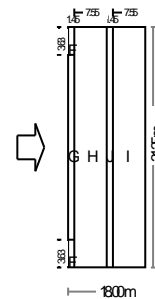
Roof type: Sharp eaves

Zone : F, $A= 5.26\text{m}^2$, $C_{pe,10}=-1.80$, $C_{pe,1}=-2.50$

Zone : G, $A= 34.80\text{m}^2$, $C_{pe,10}=-1.20$, $C_{pe,1}=-2.00$

Zone : H, $A= 181.25\text{m}^2$, $C_{pe,10}=-0.70$, $C_{pe,1}=-1.20$

Zone : I, $A= 335.94\text{m}^2$, $C_{pe,10}=\pm 0.20$, $C_{pe,1}=\pm 0.20$



Wind pressure on roof surfaces $w_e=q(z) \cdot C_{pe}=0.542 \times C_{pe}$ [kN/m²]

(EN1991-1-4, 5.1)

F		G		H		I	
$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$
-0.976	-1.355	-0.650	-1.084	-0.379	-0.650	+0.108	-0.108

Wind forces on vertical walls

(EN1991-1-4, §7.2.2)

Wind pressure coefficients C_{pe}

(EN1991-1-4, Tab.7.1)

$h/d=7.25/18.00=0.403$, $e=14.50\text{m}$

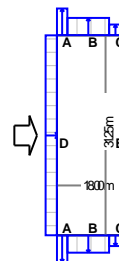
Zone : A, (2.90xh), $C_{pe,10}=-1.20$, $C_{pe,1}=-1.40$

Zone : B, (11.60xh), $C_{pe,10}=-0.80$, $C_{pe,1}=-1.10$

Zone : C, (3.50xh), $C_{pe,10}=-0.50$, $C_{pe,1}=-0.50$

Zone : D, (31.25xh), $C_{pe,10}= 0.80$, $C_{pe,1}= 1.00$

Zone : E, (31.25xh), $C_{pe,10}=-0.34$, $C_{pe,1}=-0.34$



Wind pressure on wall surfaces $w_e=q(z) \cdot C_{pe}$ [kN/m²]

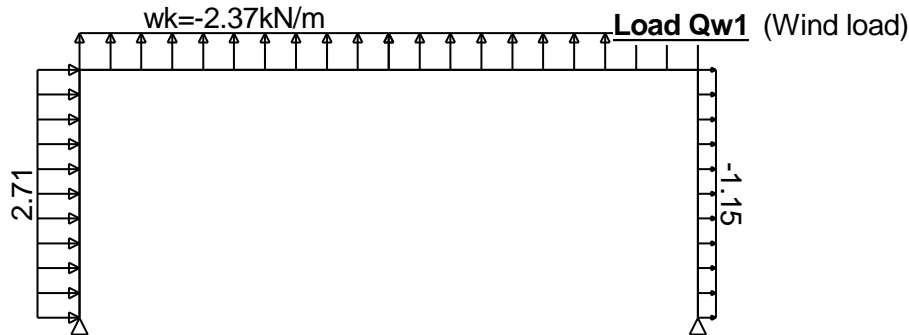
(EN1991-1-4, 5.1)

$z= 7.25 \sim 0.00\text{m}$	A		B		C		D		E	
	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$
	-0.650	-0.759	-0.434	-0.596	-0.271	-0.271	0.434	0.542	-0.185	-0.185

4.6. Wind load on frame

(EC1 EN1991-1-4:2005)

Wind pressure on vertical surface	$w_k = 0.542 \text{ kN/m}^2$
Wind internal pressure	$w_i = 0.000 \text{ kN/m}^2$
Spacing of frames	$s = 6.250 \text{ m}$
Left column	$W_{k1} = 0.434 \times 6.250 = 2.71 \text{ kN/m}$
Left rafter	$W_{k2} = -0.379 \times 6.250 = -2.37 \text{ kN/m}$
Right rafter	$W_{k3} = -0.379 \times 6.250 = -2.37 \text{ kN/m}$
Right column	$W_{k4} = -0.185 \times 6.250 = -1.15 \text{ kN/m}$

**4.7. Seismic loading**

(EC8 EN1998-1-1:2004, §3)

Horizontal acceleration ratio (§3.2.2.2)	$a_{gr}/g = 0.040$	
Verti./horiz. acceleration (§3.2.2.3)	$avg/a_{gr} = 0.90$	
Importance factor (§3.2.1, T.4.3)	$\gamma_i = 1.00$	
Soil factor [horizontal] (§3.2.2.2)	$S = 1.00$	
Behavior factor [horizontal] (§3.2.2.5)	$q = 1.50$	
Behavior factor [vertical]	$q_v = 1.50$	
Spectral shape factor [horizontal] (§3.2.2.5)	$\beta_h(T) = 2.50$	
Spectral shape factor [vertical] (§3.2.2.3)	$\beta_v(T) = 3.00$	
Correction factor (§4.3.3.2.2.1)	$\lambda = 1.00$	
Force distribution $\zeta = z_i W_i / \sum z_j W_j$ (§4.3.3.2.3)	$\zeta = 1.50$	
Fundamental vibration period (§4.3.3.2.2.3)	$(\text{sec}) = 0.81$	
Live load combination factor (EC0 T.A1.1)	$\psi_2 = 0.30$	
Snow load combination factor (EC0 T.A1.1)	$\psi_2 = 0.20$	
Characteristic spectral periods [horizontal]	$T_b = 0.15 \text{ sec}, T_c = 0.50 \text{ sec}, T_d = 2.00 \text{ sec}$	
Characteristic spectral periods [vertical]	$T_b = 0.05 \text{ sec}, T_c = 0.15 \text{ sec}, T_d = 1.00 \text{ sec}$	
$S_d(T_1 = 0.81 \text{ s}) = 0.040 \times 1.00 \times 1.00 \times [(2.50/1.50) \times (0.50/0.813)] = 0.402 \text{ m/s}^2$	(EC8 §3.2.2.5(4), Eq.3.13)	
$S_v(T_1 = 0.81 \text{ s}) = 0.90 \times 0.040 \times 1.00 \times [(3.00/1.50) \times (0.15/0.813)] = 0.130 \text{ m/s}^2$	(EC8 §3.2.2.5(5))	

5. Design values of Actions

(EN1990 NA Latvia LVS, §6.4, §6.5)

5.1. Load combination factors

(EN1990 Tab.A1.1)

Category H (roofs)

 $Q_k \psi_0=0.00, \psi_1=0.00, \psi_2=0.00$

Snow loads on buildings

 $Q_s \psi_0=0.50, \psi_1=0.20, \psi_2=0.00$

Wind loads on buildings

 $Q_w \psi_0=0.60, \psi_1=0.20, \psi_2=0.00$ **5.2. Ultimate Limit State (ULS) (EQU)**

(EN1990 §6.4.3.2, T.A1.2A)

$$E_d = \gamma_G \cdot G_k + \gamma_Q \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10})$$

 $\gamma_{G,\text{sup}}=1.35$ (Unfavorable) $\gamma_{G,\text{inf}}=0.90$ (Favorable) $\gamma_Q = 1.30$ (Unfavorable) $\gamma_Q = 0.00$ (Favorable)**Load combinations (ULS)(EQU),****Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

$$L.C. 101: 1.35G_k + 1.30Q_k \quad (\text{Eq.6.10})$$

$$L.C. 102: 1.35G_k + 1.30Q_{s1} \quad (\text{Eq.6.10})$$

$$L.C. 103: 1.35G_k + 1.30Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 111: 0.90G_k + 1.30Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 121: 1.35G_k + 1.30Q_{s1} + 0.60 \times 1.30Q_{w1} = 1.35xG_k + 1.30Q_{s1} + 0.78Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 122: 1.35G_k + 1.30Q_{w1} + 0.50 \times 1.30Q_{s1} = 1.35xG_k + 1.30Q_{w1} + 0.65Q_{s1} \quad (\text{Eq.6.10})$$

5.3. Ultimate Limit State (ULS) (STR)

(EN1990 §6.4.3.2, T.A1.2B)

$$E_d = \gamma_G \cdot G_k + \gamma_Q \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10})$$

$$E_d = \gamma_G \cdot G_k + \gamma_Q \cdot \psi_0 \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10a})$$

$$E_d = \xi \cdot \gamma_G \cdot G_k + \gamma_Q \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10b})$$

 $\gamma_{G,\text{sup}}=1.35$ (Unfavorable) $\gamma_{G,\text{inf}}=1.00$ (Favorable) $\gamma_Q = 1.50$ (Unfavorable) $\gamma_Q = 0.00$ (Favorable) $\xi=0.850, \xi \cdot \gamma_G=0.850 \times 1.35=1.15$ **Load combinations (ULS)(STR),****Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

$$L.C. 201: 1.35G_k + 1.50Q_k \quad (\text{Eq.6.10})$$

$$L.C. 202: 1.35G_k + 1.50Q_{s1} \quad (\text{Eq.6.10})$$

$$L.C. 203: 1.35G_k + 1.50Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 210: 1.00G_k + 1.50Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 211: 1.35G_k + 1.50Q_{s1} + 0.60 \times 1.50Q_{w1} = 1.35xG_k + 1.50Q_{s1} + 0.90Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 212: 1.35G_k + 1.50Q_{w1} + 0.50 \times 1.50Q_{s1} = 1.35xG_k + 1.50Q_{w1} + 0.75Q_{s1} \quad (\text{Eq.6.10})$$

$$L.C. 231: 1.35G_k + 1.50 \times 0.50Q_{s1} + 1.50 \times 0.60Q_{w1} = 1.35xG_k + 0.75Q_{s1} + 0.90Q_{w1} \quad (\text{Eq.6.10a})$$

$$L.C. 251: 0.850 \times 1.35G_k + 1.50Q_{s1} + 1.50 \times 0.60Q_{w1} = 1.15xG_k + 1.50Q_{s1} + 0.90Q_{w1} \quad (\text{Eq.6.10b})$$

$$L.C. 252: 0.850 \times 1.35G_k + 1.50Q_{w1} + 1.50 \times 0.50Q_{s1} = 1.15xG_k + 1.50Q_{w1} + 0.75Q_{s1} \quad (\text{Eq.6.10b})$$

5.4. Serviceability Limit State (SLS)

(EN1990 §6.5.3, T.A1.4)

$$E_d = G_k + Q_{k1} + \psi_0 \cdot Q_{k2} + \psi_0 \cdot Q_{k3} \quad (\text{Characteristic combination}) \quad (\text{Eq.6.14})$$

$$E_d = G_k + \psi_1 \cdot Q_{k1} + \psi_2 \cdot Q_{k2} + \psi_2 \cdot Q_{k3} \quad (\text{Frequent combination}) \quad (\text{Eq.6.15})$$

$$E_d = G_k + \psi_2 \cdot Q_{k1} + \psi_2 \cdot Q_{k2} + \psi_2 \cdot Q_{k3} \quad (\text{Quasi-permanent combination}) \quad (\text{Eq.6.16})$$

Load combinations (SLS)**Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

L.C. 301: $G_k + Q_k$	(Eq.6.14a)
L.C. 302: $G_k + Q_{s1}$	(Eq.6.14a)
L.C. 303: $G_k + Q_{w1}$	(Eq.6.14a)
L.C. 311: $G_k + Q_{s1} + 0.60Q_{w1}$	(Eq.6.14a)
L.C. 312: $G_k + Q_{w1} + 0.50Q_{s1}$	(Eq.6.14a)
L.C. 331: $G_k + 0.20Q_{s1} + 0.00Q_{w1}$	(Eq.6.15a)
L.C. 332: $G_k + 0.20Q_{w1} + 0.00Q_{s1}$	(Eq.6.15a)
L.C. 351: $G_k + 0.00Q_{s1} + 0.00Q_{w1}$	(Eq.6.16a)

5.5. Ultimate Limit State (ULS) Seismic situation

$$E_d = G_k + A_{ed} + \psi_2 \cdot Q_{k1} + \psi_2 \cdot Q_{k2} + \psi_2 \cdot Q_{k3} \quad (\text{Eq.6.12b})$$

Snow load Q_s , Wind load Q_w , Seismic load A_{ed}

L.C. 601: $G_k + 0.20Q_{s1} + A_{ed}$	(Eq.6.14a)
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5.6. Summary of load combination**Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

1 L.C. 101 (ULS) (EQU)	$1.35G_k + 1.30Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
2 L.C. 102 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 1.30Q_{s1} + 0.00Q_{w1}$
3 L.C. 103 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 0.00Q_{s1} + 1.30Q_{w1}$
4 L.C. 111 (ULS) (EQU)	$0.90G_k + 0.00Q_k + 0.00Q_{s1} + 1.30Q_{w1}$
5 L.C. 121 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 1.30Q_{s1} + 0.78Q_{w1}$
6 L.C. 122 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 0.65Q_{s1} + 1.30Q_{w1}$
7 L.C. 201 (ULS) (STR)	$1.35G_k + 1.50Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
8 L.C. 202 (ULS) (STR)	$1.35G_k + 0.00Q_k + 1.50Q_{s1} + 0.00Q_{w1}$
9 L.C. 203 (ULS) (STR)	$1.35G_k + 0.00Q_k + 0.00Q_{s1} + 1.50Q_{w1}$
10 L.C. 210 (ULS) (STR)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 1.50Q_{w1}$
11 L.C. 211 (ULS) (STR)	$1.35G_k + 0.00Q_k + 1.50Q_{s1} + 0.90Q_{w1}$
12 L.C. 212 (ULS) (STR)	$1.35G_k + 0.00Q_k + 0.75Q_{s1} + 1.50Q_{w1}$
13 L.C. 231 (ULS) (STR)	$1.35G_k + 0.00Q_k + 0.75Q_{s1} + 0.90Q_{w1}$
14 L.C. 251 (ULS) (STR)	$1.15G_k + 0.00Q_k + 1.50Q_{s1} + 0.90Q_{w1}$
15 L.C. 252 (ULS) (STR)	$1.15G_k + 0.00Q_k + 0.75Q_{s1} + 1.50Q_{w1}$
16 L.C. 301 (SLS)	$1.00G_k + 1.00Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
17 L.C. 302 (SLS)	$1.00G_k + 0.00Q_k + 1.00Q_{s1} + 0.00Q_{w1}$
18 L.C. 303 (SLS)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 1.00Q_{w1}$
19 L.C. 311 (SLS)	$1.00G_k + 0.00Q_k + 1.00Q_{s1} + 0.60Q_{w1}$
20 L.C. 312 (SLS)	$1.00G_k + 0.00Q_k + 0.50Q_{s1} + 1.00Q_{w1}$
21 L.C. 331 (SLS)	$1.00G_k + 0.00Q_k + 0.20Q_{s1} + 0.00Q_{w1}$
22 L.C. 332 (SLS)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 0.20Q_{w1}$
23 L.C. 351 (SLS)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
24 L.C. 601 (SEISM)	$1.00G_k + 0.00Q_k + 0.20Q_{s1} + 0.00Q_{w1} + A_{ed}$

6. Steel sections

6.1. Column section

Steel cross-section properties

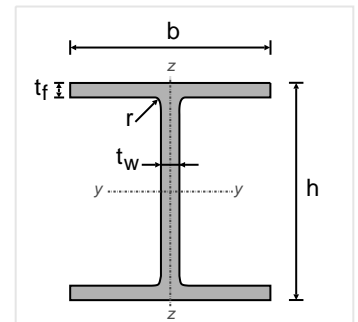
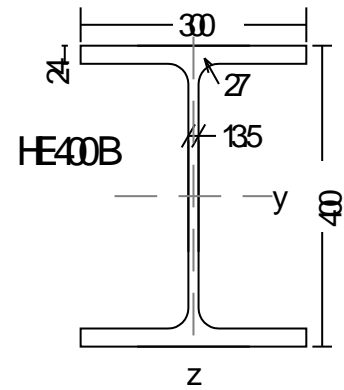
Cross-section HE 400 B-S 275

Dimensions of cross section

Depth of cross section	h=	400.00	mm
Width of cross section	b=	300.00	mm
Web depth	hw=	376.00	mm
Depth of straight portion of web	dw=	298.00	mm
Web thickness	tw=	13.50	mm
Flange thickness	tf=	24.00	mm
Radius of root fillet	r=	27.00	mm
Mass	=	155.00	Kg/m

Properties of cross section

Area	A=	19780	mm ²		
Second moment of area	Iy=	576.80x10 ⁶	mm ⁴	Iz=	108.20x10 ⁶ mm ⁴
Section modulus	Wy=	2884.0x10 ³	mm ³	Wz=	721.30x10 ³ mm ³
Plastic section modulus	Wpy=	3232.0x10 ³	mm ³	Wpz=	1104.0x10 ³ mm ³
Radius of gyration	iy=	170.8	mm	iz=	74.0 mm
Shear area	Avz=	7000	mm ²	Avy=	14400 mm ²
Torsional constant	It=	3.557x10 ⁶	mm ⁴	ip=	186 mm
Torsional modulus	Wt=	148.23x10 ³	mm ³		
Warping constant	Iw=	3817.2x10 ⁹	mm ⁶		



6.2. Rafter section

Steel cross-section properties

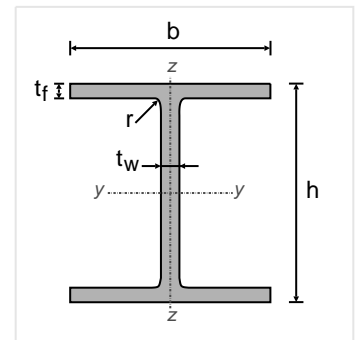
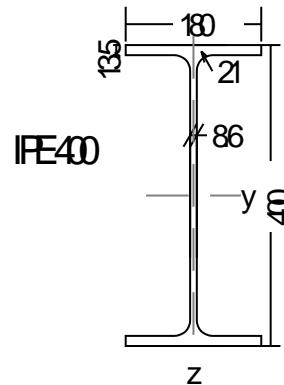
Cross-section IPE 400-S 275

Dimensions of cross section

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

Properties of cross section

Area	A=	8446	mm ²		
Second moment of area	Iy=	231.30x10 ⁶	mm ⁴	Iz=	13.180x10 ⁶ mm ⁴
Section modulus	Wy=	1156.0x10 ³	mm ³	Wz=	146.40x10 ³ mm ³
Plastic section modulus	Wpy=	1307.0x10 ³	mm ³	Wpz=	229.00x10 ³ mm ³
Radius of gyration	iy=	165.5	mm	iz=	39.5 mm
Shear area	Avz=	4269	mm ²	Avy=	4860 mm ²
Torsional constant	It=	0.511x10 ⁶	mm ⁴	ip=	170 mm
Torsional modulus	Wt=	37.834x10 ³	mm ³		
Warping constant	Iw=	490.05x10 ⁹	mm ⁶		



6.3. Haunch section at haunch end

Steel cross-section properties

Welded section

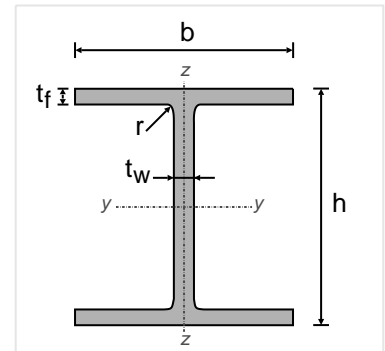
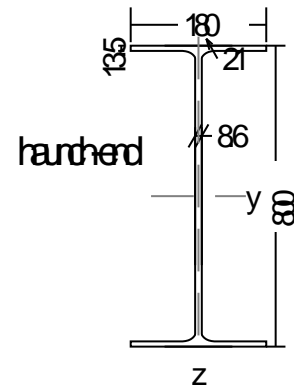
Cross-section haunch-end-S 275

Dimensions of cross section

Depth of cross section	h=	800.00 mm
Width of cross section	b=	180.00 mm
Web depth	hw=	786.50 mm
Depth of straight portion of web	dw=	713.60 mm
Web thickness	tw=	8.60 mm
Flange thickness	tf=	13.50 mm
Radius of root fillet	r=	21.00 mm
Mass	=	90.39 Kg/m

Properties of cross section

Area	A=	11508 mm ²	
Second moment of area	Iy=	1082.7x10 ⁶ mm ⁴	Iz=13.163x10 ⁶ mm ⁴
Section modulus	Wy=	2706.7x10 ³ mm ³	Wz=146.26x10 ³ mm ³
Plastic section modulus	Wpy=	3195.9x10 ³ mm ³	Wpz=232.99x10 ³ mm ³
Radius of gyration	iy=	306.7 mm	iz= 33.8 mm
Shear area	Avz=	6764 mm ²	Avy= 4860 mm ²
Torsional constant	It=	0.454x10 ⁶ mm ⁴	ip= 309 mm
Torsional modulus	Wt=	33.646x10 ³ mm ³	
Warping constant	Iw=	2029.3x10 ⁹ mm ⁶	
Weld	a=	21.0x10 ⁹ mm	



6.4. Haunch section at haunch-middle

Steel cross-section properties

Welded section

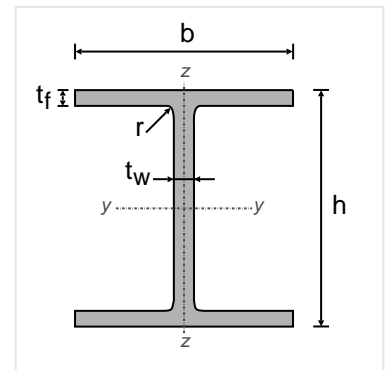
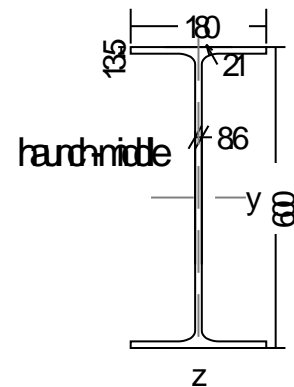
Cross-section haunch-middle-S 275

Dimensions of cross section

Depth of cross section	h=	600.00 mm
Width of cross section	b=	180.00 mm
Web depth	hw=	586.50 mm
Depth of straight portion of web	dw=	513.60 mm
Web thickness	tw=	8.60 mm
Flange thickness	tf=	13.50 mm
Radius of root fillet	r=	21.00 mm
Mass	=	76.88 Kg/m

Properties of cross section

Area	A=	9788 mm ²	
Second moment of area	Iy=	552.84x10 ⁶ mm ⁴	Iz=13.152x10 ⁶ mm ⁴
Section modulus	Wy=	1842.8x10 ³ mm ³	Wz=146.14x10 ³ mm ³
Plastic section modulus	Wpy=	2131.1x10 ³ mm ³	Wpz=229.29x10 ³ mm ³
Radius of gyration	iy=	237.7 mm	iz= 36.7 mm
Shear area	Avz=	5044 mm ²	Avy= 4860 mm ²
Torsional constant	It=	0.412x10 ⁶ mm ⁴	ip= 240 mm
Torsional modulus	Wt=	30.505x10 ³ mm ³	
Warping constant	Iw=	1128.4x10 ⁹ mm ⁶	
Weld	a=	21.0x10 ⁹ mm	



7. Finite Element Analysis

(EN1993-1-1, §5.1)

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

Linear-elastic analysis is used for the design of static loads.
 The seismic design is based on lateral force method and on dynamic analysis by modal superposition spectrum analysis.

7.1. Data used for elastic analysis**Nodal points**

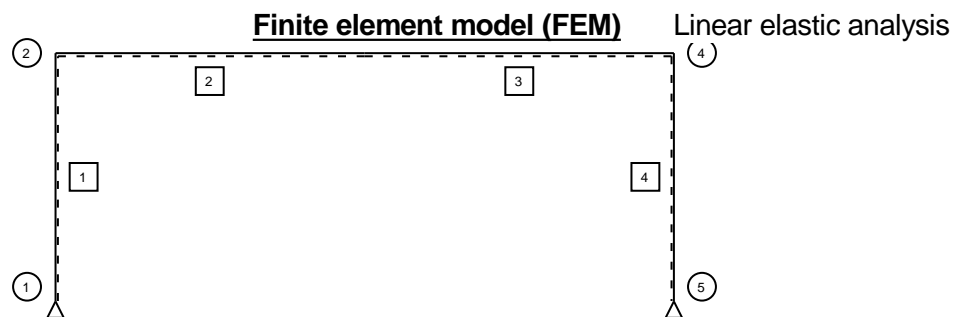
Node	x [mm]	y[mm]
1	0	0
2	0	7250
3	9000	7250
4	18000	7250
5	18000	0

Supports

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

Elements

Element	node 1	node 2	length(mm)	angle(°)	E(GPa)	A(mm ²)	I(mm ⁴)
1	1	2	7250	90.00	210	19780	576800x10 ³
2	2	3	9000	0.00	210	8446	231300x10 ³
3	3	4	9000	0.00	210	8446	231300x10 ³
4	4	5	7250	270.00	210	19780	576800x10 ³



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

L.C.		Left column 1			Left rafter 2			Right rafter 3			Right column 4		
		q	qy	qx	q	qy	qx	q	qy	qx	q	qy	qx
101	ULS-EQU	0.00	2.05	0	0.00	8.50	0	0.00	8.50	0	0.00	2.05	0
102	ULS-EQU	0.00	2.05	0	0.00	13.05	0	0.00	13.05	0	0.00	2.05	0
103	ULS-EQU	3.52	2.05	0	-3.08	5.25	0	-3.08	5.25	0	-1.50	2.05	0
111	ULS-EQU	3.52	1.37	0	-3.08	3.50	0	-3.08	3.50	0	-1.50	1.37	0
121	ULS-EQU	2.11	2.05	0	-1.85	13.05	0	-1.85	13.05	0	-0.90	2.05	0
122	ULS-EQU	3.52	2.05	0	-3.08	9.15	0	-3.08	9.15	0	-1.50	2.05	0
201	ULS-STR	0.00	2.05	0	0.00	9.00	0	0.00	9.00	0	0.00	2.05	0
202	ULS-STR	0.00	2.05	0	0.00	14.25	0	0.00	14.25	0	0.00	2.05	0
203	ULS-STR	4.07	2.05	0	-3.56	5.25	0	-3.56	5.25	0	-1.73	2.05	0
210	ULS-STR	4.07	1.52	0	-3.56	3.89	0	-3.56	3.89	0	-1.73	1.52	0
211	ULS-STR	2.44	2.05	0	-2.13	14.25	0	-2.13	14.25	0	-1.04	2.05	0
212	ULS-STR	4.07	2.05	0	-3.56	9.75	0	-3.56	9.75	0	-1.73	2.05	0
231	ULS-STR	2.44	2.05	0	-2.13	9.75	0	-2.13	9.75	0	-1.04	2.05	0
251	ULS-STR	2.44	1.75	0	-2.13	13.47	0	-2.13	13.47	0	-1.04	1.75	0
252	ULS-STR	4.07	1.75	0	-3.56	8.97	0	-3.56	8.97	0	-1.73	1.75	0
301	SLS	0.00	1.52	0	0.00	6.39	0	0.00	6.39	0	0.00	1.52	0
302	SLS	0.00	1.52	0	0.00	9.89	0	0.00	9.89	0	0.00	1.52	0
303	SLS	2.71	1.52	0	-2.37	3.89	0	-2.37	3.89	0	-1.15	1.52	0
311	SLS	1.63	1.52	0	-1.42	9.89	0	-1.42	9.89	0	-0.69	1.52	0
312	SLS	2.71	1.52	0	-2.37	6.89	0	-2.37	6.89	0	-1.15	1.52	0
331	SLS	0.00	1.52	0	0.00	5.09	0	0.00	5.09	0	0.00	1.52	0
332	SLS	0.54	1.52	0	-0.47	3.89	0	-0.47	3.89	0	-0.23	1.52	0
351	SLS	0.00	1.52	0	0.00	3.89	0	0.00	3.89	0	0.00	1.52	0
601	SEISM	0.00	1.52	0	0.00	5.09	0	0.00	5.09	0	0.00	1.52	0

8. Results of static-linear-elastic analysis**8.1. Displacements [mm]**

L.C.		Hor. defl. Column Dx mm	Vert. defl. Apex Dy mm	Bending Defl. Rafter w mm
101	ULS-EQU	0.150	63.669	10.258
102	ULS-EQU	0.230	97.738	15.748
103	ULS-EQU	32.005	15.695	11.568
111	ULS-EQU	31.974	2.584	9.456
121	ULS-EQU	19.377	83.555	18.887
122	ULS-EQU	32.073	44.897	16.273
201	ULS-STR	0.159	67.413	10.861
202	ULS-STR	0.251	106.723	17.196
203	ULS-STR	36.914	12.058	12.373
210	ULS-STR	36.890	1.861	10.730
211	ULS-STR	22.344	90.358	20.817
212	ULS-STR	36.994	45.753	17.802
231	ULS-STR	22.265	56.663	15.388
251	ULS-STR	22.331	84.530	19.879
252	ULS-STR	36.980	39.926	16.863
301	SLS	0.113	47.856	7.710
302	SLS	0.174	74.063	11.933
303	SLS	24.616	10.953	8.718
311	SLS	14.903	63.152	14.348
312	SLS	24.669	33.416	12.338
331	SLS	0.090	38.122	6.142
332	SLS	4.978	25.500	5.498
351	SLS	0.069	29.137	4.694

8.2. Reactions at the supports

		Horizontal Force Hed [kN], Vertical Force Ved [kN], Moment Med [kNm]					
		Left support 1			Right support 2		
L.C.		Hed,1 kN	Ved,1 kN	Med,1 kNm	Hed,2 kN	Ved,2 kN	Med,2 kNm
101	ULS-EQU	29.5	91.4	0.0	-29.5	91.4	0.0
102	ULS-EQU	45.3	132.3	0.0	-45.3	132.3	0.0
103	ULS-EQU	-14.2	27.1	0.0	-22.2	41.7	0.0
111	ULS-EQU	-20.3	6.3	0.0	-16.1	21.0	0.0
121	ULS-EQU	25.9	111.3	0.0	-47.7	120.1	0.0
122	ULS-EQU	-0.7	62.2	0.0	-35.7	76.8	0.0
201	ULS-STR	31.3	95.9	0.0	-31.3	95.9	0.0
202	ULS-STR	49.5	143.1	0.0	-49.5	143.1	0.0
203	ULS-STR	-19.2	21.7	0.0	-22.8	38.6	0.0
210	ULS-STR	-24.0	5.6	0.0	-18.1	22.5	0.0
211	ULS-STR	27.0	118.9	0.0	-52.2	129.0	0.0
212	ULS-STR	-3.6	62.2	0.0	-38.4	79.1	0.0
231	ULS-STR	11.4	78.4	0.0	-36.6	88.5	0.0
251	ULS-STR	24.3	109.6	0.0	-49.5	119.8	0.0
252	ULS-STR	-6.3	53.0	0.0	-35.7	69.9	0.0
301	SLS	22.2	68.5	0.0	-22.2	68.5	0.0
302	SLS	34.4	100.0	0.0	-34.4	100.0	0.0
303	SLS	-11.5	19.0	0.0	-16.6	30.3	0.0
311	SLS	19.4	83.8	0.0	-36.2	90.6	0.0
312	SLS	-1.0	46.0	0.0	-27.0	57.3	0.0
331	SLS	17.7	56.8	0.0	-17.7	56.8	0.0
332	SLS	8.5	40.6	0.0	-14.1	42.9	0.0
351	SLS	13.5	46.0	0.0	-13.5	46.0	0.0

8.3. Axial forces Ned [kN]

L.C.		Left column 1 Ned,1	Left rafter 2 Ned,2	Right rafter 3 Ned,3	Right column 4 Ned,4
101	ULS-EQU	-84.0	-29.5	-29.5	-84.0
102	ULS-EQU	-124.9	-45.3	-45.3	-124.9
103	ULS-EQU	-19.6	-11.3	-11.3	-34.3
111	ULS-EQU	-1.4	-5.2	-5.2	-16.1
121	ULS-EQU	-103.9	-41.2	-41.2	-112.7
122	ULS-EQU	-54.7	-24.9	-24.9	-69.4
201	ULS-STR	-88.5	-31.3	-31.3	-88.5
202	ULS-STR	-135.7	-49.5	-49.5	-135.7
203	ULS-STR	-14.2	-10.2	-10.2	-31.2
210	ULS-STR	0.0	-5.5	-5.5	-17.0
211	ULS-STR	-111.4	-44.7	-44.7	-121.6
212	ULS-STR	-54.7	-25.9	-25.9	-71.7
231	ULS-STR	-70.9	-29.1	-29.1	-81.1
251	ULS-STR	-103.3	-42.0	-42.0	-113.5
252	ULS-STR	-46.6	-23.2	-23.2	-63.5
301	SLS	-63.0	-22.2	-22.2	-63.0
302	SLS	-94.5	-34.4	-34.4	-94.5
303	SLS	-13.5	-8.2	-8.2	-24.8
311	SLS	-78.3	-31.2	-31.2	-85.1
312	SLS	-40.5	-18.6	-18.6	-51.8
331	SLS	-51.3	-17.7	-17.7	-51.3
332	SLS	-35.1	-12.4	-12.4	-37.4
351	SLS	-40.5	-13.5	-13.5	-40.5

8.4. Shearing forces Ved [kN]

L.C.		Left column 1 VedA,1 VedB,1		Left rafter 2 VedA,2 VedC,2 VedB,2			Right rafter 3 VedA,3 VedC,3 VedB,3			Right column 4 VedA,4 VedB,4	
101	ULS-EQU	-29.5	-29.5	76.5	61.2	0.0	0.0	-61.2	-76.5	29.5	29.5
102	ULS-EQU	-45.3	-45.3	117.5	94.0	0.0	0.0	-94.0	-117.5	45.3	45.3
103	ULS-EQU	14.2	-11.3	12.2	8.3	-7.3	-7.3	-23.0	-26.9	11.3	22.2
111	ULS-EQU	20.3	-5.2	-3.6	-4.3	-7.3	-7.3	-10.3	-11.1	5.2	16.1
121	ULS-EQU	-25.9	-41.2	96.4	76.3	-4.4	-4.4	-85.1	-105.2	41.2	47.7
122	ULS-EQU	0.7	-24.9	47.3	36.4	-7.3	-7.3	-51.0	-62.0	24.9	35.7
201	ULS-STR	-31.3	-31.3	81.0	64.8	0.0	0.0	-64.8	-81.0	31.3	31.3
202	ULS-STR	-49.5	-49.5	128.3	102.6	0.0	0.0	-102.6	-128.3	49.5	49.5
203	ULS-STR	19.2	-10.2	6.8	3.7	-8.5	-8.5	-20.7	-23.7	10.2	22.8
210	ULS-STR	24.0	-5.5	-5.5	-6.1	-8.5	-8.5	-10.9	-11.5	5.5	18.1
211	ULS-STR	-27.0	-44.7	104.0	82.2	-5.1	-5.1	-92.3	-114.1	44.7	52.2
212	ULS-STR	3.6	-25.9	47.3	36.1	-8.5	-8.5	-53.1	-64.2	25.9	38.4
231	ULS-STR	-11.4	-29.1	63.5	49.8	-5.1	-5.1	-59.9	-73.6	29.1	36.6
251	ULS-STR	-24.3	-42.0	97.0	76.6	-5.1	-5.1	-86.7	-107.1	42.0	49.5
252	ULS-STR	6.3	-23.2	40.3	30.5	-8.5	-8.5	-47.5	-57.2	23.2	35.7
301	SLS	-22.2	-22.2	57.5	46.0	0.0	0.0	-46.0	-57.5	22.2	22.2
302	SLS	-34.4	-34.4	89.0	71.2	0.0	0.0	-71.2	-89.0	34.4	34.4
303	SLS	11.5	-8.2	8.0	5.3	-5.6	-5.6	-16.6	-19.3	8.2	16.6
311	SLS	-19.4	-31.2	72.8	57.6	-3.4	-3.4	-64.3	-79.6	31.2	36.2
312	SLS	1.0	-18.6	35.0	26.9	-5.6	-5.6	-38.2	-46.3	18.6	27.0
331	SLS	-17.7	-17.7	45.8	36.6	0.0	0.0	-36.6	-45.8	17.7	17.7
332	SLS	-8.5	-12.4	29.6	23.5	-1.1	-1.1	-25.7	-31.9	12.4	14.1
351	SLS	-13.5	-13.5	35.0	28.0	0.0	0.0	-28.0	-35.0	13.5	13.5

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

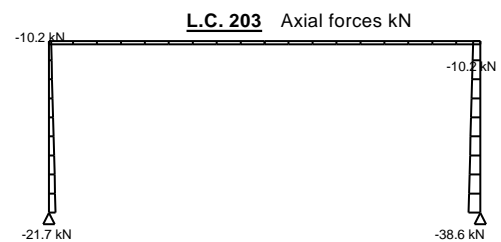
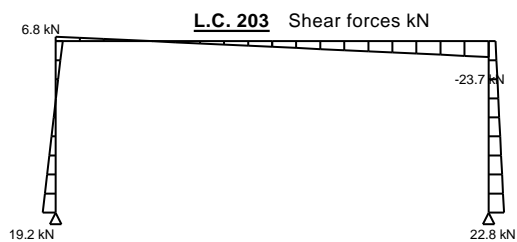
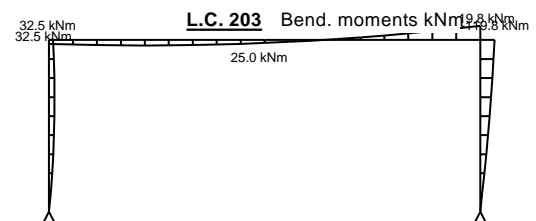
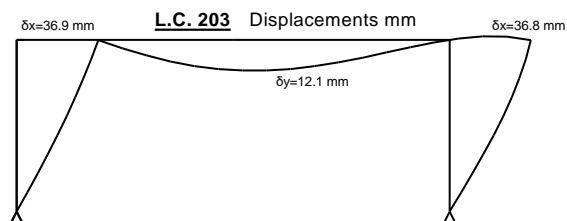
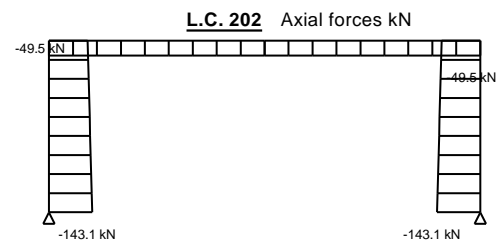
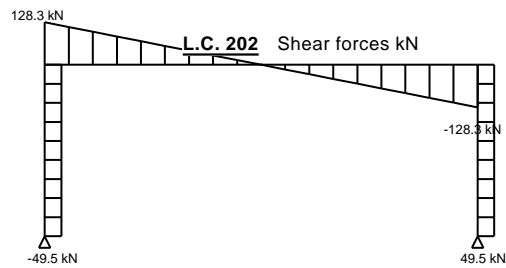
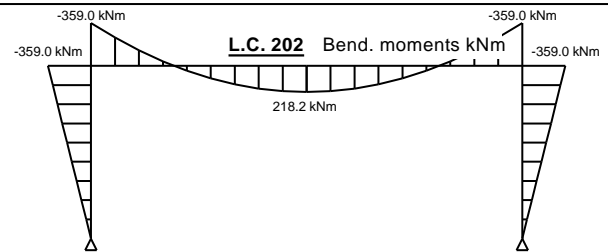
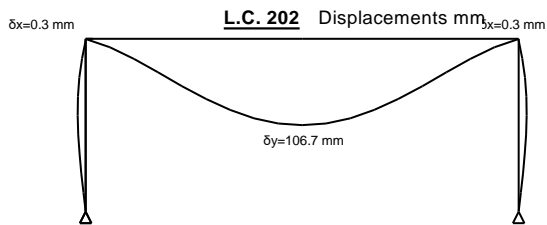
8.5. Bending moments Med [kNm]

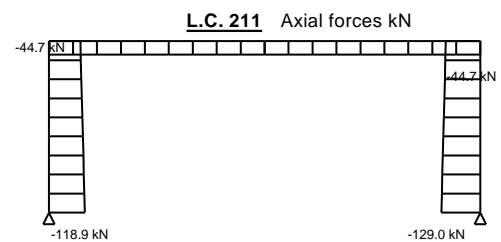
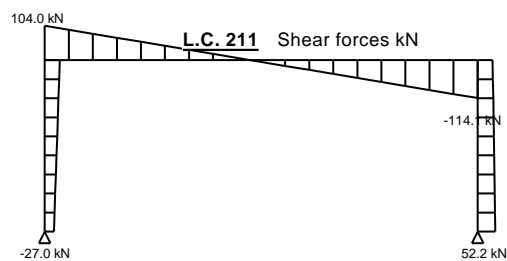
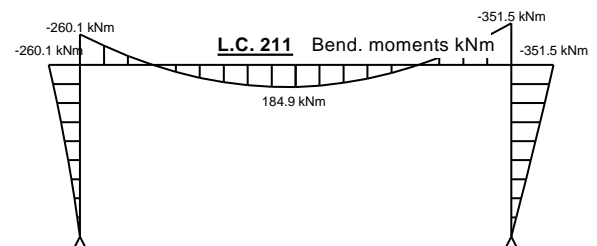
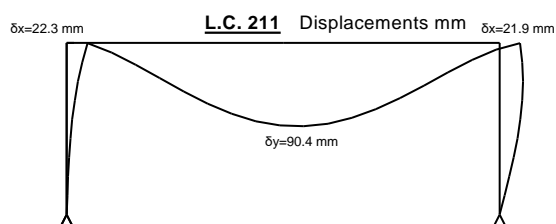
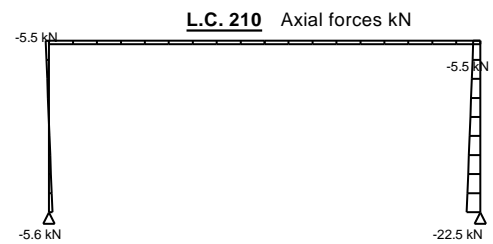
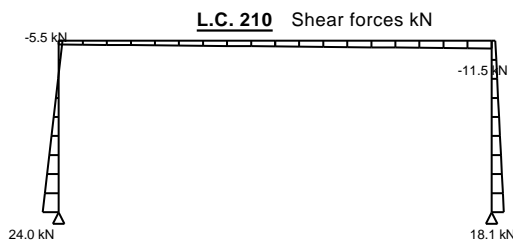
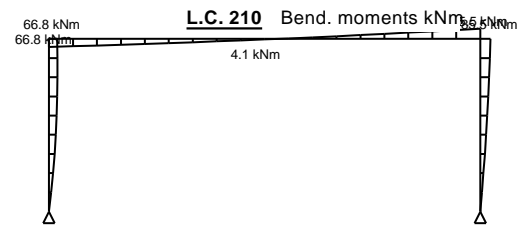
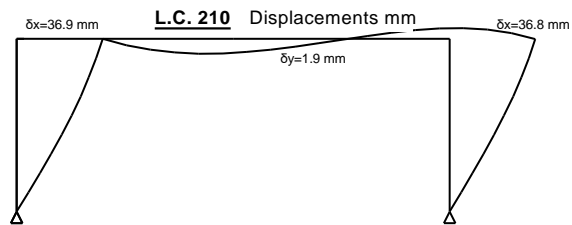
L.C.		Left column 1			Right column 4		
		MedA,1	MedM,1	MedB,1	MedA,4	MedM,4	MedB,4
101	ULS-EQU	0.0	-107.1	-214.1	-214.1	-107.1	0.0
102	ULS-EQU	0.0	-164.4	-328.8	-328.8	-164.4	0.0
103	ULS-EQU	0.0	28.7	10.6	-121.5	-60.7	0.0
111	ULS-EQU	0.0	58.5	54.6	-77.4	-38.7	0.0
121	ULS-EQU	0.0	-121.5	-243.1	-322.3	-161.1	0.0
122	ULS-EQU	0.0	0.1	-87.7	-219.7	-109.9	0.0
201	ULS-STR	0.0	-113.4	-226.7	-226.7	-113.4	0.0
202	ULS-STR	0.0	-179.5	-359.0	-359.0	-179.5	0.0
203	ULS-STR	0.0	45.4	32.5	-119.8	-59.9	0.0
210	ULS-STR	0.0	70.6	66.8	-85.5	-42.8	0.0
211	ULS-STR	0.0	-130.1	-260.1	-351.5	-175.8	0.0
212	ULS-STR	0.0	1.6	-80.8	-233.2	-116.6	0.0
231	ULS-STR	0.0	-73.4	-146.7	-238.2	-119.1	0.0
251	ULS-STR	0.0	-120.3	-240.5	-331.9	-166.0	0.0
252	ULS-STR	0.0	4.9	-61.2	-213.6	-106.8	0.0
301	SLS	0.0	-80.5	-161.0	-161.0	-80.5	0.0
302	SLS	0.0	-124.6	-249.1	-249.1	-124.6	0.0
303	SLS	0.0	24.2	11.9	-89.7	-44.8	0.0
311	SLS	0.0	-91.6	-183.2	-244.1	-122.1	0.0
312	SLS	0.0	0.2	-63.7	-165.2	-82.6	0.0
331	SLS	0.0	-64.1	-128.2	-128.2	-64.1	0.0
332	SLS	0.0	-38.0	-76.0	-96.3	-48.2	0.0
351	SLS	0.0	-49.0	-98.0	-98.0	-49.0	0.0

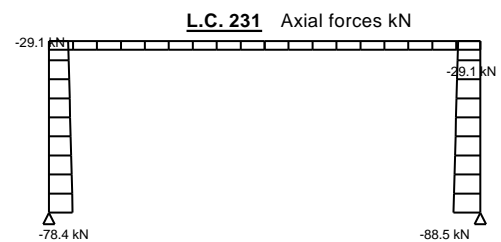
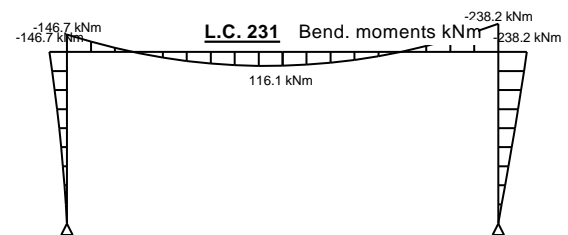
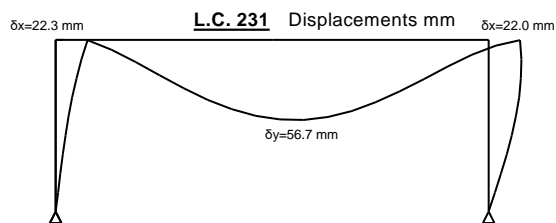
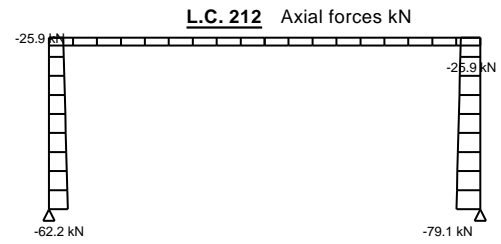
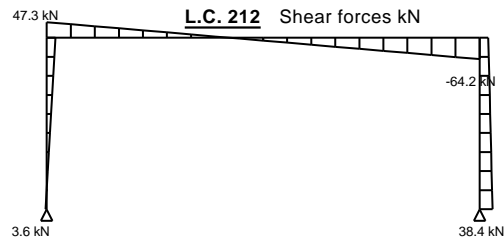
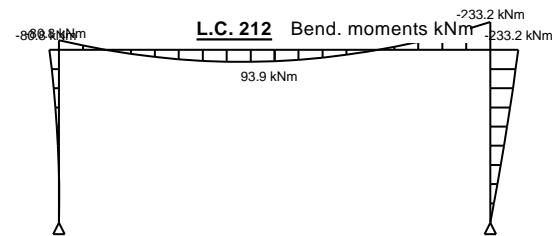
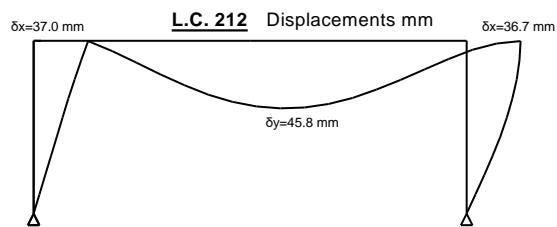
A:left end, C:haunch end, M: span, B: right end

L.C.		Left rafter 2				Right rafter 3			
		MedA,2	MedC2	MedM,2	MedB,2	MedA,3	MedM,3	MedC3	MedB,3
101	ULS-EQU	-214.1	-90.2	-42.0	130.2	130.2	130.2	-90.2	-214.1
102	ULS-EQU	-328.8	-138.5	199.8	199.8	199.8	199.8	-138.5	-328.8
103	ULS-EQU	10.6	29.0	44.8	32.4	32.4	-44.6	-76.7	-121.5
111	ULS-EQU	54.6	47.5	30.1	5.6	5.6	-35.9	-58.1	-77.4
121	ULS-EQU	-243.1	-87.7	171.9	171.0	171.0	-75.6	-151.0	-322.3
122	ULS-EQU	-87.7	-12.4	96.5	92.1	92.1	-63.8	-118.0	-219.7
201	ULS-STR	-226.7	-95.5	-44.5	137.8	137.8	137.8	-95.5	-226.7
202	ULS-STR	-359.0	-151.2	-70.4	218.2	218.2	218.2	-151.2	-359.0
203	ULS-STR	32.5	42.0	46.1	25.0	25.0	-47.4	-79.9	-119.8
210	ULS-STR	66.8	56.4	35.5	4.1	4.1	-40.7	-65.4	-85.5
211	ULS-STR	-260.1	-92.6	186.0	184.9	184.9	-83.3	-165.7	-351.5
212	ULS-STR	-80.8	-5.7	99.7	93.9	93.9	-69.6	-127.6	-233.2
231	ULS-STR	-146.7	-44.8	117.7	116.1	116.1	-61.1	-118.0	-238.2
251	ULS-STR	-240.5	-84.3	174.2	173.0	173.0	-79.4	-157.4	-331.9
252	ULS-STR	-61.2	2.5	88.6	82.0	82.0	-65.8	-119.4	-213.6
301	SLS	-161.0	-67.8	-31.6	97.8	97.8	97.8	-67.8	-161.0
302	SLS	-249.1	-104.9	151.4	151.4	151.4	151.4	-104.9	-249.1
303	SLS	11.9	23.9	33.1	22.6	22.6	-33.5	-57.4	-89.7
311	SLS	-183.2	-65.8	129.9	129.3	129.3	-57.4	-114.6	-244.1
312	SLS	-63.7	-8.0	72.1	68.5	68.5	-48.3	-89.2	-165.2
331	SLS	-128.2	-54.0	77.9	77.9	77.9	77.9	-54.0	-128.2
332	SLS	-76.0	-28.2	52.4	52.2	52.2	-22.1	-44.5	-96.3
351	SLS	-98.0	-41.3	-19.2	59.6	59.6	59.6	-41.3	-98.0

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







9. Results of dynamic analysis

9.1. Eigenfrequencies and Eigenperiods of the structure

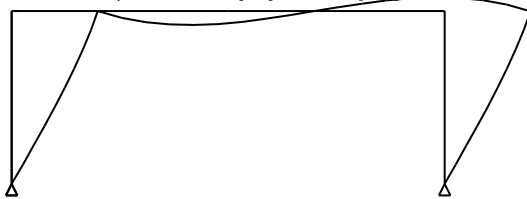
Mass of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$

Total vertical load of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$, $G=113.7$ kN

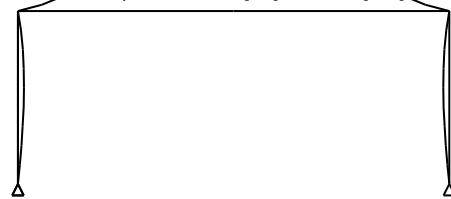
Mass of building: $m=113.660 \times 1000 / 9.81 = 11.59 \times 10^3$ kg

1	f=	1.230 Hz	T=	0.813 sec
2	f=	3.203 Hz	T=	0.312 sec
3	f=	10.184 Hz	T=	0.098 sec
4	f=	21.273 Hz	T=	0.047 sec
5	f=	30.560 Hz	T=	0.033 sec
6	f=	52.032 Hz	T=	0.019 sec
7	f=	61.248 Hz	T=	0.016 sec
8	f=	65.695 Hz	T=	0.015 sec
9	f=	104.240 Hz	T=	0.010 sec
10	f=	142.826 Hz	T=	0.007 sec
11	f=	149.742 Hz	T=	0.007 sec

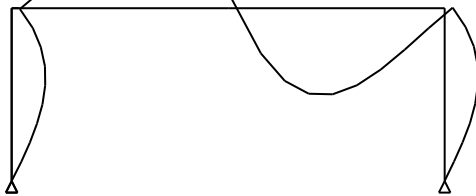
Mode shape :1, f=1.230[Hz], T=0.813[sec]



Mode shape :2, f=3.203[Hz], T=0.312[sec]



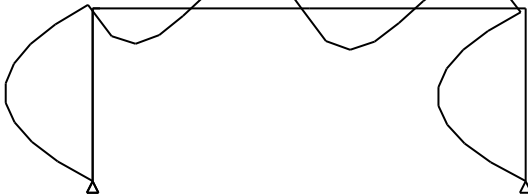
Mode shape :3, f=10.184[Hz], T=0.098[sec]



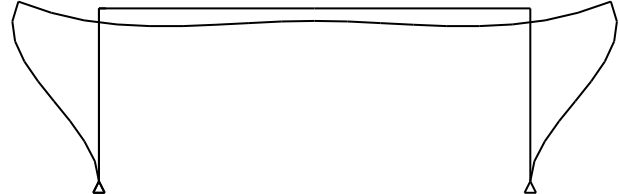
Mode shape :4, f=21.273[Hz], T=0.047[sec]



Mode shape :5, f=30.560[Hz], T=0.033[sec]



Mode shape :6, f=52.032[Hz], T=0.019[sec]



9.2. Seismic action, Lateral Force Method

(EN1998-1-1, §4.3.3.2)

Determination of Base Shear Force

(EN1998-1-1, §4.3.3.2.2)

Approximate value fundamental period of vibration T_1

(EC8 §4.3.3.2.2(3), Eq.4.6)

$$T_1 = 0.085 \cdot H^{0.75} = 0.085 \times 7.25^{0.75} = 0.376 \text{ sec}$$

Value of fundamental period of vibration resulting from dynamic analysis $T_1 = 0.813 \text{ sec}$ From Horizontal design spectrum with period of vibration $T_1 = 0.813 \text{ sec}$ we obtain

$$S_d(T_1) = 0.040 \times 1.00 \times 1.00 \times [(2.50/1.50) \times (0.50/0.813)] = 0.041 \cdot g = 0.402 \text{ m/s}^2$$

From Vertical design spectrum with period of vibration $T_1 = 0.813 \text{ sec}$ we obtain

$$S_{ve}(T_1) = 0.90 \times 0.040 \times 1.00 \times [(3.00/1.50) \times (0.15/0.813)] = 0.013 \cdot g = 0.130 \text{ m/s}^2$$

Total vertical load of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$, $G = 113.7 \text{ kN}$ Mass of building: $m = 113.660 \times 1000 / 9.81 = 11.59 \times 10^3 \text{ kg}$ Seismic Base shear force $F_b = S(T_1) \cdot m \cdot \lambda$

(EC8 §4.3.3.2.2, Eq.4.5)

$$S(T_1) = 0.402 \text{ m/s}^2, m = 11.59 \times 10^3 \text{ kg}, \lambda = 1.00, F_b = 0.402 \times 11.59 \times 1.00 = 4.7 \text{ kN}$$

Amplification factor for torsional effects

(EC8 §4.3.3.2.3, Eq.4.12)

$$\delta = 1 + 0.60 \cdot x / L_c = 1 + 0.60 \times 15625 / 31250 = 1.30$$

Seismic Base shear force $F_b = 1.30 \times 4.7 = 6.1 \text{ kN}$ Horizontal seismic force $F_b = 6.06 \text{ kN}$ is applied at lever $H = 7.250 \text{ m}$ Vertical seismic force $F_v = 0.130 \times 11.59 = 1.5 \text{ kN}$ **Displacements and internal forces M,V,N from linear elastic analysis****Displacements [mm]**

L.C.	Hor. defl. Column Dx mm	Vert. defl. Apex Dy mm	Bending Defl. Rafter w mm
602	9.939	38.131	6.142

Reactions at the supports**Horizontal Force Hed [kN], Vertical Force Ved [kN], Moment Med [kNm]**

L.C.	Left support 1			Right support 2		
	Hed,1 kN	Ved,1 kN	Med,1 kNm	Hed,2 kN	Ved,2 kN	Med,2 kNm
602	14.7	54.4	0.0	-20.7	59.3	0.0

Axial forces Ned [kN]

L.C.	Left column 1 Ned,1	Left rafter 2 Ned,2	Right rafter 3 Ned,3	Right column 4 Ned,4
602	-48.9	-20.7	-20.7	-53.8

Shearing forces Ved [kN]

L.C.	Left column 1		Left rafter 2			Right rafter 3			Right column 4	
	VedA,1	VedB,1	VedA,2	VedC,2	VedB,2	VedA,3	VedC,3	VedB,3	VedA,4	VedB,4
602	-14.7	-14.7	43.4	0.0	-2.4	-2.4	0.0	-48.3	20.7	20.7

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

Bending moments Med [kNm]

L.C.	Left column 1			Right column 4		
	MedA,1	MedM,1	MedB,1	MedA,4	MedM,4	MedB,4
602	0.0	-53.1	-106.2	-150.2	-75.1	0.0

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

L.C.	Left rafter 2			Right rafter 3		
	MedA,2	MedC,2	MedM,2	MedA,3	MedM,3	MedB,3
602	-106.2		77.9	77.9		-150.2

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

Maximum internal forces, Lateral Force Method

(EC8 §4.3.3.2.2)

Columns

NedA	=	-59.3kN
NedB	=	-48.3kN
VedA	=	20.7kN
VedB	=	20.7kN
MedA	=	0.0kNm
MedB	=	150.2kNm
Nedmax	=	-59.3kN
Vedmax	=	20.7kN
Medmax	=	150.2kNm

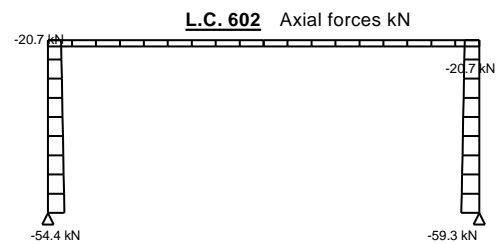
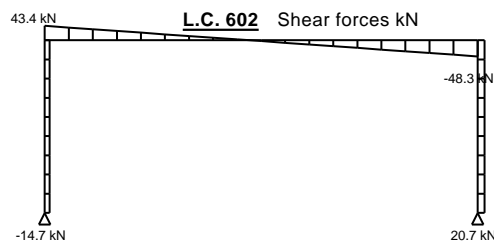
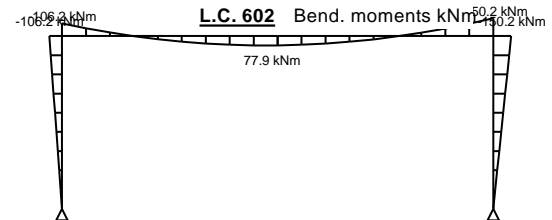
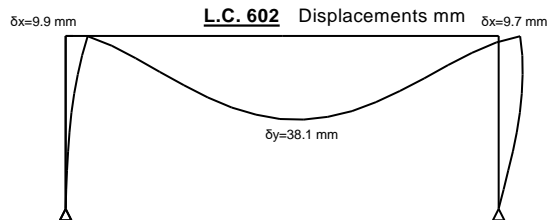
Horizontal deflection at the top of column $dx = 9.9 \text{ mm}$

Rafters

NedA	=	-20.7kN
NedB	=	-20.7kN
VedA	=	48.3kN
VedB	=	2.4kN
MedA	=	150.2kNm
MedB	=	77.9kNm
Nedmax	=	-20.7kN
Vedmax	=	48.3kN
Medmax	=	150.2kNm

Estimate of seismic forces at supports

max Downwards support force $F_{v-} = -59.3 - 1.5/2 = -60.0$ kN
max Upwards support force $F_{v+} = -54.4 + 1.5/2 = 0.0$ kN

**9.3. Seismic action, Modal Response Spectrum Analysis**

(EN1998-1-1, §4.3.3.3)

Effective modal masses of the structure

(EN1998-1-1, §4.3.3.3.1 (3))

From eigenvalue analysis we have the structure eigenperiods and eigenvectors
From response spectrum we obtain the modal spectral acceleration values

	T sec	Sd(T)/g	Sd(T)m/s ²	Sve(T)/g	Sve(T)m/s ²
1	0.813	0.041	0.402	0.013	0.130
2	0.312	0.067	0.654	0.035	0.339
3	0.098	0.053	0.518	0.072	0.706
4	0.047	0.039	0.385	0.069	0.678
5	0.033	0.035	0.347	0.055	0.544
6	0.019	0.032	0.312	0.042	0.416
7	0.016	0.031	0.304	0.040	0.389
8	0.015	0.031	0.301	0.039	0.379

Sd(T):horizontal elastic spectrum, Sve(T):vertical elastic spectrum

Modal masses $M_i = \Phi_i^T \cdot m \cdot \Phi_i$
 Modal excitations $L_i = \Phi_i^T \cdot m \cdot I_i$
 Effective modal masses $m_i = L_i^2 / M_i$
 Modal amplitudes $q_i = (L_i / M_i) \cdot S_d(T) \cdot T^2 / 4\pi^2$
 Modal displacement vectors $u_i = \delta \cdot q_i \cdot \Phi_i$
 Modal internal forces $E_i = K_e \cdot u_i$
 Maximum internal forces $E_e = \sqrt{[\sum E_i^2]}$
 Amplification factor for torsional effects (EC8 §4.3.3.2.3, Eq.4.12)
 $\delta = 1 + 0.60 \cdot x / L_c = 1 + 0.60 \times 15625 / 31250 = 1.30$

	T sec	Mi kg	Li kg	mi kg	qi m
1	0.813	1.000×10^3	3.326×10^3	3.326×10^3 (0.29xMtot)	0.02911
2	0.312	1.000×10^3	2.405×10^3	2.405×10^3 (0.21xMtot)	0.00505
3	0.098	1.000×10^3	0.660×10^3	0.660×10^3 (0.06xMtot)	0.00011
4	0.047	1.000×10^3	-1.450×10^3	1.450×10^3 (0.13xMtot)	-0.00004
5	0.033	1.000×10^3	1.324×10^3	1.324×10^3 (0.11xMtot)	0.00002
6	0.019	1.000×10^3	0.468×10^3	0.468×10^3 (0.04xMtot)	0.00000
7	0.016	1.000×10^3	-0.887×10^3	0.887×10^3 (0.08xMtot)	0.00000
8	0.015	1.000×10^3	-1.297×10^3	1.068×10^3 (0.09xMtot)	0.00000
Mtot=		11.586×10^3 Kg		11.586×10^3 (1.00xMtot)	

Modal displacement vectors

u1	u2	u3	u4	u5	u6	u7	u8
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}
1.446×10^{-3}	0.071×10^{-3}	6.747×10^{-6}	-9.594×10^{-6}	-6.090×10^{-6}	-0.007×10^{-6}	2.201×10^{-6}	0.000×10^{-6}
9.062×10^{-3}	-0.005×10^{-3}	4.603×10^{-6}	1.219×10^{-6}	-0.592×10^{-6}	-0.891×10^{-6}	0.312×10^{-6}	0.000×10^{-6}
0.004×10^{-3}	0.004×10^{-3}	-0.345×10^{-6}	0.917×10^{-6}	0.457×10^{-6}	0.077×10^{-6}	0.272×10^{-6}	0.000×10^{-6}
0.855×10^{-3}	-0.142×10^{-3}	-10.714×10^{-6}	13.580×10^{-6}	5.678×10^{-6}	0.149×10^{-6}	0.301×10^{-6}	0.000×10^{-6}
9.068×10^{-3}	0.000×10^{-3}	4.834×10^{-6}	0.000×10^{-6}	-0.957×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}
0.000×10^{-3}	2.807×10^{-3}	0.000×10^{-6}	-8.842×10^{-6}	0.000×10^{-6}	-0.136×10^{-6}	-0.317×10^{-6}	0.000×10^{-6}
-0.423×10^{-3}	0.000×10^{-3}	30.907×10^{-6}	0.000×10^{-6}	5.858×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}
9.062×10^{-3}	0.005×10^{-3}	4.603×10^{-6}	-1.219×10^{-6}	-0.592×10^{-6}	0.891×10^{-6}	-0.312×10^{-6}	0.000×10^{-6}
-0.004×10^{-3}	0.004×10^{-3}	0.345×10^{-6}	0.917×10^{-6}	-0.457×10^{-6}	0.077×10^{-6}	0.272×10^{-6}	0.000×10^{-6}
0.855×10^{-3}	0.142×10^{-3}	-10.714×10^{-6}	-13.580×10^{-6}	5.678×10^{-6}	-0.149×10^{-6}	-0.301×10^{-6}	0.000×10^{-6}
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}
1.446×10^{-3}	-0.071×10^{-3}	6.747×10^{-6}	9.594×10^{-6}	-6.090×10^{-6}	0.007×10^{-6}	-2.201×10^{-6}	0.000×10^{-6}

Modal internal forces

	0.813s	0.312s	0.098s	0.047s	0.033s	0.019s	0.016s	0.015s
Columns								
NA kN	-2.4	-2.5	0.2	-0.5	-0.3	0.0	-0.2	-0.3
NB kN	-2.4	-2.5	0.2	-0.5	-0.3	0.0	-0.2	-0.3
VA kN	2.8	1.0	0.1	-0.1	0.0	0.0	0.0	0.0
VB kN	-2.8	-1.0	-0.1	0.1	0.0	0.0	0.0	0.0
MA kNm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
MB kNm	-19.9	-7.1	-0.6	0.6	0.2	0.0	0.1	0.1
Rafters								
NA kN	-1.3	-1.0	0.0	0.2	0.1	-0.2	0.1	0.0
NB kN	-1.3	-1.0	0.0	0.2	0.1	-0.2	0.1	0.0
VA kN	-2.2	-1.9	-0.1	-0.1	0.0	0.0	0.0	0.0
VB kN	2.2	1.9	0.1	0.1	0.0	0.0	0.0	0.0
MA kNm	20.3	8.3	0.0	0.4	0.2	0.0	0.0	0.0
MB kNm	-0.2	8.8	0.6	0.1	0.2	0.0	0.0	0.0

Maximum internal forces, Modal Response Spectrum Analysis

(EC8 §4.3.3.3.2 (2))

Column										
NedA =	$-56.8 - \sqrt{[2.4^2 + 2.5^2 + 0.2^2 + 0.5^2 + 0.3^2 + 0.0^2 + 0.2^2 + 0.3^2]}$									-60.3 kN
NedB =	$-45.8 - \sqrt{[2.4^2 + 2.5^2 + 0.2^2 + 0.5^2 + 0.3^2 + 0.0^2 + 0.2^2 + 0.3^2]}$									-49.3 kN
VedA =	$-17.7 - \sqrt{[2.8^2 + 1.0^2 + 0.1^2 + 0.1^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2]}$									20.6 kN
VedB =	$-17.7 - \sqrt{[2.8^2 + 1.0^2 + 0.1^2 + 0.1^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2]}$									20.6 kN
MedA =	$0.0 + \sqrt{[0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2]}$									0.0 kNm
MedB =	$-128.2 - \sqrt{[19.9^2 + 7.1^2 + 0.6^2 + 0.6^2 + 0.2^2 + 0.0^2 + 0.1^2 + 0.1^2]}$									-149.3 kNm
Nedmax=	-60.3 kN									
Vedmax=	20.6 kN									
Medmax=	149.3 kNm									

Horizontal deflection at the top of column dx=9.2 mm

Rafter

Ned A=	-17.7-√[1.3 ² +	1.0 ² +	0.0 ² +	0.2 ² +	0.1 ² +	0.2 ² +	0.1 ² +	0.0 ²]	=	-19.3 kN
Ned B=	-17.7-√[1.3 ² +	1.0 ² +	0.0 ² +	0.2 ² +	0.1 ² +	0.2 ² +	0.1 ² +	0.0 ²]	=	-19.3 kN
Ved A=	45.8+√[2.2 ² +	1.9 ² +	0.1 ² +	0.1 ² +	0.0 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	48.7 kN
Ved B=	0.0-√[2.2 ² +	1.9 ² +	0.1 ² +	0.1 ² +	0.0 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	2.9 kN
Med A=	-128.2-√[20.3 ² +	8.3 ² +	0.0 ² +	0.4 ² +	0.2 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	-150.2 kNm
Med B=	77.9+√[0.2 ² +	8.8 ² +	0.6 ² +	0.1 ² +	0.2 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	86.7 kNm

Nedmax= -19.3 kN
Vedmax= 48.7 kN
Medmax= 150.2 kNm

9.4. Design for seismic loading

(EN1998-1-1, §6)

Maximum design values for Deflections, internal forces and moments

Columns

NedA = -60.3kN
NedB = -49.3kN
VedA = 20.7kN
VedB = 20.7kN
MedA = 0.0kNm
MedB = 150.2kNm
Nedmax = -60.3kN
Vedmax = 20.7kN
Medmax = 150.2kNm

Horizontal deflection at the top of column dx=9.9 mm

Rafters

NedA = -20.7kN
NedB = -20.7kN
VedA = 48.7kN
VedB = 2.9kN
MedA = 150.2kNm
MedB = 86.7kNm
Nedmax = -20.7kN
Vedmax = 48.7kN
Medmax = 150.2kNm, at haunch-start Medmax = 120.1kNm

9.5. Second Order effects

(EC8 §4.4.2.2(2))

$$\theta = P_{tot} \cdot d_r / (V_{tot} \cdot h) = 113.7 \times 9.9 / (6.1 \times 7250) = 0.026$$

$$\theta = 0.026 \leq 0.20, \quad 1/(1-\theta) = 1/(1-0.026) = 1.026$$

Second Order effects are taken into account by multiplying the seismic actions by a factor $1/(1-\theta) = 1.026$

(EC8 §4.4.2.2(3))

9.6. Maximum forces and bending moments for seismic load

(EC8 §6.6)

Columns

(EC8 §6.6.3(1))

Ned=Ned,g+1.1·γov·Ω·[1/(1-θ)]·Ned,e
Ved=Ved,g+1.1·γov·Ω·[1/(1-θ)]·Ved,e
Med=Med,g+1.1·γov·Ω·[1/(1-θ)]·Med,e

Rafters

(EC8 §6.6.2(2))

Ned=Ned,g+[1/(1-θ)]·Ned,e
Ved=Ved,g+[1/(1-θ)]·Ved,e
Med=Med,g+[1/(1-θ)]·Med,e

$$\gamma_{ov} = 1.25$$

(EC8 §6.2(3))

$$\Omega = M_{pl,rd} / Med = [10^{-6}] \times (1.307 \times 10^6 \times 275 / 1.00) / 120.1 = 2.992$$

(EC8 §6.6.3(1))

Columns

(EC8 §6.6.3(1))

NedA = -56.8+1.10x1.25x2.992x1.026x(56.8- 60.3) = -71.6kN
NedB = -45.8+1.10x1.25x2.992x1.026x(45.8- 49.3) = -60.6kN
VedA = 17.7+1.10x1.25x2.992x1.026x(20.7- 17.7) = 30.5kN
VedB = 17.7+1.10x1.25x2.992x1.026x(20.7- 17.7) = 30.5kN
MedA = 0.0+1.10x1.25x2.992x1.026x(0.0- 0.0) = 0.0kNm
MedB = 128.2+1.10x1.25x2.992x1.026x(150.2- 128.2) = 220.9kNm
Nedmax= -71.6kN
Vedmax= 30.5kN
Medmax= 220.9kNm

Rafters

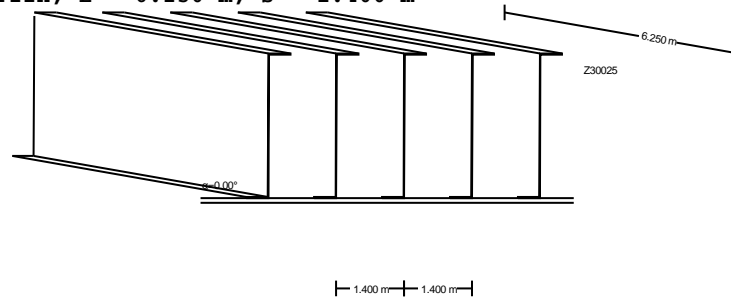
(EC8 §6.6.2(2))

NedA =	-17.7+1.026x(17.7-	20.7)=	-20.8kN
NedB =	-17.7+1.026x(17.7-	20.7)=	-20.8kN
VedA =	45.8+1.026x(48.7-	45.8)=	48.8kN
VedB =	0.0+1.026x(2.9-	0.0)=	3.0kN
MedA =	128.2+1.026x(150.2-	128.2)=	150.8kNm
MedB =	77.9+1.026x(86.7-	77.9)=	87.0kNm
Nedmax=	-20.8kN			
Vedmax=	48.8kN			
Medmax=	150.8kNm			

10. Design of Purlins

Purlin laterally restrained, Z30025 S 275

Simply supported purlin, L= 6.250 m, s= 1.400 m



10.1. Materials, Purlins

Steel: S 275

(EN1993-1-1, §3.2)

$t \leq 40$ mm, Yield strength $f_y = 275$ N/mm², Ultimate strength $f_u = 430$ N/mm²

$40 \text{ mm} < t \leq 80$ mm, Yield strength $f_y = 255$ N/mm², Ultimate strength $f_u = 410$ N/mm²

Modulus of elasticity $E = 210000$ N/mm², Poisson ratio $\nu = 0.30$, Unit mass $\rho = 7850$ Kg/m³

Partial safety factors for actions

(EN1990, Annex A1)

$\gamma_{G, \text{sup}} = 1.35$, $\gamma_Q = 1.50$, $\gamma_{G, \text{inf}} = 1.00$, $\psi_0 = 0.70$

Partial factors for materials

(EN1993-1-1, §6.1)

$\gamma_{M0} = 1.00$, $\gamma_{M1} = 1.00$, $\gamma_{M2} = 1.25$

10.2. Loading, Purlins

(EN1991-1-1)

Roof loads

Roof slope	$\alpha = 0.00^\circ$	
Load of roof covering	$g_{k1} = 0.200$ kN/m ²	(EN1991-1-1 §5)
Imposed load (category H)	$q_k = 0.400$ kN/m ²	(EN1991-1-1 §6.3.4.2)
Snow load	$q_{sk} = 0.960$ kN/m ²	(EN1991-1-3 §5.3)
Wind pressure	$w_k = 0.000$ kN/m ²	(EN1991-1-4 §7.2)
Wind uplift	$w_k = -0.379$ kN/m ²	

Load on purlin

Purlin spacing	$s = 1.400$ m
Load of roof covering	$G_{k1} = 1.400 \times 0.200 = 0.28$ kN/m
Purlin weight	$G_{k2} = 0.10$ kN/m
Permanent load	$G_k = G_{k1} + G_{k2} = 0.28 + 0.10 = 0.38$ kN/m
Imposed load (category H)	$Q_{kk} = 1.400 \times 0.400 = 0.56$ kN/m
Snow load	$Q_{sk} = 1.400 \times 0.960 = 1.34$ kN/m
Wind uplift	$Q_{wk} = -1.400 \times 0.379 = -0.53$ kN/m

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

Permanent load	$G_{k,z} = 0.38 \times \cos(0.00) = 0.38$ kN/m, $G_{k,y} = 0.38 \times \sin(0.00) = 0.00$ kN/m
Imposed load (category H)	$Q_{kk,z} = 0.56 \times \cos(0.00) = 0.56$ kN/m, $Q_{kk,y} = 0.56 \times \sin(0.00) = 0.00$ kN/m
Snow load	$Q_{sk,z} = 1.34 \times \cos(0.00) = 1.34$ kN/m, $Q_{sk,y} = 1.34 \times \sin(0.00) = 0.00$ kN/m
Wind pressure	$Q_{wk,z} = 0.00$ kN/m, $Q_{wk,y} = 0.00$ kN/m
Wind uplift	$Q_{wk,z} = -0.53$ kN/m, $Q_{wk,y} = 0.00$ kN/m

10.3. Design values of Actions, Load combinations, Purlins

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

Sagging $\gamma_G \cdot \sup \cdot G_k, z + \gamma_Q \cdot Q_k, z + \gamma_Q \cdot \psi_0 \cdot Q_{wk}, z = 1.35 \times 0.38 + 1.50 \times 1.34 + 1.50 \times 0.60 \times 0.00 = 2.52 \text{ kN/m}$
 Hogging $\gamma_G \cdot \inf \cdot G_k, z - \gamma_Q \cdot Q_{wk}, z = 1.00 \times 0.38 - 1.50 \times 0.53 = -0.41 \text{ kN/m}$

Serviceability Limit State (SLS), Load combinations

(EN1990 §6.5.3, T.A1.4)

Sagging $G_k, z + Q_k, z + \psi_0 \cdot Q_{wk}, z = 0.38 + 1.34 + 0.60 \times 0.00 = 1.72 \text{ kN/m}$
 Hogging $G_k, z + Q_{wk}, z = 0.38 - 0.53 = -0.15 \text{ kN/m}$

10.4. Design actions, Purlins

Design actions, Ultimate Limit State

Sagging $M_{yed} = 2.52 \times 6.250^2 / 8 = 12.32 \text{ kNm}$, $V_{zed} = 2.52 \times 6.250 / 2 = 7.88 \text{ kN}$
 Hogging $M_{yed} = -0.41 \times 6.250^2 / 8 = -2.03 \text{ kNm}$, $V_{zed} = 0.41 \times 6.250 / 2 = 1.30 \text{ kN}$

Design actions, Serviceability Limit State (SLS)

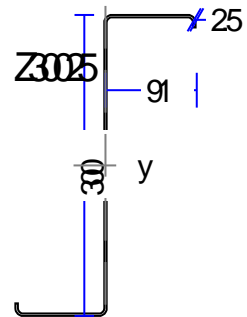
Sagging $M_{yed} = 1.72 \times 6.250^2 / 8 = 8.40 \text{ kNm}$, $V_{zed} = 1.72 \times 6.250 / 2 = 5.38 \text{ kN}$
 Hogging $M_{yed} = -0.15 \times 6.250^2 / 8 = -0.73 \text{ kNm}$, $V_{zed} = 0.15 \times 6.250 / 2 = 0.47 \text{ kN}$

10.5. Steel cross-section properties, Purlins

Cross-section Z30025-S 275

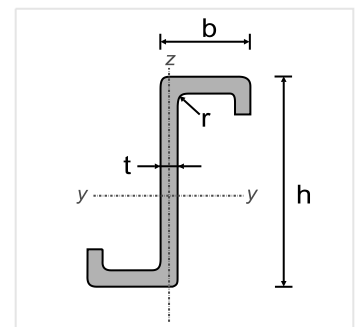
Dimensions of cross section

Depth of cross section $h = 300.00 \text{ mm}$
 Width of cross section $b = 91.00 \text{ mm}$
 Web depth $h_w = 297.50 \text{ mm}$
 Depth of straight portion of web $d_w = 285.00 \text{ mm}$
 Web thickness $t_w = 2.50 \text{ mm}$
 Flange thickness $t_f = 2.50 \text{ mm}$
 Radius of root fillet $r = 5.00 \text{ mm}$
 Mass $= 9.91 \text{ Kg/m}$



Properties of cross section

Area $A = 1265 \text{ mm}^2$
 Second moment of area $I_y = 16.791 \times 10^6 \text{ mm}^4$, $I_z = 0.314 \times 10^6 \text{ mm}^4$
 Section modulus $W_y = 110.07 \times 10^3 \text{ mm}^3$, $W_z = 3.450 \times 10^3 \text{ mm}^3$
 Plastic section modulus $W_{py} = 110.07 \times 10^3 \text{ mm}^3$, $W_{pz} = 3.450 \times 10^3 \text{ mm}^3$
 Radius of gyration $i_y = 115.2 \text{ mm}$, $i_z = 15.8 \text{ mm}$
 Shear area $A_{vz} = 744 \text{ mm}^2$, $A_{vy} = 455 \text{ mm}^2$
 Torsional constant $I_t = 0.003 \times 10^6 \text{ mm}^4$, $i_p = 116 \text{ mm}$
 Torsional modulus $W_t = 1.263 \times 10^3 \text{ mm}^3$
 Warping constant $I_w = 6.947 \times 10^9 \text{ mm}^6$

**10.6. Serviceability Limit State (SLS), Purlins**

(EN1993-1-1, §7)

Purlin deflections, Sagging

Loading $G+Q$: $w = 5 \times 1.72 \times 6250^4 / (384 \times 2.1 \times 10^5 \times 16.791 \times 10^6) = 9.69 \text{ mm} = L/645 < L/200$

Loading Q : $w = 5 \times 1.34 \times 6250^4 / (384 \times 2.1 \times 10^5 \times 16.791 \times 10^6) = 7.55 \text{ mm} = L/828 < L/250$

Purlin deflections, Hogging

Loading $G+Q$: $w = 5 \times -0.15 \times 6250^4 / (384 \times 2.1 \times 10^5 \times 16.791 \times 10^6) = -0.85 \text{ mm} = L/7393 < L/200$

Loading Q : $w = 5 \times -0.53 \times 6250^4 / (384 \times 2.1 \times 10^5 \times 16.791 \times 10^6) = -2.99 \text{ mm} = L/2091 < L/250$

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

10.7. Classification of steel cross-section, Bending My (Purlin section)

(EN1993-1-1, §5.5)

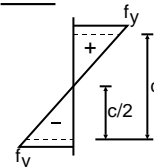
Web

$c = 300.0 - 2 \times 2.5 - 2 \times 5.0 = 285.0 \text{ mm}$, $t = 2.5 \text{ mm}$, $c/t = 285.0 / 2.5 = 114.00$

S 275, $t = 2.5 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$

$83\epsilon = 83 \times 0.92 = 76.36 < c/t = 114.00 \leq 124\epsilon = 124 \times 0.92 = 114.08$

The web is class 3 (EN1993-1-1, Tab.5.2)



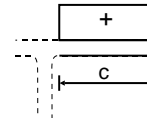
Flange

$c=91.0 \text{ mm}$, $t=2.5 \text{ mm}$, $c/t=91.0/2.5=36.40$

S 275, $t=2.5 \leq 40 \text{ mm}$, $f_y=275 \text{ N/mm}^2$, $\varepsilon=(235/275)^{0.5}=0.92$

$c/t=36.40 > 14\varepsilon=14 \times 0.92=12.88$

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



Overall classification of cross-section is Class 4, Bending $M_{y,ed}$

Effective cross-section properties of Class 4 cross-sections

(EN1993-1-1, §6.2.2.5)

Effective area $A_{eff}=1265 \text{ mm}^2$

Flange

$\bar{\lambda}_p=(b/t)/[28.40\varepsilon\sqrt{(\kappa\sigma)}]$

(EN1993-1-5, §4.4.2, Eq.4.2, Tab1.4.1)

$b=c=91.0 \text{ mm}$, $t=2.5 \text{ mm}$, $\varepsilon=0.92$, $\psi=1.00$, $\kappa\sigma=0.43$, $\bar{\lambda}_p=2.125$

$\bar{\lambda}_p=2.125 > 0.673$ $\rho=[1-0.188/2.125]/2.125=0.429$ ($\rho < 1.0$), $c_{eff}=\rho \cdot c=0.429 \times 91=39.0 \text{ mm}$

Effective area $A_{eff}=0.1 \times (91.0-39.0) \times 2.50=-130 \text{ mm}^2$

$e_{my} \frac{1}{2}(300.0-2.5) \times (0/-130-1)=-148.75 \text{ mm}$, $I_{y,eff}=0.000 \times 10^6 \text{ mm}^4$

Effective section modulus $W_{y,eff}=0.000 \times 10^6 / (300.0/2 + -148.75)=0.000 \times 10^3 \text{ mm}^3$

10.8. Resistance of cross-section, Purlin section

(EN1993-1-1, §6.2)

Effective cross-section properties of Class 4 cross-sections

(EN1993-1-1, §6.2.2.5)

Effective area $A_{eff}=1265 \text{ mm}^2$

$e_{my}=0.00 \text{ mm}$, $I_{y,eff}=16.791 \times 10^6 \text{ mm}^4$

Effective section modulus $W_{y,eff}=16.791 \times 10^6 / (300.0/2 + 0.00)=111.94 \times 10^3 \text{ mm}^3$

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

$M_{y,ed}=12.32 \text{ kNm}$

Bending Resistance $M_{cy,rd}=W_{eff,y} \cdot f_y / \gamma_{M0}=[10^{-6}] \times 111.94 \times 10^3 \times 275 / 1.00=30.78 \text{ kNm}$

$M_{y,ed}=12.32 \text{ kNm} < 30.78 \text{ kNm} = M_{y,rd} = M_{ply,rd}$, Is verified

$M_{y,ed}/M_{y,rd}=12.32/30.78=0.400 < 1$

Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

$V_{z,ed}=7.88 \text{ kN}$

$A_v=h_w \cdot t=297.5 \times 2.5=744 \text{ mm}^2$

Plastic Shear Resistance $V_{pl,z,rd}=A_v(f_y/\sqrt{3})/\gamma_{M0}=[10^{-3}] \times 744 \times (275/1.73)/1.00=118.09 \text{ kN}$

$V_{z,ed}=7.88 \text{ kN} < 118.09 \text{ kN} = V_{z,rd}=V_{pl,z,rd}$, Is verified

$V_{z,ed}/V_{z,rd}=7.88/118.09=0.067 < 1$

$h_w/t_w=(300.0-2 \times 2.5)/2.5=297.5/2.5=119.00 > 72 \times 0.92/1.00=72\varepsilon/\eta=66.24$ ($\eta=1.00$)

S 275, $t=2.5 \leq 40 \text{ mm}$, $f_y=275 \text{ N/mm}^2$, $\varepsilon=(235/275)^{0.5}=0.92$

Shear buckling resistance must be verified (EC3 §6.2.6.6)

Shear buckling resistance

(EC3 EN1993-1-5:2006, §5)

$\bar{\lambda}_w=(285.0/2.5)/(37.4 \times 0.92 \times \sqrt{(5.34)})=1.434$, $K_t=5.34$

(EC3-1-5 §5, Eq.5.6, A.3)

$\bar{\lambda}_w=1.434 > 1.08$, $\chi_v=0.83/1.434=0.579$

(EC3-1-5 Tab.5.1)

$V_{b,rd}=\chi_v \cdot f_{yw} \cdot h_w \cdot t / (\sqrt{3} \gamma_{M1})=0.001 \times 275 \times 0.579 \times 285.0 \times 2.5 / (1.73 \times 1.00)=65.49 \text{ kN}$

(EC3-1-5 Tab.5.1)

$V_{ed}=8 \text{ kN} < 65 = V_{b,rd} \text{ kN}$, Is verified

$V_{ed}/V_{b,rd}=7.88/65.49=0.120 < 1$

Ultimate Limit State, Verification for axial force, shear and bending

(EN1993-1-1, §6.2.9)

$N_{ed}=0.00 \text{ kN}$, $V_{z,ed}=7.88 \text{ kN}$, $M_{y,ed}=12.32 \text{ kNm}$

$M_{y,ed}=12.32 \text{ kNm}$

$M_{c,y,rd}=30.78 \text{ kNm}$, $V_{pl,z,rd}=65.49 \text{ kN}$

$N_{ed}=0 \text{ kN}$, Effect of axial force is neglected

(EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

$V_{ed}=7.88 \text{ kN} \leq 0.50 \times 65.49=0.50 \times V_{pl,rd}=32.74 \text{ kN}$

Effect of shear force is neglected

(EC3 §6.2.8.2)

10.9. Lateral restraining of sheeting, Purlins

(EC3 EN1993-1-3:2005, §10.1)

Sheeting thickness $t_w=0.100$ mm, Profile depth $h_w=5.0$ mm

Shear stiffness of sheeting

(EN1993-1-3, §10.1.1Eq.10.1b)

$$S = t^{1.5} (50 + 10b^{0.33}) s / h_w = 0.100^{1.5} \times (50 + 10 \times 6250^{0.33}) \times 1400 / 5.0 = 2074 \text{ kNm/m}$$

Minimum required shear stiffness, for laterally restrained purlin

(§10.1.1Eq.10.1a)

$$S_{min} = [\pi^2 E \cdot I_w / L^2 + G \cdot I_t + \pi^2 E \cdot I_z (h/2)^2 / L^2] \cdot 70 / h = [\pi^2 \times 2.1 \times 10^5 \times 6.947 \times 10^9 / 6250^2 + 8.1 \times 10^4 \times 0.003 \times 10^6 + \pi^2 \times 2.1 \times 10^5 \times 0.314 \times 10^6 \times 150^2 / 6250^2] \times 70 / 300^2 \times 10^{-3} = 777 \text{ kNm/m}$$

$$s = 2074 \text{ kNm/m} > 777 \text{ kNm/m}$$

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

Rotational restraint given by the sheeting $C_d = 1 / (1/C_d,a + 1/C_d,c)$

(EN1993-1-3, §10.1.5.2)

$$C_d,c = k \cdot E \cdot I_{eff} / s, \quad k=2, \quad I_{eff} = 0.3 \times 0.10 \times 4.90^2 = 1 \text{ mm}^4/\text{m}, \quad s = 1400 \text{ mm}$$

(Eq.10.16)

$$C_d,c = [10^{-3}] 2 \times 2.1 \times 10^5 \times 0.7 / 1400 = 0.2 \text{ kNm/m}$$

$$C_d,a = C100 \cdot k_{ba} \cdot k_t \cdot k_{br} \cdot k_a \cdot k_{bt}$$

(EN1993-1-3, Eq.10.17)

$$C100 = 2.0, \quad k_{ba} = 1.25 \times 300 / 100 = 3.75, \quad k_t = (0.10 / 0.75)^{1.5} = 0.05, \quad k_{br} = 1.0, \quad k_a = 1.0, \quad k_{bt} = 1.0$$

$$C_d,a = 2.0 \times 3.75 \times 0.05 \times 1.0 \times 1.0 \times 1.0 = 0.4 \text{ kNm/m}$$

$$C_d = C_d,a = 0.1 \text{ kNm/m}$$

10.10. Lateral torsional buckling (Purlin laterally restrained)

(EN1993-1-1, §6.3.2)

Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)

Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961

$$M_{cr} = C1 \cdot [\pi^2 E I_z / (kL)^2] \{ \sqrt{[(kz/kw)^2 (I_w/I_z) + (kL)^2 G I_t, eq / (\pi^2 E I_z) + (C2 \cdot z_g - C3 \cdot z_j)^2]} - (C2 \cdot z_g - C3 \cdot z_j) \}$$

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

$$G = E / (2(1+\nu)) = 210000 / (2(1+0.30)) = 80769 \text{ N/mm}^2, \quad I_t, eq = I_t + C_d \cdot (kL)^2 / (\pi^2 G)$$

Hogging

$$k \cdot L = 6250 \text{ mm}, \quad z_g = -150 \text{ mm}, \quad z_j = 0 \text{ mm}$$

(EN1993:2002 Eq.C.11)

$$k_y = 1.0, \quad k_z = 1.0, \quad k_w = 1.0, \quad C1 = 1.127, \quad C2 = 0.454, \quad C3 = 0.000$$

$$M_{cr} = [10^{-6}] 1.127 \times [\pi^2 \times 2.1 \times 10^5 \times 0.314 \times 10^6 / 6250^2]$$

$$\times \{ [(1.0/1.0)^2 \times (6.947 \times 10^9 / 0.314 \times 10^6)$$

$$+ 6250^2 \times 8.1 \times 10^4 \times 0.010 \times 10^6 / (\pi^2 \times 2.1 \times 10^5 \times 0.314 \times 10^6)$$

$$+ (-0.454 \times 150)^2 \}^{0.5} - (-0.454 \times 150) \} = 6.4 \text{ kNm}$$

$$I_t, eq = (0.003 \times 10^6 + 10^3 \times 0.1 \times 6250^2 / (\pi^2 \times 8.1 \times 10^4)) = 0.010 \times 10^6 \text{ mm}^4$$

$$\bar{\lambda}, lt = \sqrt{(W_{pl,y} \cdot f_y / M_{cr})} = \sqrt{[10^{-6}] \times 111.94 \times 10^3 \times 275 / 6.4} = 2.193$$

(EC3 Eq.6.56)

$$h/b = 300/91 = 3.30 > 2.00 \text{ buckling curve: c}$$

$$\text{imperfection factor: } \alpha, lt = 0.49, \quad \beta = 0.75, \quad \chi, lt = 0.208$$

(T.6.3, T.6.5, Fig.6.4)

$$\Phi, lt = 0.5 [1 + \alpha, lt (\bar{\lambda}, lt - \bar{\lambda}, lto) + \beta \bar{\lambda}, lt^2] = 0.5 \times [1 + 0.49 \times (2.193 - 0.40) + 0.75 \times 2.193^2] = 2.743$$

$$\chi, lt = 1 / [\Phi, lt + \sqrt{(\Phi, lt^2 - \beta \bar{\lambda}, lt^2)}] = 1 / [2.743 + \sqrt{(2.743^2 - 0.75 \times 2.743^2)}] = 0.212$$

$$\text{Reduction factor } \chi, lt = 1 / [\Phi, lt + \sqrt{(\Phi, lt^2 - \beta \bar{\lambda}, lt^2)}], \quad \chi, lt \leq 1.0, \quad 1/\bar{\lambda}, lt^2, \quad \chi, lt = 0.208$$

(Eq.6.57)

$$M_{b,rd} = \chi, lt \cdot W_{eff,y} \cdot f_y / \gamma_{M1} = 0.208 \times [10^{-6}] \times 111.94 \times 10^3 \times 275 / 1.00 = 6.40 \text{ kNm}$$

(EC3 Eq.6.55)

$$M_{y,ed} = 2.03 \text{ kNm} < 6.40 \text{ kNm} = M_{b,rd}, \quad \text{Is verified}$$

$$M_{y,ed} / M_{b,rd} = 2.03 / 6.40 = 0.317 < 1$$

11. Global analysis

(EN1993-1-1, §5.2)

11.1. Effects of deformed geometry of the structure

(EN1993-1-1, §5.2.1)

$$\alpha_{cr} = (H_{nhf}/V_{ed})(h/\delta h, ed)$$

(Eq.5.2)

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

Vertical reaction at the base of column

V_{ed}= 143.1 kN

Horizontal reaction at the base of column

H_{ed}= 49.5 kN

Axial force at rafters

N_{red}= 49.5 kN

Notional horizontal force applied at the top of the columns

H_{nhf}= 1.0 kN

Horizontal deflection at column top for notional force

 $\delta h, ed = 3.25$ mm

$$\alpha_{cr} = (1.0/143.1)(7250/3.25) = 15.58$$

(Eq.5.2)

Check axial compression of rafters. Axial compression is significant if

(§5.2.1, (4)B)

$$\lambda = \sqrt{(A \cdot f_y / N_{cr})} > 0.3 \sqrt{(A \cdot f_y / N_{ed})}, \quad N_{ed} > 0.09 N_{cr}$$

(§5.2.1 Eq.5.3)

Development length of the rafter pair from column to column $L = 18000 / \cos 0.00^\circ = 18000$ mm

$$N_{cr} = \pi^2 EI / L^2 = \pi^2 \times 210 \times 231.30 \times 10^6 / (18000)^2 = 1479.6 \text{ kN}$$

Maximum axial force in the rafters $N_{ed} = 49.5$ kN, L.C. 202: 1.35Gk+1.50Qs1

$$\lambda = \sqrt{(8446 \times 275 / 1479618)} = 1.25 \leq 0.3 \sqrt{(8446 \times 275 / 49516)} = 2.05$$

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

$$\alpha_{cr} = 15.58 > 10$$

(Eq.5.1)

First-order elastic analysis may be used

(§5.2.2.1)

$$\text{Amplification factor for design moments } \delta = 1 / (1 - 1/\alpha_{cr}) = 1 / (1 - 1/15.58) = 1.07$$

(Eq.5.4)

11.2. Imperfections for global analysis

(EN1993-1-1, §5.3.2)

$$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m \cdot \delta = (1/200) \times 0.743 \times 0.866 \times 1.069 = 3.437 \times 10^{-3} = 1/291$$

(Eq.5.5)

$$\phi_0 = 1/200, \quad \alpha_h = 2/\sqrt{h} = 2/\sqrt{7.250} = 0.743 \quad 2/3 \leq \alpha_h \leq 1.0, \quad \alpha_m = \sqrt{(0.5(1+1/2))} = 0.866$$

Sway imperfection may be disregarded where $H_{ed} > 0.15 V_{ed}$

(§5.3.2(4) Eq.5.7)

$$\text{Effect of initial sway imperfection } H_{eq} = 3.437 \times 10^{-3} \times V_{ed}$$

(§5.3.2 (5))

11.3. Sway imperfections for columns

(EN1993-1-1, §5.3.2)

Reactions at the supports, Horizontal Force H_{ed} [kN], Vertical Force V_{ed} [kN]

		Left support 1		Right support 2		Hed1+Hed2	Ved1+Ved2	Hed/Vhe	$\phi \cdot V_{ed}$ Heq kN
		Hed,1	Ved,1	Hed,2	Ved,2	Hed	Ved		
201	ULS-STR	31.3	95.9	-31.3	95.9	0.0	191.8	0.00	0.330
202	ULS-STR	49.5	143.1	-49.5	143.1	0.0	286.3	0.00	0.492
203	ULS-STR	-19.2	21.7	-22.8	38.6	-42.0	60.3	0.70	0.000
210	ULS-STR	-24.0	5.6	-18.1	22.5	-42.0	28.0	1.50	0.000
211	ULS-STR	27.0	118.9	-52.2	129.0	-25.2	247.9	0.10	0.443
212	ULS-STR	-3.6	62.2	-38.4	79.1	-42.0	141.3	0.30	0.000
231	ULS-STR	11.4	78.4	-36.6	88.5	-25.2	166.9	0.15	0.000
251	ULS-STR	24.3	109.6	-49.5	119.8	-25.2	229.5	0.11	0.412
252	ULS-STR	-6.3	53.0	-35.7	69.9	-42.0	122.8	0.34	0.000

11.4. Internal forces and bending moments with imperfection effect**11.5. Axial forces Ned [kN]**

		Left column 1	Left rafter 2	Right rafter 3	Right column 4
		Ned,1	Ned,2	Ned,3	Ned,4
201	ULS-STR	-88.3	-31.4	-31.4	-88.6
202	ULS-STR	-135.5	-49.8	-49.8	-135.9
203	ULS-STR	-14.2	-10.2	-10.2	-31.2
210	ULS-STR	0.0	-5.5	-5.5	-17.0
211	ULS-STR	-111.2	-44.9	-44.9	-121.8
212	ULS-STR	-54.7	-25.9	-25.9	-71.7
231	ULS-STR	-70.9	-29.1	-29.1	-81.1
251	ULS-STR	-103.1	-42.2	-42.2	-113.6
252	ULS-STR	-46.6	-23.2	-23.2	-63.5

11.6. Shearing forces Ved [kN]

L.C.		Left column 1		Left rafter 2			Right rafter 3			Right column 4	
		VedA,1	VedB,1	VedA,2	VedC,2	VedB,2	VedA,3	VedC,3	VedB,3	VedA,4	VedB,4
201	ULS-STR	-31.1	-31.1	80.9	64.7	-0.1	-0.1	-64.9	-81.1	31.4	31.4
202	ULS-STR	-49.3	-49.3	128.1	102.4	-0.2	-0.2	-102.8	-128.5	49.8	49.8
203	ULS-STR	19.2	-10.2	6.8	3.7	-8.5	-8.5	-20.7	-23.7	10.2	22.8
210	ULS-STR	24.0	-5.5	-5.5	-6.1	-8.5	-8.5	-10.9	-11.5	5.5	18.1
211	ULS-STR	-26.8	-44.5	103.8	82.0	-5.3	-5.3	-92.5	-114.3	44.9	52.5
212	ULS-STR	3.6	-25.9	47.3	36.1	-8.5	-8.5	-53.1	-64.2	25.9	38.4
231	ULS-STR	-11.4	-29.1	63.5	49.8	-5.1	-5.1	-59.9	-73.6	29.1	36.6
251	ULS-STR	-24.1	-41.8	96.8	76.4	-5.2	-5.2	-86.9	-107.3	42.2	49.8
252	ULS-STR	6.3	-23.2	40.3	30.5	-8.5	-8.5	-47.5	-57.2	23.2	35.7

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

11.7. Bending moments Med [kNm]

L.C.		Left column 1			Right column 4		
		MedA,1	MedM,1		MedA,4	MedM,4	
201	ULS-STR	0.0	-112.8	-225.5	-227.9	-114.0	0.0
202	ULS-STR	0.0	-178.6	-357.2	-360.8	-180.4	0.0
203	ULS-STR	0.0	45.4	32.5	-119.8	-59.9	0.0
210	ULS-STR	0.0	70.6	66.8	-85.5	-42.8	0.0
211	ULS-STR	0.0	-129.2	-258.5	-353.1	-176.6	0.0
212	ULS-STR	0.0	1.6	-80.8	-233.2	-116.6	0.0
231	ULS-STR	0.0	-73.4	-146.7	-238.2	-119.1	0.0
251	ULS-STR	0.0	-119.5	-239.0	-333.4	-166.7	0.0
252	ULS-STR	0.0	4.9	-61.2	-213.6	-106.8	0.0

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

L.C.		Left rafter 2			Right rafter 3		
		MedA,2	MedC2	MedM,2	MedA,3	MedM,3	MedC3
201	ULS-STR	-225.5	-94.5	137.8	137.8	-45.1	-96.5
202	ULS-STR	-357.2	-149.8	218.2	218.2	-71.3	-152.6
203	ULS-STR	32.5	42.0	46.1	25.0	-47.4	-79.9
210	ULS-STR	66.8	56.4	35.5	4.1	-40.7	-65.4
211	ULS-STR	-258.5	-91.3	186.1	184.9	-84.1	-167.0
212	ULS-STR	-80.8	-5.7	99.7	93.9	-69.6	-127.6
231	ULS-STR	-146.7	-44.8	117.7	116.1	-61.1	-118.0
251	ULS-STR	-239.0	-83.1	174.3	173.0	-80.2	-158.6
252	ULS-STR	-61.2	2.5	88.6	82.0	-65.8	-119.4

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

12. Serviceability Limit State (SLS)

(EN1993-1-1, §7)

12.1. Vertical deflection at the apex

(EN1993-1-1, §7.2.1)

Maximum vertical deflection, L.C. 302: $G_k + Q_{s1}$ $D_y = 74.1 \text{ mm} = 18000/243 = L/243$
 Vertical deflection due to imposed load only $D_y = 18.7 \text{ mm} = 18000/401 = L/963$
 Vertical deflection due to snow only $D_y = 44.9 \text{ mm} = 18000/401 = L/401$
 Limit for vertical deflection $L/200$, Is verified

12.2. Horizontal deflection at the top of column

(EN1993-1-1, §7.2.2)

Maximum horizontal deflection, L.C. 312: $G_k + Q_{w1} + 0.50Q_{s1}$ $D_x = 24.7 \text{ mm} = 7250/294 = h/294$
 Horizontal deflection due to wind only $D_x = 24.5 \text{ mm} = 7250/296 = h/296$
 Limit for horizontal deflection $H/150$, Is verified

12.3. Dynamic effects

(EN1993-1-1, §7.2.3)

Eigenfrequencies and Eigenperiods of the structure

Mass of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$

1	f=	1.230 Hz	T=	0.813 sec
2	f=	3.203 Hz	T=	0.312 sec
3	f=	10.184 Hz	T=	0.098 sec
4	f=	21.273 Hz	T=	0.047 sec
5	f=	30.560 Hz	T=	0.033 sec
6	f=	52.032 Hz	T=	0.019 sec
7	f=	61.248 Hz	T=	0.016 sec
8	f=	65.695 Hz	T=	0.015 sec
9	f=	104.240 Hz	T=	0.010 sec
10	f=	142.826 Hz	T=	0.007 sec
11	f=	149.742 Hz	T=	0.007 sec

13. Column verification (Ultimate Limit State)

(EN1993-1-1, §6)

Profile : HE 400 B-S 275**Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1**

Ned = 143.3 kN

Ved = 49.8 kN

Myed = 360.8 kNm, Mzed = 0.0 kNm

Myed = 330.9 kNm (Column top under the haunch)

Buckling length, In-plane buckling Lcr,y = 7250mm (System length) (EC3 §5.5.2.(7))

Buckling length, Out-of-plane buckling Lcr,z = 6650mm (Column height without haunch)

Buckling length, Torsional buckling Lcr,t = 6650mm

Buckling length, Lateral torsional buckling Lcr,lt = 6650mm

Maximum design values for seismic loading

Ned = 71.6 kN

Ved = 30.5 kN

Myed = 220.9 kNm, Mzed = 0.0 kNm

Myed = 202.6 kNm (Column top under the haunch)

13.1. Classification of steel cross-section, Column

(EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{y,ed}/W_{el,y} \pm M_{z,ed}/W_{el,z}$ $\sigma = [10^3]143.30/19780 \pm [10^6]360.80/2884.0 \times 10^3 \pm [10^6]0.00/721.3 \times 10^3$ $\sigma_1 = 132 \text{ N/mm}^2$, $\sigma_2 = -118 \text{ N/mm}^2$ (compression positive)**Web**

c = 400.0 - 2x24.0 - 2x27.0 = 298.0 mm, t = 13.5 mm, c/t = 298.0/13.5 = 22.07

S 275, t = 13.5 ≤ 40 mm, f_y = 275 N/mm², ε = (235/275)^{0.5} = 0.92

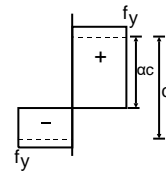
Position of neutral axis for combined Bending and compression

Ned/(2t_w·f_y/γ_{M0}) = 143300/(2x13.5x275/1.00) = 19.3 mm

α = (298.0/2 + 19.3)/298.0 = 0.565 > 0.5

c/t = 22.07 ≤ 396x0.92/(13x0.565-1) = 57.45

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

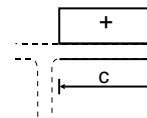
**Flange**

c = 300.0/2 - 13.5/2 - 27.0 = 116.3 mm, t = 24.0 mm, c/t = 116.3/24.0 = 4.84

S 275, t = 24.0 ≤ 40 mm, f_y = 275 N/mm², ε = (235/275)^{0.5} = 0.92

c/t = 4.84 ≤ 9ε = 9x0.92 = 8.28

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

**Overall classification of cross-section is Class 1, Bending and compression N_{c,ed}+M_{y,ed}****13.2. Resistance of cross-section, Column (Ultimate Limit State)**

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

N_{c,ed} = 143.30 kNCompression Resistance N_{plrd} = A·f_y/γ_{M0} = [10⁻³]x19780x275/1.00 = 5439.50 kNN_{ed} = 143.30 kN < 5439.50 kN = N_{c,rd} = N_{plrd}, Is verifiedN_{ed}/N_{c,rd} = 143.30/5439.50 = 0.026 < 1**Ultimate Limit State, Verification for bending moment y-y**

(EN1993-1-1, §6.2.5)

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

M_{y,ed} = 360.80 kNmBending Resistance M_{pl,y,rd} = W_{pl,y}·f_y/γ_{M0} = [10⁻⁶]x3232.0x10³x275/1.00 = 888.80 kNmM_{y,ed} = 360.80 kNm < 888.80 kNm = M_{y,rd} = M_{pl,y,rd}, Is verifiedM_{y,ed}/M_{y,rd} = 360.80/888.80 = 0.406 < 1

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

Maximum design values. Verification for load case: L.C. 211: 1.35xGk+1.50Qs1+0.90Qw1

Vz.ed= 52.50 kN $A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 19780 - 2 \times 300.0 \times 24.0 + (13.5 + 2 \times 27.0) \times 24.0 = 7000 \text{ mm}^2$ (EC3 §6.2.6.3) $A_v = 7000 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (400.0 - 2 \times 24.0) \times 13.5 = 1.00 \times 376.0 \times 13.5 = 5076 \text{ mm}^2$ Plastic Shear Resistance $V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 7000 \times (275 / 1.73) / 1.00 = 1111.40 \text{ kN}$ $V_{z,ed} = 52.50 \text{ kN} < 1111.40 \text{ kN} = V_{z,rd} = V_{pl,z,rd}$, Is verified $V_{z,ed} / V_{z,rd} = 52.50 / 1111.40 = 0.047 < 1$ $h_w / t_w = (400.0 - 2 \times 24.0) / 13.5 = 376.0 / 13.5 = 27.85 \leq 72 \times 0.92 / 1.00 = 72 \varepsilon / \eta = 66.24$ ($\eta = 1.00$)S 275, $t = 13.5 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\varepsilon = (235 / 275)^{0.5} = 0.92$

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

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

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

N.ed= 143.30kN (Compression), Vz.ed= 49.80kN, My.ed= 360.80kNm $N_{pl,rd} = 5439.50 \text{ kN}$, $M_{pl,y,rd} = 888.80 \text{ kNm}$, $V_{pl,z,rd} = 1111.40 \text{ kN}$ $N_{ed} = 143.30 \text{ kN} \leq 0.25 \times 5439.50 = 0.25 \times N_{pl,rd} = 1359.88 \text{ kN}$ $N_{ed} = 143.30 \text{ kN} \leq [10^{-3}] \times 0.5 \times 376.0 \times 13.5 \times 275 / 1.00 = 0.5 h_w \cdot t_w \cdot f_y / \gamma_{M0} = 697.95 \text{ kN}$ $n = N_{ed} / N_{pl,rd} = 143 / 5440 = 0.026$

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

 $V_{ed} = 49.80 \text{ kN} \leq 0.50 \times 1111.40 = 0.50 \times V_{pl,rd} = 555.70 \text{ kN}$

Effect of shear force is neglected (EC3 §6.2.8.2)

 $M_{y,ed} = 360.80 \text{ kNm} < 888.80 \text{ kNm} = M_{pl,y,rd}$, Is verified $M_{y,ed} / M_{pl,y,rd} = 360.80 / 888.80 = 0.406 < 1$ **13.3. Flexural Buckling, Column (Ultimate Limit State)** (EN1993-1-1, §6.3.1)**Nc.ed=143.30 kN, Lcr,y=7.250 m, Lcr,z=6.650 m**

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

Buckling lengths: $L_{cr,y} = 1.000 \times 7250 = 7250 \text{ mm}$, $L_{cr,z} = 0.917 \times 7250 = 6650 \text{ mm}$

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

 $\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (7250 / 170.8) \times (1 / 86.39) = 0.491$ $\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (6650 / 74.0) \times (1 / 86.39) = 1.041$ $\lambda_1 = \pi \sqrt{(E / f_y)} = 93.9 \varepsilon = 86.39$, $\varepsilon = \sqrt{(235 / f_y)} = 0.92$ $h/b = 400 / 300 = 1.33 > 1.20$, $t_f = 24.0 \text{ mm} \leq 40 \text{ mm}$ y-y buckling curve: a, imperfection factor: $\alpha_y = 0.21$, $\chi_y = 0.927$ (T.6.2, T.6.1, Fig.6.4) $\Phi_y = 0.5 [1 + \alpha_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.5 \times [1 + 0.21 \times (0.491 - 0.2) + 0.491^2] = 0.651$ $\chi_y = 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}] = 1 / [0.651 + \sqrt{(0.651^2 - 0.491^2)}] = 0.927 \leq 1$ $\chi_y = 0.927$ z-z buckling curve: b, imperfection factor: $\alpha_z = 0.34$, $\chi_z = 0.571$ $\Phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = 0.5 \times [1 + 0.34 \times (1.041 - 0.2) + 1.041^2] = 1.185$ $\chi_z = 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}] = 1 / [1.185 + \sqrt{(1.185^2 - 1.041^2)}] = 0.571 \leq 1$ $\chi_z = 0.571$ Reduction factor $\chi = 1 / [\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}]$, $\chi \leq 1.0$, $\Phi = 0.5 [1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2]$, $\chi = 0.571$ (EC3 Eq.6.49) $N_{b,rd} = \chi \cdot A \cdot f_y / \gamma_{M1} = 0.571 \times [10^{-3}] \times 19780 \times 275 / 1.00 = 3105.95 \text{ kN}$ (EC3 Eq.6.47) $N_{c,ed} = 143.30 \text{ kN} < 3105.95 \text{ kN} = N_{b,rd}$, Is verified $N_{c,ed} / N_{b,rd} = 143.30 / 3105.95 = 0.046 < 1$ **13.4. Lateral torsional buckling, Column (ULS)** (EN1993-1-1, §6.3.2)**My.ed=330.91 kN, L=7.250m, Lcr,y=7.250m, Lcr,z=6.650m, Lcr,lt=6.650m**

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

Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)

Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961

 $M_{cr} = C_1 \cdot [\pi^2 E I_z / (k L)^2] \{ \sqrt{[(k z / k_w)^2 (I_w / I_z) + (k L)^2 G I_t / (\pi^2 E I_z) + (C_2 \cdot z_g - C_3 \cdot z_j)^2]} - (C_2 \cdot z_g - C_3 \cdot z_j) \}$ Method of computation C_1, C_2, C_3 : ECCS 119/Galea SNO30a-EN-EU Access Steel 2006 $\psi = M_b / M_a = 0.0 / -330.9 = 0.00$, $C_1 = 1.770$, $C_2 = 0.000$, $C_3 = 1.000$, $G = E / (2(1 + \nu)) = 210000 / (2(1 + 0.30)) = 80769 \text{ N/mm}^2$ $k \cdot L = 6650 \text{ mm}$, $z_g = h / 2 = 400 / 2 = 200 \text{ mm}$, $z_j = 0 \text{ mm}$ (EN1993:2002 Eq.C.11) $k_y = 1.0$, $k_z = 1.0$, $k_w = 1.0$, $C_1 = 1.770$, $C_2 = 0.000$, $C_3 = 1.000$ $M_{cr} = [10^{-6}] 1.770 \times [\pi^2 \times 2.1 \times 10^5 \times 108.20 \times 10^6 / 6650^2]$ $\times \{ [(1.0 / 1.0)^2 \times (3817.2 \times 10^9 / 108.20 \times 10^6)]$ $+ 6650^2 \times 8.1 \times 10^4 \times 3.557 \times 10^6 / (\pi^2 \times 2.1 \times 10^5 \times 108.20 \times 10^6) \}^{0.5} = 2721.6 \text{ kNm}$

$\bar{\lambda}, l_t = \sqrt{(W_{pl,y} \cdot f_y / M_{cr})} = \sqrt{([10^{-6}] \times 3232.0 \times 10^3 \times 275 / 2721.6)} = 0.571$ (EC3 Eq.6.56)
 $h/b = 400/300 = 1.33 < 2.00$ buckling curve: b
 imperfection factor: $\alpha, l_t = 0.34$, $\beta = 0.75$, $\chi, l_t = 0.930$ (T.6.3, T.6.5, Fig.6.4)
 $\Phi, l_t = 0.5[1 + \alpha, l_t(\bar{\lambda}, l_t - \bar{\lambda}, l_{to}) + \beta \bar{\lambda}, l_t^2] = 0.5 \times [1 + 0.34 \times (0.571 - 0.40) + 0.75 \times 0.571^2] = 0.652$
 $\chi, l_t = 1 / [\Phi, l_t + \sqrt{(\Phi, l_t^2 - \beta \bar{\lambda}, l_t^2)}] = 1 / [0.652 + \sqrt{(0.652^2 - 0.75 \times 0.652^2)}] = 0.930$
 Reduction factor $\chi, l_t = 1 / [\Phi, l_t + \sqrt{(\Phi, l_t^2 - \beta \bar{\lambda}, l_t^2)}]$, $\chi, l_t \leq 1.0$, $1 / \bar{\lambda}, l_t^2$, $\chi, l_t = 0.930$ (Eq.6.57)
 $\chi, l_t, mod = \chi, l_t / f$, $\chi, l_t, mod \leq 1$, $\chi, l_t, mod \leq 1 / \bar{\lambda}, l_t^2 = 1 / 0.571^2 = 3.06$ (EC3 §6.3.2.3(2), Eq.6.58)
 $K_c = 1 / (1.33 - 0.33\psi) = 0.752$, $\psi = 0.00$ (EC3 Tab.6.6)
 $f = 1 - 0.5(1 - K_c)[1 - 2.0(\bar{\lambda}, l_t - 0.8)^2] = 1 - 0.5 \times (1 - 0.752)[1 - 2.0 \times (0.571 - 0.8)^2] = 0.889$, $f \leq 1.0$
 $\chi, l_t, mod = \chi, l_t / f = 0.930 / 0.889 = 1.046$, $\chi, l_t, mod \leq 1.0$, $\chi, l_t, mod \leq 3.06$, $\chi, l_t, mod = 1.000$
 $M_{b,rd} = \chi, l_t \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 1.000 \times [10^{-6}] \times 3232.0 \times 10^3 \times 275 / 1.00 = 888.80 \text{ kNm}$ (EC3 Eq.6.55)
 $M_{y,ed} = 330.91 \text{ kNm} < 888.80 \text{ kNm} = M_{b,rd}$, Is verified
 $M_{y,ed} / M_{b,rd} = 330.91 / 888.80 = 0.372 < 1$

13.5. Axial force and bending moment, Column (ULS)

(EN1993-1-1, §6.3.3)

Ned=143.30 kN, My,ed=330.91 kNm

$N_{ed} / (\chi_y \cdot N_{rk} / \gamma_{M1}) + k_{yy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk} / \gamma_{M1}) \leq 1$ (EC3 Eq.6.61)
 $N_{ed} / (\chi_z \cdot N_{rk} / \gamma_{M1}) + k_{zy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk} / \gamma_{M1}) \leq 1$ (EC3 Eq.6.62)
 $N_{rk} = A \cdot f_y = [10^{-3}] \times 19780 \times 275 = 5439.5 \text{ kN}$ (Tab.6.7)
 $M_{y,rk} = W_{pl,y} \cdot f_y = [10^{-6}] \times 3232.0 \times 10^3 \times 275 = 888.8 \text{ kNm}$
 $\chi_y \cdot N_{rk} / \gamma_{M1} = \chi_y \cdot A \cdot f_y / \gamma_{M1} = 0.927 \times [10^{-3}] \times 19780 \times 275 / 1.00 = 5042.4 \text{ kN}$
 $\chi_z \cdot N_{rk} / \gamma_{M1} = \chi_z \cdot A \cdot f_y / \gamma_{M1} = 0.571 \times [10^{-3}] \times 19780 \times 275 / 1.00 = 3106.0 \text{ kN}$
 $\chi_{LT} \cdot M_{y,rk} / \gamma_{M1} = \chi_{LT} \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 1.000 \times [10^{-6}] \times 3232.0 \times 10^3 \times 275 / 1.00 = 888.8 \text{ kNm}$

Interaction factors, Method of computation: Method 1 Annex A

(EC3 Annex A)

$k_{yy} = C_{my} \cdot C_{mLT}(\mu_y / (1 - N_{ed} / N_{cr,y})) (1 / C_{yy})$, $\mu_y = (1 - N_{ed} / N_{cr,y}) / (1 - \chi_y \cdot N_{ed} / N_{cr,y})$ (EC3 Tab.A.1)
 $k_{zy} = C_{my} \cdot C_{mLT}(\mu_z / (1 - N_{ed} / N_{cr,y})) (1 / C_{zy}) 0.60 \sqrt{(w_y / w_z)}$, $\mu_z = (1 - N_{ed} / N_{cr,z}) / (1 - \chi_z \cdot N_{ed} / N_{cr,z})$

$N_{cr,y} = \pi^2 E I_y / l_{cr,y}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 576.80 \times 10^6 / 7250^2 = 22744 \text{ kN}$
 $N_{cr,z} = \pi^2 E I_z / l_{cr,z}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 108.20 \times 10^6 / 6650^2 = 5071 \text{ kN}$
 $N_{cr,t} = (1 / i_p^2) \times (G \cdot I_t + \pi^2 E I_w / L_{cr,t}^2)$ (EC3 NCCI SN003b-EN-EU)
 $N_{cr,t} = [10^{-3}] \times (1 / 186^2) [80769 \times 3.557 \times 10^6 + \pi^2 \times 210000 \times 3817.2 \times 10^9 / 6650^2] = 13463 \text{ kN}$

$\mu_y = (1 - N_{ed} / N_{cr,y}) / (1 - \chi_y \cdot N_{ed} / N_{cr,y}) = (1 - 143.3 / 22744) / (1 - 0.927 \times 143.3 / 22744) = 1.000$
 $\mu_z = (1 - N_{ed} / N_{cr,z}) / (1 - \chi_z \cdot N_{ed} / N_{cr,z}) = (1 - 143.3 / 5071) / (1 - 0.571 \times 143.3 / 5071) = 0.988$
 $alt = 1 - I_t / I_y > 0 = 1 - 3.557 \times 10^6 / 576.80 \times 10^6 = 0.994$ (EC3 Annex A.1)

$w_y = W_{pl,y} / W_{el,y} \leq 1.50$, $w_y = 3.232 \times 10^6 / 2.884 \times 10^6 = 1.121 \leq 1.50$ (EC3 Annex A.1)
 $w_z = W_{pl,z} / W_{el,z} \leq 1.50$, $w_z = 1.104 \times 10^6 / 0.721 \times 10^6 = 1.531 > 1.50$, $w_z = 1.50$
 $n_{pl} = N_{ed} / (N_{rk} / \gamma_{M1}) = 143.30 / (5439.50 / 1.00) = 0.026$

$\bar{\lambda}_{max} = \max(0.491, 1.041) = 1.040$ (EC3 Annex A.1)
 $M_{cr,o} = (1.00 / 1.77) \times 2721.60 = 1537.6$, $C_1 = 1.00$
 $\bar{\lambda}_o = \sqrt{([10^{-6}] \times 3232.0 \times 10^3 \times 275 / 1537.6)} = 0.760$
 $\bar{\lambda}_o, lim = 0.2 \sqrt{C_1 [(1 - N_{ed} / N_{cr,z}) (1 - N_{ed} / N_{cr,t})]^{0.25}}$ (EC3 Annex A.1)
 $\bar{\lambda}_o, lim = 0.2 \sqrt{1.770 [(1 - 143.3 / 5071) (1 - 143.3 / 13463)]^{0.25}} = 0.263$
 $\varepsilon_y = (M_{y,ed} / N_{ed}) (A / W_{el}) = ([10^3] \times 330.91 / 143.30) \times (19780.0 / 2884.0 \times 10^3) = 15.84$

$C_{my,o} = 0.79 + 0.21\psi + 0.36(\psi - 0.33) \times (143.30 / 22744.0) = 1.002$, ($\psi = 1.00$) (EC3 Annex A, T.A.1)
 $\bar{\lambda}_o = 0.760 > \bar{\lambda}_o, lim = 0.263$
 $C_{my} = C_{my,o} + (1 - C_{my,o}) (\sqrt{\varepsilon_y \cdot alt}) / (1 + \sqrt{\varepsilon_y \cdot alt}) =$
 $= 1.002 + (1 - 1.002) \times (\sqrt{15.838 \times 0.994}) / (1 + \sqrt{15.838 \times 0.994}) = 1.000$
 $C_{mLT} = C_{my}^2 \cdot alt / \sqrt{[(1 - N_{ed} / N_{cr,z}) (1 - N_{ed} / N_{cr,t})]} \geq 1$
 $C_{mLT} = 1.000^2 \times 0.994 / \sqrt{[(1 - 143.3 / 5071.0) (1 - 143.3 / 13463.0)]} = 1.014$, $C_{mLT} = 1.014$

$$C_{yy}=1+(w_y-1)[(2-1.6C_{my}^2 \cdot \bar{\lambda}_{max}/w_y-1.6C_{my}^2 \cdot \bar{\lambda}_{max}^2/w_y)n_{pl}-b_{lt}] \geq W_{el,y}/W_{pl,y} \quad (\text{Annex A, T.A.1})$$

$$b_{lt}=0.5alt \cdot \bar{\lambda}_o^2 [M_{y,ed}/(\chi_{lt} \cdot M_{pl,y,rd})] (M_{z,ed}/M_{pl,z,rd}) =$$

$$=0.5 \times 0.994 \times 0.760^2 [330.9/(1.000 \times 793.1)] (0.0/198.4) = 0.000$$

$$C_{yy}=1+(1.121-1)[(2-1.6 \times 1.000^2 \times 1.040/1.121-1.6 \times 1.000^2 \times 1.040^2/1.121) \times 0.026-0.000]=0.997$$

$$C_{yy} \geq 2884.0 \times 10^3 / 3232.0 \times 10^3 = 0.892, \quad C_{yy}=0.997$$

$$C_{zy}=1+(w_y-1)[(2-14.0C_{my}^2 \cdot \bar{\lambda}_{max}^2/w_y^5)n_{pl}-d_{lt}] \geq 0.6 \sqrt{(w_y/w_z)} (W_{el,y}/W_{pl,y}) \quad (\text{Annex A, T.A.1})$$

$$d_{lt}=2alt \cdot [\bar{\lambda}_o/(0.1+\bar{\lambda}_z^4)] [M_{y,ed}/(C_{my} \cdot \chi_{lt} \cdot M_{pl,y,rd})] [M_{z,ed}/(C_{mz} \cdot M_{pl,z,rd})] =$$

$$=20.994 \times [0.760/(0.1+1.041^4)] [330.9/(1.000 \times 1.000 \times 793.1)] [0.0/(0.000 \times 198.4)] = 0.000$$

$$C_{zy}=1+(1.121-1)[(2-14.0 \times 1.000^2 \times 1.040^2/1.121^5) \times 0.026-0.000]=0.979$$

$$C_{zy} \geq 0.6 \sqrt{(1.121/1.500)} (2884.0 \times 10^3 / 3232.0 \times 10^3) = 0.463, \quad C_{zy}=0.979$$

$$C_{yy}=0.997, \quad C_{zy}=0.979 \quad (\text{Annex A, T.A.1})$$

$$k_{yy}=1.000 \times 1.014 \times 1.000 / (1-143.30/22744.0) \times (1/0.997) = 1.023$$

$$k_{zy}=1.000 \times 1.014 \times 0.988 / (1-143.30/22744.0) \times (1/0.979) \times 0.6 \times \sqrt{(1.121/1.500)} = 0.534$$

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

$$N_{ed}/(\chi_y \cdot N_{rk}/\gamma_{M1}) + k_{yy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) = \quad (\text{EC3 Eq.6.61})$$

$$143.3/(0.927 \times 5439.5/1.00) + 1.023 \times 330.9/(1.000 \times 888.8/1.00) = 0.028 + 0.381 = 0.409$$

$$0.409 < 1.000, \quad \text{Is verified}$$

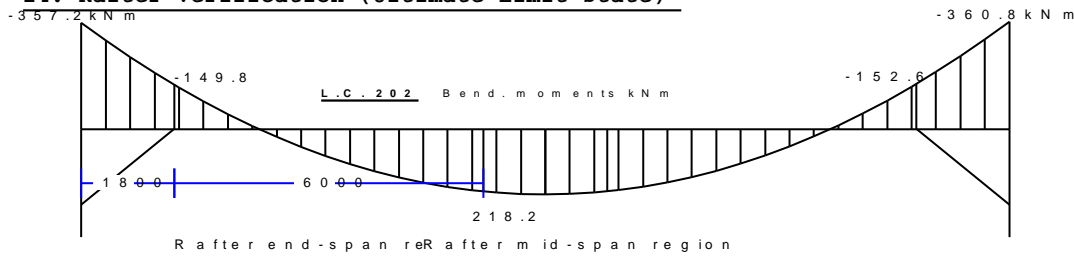
$$N_{ed}/(\chi_z \cdot N_{rk}/\gamma_{M1}) + k_{zy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) = \quad (\text{EC3 Eq.6.62})$$

$$143.3/(0.571 \times 5439.5/1.00) + 0.534 \times 330.9/(1.000 \times 888.8/1.00) = 0.046 + 0.199 = 0.245$$

$$0.245 < 1.000, \quad \text{Is verified}$$

14. Rafter verification (Ultimate Limit State)

(EN1993-1-1, §6)

**Profile : IPE 400-S 275****Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1**

Ned = 49.8 kN
 Ved = 102.8 kN
 Myed = 218.2 kNm, Mzed = 0.0 kNm
 Myed = 218.2 kNm (at mid-span)
 Myed = -152.6 kNm (at haunch-start)
 Myed = -335.1 kNm (at haunch end)
 Myed = -360.8 kNm (at column axis point)

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

Ned = 5.5 kN
 Ved = 10.9 kN
 Myed = -40.7 kNm

Maximum design values for seismic loading

Ned = 20.8 kN
 Ved = 48.8 kN
 Myed = 120.6 kNm, Mzed = 0.0 kNm
 Myed = 120.6 kNm (at haunch-start)
 Myed = 147.4 kNm (at haunch end)

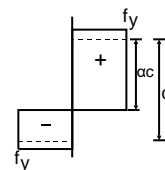
14.1. Classification of steel cross-section, Rafter

(EN1993-1-1, §5.5)

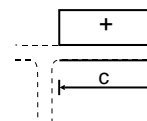
Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{yed}/W_{el,y} \pm M_{zed}/W_{el,z}$
 $\sigma = [10^3]49.80/8446 \pm [10^6]218.20/1156.0 \times 10^3 \pm [10^6]0.00/146.4 \times 10^3$
 $\sigma_1 = 195 \text{ N/mm}^2$, $\sigma_2 = -183 \text{ N/mm}^2$ (compression positive)

Web

$c = 400.0 - 2 \times 13.5 - 2 \times 21.0 = 331.0 \text{ mm}$, $t = 8.6 \text{ mm}$, $c/t = 331.0/8.6 = 38.49$
 S 275, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 Position of neutral axis for combined Bending and compression
 $N_{ed}/(2t_w \cdot f_y / \gamma_{M0}) = 49800 / (2 \times 8.6 \times 275 / 1.00) = 10.5 \text{ mm}$
 $\alpha = (331.0/2 + 10.5) / 331.0 = 0.532 > 0.5$
 $c/t = 38.49 \leq 396 \times 0.92 / (13 \times 0.532 - 1) = 61.61$
 The web is class 1 (EN1993-1-1, Tab.5.2)

**Flange**

$c = 180.0/2 - 8.6/2 - 21.0 = 64.7 \text{ mm}$, $t = 13.5 \text{ mm}$, $c/t = 64.7/13.5 = 4.79$
 S 275, $t = 13.5 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 $c/t = 4.79 \leq 9 \epsilon = 9 \times 0.92 = 8.28$
 The flange is class 1 (EN1993-1-1, Tab.5.2)

**Overall classification of cross-section is Class 1, Bending and compression $N_{c,ed} + M_{y,ed}$**

14.2. Resistance of cross-section, Rafter (Ultimate Limit State)

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

Nc.ed= 49.80 kNCompression Resistance $N_{plrd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 8446 \times 275 / 1.00 = 2322.65 \text{ kN}$

Ned= 49.80 kN < 2322.65 kN =Nc,rd=Nplrd, Is verified

Ned/Nc,rd= 49.80/2322.65= 0.021<1

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

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

My.ed=218.20 kNmBending Resistance $M_{ply,rd} = W_{ply} \cdot f_y / \gamma_{M0} = [10^{-6}] \times 1307.0 \times 10^3 \times 275 / 1.00 = 359.42 \text{ kNm}$

My,ed= 218.20 kNm < 359.42 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 218.20/359.42= 0.607<1

Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

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

Vz.ed=102.80 kN $A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 8446 - 2 \times 180.0 \times 13.5 + (8.6 + 2 \times 21.0) \times 13.5 = 4269 \text{ mm}^2$

(EC3 §6.2.6.3)

 $A_v = 4269 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (400.0 - 2 \times 13.5) \times 8.6 = 1.00 \times 386.5 \times 8.6 = 3324 \text{ mm}^2$ Plastic Shear Resistance $V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 4269 \times (275 / 1.73) / 1.00 = 677.81 \text{ kN}$

Vz,ed= 102.80 kN < 677.81 kN =Vz,rd=Vpl,z,rd, Is verified

Vz,ed/Vz,rd= 102.80/677.81= 0.152<1

 $h_w / t_w = (400.0 - 2 \times 13.5) / 8.6 = 386.5 / 8.6 = 44.94 \leq 72 \times 0.92 / 1.00 = 72 \varepsilon / \eta = 66.24$ ($\eta = 1.00$)S 275, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\varepsilon = (235 / 275)^{0.5} = 0.92$

Shear buckling resistance is not necessary to be verified

(EC3 §6.2.6.6)

Ultimate Limit State, Verification for axial force, shear and bending

(EN1993-1-1, §6.2.9)

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

N.ed= 49.80kN (Compression), Vz.ed= 102.80kN, My.ed= 218.20kNm $N_{plrd} = 2322.65 \text{ kN}$, $M_{pl,y,rd} = 359.42 \text{ kNm}$, $V_{pl,z,rd} = 677.81 \text{ kN}$

Ned=49.80kN <= 0.25x2322.65=0.25xNplrd=580.66kN

Ned=49.80kN <= $[10^{-3}] \times 0.5 \times 386.5 \times 8.6 \times 275 / 1.00 = 0.5 h_w \cdot t_w \cdot f_y / \gamma_{M0} = 457.04 \text{ kN}$

n=Ned/Nplrd=50/2323= 0.021

Effect of axial force is neglected

(EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

Ved=102.80kN <= 0.50x677.81=0.50xVpl,rd=338.90kN

Effect of shear force is neglected

(EC3 §6.2.8.2)

My,ed= 218.20 kNm < 359.42 kNm =Mply,rd, Is verified

My,ed/Mply,rd= 218.20/359.42= 0.607<1

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

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

Ned = 49.8 kN

Ved = 128.5 kN

Myed = 218.2 kNm, Mzed =0.0 kNm

Rafter length $L_r = 18000 \text{ mm}$

Buckling length, In-plane buckling

 $\alpha_{cr} = 15.58$, Ned=49.8kN, $L_{cr,y} = \pi \sqrt{EI / \alpha_{cr} \cdot Ned} \leq L_r = 18000 \text{ mm}$ $L_{cr,y} = \pi \sqrt{[210000 \times 231.30 \times 10^6] / (15.58 \times 49.8 \times 10^3)} = 24867 \text{ mm}$, $L_{cr,y} = 18000 \text{ mm}$ Buckling length, In-plane buckling $L_{cr,y} = 18000 \text{ mm}$ (System length)Buckling length, Out-of-plane buckling $L_{cr,z} = 1400 \text{ mm}$ (Purlin spacing)**14.4. Flexural Buckling, Rafter mid-span region (Ultimate Limit State)**

(EN1993-1-1, §6.3.1)

Nc.ed=49.76 kN, Lcr,y=18.000 m, Lcr,z=1.400 m

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

Buckling lengths: $L_{cr,y} = 1.000 \times 18000 = 18000 \text{ mm}$, $L_{cr,z} = 0.078 \times 18000 = 1400 \text{ mm}$

Non-dimensional slenderness (Cross-section Class: 1)

(EC3 §6.3.1.3)

 $\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (18000 / 165.5) \times (1 / 86.39) = 1.259$ $\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (1400 / 39.5) \times (1 / 86.39) = 0.410$ $\lambda_1 = \pi \sqrt{(E / f_y)} = 93.9 \varepsilon = 86.39$, $\varepsilon = \sqrt{(235 / f_y)} = 0.92$

$h/b=400/180=2.22 \geq 1.20$, $t_f=13.5\text{mm} \leq 40\text{ mm}$
 $y-y$ buckling curve: a, imperfection factor: $\alpha_y=0.21$, $\chi_y=0.494$ (T.6.2, T.6.1, Fig.6.4)
 $\Phi_y=0.5[1+\alpha_y(\bar{\lambda}_y-0.2)+\bar{\lambda}_y^2]=0.5x[1+0.21x(1.259-0.2)+1.259^2]=1.404$
 $\chi_y=1/[\Phi_y+\sqrt{(\Phi_y^2-\bar{\lambda}_y^2)}]=1/[1.404+\sqrt{(1.404^2-1.259^2)}]=0.494 \leq 1$ $\chi_y=0.494$
 $z-z$ buckling curve: b, imperfection factor: $\alpha_z=0.34$, $\chi_z=0.922$
 $\Phi_z=0.5[1+\alpha_z(\bar{\lambda}_z-0.2)+\bar{\lambda}_z^2]=0.5x[1+0.34x(0.410-0.2)+0.410^2]=0.620$
 $\chi_z=1/[\Phi_z+\sqrt{(\Phi_z^2-\bar{\lambda}_z^2)}]=1/[0.620+\sqrt{(0.620^2-0.410^2)}]=0.922 \leq 1$ $\chi_z=0.922$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi \leq 1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.494$ (EC3 Eq.6.49)
 $N_{b,rd}=\chi \cdot A \cdot f_y/\gamma_{M1}=0.494x[10^{-3}]x8446x275/1.00=1147.39\text{ kN}$ (EC3 Eq.6.47)
 $N_{c,ed}=49.76\text{ kN} < 1147.39\text{ kN} = N_{b,rd}$, Is verified
 $N_{c,ed}/N_{b,rd}=49.76/1147.39=0.043 < 1$

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

$M_{y,ed}=218.20\text{ kN}$, $L=18.000\text{m}$, $L_{cr,y}=18.000\text{m}$, $L_{cr,z}=1.400\text{m}$, $L_{cr,lt}=1.400\text{m}$
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961
 $M_{cr}=C1 \cdot [\pi^2 EI_z / (kL)^2] \{ \sqrt{[(kz/kw)^2 (I_w/I_z) + (kL)^2 GIt / (\pi^2 EI_z) + (C2 \cdot z_g - C3 \cdot z_j)^2]} - (C2 \cdot z_g - C3 \cdot z_j) \}$
Method of computation $C1, C2, C3$: ECCS 119/Galea SN030a-EN-EU Access Steel 2006
 $\mu=Mo/M=qL^2/8M=3.5/218.2=0.02$, $\psi=Mb/Ma=-39.5/218.2=-0.18$, $C1=1.613$, $C2=0.060$
 $G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4\text{ N/mm}^2$
 $k \cdot L=1400\text{mm}$, $z_g=h/2=400/2=200\text{mm}$, $z_j=0\text{mm}$ (EN1993:2002 Eq.C.11)
 $k_y=1.0$, $k_z=1.0$, $k_w=1.0$, $C1=1.613$, $C2=0.060$, $C3=0.000$
 $M_{cr}=[10^{-6}]1.613x[\pi^2x2.1x10^5x13.180x10^6/1400^2]$
 $x\{ [(1.0/1.0)^2x(490.05x10^9/13.180x10^6)$
 $+1400^2x8.1x10^4x0.511x10^6/(\pi^2x2.1x10^5x13.180x10^6)$
 $+(0.060x200)^2]^{0.5}-(0.060x200) \}=4243.4\text{ kNm}$

$\bar{\lambda}_{lt}=\sqrt{(W_{pl,y} \cdot f_y / M_{cr})}=\sqrt{[10^{-6}]x1307.0x10^3x275/4243.4}=0.291$ (EC3 Eq.6.56)
 $\bar{\lambda}_{lt} \leq 0.40$, $\chi_{lt}=1.00$ (EC3 §6.3.2.2.4)

$\chi_{lt,mod}=\chi_{lt}/f$, $\chi_{lt,mod} \leq 1$, $\chi_{lt,mod} \leq 1/\bar{\lambda}_{lt}^2=1/0.291^2=11.81$ (EC3 §6.3.2.3(2), Eq.6.58)
 $K_c=1.00$ (EC3 Tab.6.6)
 $f=1-0.5(1-K_c)[1-2.0(\bar{\lambda}_{lt}-0.8)^2]=1-0.5x(1-1.000)[1-2.0x(0.291-0.8)^2]=1.000$, $f \leq 1.0$
 $\chi_{lt,mod}=\chi_{lt}/f=1.000/1.000=1.000$, $\chi_{lt,mod} \leq 1.0$, $\chi_{lt,mod} \leq 11.81$, $\chi_{lt,mod}=1.000$

$M_{b,rd}=\chi_{lt} \cdot W_{pl,y} \cdot f_y/\gamma_{M1}=1.000x[10^{-6}]x1307.0x10^3x275/1.00=359.42\text{ kNm}$ (EC3 Eq.6.55)
 $M_{y,ed}=218.20\text{ kNm} < 359.42\text{ kNm} = M_{b,rd}$, Is verified
 $M_{y,ed}/M_{b,rd}=218.20/359.42=0.607 < 1$

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

$N_{ed}=49.76\text{ kN}$, $M_{y,ed}=218.20\text{ kNm}$

$N_{ed}/(\chi_y \cdot N_{rk}/\gamma_{M1}) + k_{yy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) \leq 1$ (EC3 Eq.6.61)

$N_{ed}/(\chi_z \cdot N_{rk}/\gamma_{M1}) + k_{zy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) \leq 1$ (EC3 Eq.6.62)

$N_{rk}=A \cdot f_y=[10^{-3}]x8446x275=2322.6\text{ kN}$ (Tab.6.7)

$M_{y,rk}=W_{pl,y} \cdot f_y=[10^{-6}]x1307.0x10^3x275=359.4\text{ kNm}$

$\chi_y \cdot N_{rk}/\gamma_{M1}=\chi_y \cdot A \cdot f_y/\gamma_{M1}=0.494x[10^{-3}]x8446x275/1.00=1147.4\text{ kN}$

$\chi_z \cdot N_{rk}/\gamma_{M1}=\chi_z \cdot A \cdot f_y/\gamma_{M1}=0.922x[10^{-3}]x8446x275/1.00=2141.5\text{ kN}$

$\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}=\chi_{LT} \cdot W_{pl,y} \cdot f_y/\gamma_{M1}=1.000x[10^{-6}]x1307.0x10^3x275/1.00=359.4\text{ kNm}$

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

$k_{yy}=C_{my} \cdot C_{mLT}(\mu_y/(1-N_{ed}/N_{cr,y}))(1/C_{yy})$, $\mu_y=(1-N_{ed}/N_{cr,y})/(1-\chi_y \cdot N_{ed}/N_{cr,y})$ (EC3 Tab.A.1)

$k_{zy}=C_{my} \cdot C_{mLT}(\mu_z/(1-N_{ed}/N_{cr,y}))(1/C_{zy})0.60\sqrt{(w_y/w_z)}$, $\mu_z=(1-N_{ed}/N_{cr,z})/(1-\chi_z \cdot N_{ed}/N_{cr,z})$

$N_{cr,y}=\pi^2 EI_y/l_{cr,y}^2=3.14^2x[10^{-3}]x210000x231.30x10^6/18000^2=1480\text{ kN}$

$N_{cr,z}=\pi^2 EI_z/l_{cr,z}^2=3.14^2x[10^{-3}]x210000x13.180x10^6/1400^2=13937\text{ kN}$

$N_{cr,t}=(1/i_p^2)x(G \cdot I_t + \pi^2 EI_w/L_{cr,t}^2)$ (EC3 NCCI SN003b-EN-EU)

$N_{cr,t}=[10^{-3}]x(1/170^2)[80769x0.511x10^6+\pi^2x210000x490.05x10^9/1400^2]=19328\text{ kN}$

$\mu_y = (1 - N_{ed}/N_{cr,y}) / (1 - \chi_y \cdot N_{ed}/N_{cr,y}) = (1 - 49.8 / 1480) / (1 - 0.494 \times 49.8 / 1480) = 0.983$
 $\mu_z = (1 - N_{ed}/N_{cr,z}) / (1 - \chi_z \cdot N_{ed}/N_{cr,z}) = (1 - 49.8 / 13937) / (1 - 0.922 \times 49.8 / 13937) = 1.000$
 $alt = 1 - It/It_y > 0 = 1 - 0.511 \times 10^6 / 231.30 \times 10^6 = 0.998$ (EC3 Annex A.1)

$w_y = W_{pl,y}/W_{el,y} \leq 1.50$, $w_y = 1.307 \times 10^6 / 1.156 \times 10^6 = 1.131 \leq 1.50$ (EC3 Annex A.1)
 $w_z = W_{pl,z}/W_{el,z} \leq 1.50$, $w_z = 0.229 \times 10^6 / 0.146 \times 10^6 = 1.564 > 1.50$, $w_z = 1.50$
 $n_{pl} = N_{ed} / (N_{rk}/\gamma_{M1}) = 49.76 / (2322.60 / 1.00) = 0.021$

$\bar{\lambda}_{max} = \max(1.259, 0.410) = 1.260$ (EC3 Annex A.1)
 $M_{cr,o} = (1.00 / 1.61) \times 4243.40 = 2630.1$, $C1 = 1.00$
 $\bar{\lambda}_o = \sqrt{([10^{-6}] \times 1307.0 \times 10^3 \times 275 / 2630.1)} = 0.370$
 $\bar{\lambda}_o, lim = 0.2 \sqrt{C1 [(1 - N_{ed}/N_{cr,z})(1 - N_{ed}/N_{cr,t})]^{0.25}}$ (EC3 Annex A.1)
 $\bar{\lambda}_o, lim = 0.2 \sqrt{1.613 [(1 - 49.8 / 13937)(1 - 49.8 / 19328)]^{0.25}} = 0.254$
 $\varepsilon_y = (M_{y,ed}/N_{ed})(A/W_{el}) = ([10^3] \times 218.20 / 49.76) \times (8446.0 / 1156.0 \times 10^3) = 32.04$

$C_{my,o} = 0.79 + 0.21\psi + 0.36(\psi - 0.33) \times (49.76 / 1480.0) = 1.008$, ($\psi = 1.00$) (EC3 Annex A, T.A.1)
 $\bar{\lambda}_o = 0.370 > \bar{\lambda}_o, lim = 0.254$
 $C_{my} = C_{my,o} + (1 - C_{my,o})(\sqrt{\varepsilon_y \cdot alt}) / (1 + \sqrt{\varepsilon_y \cdot alt}) =$
 $= 1.008 + (1 - 1.008) \times (\sqrt{32.037 \times 0.998}) / (1 + \sqrt{32.037 \times 0.998}) = 1.001$
 $C_{m1t} = C_{my}^2 \cdot alt / \sqrt{[(1 - N_{ed}/N_{cr,z})(1 - N_{ed}/N_{cr,t})]} \geq 1$
 $C_{m1t} = 1.001^2 \times 0.998 / \sqrt{[(1 - 49.8 / 13937.0)(1 - 49.8 / 19328.0)]} = 1.003$, $C_{m1t} = 1.003$

$C_{yy} = 1 + (w_y - 1)[(2 - 1.6C_{my}^2 \cdot \bar{\lambda}_{max}/w_y - 1.6C_{my}^2 \cdot \bar{\lambda}_{max}^2/w_y)n_{pl} - blt] \geq W_{el,y}/W_{pl,y}$ (Annex A, T.A.1)
 $blt = 0.5alt \cdot \bar{\lambda}_o^2 [M_{y,ed}/(\chi_{lt} \cdot M_{pl,y,rd})] [M_{z,ed}/M_{pl,z,rd}] =$
 $= 0.5 \times 0.998 \times 0.370^2 [0.0 / (1.000 \times 317.9)] [0.0 / 40.3] = 0.000$
 $C_{yy} = 1 + (1.131 - 1)[(2 - 1.6 \times 1.001^2 \times 1.260 / 1.131 - 1.6 \times 1.001^2 \times 1.260^2 / 1.131) \times 0.021 - 0.000] = 0.994$
 $C_{yy} > 1156.0 \times 10^3 / 1307.0 \times 10^3 = 0.884$, $C_{yy} = 0.994$

$C_{zy} = 1 + (w_y - 1)[(2 - 14.0C_{my}^2 \cdot \bar{\lambda}_{max}^2/w_y^5)n_{pl} - dlt] \geq 0.6 \sqrt{(w_y/w_z)} (W_{el,y}/W_{pl,y})$ (Annex A, T.A.1)
 $dlt = 2alt \cdot [\bar{\lambda}_o / (0.1 + \bar{\lambda}_z^4)] [M_{y,ed}/(C_{my} \cdot \chi_{lt} \cdot M_{pl,y,rd})] [M_{z,ed}/(C_{mz} \cdot M_{pl,z,rd})] =$
 $= 20.998 \times [0.370 / (0.1 + 0.410^4)] [0.0 / (1.001 \times 1.000 \times 317.9)] [0.0 / (0.000 \times 40.3)] = 0.000$
 $C_{zy} = 1 + (1.131 - 1)[(2 - 14.0 \times 1.001^2 \times 1.260^2 / 1.131^5) \times 0.021 - 0.000] = 0.972$
 $C_{zy} > 0.6 \sqrt{(1.131 / 1.500)} (1156.0 \times 10^3 / 1307.0 \times 10^3) = 0.461$, $C_{zy} = 0.972$

$C_{yy} = 0.994$, $C_{zy} = 0.972$ (Annex A, T.A.1)
 $k_{yy} = 1.001 \times 1.003 \times 0.983 / (1 - 49.76 / 1480.0) \times (1 / 0.994) = 1.027$
 $k_{zy} = 1.001 \times 1.003 \times 1.000 / (1 - 49.76 / 1480.0) \times (1 / 0.972) \times 0.6 \sqrt{(1.131 / 1.500)} = 0.557$

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
 $N_{ed} / (\chi_y \cdot N_{rk}/\gamma_{M1}) + k_{yy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) =$ (EC3 Eq.6.61)
 $49.8 / (0.494 \times 2322.6 / 1.00) + 1.027 \times 218.2 / (1.000 \times 359.4 / 1.00) = 0.043 + 0.624 = 0.667$
 $0.667 < 1.000$, Is verified

$N_{ed} / (\chi_z \cdot N_{rk}/\gamma_{M1}) + k_{zy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) =$ (EC3 Eq.6.62)
 $49.8 / (0.922 \times 2322.6 / 1.00) + 0.557 \times 218.2 / (1.000 \times 359.4 / 1.00) = 0.023 + 0.338 = 0.361$
 $0.361 < 1.000$, Is verified

14.7. Buckling resistance, Rafter end-span region (Ultimate Limit State)
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
 $N_{ed} = 49.8$ kN
 $V_{ed} = 128.5$ kN
 $M_{y,ed} = 207.7$ kNm, $M_{z,ed} = 0.0$ kNm
 Rafter length $L_r = 18000$ mm
 Buckling length, In-plane buckling $L_{cr,y} = 18000$ mm (System length)
 Buckling length, Out-of-plane buckling $L_{cr,z} = 6000$ mm (Torsional restrains of rafters)

14.8. Flexural Buckling, Rafter end-span region (Ultimate Limit State) (EN1993-1-1, §6.3.1)
 $N_{c,ed} = 49.76$ kN, $L_{cr,y} = 18.000$ m, $L_{cr,z} = 6.000$ m
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
 Buckling lengths: $L_{cr,y} = 1.000 \times 18000 = 18000$ mm, $L_{cr,z} = 0.333 \times 18000 = 6000$ mm
 Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)
 $\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (18000 / 165.5) \times (1 / 86.39) = 1.259$
 $\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (6000 / 39.5) \times (1 / 86.39) = 1.758$
 $\lambda_1 = \pi \sqrt{(E / f_y)} = 93.9 \varepsilon = 86.39$, $\varepsilon = \sqrt{(235 / f_y)} = 0.92$

$h/b=400/180=2.22>1.20$, $t_f=13.5\text{mm}\leq 40\text{ mm}$
 $y-y$ buckling curve: a , imperfection factor: $\alpha_y=0.21$, $\chi_y=0.494$ (T.6.2, T.6.1, Fig.6.4)
 $\Phi_y=0.5[1+\alpha_y(\bar{\lambda}_y-0.2)+\bar{\lambda}_y^2]=0.5x[1+0.21x(1.259-0.2)+1.259^2]=1.404$
 $\chi_y=1/[\Phi_y+\sqrt{(\Phi_y^2-\bar{\lambda}_y^2)}]=1/[1.404+\sqrt{(1.404^2-1.259^2)}]=0.494 \leq 1$ $\chi_y=0.494$
 $z-z$ buckling curve: b , imperfection factor: $\alpha_z=0.34$, $\chi_z=0.263$
 $\Phi_z=0.5[1+\alpha_z(\bar{\lambda}_z-0.2)+\bar{\lambda}_z^2]=0.5x[1+0.34x(1.758-0.2)+1.758^2]=2.310$
 $\chi_z=1/[\Phi_z+\sqrt{(\Phi_z^2-\bar{\lambda}_z^2)}]=1/[2.310+\sqrt{(2.310^2-1.758^2)}]=0.263 \leq 1$ $\chi_z=0.263$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi\leq 1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.263$ (EC3 Eq.6.49)
 $N_{b,rd}=\chi\cdot A\cdot f_y/\gamma_{M1}=0.263x[10^{-3}]x8446x275/1.00=610.86\text{ kN}$ (EC3 Eq.6.47)
 $N_{c,ed}=49.76\text{ kN} < 610.86\text{ kN}=N_{b,rd}$, Is verified
 $N_{c,ed}/N_{b,rd}=49.76/610.86=0.081<1$

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

$M_{y,ed}=207.70\text{ kN}$, $L=18.000\text{m}$, $L_{cr,y}=18.000\text{m}$, $L_{cr,z}=6.000\text{m}$, $L_{t,lt}=6.000\text{m}$
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961
 $M_{cr}=C_1\cdot[\pi^2EI_z/(kL)^2]\{[\sqrt{(k_z/k_w)^2(I_w/I_z)+(kL)^2GI_t/(\pi^2EI_z)}+(C_2\cdot z_g-C_3\cdot z_j)^2]-(C_2\cdot z_g-C_3\cdot z_j)\}$
Method of computation C_1, C_2, C_3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
 $\mu=Mo/M=qL^2/8M=64.1/207.7=0.31$, $\psi=M_b/M_a=-152.6/207.7=-0.73$, $C_1=1.771$, $C_2=0.180$
 $G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4\text{ N/mm}^2$
 $k\cdot L=6000\text{mm}$, $z_g=h/2=400/2=200\text{mm}$, $z_j=0\text{mm}$ (EN1993:2002 Eq.C.11)
 $k_y=1.0$, $k_z=1.0$, $k_w=1.0$, $C_1=1.771$, $C_2=0.180$, $C_3=0.000$
 $M_{cr}=[10^{-6}]1.771x[\pi^2x2.1x10^5x13.180x10^6/6000^2]$
 $x\{[(1.0/1.0)^2x(490.05x10^9/13.180x10^6)$
 $+6000^2x8.1x10^4x0.511x10^6/(\pi^2x2.1x10^5x13.180x10^6)$
 $+(0.180x200)^2]^{0.5}-(0.180x200)\}=361.1\text{ kNm}$

 $\bar{\lambda}_{lt}=\sqrt{(W_{pl,y}\cdot f_y/M_{cr})}=\sqrt{[10^{-6}]x1307.0x10^3x275/361.1}=0.998$ (EC3 Eq.6.56)
 $h/b=400/180=2.22>2.00$ buckling curve: c
imperfection factor: $\alpha_{lt}=0.49$, $\beta=0.75$, $\chi_{lt}=0.641$ (T.6.3, T.6.5, Fig.6.4)
 $\Phi_{lt}=0.5[1+\alpha_{lt}(\bar{\lambda}_{lt}-\bar{\lambda}_{lt0})+\beta\bar{\lambda}_{lt}^2]=0.5x[1+0.49x(0.998-0.40)+0.75x0.998^2]=1.020$
 $\chi_{lt}=1/[\Phi_{lt}+\sqrt{(\Phi_{lt}^2-\beta\bar{\lambda}_{lt}^2)}]=1/[1.020+\sqrt{(1.020^2-0.75x1.020^2)}]=0.641$
Reduction factor $\chi_{lt}=1/[\Phi_{lt}+\sqrt{(\Phi_{lt}^2-\beta\bar{\lambda}_{lt}^2)}]$, $\chi_{lt}\leq 1.0$, $1/\bar{\lambda}_{lt}^2$, $\chi_{lt}=0.641$ (Eq.6.57)

 $\chi_{lt,mod}=\chi_{lt}/f$, $\chi_{lt,mod}\leq 1$, $\chi_{lt,mod}\leq 1/\bar{\lambda}_{lt}^2=1/0.998^2=1.00$ (EC3 §6.3.2.3(2), Eq.6.58)
 $K_c=1/(1.33-0.33\psi)=0.752$, $\psi=0.00$ (EC3 Tab.6.6)
 $f=1-0.5(1-K_c)[1-2.0(\bar{\lambda}_{lt}-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.998-0.8)^2]=0.886$, $f\leq 1.0$
 $\chi_{lt,mod}=\chi_{lt}/f=0.641/0.886=0.724$, $\chi_{lt,mod}\leq 1.0$, $\chi_{lt,mod}\leq 1.00$, $\chi_{lt,mod}=0.724$

 $M_{b,rd}=\chi_{lt}\cdot W_{pl,y}\cdot f_y/\gamma_{M1}=0.724x[10^{-6}]x1307.0x10^3x275/1.00=260.22\text{ kNm}$ (EC3 Eq.6.55)
 $M_{y,ed}=207.70\text{ kNm} < 260.22\text{ kNm}=M_{b,rd}$, Is verified
 $M_{y,ed}/M_{b,rd}=207.70/260.22=0.798<1$

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

$N_{ed}=49.76\text{ kN}$, $M_{y,ed}=207.70\text{ kNm}$

 $N_{ed}/(\chi_y\cdot N_{rk}/\gamma_{M1})+k_{yy}\cdot M_{y,ed}/(\chi_{LT}\cdot M_{y,rk}/\gamma_{M1})\leq 1$ (EC3 Eq.6.61)
 $N_{ed}/(\chi_z\cdot N_{rk}/\gamma_{M1})+k_{zy}\cdot M_{y,ed}/(\chi_{LT}\cdot M_{y,rk}/\gamma_{M1})\leq 1$ (EC3 Eq.6.62)
 $N_{rk}=A\cdot f_y=[10^{-3}]x8446x275=2322.6\text{ kN}$ (Tab.6.7)
 $M_{y,rk}=W_{pl,y}\cdot f_y=[10^{-6}]x1307.0x10^3x275=359.4\text{ kNm}$
 $\chi_y\cdot N_{rk}/\gamma_{M1}=\chi_y\cdot A\cdot f_y/\gamma_{M1}=0.494x[10^{-3}]x8446x275/1.00=1147.4\text{ kN}$
 $\chi_z\cdot N_{rk}/\gamma_{M1}=\chi_z\cdot A\cdot f_y/\gamma_{M1}=0.263x[10^{-3}]x8446x275/1.00=610.9\text{ kN}$
 $\chi_{LT}\cdot M_{y,rk}/\gamma_{M1}=\chi_{LT}\cdot W_{pl,y}\cdot f_y/\gamma_{M1}=0.724x[10^{-6}]x1307.0x10^3x275/1.00=260.2\text{ kNm}$

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

$k_{yy}=C_{my}\cdot C_{mLT}(\mu_y/(1-N_{ed}/N_{cr,y}))(1/C_{yy})$, $\mu_y=(1-N_{ed}/N_{cr,y})/(1-\chi_y\cdot N_{ed}/N_{cr,y})$ (EC3 Tab.A.1)
 $k_{zy}=C_{my}\cdot C_{mLT}(\mu_z/(1-N_{ed}/N_{cr,y}))(1/C_{zy})0.60\sqrt{(w_y/w_z)}$, $\mu_z=(1-N_{ed}/N_{cr,z})/(1-\chi_z\cdot N_{ed}/N_{cr,z})$

$N_{cr,y} = \pi^2 EI_y / l_{cr,y}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 231.30 \times 10^6 / 18000^2 = 1480 \text{ kN}$
 $N_{cr,z} = \pi^2 EI_z / l_{cr,z}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 13.180 \times 10^6 / 6000^2 = 759 \text{ kN}$
 $N_{cr,t} = (1 / i_p^2) \times (G \cdot I_t + \pi^2 EI_w / L_{cr,t}^2)$ (EC3 NCCI SN003b-EN-EU)
 $N_{cr,t} = [10^{-3}] \times (1 / 170^2) [80769 \times 0.511 \times 10^6 + \pi^2 \times 210000 \times 490.05 \times 10^9 / 6000^2] = 2400 \text{ kN}$

$\mu_y = (1 - N_{ed} / N_{cr,y}) / (1 - \chi_y \cdot N_{ed} / N_{cr,y}) = (1 - 49.8 / 1480) / (1 - 0.494 \times 49.8 / 1480) = 0.983$
 $\mu_z = (1 - N_{ed} / N_{cr,z}) / (1 - \chi_z \cdot N_{ed} / N_{cr,z}) = (1 - 49.8 / 759) / (1 - 0.263 \times 49.8 / 759) = 0.951$
 $alt = 1 - I_t / I_y > 0 = 1 - 0.511 \times 10^6 / 231.30 \times 10^6 = 0.998$ (EC3 Annex A.1)

$w_y = W_{pl,y} / W_{el,y} < 1.50$, $w_y = 1.307 \times 10^6 / 1.156 \times 10^6 = 1.131 < 1.50$ (EC3 Annex A.1)
 $w_z = W_{pl,z} / W_{el,z} < 1.50$, $w_z = 0.229 \times 10^6 / 0.146 \times 10^6 = 1.564 > 1.50$, $w_z = 1.50$
 $n_{pl} = N_{ed} / (N_{rk} / \gamma_{M1}) = 49.76 / (2322.60 / 1.00) = 0.021$

$\bar{\lambda}_{max} = \max(1.259, 1.758) = 1.760$ (EC3 Annex A.1)
 $M_{cr,o} = (1.00 / 1.77) \times 361.10 = 203.9$, $C1 = 1.00$
 $\bar{\lambda}_o = \sqrt{([10^{-6}] \times 1307.0 \times 10^3 \times 275 / 203.9)} = 1.330$
 $\bar{\lambda}_{o,lim} = 0.2 \sqrt{C1 [(1 - N_{ed} / N_{cr,z}) (1 - N_{ed} / N_{cr,t})]^{0.25}}$ (EC3 Annex A.1)
 $\bar{\lambda}_{o,lim} = 0.2 \sqrt{1.771 [(1 - 49.8 / 759) (1 - 49.8 / 2400)]^{0.25}} = 0.260$
 $\varepsilon_y = (M_y, ed / N_{ed}) (A / W_{el}) = ([10^3] \times 207.70 / 49.76) \times (8446.0 / 1156.0 \times 10^3) = 30.50$

$C_{my,o} = 0.79 + 0.21\psi + 0.36(\psi - 0.33) \times (49.76 / 1480.0) = 1.008$, ($\psi = 1.00$) (EC3 Annex A, T.A.1)
 $\bar{\lambda}_o = 1.330 > \bar{\lambda}_{o,lim} = 0.260$
 $C_{my} = C_{my,o} + (1 - C_{my,o}) (\sqrt{\varepsilon_y \cdot alt}) / (1 + \sqrt{\varepsilon_y \cdot alt}) =$
 $= 1.008 + (1 - 1.008) \times (\sqrt{30.495 \times 0.998}) / (1 + \sqrt{30.495 \times 0.998}) = 1.001$
 $C_{m1t} = C_{my}^2 \cdot alt / \sqrt{[(1 - N_{ed} / N_{cr,z}) (1 - N_{ed} / N_{cr,t})]} > 1$
 $C_{m1t} = 1.001^2 \times 0.998 / \sqrt{[(1 - 49.8 / 759.0) (1 - 49.8 / 2400.0)]} = 1.045$, $C_{m1t} = 1.045$

$C_{yy} = 1 + (w_y - 1) [(2 - 1.6 C_{my}^2 \cdot \bar{\lambda}_{max} / w_y - 1.6 C_{my}^2 \cdot \bar{\lambda}_{max}^2 / w_y) n_{pl} - blt] > W_{el,y} / W_{pl,y}$ (Annex A, T.A.1)
 $blt = 0.5 alt \cdot \bar{\lambda}_o^2 [M_y, ed / (\chi_{lt} \cdot M_{pl,y}, rd)] (M_z, ed / M_{pl,z}, rd) =$
 $= 0.5 \times 0.998 \times 1.330^2 [0.0 / (0.724 \times 317.9)] (0.0 / 40.3) = 0.000$
 $C_{yy} = 1 + (1.131 - 1) [(2 - 1.6 \times 1.001^2 \times 1.760 / 1.131 - 1.6 \times 1.001^2 \times 1.760^2 / 1.131) \times 0.021 - 0.000] = 0.987$
 $C_{yy} > 1156.0 \times 10^3 / 1307.0 \times 10^3 = 0.884$, $C_{yy} = 0.987$

$C_{zy} = 1 + (w_y - 1) [(2 - 14.0 C_{my}^2 \cdot \bar{\lambda}_{max}^2 / w_y^5) n_{pl} - dlt] > 0.6 \sqrt{(w_y / w_z)} (W_{el,y} / W_{pl,y})$ (Annex A, T.A.1)
 $dlt = 2 alt \cdot [\bar{\lambda}_o / (0.1 + \bar{\lambda}_z^4)] [M_y, ed / (C_{my} \cdot \chi_{lt} \cdot M_{pl,y}, rd)] [M_z, ed / (C_{mz} \cdot M_{pl,z}, rd)] =$
 $= 20.998 \times [1.330 / (0.1 + 1.758^4)] [0.0 / (1.001 \times 0.724 \times 317.9)] [0.0 / (0.000 \times 40.3)] = 0.000$
 $C_{zy} = 1 + (1.131 - 1) [(2 - 14.0 \times 1.001^2 \times 1.760^2 / 1.131^5) 0.021 - 0.000] = 0.941$
 $C_{zy} > 0.6 \sqrt{(1.131 / 1.500)} (1156.0 \times 10^3 / 1307.0 \times 10^3) = 0.461$, $C_{zy} = 0.941$

$C_{yy} = 0.987$, $C_{zy} = 0.941$ (Annex A, T.A.1)
 $k_{yy} = 1.001 \times 1.045 \times 0.983 / (1 - 49.76 / 1480.0) \times (1 / 0.987) = 1.078$
 $k_{zy} = 1.001 \times 1.045 \times 0.951 / (1 - 49.76 / 1480.0) \times (1 / 0.941) \times 0.6 \sqrt{(1.131 / 1.500)} = 0.570$

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

$N_{ed} / (\chi_y \cdot N_{rk} / \gamma_{M1}) + k_{yy} \cdot M_y, ed / (\chi_{LT} \cdot M_y, rk / \gamma_{M1}) =$ (EC3 Eq.6.61)
 $49.8 / (0.494 \times 2322.6 / 1.00) + 1.078 \times 207.7 / (0.724 \times 359.4 / 1.00) = 0.043 + 0.860 = 0.904$
 $0.904 < 1.000$, Is verified

$N_{ed} / (\chi_z \cdot N_{rk} / \gamma_{M1}) + k_{zy} \cdot M_y, ed / (\chi_{LT} \cdot M_y, rk / \gamma_{M1}) =$ (EC3 Eq.6.62)
 $49.8 / (0.263 \times 2322.6 / 1.00) + 0.570 \times 207.7 / (0.724 \times 359.4 / 1.00) = 0.081 + 0.455 = 0.536$
 $0.536 < 1.000$, Is verified

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

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

Ned = 5.5 kN

Ved = 10.9 kN

Myed = 40.7 kNm, Mzed = 0.0 kNm

Rafter length $L_r = 18000 \text{ mm}$

Buckling length, In-plane buckling $L_{cr,y} = 18000 \text{ mm}$ (System length)

Buckling length, Out-of-plane buckling $L_{cr,z} = 6000 \text{ mm}$ (Torsional restrains of rafters)

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

$N_{c,ed}=5.52 \text{ kN}$, $L_{cr,y}=18.000 \text{ m}$, $L_{cr,z}=6.000 \text{ m}$

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

Buckling lengths: $L_{cr,y}=1.000 \times 18000=18000 \text{ mm}$, $L_{cr,z}=0.333 \times 18000=6000 \text{ mm}$

Non-dimensional slenderness (Cross-section Class: 1)

(EC3 §6.3.1.3)

$$\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (18000 / 165.5) \times (1 / 86.39) = 1.259$$

$$\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (6000 / 39.5) \times (1 / 86.39) = 1.758$$

$$\lambda_1 = \pi \sqrt{E / f_y} = 93.9 \text{ } \varepsilon = 86.39, \quad \varepsilon = \sqrt{(235 / f_y)} = 0.92$$

$$h/b = 400/180 = 2.22 > 1.20, \quad t_f = 13.5 \text{ mm} \leq 40 \text{ mm}$$

$$y-y \text{ buckling curve: } a, \text{ imperfection factor: } \alpha_y = 0.21, \quad \chi_y = 0.494$$

(T.6.2, T.6.1, Fig.6.4)

$$\Phi_y = 0.5 [1 + \alpha_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.5 \times [1 + 0.21 \times (1.259 - 0.2) + 1.259^2] = 1.404$$

$$\chi_y = 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}] = 1 / [1.404 + \sqrt{(1.404^2 - 1.259^2)}] = 0.494 \leq 1 \quad \chi_y = 0.494$$

$$z-z \text{ buckling curve: } b, \text{ imperfection factor: } \alpha_z = 0.34, \quad \chi_z = 0.263$$

$$\Phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = 0.5 \times [1 + 0.34 \times (1.758 - 0.2) + 1.758^2] = 2.310$$

$$\chi_z = 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}] = 1 / [2.310 + \sqrt{(2.310^2 - 1.758^2)}] = 0.263 \leq 1 \quad \chi_z = 0.263$$

$$\text{Reduction factor } \chi = 1 / [\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}], \quad \chi \leq 1.0, \quad \Phi = 0.5 [1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2], \quad \chi = 0.263$$

(EC3 Eq.6.49)

$$N_{b,rd} = \chi \cdot A \cdot f_y / \gamma_{M1} = 0.263 \times [10^{-3}] \times 8446 \times 275 / 1.00 = 610.86 \text{ kN}$$

(EC3 Eq.6.47)

$$N_{c,ed} = 5.52 \text{ kN} < 610.86 \text{ kN} = N_{b,rd}, \quad \text{Is verified}$$

$$N_{c,ed} / N_{b,rd} = 5.52 / 610.86 = 0.009 < 1$$

14.13. Lateral torsional buckling, Rafter-Uplift conditions (ULS)

(EN1993-1-1, §6.3.2)

$M_{y,ed}=40.69 \text{ kNm}$, $L=18.000 \text{ m}$, $L_{cr,y}=18.000 \text{ m}$, $L_{cr,z}=6.000 \text{ m}$, $L_{cr,lt}=6.000 \text{ m}$

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

Hogging

$$k \cdot L = 6000 \text{ mm}, \quad z_g = -200 \text{ mm}, \quad z_j = 0 \text{ mm}$$

(EN1993:2002 Eq.C.11)

$$k_y = 1.0, \quad k_z = 1.0, \quad k_w = 1.0, \quad C1 = 1.000, \quad C2 = 0.000, \quad C3 = 1.000$$

$$M_{cr} = [10^{-6}] 1.000 \times [\pi^2 \times 2.1 \times 10^5 \times 13.180 \times 10^6 / 6000^2]$$

$$\times \{ [(1.0/1.0)^2 \times (490.05 \times 10^9 / 13.180 \times 10^6)]$$

$$+ 6000^2 \times 8.1 \times 10^4 \times 0.511 \times 10^6 / (\pi^2 \times 2.1 \times 10^5 \times 13.180 \times 10^6)]^{0.5} \} = 229.6 \text{ kNm}$$

$$\bar{\lambda}_{lt} = \sqrt{(W_{pl,y} \cdot f_y / M_{cr})} = \sqrt{[10^{-6}] \times 1307.0 \times 10^3 \times 275 / 229.6} = 1.251$$

(EC3 Eq.6.56)

$$h/b = 400/180 = 2.22 > 2.00 \text{ buckling curve: } c$$

$$\text{imperfection factor: } \alpha_{lt} = 0.49, \quad \beta = 0.75, \quad \chi_{lt} = 0.499$$

(T.6.3, T.6.5, Fig.6.4)

$$\Phi_{lt} = 0.5 [1 + \alpha_{lt} (\bar{\lambda}_{lt} - 0.2) + \bar{\lambda}_{lt}^2] = 0.5 \times [1 + 0.49 \times (1.251 - 0.2) + 1.251^2] = 1.296$$

$$\chi_{lt} = 1 / [\Phi_{lt} + \sqrt{(\Phi_{lt}^2 - \bar{\lambda}_{lt}^2)}] = 1 / [1.296 + \sqrt{(1.296^2 - 1.251^2)}] = 0.499$$

$$\text{Reduction factor } \chi_{lt} = 1 / [\Phi_{lt} + \sqrt{(\Phi_{lt}^2 - \bar{\lambda}_{lt}^2)}], \quad \chi_{lt} \leq 1.0, \quad 1 / \bar{\lambda}_{lt}^2, \quad \chi_{lt} = 0.499$$

(Eq.6.57)

$$\chi_{lt,mod} = \chi_{lt} / f, \quad \chi_{lt,mod} \leq 1, \quad \chi_{lt,mod} \leq 1 / \bar{\lambda}_{lt}^2 = 1 / 1.251^2 = 0.64$$

(EC3 §6.3.2.3(2), Eq.6.58)

$$K_c = 1 / (1.33 - 0.33\psi) = 0.752, \quad \psi = 0.00$$

(EC3 Tab.6.6)

$$f = 1 - 0.5(1 - K_c)[1 - 2.0(\bar{\lambda}_{lt} - 0.8)^2] = 1 - 0.5 \times (1 - 0.752)[1 - 2.0 \times (1.251 - 0.8)^2] = 0.926, \quad f \leq 1.0$$

$$\chi_{lt,mod} = \chi_{lt} / f = 0.499 / 0.926 = 0.539, \quad \chi_{lt,mod} \leq 1.0, \quad \chi_{lt,mod} \leq 0.64, \quad \chi_{lt,mod} = 0.539$$

$$M_{b,rd} = \chi_{lt} \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 0.539 \times [10^{-6}] \times 1307.0 \times 10^3 \times 275 / 1.00 = 193.73 \text{ kNm}$$

(EC3 Eq.6.55)

$$M_{y,ed} = 40.69 \text{ kNm} < 193.73 \text{ kNm} = M_{b,rd}, \quad \text{Is verified}$$

$$M_{y,ed} / M_{b,rd} = 40.69 / 193.73 = 0.210 < 1$$

15. Haunch verification (Ultimate Limit State)

(EN1993-1-1, §6)

The haunch is fabricated by cutting and welding of an IPE 400 section - S 275

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

at haunch end	at haunch-middle	at haunch-start
Ned = 49.8 kN	Ned = 49.8 kN	Ned = 49.8 kN
Ved = 128.5 kN	Ved = 115.6 kN	Ved = 102.8 kN
Myed = 335.1 kNm	Myed = 250.9 kNm	Myed = 152.6 kNm

Buckling length, In-plane buckling $L_{cr,y}=1800\text{mm}$ Buckling length, Out-of-plane buckling $L_{cr,z}=1800\text{mm}$

Maximum design values. Verification for load case: Seismic loading

at haunch end	at haunch-middle	at haunch-start
Ned = 20.8 kN	Ned = 20.8 kN	Ned = 20.8 kN
Ved = 48.8 kN	Ved = 48.8 kN	Ved = 48.8 kN
Myed = 147.4 kNm	Myed = 135.7 kNm	Myed = 120.6 kNm

15.1. Classification of steel cross-section, at haunch end

(EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{y,ed}/W_{el,y} \pm M_{z,ed}/W_{el,z}$

$$\sigma = [10^3]49.80/11508 \pm [10^6]335.10/2706.7 \times 10^3 \pm [10^6]0.00/146.3 \times 10^3$$

$$\sigma_1 = 128 \text{ N/mm}^2, \sigma_2 = -119 \text{ N/mm}^2 \text{ (compression positive)}$$

Web

$$c = 800.0 - 2 \times 13.5 - 2 \times 21.0 = 731.0 \text{ mm}, t = 8.6 \text{ mm}, c/t = 731.0/8.6 = 85.00$$

$$S 275, t = 8.6 \leq 40 \text{ mm}, f_y = 275 \text{ N/mm}^2, \epsilon = (235/275)^{0.5} = 0.92$$

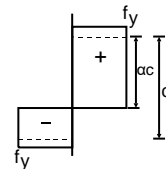
Position of neutral axis for combined Bending and compression

$$N_{ed}/(2t_w \cdot f_y / \gamma_{M0}) = 49800 / (2 \times 8.6 \times 275 / 1.00) = 10.5 \text{ mm}$$

$$\alpha = (731.0/2 + 10.5) / 731.0 = 0.514 > 0.5$$

$$c/t = 85.00 > 456 \times 0.92 / (13 \times 0.514 - 1) = 73.77$$

The web is not class 1 or 2

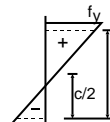


$$\sigma = N_{ed}/A \pm M_{y,ed} \cdot (0.5d)/I_y, \sigma_1 = 117 \text{ N/mm}^2, \sigma_2 = -109 \text{ N/mm}^2$$

$$\psi = -109/117 = -0.930 > -1$$

$$c/t = 85.00 \leq 42 \times 0.92 / (0.67 + 0.33 \times -0.930) = 106.42$$

The web is class 3 (EN1993-1-1, Tab.5.2)

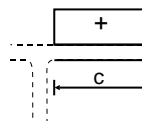
**Flange**

$$c = 180.0/2 - 8.6/2 - 21.0 = 64.7 \text{ mm}, t = 13.5 \text{ mm}, c/t = 64.7/13.5 = 4.79$$

$$S 275, t = 13.5 \leq 40 \text{ mm}, f_y = 275 \text{ N/mm}^2, \epsilon = (235/275)^{0.5} = 0.92$$

$$c/t = 4.79 \leq 9 \times 0.92 = 8.28$$

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

**Overall classification of cross-section is Class 3, Bending and compression $N_{c,ed} + M_{y,ed}$** **15.2. Resistance of cross-section, at haunch end (Ultimate Limit State)**

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

$$N_{c,ed} = 49.80 \text{ kN}$$

$$\text{Compression Resistance } N_{pl,rd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 11508 \times 275 / 1.00 = 3164.65 \text{ kN}$$

$$N_{ed} = 49.80 \text{ kN} < 3164.65 \text{ kN} = N_{c,rd} = N_{pl,rd}, \text{ Is verified}$$

$$N_{ed}/N_{c,rd} = 49.80/3164.65 = 0.016 < 1$$

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

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

$$M_{y,ed} = 335.10 \text{ kNm}$$

$$\text{Bending Resistance } M_{y,rd} = W_{el,y} \cdot f_y / \gamma_{M0} = [10^{-6}] \times 2706.7 \times 10^3 \times 275 / 1.00 = 744.34 \text{ kNm}$$

$$M_{y,ed} = 335.10 \text{ kNm} < 744.34 \text{ kNm} = M_{y,rd} = M_{pl,y,rd}, \text{ Is verified}$$

$$M_{y,ed}/M_{y,rd} = 335.10/744.34 = 0.450 < 1$$

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

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

Vz.ed=128.50 kN

$$A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 11508 - 2 \times 180.0 \times 13.5 + (8.6 + 2 \times 21.0) \times 13.5 = 7331 \text{ mm}^2 \quad (\text{EC3 §6.2.6.3})$$

$$A_v = 7331 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (800.0 - 2 \times 13.5) \times 8.6 = 1.00 \times 786.5 \times 8.6 = 6764 \text{ mm}^2$$

$$\text{Plastic Shear Resistance } V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 7331 \times (275 / 1.73) / 1.00 = 1163.94 \text{ kN}$$

$$V_{z,ed} = 128.50 \text{ kN} < 1163.94 \text{ kN} = V_{z,rd} = V_{pl,z,rd}, \text{ Is verified}$$

$$V_{z,ed} / V_{z,rd} = 128.50 / 1163.94 = 0.110 < 1$$

$$h_w / t_w = (800.0 - 2 \times 13.5) / 8.6 = 786.5 / 8.6 = 91.45 > 72 \times 0.92 / 1.00 = 72 \varepsilon / \eta = 66.24 \quad (\eta = 1.00)$$

$$S_{275}, t = 8.6 \leq 40 \text{ mm}, f_y = 275 \text{ N/mm}^2, \varepsilon = (235 / 275)^{0.5} = 0.92$$

Shear buckling resistance must be verified (EC3 §6.2.6.6)

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

$$\bar{\lambda}_w = (731.0 / 8.6) / (37.4 \times 0.92 \times \sqrt{(5.34)}) = 1.069, K_T = 5.34 \quad (\text{EC3-1-5 §5, Eq.5.6, A.3})$$

$$0.83 / \eta \leq \bar{\lambda}_w = 1.069 < 1.08, \chi_v = 0.83 / 1.069 = 0.776 \quad (\eta = 1.00) \quad (\text{EC3-1-5 Tab.5.1})$$

$$V_{b,rd} = \chi_v \cdot f_{yw} \cdot h_w \cdot t_w / (\sqrt{3} \gamma_{M1}) = 0.001 \times 275 \times 0.776 \times 731.0 \times 8.6 / (1.73 \times 1.00) = 774.98 \text{ kN} \quad (\text{EC3-1-5 Tab.5.1})$$

$$V_{ed} = 129 \text{ kN} < 775 = V_{b,rd} \text{ kN}, \text{ Is verified}$$

$$V_{ed} / V_{b,rd} = 128.50 / 774.98 = 0.166 < 1$$

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

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

N.ed= 49.80kN (Compression), Vz.ed= 128.50kN, My.ed= 335.10kNm

$$N_{pl,rd} = 3164.65 \text{ kN}, M_{el,y,rd} = 744.34 \text{ kNm}, V_{pl,z,rd} = 774.98 \text{ kN}$$

$$N_{ed} = 49.80 \text{ kN} \leq 0.25 \times 3164.65 = 0.25 \times N_{pl,rd} = 791.16 \text{ kN}$$

$$N_{ed} = 49.80 \text{ kN} \leq [10^{-3}] \times 0.5 \times 786.5 \times 8.6 \times 275 / 1.00 = 0.5 h_w \cdot t_w \cdot f_y / \gamma_{M0} = 930.04 \text{ kN}$$

$$n = N_{ed} / N_{pl,rd} = 50 / 3165 = 0.016$$

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

$$V_{ed} = 128.50 \text{ kN} \leq 0.50 \times 774.98 = 0.50 \times V_{pl,rd} = 387.49 \text{ kN}$$

Effect of shear force is neglected (EC3 §6.2.8.2)

Maximum and minimum cross-section stresses $\sigma = N_{ed} / A_{el} \pm M_{y,ed} / W_{el,y} \pm M_{z,ed} / W_{el,z}$

$$\sigma = [10^3] 0.00 / 11508 \pm [10^6] 335.10 / 2706.7 \times 10^3 \pm [10^6] 0.00 / 146.3 \times 10^3$$

$$\sigma_1 = 124 \text{ N/mm}^2, \sigma_2 = -124 \text{ N/mm}^2 \text{ (compression positive)}$$

$$\sigma_{x,ed} = 124 < 275 / 1.00 = 275 = f_y / \gamma_{M0} \text{ N/mm}^2, \text{ Is verified} \quad (\text{EC3 Eq.6.42})$$

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

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

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

Properties of equivalent T-section

$$\text{Depth of cross section } h_f = 133 \text{ mm}$$

$$\text{Width of cross section } b_f = 180 \text{ mm}$$

$$\text{Web thickness } t_w = 8.60 \text{ mm}$$

$$\text{Flange thickness } t_f = 13.50 \text{ mm}$$

$$\text{Area } A_f = 3461 \text{ mm}^2$$

$$\text{Second moment of area } I_{f,z} = 6.561 \times 10^6 \text{ mm}^4$$

$$\text{Radius of gyration } i_{f,z} = \sqrt{(6.561 \times 10^6 / 3461)} = 43.5 \text{ mm}$$

Compression in the T-section

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

$$N_{ed,f} = N_{ed} \cdot A_f / A + M_{ed} \cdot A_f / W_{el,y} = 49.8 \times 3461 / 11508 + 335.1 \times 3461 \times 10^3 / 2706.7 \times 10^3 = 443.4 \text{ kN}$$

Maximum design values. Verification for load case: Seismic loading

$$N_{ed,f} = N_{ed} \cdot A_f / A + M_{ed} \cdot A_f / W_{el,y} = 20.8 \times 3461 / 11508 + 147.4 \times 3461 \times 10^3 / 2706.7 \times 10^3 = 194.7 \text{ kN}$$

$$N_{ed} = \max(443.4, 194.7) = 443.4 \text{ kN}$$

$$\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (1800 / 43.5) \times (1 / 86.39) = 0.479$$

$$f_{z-f,z} \text{ buckling curve: c, imperfection factor: } \alpha_{f,z} = 0.49, \chi_{f,z} = 0.855 \quad (\text{T.6.2, T.6.1, Fig.6.4})$$

$$\Phi_{f,z} = 0.5 [1 + \alpha_{f,z} (\bar{\lambda}_{f,z} - 0.2) + \bar{\lambda}_{f,z}^2] = 0.5 [1 + 0.49 \times (0.479 - 0.2) + 0.479^2] = 0.683$$

$$\chi_{f,z} = 1 / [\Phi_{f,z} + \sqrt{(\Phi_{f,z}^2 - \bar{\lambda}_{f,z}^2)}] = 1 / [0.683 + \sqrt{(0.683^2 - 0.479^2)}] = 0.855 \leq 1 \quad \chi_{f,z} = 0.855$$

$$N_{b,rd} = \chi_z \cdot A \cdot f_y / \gamma_{M1} = 0.855 \times 3461 \times 275 / 1.00 = 813.67 \text{ kN} \quad (\text{EC3 Eq.6.47})$$

$$N_{c,ed} = 443.41 \text{ kN} < 813.67 \text{ kN} = N_{b,rd}, \text{ Is verified}$$

$$N_{c,ed} / N_{b,rd} = 443.41 / 813.67 = 0.545 < 1$$

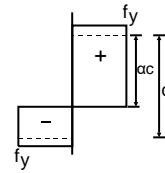
15.4. Classification of steel cross-section, at haunch-middle

(EN1993-1-1, §5.5)

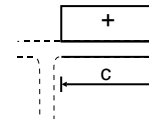
Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{y,ed}/W_{el,y} \pm M_{z,ed}/W_{el,z}$
 $\sigma = [10^3]49.80/9788 \pm [10^6]250.90/1842.8 \times 10^3 \pm [10^6]0.00/146.1 \times 10^3$
 $\sigma_1 = 141 \text{ N/mm}^2$, $\sigma_2 = -131 \text{ N/mm}^2$ (compression positive)

Web

$c = 600.0 - 2 \times 13.5 - 2 \times 21.0 = 531.0 \text{ mm}$, $t = 8.6 \text{ mm}$, $c/t = 531.0/8.6 = 61.74$
 $S 275$, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 Position of neutral axis for combined Bending and compression
 $N_{ed}/(2 \cdot t \cdot w \cdot f_y / \gamma_{M0}) = 49800 / (2 \times 8.6 \times 275 / 1.00) = 10.5 \text{ mm}$
 $\alpha = (531.0 / 2 + 10.5) / 531.0 = 0.520 > 0.5$
 $c/t = 61.74 \leq 396 \times 0.92 / (13 \times 0.520 - 1) = 63.27$
 The web is class 1 (EN1993-1-1, Tab.5.2)

Flange

$c = 180.0 / 2 - 8.6 / 2 - 21.0 = 64.7 \text{ mm}$, $t = 13.5 \text{ mm}$, $c/t = 64.7/13.5 = 4.79$
 $S 275$, $t = 13.5 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 $c/t = 4.79 \leq 9 \epsilon = 9 \times 0.92 = 8.28$
 The flange is class 1 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 1, Bending and compression $N_{c,ed} + M_{y,ed}$

15.5. Resistance of cross-section, at haunch-middle (Ultimate Limit State)

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

$N_{c,ed} = 49.80 \text{ kN}$

Compression Resistance $N_{pl,rd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 9788 \times 275 / 1.00 = 2691.64 \text{ kN}$

$N_{ed} = 49.80 \text{ kN} < 2691.64 \text{ kN} = N_{c,rd} = N_{pl,rd}$, Is verified

$N_{ed}/N_{c,rd} = 49.80/2691.64 = 0.019 < 1$

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

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

$M_{y,ed} = 250.90 \text{ kNm}$

Bending Resistance $M_{pl,y,rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = [10^{-6}] \times 2131.1 \times 10^3 \times 275 / 1.00 = 586.05 \text{ kNm}$

$M_{y,ed} = 250.90 \text{ kNm} < 586.05 \text{ kNm} = M_{y,rd} = M_{pl,y,rd}$, Is verified

$M_{y,ed}/M_{y,rd} = 250.90/586.05 = 0.428 < 1$

Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

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

$V_{z,ed} = 115.60 \text{ kN}$

$A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 9788 - 2 \times 180.0 \times 13.5 + (8.6 + 2 \times 21.0) \times 13.5 = 5611 \text{ mm}^2$

(EC3 §6.2.6.3)

$A_v = 5611 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (600.0 - 2 \times 13.5) \times 8.6 = 1.00 \times 586.5 \times 8.6 = 5044 \text{ mm}^2$

Plastic Shear Resistance $V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 5611 \times (275 / 1.73) / 1.00 = 890.85 \text{ kN}$

$V_{z,ed} = 115.60 \text{ kN} < 890.85 \text{ kN} = V_{z,rd} = V_{pl,z,rd}$, Is verified

$V_{z,ed}/V_{z,rd} = 115.60/890.85 = 0.130 < 1$

$h_w/t_w = (600.0 - 2 \times 13.5) / 8.6 = 586.5 / 8.6 = 68.20 > 72 \times 0.92 / 1.00 = 72 \epsilon / \eta = 66.24$ ($\eta = 1.00$)

$S 275$, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$

Shear buckling resistance must be verified

(EC3 §6.2.6.6)

Shear buckling resistance

(EC3 EN1993-1-5:2006, §5)

$\bar{\lambda}_w = (531.0 / 8.6) / (37.4 \times 0.92 \times \sqrt{5.34}) = 0.777$, $K_t = 5.34$

(EC3-1-5 §5, Eq.5.6, A.3)

$\bar{\lambda}_w = 0.777 < 0.83 / \eta$, $\chi_w = \eta = 1.000$ ($\eta = 1.00$)

(EC3-1-5 Tab.5.1)

$V_{b,rd} = \chi_w \cdot f_{yw} \cdot h_w \cdot t / (\sqrt{3} \gamma_{M1}) = 0.001 \times 275 \times 1.000 \times 531.0 \times 8.6 / (1.73 \times 1.00) = 725.07 \text{ kN}$

(EC3-1-5 Tab.5.1)

$V_{ed} = 116 \text{ kN} < 725 = V_{b,rd} \text{ kN}$, Is verified

$V_{ed}/V_{b,rd} = 115.60/725.07 = 0.159 < 1$

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

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

N.ed= 49.80kN (Compression), Vz.ed= 115.60kN, My.ed= 250.90kNm

Nplrd=2691.64kN, Mpl,y,rd=586.05kNm, Vpl,z,rd=725.07kN

Ned=49.80kN <= 0.25x2691.64=0.25xNplrd=672.91kN

Ned=49.80kN <= $[10^{-3}] \times 0.5 \times 586.5 \times 8.6 \times 275 / 1.00 = 0.5 \text{hw} \cdot \text{tw} \cdot \text{fy} / \gamma_{M0} = 693.54 \text{ kN}$

n=Ned/Nplrd=50/2692= 0.019

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

Ved=115.60kN <= 0.50x725.07=0.50xVpl,rd=362.54kN

Effect of shear force is neglected (EC3 §6.2.8.2)

My,ed= 250.90 kNm < 586.05 kNm =Mply,rd, Is verified

My,ed/Mply,rd= 250.90/586.05= 0.428<1

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

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

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

Properties of equivalent T-section

Depth of cross section hf = 100 mm

Width of cross section bf = 180 mm

Web thickness tw = 8.60 mm

Flange thickness tf = 13.50 mm

Area Af = 3174 mm²Second moment of area If,z = 6.561x10⁶ mm⁴Radius of gyration if,z = $\sqrt{(6.561 \times 10^6 / 3174)} = 45.5 \text{ mm}$ Compression in the T-section

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

Ned,f=Ned·Af/A+Med·Af/Wel,y=49.8x3174/9788+250.9x3174x10³/1842.8x10³=448.3kN

Maximum design values. Verification for load case: Seismic loading

Ned,f=Ned·Af/A+Med·Af/Wel,y=20.8x3174/9788+135.7x3174x10³/1842.8x10³=240.4kN

Ned=max(448.3,240.4)= 448.3 kN

 $\bar{\lambda}_z = \sqrt{(A \cdot \text{fy} / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (1800 / 45.5) \times (1 / 86.39) = 0.458$

f,z=f,z buckling curve:c, imperfection factor:αf,z=0.49, χf,z=0.866 (T.6.2,T.6.1, Fig.6.4)

 $\Phi_{f,z} = 0.5[1 + \alpha_{f,z}(\bar{\lambda}_{f,z} - 0.2) + \bar{\lambda}_{f,z}^2] = 0.5[1 + 0.49 \times (0.458 - 0.2) + 0.458^2] = 0.668$ $\chi_{f,z} = 1 / [\Phi_{f,z} + \sqrt{(\Phi_{f,z}^2 - \bar{\lambda}_{f,z}^2)}] = 1 / [0.668 + \sqrt{(0.668^2 - 0.458^2)}] = 0.866 <= 1 \quad \chi_{f,z} = 0.866$

Nb,rd=χz·A·fy/γM1= 0.866x3174x275/1.00=755.86kN (EC3 Eq.6.47)

Nc,ed= 448.33 kN < 755.86 kN =Nb,rd, Is verified

Nc,ed/Nb,rd= 448.33/755.86= 0.593<1

Connections16. Connection data

(EN1993-1-8)

16.1. Bolt connection data (eave, apex)

(EN1993-1-8)

Type of connection	End-plate connection, non-preloaded bolts	
Category of connection	Category A: Bearing type	(EC3-1-8 §3.4.1)
	Category D: Non-preloaded	(EC3-1-8 §3.4.2)
End Plate	Thickness $t_p=25$ mm, S 275	
Plate of Eave connection	180x935x25 mm, S 275	
Bolts	M24, Strength grade 10.9	
Bolt diameter	$d = 24$ mm	
Diameter of holes	$d_o = 26$ mm	
Nominal area	$\pi d^2/4 = \pi \times 24^2/4 = 452.4$ mm ²	
Tensile stress area	$A_s = 353.0$ mm ²	
Bolt strength grade	10.9, $f_{yb}=900$ N/mm ² , $f_{ub}=1000$ N/mm ²	(EC3-1-8 §3.1.1)

16.2. Edge distances and spacing of bolts (eave, apex)

(EN1993-1-8, §3.5, Tab.3.3)

Minimum edge distances	$e_1=1.2d_o=1.2 \times 26=32$ mm $e_2=1.2d_o=1.2 \times 26=32$ mm
Maximum edge distances	$e_1=4t+40=4 \times 24.0+40=137$ mm $e_2=4t+40=4 \times 24.0+40=137$ mm
Minimum spacing of bolts	$p_1=2.2d_o=2.2 \times 26=58$ mm $p_2=2.4d_o=2.4 \times 26=63$ mm
Maximum spacing of bolts	$p_1=\min(14t, 200)=\min(14 \times 24.0, 200)=200$ mm $p_2=\min(14t, 200)=\min(14 \times 24.0, 200)=200$ mm
Distance of plate edge to bolt line	$e_1=e_2=e_x=45$ mm
Distance of section edge to bolt line	$e_c=41$ mm
Distance of flange edge to bolt line	$e_f=45$ mm
Pitch between bolt rows	$p_1=p_3=p=90$ mm
Spacing between cross centers	$p_2=g=w=90$ mm
Flange to end-plate weld	$a_{tf} \geq 0.55t_f=0.55 \times 13.5=8$ mm
Web to end-plate weld	$a_w \geq 0.55t_w=0.55 \times 8.6=6$ mm

16.3. Design resistance of individual bolts (eave, apex)

(EC3-1-8 §3.6.1, Tab.3.4)

Bolt strength grade=10.9, $f_{ub}=1000$ N/mm ² , $A_s=353.0$ mm ² , $\gamma_{M2}=1.25$	
Tension resistance of bolts	$F_{t,rd}=k_2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($k_2=0.90$) $F_{t,rd}=[10^{-3}] \times 0.90 \times 1000 \times 353.0 / 1.25=254$ kN
Shear resistance of bolts	$F_{v,rd}=\alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($\alpha_v=0.50$) $F_{v,rd}=[10^{-3}] \times 0.50 \times 1000 \times 353.0 / 1.25=141$ kN

17. Eave connection**17.1. Basic data (Eave connection)**Design forces of connection (Eave connection)

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

Ned = -49.8 kN

Ved = 128.1 kN

Med = -335.2 kNm

Maximum design values for actions (Seismic loading)

Ned = -20.8 kN

Ved = 48.8 kN

Med = -141.0 kNm

17.2. Connection data (Eave connection)Bolt connection data

End Plate 180x935x25 mm, S 275

Bolts M24, Bolt strength grade 10.9

Number of Bolts top 2x3=6

bottom 2x1=2

Total number of bolts =8

Diameter of holes do = 26 mm

Shear plane of bolt through the threaded portion

Edge distances and spacing of bolts

Distance of plate edge to bolt line e1=e2=ex= 45 mm

Distance of section edge to bolt line ec= 41 mm

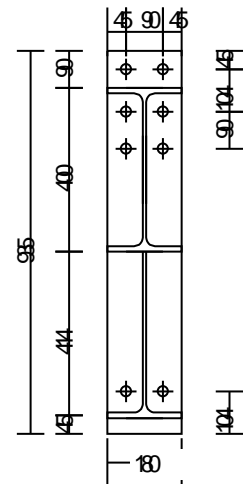
Distance of flange edge to bolt line ef= 45 mm

Pitch between bolt rows p1=p3=p= 90 mm

Spacing between cross centers p2=g=w= 90 mm

Flange to end-plate weld atf>= 0.55tf=0.55x13.5= 8 mm

Web to end-plate weld aw>= 0.55tw=0.55x 8.6= 6 mm

Compression stiffener at the bottom of haunch

Compression stiffener with thickness ts= 25.0 mm

17.3. Connection geometry of end-plate (Eave connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=45 mm, emin=45 mm

 $m_x, x = (90 - 8.6 - 2 \times 0.8 \times 6 \times \sqrt{2}) / 2 = 33.9 \text{ mm}$ $m_x, y = 45 - 0.8 \times 8 \times \sqrt{2} = 35.9 \text{ mm}$ $n_x, x = \text{emin} \leq 1.25m_x, x = \min(45.0, 1.25 \times 33.9) = 42.4 \text{ mm}$ $n_x, y = \text{emin} \leq 1.25m_x, y = \min(45.0, 1.25 \times 35.9) = 44.9 \text{ mm}$ $\min(m_x, x, m_x, y) = \min(33.9, 35.9) = 33.9 \text{ mm}$, $\max(m_x, x, m_x, y) = \max(33.9, 35.9) = 35.9 \text{ mm}$ $\min(n_x, x, n_x, y) = \min(42.4, 44.9) = 42.4 \text{ mm}$, $\max(n_x, x, n_x, y) = \max(42.4, 44.9) = 44.9 \text{ mm}$ **17.4. Effective lengths of end-plate (Eave connection)**

(EC3-1-8 §6.2.6.5 Tab.6.6)

Bolt-row outside tension flange of beam $l_{eff} = 2 \cdot \pi \cdot m_x = 2 \cdot \pi \cdot 35.9 = 225.6 \text{ mm}$ $= \pi \cdot m_x + w = \pi \cdot 35.9 + 90.0 = 202.8 \text{ mm}$ $= \pi \cdot m_x + 2e = \pi \cdot 35.9 + 2 \times 45.0 = 202.8 \text{ mm}$ $= 4m_x + 1.25e_x = 4 \times 35.9 + 1.25 \times 45.0 = 199.9 \text{ mm}$ $= e + 2m_x + 0.625e_x = 45.0 + 2 \times 35.9 + 0.625 \times 45.0 = 144.9 \text{ mm}$ $= 0.5b_p = 0.5 \times 180 = 90.0 \text{ mm}$ $= 0.5w + 2m_x + 0.625e_x = 0.5 \times 90.0 + 2 \times 35.9 + 0.625 \times 45.0 = 144.9 \text{ mm}$ $l_{eff, lb} = \min(225.6, 202.8, 202.8, 199.9, 144.9, 90.0, 144.9) = 90.0 \text{ mm}$ $l_{eff, lb} = 90.0 \text{ mm}$

Bolt next to tension flange alone

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 33.9 = 213.0 \text{ mm} \\
 &= \alpha \cdot m = 6.28 \times 33.9 = 213.0 \text{ mm} \quad (\lambda_1 = \lambda_2 = m / (m + e) = 0.43, \alpha = 6.28) \quad (\text{EC3-1-8 Fig.6.11}) \\
 l_{eff,2b} &= \min(213.0, 213.0) = 213.0 \text{ mm} \\
 l_{eff,2b} &= 213.0 \text{ mm}
 \end{aligned}$$

Bolt next to tension flange in a group

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 33.9 = 213.0 \text{ mm} \\
 &= \alpha \cdot m = 6.28 \times 33.9 = 213.0 \text{ mm} \quad (\lambda_1 = \lambda_2 = m / (m + e) = 0.43, \alpha = 6.28) \\
 &= \pi m + p = \pi \times 33.9 + 90.0 = 196.5 \text{ mm} \\
 &= 0.5p + \alpha \cdot m - (2m + 0.625e) = 0.5 \times 90.0 + 6.3 \times 33.9 - (2 \times 33.9 + 0.625 \times 45.0) = 162.1 \text{ mm} \\
 l_{eff,3b} &= \min(213.0, 213.0, 196.5, 162.1) = 162.1 \text{ mm} \\
 l_{eff,3b} &= 162.1 \text{ mm}
 \end{aligned}$$

Inner Bolt-row in a group

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 33.9 = 213.0 \text{ mm} \\
 &= 4m + 1.25e = 4 \times 33.9 + 1.25 \times 45.0 = 191.9 \text{ mm} \\
 &= 2p = 2 \times 90.0 = 180.0 \text{ mm} \\
 &= p = 90.0 \text{ mm} \\
 l_{eff,4b} &= \min(213.0, 191.9, 180.0, 90.0) = 90.0 \text{ mm} \\
 l_{eff,4b} &= 90.0 \text{ mm}
 \end{aligned}$$

17.5. End-Plate, Resistance of T-stub flange (Eave connection)

(EC3-1-8 §6.2.4.1, Tab.6.2)

Bolt-row outside tension flange of beam

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 25.0^2 \times 275 / 1.00 = 3.867 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4 M_{pl,1,rd} / m = [10^3] \times 4 \times 3.867 / 35.9 = 431 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2 M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 3.867 + 44.9 \times 2 \times 254) / (35.9 + 44.9) = 378 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(431, 378, 508) = 378 \text{ kN}
 \end{aligned}$$

Bolt next to tension flange alone

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 213.0 \times 25.0^2 \times 275 / 1.00 = 9.152 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4 M_{pl,1,rd} / m = [10^3] \times 4 \times 9.152 / 33.9 = 1080 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2 M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 9.152 + 42.4 \times 2 \times 254) / (33.9 + 42.4) = 522 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(1080, 522, 508) = 508 \text{ kN}
 \end{aligned}$$

Bolt next to tension flange in a group

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 162.1 \times 25.0^2 \times 275 / 1.00 = 6.965 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4 M_{pl,1,rd} / m = [10^3] \times 4 \times 6.965 / 33.9 = 822 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2 M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 6.965 + 42.4 \times 2 \times 254) / (33.9 + 42.4) = 465 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(822, 465, 508) = 465 \text{ kN}
 \end{aligned}$$

Inner Bolt-row in a group

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 25.0^2 \times 275 / 1.00 = 3.867 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4 M_{pl,1,rd} / m = [10^3] \times 4 \times 3.867 / 35.9 = 431 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2 M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 3.867 + 44.9 \times 2 \times 254) / (35.9 + 44.9) = 378 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(431, 378, 508) = 378 \text{ kN}
 \end{aligned}$$

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

(EC3-1-8 §6.2.6.7)

$$\begin{aligned}
 F_{c,fb,rd} &= M_{c,rd} / (h - t_f), \quad M_{c,rd} = W_{el,y} \cdot f_y / \gamma_{M0} \\
 W_{el,y} &= (180 \times 13.5 \times 786.5^2 + 8.6 \times 773.0^3 / 6) / 800 = 2706.5 \times 10^3 \text{ mm}^3 \\
 M_{c,rd} &= [10^{-6}] \times 2706.5 \times 10^3 \times 275 / 1.00 = 744 \text{ kNm}, \quad F_{c,fb,rd} = [10^3] \times 744 / 786.5 = 946 \text{ kN} \\
 F_{c,fb,rd,max} &= (1/0.8) b \cdot t \cdot f_y / \gamma_{M0} = (1/0.8) \times [10^{-3}] \times 180.0 \times 13.5 \times 275 / 1.00 = 835 \text{ kN} \quad (h > 600 \text{ mm}) \\
 F_{c,fb,rd} &= \min(946, 835) = 835 \text{ kN}
 \end{aligned}$$

17.7. Rafter web in tension (Eave connection)

(EC3-1-8 §6.2.6.8)

$F_{t,wb,rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_y / \gamma_{M0}$
 $b_{eff,t,wb} = \min(l_{eff,3b}, l_{eff,4b}) = \min(162.1, 90.0) = 90.0 \text{ mm}$
 $F_{t,wb,rd} = [10^{-3}] \times 90.0 \times 8.6 \times 275 / 1.00 = 213 \text{ kN}$

$\min F_{t,rd} = \min(378, 508, 465, 378, 213) = 213 \text{ kN}$

17.8. Connection geometry of column-side (Eave connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

$e = e_x = 45 \text{ mm}, e_{min} = 45 \text{ mm}$
 $m_x = (90 - 13.5 - 2 \times 0.8 \times 27) / 2 = 16.6 \text{ mm}$
 $m_x = 45 - 0.8 \times 8 \times \sqrt{2} = 35.9 \text{ mm}$
 $n_x = e_{min} \leq 1.25 m_x, x = \min(45.0, 1.25 \times 16.6) = 20.8 \text{ mm}$
 $n_x = e_{min} \leq 1.25 m_x, y = \min(45.0, 1.25 \times 35.9) = 44.9 \text{ mm}$
 $\min(m_x, x, m_x, y) = \min(16.6, 35.9) = 16.6 \text{ mm}, \max(m_x, x, m_x, y) = \max(16.6, 35.9) = 35.9 \text{ mm}$
 $\min(n_x, x, n_x, y) = \min(20.8, 44.9) = 20.8 \text{ mm}, \max(n_x, x, n_x, y) = \max(20.8, 44.9) = 44.9 \text{ mm}$

17.9. Effective lengths of column-side (Eave connection)

(EC3-1-8 §6.2.6.4 Tab.6.4)

End Bolt-row in a group

$l_{eff} = 2 \pi \cdot m = 2 \pi \times 16.6 = 104.3 \text{ mm}$
 $= \pi \cdot m + 2e_1 = \pi \times 16.6 + 2 \times 45.0 = 142.2 \text{ mm}$
 $= 4m + 1.25e = 4 \times 16.6 + 1.25 \times 45.0 = 122.7 \text{ mm}$
 $= 2m + 0.63e + e_1 = 2 \times 16.6 + 0.63 \times 45.0 + 45.0 = 106.3 \text{ mm}$
 $= \pi \cdot m + p = \pi \times 16.6 + 90.0 = 142.2 \text{ mm}$
 $= 2e_1 + p = 2 \times 45.0 + 90.0 = 180.0 \text{ mm}$
 $= 2m + 0.63e + 0.5p = 2 \times 16.6 + 0.63 \times 45.0 + 0.5 \times 90.0 = 106.3 \text{ mm}$
 $= e_1 + 0.5p = 45.0 + 0.5 \times 90.0 = 90.0 \text{ mm}$
 $l_{eff,1c} = \min(104.3, 142.2, 122.7, 106.3, 142.2, 180.0, 106.3, 90.0) = 90.0 \text{ mm}$
 $l_{eff,1c} = 90.0 \text{ mm}$

Inner Bolt-row in a group

$l_{eff} = 2 \pi \cdot m = 2 \pi \times 16.6 = 104.3 \text{ mm}$
 $= 4m + 1.25e = 4 \times 16.6 + 1.25 \times 45.0 = 122.7 \text{ mm}$
 $= 2p = 2 \times 90.0 = 180.0 \text{ mm}$
 $= p = 90.0 \text{ mm}$
 $l_{eff,2c} = \min(104.3, 122.7, 180.0, 90.0) = 90.0 \text{ mm}$
 $l_{eff,2c} = 90.0 \text{ mm}$

17.10. Column-Side, Resistance of T-stub flange (Eave connection)

(EC3-1-8 §6.2.4.1, Tab.6.2)

End Bolt-row in a group

$M_{pl,1,rd} = M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 24.0^2 \times 275 / 1.00 = 3.564 \text{ kNm}$
Mode 1 $F_{t,1,rd} = 4 M_{pl,1,rd} / m = [10^3] \times 4 \times 3.564 / 16.6 = 859 \text{ kN}$
Mode 2 $F_{t,2,rd} = (2 M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 3.564 + 20.8 \times 2 \times 254) / (16.6 + 20.8) = 473 \text{ kN}$
Mode 3 $F_{t,3,rd} = \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN}$
 $F_{t,rd} = \min(859, 473, 508) = 473 \text{ kN}$

Inner Bolt-row in a group

$M_{pl,1,rd} = M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 24.0^2 \times 275 / 1.00 = 3.564 \text{ kNm}$
Mode 1 $F_{t,1,rd} = 4 M_{pl,1,rd} / m = [10^3] \times 4 \times 3.564 / 16.6 = 859 \text{ kN}$
Mode 2 $F_{t,2,rd} = (2 M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 3.564 + 20.8 \times 2 \times 254) / (16.6 + 20.8) = 473 \text{ kN}$
Mode 3 $F_{t,3,rd} = \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN}$
 $F_{t,rd} = \min(859, 473, 508) = 473 \text{ kN}$

17.11. Column-web in transverse tension (Eave connection)

(EC3-1-8 §6.2.6.3)

$F_{t,wc,rd} = \omega \cdot b_{eff,t,wc} \cdot t_{wc} \cdot f_y / \gamma_{M0}$
 $\beta = 1, \omega = \omega_1 = 1 / \sqrt{[1 + 1.3 (b_{eff,c} \cdot t_{wc} / A_{vc})^2]}, b_{eff,c} = 90.0 \text{ mm}$ (EC3-1-8 §6.2.6.2, Tab.6.3)
 $\omega = 1 / \sqrt{[1 + 1.3 \times (90.0 \times 13.5 / 7000)^2]} = 0.98$
 $F_{t,wc,rd} = [10^{-3}] \times 0.98 \times 90.0 \times 13.5 \times 275 / 1.00 = 327 \text{ kN}$

17.12. Design resistance of compression stiffener (Eave connection)

(EC3-1-5 §9.1)

Compression stiffener at the bottom of haunch $t_s = 25.0$ mm

$$f_y = 275 \text{ N/mm}^2, \quad b_s = (180 - 13.5 - 2 \times 27.0) / 2 = 56.2 \text{ mm}, \quad t_s = 25.0 \text{ mm}, \quad t_w = 13.5 \text{ mm}, \quad \varepsilon = \sqrt{(235 / f_y)} = 0.92$$

$$A_{eff,s} = 2 \times 56.2 \times 25.0 + (2 \times 15 \times 0.92 \times 13.5 + 25.0) \times 13.5 = 8178 \text{ mm}^2 \quad (\text{EC3-1-5 §9.1(2)})$$

$$I_{eff,s} = \min(56.2, 14 \times 0.92 \times 25.0) = \min(56.2, 322.00) = 56.2 \text{ mm}, \quad (\text{EC3 Tab.5.2})$$

$$I_{eff,s} = (2 \times 56.2 + 13.5)^3 \times 25.0 / 12 = 4157.5 \times 10^3 \text{ mm}^4$$

$$i_{eff,s} = \sqrt{(4157.5 \times 10^3 / 8178)} = 22.5 \text{ mm}, \quad \lambda_1 = \pi \sqrt{(E / f_y)} = 93.9, \quad \varepsilon = 86.39$$

$$L_{cr} = 0.75 \times (400 - 2 \times 24.0) = 264.0 \text{ mm} \quad (\text{EC3-1-5 §9.4(2)})$$

$$\bar{\lambda} = L_{cr} / (i_{eff,s} \times \lambda_1) = 264.0 / (22.5 \times 86.39) = 0.14 \quad (\text{EC3 §6.3.1.3(1)})$$

$$\bar{\lambda} < 0.20, \quad \chi = 1.00 \quad (\text{EC3 §6.3.1.2.4})$$

$$F_{c,wc,rd} = \chi \cdot A_{eff,s} \cdot f_y / \gamma_{M1} = 1.000 \times 8178 \times 275 / 1.00 = 2249 \text{ kN} > F_{c,fb,rd} = 835 \text{ kN}$$

Compression stiffener, Is verified

17.13. Moment resistance of connection (Eave connection)

(EN1993-1-8, §6.2.7.2)

$$M_{j,rd} = \sum h_r \cdot F_{tr,rd} \quad (\text{EN1993-1-8, §6.2.7.2 Eq.6.25})$$

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

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

Force distribution in bolt rows

$$\text{Bolt-row 1, } h_r = 838.3 \text{ mm, } F_{t,rd} = 378 \text{ kN}$$

$$\text{Bolt-row 2, } h_r = 734.8 \text{ mm, } F_{t,rd} = 465 \text{ kN}$$

$$\text{Bolt-row 3, } h_r = 644.8 \text{ mm, } F_{t,rd} = 378 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} = 378 + 465 + 378 = 1221 \text{ kN}$$

End-plate in bending (EC3-1-8 §6.2.4.4)

Force distribution in bolt rows

$$\text{Bolt-row 1, } h_r = 838.3 \text{ mm, } F_{t,rd} = 473 \text{ kN}$$

$$\text{Bolt-row 2, } h_r = 734.8 \text{ mm, } F_{t,rd} = 473 \text{ kN}$$

$$\text{Bolt-row 3, } h_r = 644.8 \text{ mm, } F_{t,rd} = 473 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} = 473 + 473 + 473 = 1419 \text{ kN}$$

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

$$F_{t,wb,rd} = 213 \text{ kN}$$

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

$$F_{c,fb,rd} = 835 \text{ kN}$$

$$F_{t,rd} \leq F_{t,wb,rd} = 213 \text{ kN}, \quad F_{c,ed} = \sum F_{t,rd} \leq F_{c,fb,rd} = 835 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} \leq F_{c,wc,rd} = 2249 \text{ kN}$$

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

$$\text{Bolt-row 1, } h_r = 838.3 \text{ mm, } F_{t,rd} = 213 \text{ kN}$$

$$\text{Bolt-row 2, } h_r = 734.8 \text{ mm, } F_{t,rd} = 213 \text{ kN}$$

$$\text{Bolt-row 3, } h_r = 644.8 \text{ mm, } F_{t,rd} = 213 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} = 213 + 213 + 213 = 639 \text{ kN}$$

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

$$M_{j,rd} = [10^{-3}] \times [213 \times 838.3 + 213 \times 734.8 + 213 \times 644.8]$$

$$M_{j,rd} = 472 \text{ kNm}$$

$$M_{ed} = 335.2 \text{ kNm} < 472.4 \text{ kNm} = M_{j,rd}, \quad \text{Is verified}$$

17.14. Shear resistance (Eave connection)

(EN1993-1-8, §3.6.1 Tab.3.4)

Shear resistance of bolts

$$F_{v,rd} = \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2} = [10^{-3}] \times 0.50 \times 1000 \times 353.0 / 1.25 = 141 \text{ kN}$$

Shear plane of bolt: through the threaded portion

Bearing resistance of bolts

$$F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2}$$

End-Plate

$t = 25.0 \text{ mm}$, $d = 24 \text{ mm}$, $d_o = 26 \text{ mm}$, $e_1 = 45 \text{ mm}$, $e_2 = 45 \text{ mm}$, $p_1 = 90 \text{ mm}$, $f_{ub} = 1000 \text{ kN/mm}^2$, $f_u = 430 \text{ kN/mm}^2$,
 $\alpha_b = \min[f_{ub}/f_u, 1.0, e_1/3d_o, p_1/3d_o - 1/4] =$
 $= \min[1000/430, 1.0, 45/(3 \times 26), 90/(3 \times 26) - 0.25] = 0.58$
 $k_1 = \min[2.8e_2/d_o - 1.7, 1.4p_2/d_o - 1.7, 2.5] = \min[2.8 \times 45/26 - 1.7, 1.4 \times 90/26 - 1.7, 2.5] = 2.50$
 $F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.58 \times 430 \times 24 \times 25.0 / 1.25 = 298 \text{ kN}$

Column-Side

$t = 24.0 \text{ mm}$, $d = 24 \text{ mm}$, $d_o = 26 \text{ mm}$, $e_1 = 45 \text{ mm}$, $e_2 = 45 \text{ mm}$, $p_1 = 90 \text{ mm}$, $f_{ub} = 1000 \text{ kN/mm}^2$, $f_u = 430 \text{ kN/mm}^2$,
 $\alpha_b = \min[f_{ub}/f_u, 1.0, e_1/3d_o, p_1/3d_o - 1/4] =$
 $= \min[1000/430, 1.0, 45/(3 \times 26), 90/(3 \times 26) - 0.25] = 0.58$
 $k_1 = \min[2.8e_2/d_o - 1.7, 1.4p_2/d_o - 1.7, 2.5] = \min[2.8 \times 45/26 - 1.7, 1.4 \times 90/26 - 1.7, 2.5] = 2.50$
 $F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.58 \times 430 \times 24 \times 24.0 / 1.25 = 286 \text{ kN}$

Design resistance of one bolt in shear = $\min(141, 298, 286) = 141 \text{ kN}$

Bending moment and shear

(EN1993-1-8, §3.6.1 Tab.3.4)

Maximum tension force in bolts

$$F_{t,ed} = 213/2 = 106 \text{ kN}$$

Reduction of shear resistance due to bending

$$\rho = 1 - F_{t,ed} / 1.40 F_{t,rd} = 1 - 106 / (1.40 \times 254) = 0.70$$

Shear acting together with bending moment for all the bolts

$$V_{rd} = 8 \times 0.70 \times 141 = 790 \text{ kN}$$

$V_{ed} = 128 \text{ kN} < 790 \text{ kN} = V_{rd}$, Is verified

18. Column base Connection**18.1. Basic data (Base connection)**Design forces of connection (Base connection)

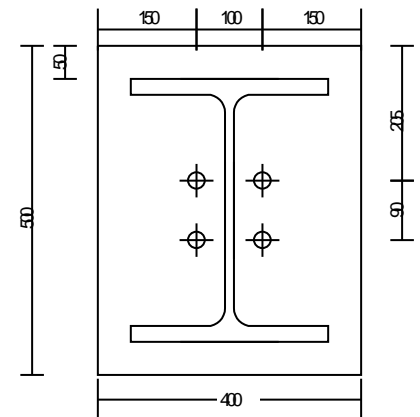
Axial force (compression) Ned=-143 kN, L.C. 202: 1.35Gk+1.50Qs1
 Axial force (tension) Ned= 0 kN,
 Shear force Ved= 52 kN, L.C. 211: 1.35Gk+1.50Qs1+0.60x1.50Qw1= 1.35xGk+1.50Qs1+0.90Qw1
 Moment Med= 0 kNm,

Seismic loading

Compression force at base Ned= -60 kN
 Tension force at base Ned= 0 kN
 Shear force at base Ved= 21 kN

Connection data (Base connection)

Base plate steel grade 500x400x30 mm, S 275
 Anchor bolts M24, Grade 5.6
 Shear plane of bolt through the threaded portion
 middle 2x2=4
 Total number of bolts =4
 Diameter of holes do = 26 mm
 Steel section for column HE 400 B, S 275
 Spacing between cross centers 100 mm
 Flange to end-plate weld 14 mm
 Web to end-plate weld 8 mm

Edge distances and spacing of bolts

Distance of plate edge to bolt line $e_1=e_2=e_x=150$ mm
 Distance of section edge to bolt line $e_c=43$ mm
 Distance of flange edge to bolt line $e_f=45$ mm
 Pitch between bolt rows $p_1=p_3=p=90$ mm
 Spacing between cross centers $p_2=g=w=100$ mm
 Flange to end-plate weld $a_{tf} \geq 0.55t_f = 0.55 \times 24.0 = 14$ mm
 Web to end-plate weld $a_{tw} \geq 0.55t_w = 0.55 \times 13.5 = 8$ mm

Concrete of foundation

Concrete-Steel class C25/30-B500C (EC2 §3.1, §3.2)
 Partial factors for materials $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 §2.4.2.4)
 Design compressive strength $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 1.00 \times 25 / 1.50 = 16.67$ N/mm² (EC2 §3.1.6)
 Design tensile strength $f_{ctd} = \alpha_{ct} \cdot f_{ctk05} / \gamma_c = 1.00 \times 2 / 1.50 = 1.20$ N/mm²
 Bearing strength $f_{jd} = \beta \cdot \sqrt{A_{c1} / A_{co}} \cdot f_{cd} = (2/3) \times 1.5 \times 16.67 = 16.67$ N/mm² (EC2 §6.7)

18.2. Design resistance of individual bolts (Base connection)

(EC3-1-8 §3.6.1, Tab.3.4)

Bolt strength grade=5.6, $f_{ub}=500$ N/mm², $A_s=353.0$ mm², $\gamma_{M2}=1.25$
 Tension resistance of bolts $F_{t,rd} = k_2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($k_2=0.90$)
 $F_{t,rd} = [10^{-3}] \times 0.90 \times 500 \times 353.0 / 1.25 = 127$ kN
 Shear resistance of bolts $F_{v,rd} = \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($\alpha_v=0.60$)
 $F_{v,rd} = [10^{-3}] \times 0.60 \times 500 \times 353.0 / 1.25 = 85$ kN

18.3. Connection geometry of end-plate (Base connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

$e=e_x=150$ mm, $e_{min}=150$ mm
 $m_{x,x} = (100 - 13.5 - 2 \times 0.8 \times 8 \times \sqrt{2}) / 2 = 34.2$ mm
 $m_{x,y} = 34.2$ mm
 $n_{x,x} = e_{min} \leq 1.25 m_{x,x} = \min(150.0, 1.25 \times 34.2) = 42.8$ mm
 $n_{x,y} = e_{min} \leq 1.25 m_{x,y} = \min(150.0, 1.25 \times 34.2) = 42.8$ mm
 $\min(m_{x,x}, m_{x,y}) = \min(34.2, 34.2) = 34.2$ mm, $\max(m_{x,x}, m_{x,y}) = \max(34.2, 34.2) = 34.2$ mm
 $\min(n_{x,x}, n_{x,y}) = \min(42.8, 42.8) = 42.8$ mm, $\max(n_{x,x}, n_{x,y}) = \max(42.8, 42.8) = 42.8$ mm

18.4. Effective lengths of end-plate (Base connection)

(EC3-1-8 §6.2.6.5 Tab.6.6)

Inner Bolt-row in a group

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 34.2 = 214.9 \text{ mm} \\
 &= 4m + 1.25e = 4 \times 34.2 + 1.25 \times 150.0 = 324.3 \text{ mm} \\
 &= 2p = 2 \times 90.0 = 180.0 \text{ mm} \\
 &= p = 90.0 \text{ mm} \\
 l_{eff,4b} &= \min(214.9, 324.3, 180.0, 90.0) = 90.0 \text{ mm} \\
 l_{eff,4b} &= 90.0 \text{ mm}
 \end{aligned}$$

18.5. End-Plate, Resistance of T-stub flange (Base connection)

(EC3-1-8 §6.2.4.1, Tab.6.2)

Inner Bolt-row in a group

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 30.0^2 \times 275 / 1.00 = 5.569 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4M_{pl,1,rd} / m = [10^3] \times 4 \times 5.569 / 34.2 = 651 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m+n) = ([10^3] \times 2 \times 5.569 + 42.8 \times 2 \times 127) / (34.2 + 42.8) = 286 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 127 = 254 \text{ kN} \\
 F_{t,rd} &= \min(651, 286, 254) = 254 \text{ kN}
 \end{aligned}$$

18.6. Column web in tension (Base connection)

(EC3-1-8 §6.2.6.8)

$$\begin{aligned}
 F_{t,wb,rd} &= b_{eff,t,wb} \cdot t_{wb} \cdot f_y / \gamma_{M0} \\
 b_{eff,t,wb} &= l_{eff} = l_{eff,4b} = 90.0 \text{ mm} \\
 F_{t,wb,rd} &= [10^{-3}] \times 90.0 \times 13.5 \times 275 / 1.00 = 334 \text{ kN}
 \end{aligned}$$

$$\min F_{t,rd} = \min(254, 334) = 254 \text{ kN}$$

18.7. Tension resistance of connection

(EN1993-1-8, §6.2.4)

$$\begin{aligned}
 \text{Uplift force of connection} \quad F_{t,ed} &= 0 \text{ kN} \\
 \text{Tension resistance of connection} \quad F_{t,rd} &= 2 \times 254 = 508 \text{ kN} \\
 N_{ed} &= 0 \text{ kN} < 508 \text{ kN} = N_{rd}, \text{ Is verified}
 \end{aligned}$$

18.8. Shear resistance (Base connection)

(EN1993-1-8, §3.6.1 Tab.3.4)

Shear resistance of bolts

$$\begin{aligned}
 F_{v,rd} &= \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2} = [10^{-3}] \times 0.60 \times 500 \times 353.0 / 1.25 = 85 \text{ kN} \\
 \text{Shear plane of bolt: through the threaded portion}
 \end{aligned}$$

Bearing resistance of bolts

$$\begin{aligned}
 F_{b,rd} &= k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} \\
 t &= 30.0 \text{ mm}, d = 24 \text{ mm}, d_o = 26 \text{ mm}, e_1 = 150 \text{ mm}, e_2 = 150 \text{ mm}, p_1 = 90 \text{ mm}, f_{ub} = 500 \text{ kN/mm}^2, f_u = 430 \text{ kN/mm}^2, \\
 \alpha_b &= \min[f_{ub}/f_u, 1.0, e_1/3d_o, p_1/3d_o - 1/4] = \\
 &= \min[500/430, 1.0, 150/(3 \times 26), 90/(3 \times 26) - 0.25] = 0.90 \\
 k_1 &= \min[2.8e_2/d_o - 1.7, 1.4p_2/d_o - 1.7, 2.5] = \min[2.8 \times 150/26 - 1.7, 1.4 \times 100/26 - 1.7, 2.5] = 2.50 \\
 F_{b,rd} &= k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.90 \times 430 \times 24 \times 30.0 / 1.25 = 560 \text{ kN}
 \end{aligned}$$

$$\text{Design resistance of one bolt in shear} = \min(85, 560) = 85 \text{ kN}$$

Tension and shear

(EN1993-1-8, §3.6.1 Tab.3.4)

$$\begin{aligned}
 \text{Maximum tension force in bolts} \quad F_{t,ed} &= 254 / 2 = 127 \text{ kN} \\
 \text{Reduction of shear resistance due to tension} \\
 \rho &= 1 - F_{t,ed} / 1.40 F_{t,rd} = 1 - 127 / (1.40 \times 127) = 0.29 \\
 \text{Shear acting together with tension for all the bolts} \\
 V_{rd} &= 4 \times 0.29 \times 85 = 99 \text{ kN}
 \end{aligned}$$

$$V_{ed} = 52 \text{ kN} < 99 \text{ kN} = V_{rd}, \text{ Is verified}$$

18.9. Bearing resistance (Base connection)

(EN1993-1-8, §6.2.5)

Compression resistance of T-stub flange $F_{c,rd} = f_{jd} \cdot b_{eff} \cdot l_{eff}$ (§6.2.5(3)Eq.6.4), §6.2.5(7)
 $f_{jd} = \beta \cdot \sqrt{(A_{c1}/A_{c0})} \cdot f_{cd} = (2/3) \cdot \sqrt{(2.25)} \cdot 16.67 = 16.67 \text{ N/mm}^2$ (EC2 EN1992-1-1:2004, §6.7,Eq.6.63)
 $h = 400.0 \text{ mm}$, $b = 300.0 \text{ mm}$, $t_f = 24.0 \text{ mm}$, $t_w = 13.5 \text{ mm}$, $t_p = 30.0 \text{ mm}$
 $c = t_p \cdot (f_y / (3f_{jd} \cdot \gamma_{M0}))^{0.5} = 30 \times (275.00 / (3 \times 16.67 \times 1.00))^{0.5} = 70.3$, < 50.0 , $c = 50.0 \text{ mm}$ (Eq.6.5)
 $2c + b_f = 2 \times 50.0 + 300 = 400.0 \text{ mm} \leq b_p = 400 \text{ mm}$, $l_{eff} = 400.0 \text{ mm}$
 $A_{c0,f} = l_{eff} \cdot (2c + t_f) = 400.0 \times (2 \times 50.0 + 24.0) = 49600 \text{ mm}^2$ (EC3-1-8, Fig.6.4)
 $A_{c0,w} = (h - 2t_f - 2c) \cdot (t_w + 2c) = (400.0 - 2 \times 24.0 - 2 \times 50.0) \times (13.5 + 2 \times 50.0) = 28602 \text{ mm}^2$
 $N_{j,rd} = [10^{-3}] \times 16.7 \times (2 \times 49600 + 28602) = [10^{-3}] \times 16.7 \times 127802 = 2134 \text{ kN}$
 $N_{j,ed} = 143 \text{ kN} < 2134 \text{ kN} = N_{j,rd}$, Is verified

Bending resistance of base plate

(EN1993-1-8, §6.2.6.10)

$M_{p,rd} = W_{el} \cdot f_y / \gamma_{M0} = [10^{-6}] (400 \times 30.0^2 / 6) \times 275 / 1.0 = 16 \text{ kNm}$ (§6.2.5)
 $M_{p,ed} = b_p \cdot q_{ed} \cdot c^2 / 2 = [10^{-6}] [400 \times 143339 / (2 \times 49600 + 28602.0)] \times 50.0^2 / 2 = 1 \text{ kNm}$
 $M_{p,ed} = 1.0 \text{ kNm} < 16.0 \text{ kNm} = M_{p,rd}$, Is verified

18.10. Anchoring resistance (Base connection)

(EN1993-1-8, §6.2.6.12)

Anchoring with washer plate

(§6.2.6.12, CEN/TS 1992-4-2)

Number of fasteners: 4, of minimum length 250mm, and minimum diameter of anchor head 60mm
 $\gamma_{Mp} = \gamma_{Msp} = \gamma_{Ms} = \gamma_{Mc} = 1.50$ (CEN/TS 1992-4-1:2009 §4.4.3.1)
Pull-out failure of fasteners (CEN/TS 1992-4-2:2009 §6.2.4)
Diameter of anchor head $d_h = 60 \text{ mm}$, $A_h = \pi / 4 (60^2 - 24^2) = 2375 \text{ mm}^2$
 $N_{rk,p} = 6 \cdot A_h \cdot f_{ck} \cdot \psi = [10^{-3}] \times 6 \times 2375 \times 25.0 \times 1.0 = 356 \text{ kN}$, ($\psi = 1.0$) (CEN/TS 1992-4-2:2009 Eq.2)
 $N_{rd,p} = N_{rk,p} / \gamma_{Mp} = 356 / 1.50 = 237 \text{ kN}$, $N_{jrd,p} = 4 \times 237 = 948 \text{ kN}$
 $N_{j,ed} = 0 \text{ kN} < 948 \text{ kN} = N_{jrd,p}$, Is verified

Concrete cone failure

(CEN/TS 1992-4-2:2009 §6.2.5)

$N_{rk,c} = N_{rk0,c} \cdot (A_{c,N} / A_{c0,N}) \cdot \psi_{sn} \cdot \psi_{ren} \cdot \psi_{ecn}$ (CEN/TS 1992-4-2:2009 Eq.4)
 $N_{rk0,c} = k_{cr} \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$ (CEN/TS 1992-4-2:2009 Eq.5)
 $k_{cr} = 8.5$, $h_{ef} = 250 \text{ mm}$, $N_{rk0,c} = [10^{-3}] \times 8.5 \times 25.0^{0.5} \times 250^{1.5} = 168 \text{ kN}$
 $A_{c,N} / A_{c0,N} = 1.0$, $\psi_{sn} = 0.70$, $\psi_{ren} = 0.5 + 250 / 200 \leq 1$, $\psi_{ren} = 1.00$, $\psi_{ecn} = 1.00$ (CEN/TS 1992-4-2Eq.8,9,10)
 $N_{rk,c} = 168 \times 1.00 \times 0.70 \times 1.00 \times 1.00 = 118 \text{ kN}$
 $N_{rd,c} = N_{rk,c} / \gamma_{Mc} = 118 / 1.50 = 79 \text{ kN}$, $N_{jrd,c} = 4 \times 79 = 316 \text{ kN}$
 $N_{j,ed} = 0 \text{ kN} < 316 \text{ kN} = N_{jrd,c}$, Is verified

Splitting failure

(CEN/TS 1992-4-2:2009 §6.2.6)

$N_{rk,sp} = N_{rk0} \cdot (A_{c,N} / A_{c0,N}) \cdot \psi_{sn} \cdot \psi_{ren} \cdot \psi_{ecn} \cdot \psi_{hsp}$ (CEN/TS 1992-4-2:2009 Eq.18)
 $N_{rk0} = \min(N_{rk,p}, N_{rk0}) = \min(356, 168) = 168 \text{ kN}$
 $A_{c,N} / A_{c0,N} = 1.0$, $\psi_{sn} = 0.70$, $\psi_{ren} = 1.00$, $\psi_{ecn} = 1.00$, $\psi_{hsp} = 1.00$
 $N_{rk,sp} = 168 \times 1.00 \times 0.70 \times 1.00 \times 1.00 \times 1.00 = 118 \text{ kN}$
 $N_{rd,sp} = N_{rk,sp} / \gamma_{Mc} = 118 / 1.50 = 79 \text{ kN}$, $N_{jrd,sp} = 4 \times 79 = 316 \text{ kN}$
 $N_{j,ed} = 0 \text{ kN} < 316 \text{ kN} = N_{jrd,sp}$, Is verified

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 - 15.5. Resistance of cross-section, at haunch-middle (Ultimate Limit State)
 - 15.6. Out-of-plane buckling, at haunch-middle (Ultimate Limit State)
- 16. Connection data
 - 16.1. Bolt connection data (eave, apex)
 - 16.2. Edge distances and spacing of bolts (eave, apex)
 - 16.3. Design resistance of individual bolts (eave, apex)
- 17. Eave connection
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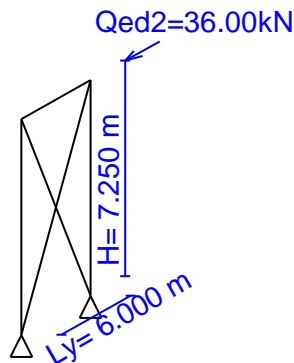
- 17.3. Connection geometry of end-plate (Eave connection)
- 17.4. Effective lengths of end-plate (Eave connection)
- 17.5. End-Plate, Resistance of T-stub flange (Eave connection)
- 17.6. Rafter flange and web in compression (Eave connection)
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STOGINĖ NR.2 (jungtys)

1. BRACE-001

Vertical bracing system

(EC3 EN1993-1-1:2005,)

Design of lateral bracing system $H=16.000\text{m}$, $L_y=7.250\text{m}$, $Q_{ed2}=4.00\text{kN}$ 1.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
 EN1991-1-1:2002, Eurocode 1-1 Actions on structures
 EN1993-1-1:2005, Eurocode 3 1-1 Design of steel structures
 EN1993-1-3:2005, Eurocode 3 1-3 Cold-formed members
 EN1993-1-5:2006, Eurocode 3 1-5 Plated structural elements
 EN1993-1-8:2005, Eurocode 3 1-8 Design of Joints

1.2. Materials

Steel: S 275 N/NL (EN1993-1-1, §3.2)

$t \leq 40 \text{ mm}$, Yield strength $f_y = 275 \text{ N/mm}^2$, Ultimate strength $f_u = 390 \text{ N/mm}^2$
 $40\text{mm} < t \leq 80 \text{ mm}$, Yield strength $f_y = 255 \text{ N/mm}^2$, Ultimate strength $f_u = 370 \text{ N/mm}^2$
 Modulus of elasticity $E = 210000 \text{ N/mm}^2$, Poisson ratio $\nu = 0.30$, Unit mass $\rho = 7850 \text{ Kg/m}^3$

Partial safety factors for actions (EN1990, Annex A1)

$\gamma_G = 1.35$, $\gamma_Q = 1.30$

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

$\gamma_{M0} = 1.00$, $\gamma_{M1} = 1.00$, $\gamma_{M2} = 1.25$

1.3. Dimensions and loads

(EN1991-1-1)

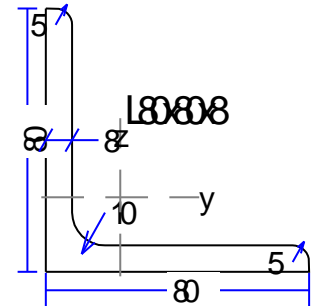
$H = 7.250 \text{ m}$

$L_y = 6.000 \text{ m}$

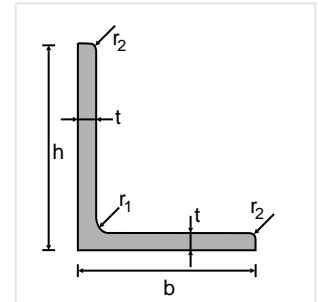
Load on bracing system, roof level $Q_{ed2} = 36.000 \text{ kN}$

1.4. Steel cross-section properties**Cross-section L80x80x8 -S 275 N/NL****Dimensions of cross section**

Depth of cross section	h=	80.00 mm
Width of cross section	b=	80.00 mm
Web depth	hw=	80.00 mm
Depth of straight portion of web	dw=	80.00 mm
Web thickness	tw=	8.00 mm
Flange thickness	tf=	8.00 mm
Radius of root fillet	r=	10.00 mm
Mass	=	9.66 Kg/m

**Properties of cross section**

Area	A=	1230 mm ²	
Second moment of area	Iy=	0.723x10 ⁶ mm ⁴	Iz= 0.723x10 ⁶ mm ⁴
Second moment of area	Iu=	1.150x10 ⁶ mm ⁴	Iv= 0.296x10 ⁶ mm ⁴
Section modulus	Wy=	12.600x10 ³ mm ³	Wz=12.600x10 ³ mm ³
Plastic section modulus	Wpy=	54.272x10 ³ mm ³	Wpz=26.624x10 ³ mm ³
Radius of gyration	iy=	24.2 mm	iz= 24.2 mm
Radius of gyration	iu=	30.6 mm	iv= 15.5 mm
Shear area	Avz=	662 mm ²	Avy= 640 mm ²
Torsional constant	It=	0.037x10 ⁶ mm ⁴	ip= 34 mm
Torsional modulus	Wt=	4.673x10 ³ mm ³	
Warping constant	Iw=	0.442x10 ⁹ mm ⁶	

**1.5. Horizontal loadings****Vertical (wall) braced girder**

The vertical brace system is loaded with point horizontal load Qed2=36.00kN

at the top of the column h= 7.250m.

Length of braced girder members 9.411 m, inclination $\varphi=50.39^\circ$, $\tan\varphi=7.250/6.000=1.208$

Forces in bracing members

Tension Nted2= 1.00x36.0/cos50.39= 56.5 kN

Compression on columns Nced2=56.5xsin50.39=43.5kN

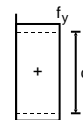
1.6. Classification of steel cross-section, Compression Nc (Bracing member)

(EN1993-1-1, §5.5)

$$h/t=80.0/8.0=10.00, (b+h)/2t=(80.0+80.0)/(2 \times 8.0)=10.00$$

$$S 275 \text{ N/NL}, t= 8.0 \leq 40 \text{ mm}, f_y=275 \text{ N/mm}^2, \varepsilon=(235/275)^{0.5}=0.92$$

$$h/t=10.00 \leq 15\varepsilon=13.80, (b+h)/2t=10.00 \leq 11.5\varepsilon=10.58$$



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

1.7. Resistance of cross-section, Bracing member

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for tension

(EN1993-1-1, §6.2.3)

Nt.ed= 56.50 kN

$$\text{Tension Resistance } N_{plrd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 1230 \times 275 / 1.00 = 338.25 \text{ kN}$$

$$N_{t,ed} = 56.50 \text{ kN} < 338.25 \text{ kN} = N_{t,rd} = N_{plrd}, \text{ Is verified}$$

$$N_{t,ed}/N_{t,rd} = 56.50/338.25 = 0.167 < 1$$

1.8. Bolts connecting braces

<u>Bolt connection data, Bracing member</u>		(EN1993-1-8)
Type of connection	End-plate connection, non-preloaded bolts	
Category of connection	Category A: Bearing type	(EC3-1-8 §3.4.1)
Connected members	Thickness t=8 mm	
Bolts	M24, Strength grade 8.8	
Bolt diameter	d = 24 mm	
Diameter of holes	do = 26 mm	
Nominal area	$\pi d^2/4 = \pi \times 24^2/4 = 452.4 \text{ mm}^2$	
Tensile stress area	As = 452.4 mm ²	
Bolt strength grade	8.8, fyb=640N/mm ² , fub=800N/mm ²	(EC3-1-8 §3.1.1)

Shear resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)

$$F_{v,rd} = \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2} = [10^{-3}] \times 0.60 \times 800 \times 452.4 / 1.25 = 173.7 \text{ kN}$$

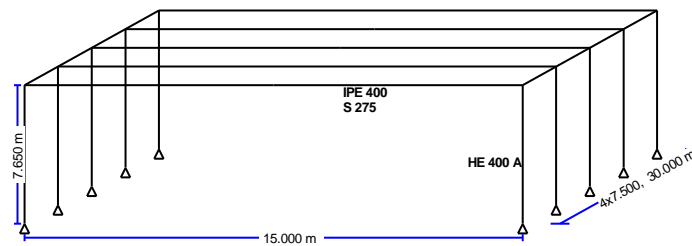
Bearing resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)

$$F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2}$$

t=8.0mm, d=24mm, do=26mm, e1=50mm, e2=50mm, p1=100mm, fub=800kN/mm², fu=430kN/mm²,
 $\alpha_b = \min[f_{ub}/f_u, 1, e_1/3d_o, p_1/3d_o - 1/4] = \min[800/430, 1, 50/(3 \times 26), 100/(3 \times 26) - 0.25] = 0.64$
 $k_1 = \min[2.8e_2/d_o - 1.7, 2.5] = \min[2.8 \times 50/26 - 1.7, 2.5] = 2.50$
 $F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.64 \times 430 \times 24 \times 8.0 / 1.25 = 105.8 \text{ kN}$

Necessary bolts per brace 56.5/105.8= 1 M24, Grade 8.8

PFR 01/09/2022

STOGINĖ NR. 3 (karkasas)**1. Design codes**

EN1990:2002, Eurocode 0 Basis of Structural Design
 EN1991-1-1:2002, Eurocode 1-1 Actions on structures
 EN1991-1-3:2003, Eurocode 1-3 Snow loads
 EN1991-1-4:2005, Eurocode 1-4 Wind actions
 EN1993-1-1:2005, Eurocode 3 1-1 Design of steel structures
 EN1993-1-3:2005, Eurocode 3 1-3 Cold-formed members
 EN1993-1-5:2006, Eurocode 3 1-5 Plated structural elements
 EN1993-1-8:2005, Eurocode 3 1-8 Design of Joints
 CEN/TS 1992-4-1:2009, Design of fastenings in concrete, General
 CEN/TS 1992-4-2:2009, Design of fastenings, Headed Fasteners
 EN1998-1-1:2004, Eurocode 8 Design in earthquake environment

2. Basic data**2.1. Geometry of frame structure**

Bay width	$L = 15.000 \text{ m}$
Total height(max)	$H = 7.650 \text{ m}$
Column height	$H1 = 7.650 \text{ m}$
Total length	$B = 30.000 \text{ m (4x7.500m)}$
Spacing of frames	$s = 7.500 \text{ m}$
Roof slope	$\alpha = 0.00^\circ$
Haunch size	$L1 = L/10.0 = 1.500 \text{ m}$
Cladding	Sheeting thickness $t_w = 0.100 \text{ mm}$, Profile depth $h_w = 5.0 \text{ mm}$
Purlin spacing	$= 1.400 \text{ m}$
	Purlin laterally restrained, Simply supported purlin

2.2. Steel sections

Column section	HE 400 A - S 275
Rafter section	IPE 400 - S 275
Purlin section	Z30030 - S 275
Transverse restraint system	L90x90x8 - S 275
Lateral bracing of columns	$L_{m1} = 7.050 \text{ m}$
Torsional restrains of rafters	$L_{m2} = 4.000 \text{ m}$
Compression stiffener at the bottom of haunch	

2.3. Steel joints

Type of connection	End-plate connection, non-preloaded bolts
Category of connection	Category A: Bearing type Category D: Non-preloaded
End Plate	Thickness $t_p=20$ mm, S 275
Bolts	M24, Grade 10.9

3. Materials and Code parameters

3.1. Materials

Steel: S 275 (EN1993-1-1, §3.2)

$t \leq 40$ mm, Yield strength $f_y = 275$ N/mm², Ultimate strength $f_u = 430$ N/mm²

$40\text{mm} < t \leq 80$ mm, Yield strength $f_y = 255$ N/mm², Ultimate strength $f_u = 410$ N/mm²

Modulus of elasticity $E = 210000$ N/mm², Poisson ratio $\nu = 0.30$, Unit mass $\rho = 7850$ Kg/m³

Partial factors for materials

(EN1993-1-1, §6.1)

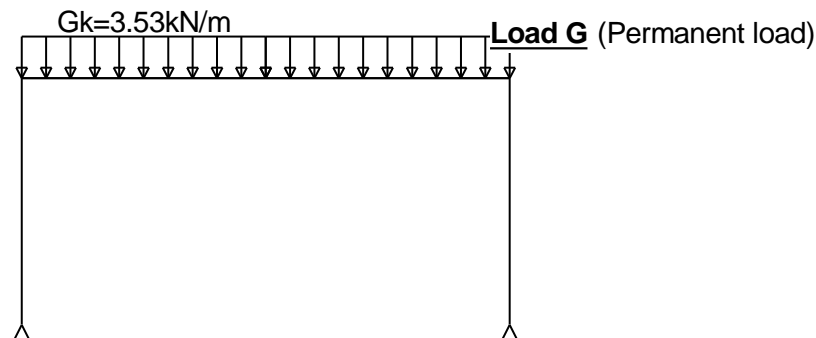
$\gamma_{M0} = 1.00$, $\gamma_{M1} = 1.00$, $\gamma_{M2} = 1.25$

4. Loads

4.1. Permanent loads

(EN1991-1-1)

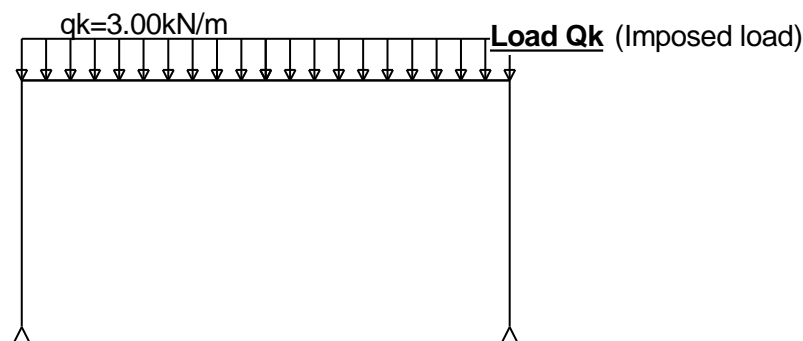
Self weight of purlins and finishing	$g_{k1} = 0.300 + 0.117/1.400 = 0.384$ kN/m ²
Self weight of ceiling under the roof	$g_{k2} = 0.000$ kN/m ² $g_k = g_{k1} + g_{k2} = 0.384$ kN/m ²
Spacing of frames	$s = 7.500$ m
Roof load on frame	$(g_{k1} + g_{k2}) \cdot s = 0.384 \times 7.500 = 2.88$ kN/m
Self weight of rafters	$G(\text{IPE } 400) = 0.65$ kN/m
Permanent load on frame	$G_k = 2.88 + 0.65 = 3.53$ kN/m
Self weight of columns	$G(\text{HE } 400 \text{ A}) = 1.23$ kN/m



4.2. Imposed loads

(EC1 EN1991-1-1:2002 Tab.6.10)

Roof slope	$\alpha = 0.00^\circ$
Imposed load (category H)	$q_k = 0.40$ kN/m ²
Roof load on frame	$q_k \cdot s = 0.40 \times 7.500 = 3.00$ kN/m



4.3. Snow load

(EC1 EN1991-1-3:2003)

Snow load on the ground

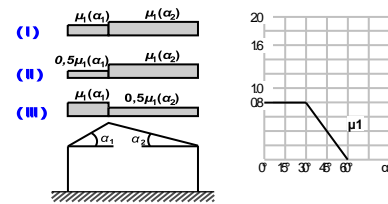
(EN1991-1-3 §4, Annex C)

Characteristic value of snow load on the ground: $s_k = 1.200$ kN/m²

Snow load on the roof

(EC1 EN1991-1-3:2003 §5)

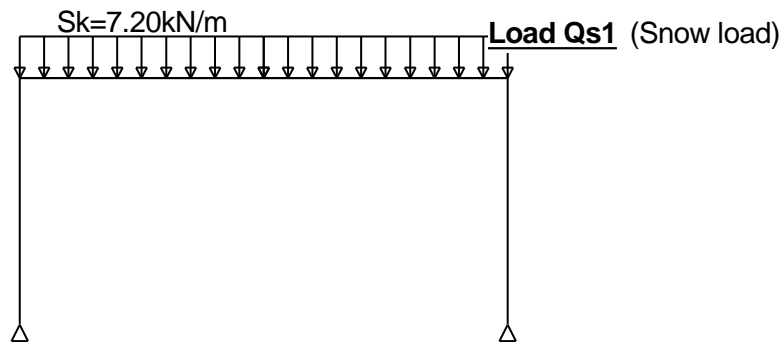
pitched roof (EC1-1-3 §5.3.3))

Angle of pitch of roof : $\alpha_1=0.000^\circ$ Angle of pitch of roof : $\alpha_2=0.000^\circ$ Exposure coefficient : $C_e=1.000$ (EC1-1-3 §5.2(7))Thermal coefficient : $C_t=1.000$ (EC1-1-3 §5.2(8))Shape coefficients $\mu_1(\alpha_1)=\mu_1(\alpha_2)=0.800$ (EC1-1-3 T.5.2)) $S(\alpha_1)=\mu_1(\alpha_1) \cdot C_e \cdot C_t \cdot S_k = 0.800 \times 1.000 \times 1.000 \times 1.200 = 0.960 \text{ kN/m}^2$ $S(\alpha_2)=\mu_1(\alpha_2) \cdot C_e \cdot C_t \cdot S_k = 0.800 \times 1.000 \times 1.000 \times 1.200 = 0.960 \text{ kN/m}^2$ **Snow load**

(EC1 EN1991-1-3:2003, §5.2, §5.3.3)

Load case (I) , $S(\text{Left})=S(\alpha_1) = 0.960 \text{ kN/m}^2$, $S(\text{Right})=S(\alpha_2) = 0.960 \text{ kN/m}^2$ **4.4. Snow load on frame**

(EC1 EN1991-1-3:2003)

Snow load on the ground $s_k = 1.200 \text{ kN/m}^2$ Snow load on the roof $S_k = 0.8 \times 1.200 \times 1.00 \times 1.00 = 0.960 \text{ kN/m}^2$ Spacing of frames $s = 7.500 \text{ m}$ Snow load on frame $S_{k1} = 0.960 \times 7.500 / \cos 0.00^\circ = 7.20 \text{ kN/m}$ $S_{k2} = 0.5 \times 0.960 \times 7.500 / \cos 0.00^\circ = 3.60 \text{ kN/m}$ Load case(I) $S_{k1} = 7.20 \text{ kN/m}$, $S_{k2} = 7.20 \text{ kN/m}$ **4.5. Wind load**

(EC1 EN1991-1-4:2005)

Reference velocity

(EN1991-1-4, §4.2)

 $v_{bo} = 0.00 \text{ m/s}$, Zone: 2 $v_b = C_{dir} \cdot C_{season} \cdot V_{bo} = 24.00 \text{ m/s}$ **Terrain effects**

(EN1991-1-4, §4.3.2, Annex A)

Terrain category : III

(EN1991-1-4, Tab.4.1)

Area with regular cover of vegetation or buildings (villages, suburban terrain, forest)

Roughness factor $C_r(z)$

(EN1991-1-4, §4.3.2)

Terrain category:III, $z=7.650 \text{ m}$, $z_o=0.300 \text{ m}$, $z_{min}=5 \text{ m}$, $z_{max}=200 \text{ m}$, $z_{oII}=0.050 \text{ m}$ $kr = 0.19 \cdot (0.300/0.05)^{0.07} = 0.215$ $C_r(z) = kr \cdot \ln(z/z_o) = 0.215 \times \ln(7.650/0.300) = 0.698$ Orography factor $C_o(z)$

(EN1991-1-4, §4.3.3)

 $C_o(z) = 1.000$

(EN1991-1-4, §4.3.3)

Turbulence factor K_t

(EN1991-1-4, §4.4)

 $K_t = 1.000$ Exposure factor $C_e(z)$

(EN1991-1-4, §4.5)

Terrain category: III

(EN1991-1-4, Tab.4.1)

 $z = 7.65 \text{ m}$, $kr = 0.215$, $lv(z) = 0.309$, $C_e(z) = 1.538$ (EC1 EN1991-1-4:2005, eq.A. 4.8,4.7,4.4,4.3) $q(z) = C_e(z) \cdot (\frac{1}{2} \rho) \cdot V_b^2 = [0.001] \times 1.538 \times 0.625 \times 24.00^2 = 0.554 \text{ kN/m}^2$

Wind peak velocity pressure $q(z)=C_e(z) \cdot q_b = C_e(z) \cdot (0.625) \cdot V_b^2$

(EN1991-1-4, §4.5)

$V_b=24.00\text{m/sec}$

$z=7.650\text{m}$

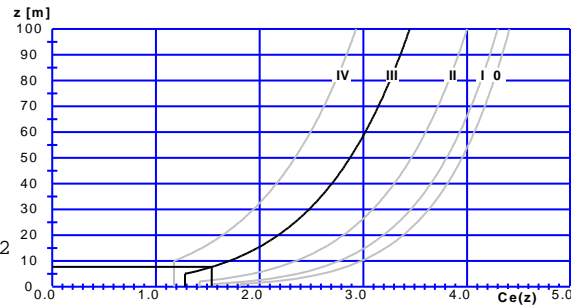
$C_r(z)=0.698$

$C_o(z)=1.000$

$K_t=1.000$

$C_e(z)=1.538$

$$\begin{aligned} q(z) &= C_e(z) \cdot (\frac{1}{2}\rho) \cdot V_b^2 \\ &= [0.001] \times 1.538 \times 0.625 \times 2 \\ &= 0.554 \text{ kN/m}^2 \end{aligned}$$



Wind forces on flat roof, wind direction: 0.00

(EN1991-1-4, §7.2.3)

Wind pressure coefficients C_{pe}

(EN1991-1-4, Tab. 7.2)

wind direction: $\theta=0.00$

$b=30.00\text{m}$, $d=15.00\text{m}$, $h=7.65\text{m}$, $e=\min(b, 2h)=15.30\text{m}$

$e/4=3.83\text{m}$, $e/10=1.53\text{m}$, $e/2=7.65\text{m}$

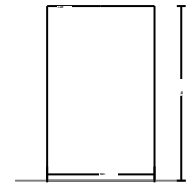
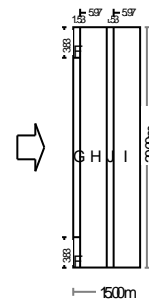
Roof type: Sharp eaves

Zone : F, $A= 5.85\text{m}^2$, $C_{pe,10}=-1.80$, $C_{pe,1}=-2.50$

Zone : G, $A= 34.20\text{m}^2$, $C_{pe,10}=-1.20$, $C_{pe,1}=-2.00$

Zone : H, $A= 183.60\text{m}^2$, $C_{pe,10}=-0.70$, $C_{pe,1}=-1.20$

Zone : I, $A= 220.50\text{m}^2$, $C_{pe,10}=\pm 0.20$, $C_{pe,1}=\pm 0.20$



Wind pressure on roof surfaces $w_e=q(z) \cdot C_{pe}=0.554 \times C_{pe}$ [kN/m²]

(EN1991-1-4, 5.1)

F		G		H		I	
$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$
-0.997	-1.385	-0.665	-1.108	-0.388	-0.665	+0.111	-0.111

Wind forces on vertical walls

(EN1991-1-4, §7.2.2)

Wind pressure coefficients C_{pe}

(EN1991-1-4, Tab.7.1)

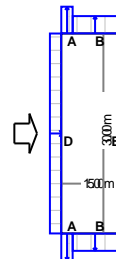
$h/d=7.65/15.00=0.510$, $e=15.30\text{m}$

Zone : A, (3.06xh), $C_{pe,10}=-1.20$, $C_{pe,1}=-1.40$

Zone : B, (11.94xh), $C_{pe,10}=-0.80$, $C_{pe,1}=-1.10$

Zone : D, (30.00xh), $C_{pe,10}= 0.80$, $C_{pe,1}= 1.00$

Zone : E, (30.00xh), $C_{pe,10}=-0.37$, $C_{pe,1}=-0.37$



Wind pressure on wall surfaces $w_e=q(z) \cdot C_{pe}$ [kN/m²]

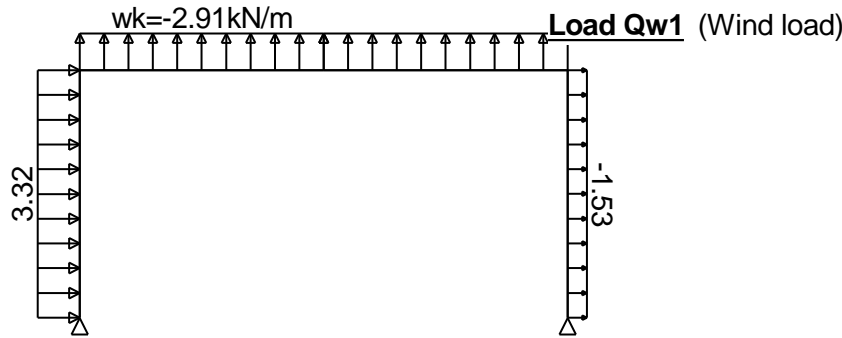
(EN1991-1-4, 5.1)

A		B		D		E		
$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	$w_{e,10}$	$w_{e,1}$	
$z= 7.65 \sim 0.00\text{m}$,	-0.665	-0.776	-0.443	-0.609	0.443	0.554	-0.205	-0.205

4.6. Wind load on frame

(EC1 EN1991-1-4:2005)

Wind pressure on vertical surface	$w_k = 0.554 \text{ kN/m}^2$
Wind internal pressure	$w_i = 0.000 \text{ kN/m}^2$
Spacing of frames	$s = 7.500 \text{ m}$
Left column	$W_{k1} = 0.443 \times 7.500 = 3.32 \text{ kN/m}$
Left rafter	$W_{k2} = -0.388 \times 7.500 = -2.91 \text{ kN/m}$
Right rafter	$W_{k3} = -0.388 \times 7.500 = -2.91 \text{ kN/m}$
Right column	$W_{k4} = -0.205 \times 7.500 = -1.53 \text{ kN/m}$

**4.7. Seismic loading**

(EC8 EN1998-1-1:2004, §3)

Horizontal acceleration ratio (§3.2.2.2)	$a_{gr}/g = 0.040$	
Verti./horiz. acceleration (§3.2.2.3)	$avg/a_{gr} = 0.90$	
Importance factor (§3.2.1, T.4.3)	$\gamma_i = 1.00$	
Soil factor [horizontal] (§3.2.2.2)	$S = 1.00$	
Behavior factor [horizontal] (§3.2.2.5)	$q = 1.50$	
Behavior factor [vertical]	$q_v = 1.50$	
Spectral shape factor [horizontal] (§3.2.2.5)	$\beta_h(T) = 2.50$	
Spectral shape factor [vertical] (§3.2.2.3)	$\beta_v(T) = 3.00$	
Correction factor (§4.3.3.2.2.1)	$\lambda = 1.00$	
Force distribution $\zeta = z_i w_i / \sum z_j w_j$ (§4.3.3.2.3)	$\zeta = 1.50$	
Fundamental vibration period (§4.3.3.2.2.3)	$(\text{sec}) = 0.77$	
Live load combination factor (EC0 T.A1.1)	$\psi_2 = 0.30$	
Snow load combination factor (EC0 T.A1.1)	$\psi_2 = 0.20$	
Characteristic spectral periods [horizontal]	$T_b = 0.15 \text{ sec}, T_c = 0.50 \text{ sec}, T_d = 2.00 \text{ sec}$	
Characteristic spectral periods [vertical]	$T_b = 0.05 \text{ sec}, T_c = 0.15 \text{ sec}, T_d = 1.00 \text{ sec}$	
$S_d(T_1 = 0.77 \text{ s}) = 0.040 \times 1.00 \times 1.00 \times [(2.50/1.50) \times (0.50/0.772)] = 0.424 \text{ m/s}^2$	(EC8 §3.2.2.5(4), Eq.3.13)	
$S_v(T_1 = 0.77 \text{ s}) = 0.90 \times 0.040 \times 1.00 \times [(3.00/1.50) \times (0.15/0.772)] = 0.137 \text{ m/s}^2$	(EC8 §3.2.2.5(5))	

5. Design values of Actions

(EN1990 NA Latvia LVS, §6.4, §6.5)

5.1. Load combination factors

(EN1990 Tab.A1.1)

Category H (roofs)

 $Q_k \psi_0=0.00, \psi_1=0.00, \psi_2=0.00$

Snow loads on buildings

 $Q_s \psi_0=0.50, \psi_1=0.20, \psi_2=0.00$

Wind loads on buildings

 $Q_w \psi_0=0.60, \psi_1=0.20, \psi_2=0.00$ **5.2. Ultimate Limit State (ULS) (EQU)**

(EN1990 §6.4.3.2, T.A1.2A)

$$E_d = \gamma_G \cdot G_k + \gamma_Q \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10})$$

$$\gamma_{G,\text{sup}}=1.35 \text{ (Unfavorable)}$$

$$\gamma_{G,\text{inf}}=0.90 \text{ (Favorable)}$$

$$\gamma_Q = 1.30 \text{ (Unfavorable)}$$

$$\gamma_Q = 0.00 \text{ (Favorable)}$$

Load combinations (ULS)(EQU),**Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

$$L.C. 101: 1.35G_k + 1.30Q_k \quad (\text{Eq.6.10})$$

$$L.C. 102: 1.35G_k + 1.30Q_{s1} \quad (\text{Eq.6.10})$$

$$L.C. 103: 1.35G_k + 1.30Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 111: 0.90G_k + 1.30Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 121: 1.35G_k + 1.30Q_{s1} + 0.60 \times 1.30Q_{w1} = 1.35xG_k + 1.30Q_{s1} + 0.78Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 122: 1.35G_k + 1.30Q_{w1} + 0.50 \times 1.30Q_{s1} = 1.35xG_k + 1.30Q_{w1} + 0.65Q_{s1} \quad (\text{Eq.6.10})$$

5.3. Ultimate Limit State (ULS) (STR)

(EN1990 §6.4.3.2, T.A1.2B)

$$E_d = \gamma_G \cdot G_k + \gamma_Q \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10})$$

$$E_d = \gamma_G \cdot G_k + \gamma_Q \cdot \psi_0 \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10a})$$

$$E_d = \xi \cdot \gamma_G \cdot G_k + \gamma_Q \cdot Q_{k1} + \gamma_Q \cdot \psi_0 \cdot Q_{k2} \quad (\text{Eq.6.10b})$$

$$\gamma_{G,\text{sup}}=1.35 \text{ (Unfavorable)}$$

$$\gamma_{G,\text{inf}}=1.00 \text{ (Favorable)}$$

$$\gamma_Q = 1.50 \text{ (Unfavorable)}$$

$$\gamma_Q = 0.00 \text{ (Favorable)}$$

$$\xi=0.850, \xi \cdot \gamma_G=0.850 \times 1.35=1.15$$

Load combinations (ULS)(STR),**Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

$$L.C. 201: 1.35G_k + 1.50Q_k \quad (\text{Eq.6.10})$$

$$L.C. 202: 1.35G_k + 1.50Q_{s1} \quad (\text{Eq.6.10})$$

$$L.C. 203: 1.35G_k + 1.50Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 210: 1.00G_k + 1.50Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 211: 1.35G_k + 1.50Q_{s1} + 0.60 \times 1.50Q_{w1} = 1.35xG_k + 1.50Q_{s1} + 0.90Q_{w1} \quad (\text{Eq.6.10})$$

$$L.C. 212: 1.35G_k + 1.50Q_{w1} + 0.50 \times 1.50Q_{s1} = 1.35xG_k + 1.50Q_{w1} + 0.75Q_{s1} \quad (\text{Eq.6.10})$$

$$L.C. 231: 1.35G_k + 1.50 \times 0.50Q_{s1} + 1.50 \times 0.60Q_{w1} = 1.35xG_k + 0.75Q_{s1} + 0.90Q_{w1} \quad (\text{Eq.6.10a})$$

$$L.C. 251: 0.850 \times 1.35G_k + 1.50Q_{s1} + 1.50 \times 0.60Q_{w1} = 1.15xG_k + 1.50Q_{s1} + 0.90Q_{w1} \quad (\text{Eq.6.10b})$$

$$L.C. 252: 0.850 \times 1.35G_k + 1.50Q_{w1} + 1.50 \times 0.50Q_{s1} = 1.15xG_k + 1.50Q_{w1} + 0.75Q_{s1} \quad (\text{Eq.6.10b})$$

5.4. Serviceability Limit State (SLS)

(EN1990 §6.5.3, T.A1.4)

$$E_d = G_k + Q_{k1} + \psi_0 \cdot Q_{k2} + \psi_0 \cdot Q_{k3} \text{ (Characteristic combination)} \quad (\text{Eq.6.14})$$

$$E_d = G_k + \psi_1 \cdot Q_{k1} + \psi_2 \cdot Q_{k2} + \psi_2 \cdot Q_{k3} \text{ (Frequent combination)} \quad (\text{Eq.6.15})$$

$$E_d = G_k + \psi_2 \cdot Q_{k1} + \psi_2 \cdot Q_{k2} + \psi_2 \cdot Q_{k3} \text{ (Quasi-permanent combination)} \quad (\text{Eq.6.16})$$

Load combinations (SLS)**Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

L.C. 301: $G_k + Q_k$	(Eq.6.14a)
L.C. 302: $G_k + Q_{s1}$	(Eq.6.14a)
L.C. 303: $G_k + Q_{w1}$	(Eq.6.14a)
L.C. 311: $G_k + Q_{s1} + 0.60Q_{w1}$	(Eq.6.14a)
L.C. 312: $G_k + Q_{w1} + 0.50Q_{s1}$	(Eq.6.14a)
L.C. 331: $G_k + 0.20Q_{s1} + 0.00Q_{w1}$	(Eq.6.15a)
L.C. 332: $G_k + 0.20Q_{w1} + 0.00Q_{s1}$	(Eq.6.15a)
L.C. 351: $G_k + 0.00Q_{s1} + 0.00Q_{w1}$	(Eq.6.16a)

5.5. Ultimate Limit State (ULS) Seismic situation

$$E_d = G_k + A_{ed} + \psi_2 \cdot Q_{k1} + \psi_2 \cdot Q_{k2} + \psi_2 \cdot Q_{k3} \quad (\text{Eq.6.12b})$$

Snow load Q_s , Wind load Q_w , Seismic load A_{ed}

L.C. 601: $G_k + 0.20Q_{s1} + A_{ed}$	(Eq.6.14a)
---------------------------------------	------------

5.6. Summary of load combination**Permanent load G_k , Imposed load Q_k , Snow load Q_{s1} , Wind load Q_{w1}**

1 L.C. 101 (ULS) (EQU)	$1.35G_k + 1.30Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
2 L.C. 102 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 1.30Q_{s1} + 0.00Q_{w1}$
3 L.C. 103 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 0.00Q_{s1} + 1.30Q_{w1}$
4 L.C. 111 (ULS) (EQU)	$0.90G_k + 0.00Q_k + 0.00Q_{s1} + 1.30Q_{w1}$
5 L.C. 121 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 1.30Q_{s1} + 0.78Q_{w1}$
6 L.C. 122 (ULS) (EQU)	$1.35G_k + 0.00Q_k + 0.65Q_{s1} + 1.30Q_{w1}$
7 L.C. 201 (ULS) (STR)	$1.35G_k + 1.50Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
8 L.C. 202 (ULS) (STR)	$1.35G_k + 0.00Q_k + 1.50Q_{s1} + 0.00Q_{w1}$
9 L.C. 203 (ULS) (STR)	$1.35G_k + 0.00Q_k + 0.00Q_{s1} + 1.50Q_{w1}$
10 L.C. 210 (ULS) (STR)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 1.50Q_{w1}$
11 L.C. 211 (ULS) (STR)	$1.35G_k + 0.00Q_k + 1.50Q_{s1} + 0.90Q_{w1}$
12 L.C. 212 (ULS) (STR)	$1.35G_k + 0.00Q_k + 0.75Q_{s1} + 1.50Q_{w1}$
13 L.C. 231 (ULS) (STR)	$1.35G_k + 0.00Q_k + 0.75Q_{s1} + 0.90Q_{w1}$
14 L.C. 251 (ULS) (STR)	$1.15G_k + 0.00Q_k + 1.50Q_{s1} + 0.90Q_{w1}$
15 L.C. 252 (ULS) (STR)	$1.15G_k + 0.00Q_k + 0.75Q_{s1} + 1.50Q_{w1}$
16 L.C. 301 (SLS)	$1.00G_k + 1.00Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
17 L.C. 302 (SLS)	$1.00G_k + 0.00Q_k + 1.00Q_{s1} + 0.00Q_{w1}$
18 L.C. 303 (SLS)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 1.00Q_{w1}$
19 L.C. 311 (SLS)	$1.00G_k + 0.00Q_k + 1.00Q_{s1} + 0.60Q_{w1}$
20 L.C. 312 (SLS)	$1.00G_k + 0.00Q_k + 0.50Q_{s1} + 1.00Q_{w1}$
21 L.C. 331 (SLS)	$1.00G_k + 0.00Q_k + 0.20Q_{s1} + 0.00Q_{w1}$
22 L.C. 332 (SLS)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 0.20Q_{w1}$
23 L.C. 351 (SLS)	$1.00G_k + 0.00Q_k + 0.00Q_{s1} + 0.00Q_{w1}$
24 L.C. 601 (SEISM)	$1.00G_k + 0.00Q_k + 0.20Q_{s1} + 0.00Q_{w1} + A_{ed}$

6. Steel sections

6.1. Column section

Steel cross-section properties

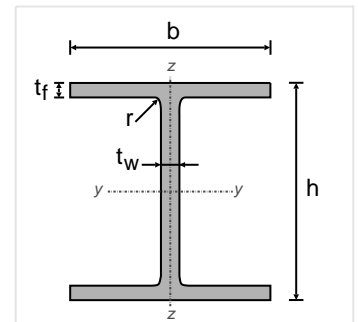
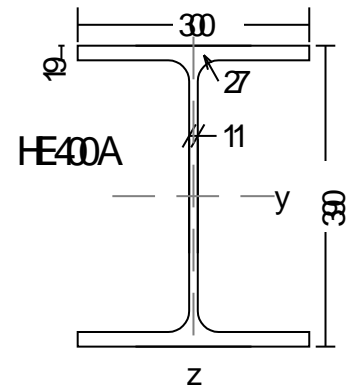
Cross-section HE 400 A-S 275

Dimensions of cross section

Depth of cross section	h=	390.00	mm
Width of cross section	b=	300.00	mm
Web depth	hw=	371.00	mm
Depth of straight portion of web	dw=	298.00	mm
Web thickness	tw=	11.00	mm
Flange thickness	tf=	19.00	mm
Radius of root fillet	r=	27.00	mm
Mass	=	125.00	Kg/m

Properties of cross section

Area	A=	15900	mm ²		
Second moment of area	Iy=	450.70x10 ⁶	mm ⁴	Iz=	85.640x10 ⁶ mm ⁴
Section modulus	Wy=	2311.0x10 ³	mm ³	Wz=	571.00x10 ³ mm ³
Plastic section modulus	Wpy=	2562.0x10 ³	mm ³	Wpz=	872.90x10 ³ mm ³
Radius of gyration	iy=	168.4	mm	iz=	73.4 mm
Shear area	Avz=	5735	mm ²	Avy=	11400 mm ²
Torsional constant	It=	1.890x10 ⁶	mm ⁴	ip=	184 mm
Torsional modulus	Wt=	99.494x10 ³	mm ³		
Warping constant	Iw=	2942.1x10 ⁹	mm ⁶		



6.2. Rafter section

Steel cross-section properties

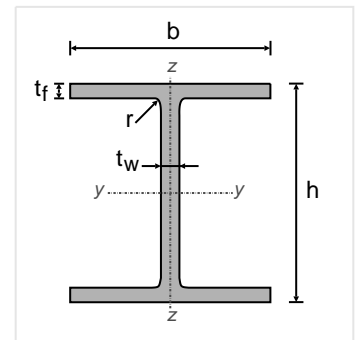
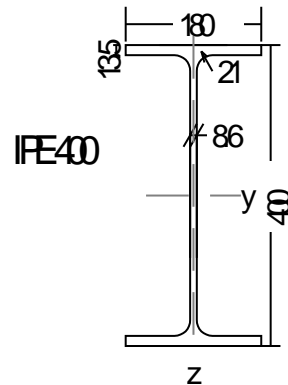
Cross-section IPE 400-S 275

Dimensions of cross section

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

Properties of cross section

Area	A=	8446	mm ²		
Second moment of area	Iy=	231.30x10 ⁶	mm ⁴	Iz=	13.180x10 ⁶ mm ⁴
Section modulus	Wy=	1156.0x10 ³	mm ³	Wz=	146.40x10 ³ mm ³
Plastic section modulus	Wpy=	1307.0x10 ³	mm ³	Wpz=	229.00x10 ³ mm ³
Radius of gyration	iy=	165.5	mm	iz=	39.5 mm
Shear area	Avz=	4269	mm ²	Avy=	4860 mm ²
Torsional constant	It=	0.511x10 ⁶	mm ⁴	ip=	170 mm
Torsional modulus	Wt=	37.834x10 ³	mm ³		
Warping constant	Iw=	490.05x10 ⁹	mm ⁶		



6.3. Haunch section at haunch end

Steel cross-section properties

Welded section

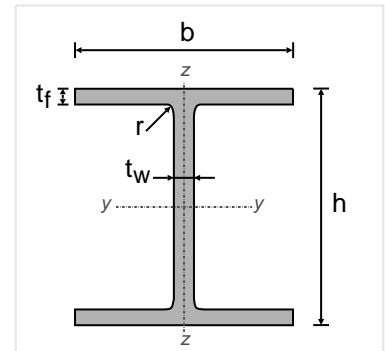
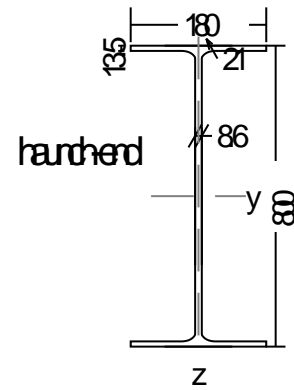
Cross-section haunch-end-S 275

Dimensions of cross section

Depth of cross section	h=	800.00	mm
Width of cross section	b=	180.00	mm
Web depth	hw=	786.50	mm
Depth of straight portion of web	dw=	713.60	mm
Web thickness	tw=	8.60	mm
Flange thickness	tf=	13.50	mm
Radius of root fillet	r=	21.00	mm
Mass	=	90.39	Kg/m

Properties of cross section

Area	A=	11508	mm ²		
Second moment of area	Iy=	1082.7x10 ⁶	mm ⁴	Iz=	13.163x10 ⁶ mm ⁴
Section modulus	Wy=	2706.7x10 ³	mm ³	Wz=	146.26x10 ³ mm ³
Plastic section modulus	Wpy=	3195.9x10 ³	mm ³	Wpz=	232.99x10 ³ mm ³
Radius of gyration	iy=	306.7	mm	iz=	33.8 mm
Shear area	Avz=	6764	mm ²	Avy=	4860 mm ²
Torsional constant	It=	0.454x10 ⁶	mm ⁴	ip=	309 mm
Torsional modulus	Wt=	33.646x10 ³	mm ³		
Warping constant	Iw=	2029.3x10 ⁹	mm ⁶		
Weld	a=	21.0x10 ⁹	mm		



6.4. Haunch section at haunch-middle

Steel cross-section properties

Welded section

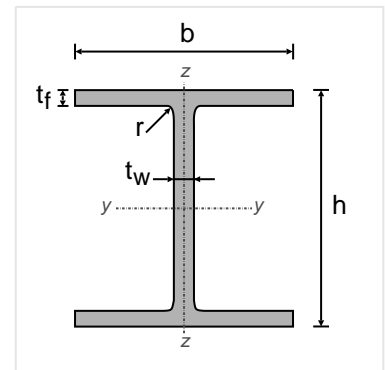
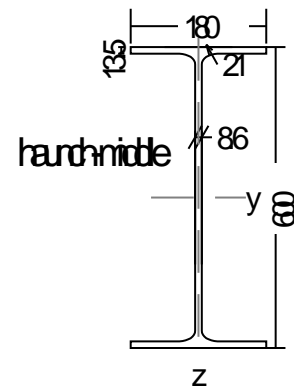
Cross-section haunch-middle-S 275

Dimensions of cross section

Depth of cross section	h=	600.00	mm
Width of cross section	b=	180.00	mm
Web depth	hw=	586.50	mm
Depth of straight portion of web	dw=	513.60	mm
Web thickness	tw=	8.60	mm
Flange thickness	tf=	13.50	mm
Radius of root fillet	r=	21.00	mm
Mass	=	76.88	Kg/m

Properties of cross section

Area	A=	9788	mm ²		
Second moment of area	Iy=	552.84x10 ⁶	mm ⁴	Iz=	13.152x10 ⁶ mm ⁴
Section modulus	Wy=	1842.8x10 ³	mm ³	Wz=	146.14x10 ³ mm ³
Plastic section modulus	Wpy=	2131.1x10 ³	mm ³	Wpz=	229.29x10 ³ mm ³
Radius of gyration	iy=	237.7	mm	iz=	36.7 mm
Shear area	Avz=	5044	mm ²	Avy=	4860 mm ²
Torsional constant	It=	0.412x10 ⁶	mm ⁴	ip=	240 mm
Torsional modulus	Wt=	30.505x10 ³	mm ³		
Warping constant	Iw=	1128.4x10 ⁹	mm ⁶		
Weld	a=	21.0x10 ⁹	mm		



7. Finite Element Analysis

(EN1993-1-1, §5.1)

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

Linear-elastic analysis is used for the design of static loads.
 The seismic design is based on lateral force method and on dynamic analysis by modal superposition spectrum analysis.

7.1. Data used for elastic analysis**Nodal points**

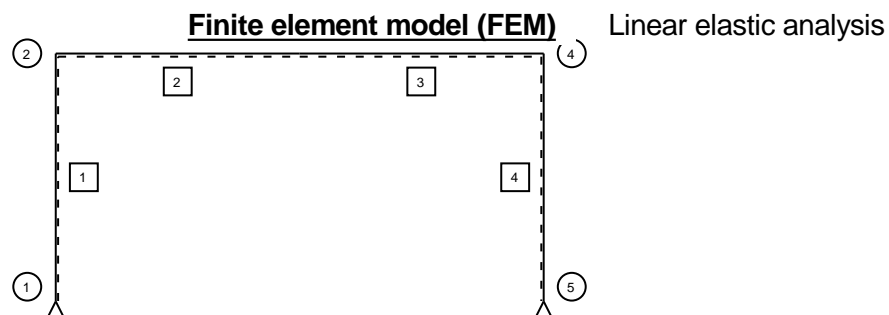
Node	x [mm]	y[mm]
1	0	0
2	0	7650
3	7500	7650
4	15000	7650
5	15000	0

Supports

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

Elements

Element	node 1	node 2	length(mm)	angle(°)	E(GPa)	A(mm ²)	I(mm ⁴)
1	1	2	7650	90.00	210	15900	450700x10 ³
2	2	3	7500	0.00	210	8446	231300x10 ³
3	3	4	7500	0.00	210	8446	231300x10 ³
4	4	5	7650	270.00	210	15900	450700x10 ³



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

L.C.		Left column 1			Left rafter 2			Right rafter 3			Right column 4		
		q	qy	qx	q	qy	qx	q	qy	qx	q	qy	qx
101	ULS-EQU	0.00	1.66	0	0.00	8.67	0	0.00	8.67	0	0.00	1.66	0
102	ULS-EQU	0.00	1.66	0	0.00	14.13	0	0.00	14.13	0	0.00	1.66	0
103	ULS-EQU	4.32	1.66	0	-3.78	4.77	0	-3.78	4.77	0	-1.99	1.66	0
111	ULS-EQU	4.32	1.11	0	-3.78	3.18	0	-3.78	3.18	0	-1.99	1.11	0
121	ULS-EQU	2.59	1.66	0	-2.27	14.13	0	-2.27	14.13	0	-1.20	1.66	0
122	ULS-EQU	4.32	1.66	0	-3.78	9.45	0	-3.78	9.45	0	-1.99	1.66	0
201	ULS-STR	0.00	1.66	0	0.00	9.27	0	0.00	9.27	0	0.00	1.66	0
202	ULS-STR	0.00	1.66	0	0.00	15.57	0	0.00	15.57	0	0.00	1.66	0
203	ULS-STR	4.99	1.66	0	-4.36	4.77	0	-4.36	4.77	0	-2.30	1.66	0
210	ULS-STR	4.99	1.23	0	-4.36	3.53	0	-4.36	3.53	0	-2.30	1.23	0
211	ULS-STR	2.99	1.66	0	-2.62	15.57	0	-2.62	15.57	0	-1.38	1.66	0
212	ULS-STR	4.99	1.66	0	-4.36	10.17	0	-4.36	10.17	0	-2.30	1.66	0
231	ULS-STR	2.99	1.66	0	-2.62	10.17	0	-2.62	10.17	0	-1.38	1.66	0
251	ULS-STR	2.99	1.41	0	-2.62	14.86	0	-2.62	14.86	0	-1.38	1.41	0
252	ULS-STR	4.99	1.41	0	-4.36	9.46	0	-4.36	9.46	0	-2.30	1.41	0
301	SLS	0.00	1.23	0	0.00	6.53	0	0.00	6.53	0	0.00	1.23	0
302	SLS	0.00	1.23	0	0.00	10.73	0	0.00	10.73	0	0.00	1.23	0
303	SLS	3.32	1.23	0	-2.91	3.53	0	-2.91	3.53	0	-1.53	1.23	0
311	SLS	1.99	1.23	0	-1.75	10.73	0	-1.75	10.73	0	-0.92	1.23	0
312	SLS	3.32	1.23	0	-2.91	7.13	0	-2.91	7.13	0	-1.53	1.23	0
331	SLS	0.00	1.23	0	0.00	4.97	0	0.00	4.97	0	0.00	1.23	0
332	SLS	0.66	1.23	0	-0.58	3.53	0	-0.58	3.53	0	-0.31	1.23	0
351	SLS	0.00	1.23	0	0.00	3.53	0	0.00	3.53	0	0.00	1.23	0
601	SEISM	0.00	1.23	0	0.00	4.97	0	0.00	4.97	0	0.00	1.23	0

8. Results of static-linear-elastic analysis**8.1. Displacements [mm]**

L.C.		Hor. defl. Column Dx mm	Vert. defl. Apex Dy mm	Bending Defl. Rafter w mm
101	ULS-EQU	0.078	36.565	7.410
102	ULS-EQU	0.127	59.595	12.079
103	ULS-EQU	48.484	3.416	9.447
111	ULS-EQU	48.470	3.290	8.089
121	ULS-EQU	29.191	49.576	15.303
122	ULS-EQU	48.526	23.156	13.449
201	ULS-STR	0.083	39.096	7.923
202	ULS-STR	0.139	65.669	13.311
203	ULS-STR	55.937	0.846	10.274
210	ULS-STR	55.926	4.369	9.217
211	ULS-STR	33.676	54.108	17.030
212	ULS-STR	55.985	23.623	14.891
231	ULS-STR	33.627	31.331	12.412
251	ULS-STR	33.670	51.128	16.426
252	ULS-STR	55.979	20.643	14.288
301	SLS	0.059	27.554	5.584
302	SLS	0.096	45.270	9.176
303	SLS	37.294	2.054	7.151
311	SLS	22.454	37.562	11.655
312	SLS	37.327	17.239	10.230
331	SLS	0.045	20.974	4.250
332	SLS	7.484	12.331	3.845
351	SLS	0.032	14.900	3.019

8.2. Reactions at the supports

Horizontal Force		Hed [kN], Vertical ForceVed [kN],			Moment Med [kNm]		
		Left support 1			Right support 2		
L.C.		Hed,1 kN	Ved,1 kN	Med,1 kNm	Hed,2 kN	Ved,2 kN	Med,2 kNm
101	ULS-EQU	18.4	77.7	0.0	-18.4	77.7	0.0
102	ULS-EQU	29.9	118.6	0.0	-29.9	118.6	0.0
103	ULS-EQU	-26.3	7.8	0.0	-22.0	32.4	0.0
111	ULS-EQU	-29.7	-8.4	0.0	-18.6	16.3	0.0
121	ULS-EQU	8.1	94.2	0.0	-37.1	109.0	0.0
122	ULS-EQU	-16.4	42.9	0.0	-31.9	67.5	0.0
201	ULS-STR	19.6	82.2	0.0	-19.6	82.2	0.0
202	ULS-STR	33.0	129.4	0.0	-33.0	129.4	0.0
203	ULS-STR	-31.9	1.5	0.0	-23.8	29.9	0.0
210	ULS-STR	-34.5	-11.1	0.0	-21.2	17.4	0.0
211	ULS-STR	7.8	101.3	0.0	-41.2	118.3	0.0
212	ULS-STR	-20.5	42.0	0.0	-35.3	70.4	0.0
231	ULS-STR	-3.7	60.8	0.0	-29.8	77.8	0.0
251	ULS-STR	6.3	94.1	0.0	-39.7	111.2	0.0
252	ULS-STR	-22.0	34.8	0.0	-33.8	63.3	0.0
301	SLS	13.8	58.4	0.0	-13.8	58.4	0.0
302	SLS	22.7	89.9	0.0	-22.7	89.9	0.0
303	SLS	-20.5	4.6	0.0	-16.6	23.5	0.0
311	SLS	5.9	71.1	0.0	-28.2	82.5	0.0
312	SLS	-12.9	31.6	0.0	-24.3	50.5	0.0
331	SLS	10.5	46.7	0.0	-10.5	46.7	0.0
332	SLS	1.9	29.6	0.0	-9.3	33.4	0.0
351	SLS	7.5	35.9	0.0	-7.5	35.9	0.0

8.3. Axial forces Ned [kN]

L.C.	Left column 1 Ned,1	Left rafter 2 Ned,2	Right rafter 3 Ned,3	Right column 4 Ned,4
101 ULS-EQU	-71.3	-18.4	-18.4	-71.3
102 ULS-EQU	-112.3	-29.9	-29.9	-112.3
103 ULS-EQU	-1.4	-6.7	-6.7	-26.1
111 ULS-EQU	12.6	-3.4	-3.4	-12.0
121 ULS-EQU	-87.9	-27.9	-27.9	-102.7
122 ULS-EQU	-36.5	-16.7	-16.7	-61.2
201 ULS-STR	-75.8	-19.6	-19.6	-75.8
202 ULS-STR	-123.1	-33.0	-33.0	-123.1
203 ULS-STR	4.8	-6.2	-6.2	-23.6
210 ULS-STR	15.8	-3.6	-3.6	-12.7
211 ULS-STR	-94.9	-30.7	-30.7	-112.0
212 ULS-STR	-35.7	-17.7	-17.7	-64.1
231 ULS-STR	-54.4	-19.2	-19.2	-71.5
251 ULS-STR	-88.7	-29.2	-29.2	-105.8
252 ULS-STR	-29.4	-16.2	-16.2	-57.9
301 SLS	-53.7	-13.8	-13.8	-53.7
302 SLS	-85.2	-22.7	-22.7	-85.2
303 SLS	0.1	-4.9	-4.9	-18.8
311 SLS	-66.4	-21.2	-21.2	-77.8
312 SLS	-26.9	-12.5	-12.5	-45.8
331 SLS	-42.0	-10.5	-10.5	-42.0
332 SLS	-24.9	-7.0	-7.0	-28.7
351 SLS	-31.2	-7.5	-7.5	-31.2

8.4. Shearing forces Ved [kN]

L.C.	Left column 1 VedA,1 VedB,1		Left rafter 2 VedA,2 VedC,2 VedB,2			Right rafter 3 VedA,3 VedC,3 VedB,3			Right column 4 VedA,4 VedB,4	
101 ULS-EQU	-18.4	-18.4	65.0	52.0	0.0	0.0	-52.0	-65.0	18.4	18.4
102 ULS-EQU	-29.9	-29.9	105.9	84.8	0.0	0.0	-84.8	-105.9	29.9	29.9
103 ULS-EQU	26.3	-6.7	-4.9	-6.4	-12.3	-12.3	-18.2	-19.7	6.7	22.0
111 ULS-EQU	29.7	-3.4	-16.9	-15.9	-12.3	-12.3	-8.7	-7.8	3.4	18.6
121 ULS-EQU	-8.1	-27.9	81.5	63.7	-7.4	-7.4	-78.5	-96.3	27.9	37.1
122 ULS-EQU	16.4	-16.7	30.2	21.7	-12.3	-12.3	-46.3	-54.8	16.7	31.9
201 ULS-STR	-19.6	-19.6	69.5	55.6	0.0	0.0	-55.6	-69.5	19.6	19.6
202 ULS-STR	-33.0	-33.0	116.7	93.4	0.0	0.0	-93.4	-116.7	33.0	33.0
203 ULS-STR	31.9	-6.2	-11.2	-11.8	-14.2	-14.2	-16.6	-17.2	6.2	23.8
210 ULS-STR	34.5	-3.6	-20.5	-19.2	-14.2	-14.2	-9.2	-8.0	3.6	21.2
211 ULS-STR	-7.8	-30.7	88.6	69.2	-8.5	-8.5	-86.2	-105.6	30.7	41.2
212 ULS-STR	20.5	-17.7	29.3	20.6	-14.2	-14.2	-49.0	-57.7	17.7	35.3
231 ULS-STR	3.7	-19.2	48.1	36.8	-8.5	-8.5	-53.8	-65.1	19.2	29.8
251 ULS-STR	-6.3	-29.2	83.3	64.9	-8.5	-8.5	-82.0	-100.3	29.2	39.7
252 ULS-STR	22.0	-16.2	24.0	16.4	-14.2	-14.2	-44.8	-52.4	16.2	33.8
301 SLS	-13.8	-13.8	49.0	39.2	0.0	0.0	-39.2	-49.0	13.8	13.8
302 SLS	-22.7	-22.7	80.5	64.4	0.0	0.0	-64.4	-80.5	22.7	22.7
303 SLS	20.5	-4.9	-4.8	-5.7	-9.5	-9.5	-13.2	-14.1	4.9	16.6
311 SLS	-5.9	-21.2	61.7	48.2	-5.7	-5.7	-59.6	-73.1	21.2	28.2
312 SLS	12.9	-12.5	22.2	15.9	-9.5	-9.5	-34.8	-41.1	12.5	24.3
331 SLS	-10.5	-10.5	37.3	29.8	0.0	0.0	-29.8	-37.3	10.5	10.5
332 SLS	-1.9	-7.0	20.2	15.8	-1.9	-1.9	-19.6	-24.0	7.0	9.3
351 SLS	-7.5	-7.5	26.5	21.2	0.0	0.0	-21.2	-26.5	7.5	7.5

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

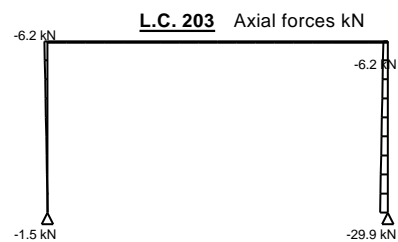
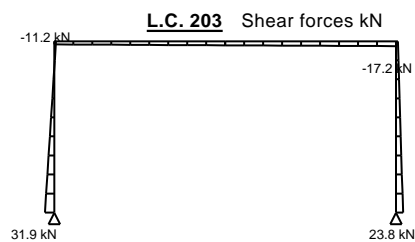
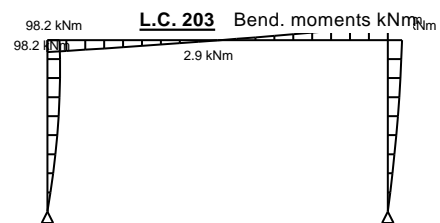
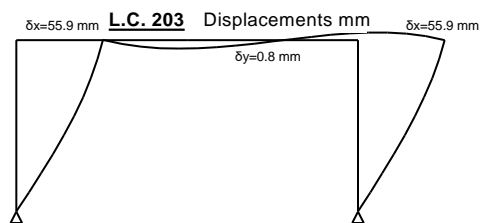
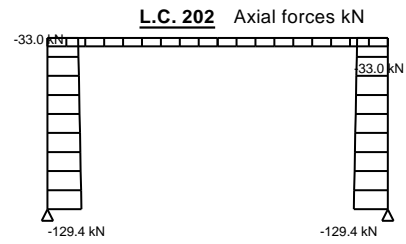
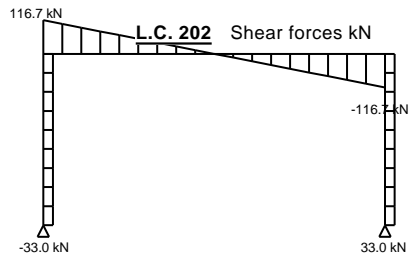
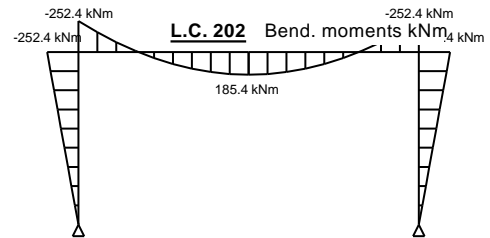
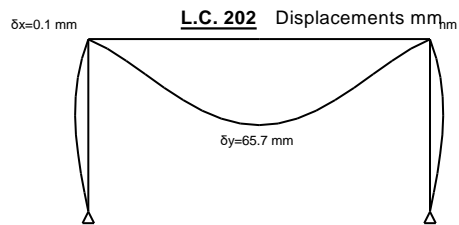
8.5. Bending moments Med [kNm]

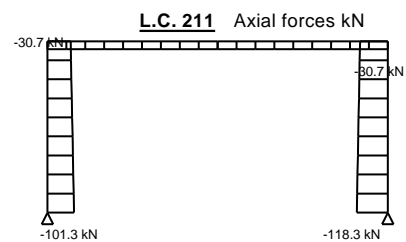
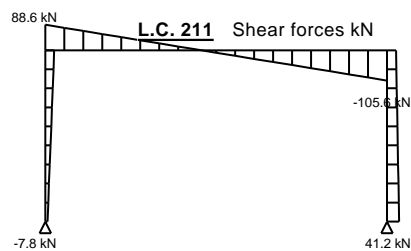
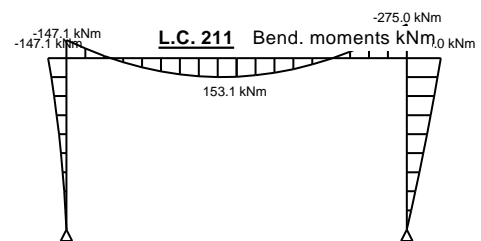
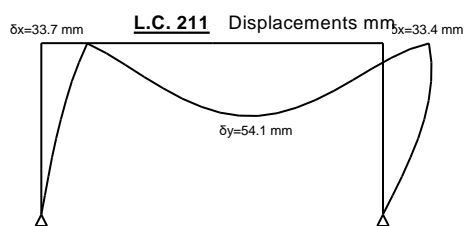
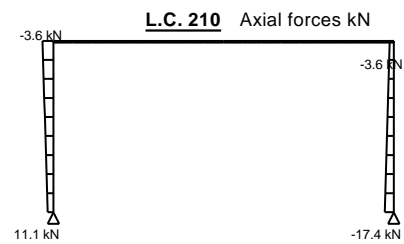
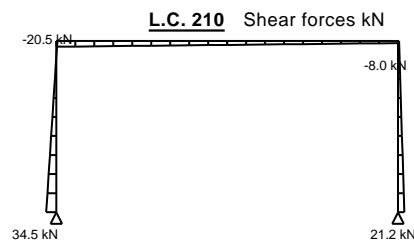
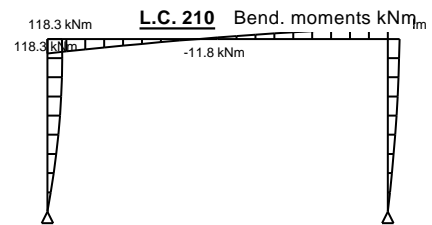
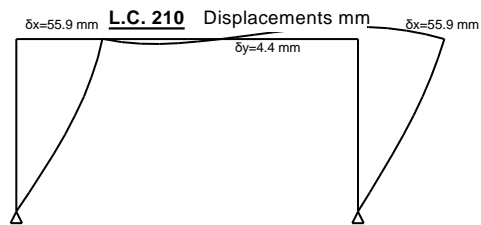
L.C.		Left column 1			Right column 4		
		MedA,1	MedM,1	MedB,1	MedA,4	MedM,4	MedB,4
101	ULS-EQU	0.0	-70.2	-140.5	-140.5	-70.2	0.0
102	ULS-EQU	0.0	-114.5	-229.0	-229.0	-114.5	0.0
103	ULS-EQU	0.0	80.1	74.8	-110.0	-55.0	0.0
111	ULS-EQU	0.0	101.9	100.6	-84.2	-42.1	0.0
121	ULS-EQU	0.0	-68.9	-137.8	-248.6	-124.3	0.0
122	ULS-EQU	0.0	31.1	-1.0	-185.9	-92.9	0.0
201	ULS-STR	0.0	-75.1	-150.2	-150.2	-75.1	0.0
202	ULS-STR	0.0	-126.2	-252.4	-252.4	-126.2	0.0
203	ULS-STR	0.0	102.1	98.2	-115.0	-57.5	0.0
210	ULS-STR	0.0	119.6	118.3	-95.0	-47.5	0.0
211	ULS-STR	0.0	-73.5	-147.1	-275.0	-137.5	0.0
212	ULS-STR	0.0	42.0	10.7	-202.6	-101.3	0.0
231	ULS-STR	0.0	2.2	-59.5	-187.5	-93.7	0.0
251	ULS-STR	0.0	-67.8	-135.6	-263.6	-131.8	0.0
252	ULS-STR	0.0	48.4	22.1	-191.1	-95.6	0.0
301	SLS	0.0	-52.9	-105.9	-105.9	-52.9	0.0
302	SLS	0.0	-87.0	-174.0	-174.0	-87.0	0.0
303	SLS	0.0	63.4	59.8	-82.4	-41.2	0.0
311	SLS	0.0	-51.9	-103.8	-189.1	-94.5	0.0
312	SLS	0.0	25.0	1.4	-140.8	-70.4	0.0
331	SLS	0.0	-40.3	-80.6	-80.6	-40.3	0.0
332	SLS	0.0	-16.9	-33.8	-62.3	-31.1	0.0
351	SLS	0.0	-28.6	-57.2	-57.2	-28.6	0.0

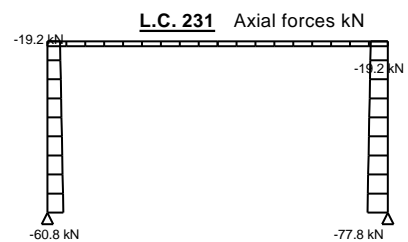
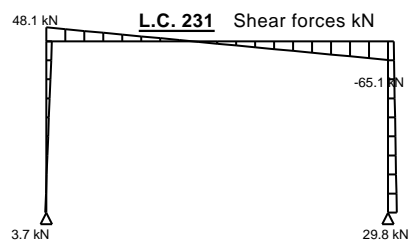
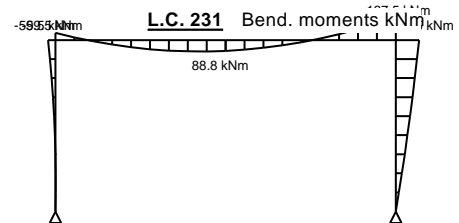
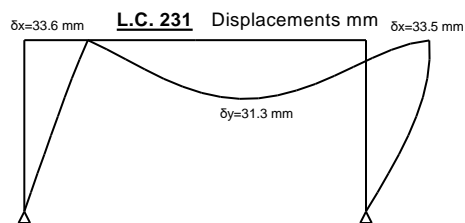
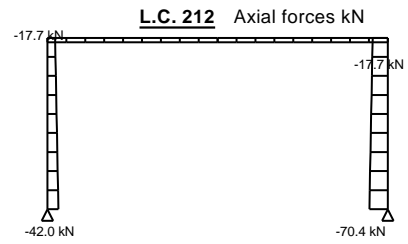
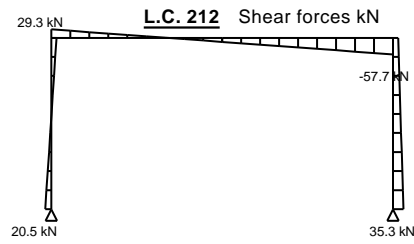
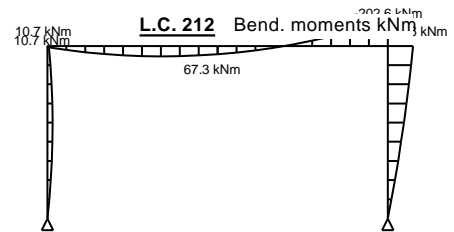
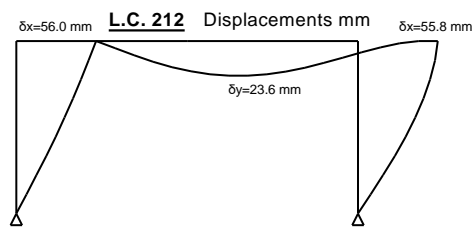
A:left end, C:haunch end, M: span, B: right end

L.C.		Left rafter 2				Right rafter 3			
		MedA,2	MedC2	MedM,2	MedB,2	MedA,3	MedM,3	MedC3	MedB,3
101	ULS-EQU	-140.5	-52.8	-18.6	103.2	103.2	-18.6	-52.8	-140.5
102	ULS-EQU	-229.0	-86.0	-30.4	168.3	168.3	-86.0	-86.0	-229.0
103	ULS-EQU	74.8	66.3	42.5	10.1	10.1	-49.9	-81.5	-110.0
111	ULS-EQU	100.6	76.0	45.9	-8.8	-8.8	-46.5	-71.9	-84.2
121	ULS-EQU	-137.8	-28.8	142.6	140.3	140.3	-54.2	-117.5	-248.6
122	ULS-EQU	-1.0	37.8	79.3	65.9	65.9	-60.0	-110.0	-185.9
201	ULS-STR	-150.2	-56.4	-19.9	110.4	110.4	-19.9	-56.4	-150.2
202	ULS-STR	-252.4	-94.8	-33.5	185.4	185.4	-33.5	-94.8	-252.4
203	ULS-STR	98.2	81.0	50.6	2.9	2.9	-56.0	-89.6	-115.0
210	ULS-STR	118.3	88.5	53.2	-11.8	-11.8	-53.4	-82.1	-95.0
211	ULS-STR	-147.1	-28.8	155.9	153.1	153.1	-60.9	-131.1	-275.0
212	ULS-STR	10.7	48.1	84.7	67.3	67.3	-67.6	-122.5	-202.6
231	ULS-STR	-59.5	4.1	93.6	88.8	88.8	-49.3	-98.2	-187.5
251	ULS-STR	-135.6	-24.5	147.7	144.7	144.7	-59.4	-126.8	-263.6
252	ULS-STR	22.1	52.4	78.7	58.9	58.9	-66.1	-118.2	-191.1
301	SLS	-105.9	-39.8	-14.0	77.8	77.8	-14.0	-39.8	-105.9
302	SLS	-174.0	-65.3	-23.1	127.8	127.8	-23.1	-65.3	-174.0
303	SLS	59.8	51.8	33.0	6.2	6.2	-38.1	-61.9	-82.4
311	SLS	-103.8	-21.3	108.1	106.3	106.3	-41.4	-89.6	-189.1
312	SLS	1.4	29.9	59.7	49.0	49.0	-45.9	-83.8	-140.8
331	SLS	-80.6	-30.3	-10.7	59.2	59.2	-10.7	-30.3	-80.6
332	SLS	-33.8	-6.8	35.5	34.9	34.9	-13.7	-29.6	-62.3
351	SLS	-57.2	-21.5	-7.6	42.0	42.0	42.0	-21.5	-57.2

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







9. Results of dynamic analysis

9.1. Eigenfrequencies and Eigenperiods of the structure

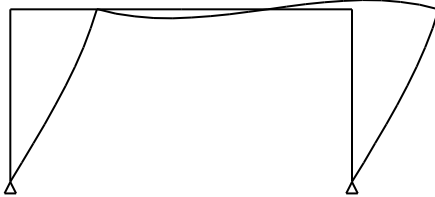
Mass of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$

Total vertical load of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$, $G=93.4$ kN

Mass of building: $m=93.369 \times 1000 / 9.81 = 9.52 \times 10^3$ kg

1	f=	1.296 Hz	T=	0.772 sec
2	f=	4.415 Hz	T=	0.226 sec
3	f=	13.667 Hz	T=	0.073 sec
4	f=	23.609 Hz	T=	0.042 sec
5	f=	33.820 Hz	T=	0.030 sec
6	f=	57.224 Hz	T=	0.017 sec
7	f=	63.558 Hz	T=	0.016 sec
8	f=	65.592 Hz	T=	0.015 sec
9	f=	122.342 Hz	T=	0.008 sec
10	f=	137.031 Hz	T=	0.007 sec
11	f=	148.601 Hz	T=	0.007 sec

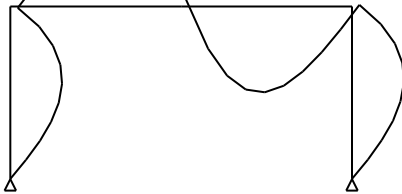
Mode shape :1, f=1.296[Hz], T=0.772[sec]



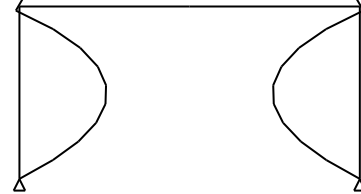
Mode shape :2, f=4.415[Hz], T=0.226[sec]



Mode shape :3, f=13.667[Hz], T=0.073[sec]



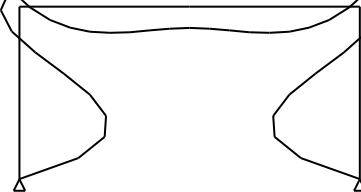
Mode shape :4, f=23.609[Hz], T=0.042[sec]



Mode shape :5, f=33.820[Hz], T=0.030[sec]



Mode shape :6, f=57.224[Hz], T=0.017[sec]



9.2. Seismic action, Lateral Force Method

(EN1998-1-1, §4.3.3.2)

Determination of Base Shear Force

(EN1998-1-1, §4.3.3.2.2)

Approximate value fundamental period of vibration T_1

(EC8 §4.3.3.2.2(3), Eq.4.6)

$$T_1 = 0.085 \cdot H^{0.75} = 0.085 \times 7.65^{0.75} = 0.391 \text{ sec}$$

Value of fundamental period of vibration resulting from dynamic analysis $T_1 = 0.772 \text{ sec}$ From Horizontal design spectrum with period of vibration $T_1 = 0.772 \text{ sec}$ we obtain

$$S_d(T_1) = 0.040 \times 1.00 \times 1.00 \times [(2.50/1.50) \times (0.50/0.772)] = 0.043 \cdot g = 0.424 \text{ m/s}^2$$

From Vertical design spectrum with period of vibration $T_1 = 0.772 \text{ sec}$ we obtain

$$S_{ve}(T_1) = 0.90 \times 0.040 \times 1.00 \times [(3.00/1.50) \times (0.15/0.772)] = 0.014 \cdot g = 0.137 \text{ m/s}^2$$

Total vertical load of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$, $G = 93.4 \text{ kN}$ Mass of building: $m = 93.369 \times 1000 / 9.81 = 9.52 \times 10^3 \text{ kg}$ Seismic Base shear force $F_b = S(T_1) \cdot m \cdot \lambda$

(EC8 §4.3.3.2.2, Eq.4.5)

$$S(T_1) = 0.424 \text{ m/s}^2, m = 9.52 \times 10^3 \text{ kg}, \lambda = 1.00, F_b = 0.424 \times 9.52 \times 1.00 = 4.0 \text{ kN}$$

Amplification factor for torsional effects

(EC8 §4.3.3.2.3, Eq.4.12)

$$\delta = 1 + 0.60 \cdot x / L_c = 1 + 0.60 \times 15000 / 30000 = 1.30$$

Seismic Base shear force $F_b = 1.30 \times 4.0 = 5.2 \text{ kN}$ Horizontal seismic force $F_b = 5.24 \text{ kN}$ is applied at lever $H = 7.650 \text{ m}$ Vertical seismic force $F_v = 0.137 \times 9.52 = 1.3 \text{ kN}$ **Displacements and internal forces M,V,N from linear elastic analysis****Displacements [mm]**

L.C.	Hor. defl. Column Dx mm	Vert. defl. Apex Dy mm	Bending Defl. Rafter w mm
602	9.538	20.979	4.250

Reactions at the supports**Horizontal Force Hed [kN], Vertical Force Ved [kN], Moment Med [kNm]**

L.C.	Left support 1			Right support 2		
	Hed,1 kN	Ved,1 kN	Med,1 kNm	Hed,2 kN	Ved,2 kN	Med,2 kNm
602	7.9	44.0	0.0	-13.2	49.4	0.0

Axial forces Ned [kN]

L.C.	Left column 1 Ned,1	Left rafter 2 Ned,2	Right rafter 3 Ned,3	Right column 4 Ned,4
602	-39.3	-13.2	-13.2	-44.7

Shearing forces Ved [kN]

L.C.	Left column 1		Left rafter 2			Right rafter 3			Right column 4	
	VedA,1	VedB,1	VedA,2	VedC,2	VedB,2	VedA,3	VedC,3	VedB,3	VedA,4	VedB,4
602	-7.9	-7.9	34.6	0.0	-2.7	-2.7	0.0	-39.9	13.2	13.2

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

Bending moments Med [kNm]

L.C.	Left column 1			Right column 4		
	MedA,1	MedM,1	MedB,1	MedA,4	MedM,4	MedB,4
602	0.0	-30.3	-60.5	-100.6	-50.3	0.0

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

L.C.	Left rafter 2			Right rafter 3		
	MedA,2	MedC,2	MedM,2	MedA,3	MedM,3	MedB,3
602	-60.5		59.2	59.2		-100.6

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

Maximum internal forces, Lateral Force Method

(EC8 §4.3.3.2.2)

Columns

NedA	=	-49.4 kN
NedB	=	-39.9 kN
VedA	=	13.2 kN
VedB	=	13.2 kN
MedA	=	0.0 kNm
MedB	=	100.6 kNm
Nedmax	=	-49.4 kN
Vedmax	=	13.2 kN
Medmax	=	100.6 kNm

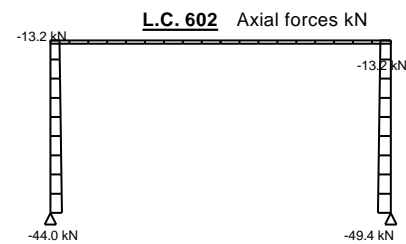
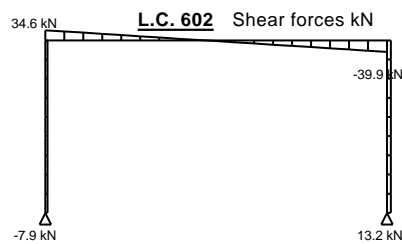
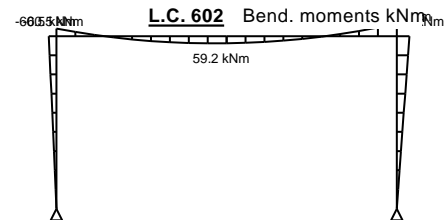
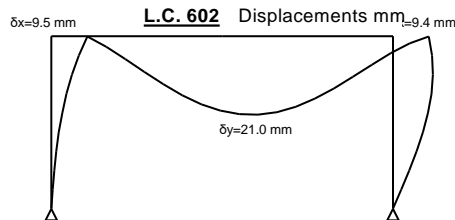
Horizontal deflection at the top of column $dx = 9.5 \text{ mm}$

Rafters

NedA	=	-13.2kN
NedB	=	-13.2kN
VedA	=	39.9kN
VedB	=	2.7kN
MedA	=	100.6kNm
MedB	=	59.2kNm
Nedmax	=	-13.2kN
Vedmax	=	39.9kN
Medmax	=	100.6kNm

Estimate of seismic forces at supports

max Downwards support force $F_{v-} = -49.4 - 1.3/2 = -50.0$ kN
max Upwards support force $F_{v+} = -44.0 + 1.3/2 = 0.0$ kN

**9.3. Seismic action, Modal Response Spectrum Analysis**

(EN1998-1-1, §4.3.3.3)

Effective modal masses of the structure

(EN1998-1-1, §4.3.3.3.1 (3))

From eigenvalue analysis we have the structure eigenperiods and eigenvectors
From response spectrum we obtain the modal spectral acceleration values

	T sec	Sd(T)/g	Sd(T)m/s ²	Sve(T)/g	Sve(T)m/s ²
1	0.772	0.043	0.424	0.014	0.137
2	0.226	0.067	0.654	0.048	0.468
3	0.073	0.046	0.453	0.072	0.706
4	0.042	0.038	0.372	0.065	0.634
5	0.030	0.035	0.339	0.052	0.514
6	0.017	0.031	0.307	0.041	0.400
7	0.016	0.031	0.303	0.039	0.384
8	0.015	0.031	0.301	0.039	0.379

Sd(T):horizontal elastic spectrum, Sve(T):vertical elastic spectrum

Modal masses $M_i = \Phi_i^T \cdot m \cdot \Phi_i$
 Modal excitations $L_i = \Phi_i^T \cdot m \cdot I_i$
 Effective modal masses $m_i = L_i^2 / M_i$
 Modal amplitudes $q_i = (L_i / M_i) \cdot S_d(T) \cdot T^2 / 4\pi^2$
 Modal displacement vectors $u_i = \delta \cdot q_i \cdot \Phi_i$
 Modal internal forces $E_i = K_e \cdot u_i$
 Maximum internal forces $E_e = \sqrt{[\sum E_i^2]}$
 Amplification factor for torsional effects (EC8 §4.3.3.2.3, Eq.4.12)
 $\delta = 1 + 0.60 \cdot x / L_c = 1 + 0.60 \times 15000 / 30000 = 1.30$

	T sec	Mi kg	Li kg	mi kg	qi m
1	0.772	1.000×10^3	3.017×10^3	3.017×10^3 (0.32xMtot)	0.02507
2	0.226	1.000×10^3	2.132×10^3	2.132×10^3 (0.22xMtot)	0.00236
3	0.073	1.000×10^3	0.344×10^3	0.344×10^3 (0.04xMtot)	0.00003
4	0.042	1.000×10^3	-1.254×10^3	1.254×10^3 (0.13xMtot)	-0.00003
5	0.030	1.000×10^3	0.799×10^3	0.799×10^3 (0.08xMtot)	0.00001
6	0.017	1.000×10^3	1.095×10^3	1.095×10^3 (0.12xMtot)	0.00000
7	0.016	1.000×10^3	0.074×10^3	0.074×10^3 (0.01xMtot)	0.00000
8	0.015	1.000×10^3	1.229×10^3	0.803×10^3 (0.08xMtot)	0.00000
Mtot=		9.518×10^3 Kg		9.518×10^3 (1.00xMtot)	

Modal displacement vectors

u1	u2	u3	u4	u5	u6	u7	u8
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	
1.376×10^{-3}	0.061×10^{-3}	2.942×10^{-6}	-9.215×10^{-6}	-4.206×10^{-6}	2.873×10^{-6}	-0.070×10^{-6}	
8.679×10^{-3}	-0.003×10^{-3}	1.136×10^{-6}	0.721×10^{-6}	-0.308×10^{-6}	-0.287×10^{-6}	-0.103×10^{-6}	
0.006×10^{-3}	0.004×10^{-3}	-0.211×10^{-6}	0.873×10^{-6}	0.200×10^{-6}	0.549×10^{-6}	0.002×10^{-6}	
0.646×10^{-3}	-0.120×10^{-3}	-4.509×10^{-6}	10.457×10^{-6}	2.492×10^{-6}	0.810×10^{-6}	0.002×10^{-6}	
8.683×10^{-3}	0.000×10^{-3}	1.206×10^{-6}	0.000×10^{-6}	-0.456×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	
0.000×10^{-3}	1.466×10^{-3}	0.000×10^{-6}	-4.980×10^{-6}	0.000×10^{-6}	-0.678×10^{-6}	-0.002×10^{-6}	
-0.315×10^{-3}	0.000×10^{-3}	9.270×10^{-6}	0.000×10^{-6}	3.241×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	
8.679×10^{-3}	0.003×10^{-3}	1.136×10^{-6}	-0.721×10^{-6}	-0.308×10^{-6}	0.287×10^{-6}	0.103×10^{-6}	
-0.006×10^{-3}	0.004×10^{-3}	0.211×10^{-6}	0.873×10^{-6}	-0.200×10^{-6}	0.549×10^{-6}	0.002×10^{-6}	
0.646×10^{-3}	0.120×10^{-3}	-4.509×10^{-6}	-10.457×10^{-6}	2.492×10^{-6}	-0.810×10^{-6}	-0.002×10^{-6}	
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	
0.000×10^{-3}	0.000×10^{-3}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	0.000×10^{-6}	
1.376×10^{-3}	-0.061×10^{-3}	2.942×10^{-6}	9.215×10^{-6}	-4.206×10^{-6}	-2.873×10^{-6}	0.070×10^{-6}	

Modal internal forces

	0.772s	0.226s	0.073s	0.042s	0.030s	0.017s	0.016s	0.015s
Columns								
NA kN	-2.5	-1.9	0.1	-0.4	-0.1	-0.2	0.0	-0.3
NB kN	-2.5	-1.9	0.1	-0.4	-0.1	-0.2	0.0	-0.3
VA kN	2.4	0.6	0.0	0.0	0.0	0.0	0.0	0.0
VB kN	-2.4	-0.6	0.0	0.0	0.0	0.0	0.0	0.0
MA kNm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
MB kNm	-18.2	-4.4	-0.2	0.3	0.0	0.1	0.0	0.1
Rafters								
NA kN	-1.1	-0.6	0.0	0.2	0.0	-0.1	0.0	0.0
NB kN	-1.1	-0.6	0.0	0.2	0.0	-0.1	0.0	0.0
VA kN	-2.5	-1.5	0.0	-0.1	0.0	0.0	0.0	0.0
VB kN	2.5	1.5	0.0	0.1	0.0	0.0	0.0	0.0
MA kNm	18.5	5.1	0.0	0.3	0.1	0.0	0.0	0.0
MB kNm	-0.1	6.2	0.2	0.1	0.1	0.0	0.0	0.0

Maximum internal forces, Modal Response Spectrum Analysis

(EC8 §4.3.3.3.2 (2))

Column										
NedA =	$-46.7 - \sqrt{[2.5^2 + 1.9^2 + 0.1^2 + 0.4^2 + 0.1^2 + 0.2^2 + 0.0^2 + 0.3^2]}$									-49.9 kN
NedB =	$-37.3 - \sqrt{[2.5^2 + 1.9^2 + 0.1^2 + 0.4^2 + 0.1^2 + 0.2^2 + 0.0^2 + 0.3^2]}$									-40.5 kN
VedA =	$-10.5 - \sqrt{[2.4^2 + 0.6^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2]}$									13.0 kN
VedB =	$-10.5 - \sqrt{[2.4^2 + 0.6^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2]}$									13.0 kN
MedA =	$0.0 + \sqrt{[0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2 + 0.0^2]}$									0.0 kNm
MedB =	$-80.6 - \sqrt{[18.2^2 + 4.4^2 + 0.2^2 + 0.3^2 + 0.0^2 + 0.1^2 + 0.0^2 + 0.1^2]}$									-99.3 kNm
Nedmax=	-49.9 kN									
Vedmax=	13.0 kN									
Medmax=	99.3 kNm									

Horizontal deflection at the top of column dx=8.7 mm

Rafter

Ned A=	-10.5-√[1.1 ² +	0.6 ² +	0.0 ² +	0.2 ² +	0.0 ² +	0.1 ² +	0.0 ² +	0.0 ²]	=	-11.8 kN
Ned B=	-10.5-√[1.1 ² +	0.6 ² +	0.0 ² +	0.2 ² +	0.0 ² +	0.1 ² +	0.0 ² +	0.0 ²]	=	-11.8 kN
Ved A=	37.3+√[2.5 ² +	1.5 ² +	0.0 ² +	0.1 ² +	0.0 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	40.2 kN
Ved B=	0.0+√[2.5 ² +	1.5 ² +	0.0 ² +	0.1 ² +	0.0 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	2.9 kN
Med A=	-80.6-√[18.5 ² +	5.1 ² +	0.0 ² +	0.3 ² +	0.1 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	-99.8 kNm
Med B=	59.2+√[0.1 ² +	6.2 ² +	0.2 ² +	0.1 ² +	0.1 ² +	0.0 ² +	0.0 ² +	0.0 ²]	=	65.4 kNm
Nedmax=	-11.8 kN										
Vedmax=	40.2 kN										
Medmax=	99.8 kNm										

9.4. Design for seismic loading

(EN1998-1-1, §6)

Maximum design values for Deflections, internal forces and moments

Columns

NedA	=	-49.9kN
NedB	=	-40.5kN
VedA	=	13.2kN
VedB	=	13.2kN
MedA	=	0.0kNm
MedB	=	100.6kNm
Nedmax	=	-49.9kN
Vedmax	=	13.2kN
Medmax	=	100.6kNm

Horizontal deflection at the top of column dx=9.5 mm

Rafters

NedA	=	-13.2kN
NedB	=	-13.2kN
VedA	=	40.2kN
VedB	=	2.9kN
MedA	=	100.6kNm
MedB	=	65.4kNm
Nedmax	=	-13.2kN
Vedmax	=	40.2kN
Medmax	=	100.6kNm, at haunch-start Medmax = 80.5kNm

9.5. Second Order effects

(EC8 §4.4.2.2(2))

$$\theta = P_{tot} \cdot d_r / (V_{tot} \cdot h) = 93.4 \times 9.5 / (5.2 \times 7650) = 0.022$$

$$\theta = 0.022 \leq 0.20, \quad 1/(1-\theta) = 1/(1-0.022) = 1.023$$

Second Order effects are taken into account by multiplying the seismic actions by a factor $1/(1-\theta) = 1.023$

(EC8 §4.4.2.2(3))

9.6. Maximum forces and bending moments for seismic load

(EC8 §6.6)

Columns

Ned	=	Ned,g + 1.1 · γ _{ov} · Ω · [1/(1-θ)] · Ned,e
Ved	=	Ved,g + 1.1 · γ _{ov} · Ω · [1/(1-θ)] · Ved,e
Med	=	Med,g + 1.1 · γ _{ov} · Ω · [1/(1-θ)] · Med,e

(EC8 §6.6.3(1))

Rafters

Ned	=	Ned,g + [1/(1-θ)] · Ned,e
Ved	=	Ved,g + [1/(1-θ)] · Ved,e
Med	=	Med,g + [1/(1-θ)] · Med,e

(EC8 §6.6.2(2))

$$\gamma_{ov} = 1.25$$

(EC8 §6.2(3))

$$\Omega = M_{pl,rd} / Med = [10^{-6}] \times (1.307 \times 10^6 \times 275 / 1.00) / 80.5 = 4.465$$

(EC8 §6.6.3(1))

Columns

NedA	=	-46.7 + 1.10 × 1.25 × 4.465 × 1.023 × (-46.7 - 49.9)	=	-67.0kN
NedB	=	-37.3 + 1.10 × 1.25 × 4.465 × 1.023 × (-37.3 - 40.5)	=	-57.6kN
VedA	=	10.5 + 1.10 × 1.25 × 4.465 × 1.023 × (13.2 - 10.5)	=	27.0kN
VedB	=	10.5 + 1.10 × 1.25 × 4.465 × 1.023 × (13.2 - 10.5)	=	27.0kN
MedA	=	0.0 + 1.10 × 1.25 × 4.465 × 1.023 × (0.0 - 0.0)	=	0.0kNm
MedB	=	80.6 + 1.10 × 1.25 × 4.465 × 1.023 × (100.6 - 80.6)	=	206.4kNm
Nedmax	=	-67.0kN		
Vedmax	=	27.0kN		
Medmax	=	206.4kNm		

(EC8 §6.6.3(1))

Rafters

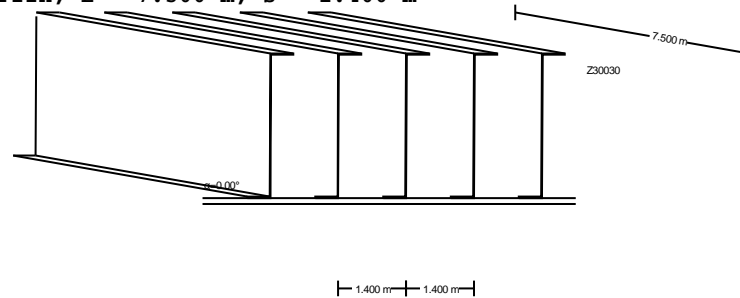
(EC8 §6.6.2(2))

NedA =	-10.5+1.023x(10.5-	13.2)=	-13.2kN
NedB =	-10.5+1.023x(10.5-	13.2)=	-13.2kN
VedA =	37.3+1.023x(40.2-	37.3)=	40.2kN
VedB =	0.0+1.023x(2.9-	0.0)=	2.9kN
MedA =	80.6+1.023x(100.6-	80.6)=	101.1kNm
MedB =	59.2+1.023x(65.4-	59.2)=	65.5kNm
Nedmax=	-13.2kN			
Vedmax=	40.2kN			
Medmax=	101.1kNm			

10. Design of Purlins

Purlin laterally restrained, Z30030 S 275

Simply supported purlin, L= 7.500 m, s= 1.400 m



10.1. Materials, Purlins

Steel: S 275

(EN1993-1-1, §3.2)

$t \leq 40$ mm, Yield strength $f_y = 275$ N/mm², Ultimate strength $f_u = 430$ N/mm²

$40 \text{ mm} < t \leq 80$ mm, Yield strength $f_y = 255$ N/mm², Ultimate strength $f_u = 410$ N/mm²

Modulus of elasticity $E = 210000$ N/mm², Poisson ratio $\nu = 0.30$, Unit mass $\rho = 7850$ Kg/m³

Partial safety factors for actions

(EN1990, Annex A1)

$\gamma_{G,sup} = 1.35$, $\gamma_Q = 1.50$, $\gamma_{G,inf} = 1.00$, $\psi_0 = 0.70$

Partial factors for materials

(EN1993-1-1, §6.1)

$\gamma_{M0} = 1.00$, $\gamma_{M1} = 1.00$, $\gamma_{M2} = 1.25$

10.2. Loading, Purlins

(EN1991-1-1)

Roof loads

Roof slope	$\alpha = 0.00^\circ$	
Load of roof covering	$g_{k1} = 0.300$ kN/m ²	(EN1991-1-1 §5)
Imposed load (category H)	$q_k = 0.400$ kN/m ²	(EN1991-1-1 §6.3.4.2)
Snow load	$q_{sk} = 0.960$ kN/m ²	(EN1991-1-3 §5.3)
Wind pressure	$w_k = 0.000$ kN/m ²	(EN1991-1-4 §7.2)
Wind uplift	$w_k = -0.388$ kN/m ²	

Load on purlin

Purlin spacing	$s = 1.400$ m
Load of roof covering	$G_{k1} = 1.400 \times 0.300 = 0.42$ kN/m
Purlin weight	$G_{k2} = 0.12$ kN/m
Permanent load	$G_k = G_{k1} + G_{k2} = 0.42 + 0.12 = 0.54$ kN/m
Imposed load (category H)	$Q_{kk} = 1.400 \times 0.400 = 0.56$ kN/m
Snow load	$Q_{sk} = 1.400 \times 0.960 = 1.34$ kN/m
Wind uplift	$Q_{wk} = -1.400 \times 0.388 = -0.54$ kN/m

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

Permanent load	$G_{k,z} = 0.54 \times \cos(0.00) = 0.54$ kN/m, $G_{k,y} = 0.54 \times \sin(0.00) = 0.00$ kN/m
Imposed load (category H)	$Q_{kk,z} = 0.56 \times \cos(0.00) = 0.56$ kN/m, $Q_{kk,y} = 0.56 \times \sin(0.00) = 0.00$ kN/m
Snow load	$Q_{sk,z} = 1.34 \times \cos(0.00) = 1.34$ kN/m, $Q_{sk,y} = 1.34 \times \sin(0.00) = 0.00$ kN/m
Wind pressure	$Q_{wk,z} = 0.00$ kN/m, $Q_{wk,y} = 0.00$ kN/m
Wind uplift	$Q_{wk,z} = -0.54$ kN/m, $Q_{wk,y} = 0.00$ kN/m

10.3. Design values of Actions, Load combinations, Purlins

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

Sagging $\gamma_G \cdot \sup \cdot G_{k,z} + \gamma_Q \cdot Q_{k,z} + \gamma_Q \cdot \psi_0 \cdot Q_{wk,z} = 1.35 \times 0.54 + 1.50 \times 1.34 + 1.50 \times 0.60 \times 0.00 = 2.74 \text{ kN/m}$
 Hogging $\gamma_G \cdot \inf \cdot G_{k,z} - \gamma_Q \cdot Q_{wk,z} = 1.00 \times 0.54 - 1.50 \times 0.54 = -0.27 \text{ kN/m}$

Serviceability Limit State (SLS), Load combinations

(EN1990 §6.5.3, T.A1.4)

Sagging $G_{k,z} + Q_{k,z} + \psi_0 \cdot Q_{wk,z} = 0.54 + 1.34 + 0.60 \times 0.00 = 1.88 \text{ kN/m}$
 Hogging $G_{k,z} + Q_{wk,z} = 0.54 - 0.54 = 0.00 \text{ kN/m}$

10.4. Design actions, Purlins

Design actions, Ultimate Limit State

Sagging $M_{yed} = 2.74 \times 7.500^2 / 8 = 19.26 \text{ kNm}$, $V_{zed} = 2.74 \times 7.500 / 2 = 10.27 \text{ kN}$
 Hogging $M_{yed} = -0.27 \times 7.500^2 / 8 = -1.90 \text{ kNm}$, $V_{zed} = 0.27 \times 7.500 / 2 = 1.01 \text{ kN}$

Design actions, Serviceability Limit State (SLS)

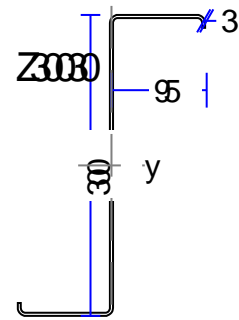
Sagging $M_{yed} = 1.88 \times 7.500^2 / 8 = 13.22 \text{ kNm}$, $V_{zed} = 1.88 \times 7.500 / 2 = 7.05 \text{ kN}$

10.5. Steel cross-section properties, Purlins

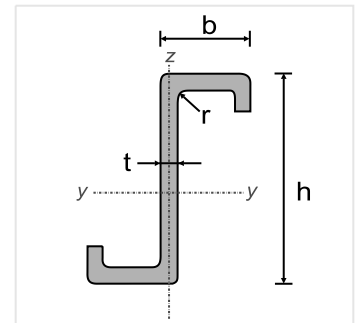
Cross-section Z30030-S 275

Dimensions of cross section

Depth of cross section $h = 300.00 \text{ mm}$
 Width of cross section $b = 95.00 \text{ mm}$
 Web depth $h_w = 297.00 \text{ mm}$
 Depth of straight portion of web $d_w = 284.00 \text{ mm}$
 Web thickness $t_w = 3.00 \text{ mm}$
 Flange thickness $t_f = 3.00 \text{ mm}$
 Radius of root fillet $r = 5.00 \text{ mm}$
 Mass $= 11.97 \text{ Kg/m}$

**Properties of cross section**

Area $A = 1508 \text{ mm}^2$
 Second moment of area $I_y = 19.836 \times 10^6 \text{ mm}^4$, $I_z = 2.271 \times 10^6 \text{ mm}^4$
 Section modulus $W_y = 118.11 \times 10^3 \text{ mm}^3$, $W_z = 22.480 \times 10^3 \text{ mm}^3$
 Plastic section modulus $W_{py} = 118.11 \times 10^3 \text{ mm}^3$, $W_{pz} = 22.480 \times 10^3 \text{ mm}^3$
 Depth of gyration $i_y = 114.7 \text{ mm}$, $i_z = 38.8 \text{ mm}$
 Shear area $A_{vz} = 891 \text{ mm}^2$, $A_{vy} = 570 \text{ mm}^2$
 Torsional constant $I_t = 0.005 \times 10^6 \text{ mm}^4$, $i_p = 121 \text{ mm}$
 Torsional modulus $W_t = 1.758 \times 10^3 \text{ mm}^3$
 Warping constant $I_w = 9.454 \times 10^9 \text{ mm}^6$



(EN1993-1-1, §7)

10.6. Serviceability Limit State (SLS), Purlins

Purlin deflections, Sagging

Loading $G+Q$: $w = 5 \times 1.88 \times 7500^4 / (384 \times 2.1 \times 10^5 \times 19.836 \times 10^6) = 18.59 \text{ mm} = L/404 < L/200$

Loading Q : $w = 5 \times 1.34 \times 7500^4 / (384 \times 2.1 \times 10^5 \times 19.836 \times 10^6) = 13.25 \text{ mm} = L/566 < L/250$

Purlin deflections, Hogging

Loading Q : $w = 5 \times 0.54 \times 7500^4 / (384 \times 2.1 \times 10^5 \times 19.836 \times 10^6) = -5.34 \text{ mm} = L/1403 < L/250$

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

10.7. Classification of steel cross-section, Bending My (Purlin section)

(EN1993-1-1, §5.5)

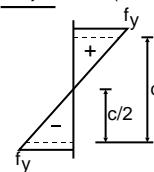
Web

$c = 300.0 - 2 \times 3.0 - 2 \times 5.0 = 284.0 \text{ mm}$, $t = 3.0 \text{ mm}$, $c/t = 284.0 / 3.0 = 94.67$

S 275, $t = 3.0 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$

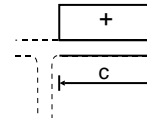
$83\epsilon = 83 \times 0.92 = 76.36 < c/t = 94.67 < 124\epsilon = 124 \times 0.92 = 114.08$

The web is class 3 (EN1993-1-1, Tab.5.2)



Flange

c=***=95.0 mm, t=3.0 mm, c/t=95.0/3.0=31.67
 S 275, t= 3.0 ≤ 40 mm, f_y=275 N/mm², ε=(235/275)^{0.5}=0.92
 c/t=31.67 > 14ε=14x0.92=12.88
 The flange is class 4 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 4, Bending My,ed

Effective cross-section properties of Class 4 cross-sections

(EN1993-1-1, §6.2.2.5)

Effective area A_{eff}=1508 mm²

Flange

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

b=c=95.0mm, t=3.0mm, ε=0.92, ψ=1.00, σ=0.43, $\bar{\lambda}_p=1.848$

$\bar{\lambda}_p=1.848 > 0.673$ ρ=[1-0.188/1.848]/1.848=0.486 (ρ<1.0), c_{eff}=ρ·c=0.486x95=46.2 mm

Effective area A_{eff}=0-1x(95.0-46.2)x3.00=-146 mm²

e_{my} ½(300.0-3.0)x(0/-146-1)=-148.50 mm, I_{y,eff}= 0.000x10⁶ mm⁴

Effective section modulus W_{y,eff}= 0.000x10⁶/(300.0/2+-148.50)= 0.000x10³ mm³

10.8. Resistance of cross-section, Purlin section

(EN1993-1-1, §6.2)

Effective cross-section properties of Class 4 cross-sections

(EN1993-1-1, §6.2.2.5)

Effective area A_{eff}=1508 mm²

e_{my}=0.00 mm, I_{y,eff}=19.836x10⁶ mm⁴

Effective section modulus W_{y,eff}=19.836x10⁶/(300.0/2+0.00)=132.24x10³ mm³

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

My,ed= 19.26 kNm

Bending Resistance M_{cy,rd}=W_{effy}·f_y/γ_{M0}=[10⁻⁶]x132.24x10³x275/1.00= 36.37kNm

My,ed= 19.26 kNm < 36.37 kNm =M_{y,rd}=M_{ply,rd}, Is verified

My,ed/My,rd= 19.26/36.37= 0.530<1

Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

Vz,ed= 10.27 kN

A_v=h_w·t=297.0x3.0=891mm²

Plastic Shear Resistance V_{pl,z,rd}=A_v(f_y/√3)/γ_{M0}= [10⁻³]x891x(275/1.73)/1.00= 141.47kN

V_{z,ed}= 10.27 kN < 141.47 kN =V_{z,rd}=V_{pl,z,rd}, Is verified

V_{z,ed}/V_{z,rd}= 10.27/141.47= 0.073<1

h_w/t_w=(300.0-2x3.0)/3.0=297.0/3.0=99.00>72ε/η=66.24 (η=1.00)

S 275, t= 3.0 ≤ 40 mm, f_y=275 N/mm², ε=(235/275)^{0.5}=0.92

Shear buckling resistance must be verified (EC3 §6.2.6.6)

Shear buckling resistance

(EC3 EN1993-1-5:2006, §5)

$\bar{\lambda}_w = (284.0/3.0) / (37.4 \times 0.92 \times \sqrt{(5.34)}) = 1.191$, K_τ=5.34

(EC3-1-5 §5, Eq.5.6, A.3)

$\bar{\lambda}_w = 1.191 > 1.08$, χ_v=0.83/1.191=0.697

(EC3-1-5 Tab.5.1)

V_{b,rd}=χ_v·f_{yw}·h_w·t/(√3γ_{M1})=0.001x275x0.697x284.0x3.0/(1.73x1.00)=94.31kN

(EC3-1-5 Tab.5.1)

V_{ed}=10 kN < 94=V_{b,rd} kN, Is verified

V_{ed}/V_{b,rd}= 10.27/94.31= 0.109<1

Ultimate Limit State, Verification for axial force, shear and bending

(EN1993-1-1, §6.2.9)

N.ed= 0.00kN, Vz.ed= 10.27kN, My.ed= 4.82kNm

My,ed= 19.26 kNm

M_{c,y,rd}=36.37kNm, V_{pl,z,rd}=94.31kN

N_{ed}=0 kN, Effect of axial force is neglected

(EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

V_{ed}=10.27kN ≤ 0.50x94.31=0.50xV_{pl,rd}=47.15kN

Effect of shear force is neglected

(EC3 §6.2.8.2)

10.9. Lateral restraining of sheeting, Purlins

(EC3 EN1993-1-3:2005, §10.1)

Sheeting thickness $t_w=0.100$ mm, Profile depth $h_w=5.0$ mm

Shear stiffness of sheeting

(EN1993-1-3, §10.1.1Eq.10.1b)

$$S = t^{1.5} (50 + 10b^{0.33}) s / h_w = 0.100^{1.5} \times (50 + 10 \times 7500^{0.33} \times 1400 / 5.0) = 2176 \text{ kNm/m}$$

Minimum required shear stiffness, for laterally restrained purlin

(§10.1.1Eq.10.1a)

$$S_{min} = [\pi^2 E \cdot I_w / L^2 + G \cdot I_t + \pi^2 E \cdot I_z (h/2)^2 / L^2] \cdot 70 / h = [\pi^2 \times 2.1 \times 10^5 \times 9.454 \times 10^9 / 7500^2 + 8.1 \times 10^4 \times 0.005 \times 10^6 + \pi^2 \times 2.1 \times 10^5 \times 2.271 \times 10^6 \times 150^2 / 7500^2] \times 70 / 300^2 \times 10^{-3} = 2067 \text{ kNm/m}$$

$$s = 2176 \text{ kNm/m} > 2067 \text{ kNm/m}$$

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

Rotational restraint given by the sheeting $C_d = 1 / (1/C_d,a + 1/C_d,c)$

(EN1993-1-3, §10.1.5.2)

$$C_d,c = k \cdot E \cdot I_{eff} / s, \quad k=2, \quad I_{eff} = 0.3 \times 0.10 \times 4.90^2 = 1 \text{ mm}^4/\text{m}, \quad s = 1400 \text{ mm}$$

(Eq.10.16)

$$C_d,c = [10^{-3}] 2 \times 2.1 \times 10^5 \times 0.7 / 1400 = 0.2 \text{ kNm/m}$$

$$C_d,a = C100 \cdot k_{ba} \cdot k_t \cdot k_{br} \cdot k_a \cdot k_{bt}$$

(EN1993-1-3, Eq.10.17)

$$C100 = 2.0, \quad k_{ba} = 1.25 \times 300 / 100 = 3.75, \quad k_t = (0.10 / 0.75)^{1.5} = 0.05, \quad k_{br} = 1.0, \quad k_a = 1.0, \quad k_{bt} = 1.0$$

$$C_d,a = 2.0 \times 3.75 \times 0.05 \times 1.0 \times 1.0 \times 1.0 = 0.4 \text{ kNm/m}$$

$$C_d = C_d,a = 0.1 \text{ kNm/m}$$

10.10. Lateral torsional buckling (Purlin laterally restrained)

(EN1993-1-1, §6.3.2)

Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)

Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961

$$M_{cr} = C1 \cdot [\pi^2 E I_z / (kL)^2] \{ \sqrt{[(kz/kw)^2 (I_w/I_z) + (kL)^2 G I_t, eq / (\pi^2 E I_z) + (C2 \cdot z_g - C3 \cdot z_j)^2]} - (C2 \cdot z_g - C3 \cdot z_j) \}$$

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

$$G = E / (2(1+\nu)) = 210000 / (2(1+0.30)) = 80769 \text{ N/mm}^2, \quad I_t, eq = I_t + C_d \cdot (kL)^2 / (\pi^2 G)$$

Hogging

$$k \cdot L = 7500 \text{ mm}, \quad z_g = -150 \text{ mm}, \quad z_j = 0 \text{ mm}$$

(EN1993:2002 Eq.C.11)

$$k_y = 1.0, \quad k_z = 1.0, \quad k_w = 1.0, \quad C1 = 1.127, \quad C2 = 0.454, \quad C3 = 0.000$$

$$M_{cr} = [10^{-6}] 1.127 \times [\pi^2 \times 2.1 \times 10^5 \times 2.271 \times 10^6 / 7500^2]$$

$$\times \{ [(1.0/1.0)^2 \times (9.454 \times 10^9 / 2.271 \times 10^6)$$

$$+ 7500^2 \times 8.1 \times 10^4 \times 0.015 \times 10^6 / (\pi^2 \times 2.1 \times 10^5 \times 2.271 \times 10^6)$$

$$+ (-0.454 \times 150)^2 \}^{0.5} - (-0.454 \times 150) \} = 20.8 \text{ kNm}$$

$$I_t, eq = (0.005 \times 10^6 + 10^3 \times 0.1 \times 7500^2 / (\pi^2 \times 8.1 \times 10^4)) = 0.015 \times 10^6 \text{ mm}^4$$

$$\bar{\lambda}, lt = \sqrt{(W_{pl,y} \cdot f_y / M_{cr})} = \sqrt{[10^{-6}] \times 132.24 \times 10^3 \times 275 / 20.8} = 1.322$$

(EC3 Eq.6.56)

$$h/b = 300/95 = 3.16 > 2.00 \text{ buckling curve: c}$$

$$\text{imperfection factor: } \alpha, lt = 0.49, \quad \beta = 0.75, \quad \chi, lt = 0.464$$

(T.6.3, T.6.5, Fig.6.4)

$$\Phi, lt = 0.5 [1 + \alpha, lt (\bar{\lambda}, lt - \bar{\lambda}, lto) + \beta \bar{\lambda}, lt^2] = 0.5 [1 + 0.49 \times (1.322 - 0.40) + 0.75 \times 1.322^2] = 1.382$$

$$\chi, lt = 1 / [\Phi, lt + \sqrt{(\Phi, lt^2 - \beta \bar{\lambda}, lt^2)}] = 1 / [1.382 + \sqrt{(1.382^2 - 0.75 \times 1.382^2)}] = 0.464$$

$$\text{Reduction factor } \chi, lt = 1 / [\Phi, lt + \sqrt{(\Phi, lt^2 - \beta \bar{\lambda}, lt^2)}], \quad \chi, lt \leq 1.0, \quad 1 / \bar{\lambda}, lt^2, \quad \chi, lt = 0.464$$

(Eq.6.57)

$$M_{b,rd} = \chi, lt \cdot W_{eff,y} \cdot f_y / \gamma_{M1} = 0.464 \times [10^{-6}] \times 132.24 \times 10^3 \times 275 / 1.00 = 16.87 \text{ kNm}$$

(EC3 Eq.6.55)

$$M_{y,ed} = 1.90 \text{ kNm} < 16.87 \text{ kNm} = M_{b,rd}, \quad \text{Is verified}$$

$$M_{y,ed} / M_{b,rd} = 1.90 / 16.87 = 0.113 < 1$$

11. Global analysis

(EN1993-1-1, §5.2)

11.1. Effects of deformed geometry of the structure

(EN1993-1-1, §5.2.1)

$$\alpha_{cr} = (H_{nhf}/V_{ed})(h/\delta h, ed)$$

(Eq.5.2)

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

Vertical reaction at the base of column

V_{ed}= 129.4 kN

Horizontal reaction at the base of column

H_{ed}= 33.0 kN

Axial force at rafters

N_{red}= 33.0 kN

Notional horizontal force applied at the top of the columns

H_{nhf}= 1.0 kN

Horizontal deflection at column top for notional force

 $\delta h, ed = 3.62$ mm

$$\alpha_{cr} = (1.0/129.4)(7650/3.62) = 16.32$$

(Eq.5.2)

Check axial compression of rafters. Axial compression is significant if

(§5.2.1, (4)B)

$$\lambda = \sqrt{(A \cdot f_y / N_{cr})} > 0.3 \sqrt{(A \cdot f_y / N_{ed})}, \quad N_{ed} > 0.09 N_{cr}$$

(§5.2.1 Eq.5.3)

Development length of the rafter pair from column to column $L = 15000 / \cos 0.00^\circ = 15000$ mm

$$N_{cr} = \pi^2 EI / L^2 = \pi^2 \times 210 \times 231.30 \times 10^6 / (15000)^2 = 2130.7 \text{ kN}$$

Maximum axial force in the rafters $N_{ed} = 33.0$ kN, L.C. 202: 1.35Gk+1.50Qs1

$$\lambda = \sqrt{(8446 \times 275 / 2130650)} = 1.04 \leq 0.3 \sqrt{(8446 \times 275 / 32989)} = 2.52$$

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

$$\alpha_{cr} = 16.32 > 10$$

(Eq.5.1)

First-order elastic analysis may be used

(§5.2.2.1)

$$\text{Amplification factor for design moments } \delta = 1 / (1 - 1/\alpha_{cr}) = 1 / (1 - 1/16.32) = 1.07$$

(Eq.5.4)

11.2. Imperfections for global analysis

(EN1993-1-1, §5.3.2)

$$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m \cdot \delta = (1/200) \times 0.723 \times 0.866 \times 1.065 = 3.336 \times 10^{-3} = 1/300$$

(Eq.5.5)

$$\phi_0 = 1/200, \quad \alpha_h = 2/\sqrt{h} = 2/\sqrt{7.650} = 0.723 \quad 2/3 \leq \alpha_h \leq 1.0, \quad \alpha_m = \sqrt{(0.5(1+1/2))} = 0.866$$

Sway imperfection may be disregarded where $H_{ed} > 0.15 V_{ed}$

(§5.3.2(4) Eq.5.7)

$$\text{Effect of initial sway imperfection } H_{eq} = 3.336 \times 10^{-3} \times V_{ed}$$

(§5.3.2 (5))

11.3. Sway imperfections for columns

(EN1993-1-1, §5.3.2)

Reactions at the supports, Horizontal Force H_{ed} [kN], Vertical Force V_{ed} [kN]

		Left support 1		Right support 2		Hed1+Hed2	Ved1+Ved2		φ·Ved
		Hed,1	Ved,1	Hed,2	Ved,2	Hed	Ved	Hed/Vhe	Heq kN
201	ULS-STR	19.6	82.2	-19.6	82.2	0.0	164.4	0.00	0.274
202	ULS-STR	33.0	129.4	-33.0	129.4	0.0	258.9	0.00	0.432
203	ULS-STR	-31.9	1.5	-23.8	29.9	-55.8	31.4	1.77	0.000
210	ULS-STR	-34.5	-11.1	-21.2	17.4	-55.8	6.3	8.81	0.000
211	ULS-STR	7.8	101.3	-41.2	118.3	-33.5	219.6	0.15	0.000
212	ULS-STR	-20.5	42.0	-35.3	70.4	-55.8	112.4	0.50	0.000
231	ULS-STR	-3.7	60.8	-29.8	77.8	-33.5	138.6	0.24	0.000
251	ULS-STR	6.3	94.1	-39.7	111.2	-33.5	205.3	0.16	0.000
252	ULS-STR	-22.0	34.8	-33.8	63.3	-55.8	98.1	0.57	0.000

11.4. Internal forces and bending moments with imperfection effect**11.5. Axial forces Ned [kN]**

		Left column 1	Left rafter 2	Right rafter 3	Right column 4
		Ned,1	Ned,2	Ned,3	Ned,4
L.C.					
201	ULS-STR	-75.7	-19.8	-19.8	-76.0
202	ULS-STR	-122.9	-33.2	-33.2	-123.3
203	ULS-STR	4.8	-6.2	-6.2	-23.6
210	ULS-STR	15.8	-3.6	-3.6	-12.7
211	ULS-STR	-94.9	-30.7	-30.7	-112.0
212	ULS-STR	-35.7	-17.7	-17.7	-64.1
231	ULS-STR	-54.4	-19.2	-19.2	-71.5
251	ULS-STR	-88.7	-29.2	-29.2	-105.8
252	ULS-STR	-29.4	-16.2	-16.2	-57.9

11.6. Shearing forces Ved [kN]

L.C.		Left column 1		Left rafter 2			Right rafter 3			Right column 4	
		VedA,1	VedB,1	VedA,2	VedC,2	VedB,2	VedA,3	VedC,3	VedB,3	VedA,4	VedB,4
201	ULS-STR	-19.5	-19.5	69.4	55.5	-0.1	-0.1	-55.7	-69.6	19.8	19.8
202	ULS-STR	-32.8	-32.8	116.5	93.2	-0.2	-0.2	-93.6	-117.0	33.2	33.2
203	ULS-STR	31.9	-6.2	-11.2	-11.8	-14.2	-14.2	-16.6	-17.2	6.2	23.8
210	ULS-STR	34.5	-3.6	-20.5	-19.2	-14.2	-14.2	-9.2	-8.0	3.6	21.2
211	ULS-STR	-7.8	-30.7	88.6	69.2	-8.5	-8.5	-86.2	-105.6	30.7	41.2
212	ULS-STR	20.5	-17.7	29.3	20.6	-14.2	-14.2	-49.0	-57.7	17.7	35.3
231	ULS-STR	3.7	-19.2	48.1	36.8	-8.5	-8.5	-53.8	-65.1	19.2	29.8
251	ULS-STR	-6.3	-29.2	83.3	64.9	-8.5	-8.5	-82.0	-100.3	29.2	39.7
252	ULS-STR	22.0	-16.2	24.0	16.4	-14.2	-14.2	-44.8	-52.4	16.2	33.8

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

11.7. Bending moments Med [kNm]

L.C.		Left column 1			Right column 4		
		MedA,1	MedM,1		MedA,4	MedM,4	
201	ULS-STR	0.0	-74.6	-149.2	-151.3	-75.6	0.0
202	ULS-STR	0.0	-125.4	-250.7	-254.0	-127.0	0.0
203	ULS-STR	0.0	102.1	98.2	-115.0	-57.5	0.0
210	ULS-STR	0.0	119.6	118.3	-95.0	-47.5	0.0
211	ULS-STR	0.0	-73.5	-147.1	-275.0	-137.5	0.0
212	ULS-STR	0.0	42.0	10.7	-202.6	-101.3	0.0
231	ULS-STR	0.0	2.2	-59.5	-187.5	-93.7	0.0
251	ULS-STR	0.0	-67.8	-135.6	-263.6	-131.8	0.0
252	ULS-STR	0.0	48.4	22.1	-191.1	-95.6	0.0

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

L.C.		Left rafter 2			Right rafter 3		
		MedA,2	MedC2	MedM,2	MedA,3	MedM,3	MedC3
201	ULS-STR	-149.2	-55.6	110.4	110.4	-20.5	-57.2
202	ULS-STR	-250.7	-93.4	185.4	185.4	-34.3	-96.1
203	ULS-STR	98.2	81.0	50.6	2.9	-56.0	-89.6
210	ULS-STR	118.3	88.5	53.2	-11.8	-53.4	-82.1
211	ULS-STR	-147.1	-28.8	155.9	153.1	-60.9	-131.1
212	ULS-STR	10.7	48.1	84.7	67.3	-67.6	-122.5
231	ULS-STR	-59.5	4.1	93.6	88.8	-49.3	-98.2
251	ULS-STR	-135.6	-24.5	147.7	144.7	-59.4	-126.8
252	ULS-STR	22.1	52.4	78.7	58.9	-66.1	-118.2

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

12. Serviceability Limit State (SLS)

(EN1993-1-1, §7)

12.1. Vertical deflection at the apex

(EN1993-1-1, §7.2.1)

Maximum vertical deflection, L.C. 302: $G_k + Q_{s1}$ $D_y = 45.3 \text{ mm} = 15000/331 = L/331$
 Vertical deflection due to imposed load only $D_y = 12.7 \text{ mm} = 15000/493 = L/1181$
 Vertical deflection due to snow only $D_y = 30.4 \text{ mm} = 15000/493 = L/493$
 Limit for vertical deflection $L/200$, Is verified

12.2. Horizontal deflection at the top of column

(EN1993-1-1, §7.2.2)

Maximum horizontal deflection, L.C. 312: $G_k + Q_{w1} + 0.50Q_{s1}$ $D_x = 37.3 \text{ mm} = 7650/205 = h/205$
 Horizontal deflection due to wind only $D_x = 37.3 \text{ mm} = 7650/205 = h/205$
 Limit for horizontal deflection $H/150$, Is verified

12.3. Dynamic effects

(EN1993-1-1, §7.2.3)

Eigenfrequencies and Eigenperiods of the structure

Mass of building, for loading: L.C. 601: $G_k + 0.20Q_{s1}$

1	f=	1.296 Hz	T=	0.772 sec
2	f=	4.415 Hz	T=	0.226 sec
3	f=	13.667 Hz	T=	0.073 sec
4	f=	23.609 Hz	T=	0.042 sec
5	f=	33.820 Hz	T=	0.030 sec
6	f=	57.224 Hz	T=	0.017 sec
7	f=	63.558 Hz	T=	0.016 sec
8	f=	65.592 Hz	T=	0.015 sec
9	f=	122.342 Hz	T=	0.008 sec
10	f=	137.031 Hz	T=	0.007 sec
11	f=	148.601 Hz	T=	0.007 sec

13. Column verification (Ultimate Limit State)

(EN1993-1-1, §6)

Profile : HE 400 A-S 275**Maximum design values. Verification for load case: L.C. 211: 1.35xGk+1.50Qs1+0.90Qw1**

Ned = 118.3 kN

Ved = 41.2 kN

Myed = 275.0 kNm, Mzed = 0.0 kNm

Myed = 256.4 kNm (Column top under the haunch)

Buckling length, In-plane buckling Lcr,y = 7650mm (System length) (EC3 §5.5.2.(7))

Buckling length, Out-of-plane buckling Lcr,z = 7050mm (Column height without haunch)

Buckling length, Torsional buckling Lcr,t = 7050mm

Buckling length, Lateral torsional buckling Lcr,lt = 7050mm

Maximum design values for seismic loading

Ned = 67.0 kN

Ved = 27.0 kN

Myed = 206.4 kNm, Mzed = 0.0 kNm

Myed = 190.2 kNm (Column top under the haunch)

13.1. Classification of steel cross-section, Column

(EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{y,ed}/W_{el,y} \pm M_{z,ed}/W_{el,z}$ $\sigma = [10^3]118.30/15900 \pm [10^6]275.00/2311.0 \times 10^3 \pm [10^6]0.00/571.0 \times 10^3$ $\sigma_1 = 126 \text{ N/mm}^2$, $\sigma_2 = -112 \text{ N/mm}^2$ (compression positive)**Web**

c = 390.0 - 2x19.0 - 2x27.0 = 298.0 mm, t = 11.0 mm, c/t = 298.0/11.0 = 27.09

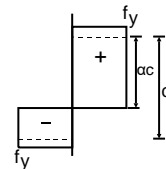
S 275, t = 11.0 ≤ 40 mm, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$

Position of neutral axis for combined Bending and compression

 $N_{ed}/(2t \cdot f_y / \gamma_{M0}) = 118300 / (2 \times 11.0 \times 275 / 1.00) = 19.6 \text{ mm}$ $\alpha = (298.0 / 2 + 19.6) / 298.0 = 0.566 > 0.5$

c/t = 27.09 ≤ 396 × 0.92 / (13 × 0.566 - 1) = 57.35

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

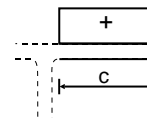
**Flange**

c = 300.0/2 - 11.0/2 - 27.0 = 117.5 mm, t = 19.0 mm, c/t = 117.5/19.0 = 6.18

S 275, t = 19.0 ≤ 40 mm, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$

c/t = 6.18 ≤ 9 × 0.92 = 8.28

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

**Overall classification of cross-section is Class 1, Bending and compression $N_{c,ed} + M_{y,ed}$** **13.2. Resistance of cross-section, Column (Ultimate Limit State)**

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

Nc.ed = 129.70 kNCompression Resistance $N_{pl,rd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 15900 \times 275 / 1.00 = 4372.50 \text{ kN}$

Ned = 129.70 kN < 4372.50 kN = Nc,rd = Npl,rd, Is verified

Ned/Nc,rd = 129.70/4372.50 = 0.030 < 1

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

Maximum design values. Verification for load case: L.C. 211: 1.35xGk+1.50Qs1+0.90Qw1

My.ed = 275.00 kNmBending Resistance $M_{pl,y,rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = [10^{-6}] \times 2562.0 \times 10^3 \times 275 / 1.00 = 704.55 \text{ kNm}$

My,ed = 275.00 kNm < 704.55 kNm = My,rd = Mpl,y,rd, Is verified

My,ed/My,rd = 275.00/704.55 = 0.390 < 1

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

Maximum design values. Verification for load case: L.C. 211: 1.35xGk+1.50Qs1+0.90Qw1

Vz,ed= 41.20 kN $A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 15900 - 2 \times 300.0 \times 19.0 + (11.0 + 2 \times 27.0) \times 19.0 = 5735 \text{ mm}^2$ (EC3 §6.2.6.3) $A_v = 5735 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (390.0 - 2 \times 19.0) \times 11.0 = 1.00 \times 371.0 \times 11.0 = 4081 \text{ mm}^2$ Plastic Shear Resistance $V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 5735 \times (275 / 1.73) / 1.00 = 910.55 \text{ kN}$ $V_{z,ed} = 41.20 \text{ kN} < 910.55 \text{ kN} = V_{z,rd} = V_{pl,z,rd}$, Is verified $V_{z,ed} / V_{z,rd} = 41.20 / 910.55 = 0.045 < 1$ $h_w / t_w = (390.0 - 2 \times 19.0) / 11.0 = 371.0 / 11.0 = 33.73 \leq 72 \times 0.92 / 1.00 = 72 \varepsilon / \eta = 66.24$ ($\eta = 1.00$)S 275, $t = 11.0 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\varepsilon = (235 / 275)^{0.5} = 0.92$

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

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

Maximum design values. Verification for load case: L.C. 211: 1.35xGk+1.50Qs1+0.90Qw1

N.ed= 118.30kN (Compression), Vz.ed= 41.20kN, My.ed= 275.00kNm $N_{pl,rd} = 4372.50 \text{ kN}$, $M_{pl,y,rd} = 704.55 \text{ kNm}$, $V_{pl,z,rd} = 910.55 \text{ kN}$ $N_{ed} = 118.30 \text{ kN} \leq 0.25 \times 4372.50 = 0.25 \times N_{pl,rd} = 1093.12 \text{ kN}$ $N_{ed} = 118.30 \text{ kN} \leq [10^{-3}] \times 0.5 \times 371.0 \times 11.0 \times 275 / 1.00 = 0.5 h_w \cdot t_w \cdot f_y / \gamma_{M0} = 561.14 \text{ kN}$ $n = N_{ed} / N_{pl,rd} = 118 / 4373 = 0.027$

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

 $V_{ed} = 41.20 \text{ kN} \leq 0.50 \times 910.55 = 0.50 \times V_{pl,rd} = 455.27 \text{ kN}$

Effect of shear force is neglected (EC3 §6.2.8.2)

 $M_{y,ed} = 275.00 \text{ kNm} < 704.55 \text{ kNm} = M_{pl,y,rd}$, Is verified $M_{y,ed} / M_{pl,y,rd} = 275.00 / 704.55 = 0.390 < 1$ **13.3. Flexural Buckling, Column (Ultimate Limit State)** (EN1993-1-1, §6.3.1)**Nc,ed=129.70 kN, Lcr,y=7.650 m, Lcr,z=7.050 m**

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

Buckling lengths: $L_{cr,y} = 1.000 \times 7650 = 7650 \text{ mm}$, $L_{cr,z} = 0.922 \times 7650 = 7050 \text{ mm}$

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

 $\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (7650 / 168.4) \times (1 / 86.39) = 0.526$ $\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (7050 / 73.4) \times (1 / 86.39) = 1.112$ $\lambda_1 = \pi \sqrt{(E / f_y)} = 93.9 \varepsilon = 86.39$, $\varepsilon = \sqrt{(235 / f_y)} = 0.92$ $h/b = 390 / 300 = 1.30 > 1.20$, $t_f = 19.0 \text{ mm} \leq 40 \text{ mm}$ y-y buckling curve: a, imperfection factor: $\alpha_y = 0.21$, $\chi_y = 0.916$ (T.6.2, T.6.1, Fig.6.4) $\Phi_y = 0.5 [1 + \alpha_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.5 \times [1 + 0.21 \times (0.526 - 0.2) + 0.526^2] = 0.673$ $\chi_y = 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}] = 1 / [0.673 + \sqrt{(0.673^2 - 0.526^2)}] = 0.916 \leq 1$ $\chi_y = 0.916$ z-z buckling curve: b, imperfection factor: $\alpha_z = 0.34$, $\chi_z = 0.528$ $\Phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = 0.5 \times [1 + 0.34 \times (1.112 - 0.2) + 1.112^2] = 1.273$ $\chi_z = 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}] = 1 / [1.273 + \sqrt{(1.273^2 - 1.112^2)}] = 0.528 \leq 1$ $\chi_z = 0.528$ Reduction factor $\chi = 1 / [\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}]$, $\chi \leq 1.0$, $\Phi = 0.5 [1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2]$, $\chi = 0.528$ (EC3 Eq.6.49) $N_{b,rd} = \chi \cdot A \cdot f_y / \gamma_{M1} = 0.528 \times [10^{-3}] \times 15900 \times 275 / 1.00 = 2308.68 \text{ kN}$ (EC3 Eq.6.47) $N_{c,ed} = 129.70 \text{ kN} < 2308.68 \text{ kN} = N_{b,rd}$, Is verified $N_{c,ed} / N_{b,rd} = 129.70 / 2308.68 = 0.056 < 1$ **13.4. Lateral torsional buckling, Column (ULS)** (EN1993-1-1, §6.3.2)**My,ed=256.37 kN, L=7.650m, Lcr,y=7.650m, Lcr,z=7.050m, Lcr,lt=7.050m**

Maximum design values. Verification for load case: L.C. 211: 1.35xGk+1.50Qs1+0.90Qw1

Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)

Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961

 $M_{cr} = C_1 \cdot [\pi^2 E I_z / (k L)^2] \{ \sqrt{[(k z / k_w)^2 (I_w / I_z) + (k L)^2 G I_t / (\pi^2 E I_z) + (C_2 \cdot z_g - C_3 \cdot z_j)^2]} - (C_2 \cdot z_g - C_3 \cdot z_j) \}$

Method of computation C1, C2, C3 : ECCS 119/Galea SNO30a-EN-EU Access Steel 2006

 $\psi = M_b / M_a = 0.0 / -256.4 = 0.00$, $C_1 = 1.770$, $C_2 = 0.000$, $C_3 = 1.000$, $G = E / (2(1 + \nu)) = 210000 / (2(1 + 0.30)) = 80769 \text{ N/mm}^2$ $k \cdot L = 7050 \text{ mm}$, $z_g = h / 2 = 390 / 2 = 195 \text{ mm}$, $z_j = 0 \text{ mm}$ (EN1993:2002 Eq.C.11) $k_y = 1.0$, $k_z = 1.0$, $k_w = 1.0$, $C_1 = 1.770$, $C_2 = 0.000$, $C_3 = 1.000$ $M_{cr} = [10^{-6}] 1.770 \times [\pi^2 \times 2.1 \times 10^5 \times 85.640 \times 10^6 / 7050^2]$ $\times \{ [(1.0 / 1.0)^2 \times (2942.1 \times 10^9 / 85.640 \times 10^6)]$ $+ 7050^2 \times 8.1 \times 10^4 \times 1.890 \times 10^6 / (\pi^2 \times 2.1 \times 10^5 \times 85.640 \times 10^6) \}^{0.5} = 1755.3 \text{ kNm}$

$\bar{\lambda}, l_t = \sqrt{(W_{pl,y} \cdot f_y / M_{cr})} = \sqrt{([10^{-6}] \times 2562.0 \times 10^3 \times 275 / 1755.3)} = 0.634$ (EC3 Eq.6.56)
 $h/b = 390/300 = 1.30 < 2.00$ buckling curve: b
 imperfection factor: $\alpha, l_t = 0.34$, $\beta = 0.75$, $\chi, l_t = 0.902$ (T.6.3, T.6.5, Fig.6.4)
 $\Phi, l_t = 0.5[1 + \alpha, l_t(\bar{\lambda}, l_t - \bar{\lambda}, l_{to}) + \beta \bar{\lambda}, l_t^2] = 0.5 \times [1 + 0.34 \times (0.634 - 0.40) + 0.75 \times 0.634^2] = 0.690$
 $\chi, l_t = 1 / [\Phi, l_t + \sqrt{(\Phi, l_t^2 - \beta \bar{\lambda}, l_t^2)}] = 1 / [0.690 + \sqrt{(0.690^2 - 0.75 \times 0.690^2)}] = 0.902$
 Reduction factor $\chi, l_t = 1 / [\Phi, l_t + \sqrt{(\Phi, l_t^2 - \beta \bar{\lambda}, l_t^2)}]$, $\chi, l_t \leq 1.0$, $1 / \bar{\lambda}, l_t^2$, $\chi, l_t = 0.902$ (Eq.6.57)
 $\chi, l_t, mod = \chi, l_t / f$, $\chi, l_t, mod \leq 1$, $\chi, l_t, mod \leq 1 / \bar{\lambda}, l_t^2 = 1 / 0.634^2 = 2.49$ (EC3 §6.3.2.3(2), Eq.6.58)
 $K_c = 1 / (1.33 - 0.33\psi) = 0.752$, $\psi = 0.00$ (EC3 Tab.6.6)
 $f = 1 - 0.5(1 - K_c)[1 - 2.0(\bar{\lambda}, l_t - 0.8)^2] = 1 - 0.5 \times (1 - 0.752)[1 - 2.0 \times (0.634 - 0.8)^2] = 0.883$, $f \leq 1.0$
 $\chi, l_t, mod = \chi, l_t / f = 0.902 / 0.883 = 1.022$, $\chi, l_t, mod \leq 1.0$, $\chi, l_t, mod \leq 2.49$, $\chi, l_t, mod = 1.000$
 $M_{b,rd} = \chi, l_t \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 1.000 \times [10^{-6}] \times 2562.0 \times 10^3 \times 275 / 1.00 = 704.55 \text{ kNm}$ (EC3 Eq.6.55)
 $M_{y,ed} = 256.37 \text{ kNm} < 704.55 \text{ kNm} = M_{b,rd}$, Is verified
 $M_{y,ed} / M_{b,rd} = 256.37 / 704.55 = 0.364 < 1$

13.5. Axial force and bending moment, Column (ULS)

(EN1993-1-1, §6.3.3)

Ned=118.30 kN, My,ed=256.37 kNm

$N_{ed} / (\chi_y \cdot N_{rk} / \gamma_{M1}) + k_{yy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk} / \gamma_{M1}) \leq 1$ (EC3 Eq.6.61)
 $N_{ed} / (\chi_z \cdot N_{rk} / \gamma_{M1}) + k_{zy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk} / \gamma_{M1}) \leq 1$ (EC3 Eq.6.62)
 $N_{rk} = A \cdot f_y = [10^{-3}] \times 15900 \times 275 = 4372.5 \text{ kN}$ (Tab.6.7)
 $M_{y,rk} = W_{pl,y} \cdot f_y = [10^{-6}] \times 2562.0 \times 10^3 \times 275 = 704.5 \text{ kNm}$
 $\chi_y \cdot N_{rk} / \gamma_{M1} = \chi_y \cdot A \cdot f_y / \gamma_{M1} = 0.916 \times [10^{-3}] \times 15900 \times 275 / 1.00 = 4005.2 \text{ kN}$
 $\chi_z \cdot N_{rk} / \gamma_{M1} = \chi_z \cdot A \cdot f_y / \gamma_{M1} = 0.528 \times [10^{-3}] \times 15900 \times 275 / 1.00 = 2308.7 \text{ kN}$
 $\chi_{LT} \cdot M_{y,rk} / \gamma_{M1} = \chi_{LT} \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 1.000 \times [10^{-6}] \times 2562.0 \times 10^3 \times 275 / 1.00 = 704.5 \text{ kNm}$

Interaction factors, Method of computation: Method 1 Annex A

(EC3 Annex A)

$k_{yy} = C_{my} \cdot C_{mLT}(\mu_y / (1 - N_{ed} / N_{cr,y})) (1 / C_{yy})$, $\mu_y = (1 - N_{ed} / N_{cr,y}) / (1 - \chi_y \cdot N_{ed} / N_{cr,y})$ (EC3 Tab.A.1)
 $k_{zy} = C_{my} \cdot C_{mLT}(\mu_z / (1 - N_{ed} / N_{cr,y})) (1 / C_{zy}) 0.60 \sqrt{(w_y / w_z)}$, $\mu_z = (1 - N_{ed} / N_{cr,z}) / (1 - \chi_z \cdot N_{ed} / N_{cr,z})$

$N_{cr,y} = \pi^2 E I_y / l_{cr,y}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 450.70 \times 10^6 / 7650^2 = 15962 \text{ kN}$
 $N_{cr,z} = \pi^2 E I_z / l_{cr,z}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 85.640 \times 10^6 / 7050^2 = 3571 \text{ kN}$
 $N_{cr,t} = (1 / i_p^2) \times (G \cdot I_t + \pi^2 E I_w / L_{cr,t}^2)$ (EC3 NCCI SN003b-EN-EU)
 $N_{cr,t} = [10^{-3}] \times (1 / 184^2) [80769 \times 1.890 \times 10^6 + \pi^2 \times 210000 \times 2942.1 \times 10^9 / 7050^2] = 8163 \text{ kN}$

$\mu_y = (1 - N_{ed} / N_{cr,y}) / (1 - \chi_y \cdot N_{ed} / N_{cr,y}) = (1 - 118.3 / 15962) / (1 - 0.916 \times 118.3 / 15962) = 0.999$
 $\mu_z = (1 - N_{ed} / N_{cr,z}) / (1 - \chi_z \cdot N_{ed} / N_{cr,z}) = (1 - 118.3 / 3571) / (1 - 0.528 \times 118.3 / 3571) = 0.984$
 $alt = 1 - I_t / I_y > 0 = 1 - 1.890 \times 10^6 / 450.70 \times 10^6 = 0.996$ (EC3 Annex A.1)

$w_y = W_{pl,y} / W_{el,y} \leq 1.50$, $w_y = 2.562 \times 10^6 / 2.311 \times 10^6 = 1.109 \leq 1.50$ (EC3 Annex A.1)
 $w_z = W_{pl,z} / W_{el,z} \leq 1.50$, $w_z = 0.873 \times 10^6 / 0.571 \times 10^6 = 1.529 > 1.50$, $w_z = 1.50$
 $n_{pl} = N_{ed} / (N_{rk} / \gamma_{M1}) = 118.30 / (4372.50 / 1.00) = 0.027$

$\bar{\lambda}_{max} = \max(0.526, 1.112) = 1.110$ (EC3 Annex A.1)
 $M_{cr,o} = (1.00 / 1.77) \times 1755.30 = 991.7$, $C1 = 1.00$
 $\bar{\lambda}_o = \sqrt{([10^{-6}] \times 2562.0 \times 10^3 \times 275 / 991.7)} = 0.840$
 $\bar{\lambda}_o, lim = 0.2 \sqrt{C1 [(1 - N_{ed} / N_{cr,z}) (1 - N_{ed} / N_{cr,t})]^{0.25}}$ (EC3 Annex A.1)
 $\bar{\lambda}_o, lim = 0.2 \sqrt{1.770 [(1 - 118.3 / 3571) (1 - 118.3 / 8163)]^{0.25}} = 0.263$
 $\varepsilon_y = (M_{y,ed} / N_{ed}) (A / W_{el}) = ([10^3] \times 256.37 / 118.30) \times (15900.0 / 2311.0 \times 10^3) = 14.91$

$C_{my,o} = 0.79 + 0.21\psi + 0.36(\psi - 0.33) \times (118.30 / 15962.0) = 1.002$, ($\psi = 1.00$) (EC3 Annex A, T.A.1)
 $\bar{\lambda}_o = 0.840 > \bar{\lambda}_o, lim = 0.263$
 $C_{my} = C_{my,o} + (1 - C_{my,o}) (\sqrt{\varepsilon_y \cdot alt}) / (1 + \sqrt{\varepsilon_y \cdot alt}) =$
 $= 1.002 + (1 - 1.002) \times (\sqrt{14.910 \times 0.996}) / (1 + \sqrt{14.910 \times 0.996}) = 1.000$
 $C_{mLT} = C_{my}^2 \cdot alt / \sqrt{[(1 - N_{ed} / N_{cr,z}) (1 - N_{ed} / N_{cr,t})]} \geq 1$
 $C_{mLT} = 1.000^2 \times 0.996 / \sqrt{[(1 - 118.3 / 3571.0) (1 - 118.3 / 8163.0)]} = 1.020$, $C_{mLT} = 1.020$

$$C_{yy}=1+(w_y-1)[(2-1.6C_{my}^2 \cdot \bar{\lambda}_{max}/w_y-1.6C_{my}^2 \cdot \bar{\lambda}_{max}^2/w_y)n_{pl}-blt] \geq W_{el,y}/W_{pl,y} \quad (\text{Annex A, T.A.1})$$

$$blt=0.5alt \cdot \bar{\lambda}_o^2 [M_{y,ed}/(\chi_{lt} \cdot M_{pl,y,rd})] (M_{z,ed}/M_{pl,z,rd}) =$$

$$=0.5 \times 0.996 \times 0.840^2 [256.4/(1.000 \times 635.5)] (0.0/157.0) = 0.000$$

$$C_{yy}=1+(1.109-1)[(2-1.6 \times 1.000^2 \times 1.110/1.109-1.6 \times 1.000^2 \times 1.110^2/1.109) \times 0.027-0.000]=0.996$$

$$C_{yy} \geq 2311.0 \times 10^3 / 2562.0 \times 10^3 = 0.902, \quad C_{yy}=0.996$$

$$C_{zy}=1+(w_y-1)[(2-14.0C_{my}^2 \cdot \bar{\lambda}_{max}^2/w_y^5)n_{pl}-dlt] \geq 0.6 \sqrt{(w_y/w_z)} (W_{el,y}/W_{pl,y}) \quad (\text{Annex A, T.A.1})$$

$$dlt=2alt \cdot [\bar{\lambda}_o/(0.1+\bar{\lambda}_z^4)] [M_{y,ed}/(C_{my} \cdot \chi_{lt} \cdot M_{pl,y,rd})] [M_{z,ed}/(C_{mz} \cdot M_{pl,z,rd})] =$$

$$=20.996 \times [0.840/(0.1+1.112^4)] [256.4/(1.000 \times 1.000 \times 635.5)] [0.0/(0.000 \times 157.0)] = 0.000$$

$$C_{zy}=1+(1.109-1)[(2-14.0 \times 1.000^2 \times 1.110^2/1.109^5) \times 0.027-0.000]=0.976$$

$$C_{zy} \geq 0.6 \sqrt{(1.109/1.500)} (2311.0 \times 10^3 / 2562.0 \times 10^3) = 0.465, \quad C_{zy}=0.976$$

$$C_{yy}=0.996, \quad C_{zy}=0.976 \quad (\text{Annex A, T.A.1})$$

$$k_{yy}=1.000 \times 1.020 \times 0.999 / (1-118.30/15962.0) \times (1/0.996) = 1.031$$

$$k_{zy}=1.000 \times 1.020 \times 0.984 / (1-118.30/15962.0) \times (1/0.976) \times 0.6 \times \sqrt{(1.109/1.500)} = 0.534$$

Maximum design values. Verification for load case: L.C. 211: 1.35xGk+1.50Qs1+0.90Qw1

$$N_{ed}/(\chi_y \cdot N_{rk}/\gamma_{M1}) + k_{yy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) = \quad (\text{EC3 Eq.6.61})$$

$$118.3/(0.916 \times 4372.5/1.00) + 1.031 \times 256.4/(1.000 \times 704.5/1.00) = 0.030 + 0.375 = 0.405$$

$$0.405 < 1.000, \quad \text{Is verified}$$

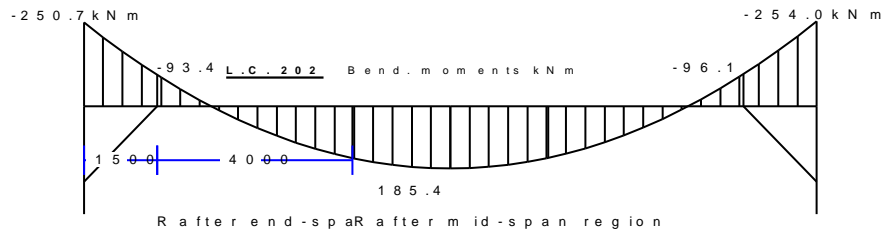
$$N_{ed}/(\chi_z \cdot N_{rk}/\gamma_{M1}) + k_{zy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) = \quad (\text{EC3 Eq.6.62})$$

$$118.3/(0.528 \times 4372.5/1.00) + 0.534 \times 256.4/(1.000 \times 704.5/1.00) = 0.051 + 0.194 = 0.246$$

$$0.246 < 1.000, \quad \text{Is verified}$$

14. Rafter verification (Ultimate Limit State)

(EN1993-1-1, §6)

**Profile : IPE 400-S 275****Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1**

Ned = 33.2 kN
 Ved = 93.6 kN
 Myed = 185.4 kNm, Mzed = 0.0 kNm
 Myed = 185.4 kNm (at mid-span)
 Myed = -96.1 kNm (at haunch-start)
 Myed = -231.2 kNm (at haunch end)
 Myed = -275.0 kNm (at column axis point)

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

Ned = 3.6 kN
 Ved = 14.2 kN
 Myed = -53.4 kNm

Maximum design values for seismic loading

Ned = 13.2 kN
 Ved = 40.2 kN
 Myed = 80.9 kNm, Mzed = 0.0 kNm
 Myed = 80.9 kNm (at haunch-start)
 Myed = 98.4 kNm (at haunch end)

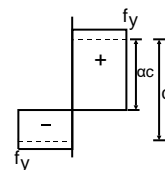
14.1. Classification of steel cross-section, Rafter

(EN1993-1-1, §5.5)

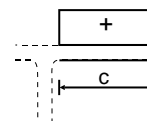
Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{yed}/W_{el,y} \pm M_{zed}/W_{el,z}$
 $\sigma = [10^3]33.20/8446 \pm [10^6]185.40/1156.0 \times 10^3 \pm [10^6]0.00/146.4 \times 10^3$
 $\sigma_1 = 164 \text{ N/mm}^2$, $\sigma_2 = -156 \text{ N/mm}^2$ (compression positive)

Web

$c = 400.0 - 2 \times 13.5 - 2 \times 21.0 = 331.0 \text{ mm}$, $t = 8.6 \text{ mm}$, $c/t = 331.0/8.6 = 38.49$
 S 275, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 Position of neutral axis for combined Bending and compression
 $N_{ed}/(2t_w \cdot f_y / \gamma_{M0}) = 33200 / (2 \times 8.6 \times 275 / 1.00) = 7.0 \text{ mm}$
 $\alpha = (331.0/2 + 7.0) / 331.0 = 0.521 > 0.5$
 $c/t = 38.49 \leq 396 \times 0.92 / (13 \times 0.521 - 1) = 63.08$
 The web is class 1 (EN1993-1-1, Tab.5.2)

**Flange**

$c = 180.0/2 - 8.6/2 - 21.0 = 64.7 \text{ mm}$, $t = 13.5 \text{ mm}$, $c/t = 64.7/13.5 = 4.79$
 S 275, $t = 13.5 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 $c/t = 4.79 \leq 9 \epsilon = 9 \times 0.92 = 8.28$
 The flange is class 1 (EN1993-1-1, Tab.5.2)

**Overall classification of cross-section is Class 1, Bending and compression $N_{c,ed} + M_{y,ed}$**

14.2. Resistance of cross-section, Rafter (Ultimate Limit State)

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

Nc.ed= 33.20 kNCompression Resistance $N_{plrd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 8446 \times 275 / 1.00 = 2322.65 \text{ kN}$

Ned= 33.20 kN < 2322.65 kN =Nc,rd=Nplrd, Is verified

Ned/Nc,rd= 33.20/2322.65= 0.014<1

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

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

My.ed=185.40 kNmBending Resistance $M_{ply,rd} = W_{ply} \cdot f_y / \gamma_{M0} = [10^{-6}] \times 1307.0 \times 10^3 \times 275 / 1.00 = 359.42 \text{ kNm}$

My,ed= 185.40 kNm < 359.42 kNm =My,rd=Mply,rd, Is verified

My,ed/My,rd= 185.40/359.42= 0.516<1

Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

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

Vz.ed= 93.60 kN $A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 8446 - 2 \times 180.0 \times 13.5 + (8.6 + 2 \times 21.0) \times 13.5 = 4269 \text{ mm}^2$

(EC3 §6.2.6.3)

 $A_v = 4269 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (400.0 - 2 \times 13.5) \times 8.6 = 1.00 \times 386.5 \times 8.6 = 3324 \text{ mm}^2$ Plastic Shear Resistance $V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 4269 \times (275 / 1.73) / 1.00 = 677.81 \text{ kN}$

Vz,ed= 93.60 kN < 677.81 kN =Vz,rd=Vpl,z,rd, Is verified

Vz,ed/Vz,rd= 93.60/677.81= 0.138<1

 $h_w / t_w = (400.0 - 2 \times 13.5) / 8.6 = 386.5 / 8.6 = 44.94 \leq 72 \times 0.92 / 1.00 = 72 \varepsilon / \eta = 66.24$ ($\eta = 1.00$)S 275, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\varepsilon = (235 / 275)^{0.5} = 0.92$

Shear buckling resistance is not necessary to be verified

(EC3 §6.2.6.6)

Ultimate Limit State, Verification for axial force, shear and bending

(EN1993-1-1, §6.2.9)

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

N.ed= 33.20kN (Compression), Vz.ed= 93.60kN, My.ed= 185.40kNm $N_{plrd} = 2322.65 \text{ kN}$, $M_{pl,y,rd} = 359.42 \text{ kNm}$, $V_{pl,z,rd} = 677.81 \text{ kN}$

Ned=33.20kN <= 0.25x2322.65=0.25xNplrd=580.66kN

Ned=33.20kN <= $[10^{-3}] \times 0.5 \times 386.5 \times 8.6 \times 275 / 1.00 = 0.5 h_w \cdot t_w \cdot f_y / \gamma_{M0} = 457.04 \text{ kN}$

n=Ned/Nplrd=33/2323= 0.014

Effect of axial force is neglected

(EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

Ved=93.60kN <= 0.50x677.81=0.50xVpl,rd=338.90kN

Effect of shear force is neglected

(EC3 §6.2.8.2)

My,ed= 185.40 kNm < 359.42 kNm =Mply,rd, Is verified

My,ed/Mply,rd= 185.40/359.42= 0.516<1

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

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

Ned = 33.2 kN

Ved = 117.0 kN

Myed = 185.4 kNm, Mzed =0.0 kNm

Rafter length $L_r = 15000 \text{ mm}$

Buckling length, In-plane buckling

 $\alpha_{cr} = 16.32$, Ned=33.2kN, $L_{cr,y} = \pi \sqrt{EI / \alpha_{cr} \cdot Ned} \leq L_r = 15000 \text{ mm}$ $L_{cr,y} = \pi \sqrt{[210000 \times 231.30 \times 10^6] / (16.32 \times 33.2 \times 10^3)} = 29743 \text{ mm}$, $L_{cr,y} = 15000 \text{ mm}$ Buckling length, In-plane buckling $L_{cr,y} = 15000 \text{ mm}$ (System length)Buckling length, Out-of-plane buckling $L_{cr,z} = 1400 \text{ mm}$ (Purlin spacing)**14.4. Flexural Buckling, Rafter mid-span region (Ultimate Limit State)**

(EN1993-1-1, §6.3.1)

Nc.ed=33.20 kN, Lcr,y=15.000 m, Lcr,z=1.400 m

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

Buckling lengths: $L_{cr,y} = 1.000 \times 15000 = 15000 \text{ mm}$, $L_{cr,z} = 0.093 \times 15000 = 1400 \text{ mm}$

Non-dimensional slenderness (Cross-section Class: 1)

(EC3 §6.3.1.3)

 $\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (15000 / 165.5) \times (1 / 86.39) = 1.049$ $\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (1400 / 39.5) \times (1 / 86.39) = 0.410$ $\lambda_1 = \pi \sqrt{(E / f_y)} = 93.9 \varepsilon = 86.39$, $\varepsilon = \sqrt{(235 / f_y)} = 0.92$

$h/b=400/180=2.22 \geq 1.20$, $t_f=13.5\text{mm} \leq 40\text{ mm}$
 $y-y$ buckling curve: a , imperfection factor: $\alpha_y=0.21$, $\chi_y=0.631$ (T.6.2, T.6.1, Fig.6.4)
 $\Phi_y=0.5[1+\alpha_y(\bar{\lambda}_y-0.2)+\bar{\lambda}_y^2]=0.5x[1+0.21x(1.049-0.2)+1.049^2]=1.139$
 $\chi_y=1/[\Phi_y+\sqrt{(\Phi_y^2-\bar{\lambda}_y^2)}]=1/[1.139+\sqrt{(1.139^2-1.049^2)}]=0.631 \leq 1$ $\chi_y=0.631$
 $z-z$ buckling curve: b , imperfection factor: $\alpha_z=0.34$, $\chi_z=0.922$
 $\Phi_z=0.5[1+\alpha_z(\bar{\lambda}_z-0.2)+\bar{\lambda}_z^2]=0.5x[1+0.34x(0.410-0.2)+0.410^2]=0.620$
 $\chi_z=1/[\Phi_z+\sqrt{(\Phi_z^2-\bar{\lambda}_z^2)}]=1/[0.620+\sqrt{(0.620^2-0.410^2)}]=0.922 \leq 1$ $\chi_z=0.922$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi \leq 1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.631$ (EC3 Eq.6.49)
 $N_{b,rd}=\chi \cdot A \cdot f_y / \gamma_{M1} = 0.631x[10^{-3}]x8446x275/1.00=1465.59\text{kN}$ (EC3 Eq.6.47)
 $N_{c,ed} = 33.20\text{ kN} < 1465.59\text{ kN} = N_{b,rd}$, Is verified
 $N_{c,ed}/N_{b,rd} = 33.20/1465.59 = 0.023 < 1$

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

$M_{y,ed}=185.42\text{ kN}$, $L=15.000\text{m}$, $L_{cr,y}=15.000\text{m}$, $L_{cr,z}=1.400\text{m}$, $L_{cr,lt}=1.400\text{m}$
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961
 $M_{cr}=C1 \cdot [\pi^2 EI_z / (kL)^2] \{ \sqrt{[(kz/kw)^2 (I_w/I_z) + (kL)^2 GIt / (\pi^2 EI_z) + (C2 \cdot z_g - C3 \cdot z_j)^2]} - (C2 \cdot z_g - C3 \cdot z_j) \}$
Method of computation $C1, C2, C3$: ECCS 119/Galea SN030a-EN-EU Access Steel 2006
 $\mu=Mo/M=qL^2/8M=3.8/185.4=0.02$, $\psi=Mb/Ma=60.0/185.4=0.32$, $C1=1.307$, $C2=0.050$
 $G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4\text{ N/mm}^2$
 $k \cdot L=1400\text{mm}$, $z_g=h/2=400/2=200\text{mm}$, $z_j=0\text{mm}$ (EN1993:2002 Eq.C.11)
 $k_y=1.0$, $k_z=1.0$, $k_w=1.0$, $C1=1.307$, $C2=0.050$, $C3=0.000$
 $M_{cr}=[10^{-6}]1.307x[\pi^2x2.1x10^5x13.180x10^6/1400^2]$
 $x\{ [(1.0/1.0)^2x(490.05x10^9/13.180x10^6)$
 $+1400^2x8.1x10^4x0.511x10^6/(\pi^2x2.1x10^5x13.180x10^6)$
 $+(0.050x200)^2]^{0.5}-(0.050x200) \} = 3471.5\text{ kNm}$

 $\bar{\lambda}_{lt}=\sqrt{(W_{pl,y} \cdot f_y / M_{cr})}=\sqrt{[10^{-6}]x1307.0x10^3x275/3471.5}=0.322$ (EC3 Eq.6.56)
 $\bar{\lambda}_{lt} \leq 0.40$, $\chi_{lt}=1.00$ (EC3 §6.3.2.2.4)

 $\chi_{lt,mod}=\chi_{lt}/f$, $\chi_{lt,mod} \leq 1$, $\chi_{lt,mod} \leq 1/\bar{\lambda}_{lt}^2=1/0.322^2=9.66$ (EC3 §6.3.2.3(2), Eq.6.58)
 $K_c=1.00$ (EC3 Tab.6.6)
 $f=1-0.5(1-K_c)[1-2.0(\bar{\lambda}_{lt}-0.8)^2]=1-0.5x(1-1.000)[1-2.0x(0.322-0.8)^2]=1.000$, $f \leq 1.0$
 $\chi_{lt,mod}=\chi_{lt}/f=1.000/1.000=1.000$, $\chi_{lt,mod} \leq 1.0$, $\chi_{lt,mod} \leq 9.66$, $\chi_{lt,mod}=1.000$

 $M_{b,rd}=\chi_{lt} \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 1.000x[10^{-6}]x1307.0x10^3x275/1.00=359.42\text{kNm}$ (EC3 Eq.6.55)
 $M_{y,ed} = 185.42\text{ kNm} < 359.42\text{ kNm} = M_{b,rd}$, Is verified
 $M_{y,ed}/M_{b,rd} = 185.42/359.42 = 0.516 < 1$

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

$N_{ed}=33.20\text{ kN}$, $M_{y,ed}=185.42\text{ kNm}$
 $N_{ed}/(\chi_y \cdot N_{rk}/\gamma_{M1}) + k_{yy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) \leq 1$ (EC3 Eq.6.61)
 $N_{ed}/(\chi_z \cdot N_{rk}/\gamma_{M1}) + k_{zy} \cdot M_{y,ed}/(\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}) \leq 1$ (EC3 Eq.6.62)
 $N_{rk}=A \cdot f_y=[10^{-3}]x8446x275=2322.6\text{ kN}$ (Tab.6.7)
 $M_{y,rk}=W_{pl,y} \cdot f_y=[10^{-6}]x1307.0x10^3x275=359.4\text{ kNm}$
 $\chi_y \cdot N_{rk}/\gamma_{M1}=\chi_y \cdot A \cdot f_y/\gamma_{M1}=0.631x[10^{-3}]x8446x275/1.00=1465.6\text{kN}$
 $\chi_z \cdot N_{rk}/\gamma_{M1}=\chi_z \cdot A \cdot f_y/\gamma_{M1}=0.922x[10^{-3}]x8446x275/1.00=2141.5\text{kN}$
 $\chi_{LT} \cdot M_{y,rk}/\gamma_{M1}=\chi_{LT} \cdot W_{pl,y} \cdot f_y/\gamma_{M1}=1.000x[10^{-6}]x1307.0x10^3x275/1.00=359.4\text{kNm}$

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

$k_{yy}=C_{my} \cdot C_{mLT}(\mu_y/(1-N_{ed}/N_{cr,y}))(1/C_{yy})$, $\mu_y=(1-N_{ed}/N_{cr,y})/(1-\chi_y \cdot N_{ed}/N_{cr,y})$ (EC3 Tab.A.1)
 $k_{zy}=C_{my} \cdot C_{mLT}(\mu_z/(1-N_{ed}/N_{cr,y}))(1/C_{zy})0.60\sqrt{(w_y/w_z)}$, $\mu_z=(1-N_{ed}/N_{cr,z})/(1-\chi_z \cdot N_{ed}/N_{cr,z})$

 $N_{cr,y}=\pi^2 EI_y / l_{cr,y}^2=3.14^2x[10^{-3}]x210000x231.30x10^6/15000^2=2131\text{ kN}$
 $N_{cr,z}=\pi^2 EI_z / l_{cr,z}^2=3.14^2x[10^{-3}]x210000x13.180x10^6/1400^2=13937\text{ kN}$
 $N_{cr,t}=(1/i_p^2)x(G \cdot I_t + \pi^2 EI_w / L_{cr,t}^2)$ (EC3 NCCI SN003b-EN-EU)
 $N_{cr,t}=[10^{-3}]x(1/170^2)[80769x0.511x10^6+\pi^2x210000x490.05x10^9/1400^2]=19328\text{ kN}$

$\mu_y = (1 - N_{ed}/N_{cr,y}) / (1 - \chi_y \cdot N_{ed}/N_{cr,y}) = (1 - 33.2 / 2131) / (1 - 0.631 \times 33.2 / 2131) = 0.994$
 $\mu_z = (1 - N_{ed}/N_{cr,z}) / (1 - \chi_z \cdot N_{ed}/N_{cr,z}) = (1 - 33.2 / 13937) / (1 - 0.922 \times 33.2 / 13937) = 1.000$
 $alt = 1 - I_t / I_y > 0 = 1 - 0.511 \times 10^6 / 231.30 \times 10^6 = 0.998$ (EC3 Annex A.1)

$w_y = W_{pl,y} / W_{el,y} \leq 1.50$, $w_y = 1.307 \times 10^6 / 1.156 \times 10^6 = 1.131 \leq 1.50$ (EC3 Annex A.1)
 $w_z = W_{pl,z} / W_{el,z} \leq 1.50$, $w_z = 0.229 \times 10^6 / 0.146 \times 10^6 = 1.564 > 1.50$, $w_z = 1.50$
 $n_{pl} = N_{ed} / (N_{rk} / \gamma_{M1}) = 33.20 / (2322.60 / 1.00) = 0.014$

$\bar{\lambda}_{max} = \max(1.049, 0.410) = 1.050$ (EC3 Annex A.1)
 $M_{cr,o} = (1.00 / 1.31) \times 3471.50 = 2655.6$, $C1 = 1.00$
 $\bar{\lambda}_o = \sqrt{([10^{-6}] \times 1307.0 \times 10^3 \times 275 / 2655.6)} = 0.370$
 $\bar{\lambda}_o, \lim = 0.2 \sqrt{C1 [(1 - N_{ed}/N_{cr,z})(1 - N_{ed}/N_{cr,t})]^{0.25}}$ (EC3 Annex A.1)
 $\bar{\lambda}_o, \lim = 0.2 \sqrt{1.307 [(1 - 33.2 / 13937)(1 - 33.2 / 19328)]^{0.25}} = 0.228$
 $\varepsilon_y = (M_{y,ed} / N_{ed}) (A / W_{el}) = ([10^3] \times 185.42 / 33.20) \times (8446.0 / 1156.0 \times 10^3) = 40.80$

$C_{my,o} = 0.79 + 0.21\psi + 0.36(\psi - 0.33) \times (33.20 / 2131.0) = 1.004$, ($\psi = 1.00$) (EC3 Annex A, T.A.1)
 $\bar{\lambda}_o = 0.370 > \bar{\lambda}_o, \lim = 0.228$
 $C_{my} = C_{my,o} + (1 - C_{my,o}) (\sqrt{\varepsilon_y \cdot alt}) / (1 + \sqrt{\varepsilon_y \cdot alt}) =$
 $= 1.004 + (1 - 1.004) \times (\sqrt{40.798 \times 0.998}) / (1 + \sqrt{40.798 \times 0.998}) = 1.001$
 $C_{m1t} = C_{my}^2 \cdot alt / \sqrt{[(1 - N_{ed}/N_{cr,z})(1 - N_{ed}/N_{cr,t})]} \geq 1$
 $C_{m1t} = 1.001^2 \times 0.998 / \sqrt{[(1 - 33.2 / 13937.0)(1 - 33.2 / 19328.0)]} = 1.002$, $C_{m1t} = 1.002$

$C_{yy} = 1 + (w_y - 1) [(2 - 1.6 C_{my}^2 \cdot \bar{\lambda}_{max} / w_y - 1.6 C_{my}^2 \cdot \bar{\lambda}_{max}^2 / w_y) n_{pl} - blt] \geq W_{el,y} / W_{pl,y}$ (Annex A, T.A.1)
 $blt = 0.5 alt \cdot \bar{\lambda}_o^2 [M_{y,ed} / (\chi_{lt} \cdot M_{pl,y,rd})] [M_{z,ed} / M_{pl,z,rd}] =$
 $= 0.5 \times 0.998 \times 0.370^2 [0.0 / (1.000 \times 317.9)] [0.0 / 40.3] = 0.000$
 $C_{yy} = 1 + (1.131 - 1) [(2 - 1.6 \times 1.001^2 \times 1.050 / 1.131 - 1.6 \times 1.001^2 \times 1.050^2 / 1.131) \times 0.014 - 0.000] = 0.998$
 $C_{yy} > 1156.0 \times 10^3 / 1307.0 \times 10^3 = 0.884$, $C_{yy} = 0.998$

$C_{zy} = 1 + (w_y - 1) [(2 - 14.0 C_{my}^2 \cdot \bar{\lambda}_{max}^2 / w_y^5) n_{pl} - dlt] \geq 0.6 \sqrt{(w_y / w_z)} (W_{el,y} / W_{pl,y})$ (Annex A, T.A.1)
 $dlt = 2 alt \cdot [\bar{\lambda}_o / (0.1 + \bar{\lambda}_z^4)] [M_{y,ed} / (C_{my} \cdot \chi_{lt} \cdot M_{pl,y,rd})] [M_{z,ed} / (C_{mz} \cdot M_{pl,z,rd})] =$
 $= 20.998 \times [0.370 / (0.1 + 0.410^4)] [0.0 / (1.001 \times 1.000 \times 317.9)] [0.0 / (0.000 \times 40.3)] = 0.000$
 $C_{zy} = 1 + (1.131 - 1) [(2 - 14.0 \times 1.001^2 \times 1.050^2 / 1.131^5) \times 0.014 - 0.000] = 0.988$
 $C_{zy} > 0.6 \sqrt{(1.131 / 1.500)} (1156.0 \times 10^3 / 1307.0 \times 10^3) = 0.461$, $C_{zy} = 0.988$

$C_{yy} = 0.998$, $C_{zy} = 0.988$ (Annex A, T.A.1)
 $k_{yy} = 1.001 \times 1.002 \times 0.994 / (1 - 33.20 / 2131.0) \times (1 / 0.998) = 1.015$
 $k_{zy} = 1.001 \times 1.002 \times 1.000 / (1 - 33.20 / 2131.0) \times (1 / 0.988) \times 0.6 \times \sqrt{(1.131 / 1.500)} = 0.537$

Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
 $N_{ed} / (\chi_y \cdot N_{rk} / \gamma_{M1}) + k_{yy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk} / \gamma_{M1}) =$ (EC3 Eq.6.61)
 $33.2 / (0.631 \times 2322.6 / 1.00) + 1.015 \times 185.4 / (1.000 \times 359.4 / 1.00) = 0.023 + 0.524 = 0.546$
 $0.546 < 1.000$, Is verified

$N_{ed} / (\chi_z \cdot N_{rk} / \gamma_{M1}) + k_{zy} \cdot M_{y,ed} / (\chi_{LT} \cdot M_{y,rk} / \gamma_{M1}) =$ (EC3 Eq.6.62)
 $33.2 / (0.922 \times 2322.6 / 1.00) + 0.537 \times 185.4 / (1.000 \times 359.4 / 1.00) = 0.016 + 0.277 = 0.293$
 $0.293 < 1.000$, Is verified

14.7. Buckling resistance, Rafter end-span region (Ultimate Limit State)
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
 $N_{ed} = 33.2$ kN
 $V_{ed} = 117.0$ kN
 $M_{y,ed} = 153.8$ kNm, $M_{z,ed} = 0.0$ kNm
 Rafter length $L_r = 15000$ mm
 Buckling length, In-plane buckling $L_{cr,y} = 15000$ mm (System length)
 Buckling length, Out-of-plane buckling $L_{cr,z} = 4000$ mm (Torsional restrains of rafters)

14.8. Flexural Buckling, Rafter end-span region (Ultimate Limit State) (EN1993-1-1, §6.3.1)
 $N_{c,ed} = 33.20$ kN, $L_{cr,y} = 15.000$ m, $L_{cr,z} = 4.000$ m
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
 Buckling lengths: $L_{cr,y} = 1.000 \times 15000 = 15000$ mm, $L_{cr,z} = 0.267 \times 15000 = 4000$ mm
 Non-dimensional slenderness (Cross-section Class: 1) (EC3 §6.3.1.3)
 $\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (15000 / 165.5) \times (1 / 86.39) = 1.049$
 $\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (4000 / 39.5) \times (1 / 86.39) = 1.172$
 $\lambda_1 = \pi \sqrt{(E / f_y)} = 93.9$, $\varepsilon = 86.39$, $\varepsilon = \sqrt{(235 / f_y)} = 0.92$

$h/b=400/180=2.22>1.20$, $t_f=13.5\text{mm}\leq 40\text{ mm}$
 $y-y$ buckling curve: a , imperfection factor: $\alpha_y=0.21$, $\chi_y=0.631$ (T.6.2, T.6.1, Fig.6.4)
 $\Phi_y=0.5[1+\alpha_y(\bar{\lambda}_y-0.2)+\bar{\lambda}_y^2]=0.5x[1+0.21x(1.049-0.2)+1.049^2]=1.139$
 $\chi_y=1/[\Phi_y+\sqrt{(\Phi_y^2-\bar{\lambda}_y^2)}]=1/[1.139+\sqrt{(1.139^2-1.049^2)}]=0.631 \leq 1$ $\chi_y=0.631$
 $z-z$ buckling curve: b , imperfection factor: $\alpha_z=0.34$, $\chi_z=0.494$
 $\Phi_z=0.5[1+\alpha_z(\bar{\lambda}_z-0.2)+\bar{\lambda}_z^2]=0.5x[1+0.34x(1.172-0.2)+1.172^2]=1.352$
 $\chi_z=1/[\Phi_z+\sqrt{(\Phi_z^2-\bar{\lambda}_z^2)}]=1/[1.352+\sqrt{(1.352^2-1.172^2)}]=0.494 \leq 1$ $\chi_z=0.494$

Reduction factor $\chi=1/[\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)}]$, $\chi\leq 1.0$, $\Phi=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$, $\chi=0.494$ (EC3 Eq.6.49)
 $N_{b,rd}=\chi\cdot A\cdot f_y/\gamma_{M1}=0.494x[10^{-3}]x8446x275/1.00=1147.39\text{ kN}$ (EC3 Eq.6.47)
 $N_{c,ed}=33.20\text{ kN} < 1147.39\text{ kN}=N_{b,rd}$, Is verified
 $N_{c,ed}/N_{b,rd}=33.20/1147.39=0.029<1$

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

$M_{y,ed}=153.84\text{ kN}$, $L=15.000\text{m}$, $L_{cr,y}=15.000\text{m}$, $L_{cr,z}=4.000\text{m}$, $L_{cr,lt}=4.000\text{m}$
Maximum design values. Verification for load case: L.C. 202: 1.35Gk+1.50Qs1
Elastic critical moment for lateral-torsional buckling (EC3 §6.3.2.2.2, EN1993:2002 AnnexC)
Timoshenko, S.P., Gere, J.M., Theory of elastic stability, McGraw-Hill, 1961
 $M_{cr}=C_1\cdot[\pi^2EI_z/(kL)^2]\{[\sqrt{(k_z/k_w)^2(I_w/I_z)+(kL)^2GI_t/(\pi^2EI_z)}+(C_2\cdot z_g-C_3\cdot z_j)^2]-(C_2\cdot z_g-C_3\cdot z_j)\}$
Method of computation C_1, C_2, C_3 : ECCS 119/Galea SN030a-EN-EU Access Steel 2006
 $\mu=Mo/M=qL^2/8M=31.1/153.8=0.20$, $\psi=M_b/M_a=-96.1/153.8=-0.62$, $C_1=1.920$, $C_2=0.127$
 $G=E/(2(1+\nu))=210000/(2(1+0.30))=80769=8.1x10^4\text{ N/mm}^2$
 $k\cdot L=4000\text{mm}$, $z_g=h/2=400/2=200\text{mm}$, $z_j=0\text{mm}$ (EN1993:2002 Eq.C.11)
 $k_y=1.0$, $k_z=1.0$, $k_w=1.0$, $C_1=1.920$, $C_2=0.127$, $C_3=0.000$
 $M_{cr}=[10^{-6}]1.920x[\pi^2x2.1x10^5x13.180x10^6/4000^2]$
 $x\{[(1.0/1.0)^2x(490.05x10^9/13.180x10^6)$
 $+4000^2x8.1x10^4x0.511x10^6/(\pi^2x2.1x10^5x13.180x10^6)$
 $+(0.127x200)^2]^{0.5}-(0.127x200)\}=732.9\text{ kNm}$

 $\bar{\lambda}_{lt}=\sqrt{(W_{pl,y}\cdot f_y/M_{cr})}=\sqrt{[10^{-6}]x1307.0x10^3x275/732.9}=0.700$ (EC3 Eq.6.56)
 $h/b=400/180=2.22>2.00$ buckling curve: c
imperfection factor: $\alpha_{lt}=0.49$, $\beta=0.75$, $\chi_{lt}=0.826$ (T.6.3, T.6.5, Fig.6.4)
 $\Phi_{lt}=0.5[1+\alpha_{lt}(\bar{\lambda}_{lt}-\bar{\lambda}_{lto})+\beta\bar{\lambda}_{lt}^2]=0.5x[1+0.49x(0.700-0.40)+0.75x0.700^2]=0.757$
 $\chi_{lt}=1/[\Phi_{lt}+\sqrt{(\Phi_{lt}^2-\beta\bar{\lambda}_{lt}^2)}]=1/[0.757+\sqrt{(0.757^2-0.75x0.757^2)}]=0.826$
Reduction factor $\chi_{lt}=1/[\Phi_{lt}+\sqrt{(\Phi_{lt}^2-\beta\bar{\lambda}_{lt}^2)}]$, $\chi_{lt}\leq 1.0$, $1/\bar{\lambda}_{lt}^2$, $\chi_{lt}=0.826$ (Eq.6.57)

 $\chi_{lt,mod}=\chi_{lt}/f$, $\chi_{lt,mod}\leq 1$, $\chi_{lt,mod}\leq 1/\bar{\lambda}_{lt}^2=1/0.700^2=2.04$ (EC3 §6.3.2.3(2), Eq.6.58)
 $K_c=1/(1.33-0.33\psi)=0.752$, $\psi=0.00$ (EC3 Tab.6.6)
 $f=1-0.5(1-K_c)[1-2.0(\bar{\lambda}_{lt}-0.8)^2]=1-0.5x(1-0.752)[1-2.0x(0.700-0.8)^2]=0.878$, $f\leq 1.0$
 $\chi_{lt,mod}=\chi_{lt}/f=0.826/0.878=0.940$, $\chi_{lt,mod}\leq 1.0$, $\chi_{lt,mod}\leq 2.04$, $\chi_{lt,mod}=0.940$

 $M_{b,rd}=\chi_{lt}\cdot W_{pl,y}\cdot f_y/\gamma_{M1}=0.940x[10^{-6}]x1307.0x10^3x275/1.00=337.86\text{ kNm}$ (EC3 Eq.6.55)
 $M_{y,ed}=153.84\text{ kNm} < 337.86\text{ kNm}=M_{b,rd}$, Is verified
 $M_{y,ed}/M_{b,rd}=153.84/337.86=0.455<1$

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

$N_{ed}=33.20\text{ kN}$, $M_{y,ed}=153.84\text{ kNm}$

 $N_{ed}/(\chi_y\cdot N_{rk}/\gamma_{M1})+k_{yy}\cdot M_{y,ed}/(\chi_{LT}\cdot M_{y,rk}/\gamma_{M1})\leq 1$ (EC3 Eq.6.61)
 $N_{ed}/(\chi_z\cdot N_{rk}/\gamma_{M1})+k_{zy}\cdot M_{y,ed}/(\chi_{LT}\cdot M_{y,rk}/\gamma_{M1})\leq 1$ (EC3 Eq.6.62)
 $N_{rk}=A\cdot f_y=[10^{-3}]x8446x275=2322.6\text{ kN}$ (Tab.6.7)
 $M_{y,rk}=W_{pl,y}\cdot f_y=[10^{-6}]x1307.0x10^3x275=359.4\text{ kNm}$
 $\chi_y\cdot N_{rk}/\gamma_{M1}=\chi_y\cdot A\cdot f_y/\gamma_{M1}=0.631x[10^{-3}]x8446x275/1.00=1465.6\text{ kN}$
 $\chi_z\cdot N_{rk}/\gamma_{M1}=\chi_z\cdot A\cdot f_y/\gamma_{M1}=0.494x[10^{-3}]x8446x275/1.00=1147.4\text{ kN}$
 $\chi_{LT}\cdot M_{y,rk}/\gamma_{M1}=\chi_{LT}\cdot W_{pl,y}\cdot f_y/\gamma_{M1}=0.940x[10^{-6}]x1307.0x10^3x275/1.00=337.9\text{ kNm}$

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

$k_{yy}=C_{my}\cdot C_{mLT}(\mu_y/(1-N_{ed}/N_{cr,y}))(1/C_{yy})$, $\mu_y=(1-N_{ed}/N_{cr,y})/(1-\chi_y\cdot N_{ed}/N_{cr,y})$ (EC3 Tab.A.1)
 $k_{zy}=C_{my}\cdot C_{mLT}(\mu_z/(1-N_{ed}/N_{cr,y}))(1/C_{zy})0.60\sqrt{(w_y/w_z)}$, $\mu_z=(1-N_{ed}/N_{cr,z})/(1-\chi_z\cdot N_{ed}/N_{cr,z})$

$N_{cr,y} = \pi^2 EI_y / l_{cr,y}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 231.30 \times 10^6 / 15000^2 = 2131 \text{ kN}$
 $N_{cr,z} = \pi^2 EI_z / l_{cr,z}^2 = 3.14^2 \times [10^{-3}] \times 210000 \times 13.180 \times 10^6 / 4000^2 = 1707 \text{ kN}$
 $N_{cr,t} = (1 / i_p^2) \times (G \cdot I_t + \pi^2 EI_w / L_{cr,t}^2)$ (EC3 NCCI SN003b-EN-EU)
 $N_{cr,t} = [10^{-3}] \times (1 / 170^2) [80769 \times 0.511 \times 10^6 + \pi^2 \times 210000 \times 490.05 \times 10^9 / 4000^2] = 3618 \text{ kN}$

$\mu_y = (1 - N_{ed} / N_{cr,y}) / (1 - \chi_y \cdot N_{ed} / N_{cr,y}) = (1 - 33.2 / 2131) / (1 - 0.631 \times 33.2 / 2131) = 0.994$
 $\mu_z = (1 - N_{ed} / N_{cr,z}) / (1 - \chi_z \cdot N_{ed} / N_{cr,z}) = (1 - 33.2 / 1707) / (1 - 0.494 \times 33.2 / 1707) = 0.990$
 $alt = 1 - I_t / I_y > 0 = 1 - 0.511 \times 10^6 / 231.30 \times 10^6 = 0.998$ (EC3 Annex A.1)

$w_y = W_{pl,y} / W_{el,y} < 1.50$, $w_y = 1.307 \times 10^6 / 1.156 \times 10^6 = 1.131 < 1.50$ (EC3 Annex A.1)
 $w_z = W_{pl,z} / W_{el,z} < 1.50$, $w_z = 0.229 \times 10^6 / 0.146 \times 10^6 = 1.564 > 1.50$, $w_z = 1.50$
 $n_{pl} = N_{ed} / (N_{rk} / \gamma_{M1}) = 33.20 / (2322.60 / 1.00) = 0.014$

$\bar{\lambda}_{max} = \max(1.049, 1.172) = 1.170$ (EC3 Annex A.1)
 $M_{cr,o} = (1.00 / 1.92) \times 732.90 = 381.8$, $C1 = 1.00$
 $\bar{\lambda}_o = \sqrt{([10^{-6}] \times 1307.0 \times 10^3 \times 275 / 381.8)} = 0.970$
 $\bar{\lambda}_{o,lim} = 0.2 \sqrt{C1 [(1 - N_{ed} / N_{cr,z})(1 - N_{ed} / N_{cr,t})]^{0.25}}$ (EC3 Annex A.1)
 $\bar{\lambda}_{o,lim} = 0.2 \sqrt{1.920 [(1 - 33.2 / 1707)(1 - 33.2 / 3618)]^{0.25}} = 0.275$
 $\varepsilon_y = (M_y, ed / N_{ed}) (A / W_{el}) = ([10^3] \times 153.84 / 33.20) \times (8446.0 / 1156.0 \times 10^3) = 33.85$

$C_{my,o} = 0.79 + 0.21\psi + 0.36(\psi - 0.33) \times (33.20 / 2131.0) = 1.004$, ($\psi = 1.00$) (EC3 Annex A, T.A.1)
 $\bar{\lambda}_o = 0.970 > \bar{\lambda}_{o,lim} = 0.275$
 $C_{my} = C_{my,o} + (1 - C_{my,o}) (\sqrt{\varepsilon_y \cdot alt}) / (1 + \sqrt{\varepsilon_y \cdot alt}) =$
 $= 1.004 + (1 - 1.004) \times (\sqrt{33.851 \times 0.998}) / (1 + \sqrt{33.851 \times 0.998}) = 1.001$
 $C_{m1t} = C_{my}^2 \cdot alt / \sqrt{[(1 - N_{ed} / N_{cr,z})(1 - N_{ed} / N_{cr,t})]} > 1$
 $C_{m1t} = 1.001^2 \times 0.998 / \sqrt{[(1 - 33.2 / 1707.0)(1 - 33.2 / 3618.0)]} = 1.015$, $C_{m1t} = 1.015$

$C_{yy} = 1 + (w_y - 1) [(2 - 1.6 C_{my}^2 \cdot \bar{\lambda}_{max} / w_y - 1.6 C_{my}^2 \cdot \bar{\lambda}_{max}^2 / w_y) n_{pl} - blt] > W_{el,y} / W_{pl,y}$ (Annex A, T.A.1)
 $blt = 0.5 alt \cdot \bar{\lambda}_o^2 [M_y, ed / (\chi_{lt} \cdot M_{pl,y}, rd)] (M_z, ed / M_{pl,z}, rd) =$
 $= 0.5 \times 0.998 \times 0.970^2 [0.0 / (0.940 \times 317.9)] (0.0 / 40.3) = 0.000$
 $C_{yy} = 1 + (1.131 - 1) [(2 - 1.6 \times 1.001^2 \times 1.170 / 1.131 - 1.6 \times 1.001^2 \times 1.170^2 / 1.131) \times 0.014 - 0.000] = 0.997$
 $C_{yy} > 1156.0 \times 10^3 / 1307.0 \times 10^3 = 0.884$, $C_{yy} = 0.997$

$C_{zy} = 1 + (w_y - 1) [(2 - 14.0 C_{my}^2 \cdot \bar{\lambda}_{max}^2 / w_y^5) n_{pl} - dlt] > 0.6 \sqrt{(w_y / w_z)} (W_{el,y} / W_{pl,y})$ (Annex A, T.A.1)
 $dlt = 2 alt \cdot [\bar{\lambda}_o / (0.1 + \bar{\lambda}_z^4)] [M_y, ed / (C_{my} \cdot \chi_{lt} \cdot M_{pl,y}, rd)] [M_z, ed / (C_{mz} \cdot M_{pl,z}, rd)] =$
 $= 20.998 \times [0.970 / (0.1 + 1.172^4)] [0.0 / (1.001 \times 0.940 \times 317.9)] [0.0 / (0.000 \times 40.3)] = 0.000$
 $C_{zy} = 1 + (1.131 - 1) [(2 - 14.0 \times 1.001^2 \times 1.170^2 / 1.131^5) 0.014 - 0.000] = 0.985$
 $C_{zy} > 0.6 \sqrt{(1.131 / 1.500)} (1156.0 \times 10^3 / 1307.0 \times 10^3) = 0.461$, $C_{zy} = 0.985$

$C_{yy} = 0.997$, $C_{zy} = 0.985$ (Annex A, T.A.1)
 $k_{yy} = 1.001 \times 1.015 \times 0.994 / (1 - 33.20 / 2131.0) \times (1 / 0.997) = 1.029$
 $k_{zy} = 1.001 \times 1.015 \times 0.990 / (1 - 33.20 / 2131.0) \times (1 / 0.985) \times 0.6 \sqrt{(1.131 / 1.500)} = 0.540$

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

$N_{ed} / (\chi_y \cdot N_{rk} / \gamma_{M1}) + k_{yy} \cdot M_y, ed / (\chi_{LT} \cdot M_y, rk / \gamma_{M1}) =$ (EC3 Eq.6.61)
 $33.2 / (0.631 \times 2322.6 / 1.00) + 1.029 \times 153.8 / (0.940 \times 359.4 / 1.00) = 0.023 + 0.469 = 0.491$
 $0.491 < 1.000$, Is verified

$N_{ed} / (\chi_z \cdot N_{rk} / \gamma_{M1}) + k_{zy} \cdot M_y, ed / (\chi_{LT} \cdot M_y, rk / \gamma_{M1}) =$ (EC3 Eq.6.62)
 $33.2 / (0.494 \times 2322.6 / 1.00) + 0.540 \times 153.8 / (0.940 \times 359.4 / 1.00) = 0.029 + 0.246 = 0.275$
 $0.275 < 1.000$, Is verified

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

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

$N_{ed} = 3.6 \text{ kN}$
 $V_{ed} = 14.2 \text{ kN}$
 $M_{yed} = 53.4 \text{ kNm}$, $M_{zed} = 0.0 \text{ kNm}$
 Rafter length $L_r = 15000 \text{ mm}$
 Buckling length, In-plane buckling $L_{cr,y} = 15000 \text{ mm}$ (System length)
 Buckling length, Out-of-plane buckling $L_{cr,z} = 4000 \text{ mm}$ (Torsional restrains of rafters)

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

$N_{c,ed}=3.61 \text{ kN}$, $L_{cr,y}=15.000 \text{ m}$, $L_{cr,z}=4.000 \text{ m}$

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

Buckling lengths: $L_{cr,y}=1.000 \times 15000=15000 \text{ mm}$, $L_{cr,z}=0.267 \times 15000=4000 \text{ mm}$

Non-dimensional slenderness (Cross-section Class: 1)

(EC3 §6.3.1.3)

$$\bar{\lambda}_y = \sqrt{(A \cdot f_y / N_{cr,y})} = (L_{cr,y} / i_y) \cdot (1 / \lambda_1) = (15000 / 165.5) \times (1 / 86.39) = 1.049$$

$$\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (4000 / 39.5) \times (1 / 86.39) = 1.172$$

$$\lambda_1 = \pi \sqrt{E / f_y} = 93.9 \text{ ε} = 86.39, \quad \varepsilon = \sqrt{(235 / f_y)} = 0.92$$

$$h/b = 400/180 = 2.22 > 1.20, \quad t_f = 13.5 \text{ mm} \leq 40 \text{ mm}$$

$$y-y \text{ buckling curve: } a, \text{ imperfection factor: } \alpha_y = 0.21, \quad \chi_y = 0.631$$

(T.6.2, T.6.1, Fig.6.4)

$$\Phi_y = 0.5 [1 + \alpha_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.5 \times [1 + 0.21 \times (1.049 - 0.2) + 1.049^2] = 1.139$$

$$\chi_y = 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}] = 1 / [1.139 + \sqrt{(1.139^2 - 1.049^2)}] = 0.631 \leq 1 \quad \chi_y = 0.631$$

$$z-z \text{ buckling curve: } b, \text{ imperfection factor: } \alpha_z = 0.34, \quad \chi_z = 0.494$$

$$\Phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = 0.5 \times [1 + 0.34 \times (1.172 - 0.2) + 1.172^2] = 1.352$$

$$\chi_z = 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}] = 1 / [1.352 + \sqrt{(1.352^2 - 1.172^2)}] = 0.494 \leq 1 \quad \chi_z = 0.494$$

$$\text{Reduction factor } \chi = 1 / [\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}], \quad \chi \leq 1.0, \quad \Phi = 0.5 [1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2], \quad \chi = 0.494$$

(EC3 Eq.6.49)

$$N_{b,rd} = \chi \cdot A \cdot f_y / \gamma_{M1} = 0.494 \times [10^{-3}] \times 8446 \times 275 / 1.00 = 1147.39 \text{ kN}$$

(EC3 Eq.6.47)

$$N_{c,ed} = 3.61 \text{ kN} < 1147.39 \text{ kN} = N_{b,rd}, \text{ Is verified}$$

$$N_{c,ed} / N_{b,rd} = 3.61 / 1147.39 = 0.003 < 1$$

14.13. Lateral torsional buckling, Rafter-Uplift conditions (ULS)

(EN1993-1-1, §6.3.2)

$M_{y,ed}=53.38 \text{ kNm}$, $L=15.000 \text{ m}$, $L_{cr,y}=15.000 \text{ m}$, $L_{cr,z}=4.000 \text{ m}$, $L_{cr,lt}=4.000 \text{ m}$

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

Hogging

$$k \cdot L = 4000 \text{ mm}, \quad z_g = -200 \text{ mm}, \quad z_j = 0 \text{ mm}$$

(EN1993:2002 Eq.C.11)

$$k_y = 1.0, \quad k_z = 1.0, \quad k_w = 1.0, \quad C1 = 1.000, \quad C2 = 0.000, \quad C3 = 1.000$$

$$M_{cr} = [10^{-6}] 1.000 \times [\pi^2 \times 2.1 \times 10^5 \times 13.180 \times 10^6 / 4000^2]$$

$$\times \{ [(1.0/1.0)^2 \times (490.05 \times 10^9 / 13.180 \times 10^6)]$$

$$+ 4000^2 \times 8.1 \times 10^4 \times 0.511 \times 10^6 / (\pi^2 \times 2.1 \times 10^5 \times 13.180 \times 10^6)]^{0.5} \} = 422.9 \text{ kNm}$$

$$\bar{\lambda}_{lt} = \sqrt{(W_{pl,y} \cdot f_y / M_{cr})} = \sqrt{[10^{-6}] \times 1307.0 \times 10^3 \times 275 / 422.9} = 0.922$$

(EC3 Eq.6.56)

$$h/b = 400/180 = 2.22 > 2.00 \text{ buckling curve: } c$$

$$\text{imperfection factor: } \alpha_{lt} = 0.49, \quad \beta = 0.75, \quad \chi_{lt} = 0.687$$

(T.6.3, T.6.5, Fig.6.4)

$$\Phi_{lt} = 0.5 [1 + \alpha_{lt} (\bar{\lambda}_{lt} - 0.2) + \bar{\lambda}_{lt}^2] = 0.5 \times [1 + 0.49 \times (0.922 - 0.2) + 0.75 \times 0.922^2] = 0.947$$

$$\chi_{lt} = 1 / [\Phi_{lt} + \sqrt{(\Phi_{lt}^2 - \beta \bar{\lambda}_{lt}^2)}] = 1 / [0.947 + \sqrt{(0.947^2 - 0.75 \times 0.947^2)}] = 0.687$$

$$\text{Reduction factor } \chi_{lt} = 1 / [\Phi_{lt} + \sqrt{(\Phi_{lt}^2 - \beta \bar{\lambda}_{lt}^2)}], \quad \chi_{lt} \leq 1.0, \quad 1 / \bar{\lambda}_{lt}^2, \quad \chi_{lt} = 0.687$$

(Eq.6.57)

$$\chi_{lt,mod} = \chi_{lt} / f, \quad \chi_{lt,mod} \leq 1, \quad \chi_{lt,mod} \leq 1 / \bar{\lambda}_{lt}^2 = 1 / 0.922^2 = 1.18$$

(EC3 §6.3.2.3(2), Eq.6.58)

$$K_c = 1 / (1.33 - 0.33\psi) = 0.752, \quad \psi = 0.00$$

(EC3 Tab.6.6)

$$f = 1 - 0.5(1 - K_c)[1 - 2.0(\bar{\lambda}_{lt} - 0.8)^2] = 1 - 0.5 \times (1 - 0.752)[1 - 2.0 \times (0.922 - 0.8)^2] = 0.880, \quad f \leq 1.0$$

$$\chi_{lt,mod} = \chi_{lt} / f = 0.687 / 0.880 = 0.781, \quad \chi_{lt,mod} \leq 1.0, \quad \chi_{lt,mod} \leq 1.18, \quad \chi_{lt,mod} = 0.781$$

$$M_{b,rd} = \chi_{lt} \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 0.781 \times [10^{-6}] \times 1307.0 \times 10^3 \times 275 / 1.00 = 280.71 \text{ kNm}$$

(EC3 Eq.6.55)

$$M_{y,ed} = 53.38 \text{ kNm} < 280.71 \text{ kNm} = M_{b,rd}, \text{ Is verified}$$

$$M_{y,ed} / M_{b,rd} = 53.38 / 280.71 = 0.190 < 1$$

15. Haunch verification (Ultimate Limit State)

(EN1993-1-1, §6)

The haunch is fabricated by cutting and welding of an IPE 400 section - S 275

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

at haunch end	at haunch-middle	at haunch-start
Ned = 33.2 kN	Ned = 33.2 kN	Ned = 33.2 kN
Ved = 117.0 kN	Ved = 105.3 kN	Ved = 93.6 kN
Myed = 231.2 kNm	Myed = 170.7 kNm	Myed = 96.1 kNm

Buckling length, In-plane buckling $L_{cr,y}=1500\text{mm}$ Buckling length, Out-of-plane buckling $L_{cr,z}=1500\text{mm}$

Maximum design values. Verification for load case: Seismic loading

at haunch end	at haunch-middle	at haunch-start
Ned = 13.2 kN	Ned = 13.2 kN	Ned = 13.2 kN
Ved = 40.2 kN	Ved = 40.2 kN	Ved = 40.2 kN
Myed = 98.4 kNm	Myed = 91.0 kNm	Myed = 80.9 kNm

15.1. Classification of steel cross-section, at haunch end

(EN1993-1-1, §5.5)

Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{ed}/W_{el,y} \pm M_{ed}/W_{el,z}$

$$\sigma = [10^3]33.20/11508 \pm [10^6]231.20/2706.7 \times 10^3 \pm [10^6]0.00/146.3 \times 10^3$$

$$\sigma_1 = 88 \text{ N/mm}^2, \sigma_2 = -83 \text{ N/mm}^2 \text{ (compression positive)}$$

Web

$$c = 800.0 - 2 \times 13.5 - 2 \times 21.0 = 731.0 \text{ mm}, t = 8.6 \text{ mm}, c/t = 731.0/8.6 = 85.00$$

$$S 275, t = 8.6 \leq 40 \text{ mm}, f_y = 275 \text{ N/mm}^2, \epsilon = (235/275)^{0.5} = 0.92$$

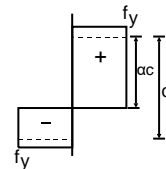
Position of neutral axis for combined Bending and compression

$$N_{ed}/(2t_w \cdot f_y / \gamma_{M0}) = 33200 / (2 \times 8.6 \times 275 / 1.00) = 7.0 \text{ mm}$$

$$\alpha = (731.0/2 + 7.0) / 731.0 = 0.510 > 0.5$$

$$c/t = 85.00 > 456 \times 0.92 / (13 \times 0.510 - 1) = 74.58$$

The web is not class 1 or 2

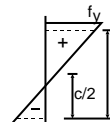


$$\sigma = N_{ed}/A \pm M_{ed} \cdot (0.5d)/I_y, \sigma_1 = 81 \text{ N/mm}^2, \sigma_2 = -75 \text{ N/mm}^2$$

$$\psi = -75/81 = -0.930 > -1$$

$$c/t = 85.00 \leq 42 \times 0.92 / (0.67 + 0.33 \times -0.930) = 106.42$$

The web is class 3 (EN1993-1-1, Tab.5.2)

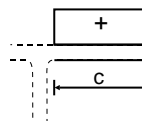
**Flange**

$$c = 180.0/2 - 8.6/2 - 21.0 = 64.7 \text{ mm}, t = 13.5 \text{ mm}, c/t = 64.7/13.5 = 4.79$$

$$S 275, t = 13.5 \leq 40 \text{ mm}, f_y = 275 \text{ N/mm}^2, \epsilon = (235/275)^{0.5} = 0.92$$

$$c/t = 4.79 \leq 9 \epsilon = 9 \times 0.92 = 8.28$$

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

**Overall classification of cross-section is Class 3, Bending and compression $N_{c,ed} + M_{y,ed}$** **15.2. Resistance of cross-section, at haunch end (Ultimate Limit State)**

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

$$N_{c,ed} = 33.20 \text{ kN}$$

$$\text{Compression Resistance } N_{pl,rd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 11508 \times 275 / 1.00 = 3164.65 \text{ kN}$$

$$N_{ed} = 33.20 \text{ kN} < 3164.65 \text{ kN} = N_{c,rd} = N_{pl,rd}, \text{ Is verified}$$

$$N_{ed}/N_{c,rd} = 33.20/3164.65 = 0.010 < 1$$

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

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

$$M_{y,ed} = 231.20 \text{ kNm}$$

$$\text{Bending Resistance } M_{y,rd} = W_{el,y} \cdot f_y / \gamma_{M0} = [10^{-6}] \times 2706.7 \times 10^3 \times 275 / 1.00 = 744.34 \text{ kNm}$$

$$M_{y,ed} = 231.20 \text{ kNm} < 744.34 \text{ kNm} = M_{y,rd} = M_{pl,y,rd}, \text{ Is verified}$$

$$M_{y,ed}/M_{y,rd} = 231.20/744.34 = 0.311 < 1$$

Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

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

Vz.ed=117.00 kN

$$A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 11508 - 2 \times 180.0 \times 13.5 + (8.6 + 2 \times 21.0) \times 13.5 = 7331 \text{ mm}^2$$

(EC3 §6.2.6.3)

$$A_v = 7331 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (800.0 - 2 \times 13.5) \times 8.6 = 1.00 \times 786.5 \times 8.6 = 6764 \text{ mm}^2$$

$$\text{Plastic Shear Resistance } V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 7331 \times (275 / 1.73) / 1.00 = 1163.94 \text{ kN}$$

$$V_{z,ed} = 117.00 \text{ kN} < 1163.94 \text{ kN} = V_{z,rd} = V_{pl,z,rd}, \text{ Is verified}$$

$$V_{z,ed} / V_{z,rd} = 117.00 / 1163.94 = 0.101 < 1$$

$$h_w / t_w = (800.0 - 2 \times 13.5) / 8.6 = 786.5 / 8.6 = 91.45 > 72 \times 0.92 / 1.00 = 72 \varepsilon / \eta = 66.24 \quad (\eta = 1.00)$$

$$S_{275}, t = 8.6 \leq 40 \text{ mm}, f_y = 275 \text{ N/mm}^2, \varepsilon = (235 / 275)^{0.5} = 0.92$$

Shear buckling resistance must be verified

(EC3 §6.2.6.6)

Shear buckling resistance

(EC3 EN1993-1-5:2006, §5)

$$\bar{\lambda}_w = (731.0 / 8.6) / (37.4 \times 0.92 \times \sqrt{(5.34)}) = 1.069, K_T = 5.34$$

(EC3-1-5 §5, Eq.5.6, A.3)

$$0.83 / \eta \leq \bar{\lambda}_w = 1.069 < 1.08, \chi_v = 0.83 / 1.069 = 0.776 \quad (\eta = 1.00)$$

(EC3-1-5 Tab.5.1)

$$V_{b,rd} = \chi_v \cdot f_{yw} \cdot h_w \cdot t_w / (\sqrt{3} \gamma_{M1}) = 0.001 \times 275 \times 0.776 \times 731.0 \times 8.6 / (1.73 \times 1.00) = 774.98 \text{ kN}$$

(EC3-1-5 Tab.5.1)

$$V_{ed} = 117 \text{ kN} < 775 = V_{b,rd} \text{ kN}, \text{ Is verified}$$

$$V_{ed} / V_{b,rd} = 117.00 / 774.98 = 0.151 < 1$$

Ultimate Limit State, Verification for axial force, shear and bending

(EN1993-1-1, §6.2.9)

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

N.ed= 33.20kN (Compression), Vz.ed= 117.00kN, My.ed= 231.20kNm

$$N_{pl,rd} = 3164.65 \text{ kN}, M_{el,y,rd} = 744.34 \text{ kNm}, V_{pl,z,rd} = 774.98 \text{ kN}$$

$$N_{ed} = 33.20 \text{ kN} \leq 0.25 \times 3164.65 = 0.25 \times N_{pl,rd} = 791.16 \text{ kN}$$

$$N_{ed} = 33.20 \text{ kN} \leq [10^{-3}] \times 0.5 \times 786.5 \times 8.6 \times 275 / 1.00 = 0.5 h_w \cdot t_w \cdot f_y / \gamma_{M0} = 930.04 \text{ kN}$$

$$n = N_{ed} / N_{pl,rd} = 33 / 3165 = 0.010$$

Effect of axial force is neglected

(EC3 §6.2.9.1 Eq.6.33, Eq.6.34, Eq.6.35)

$$V_{ed} = 117.00 \text{ kN} \leq 0.50 \times 774.98 = 0.50 \times V_{pl,rd} = 387.49 \text{ kN}$$

Effect of shear force is neglected

(EC3 §6.2.8.2)

Maximum and minimum cross-section stresses $\sigma = N_{ed} / A_{el} \pm M_{y,ed} / W_{el,y} \pm M_{z,ed} / W_{el,z}$

$$\sigma = [10^3] 0.00 / 11508 \pm [10^6] 231.20 / 2706.7 \times 10^3 \pm [10^6] 0.00 / 146.3 \times 10^3$$

$$\sigma_1 = 85 \text{ N/mm}^2, \sigma_2 = -85 \text{ N/mm}^2 \text{ (compression positive)}$$

$$\sigma_{x,ed} = 85 < 275 / 1.00 = 275 = f_y / \gamma_{M0} \text{ N/mm}^2, \text{ Is verified}$$

(EC3 Eq.6.42)

15.3. Out-of-plane buckling, at haunch end (Ultimate Limit State)

(EN1993-1-1, §6.3.2.4)

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

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

Properties of equivalent T-section

$$\text{Depth of cross section } h_f = 133 \text{ mm}$$

$$\text{Width of cross section } b_f = 180 \text{ mm}$$

$$\text{Web thickness } t_w = 8.60 \text{ mm}$$

$$\text{Flange thickness } t_f = 13.50 \text{ mm}$$

$$\text{Area } A_f = 3461 \text{ mm}^2$$

$$\text{Second moment of area } I_{f,z} = 6.561 \times 10^6 \text{ mm}^4$$

$$\text{Radius of gyration } i_{f,z} = \sqrt{(6.561 \times 10^6 / 3461)} = 43.5 \text{ mm}$$

Compression in the T-section

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

$$N_{ed,f} = N_{ed} \cdot A_f / A + M_{ed} \cdot A_f / W_{el,y} = 33.2 \times 3461 / 11508 + 231.2 \times 3461 \times 10^3 / 2706.7 \times 10^3 = 305.6 \text{ kN}$$

Maximum design values. Verification for load case: Seismic loading

$$N_{ed,f} = N_{ed} \cdot A_f / A + M_{ed} \cdot A_f / W_{el,y} = 13.2 \times 3461 / 11508 + 98.4 \times 3461 \times 10^3 / 2706.7 \times 10^3 = 129.8 \text{ kN}$$

$$N_{ed} = \max(305.6, 129.8) = 305.6 \text{ kN}$$

$$\bar{\lambda}_z = \sqrt{(A \cdot f_y / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (1500 / 43.5) \times (1 / 86.39) = 0.399$$

$$f_{z-f,z} \text{ buckling curve: c, imperfection factor: } \alpha_{f,z} = 0.49, \chi_{f,z} = 0.898 \quad (\text{T.6.2, T.6.1, Fig.6.4})$$

$$\Phi_{f,z} = 0.5 [1 + \alpha_{f,z} (\bar{\lambda}_{f,z} - 0.2) + \bar{\lambda}_{f,z}^2] = 0.5 [1 + 0.49 \times (0.399 - 0.2) + 0.399^2] = 0.628$$

$$\chi_{f,z} = 1 / [\Phi_{f,z} + \sqrt{(\Phi_{f,z}^2 - \bar{\lambda}_{f,z}^2)}] = 1 / [0.628 + \sqrt{(0.628^2 - 0.399^2)}] = 0.898 \leq 1 \quad \chi_{f,z} = 0.898$$

$$N_{b,rd} = \chi_{f,z} \cdot A \cdot f_y / \gamma_{M1} = 0.898 \times 3461 \times 275 / 1.00 = 854.59 \text{ kN}$$

(EC3 Eq.6.47)

$$N_{c,ed} = 305.58 \text{ kN} < 854.59 \text{ kN} = N_{b,rd}, \text{ Is verified}$$

$$N_{c,ed} / N_{b,rd} = 305.58 / 854.59 = 0.358 < 1$$

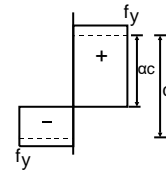
15.4. Classification of steel cross-section, at haunch-middle

(EN1993-1-1, §5.5)

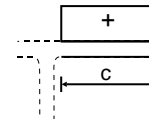
Maximum and minimum cross-section stresses $\sigma = N_{ed}/A_{el} \pm M_{y,ed}/W_{el,y} \pm M_{z,ed}/W_{el,z}$
 $\sigma = [10^3]33.20/9788 \pm [10^6]170.70/1842.8 \times 10^3 \pm [10^6]0.00/146.1 \times 10^3$
 $\sigma_1 = 96 \text{ N/mm}^2$, $\sigma_2 = -89 \text{ N/mm}^2$ (compression positive)

Web

$c = 600.0 - 2 \times 13.5 - 2 \times 21.0 = 531.0 \text{ mm}$, $t = 8.6 \text{ mm}$, $c/t = 531.0/8.6 = 61.74$
 $S 275$, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 Position of neutral axis for combined Bending and compression
 $N_{ed}/(2t \cdot f_y / \gamma_{M0}) = 33200 / (2 \times 8.6 \times 275 / 1.00) = 7.0 \text{ mm}$
 $\alpha = (531.0 / 2 + 7.0) / 531.0 = 0.513 > 0.5$
 $c/t = 61.74 \leq 396 \times 0.92 / (13 \times 0.513 - 1) = 64.23$
 The web is class 1 (EN1993-1-1, Tab.5.2)

Flange

$c = 180.0 / 2 - 8.6 / 2 - 21.0 = 64.7 \text{ mm}$, $t = 13.5 \text{ mm}$, $c/t = 64.7/13.5 = 4.79$
 $S 275$, $t = 13.5 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$
 $c/t = 4.79 \leq 9 \epsilon = 9 \times 0.92 = 8.28$
 The flange is class 1 (EN1993-1-1, Tab.5.2)



Overall classification of cross-section is Class 1, Bending and compression $N_{c,ed} + M_{y,ed}$

15.5. Resistance of cross-section, at haunch-middle (Ultimate Limit State)

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for compression

(EN1993-1-1, §6.2.4)

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

$N_{c,ed} = 33.20 \text{ kN}$

Compression Resistance $N_{pl,rd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 9788 \times 275 / 1.00 = 2691.64 \text{ kN}$

$N_{ed} = 33.20 \text{ kN} < 2691.64 \text{ kN} = N_{c,rd} = N_{pl,rd}$, Is verified

$N_{ed}/N_{c,rd} = 33.20/2691.64 = 0.012 < 1$

Ultimate Limit State, Verification for bending moment y-y

(EN1993-1-1, §6.2.5)

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

$M_{y,ed} = 170.70 \text{ kNm}$

Bending Resistance $M_{pl,y,rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = [10^{-6}] \times 2131.1 \times 10^3 \times 275 / 1.00 = 586.05 \text{ kNm}$

$M_{y,ed} = 170.70 \text{ kNm} < 586.05 \text{ kNm} = M_{y,rd} = M_{pl,y,rd}$, Is verified

$M_{y,ed}/M_{y,rd} = 170.70/586.05 = 0.291 < 1$

Ultimate Limit State, Verification for shear z

(EN1993-1-1, §6.2.6)

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

$V_{z,ed} = 105.30 \text{ kN}$

$A_v = A - 2b \cdot t_f + (t_w + 2r) t_f = 9788 - 2 \times 180.0 \times 13.5 + (8.6 + 2 \times 21.0) \times 13.5 = 5611 \text{ mm}^2$

(EC3 §6.2.6.3)

$A_v = 5611 \text{ mm}^2 > \eta \cdot h_w \cdot t_w = 1.00 \times (600.0 - 2 \times 13.5) \times 8.6 = 1.00 \times 586.5 \times 8.6 = 5044 \text{ mm}^2$

Plastic Shear Resistance $V_{pl,z,rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = [10^{-3}] \times 5611 \times (275 / 1.73) / 1.00 = 890.85 \text{ kN}$

$V_{z,ed} = 105.30 \text{ kN} < 890.85 \text{ kN} = V_{z,rd} = V_{pl,z,rd}$, Is verified

$V_{z,ed}/V_{z,rd} = 105.30/890.85 = 0.118 < 1$

$h_w/t_w = (600.0 - 2 \times 13.5) / 8.6 = 586.5 / 8.6 = 68.20 > 72 \times 0.92 / 1.00 = 72 \epsilon / \eta = 66.24$ ($\eta = 1.00$)

$S 275$, $t = 8.6 \leq 40 \text{ mm}$, $f_y = 275 \text{ N/mm}^2$, $\epsilon = (235/275)^{0.5} = 0.92$

Shear buckling resistance must be verified

(EC3 §6.2.6.6)

Shear buckling resistance

(EC3 EN1993-1-5:2006, §5)

$\bar{\lambda}_w = (531.0 / 8.6) / (37.4 \times 0.92 \times \sqrt{5.34}) = 0.777$, $K_t = 5.34$

(EC3-1-5 §5, Eq.5.6, A.3)

$\bar{\lambda}_w = 0.777 < 0.83 / \eta$, $\chi_w = \eta = 1.000$ ($\eta = 1.00$)

(EC3-1-5 Tab.5.1)

$V_{b,rd} = \chi_w \cdot f_{yw} \cdot h_w \cdot t / (\sqrt{3} \gamma_{M1}) = 0.001 \times 275 \times 1.000 \times 531.0 \times 8.6 / (1.73 \times 1.00) = 725.07 \text{ kN}$

(EC3-1-5 Tab.5.1)

$V_{ed} = 105 \text{ kN} < 725 = V_{b,rd} \text{ kN}$, Is verified

$V_{ed}/V_{b,rd} = 105.30/725.07 = 0.145 < 1$

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

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

N.ed= 33.20kN (Compression), Vz.ed= 105.30kN, My.ed= 170.70kNm

Nplrd=2691.64kN, Mpl,y,rd=586.05kNm, Vpl,z,rd=725.07kN

Ned=33.20kN <= 0.25x2691.64=0.25xNplrd=672.91kN

Ned=33.20kN <= $[10^{-3}] \times 0.5 \times 586.5 \times 8.6 \times 275 / 1.00 = 0.5 \text{hw} \cdot \text{tw} \cdot \text{fy} / \gamma_{M0} = 693.54 \text{ kN}$

n=Ned/Nplrd=33/2692= 0.012

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

Ved=105.30kN <= 0.50x725.07=0.50xVpl,rd=362.54kN

Effect of shear force is neglected (EC3 §6.2.8.2)

My,ed= 170.70 kNm < 586.05 kNm =Mply,rd, Is verified

My,ed/Mply,rd= 170.70/586.05= 0.291<1

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

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

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

Properties of equivalent T-section

Depth of cross section	hf =	100 mm
Width of cross section	bf =	180 mm
Web thickness	tw =	8.60 mm
Flange thickness	tf =	13.50 mm
Area	Af =	3174 mm ²
Second moment of area	If,z =	6.561x10 ⁶ mm ⁴
Radius of gyration	if,z =	$\sqrt{(6.561 \times 10^6 / 3174)} = 45.5 \text{ mm}$

Compression in the T-section

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

Ned,f=Ned·Af/A+Med·Af/Wel,y=33.2x3174/9788+167.7x3174x10³/1842.8x10³=299.6kN

Maximum design values. Verification for load case: Seismic loading

Ned,f=Ned·Af/A+Med·Af/Wel,y=13.2x3174/9788+91.0x3174x10³/1842.8x10³=161.0kN

Ned=max(299.6,161.0)= 299.6 kN

$\bar{\lambda}_z = \sqrt{(A \cdot \text{fy} / N_{cr,z})} = (L_{cr,z} / i_z) \cdot (1 / \lambda_1) = (1500 / 45.5) \times (1 / 86.39) = 0.382$

f,z=f,z buckling curve:c, imperfection factor:αf,z=0.49, χf,z=0.907 (T.6.2,T.6.1, Fig.6.4)

Φf,z=0.5[1+αf,z(λf,z-0.2)+λf,z²]=0.5[1+0.49x(0.382-0.2)+0.382²]=0.618

χf,z=1/[Φf,z+√(Φf,z²-λf,z²)]=1/[0.618+√(0.618²-0.382²)]=0.907 <=1 χf,z=0.907

Nb,rd=χz·A·fy/γM1= 0.907x3174x275/1.00=791.65kN (EC3 Eq.6.47)

Nc,ed= 299.60 kN < 791.65 kN =Nb,rd, Is verified

Nc,ed/Nb,rd= 299.60/791.65= 0.378<1

Connections16. Connection data

(EN1993-1-8)

16.1. Bolt connection data (eave, apex)

(EN1993-1-8)

Type of connection	End-plate connection, non-preloaded bolts	
Category of connection	Category A: Bearing type	(EC3-1-8 §3.4.1)
	Category D: Non-preloaded	(EC3-1-8 §3.4.2)
End Plate	Thickness $t_p=20$ mm, S 275	
Plate of Eave connection	180x935x20 mm, S 275	
Bolts	M24, Strength grade 10.9	
Bolt diameter	$d = 24$ mm	
Diameter of holes	$d_o = 26$ mm	
Nominal area	$\pi d^2/4 = \pi \times 24^2/4 = 452.4$ mm ²	
Tensile stress area	$A_s = 353.0$ mm ²	
Bolt strength grade	10.9, $f_{yb}=900$ N/mm ² , $f_{ub}=1000$ N/mm ²	(EC3-1-8 §3.1.1)

16.2. Edge distances and spacing of bolts (eave, apex)

(EN1993-1-8, §3.5, Tab.3.3)

Minimum edge distances	$e_1=1.2d_o=1.2 \times 26=32$ mm $e_2=1.2d_o=1.2 \times 26=32$ mm
Maximum edge distances	$e_1=4t+40=4 \times 19.0+40=117$ mm $e_2=4t+40=4 \times 19.0+40=117$ mm
Minimum spacing of bolts	$p_1=2.2d_o=2.2 \times 26=58$ mm $p_2=2.4d_o=2.4 \times 26=63$ mm
Maximum spacing of bolts	$p_1=\min(14t, 200)=\min(14 \times 19.0, 200)=200$ mm $p_2=\min(14t, 200)=\min(14 \times 19.0, 200)=200$ mm
Distance of plate edge to bolt line	$e_1=e_2=e_x=45$ mm
Distance of section edge to bolt line	$e_c=41$ mm
Distance of flange edge to bolt line	$e_f=45$ mm
Pitch between bolt rows	$p_1=p_3=p=90$ mm
Spacing between cross centers	$p_2=g=w=90$ mm
Flange to end-plate weld	$a_{tf} \geq 0.55t_f=0.55 \times 13.5=8$ mm
Web to end-plate weld	$a_w \geq 0.55t_w=0.55 \times 8.6=6$ mm

16.3. Design resistance of individual bolts (eave, apex)

(EC3-1-8 §3.6.1, Tab.3.4)

Bolt strength grade=10.9,	$f_{ub}=1000$ N/mm ² , $A_s=353.0$ mm ² , $\gamma_{M2}=1.25$
Tension resistance of bolts	$F_{t,rd}=k_2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($k_2=0.90$) $F_{t,rd}=[10^{-3}] \times 0.90 \times 1000 \times 353.0 / 1.25=254$ kN
Shear resistance of bolts	$F_{v,rd}=\alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($\alpha_v=0.50$) $F_{v,rd}=[10^{-3}] \times 0.50 \times 1000 \times 353.0 / 1.25=141$ kN

17. Eave connection**17.1. Basic data (Eave connection)**Design forces of connection (Eave connection)

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

Ned = -33.2 kN

Ved = 116.5 kN

Med = -231.3 kNm

Maximum design values for actions (Seismic loading)

Ned = -13.2 kN

Ved = 40.2 kN

Med = -93.2 kNm

17.2. Connection data (Eave connection)Bolt connection data

End Plate 180x935x20 mm, S 275

Bolts M24, Bolt strength grade 10.9

Number of Bolts top 2x2=4

bottom 2x1=2

Total number of bolts =6

Diameter of holes do = 26 mm

Shear plane of bolt through the threaded portion

Edge distances and spacing of bolts

Distance of plate edge to bolt line e1=e2=ex= 45 mm

Distance of section edge to bolt line ec= 41 mm

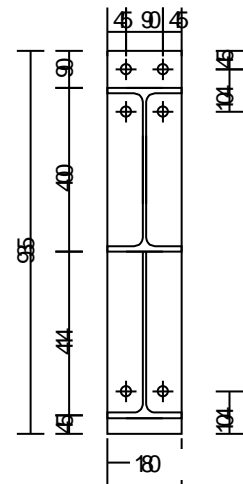
Distance of flange edge to bolt line ef= 45 mm

Pitch between bolt rows p1=p3=p= 90 mm

Spacing between cross centers p2=g=w= 90 mm

Flange to end-plate weld atf>= 0.55tf=0.55x13.5= 8 mm

Web to end-plate weld aw>= 0.55tw=0.55x 8.6= 6 mm

Compression stiffener at the bottom of haunch

Compression stiffener with thickness ts= 20.0 mm

17.3. Connection geometry of end-plate (Eave connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

e=ex=45 mm, emin=45 mm

 $m_x, x = (90 - 8.6 - 2 \times 0.8 \times 6 \times \sqrt{2}) / 2 = 33.9 \text{ mm}$ $m_x, y = 45 - 0.8 \times 8 \times \sqrt{2} = 35.9 \text{ mm}$ $n_x, x = \text{emin} \leq 1.25m_x, x = \min(45.0, 1.25 \times 33.9 = 42.4) = 42.4 \text{ mm}$ $n_x, y = \text{emin} \leq 1.25m_x, y = \min(45.0, 1.25 \times 35.9 = 44.9) = 44.9 \text{ mm}$ $\min(m_x, x, m_x, y) = \min(33.9, 35.9) = 33.9 \text{ mm}$, $\max(m_x, x, m_x, y) = \max(33.9, 35.9) = 35.9 \text{ mm}$ $\min(n_x, x, n_x, y) = \min(42.4, 44.9) = 42.4 \text{ mm}$, $\max(n_x, x, n_x, y) = \max(42.4, 44.9) = 44.9 \text{ mm}$ **17.4. Effective lengths of end-plate (Eave connection)**

(EC3-1-8 §6.2.6.5 Tab.6.6)

Bolt-row outside tension flange of beam $l_{eff} = 2 \cdot \pi \cdot m_x = 2 \cdot \pi \cdot 35.9 = 225.6 \text{ mm}$ $= \pi \cdot m_x + w = \pi \cdot 35.9 + 90.0 = 202.8 \text{ mm}$ $= \pi \cdot m_x + 2e = \pi \cdot 35.9 + 2 \times 45.0 = 202.8 \text{ mm}$ $= 4m_x + 1.25e_x = 4 \times 35.9 + 1.25 \times 45.0 = 199.9 \text{ mm}$ $= e + 2m_x + 0.625e_x = 45.0 + 2 \times 35.9 + 0.625 \times 45.0 = 144.9 \text{ mm}$ $= 0.5b_p = 0.5 \times 180 = 90.0 \text{ mm}$ $= 0.5w + 2m_x + 0.625e_x = 0.5 \times 90.0 + 2 \times 35.9 + 0.625 \times 45.0 = 144.9 \text{ mm}$ $l_{eff, lb} = \min(225.6, 202.8, 202.8, 199.9, 144.9, 90.0, 144.9) = 90.0 \text{ mm}$ $l_{eff, lb} = 90.0 \text{ mm}$

Bolt next to tension flange alone

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 33.9 = 213.0 \text{ mm} \\
 &= \alpha \cdot m = 6.28 \times 33.9 = 213.0 \text{ mm} \quad (\lambda_1 = \lambda_2 = m / (m + e) = 0.43, \alpha = 6.28) \quad (\text{EC3-1-8 Fig.6.11}) \\
 l_{eff,2b} &= \min(213.0, 213.0) = 213.0 \text{ mm} \\
 l_{eff,2b} &= 213.0 \text{ mm}
 \end{aligned}$$

Bolt next to tension flange in a group

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 33.9 = 213.0 \text{ mm} \\
 &= \alpha \cdot m = 6.28 \times 33.9 = 213.0 \text{ mm} \quad (\lambda_1 = \lambda_2 = m / (m + e) = 0.43, \alpha = 6.28) \\
 &= \pi m + p = \pi \times 33.9 + 90.0 = 196.5 \text{ mm} \\
 &= 0.5p + \alpha \cdot m - (2m + 0.625e) = 0.5 \times 90.0 + 6.3 \times 33.9 - (2 \times 33.9 + 0.625 \times 45.0) = 162.1 \text{ mm} \\
 l_{eff,3b} &= \min(213.0, 213.0, 196.5, 162.1) = 162.1 \text{ mm} \\
 l_{eff,3b} &= 162.1 \text{ mm}
 \end{aligned}$$

Inner Bolt-row in a group

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 33.9 = 213.0 \text{ mm} \\
 &= 4m + 1.25e = 4 \times 33.9 + 1.25 \times 45.0 = 191.9 \text{ mm} \\
 &= 2p = 2 \times 90.0 = 180.0 \text{ mm} \\
 &= p = 90.0 \text{ mm} \\
 l_{eff,4b} &= \min(213.0, 191.9, 180.0, 90.0) = 90.0 \text{ mm} \\
 l_{eff,4b} &= 90.0 \text{ mm}
 \end{aligned}$$

17.5. End-Plate, Resistance of T-stub flange (Eave connection)

(EC3-1-8 §6.2.4.1, Tab.6.2)

Bolt-row outside tension flange of beam

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 20.0^2 \times 275 / 1.00 = 2.475 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4M_{pl,1,rd} / m = [10^3] \times 4 \times 2.475 / 35.9 = 276 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 2.475 + 44.9 \times 2 \times 254) / (35.9 + 44.9) = 344 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(276, 344, 508) = 276 \text{ kN}
 \end{aligned}$$

Bolt next to tension flange alone

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 213.0 \times 20.0^2 \times 275 / 1.00 = 5.858 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4M_{pl,1,rd} / m = [10^3] \times 4 \times 5.858 / 33.9 = 691 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 5.858 + 42.4 \times 2 \times 254) / (33.9 + 42.4) = 436 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(691, 436, 508) = 436 \text{ kN}
 \end{aligned}$$

Bolt next to tension flange in a group

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 162.1 \times 20.0^2 \times 275 / 1.00 = 4.458 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4M_{pl,1,rd} / m = [10^3] \times 4 \times 4.458 / 33.9 = 526 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 4.458 + 42.4 \times 2 \times 254) / (33.9 + 42.4) = 399 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(526, 399, 508) = 399 \text{ kN}
 \end{aligned}$$

Inner Bolt-row in a group

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 20.0^2 \times 275 / 1.00 = 2.475 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4M_{pl,1,rd} / m = [10^3] \times 4 \times 2.475 / 35.9 = 276 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m + n) = ([10^3] \times 2 \times 2.475 + 44.9 \times 2 \times 254) / (35.9 + 44.9) = 344 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN} \\
 F_{t,rd} &= \min(276, 344, 508) = 276 \text{ kN}
 \end{aligned}$$

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

(EC3-1-8 §6.2.6.7)

$$\begin{aligned}
 F_{c,fb,rd} &= M_{c,rd} / (h - t_f), \quad M_{c,rd} = W_{el,y} \cdot f_y / \gamma_{M0} \\
 W_{el,y} &= (180 \times 13.5 \times 786.5^2 + 8.6 \times 773.0^3 / 6) / 800 = 2706.5 \times 10^3 \text{ mm}^3 \\
 M_{c,rd} &= [10^{-6}] \times 2706.5 \times 10^3 \times 275 / 1.00 = 744 \text{ kNm}, \quad F_{c,fb,rd} = [10^3] \times 744 / 786.5 = 946 \text{ kN} \\
 F_{c,fb,rd,max} &= (1/0.8) b \cdot t \cdot f_y / \gamma_{M0} = (1/0.8) \times [10^{-3}] \times 180.0 \times 13.5 \times 275 / 1.00 = 835 \text{ kN} \quad (h > 600 \text{ mm}) \\
 F_{c,fb,rd} &= \min(946, 835) = 835 \text{ kN}
 \end{aligned}$$

17.7. Rafter web in tension (Eave connection)

(EC3-1-8 §6.2.6.8)

$F_{t,wb,rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_y / \gamma_{M0}$
 $b_{eff,t,wb} = \min(l_{eff,3b}, l_{eff,4b}) = \min(162.1, 90.0) = 90.0 \text{ mm}$
 $F_{t,wb,rd} = [10^{-3}] \times 90.0 \times 8.6 \times 275 / 1.00 = 213 \text{ kN}$

$\min F_{t,rd} = \min(276, 436, 399, 276, 213) = 213 \text{ kN}$

17.8. Connection geometry of column-side (Eave connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

$e = e_x = 45 \text{ mm}$, $e_{min} = 45 \text{ mm}$
 $m_x = (90 - 11.0 - 2 \times 0.8 \times 27) / 2 = 17.9 \text{ mm}$
 $m_x = 45 - 0.8 \times 8 \times \sqrt{2} = 35.9 \text{ mm}$
 $n_x = e_{min} \leq 1.25 m_x$, $x = \min(45.0, 1.25 \times 17.9 = 22.4) = 22.4 \text{ mm}$
 $n_x = e_{min} \leq 1.25 m_x$, $y = \min(45.0, 1.25 \times 35.9 = 44.9) = 44.9 \text{ mm}$
 $\min(m_x, x, m_x, y) = \min(17.9, 35.9) = 17.9 \text{ mm}$, $\max(m_x, x, m_x, y) = \max(17.9, 35.9) = 35.9 \text{ mm}$
 $\min(n_x, x, n_x, y) = \min(22.4, 44.9) = 22.4 \text{ mm}$, $\max(n_x, x, n_x, y) = \max(22.4, 44.9) = 44.9 \text{ mm}$

17.9. Effective lengths of column-side (Eave connection)

(EC3-1-8 §6.2.6.4 Tab.6.4)

End Bolt-row in a group

$l_{eff} = 2\pi \cdot m = 2\pi \times 17.9 = 112.5 \text{ mm}$
 $= \pi \cdot m + 2e_1 = \pi \times 17.9 + 2 \times 45.0 = 146.2 \text{ mm}$
 $= 4m + 1.25e = 4 \times 17.9 + 1.25 \times 45.0 = 127.8 \text{ mm}$
 $= 2m + 0.63e + e_1 = 2 \times 17.9 + 0.63 \times 45.0 + 45.0 = 108.9 \text{ mm}$
 $= \pi \cdot m + p = \pi \times 17.9 + 90.0 = 146.2 \text{ mm}$
 $= 2e_1 + p = 2 \times 45.0 + 90.0 = 180.0 \text{ mm}$
 $= 2m + 0.63e + 0.5p = 2 \times 17.9 + 0.63 \times 45.0 + 0.5 \times 90.0 = 108.9 \text{ mm}$
 $= e_1 + 0.5p = 45.0 + 0.5 \times 90.0 = 90.0 \text{ mm}$
 $l_{eff,1c} = \min(112.5, 146.2, 127.8, 108.9, 146.2, 180.0, 108.9, 90.0) = 90.0 \text{ mm}$
 $l_{eff,1c} = 90.0 \text{ mm}$

Inner Bolt-row in a group

$l_{eff} = 2\pi \cdot m = 2\pi \times 17.9 = 112.5 \text{ mm}$
 $= 4m + 1.25e = 4 \times 17.9 + 1.25 \times 45.0 = 127.8 \text{ mm}$
 $= 2p = 2 \times 90.0 = 180.0 \text{ mm}$
 $= p = 90.0 \text{ mm}$
 $l_{eff,2c} = \min(112.5, 127.8, 180.0, 90.0) = 90.0 \text{ mm}$
 $l_{eff,2c} = 90.0 \text{ mm}$

17.10. Column-Side, Resistance of T-stub flange (Eave connection)

(EC3-1-8 §6.2.4.1, Tab.6.2)

End Bolt-row in a group

$M_{pl,1,rd} = M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 19.0^2 \times 275 / 1.00 = 2.234 \text{ kNm}$
Mode 1 $F_{t,1,rd} = 4M_{pl,1,rd} / m = [10^3] \times 4 \times 2.234 / 17.9 = 499 \text{ kN}$
Mode 2 $F_{t,2,rd} = (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m+n) = ([10^3] \times 2 \times 2.234 + 22.4 \times 2 \times 254) / (17.9 + 22.4) = 393 \text{ kN}$
Mode 3 $F_{t,3,rd} = \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN}$
 $F_{t,rd} = \min(499, 393, 508) = 393 \text{ kN}$

Inner Bolt-row in a group

$M_{pl,1,rd} = M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 19.0^2 \times 275 / 1.00 = 2.234 \text{ kNm}$
Mode 1 $F_{t,1,rd} = 4M_{pl,1,rd} / m = [10^3] \times 4 \times 2.234 / 17.9 = 499 \text{ kN}$
Mode 2 $F_{t,2,rd} = (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m+n) = ([10^3] \times 2 \times 2.234 + 22.4 \times 2 \times 254) / (17.9 + 22.4) = 393 \text{ kN}$
Mode 3 $F_{t,3,rd} = \Sigma F_{t,rd} = 2 \times 254 = 508 \text{ kN}$
 $F_{t,rd} = \min(499, 393, 508) = 393 \text{ kN}$

17.11. Column-web in transverse tension (Eave connection)

(EC3-1-8 §6.2.6.3)

$F_{t,wc,rd} = \omega \cdot b_{eff,t,wc} \cdot t_{wc} \cdot f_y / \gamma_{M0}$
 $\beta = 1$, $\omega = \omega_1 = 1 / \sqrt{[1 + 1.3(b_{eff,t,wc} / A_{vc})^2]}$, $b_{eff,t,wc} = 90.0 \text{ mm}$ (EC3-1-8 §6.2.6.2, Tab.6.3)
 $\omega = 1 / \sqrt{[1 + 1.3 \times (90.0 \times 11.0 / 5735)^2]} = 0.98$
 $F_{t,wc,rd} = [10^{-3}] \times 0.98 \times 90.0 \times 11.0 \times 275 / 1.00 = 267 \text{ kN}$

17.12. Design resistance of compression stiffener (Eave connection)

(EC3-1-5 §9.1)

Compression stiffener at the bottom of haunch $t_s = 20.0$ mm

$$f_y = 275 \text{ N/mm}^2, \quad b_s = (180 - 11.0 - 2 \times 27.0) / 2 = 57.5 \text{ mm}, \quad t_s = 20.0 \text{ mm}, \quad t_w = 11.0 \text{ mm}, \quad \varepsilon = \sqrt{235 / f_y} = 0.92$$

$$A_{eff,s} = 2 \times 57.5 \times 20.0 + (2 \times 15 \times 0.92 \times 11.0 + 20.0) \times 11.0 = 5860 \text{ mm}^2 \quad (\text{EC3-1-5 §9.1(2)})$$

$$I_{eff,s} = \min(57.5, 14 \times 0.92 \times 20.0) = \min(57.5, 257.60) = 57.5 \text{ mm}, \quad (\text{EC3 Tab.5.2})$$

$$I_{eff,s} = (2 \times 57.5 + 11.0)^3 \times 20.0 / 12 = 3334.0 \times 10^3 \text{ mm}^4$$

$$i_{eff,s} = \sqrt{(3334 \times 10^3 / 5860)} = 23.9 \text{ mm}, \quad \lambda_1 = \pi \sqrt{E / f_y} = 93.9, \quad \varepsilon = 86.39$$

$$L_{cr} = 0.75 \times (390 - 2 \times 19.0) = 264.0 \text{ mm} \quad (\text{EC3-1-5 §9.4(2)})$$

$$\bar{\lambda} = L_{cr} / (i_{eff,s} \times \lambda_1) = 264.0 / (23.9 \times 86.39) = 0.13 \quad (\text{EC3 §6.3.1.3(1)})$$

$$\bar{\lambda} < 0.20, \quad \chi = 1.00 \quad (\text{EC3 §6.3.1.2.4})$$

$$F_{c,wc,rd} = \chi \cdot A_{eff,s} \cdot f_y / \gamma_{M1} = 1.000 \times 5860 \times 275 / 1.00 = 1611 \text{ kN} > F_{c,fb,rd} = 835 \text{ kN}$$

Compression stiffener, Is verified

17.13. Moment resistance of connection (Eave connection)

(EN1993-1-8, §6.2.7.2)

$$M_{j,rd} = \sum h_r \cdot F_{tr,rd} \quad (\text{EN1993-1-8, §6.2.7.2 Eq.6.25})$$

hr: row numbering from top, distances from center of bottom (compression) flangeEnd-plate in bending (EC3-1-8 §6.2.4.5)

Force distribution in bolt rows

$$\text{Bolt-row 1, } h_r = 838.3 \text{ mm, } F_{t,rd} = 276 \text{ kN}$$

$$\text{Bolt-row 2, } h_r = 734.8 \text{ mm, } F_{t,rd} = 436 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} = 276 + 436 = 712 \text{ kN}$$

End-plate in bending (EC3-1-8 §6.2.4.4)

Force distribution in bolt rows

$$\text{Bolt-row 1, } h_r = 838.3 \text{ mm, } F_{t,rd} = 393 \text{ kN}$$

$$\text{Bolt-row 2, } h_r = 734.8 \text{ mm, } F_{t,rd} = 393 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} = 393 + 393 = 786 \text{ kN}$$

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

$$F_{t,wb,rd} = 213 \text{ kN}$$

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

$$F_{c,fb,rd} = 835 \text{ kN}$$

$$F_{t,rd} \leq F_{t,wb,rd} = 213 \text{ kN, } F_{c,ed} = \sum F_{t,rd} \leq F_{c,fb,rd} = 835 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} \leq F_{c,wc,rd} = 1611 \text{ kN}$$

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

$$\text{Bolt-row 1, } h_r = 838.3 \text{ mm, } F_{t,rd} = 213 \text{ kN}$$

$$\text{Bolt-row 2, } h_r = 734.8 \text{ mm, } F_{t,rd} = 213 \text{ kN}$$

$$F_{c,ed} = \sum F_{t,rd} = 213 + 213 = 426 \text{ kN}$$

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

$$M_{j,rd} = [10^{-3}] \times [213 \times 838.3 + 213 \times 734.8]$$

$$M_{j,rd} = 335 \text{ kNm}$$

$$M_{ed} = 231.3 \text{ kNm} < 335.0 \text{ kNm} = M_{j,rd}, \quad \text{Is verified}$$

17.14. Shear resistance (Eave connection)

(EN1993-1-8, §3.6.1 Tab.3.4)

Shear resistance of bolts

$$F_{v,rd} = \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2} = [10^{-3}] \times 0.50 \times 1000 \times 353.0 / 1.25 = 141 \text{ kN}$$

Shear plane of bolt: through the threaded portion

Bearing resistance of bolts

$$F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2}$$

End-Plate

$t = 20.0 \text{ mm}$, $d = 24 \text{ mm}$, $d_o = 26 \text{ mm}$, $e_1 = 45 \text{ mm}$, $e_2 = 45 \text{ mm}$, $p_1 = 90 \text{ mm}$, $f_{ub} = 1000 \text{ kN/mm}^2$, $f_u = 430 \text{ kN/mm}^2$,
 $\alpha_b = \min[f_{ub}/f_u, 1.0, e_1/3d_o, p_1/3d_o - 1/4] =$
 $= \min[1000/430, 1.0, 45/(3 \times 26), 90/(3 \times 26) - 0.25] = 0.58$
 $k_1 = \min[2.8e_2/d_o - 1.7, 1.4p_2/d_o - 1.7, 2.5] = \min[2.8 \times 45/26 - 1.7, 1.4 \times 90/26 - 1.7, 2.5] = 2.50$
 $F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.58 \times 430 \times 24 \times 20.0 / 1.25 = 238 \text{ kN}$

Column-Side

$t = 19.0 \text{ mm}$, $d = 24 \text{ mm}$, $d_o = 26 \text{ mm}$, $e_1 = 45 \text{ mm}$, $e_2 = 45 \text{ mm}$, $p_1 = 90 \text{ mm}$, $f_{ub} = 1000 \text{ kN/mm}^2$, $f_u = 430 \text{ kN/mm}^2$,
 $\alpha_b = \min[f_{ub}/f_u, 1.0, e_1/3d_o, p_1/3d_o - 1/4] =$
 $= \min[1000/430, 1.0, 45/(3 \times 26), 90/(3 \times 26) - 0.25] = 0.58$
 $k_1 = \min[2.8e_2/d_o - 1.7, 1.4p_2/d_o - 1.7, 2.5] = \min[2.8 \times 45/26 - 1.7, 1.4 \times 90/26 - 1.7, 2.5] = 2.50$
 $F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.58 \times 430 \times 24 \times 19.0 / 1.25 = 226 \text{ kN}$

Design resistance of one bolt in shear = $\min(141, 238, 226) = 141 \text{ kN}$

Bending moment and shear

(EN1993-1-8, §3.6.1 Tab.3.4)

Maximum tension force in bolts

$$F_{t,ed} = 213/2 = 106 \text{ kN}$$

Reduction of shear resistance due to bending

$$\rho = 1 - F_{t,ed} / 1.40 F_{t,rd} = 1 - 106 / (1.40 \times 254) = 0.70$$

Shear acting together with bending moment for all the bolts

$$V_{rd} = 6 \times 0.70 \times 141 = 592 \text{ kN}$$

$V_{ed} = 117 \text{ kN} < 592 \text{ kN} = V_{rd}$, Is verified

18. Column base Connection**18.1. Basic data (Base connection)**Design forces of connection (Base connection)

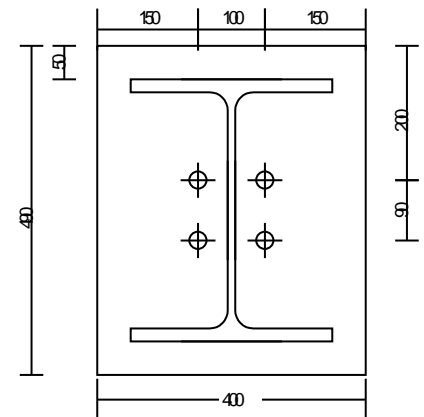
Axial force (compression) Ned=-130 kN, L.C. 202: 1.35Gk+1.50Qs1
 Axial force (tension) Ned= 11 kN, L.C. 210: 1.00Gk+1.50Qw1
 Shear force Ved= 41 kN, L.C. 211: 1.35Gk+1.50Qs1+0.60x1.50Qw1= 1.35xGk+1.50Qs1+0.90Qw1
 Moment Med= 0 kNm,

Seismic loading

Compression force at base Ned= -50 kN
 Tension force at base Ned= 0 kN
 Shear force at base Ved= 13 kN

Connection data (Base connection)

Base plate steel grade 490x400x30 mm, S 275
 Anchor bolts M24, Grade 5.6
 Shear plane of bolt through the threaded portion
 middle 2x2=4
 Total number of bolts =4
 Diameter of holes do = 26 mm
 Steel section for column HE 400 A, S 275
 Spacing between cross centers 100 mm
 Flange to end-plate weld 11 mm
 Web to end-plate weld 6 mm

Edge distances and spacing of bolts

Distance of plate edge to bolt line $e_1=e_2=e_x=150$ mm
 Distance of section edge to bolt line $e_c=45$ mm
 Distance of flange edge to bolt line $e_f=45$ mm
 Pitch between bolt rows $p_1=p_3=p=90$ mm
 Spacing between cross centers $p_2=g=w=100$ mm
 Flange to end-plate weld $a_{tf} \geq 0.55t_f = 0.55 \times 19.0 = 11$ mm
 Web to end-plate weld $a_{tw} \geq 0.55t_w = 0.55 \times 11.0 = 6$ mm

Concrete of foundation

Concrete-Steel class C25/30-B500C (EC2 §3.1, §3.2)
 Partial factors for materials $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 §2.4.2.4)
 Design compressive strength $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 1.00 \times 25 / 1.50 = 16.67$ N/mm² (EC2 §3.1.6)
 Design tensile strength $f_{ctd} = \alpha_{ct} \cdot f_{ctk05} / \gamma_c = 1.00 \times 2 / 1.50 = 1.20$ N/mm²
 Bearing strength $f_{jd} = \beta \cdot \sqrt{A_{c1} / A_{co}} \cdot f_{cd} = (2/3) \times 1.5 \times 16.67 = 16.67$ N/mm² (EC2 §6.7)

18.2. Design resistance of individual bolts (Base connection)

(EC3-1-8 §3.6.1, Tab.3.4)

Bolt strength grade=5.6, $f_{ub}=500$ N/mm², $A_s=353.0$ mm², $\gamma_{M2}=1.25$
 Tension resistance of bolts $F_{t,rd} = k_2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($k_2=0.90$)
 $F_{t,rd} = [10^{-3}] \times 0.90 \times 500 \times 353.0 / 1.25 = 127$ kN
 Shear resistance of bolts $F_{v,rd} = \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2}$, ($\alpha_v=0.60$)
 $F_{v,rd} = [10^{-3}] \times 0.60 \times 500 \times 353.0 / 1.25 = 85$ kN

18.3. Connection geometry of end-plate (Base connection)

(EC3-1-8 §6.2.4.1, Fig.6.2)

$e=e_x=150$ mm, $e_{min}=150$ mm
 $m_{x,x} = (100 - 11.0 - 2 \times 0.8 \times 6 \times \sqrt{2}) / 2 = 37.7$ mm
 $m_{x,y} = 37.7$ mm
 $n_{x,x} = e_{min} \leq 1.25 m_{x,x} = \min(150.0, 1.25 \times 37.7) = 47.1$ mm
 $n_{x,y} = e_{min} \leq 1.25 m_{x,y} = \min(150.0, 1.25 \times 37.7) = 47.1$ mm
 $\min(m_{x,x}, m_{x,y}) = \min(37.7, 37.7) = 37.7$ mm, $\max(m_{x,x}, m_{x,y}) = \max(37.7, 37.7) = 37.7$ mm
 $\min(n_{x,x}, n_{x,y}) = \min(47.1, 47.1) = 47.1$ mm, $\max(n_{x,x}, n_{x,y}) = \max(47.1, 47.1) = 47.1$ mm

18.4. Effective lengths of end-plate (Base connection)

(EC3-1-8 §6.2.6.5 Tab.6.6)

Inner Bolt-row in a group

$$\begin{aligned}
 l_{eff} &= 2\pi \cdot m_x = 2\pi \times 37.7 = 236.9 \text{ mm} \\
 &= 4m + 1.25e = 4 \times 37.7 + 1.25 \times 150.0 = 338.3 \text{ mm} \\
 &= 2p = 2 \times 90.0 = 180.0 \text{ mm} \\
 &= p = 90.0 \text{ mm} \\
 l_{eff,4b} &= \min(236.9, 338.3, 180.0, 90.0) = 90.0 \text{ mm} \\
 l_{eff,4b} &= 90.0 \text{ mm}
 \end{aligned}$$

18.5. End-Plate, Resistance of T-stub flange (Base connection)

(EC3-1-8 §6.2.4.1, Tab.6.2)

Inner Bolt-row in a group

$$\begin{aligned}
 M_{pl,1,rd} &= M_{pl,2,rd} = 0.25 l_{eff} \cdot t_f^2 \cdot f_y / \gamma_{M0} = [10^{-6}] \times 0.25 \times 90.0 \times 30.0^2 \times 275 / 1.00 = 5.569 \text{ kNm} \\
 \text{Mode 1} \quad F_{t,1,rd} &= 4M_{pl,1,rd} / m = [10^3] \times 4 \times 5.569 / 37.7 = 591 \text{ kN} \\
 \text{Mode 2} \quad F_{t,2,rd} &= (2M_{pl,2,rd} + n \Sigma F_{t,rd}) / (m+n) = ([10^3] \times 2 \times 5.569 + 47.1 \times 2 \times 127) / (37.7 + 47.1) = 272 \text{ kN} \\
 \text{Mode 3} \quad F_{t,3,rd} &= \Sigma F_{t,rd} = 2 \times 127 = 254 \text{ kN} \\
 F_{t,rd} &= \min(591, 272, 254) = 254 \text{ kN}
 \end{aligned}$$

18.6. Column web in tension (Base connection)

(EC3-1-8 §6.2.6.8)

$$\begin{aligned}
 F_{t,wb,rd} &= b_{eff,t,wb} \cdot t_{wb} \cdot f_y / \gamma_{M0} \\
 b_{eff,t,wb} &= l_{eff} = l_{eff,4b} = 90.0 \text{ mm} \\
 F_{t,wb,rd} &= [10^{-3}] \times 90.0 \times 11.0 \times 275 / 1.00 = 272 \text{ kN}
 \end{aligned}$$

$$\min F_{t,rd} = \min(254, 272) = 254 \text{ kN}$$

18.7. Tension resistance of connection

(EN1993-1-8, §6.2.4)

$$\begin{aligned}
 \text{Uplift force of connection} \quad F_{t,ed} &= 11 \text{ kN} \\
 \text{Tension resistance of connection} \quad F_{t,rd} &= 2 \times 254 = 508 \text{ kN} \\
 N_{ed} = 11 \text{ kN} < 508 \text{ kN} = N_{rd}, \quad \text{Is verified}
 \end{aligned}$$

18.8. Shear resistance (Base connection)

(EN1993-1-8, §3.6.1 Tab.3.4)

Shear resistance of bolts

$$\begin{aligned}
 F_{v,rd} &= \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2} = [10^{-3}] \times 0.60 \times 500 \times 353.0 / 1.25 = 85 \text{ kN} \\
 \text{Shear plane of bolt: through the threaded portion}
 \end{aligned}$$

Bearing resistance of bolts

$$\begin{aligned}
 F_{b,rd} &= k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} \\
 t &= 30.0 \text{ mm}, d = 24 \text{ mm}, d_o = 26 \text{ mm}, e_1 = 150 \text{ mm}, e_2 = 150 \text{ mm}, p_1 = 90 \text{ mm}, f_{ub} = 500 \text{ kN/mm}^2, f_u = 430 \text{ kN/mm}^2, \\
 \alpha_b &= \min[f_{ub}/f_u, 1.0, e_1/3d_o, p_1/3d_o - 1/4] = \\
 &= \min[500/430, 1.0, 150/(3 \times 26), 90/(3 \times 26) - 0.25] = 0.90 \\
 k_1 &= \min[2.8e_2/d_o - 1.7, 1.4p_2/d_o - 1.7, 2.5] = \min[2.8 \times 150/26 - 1.7, 1.4 \times 100/26 - 1.7, 2.5] = 2.50 \\
 F_{b,rd} &= k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.90 \times 430 \times 24 \times 30.0 / 1.25 = 560 \text{ kN}
 \end{aligned}$$

$$\text{Design resistance of one bolt in shear} = \min(85, 560) = 85 \text{ kN}$$

Tension and shear

(EN1993-1-8, §3.6.1 Tab.3.4)

$$\begin{aligned}
 \text{Maximum tension force in bolts} \quad F_{t,ed} &= 254 / 2 = 127 \text{ kN} \\
 \text{Reduction of shear resistance due to tension} \\
 \rho &= 1 - F_{t,ed} / 1.40 F_{t,rd} = 1 - 127 / (1.40 \times 127) = 0.29 \\
 \text{Shear acting together with tension for all the bolts} \\
 V_{rd} &= 4 \times 0.29 \times 85 = 99 \text{ kN}
 \end{aligned}$$

$$V_{ed} = 41 \text{ kN} < 99 \text{ kN} = V_{rd}, \quad \text{Is verified}$$

18.9. Bearing resistance (Base connection)

(EN1993-1-8, §6.2.5)

Compression resistance of T-stub flange $F_{c,rd} = f_{jd} \cdot b_{eff} \cdot l_{eff}$ (§6.2.5(3)Eq.6.4), §6.2.5(7)
 $f_{jd} = \beta \cdot \sqrt{(A_{c1}/A_{co})} \cdot f_{cd} = (2/3) \cdot \sqrt{(2.25)} \cdot 16.67 = 16.67 \text{ N/mm}^2$ (EC2 EN1992-1-1:2004, §6.7,Eq.6.63)
 $h = 390.0 \text{ mm}$, $b = 300.0 \text{ mm}$, $t_f = 19.0 \text{ mm}$, $t_w = 11.0 \text{ mm}$, $t_p = 30.0 \text{ mm}$
 $c = t_p \cdot (f_y / (3f_{jd} \cdot \gamma_{M0}))^{0.5} = 30 \times (275.00 / (3 \times 16.67 \times 1.00))^{0.5} = 70.3$, < 50.0 , $c = 50.0 \text{ mm}$ (Eq.6.5)
 $2c + b_f = 2 \times 50.0 + 300 = 400.0 \text{ mm} \leq b_p = 400 \text{ mm}$, $l_{eff} = 400.0 \text{ mm}$
 $A_{co,f} = l_{eff} \cdot (2c + t_f) = 400.0 \times (2 \times 50.0 + 19.0) = 47600 \text{ mm}^2$ (EC3-1-8, Fig.6.4)
 $A_{co,w} = (h - 2t_f - 2c) \cdot (t_w + 2c) = (390.0 - 2 \times 19.0 - 2 \times 50.0) \times (11.0 + 2 \times 50.0) = 27972 \text{ mm}^2$
 $N_{j,rd} = [10^{-3}] \times 16.7 \times (2 \times 47600 + 27972) = [10^{-3}] \times 16.7 \times 123172 = 2057 \text{ kN}$
 $N_{j,ed} = 130 \text{ kN} < 2057 \text{ kN} = N_{j,rd}$, Is verified

Bending resistance of base plate

(EN1993-1-8, §6.2.6.10)

$M_{p,rd} = W_{el} \cdot f_y / \gamma_{M0} = [10^{-6}] (400 \times 30.0^2 / 6) \times 275 / 1.0 = 16 \text{ kNm}$ (§6.2.5)
 $M_{p,ed} = b_p \cdot q_{ed} \cdot c^2 / 2 = [10^{-6}] [400 \times 129664 / (2 \times 47600 + 27972.0)] \times 50.0^2 / 2 = 1 \text{ kNm}$
 $M_{p,ed} = 1.0 \text{ kNm} < 16.0 \text{ kNm} = M_{p,rd}$, Is verified

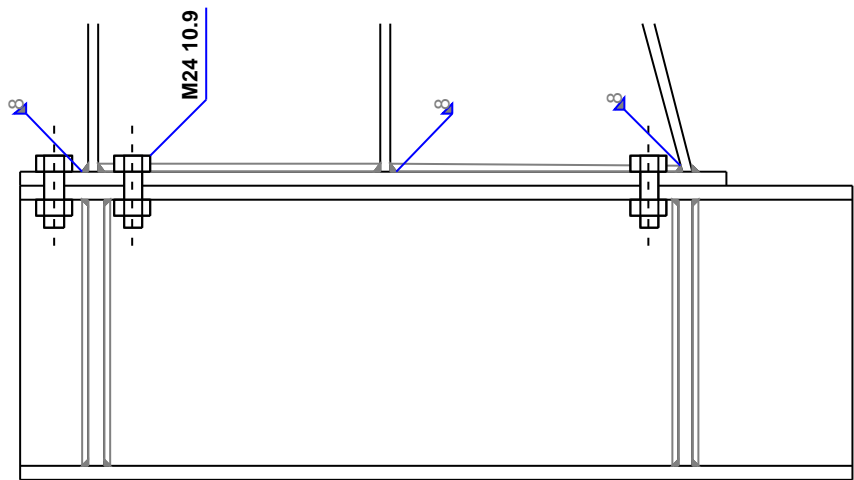
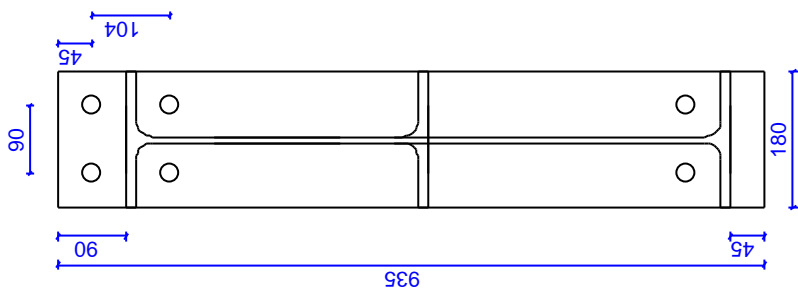
18.10. Anchoring resistance (Base connection)

(EN1993-1-8, §6.2.6.12)

Anchoring hook

(EN1992-1-1 §8.4)

Number of fasteners: 4, of diameter 24mm, $A_s = 353 \text{ mm}^2$
 Basic required anchorage length $l_{b,rqd}$, Design anchorage length $l_{bd} = 0.70 l_{b,rqd}$
 $l_{b,rqd} = (\sigma_s / 4) (\sigma_{sd} / f_{bd}) = (24/4) \times (7.8 / 1.20) = 39 \text{ mm}$
 $\sigma_{sd} = [10^3] \times 11 / (4 \times 353) = 7.8 \text{ N/mm}^2$, $f_{bd} = f_{ctd} = 1.20 \text{ N/mm}^2$
 Design anchorage length $l_{bd} = 0.70 \times 39 > (10 \times 24 / 100) \quad l_{bd} = 250 \text{ mm}$



Eave connection Scale 1:10 (mm)

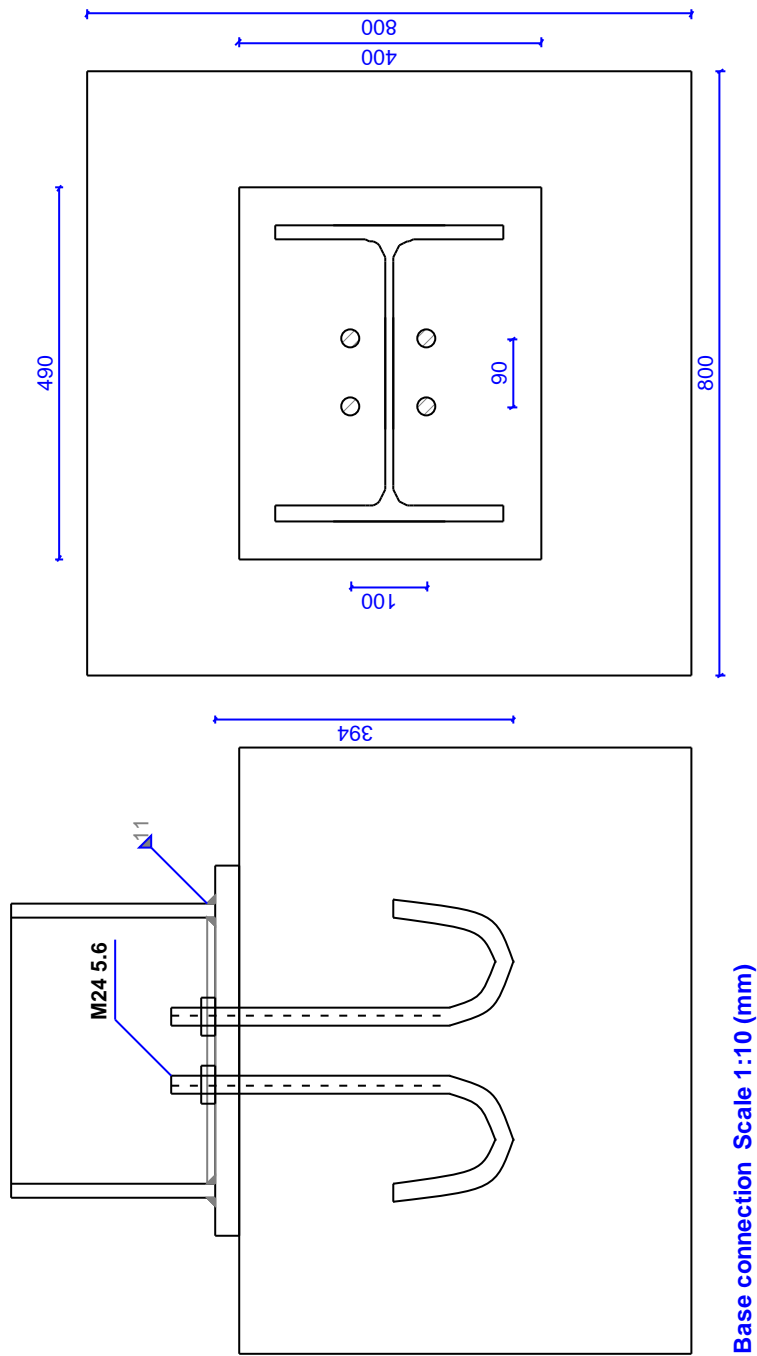


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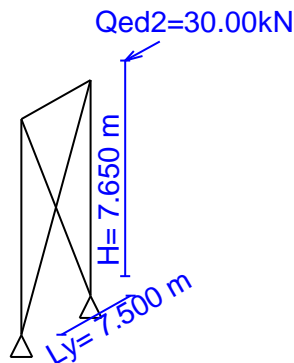
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STOGINĖ NR. 3 (jungtys)

1. BRACE-001

Vertical bracing system

(EC3 EN1993-1-1:2005,)

Design of lateral bracing system $H=16.000\text{m}$, $L_y=7.650\text{m}$, $Q_{ed2}=4.00\text{kN}$ 1.1. Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
 EN1991-1-1:2002, Eurocode 1-1 Actions on structures
 EN1993-1-1:2005, Eurocode 3 1-1 Design of steel structures
 EN1993-1-3:2005, Eurocode 3 1-3 Cold-formed members
 EN1993-1-5:2006, Eurocode 3 1-5 Plated structural elements
 EN1993-1-8:2005, Eurocode 3 1-8 Design of Joints

1.2. Materials

Steel: S 275 N/NL (EN1993-1-1, §3.2)

$t \leq 40 \text{ mm}$, Yield strength $f_y = 275 \text{ N/mm}^2$, Ultimate strength $f_u = 390 \text{ N/mm}^2$
 $40\text{mm} < t \leq 80 \text{ mm}$, Yield strength $f_y = 255 \text{ N/mm}^2$, Ultimate strength $f_u = 370 \text{ N/mm}^2$
 Modulus of elasticity $E = 210000 \text{ N/mm}^2$, Poisson ratio $\nu = 0.30$, Unit mass $\rho = 7850 \text{ Kg/m}^3$

Partial safety factors for actions (EN1990, Annex A1)

$\gamma_G = 1.35$, $\gamma_Q = 1.30$

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

$\gamma_{M0} = 1.00$, $\gamma_{M1} = 1.00$, $\gamma_{M2} = 1.25$

1.3. Dimensions and loads

(EN1991-1-1)

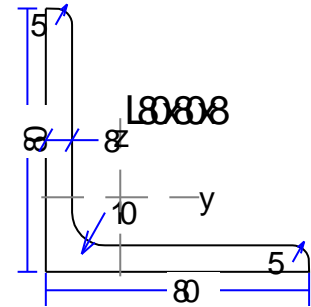
$H = 7.650 \text{ m}$

$L_y = 7.500 \text{ m}$

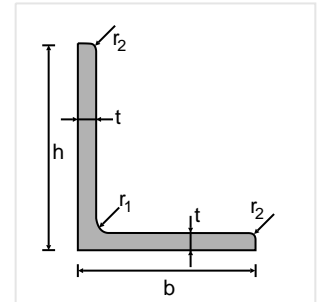
Load on bracing system, roof level $Q_{ed2} = 30.000 \text{ kN}$

1.4. Steel cross-section properties**Cross-section L80x80x8 -S 275 N/NL****Dimensions of cross section**

Depth of cross section	h=	80.00 mm
Width of cross section	b=	80.00 mm
Web depth	hw=	80.00 mm
Depth of straight portion of web	dw=	80.00 mm
Web thickness	tw=	8.00 mm
Flange thickness	tf=	8.00 mm
Radius of root fillet	r=	10.00 mm
Mass	=	9.66 Kg/m

**Properties of cross section**

Area	A=	1230 mm ²	
Second moment of area	Iy=	0.723x10 ⁶ mm ⁴	Iz= 0.723x10 ⁶ mm ⁴
Second moment of area	Iu=	1.150x10 ⁶ mm ⁴	Iv= 0.296x10 ⁶ mm ⁴
Section modulus	Wy=	12.600x10 ³ mm ³	Wz=12.600x10 ³ mm ³
Plastic section modulus	Wpy=	54.272x10 ³ mm ³	Wpz=26.624x10 ³ mm ³
Radius of gyration	iy=	24.2 mm	iz= 24.2 mm
Radius of gyration	iu=	30.6 mm	iv= 15.5 mm
Shear area	Avz=	662 mm ²	Avy= 640 mm ²
Torsional constant	It=	0.037x10 ⁶ mm ⁴	ip= 34 mm
Torsional modulus	Wt=	4.673x10 ³ mm ³	
Warping constant	Iw=	0.442x10 ⁹ mm ⁶	

**1.5. Horizontal loadings****Vertical (wall) braced girder**

The vertical brace system is loaded with point horizontal load Qed2=30.00kN

at the top of the column h= 7.650m.

Length of braced girder members 10.713 m, inclination $\varphi=45.57^\circ$, $\tan\varphi=7.650/7.500=1.020$

Forces in bracing members

Tension Nted2= 1.00x30.0/cos45.57= 42.9 kN

Compression on columns Nced2=42.9xsin45.57=30.6kN

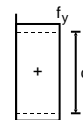
1.6. Classification of steel cross-section, Compression Nc (Bracing member)

(EN1993-1-1, §5.5)

$$h/t=80.0/8.0=10.00, (b+h)/2t=(80.0+80.0)/(2 \times 8.0)=10.00$$

$$S 275 N/NL, t= 8.0 \leq 40 \text{ mm}, f_y=275 \text{ N/mm}^2, \varepsilon=(235/275)^{0.5}=0.92$$

$$h/t=10.00 \leq 15\varepsilon=13.80, (b+h)/2t=10.00 \leq 11.5\varepsilon=10.58$$



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

1.7. Resistance of cross-section, Bracing member

(EN1993-1-1, §6.2)

Ultimate Limit State, Verification for tension

(EN1993-1-1, §6.2.3)

Nt.ed= 42.90 kN

$$\text{Tension Resistance } N_{plrd} = A \cdot f_y / \gamma_{M0} = [10^{-3}] \times 1230 \times 275 / 1.00 = 338.25 \text{ kN}$$

$$N_{t,ed} = 42.90 \text{ kN} < 338.25 \text{ kN} = N_{t,rd} = N_{plrd}, \text{ Is verified}$$

$$N_{t,ed}/N_{t,rd} = 42.90/338.25 = 0.127 < 1$$

1.8. Bolts connecting braces

<u>Bolt connection data, Bracing member</u>		(EN1993-1-8)
Type of connection	End-plate connection, non-preloaded bolts	
Category of connection	Category A: Bearing type	(EC3-1-8 §3.4.1)
Connected members	Thickness t=8 mm	
Bolts	M24, Strength grade 8.8	
Bolt diameter	d = 24 mm	
Diameter of holes	do = 26 mm	
Nominal area	$\pi d^2/4 = \pi \times 24^2/4 = 452.4 \text{ mm}^2$	
Tensile stress area	As = 452.4 mm ²	
Bolt strength grade	8.8, fyb=640N/mm ² , fub=800N/mm ²	(EC3-1-8 §3.1.1)

Shear resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)

$$F_{v,rd} = \alpha_v \cdot f_{ub} \cdot A_s / \gamma_{M2} = [10^{-3}] \times 0.60 \times 800 \times 452.4 / 1.25 = 173.7 \text{ kN}$$

Bearing resistance of bolts (EN1993-1-8, §3.6.1 Tab.3.4)

$$F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2}$$

t=8.0mm, d=24mm, do=26mm, e1=50mm, e2=50mm, p1=100mm, fub=800kN/mm², fu=430kN/mm²,
 $\alpha_b = \min[f_{ub}/f_u, 1, e_1/3d_o, p_1/3d_o - 1/4] = \min[800/430, 1, 50/(3 \times 26), 100/(3 \times 26) - 0.25] = 0.64$
 $k_1 = \min[2.8e_2/d_o - 1.7, 2.5] = \min[2.8 \times 50/26 - 1.7, 2.5] = 2.50$
 $F_{b,rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t / \gamma_{M2} = [10^{-3}] \times 2.50 \times 0.64 \times 430 \times 24 \times 8.0 / 1.25 = 105.8 \text{ kN}$

Necessary bolts per brace 42.9/105.8= 1 M24, Grade 8.8

IŠVADOS

1. Pagal pirmąją ribinę būseną – ULS:

- pjūviai parenkami 85 % laikomosios galios ribose;
- armatūra mažesnė už didžiausią leistiną, tai yra apie 1,6%;
- sekcijos, kaip ir sekcijų elementai yra stabilūs lenkimo ir vietinio sulenkimo metu;
- laikymo momentas kombinacijose viršija apvertimo momentą daugiau nei 1,5 karto;
- plieninių ir gelžbetoninių elementų įtempiai neviršija leistinų ribų.

2. Pagal antrąją ribinę būseną – SLS:

- didžiausi vertikalūs judesiai priimtinos ribose ($L/f < 1/250$);
- didžiausi horizontalūs judesiai priimtinos ribose ($h/f < 1/150$).

1. According to the first limit state – ULS:

- selected sections bear about 85% of bearing capacity;
- reinforcement is less than the maximum permissible, which is about 1,6%;
- sections, as well as section elements, are both stable during buckling and local buckling;
- the holding moment in combinations exceeds the overturning moment by more than 1,5 times;
- stresses in steel and reinforced concrete elements are within acceptable limits;

2. According to the second limit state – SLS

- maximum vertical deformations not exceed acceptable limits ($L/f < 1/250$);
- maximum horizontal deformations not exceed acceptable limits ($h/f < 1/150$).

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PROJEKTINIŲ INŽINERINIŲ GEOLOGINIŲ TYRIMŲ ATASKAITA

UŽSAKOVAS: UAB Synergy Solutions

OBJEKTAS: projektuojamas sandėliavimo paskirties pastatas ir stoginės Lakūnų g. 3, Šiaulių m.

Registracijos Lietuvos geologijos tarnyboje Nr.:

Direktorė Rūta Pranevičiūtė



2022 m. Rugsėjis, Šiauliai

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Tekstiniai priedai

1. Inžinerinių geologinių tyrimų techninė užduotis;
2. Leidimas tirti žemės gelmes;
3. Tyrimų taškų koordinacių ir altitudžių žiniaraštis;
4. Geotechninių bandymų (CPT) įrangos metrologinė patikra;
5. Grunto fizinių savybių laboratorinių tyrimų protokolas;
6. Ataskaitoje naudoti sutrumpinimai, dydžiai, žymenys ir matavimo vienetai.

Grafiniai priedai

1. Tyrimų vietos padėties vietovėje schema;
2. Tiriamojo ploto padėties vietovėje ir tyrimo vietų išdėstymo planas;
3. Gręžinių stulpeliai su geotechninio bandymo CPT kreivėmis;
4. Geologinis pjūvis.

AIŠKINAMASIS RAŠTAS

IVADAS

UAB „IGEO“ įmonė (leidimas tirti žemės gelmes 2020-04-14 Nr. 1764351), pagal su Užsakovu (UAB Synergy Solutions) suderintą techninę užduotį (1 priedas), atliko projektuojamo sandėliavimo pastato ir stoginių Lakūnų g. 3, Šiaulių m. projektinius inžinerinius geologinius tyrimus. Tyrimų tikslas – gauti objektyvią informaciją apie geologinę sklypo, kuriame yra projektuojamas ypatingas pastatas, sandarą, sudaryti pagrindų skaičiavimo schemas, išskiriant inžinerinius geologinius geotechninius sluoksnius (IGS) ir nustatyti jų būdingąsias vertes. Pagal darbų techninę užduotį (1 priedas), teritorijoje turi būti atlikti antros geotechninės kategorijos inžineriniai geologiniai tyrimai. Tyrimų vietos pagal LKS-94 koordinatas nurodytos 3 priede. Lauko ir duomenų apdorojimo darbams vadovavo Rūta Pranevičiūtė.

Lauko darbai atlikti š. m. rugpjūčio mėn. 18 dieną. Teritorijos inžinerinės geologinės sąlygos tirtos 8-uose taškuose (žr. 2 grafinį priedą). Visuose tyrimų taškuose buvo išgręžti užsakovo nurodyto gylio tiriamieji gręžiniai (žr. 3 grafinį priedą). Visose vietose, be gręžimo darbų, atliktas statinis bandymas kūginiu penetrometru (CPT) (žr. 3 grafinį priedą) ir nustatytos grunto fizinės savybės (žr. 3 lentelę).

Bandymas kūginiu penetrometru (CPT)

CPT bandymo metu, tiesiogiai matuojami ir 1 cm ilgio intervalais fiksuojami parametrai: kūginis stipris, šoninės trinties stipris ir zondavimo ilgis. Zondavimo įrangos techniniai duomenys ir kalibravimo rezultatai pateikti 4 priede. Matavimams naudojama sistema, sudaryta iš:

a) CPTU zondo Nr. GL0370 (kūgio pagrindo plotas 10 cm^2 , kūgio kampas 60° , kūgio skersmuo 35,7 mm, šoninės trinties movos plotas 150 cm^2 , maksimali apkrova kūgiui 50 kN, maksimali apkrova šoninei trinčiai 15 kN, maksimali apkrova vandens poriniam slėgiui 20 bar, leistina visų daviklių perkrova 150 %), kurio metrologinė patikra pateikta 4 tekstiniam priede;

b) zondavimo štangų (skersmuo 32 mm, ilgis 1 m);

c) duomenų registratoriaus (gylmatis, duomenų interfeisas, zondavimo kabelis 30 m, lauko kompiuteris Panasonic CF – 19;

d) programinės įrangos („Geologiniai matavimai“).

Bandymai atlikti pagal LST EN ISO 22476 – 1 reikalavimus [6].

Gręžimo darbai, pirminė gruntų klasifikacija ir bandinių paėmimo principai

Gręžiniai išgręžti sraigtniu būdu 115 mm skersmens grąžtais. Gręžimas vykdytas 1,5 m grąžtais, kaskart iškeliant po vieną grąžtą.

Gręžinio kernas tyrimų vietoje vizualiai apžiūrėtas ir atlikta pirminė grunto atpažintis nustatant pagrindinę frakciją bei aprašant antrines frakcijas. Tokiu būdu gruntas priskirtas vienam iš šešių tipų, dažniausiai nusakančių pagrindines geotechnines savybes: rieduliai, gargždas, žvyras, smėlis, dulkis ir molis. Jeigu gruntas susideda iš organinių medžiagų, jis priskiriamas organiniam gruntui. Piltinis ar perkastas gruntas priskiriamas dirbtiniams gruntams.

Laboratoriniai tyrimai

Grunto bandinių laboratorinius tyrimus atliko Klaipėdos Universiteto Jūros Tyrimų Instituto laborantas j.m.d. Mindaugas Kazbaris. Bandymų rezultatų suvestinė lentelė pateikta 5 tekstiniam priede. Atsižvelgiant į pirminės atpažinties metu nustatytą grunto tipą, parinkti atitinkami tyrimų metodai tiksliam gruntų klasifikavimui į klases:

- *granulimetrinė sudėtis* (žvyras, smėlis, dulksis ir molis). Labai rupiems gruntams neatliekama;
- *gamtinis tankis, kietųjų dalelių tankis* (molis);
- *gamtinis, takumo ir plastingumo drėgnis* (molis);
- *filtracijos koeficientas* (žvyras ir smėlis).

Ataskaitos paruošimas

Tyrimų ataskaita parengta vadovaujantis STR 1.04.02:2011 „Inžineriniai geologiniai ir geotechniniai tyrimai“ [1] ir Lietuvos geologijos tarnybos parengtų projektinių inžinerinių geologinių ir geotechninių tyrimų rekomendacijų [3] reikalavimais. Naudota programinė įranga nanoCAD 5.0, Microsoft Office (Word, Exel). Žemiau aprašoma geologinio modelio sudarymo metodika.

1. BENDRIEJI DUOMENYS APIE STATYBOS TERITORIJĄGamtinės sąlygos

Projektuojamas sandėliavimo paskirties pastatas ir stoginės yra Lakūnų g. 3, Šiaulių m. Geomorfologiniu požiūriu tyrinėta teritorija priklauso Žemaičių - Kuršo srityje esančiam Rytų Žemaičių plynaukštės rajono Šiaulių kalvoto moreninio gūbrio mikrorajonui. Reljefo absoliutiniai aukščiausi tyrimų vietose siekia 129,84 – 130,80 m.

Tyrimų plote yra paplitę trijų genetinių tipų nuogulos. Tai augalinis sluoksnis (pdIV), paskutinio apledėjimo Baltijos stadijos fliuvioglacialiniai (fIIIbl) dariniai ir paskutinio apledėjimo Baltijos stadijos kraštiniai glacialiniai (gtIIIbl) dariniai. Pagal žemės paviršiuje atsidengiančių skirtingų genetinių nuogulų tipų skaičių (3–4) tyrimo ploto geomorfologinės sąlygos yra vidutinės (1 lentelė).

Žemės paviršiaus nuolydis neviršija 10°. Sklype erozinių, termokarstinių, sufozinių ir kitų neigiamų reljefo formų nėra. Atstumas iki nepastovių šlaitų ir eroduojamų krantų didesnis nei 100 m. Pagal šiuos požymius sklypo geomorfologinės sąlygos yra paprastos.

Klimatas (pagal Meteo duomenis)

Sklypas yra vidutinių platumų klimato zonoje ir priklauso Atlanto kontinentinės miškų srities pietvakarinio posričio Vidurio Žemumos rajono Mūšos – Nevėžio parajoniui. Vidutinė metinė oro temperatūra 6,5–7,0 °C. Sausis yra šalčiausias mėnuo, kurio vidutinė oro temperatūra -3,6– -3,1. Absoliutus temperatūros minimumas -33,6 °C. Kritulių kiekis per metus 560 – 700 mm. Laikotarpio su sniego danga trukmė 75– 90 dienų. Svarbiausi procesai, sąlygojantys tarprajoninius klimato skirtumus yra adiabatinis oro leidimasis nuo gretimų aukštumų ir blogos vandens nuotėkio plokščių paviršiumi sąlygos, dirvožemių perdrėkinimas.

1 lentelė. Statybos sklypo inžinerinių geologinių sąlygų sudėtingumas pagal [1]

1. Geomorfologinės	paprastos	vidutinės	sudėtingos
Reljefo genetinių tipų skaičius	1–2	3–4	>4
Technogeniniai reljefo pokyčiai	nėra	nedideli pokyčiai	labai pakeistas reljefas

Žemės paviršiaus nuolydžiai, ⁰	<10	10–25	>25
Erozinės, termokarstinės, sufozinės ir kitos neigiamos reljefo formos	nėra	yra nedaug ir mažų	yra daug ir didelių
Atstumas iki nepastovių šlaitų ir eroduojamų krantų, m	>100	100–50	<50
2. Geologinės	paprastos	vidutinės	sudėtingos
Podirvio sluoksnio (įžemio) genezė	ikikvarterinės uolienos, pagrindinė morena, fluvio-glacialiniai, senojo aliuvio, vagos aliuvio dariniai	hipergeninė morena, limnoglacialiniai, jūriniai, eoliniai, aliuviniai dariniai	sukarstėję ikikvarterinės uolienos, kraštiniai dariniai, senvagių aliuvio, biogeninės ir technogeninės nuogulos
Įžemio grunta	Žvyras, smėlis, moreninis molis ir dulkis (jų atmainos), uoliena	molis, juostinis molis, aliuvinis molis ir dulkis, įdūlėjusi uoliena	dumblas, sapropelis, durpės, dribsmėlis, technogeniniai dariniai
Skirtingų litologinių tipų sluoksnių skaičius	<3	3–5	>5
Ikikvarterinių sluoksnių uolienos	nėra	gali būti	yra sukarstėjusių ar sudūlėjusių
Sąlygiškai silpni sluoksniai	nėra	slūgso viršutinėje pjūvio dalyje ir nedidelio storio	slūgso giliau ir didelio storio
Supiltinės, suplautinės ar perkastos storymės	nėra	planingai suformuotos, sutankintos ar sutankėjusios	betvarkės, nesutankintos ar nesutankėjusios
Sluoksniuotumo pobūdis	horizontalūs ir subhorizontalūs ištisiniai sluoksniai	įkypni nevientisi sluoksniai ir lęšiai	sudėtingos konfigūracijos sluoksniai, lęšiai, lustai
Palaidotos paleoreljefo formos	nėra	gali būti	yra palaidotų paleojrėžių
3. Hidrogeologinės	paprastos	vidutinės	sudėtingos
Gruntinio vandens slūgsojimo gylis, m	>3	2–3	<2
Galima požeminio vandens lygio kitimo amplitudė, m	<0,5	0,5–1	>1
Vandeningojo sluoksnio išplitimas	vienodas, ištisinis	diskretus, nevienodo storio	komplikuotas, sudėtingas
Duomenys apie požeminio vandens korozinį agresyvumą	vanduo neagresyvus	nustatytas silpnas agresyvumas	vanduo agresyvus
Drenažo įrenginiai ar vandens turintys vamzdynai	nėra	yra veikiantys, hidrauliškai išbandyti	neaišku arba yra netvarkingi ar neveikia
Sluoksnio vandens laidumas	vandenspara	nedidelis	didelis ar labai nevienodas
Spūdinio vandeningojo sluoksnio slūgsojimo gylis ir hidrostatinis spūdis	spūdinio sluoksnio nėra	gylis per 20 m, pjezometrinis lygis giliau nei 2 m nuo žemės paviršiaus	gylis mažesnis nei 20 m, pjezometrinis lygis mažesniame nei 2 m gylyje
Gruntinio vandens sąveika su paviršiniais vandenimis	sąveikos nėra	sąveika silpna	yra hidraulinė sąveika
Požeminio vandens iškrovis zona, šaltiniai, versmės	nėra	gretimose vietovėse	pačiame sklype
4. Geodinaminės	paprastos	vidutinės	sudėtingos
Seismingumas pagal EMS 98	iki 3 balų	iki 6 balų	daugiau kaip 6 balai
Karstinio proceso apraiškos ir reiškiniai	nėra	nėra	yra
Nuošliaužos, kitos šlaitų stabilumo pažeidos	nėra	stabilizuotos	aktyvios
Kiti geodinaminiai procesai ir reiškiniai	nėra	lokalūs	intensyvūs
Statinių deformacijos	nėra	gretimose vietovėse	pačiame sklype

Pastaba: paryškinta ta lentelės grafa, kuri tiksliausiai apibūdina sklypo sąlygas.

2. GEOLOGINĖ SANDARA

Sklypo geologinę sandarą iki 12,03 m gylio sudaro: augalinis sluoksnis (pdIV), viršutinio Pleistoceno Baltijos posvītės fluvio-glacialinės (fIIbI) nuogulos ir viršutinio Pleistoceno Baltijos posvītės kraštinės glacialinės (gtIIbI) nuogulos.

Augalinis sluoksni (pdIV) sudaro:

- Juodžemis (Or). Komplexas išskirtas visuose tyrimų taškuose. Jo storis siekia 0,10 iki 0,50 m.

Viršutinio Pleistoceno Baltijos posvitės fluvio-glacialinės nuogulos (fIIIbl) sudaro:

- Dulkingas smėlis, pilkai rudas, šlapias, vidutinio tankumo (siSa). Komplexas išskirtas visuose tyrimų taškuose. Jo storis siekia nuo 0,5 iki 3,33 m.
- Dulkingas smėlis, pilkai rudas, vandeningas, tankus (siSa). Komplexas išskirtas tyrimų taškuose Nr. 1, 7. Jo storis siekia nuo 1,50 iki 4,96 m.
- Dulkingas smėlis, rudai pilkas, šlapias, tankus (siSa). Komplexas išskirtas tyrimų taškuose Nr. 2, 4, 5, 6. Jo storis siekia nuo 5,00 iki 6,56 m.
- Blogai išrūšiuotas mažai dulkingas-molingas smėlis, pilkai rudas, šlapias, tankus (SaFP). Komplexas išskirtas tyrimų taške Nr. 7. Jo storis siekia 4,00 m.

Viršutinio Pleistoceno Baltijos posvitės kraštines glacialines nuogulas (gtIIIbl) sudaro:

- Mažo plastiškumo molis ir dulkis, pilkai rudas, vandeningas, moreninis, stiprus (CIL-SiL). Komplexas išskirtas tyrimų taškuose Nr. 1, 2, 3, 4, 6, 7. Jo storis siekia nuo 0,22 iki 3,19 m.
- Smėlingas mažo plastiškumo dulkis, pilkai rudas, šlapias, moreninis, labai stiprus (saSiL). Komplexas išskirtas visuose tyrimų taškuose. Jo storis siekia nuo 0,12 iki 5,64 m.
- Mažo plastiškumo molis, pilkai rudas, vandeningas, moreninis, vidutinio stiprumo (CIL). Komplexas išskirtas tyrimų taškuose Nr. 5, 8. Jo storis siekia nuo 2,11 iki 2,58 m.

3. HIDROGEOLOGINĖS SĄLYGOS

Gruntinis vanduo gręžimo metu pasiektas apie 0,5 m nuo žemės paviršiaus. Gruntinio vandens lygis gali kisti >1,0 m nuo išmatuoto lygio lauko darbų metu, kadangi sausuojų metu laikotarpiu gruntinio vandens lygis pažemės, o drėgnuojų – pakils.

Požeminio vandens iškrovos zonų, šaltinių, versmių nėra. Hidraulinė sąveika tarp gruntinio ir paviršinio vandens tyrimų sklype yra. Vandeningojo sluoksnio išplitimas yra nevienodo storio, o sluoksnių laidumas yra nedidelis. Tyrimų sklype nėra veikiančių, hidrauliškai išbandytų vandens vamzdinių. Spūdinio vandeningojo sluoksnio slūgsojimo gylys per 20 m, o pjezometrinis lygis yra giliau nei 2 m nuo žemės paviršiaus. Pagal hidrogeologinių požymių visumą tirtos teritorijos hidrogeologinės sąlygos turėtų būti apibrėžiamos kaip vidutinės.

4. GRUNTŲ SUDĖTIS IR INŽINERINIAI GEOLOGINIAI SLUOKSNIAI

Pagal tyrimų medžiagą išskirti 8 inžineriniai geologiniai sluoksniai (IGS), kurių aprašymai pateikti 2 lentelėje.

2 lentelė. IGS geologinis aprašymas

IGS Nr.	Sluoksnio geologinis aprašymas (pagal LST EN ISO 14688-1)
1	Juodžemis (Or). Komplexas išskirtas visuose tyrimų taškuose. Jo storis siekia 0,10 iki 0,50 m.
2	Dulkingas smėlis, pilkai rudas, šlapias, vidutinio tankumo (siSa). Komplexas išskirtas visuose tyrimų taškuose. Jo storis siekia nuo 0,5 iki 3,33 m.
3	Dulkingas smėlis, pilkai rudas, vandeningas, tankus (siSa). Komplexas išskirtas tyrimų taškuose Nr. 1, 7. Jo storis siekia nuo 1,50 iki 4,96 m.
4	Mažo plastiškumo molis ir dulkis, pilkai rudas, vandeningas, moreninis, stiprus (CIL-SiL). Komplexas

IGS Nr.	Sluoksnio geologinis aprašymas (pagal LST EN ISO 14688-1)
	išskirtas tyrimų taškuose Nr. 1, 2, 3, 4, 6, 7. Jo storis siekia nuo 0,22 iki 3,19 m.
5	Smėlingas mažo plastiškumo dulkis, pilkai rudas, šlapias, moreninis, labai stiprus (saSiL). Komplexas išskirtas visuose tyrimų taškuose. Jo storis siekia nuo 0,12 iki 5,64 m.
6	Dulkingas smėlis, rudai pilkas, šlapias, tankus (siSa). Komplexas išskirtas tyrimų taškuose Nr. 2, 4, 5, 6. Jo storis siekia nuo 5,00 iki 6,56 m.
7	Mažo plastiškumo molis, pilkai rudas, vandeningas, moreninis, vidutinio stiprumo (CIL). Komplexas išskirtas tyrimų taškuose Nr. 5, 8. Jo storis siekia nuo 2,11 iki 2,58 m.
8	Blogai išrūšiuotas mažai dulkingas-molingas smėlis, pilkai rudas, šlapias, tankus (saFP). Komplexas išskirtas tyrimų taške Nr. 7. Jo storis siekia 4,00 m.

5. GRUNTŲ FIZIKINĖS IR MECHANINĖS SAVYBĖS

Grunto CPT bandymai buvo atlikti visuose gręžinių vietose (žr. 3 grafinių priedą). Išskirtų inžinerinių geologinių sluoksnių (IGS) geotechninio zondavimo vertės, pagrindiniai statistiniai rodikliai ir fizikinių bei mechaninių savybių suvestinės vertės pateiktos 3 lentelėje.

3 lentelė. Gruntų geotechninio zondavimo verčių, pagrindinių statistinių rodiklių, fizikinių ir mechaninių savybių verčių suvestinė lentelė.

Geologinis intervalas	IGS Nr.	Grunto pavadinimas pagal ISO 14688	Skilimo stipris q_0 , MPa	Suspaustimo stipris f_0 , kPa	Deformacijos modulis, E_s , MPa	Gamtinio tankio ρ_s , Mg/m ³	Sauso grunto tankio ρ_d , Mg/m ³	Kietų dalelių (masės) tankio ρ_s , Mg/m ³	Gamtinio drėgnumo w , %	Takomo drėgnumo w_L , %	Plastinio drėgnumo w_p , %	Plastinio rodiklio I_p , %	Takomo rodiklio I_L , %
pdIV	1	Or	0.66	1.71	Netiesiniai parametrai, pagrindiniai								
fIIIb	2	siSa	7.02	43.47	31.12	1.87	1.70	2.67	10.18	21.80	-	0.00	-
	3	siSa	13.78	138.62	50.13	1.88	1.60	2.67	17.44	22.15	-	0.00	-
gtIIIb	4	CIL-siL	8.06	79.31	30.60	2.13	1.84	2.68	16.24	19.58	14.75	4.83	0.31
	5	saSiL	13.88	121.02	158.89	2.03	1.74	2.68	14.98	19.92	16.86	3.94	-0.62
fIIIb	6	siSa	12.06	145.78	45.69	1.88	1.66	2.67	13.60	23.55	-	0.00	-
gtIIIb	7	CIL	2.38	47.88	23.80	2.15	1.82	2.69	18.28	22.05	12.65	10.39	0.35
fIIIb	8	saFP	13.42	156.24	49.39	1.78	1.56	2.66	14.27	-	-	-	-

6. GEOLOGINIAI PROCESAI IR REIŠKINIAI

Iš šiuolaikinių fizinių ir geologinių procesų, kurie galėtų turėti neigiamos įtakos įrengiant ir eksploatuojant statinius, nenustatyta. Pagal karsto sufozijos pavojingumą, teritorija priskiriama nepavojingai.

7. IŠVADOS IR REKOMENDACIJOS

1. Projektuojamas sandėliavimo paskirties pastatas ir stokinės yra Lakūnų g. 3, Šiaulių m. Geomorfologiniu požiūriu tyrinėta teritorija priklauso Žemaičių - Kuršo srityje esančiam Rytų Žemaičių plynaukštės rajono Šiaulių kalvoto moreninio gūbrio mikrorajonui.
2. Reljefo absoliutiniai aukščiai tyrimų vietose siekia 129,84 – 130,80 m.
3. Pagal karsto sufozijos pavojingumą, teritorija priskiriama nepavojingai.
4. Sklypo geologinę sandarą iki 12,03 m gylio sudaro: augalinis sluoksnis (pdIV), Pleistoceno Baltijos posvitės fluvioglacialinės (fIIIb) nuosėdos. ir Pleistoceno Baltijos posvitės kraštinės glacialinės (gtIIIb) nuosėdos.
5. Gruntinis vanduo gręžimo metu pasiektas apie 0,5 m. nuo žemės paviršiaus.
6. Gruntinio vandens lygis gali kisti >1,0 m nuo išmatuoto lygio lauko darbų metu, kadangi sausuoju metų laikotarpiu gruntinio vandens lygis pažemės, o drėgnuuoju – pakils.

7. Sklypo geologiniame modelyje išskirti 8 inžineriniai geologiniai sluoksniai (IGS), kurių slūgsojimo sąlygos parodytos gręžinių litologiniuose stulpeliuose (3 grafinis priedas).
8. Apskaičiuotos IGS gruntų fizikinių mechaninių savybių būdingosios vertės pateiktos ataskaitos 7 skyriuje (3 lentelė).
9. Statybos sklypo hidrogeologinės sąlygos yra vidutinės, o geomorfologinės, geologinės ir geodinaminės – paprastos.
10. Statybos metu pastebėjus, kad pateiktas geologinis modelis neatitinka faktinės situacijos, būtina apie tai informuoti rangovą.

LITERATŪROS SĄRAŠAS

Teisės aktai ir norminiai dokumentai

1. Statybos techninis reglamentas STR 1.04.02:2011 „Inžineriniai geologiniai ir geotechniniai tyrimai“. Valstybės žinios, 2012-01-07, Nr. 5-144.
2. Statybos techninis reglamentas STR 1.01.03:2017 „Statinių klasifikavimas“. Teisės aktų registras, 2016-11-21, Nr. 27168.
3. Projektinių inžinerinių geologinių ir geotechninių tyrimų rekomendacijos. Teisės aktų registras, 2015-11-16, Nr. 18162.

Standartai

4. LST EN ISO 14688-1. Geotechniniai tyrinėjimai ir bandymai. Gruntų atpažintis ir klasifikavimas. 1 dalis. Atpažintis ir aprašymas.
5. LST EN ISO 14688-2. Geotechniniai tyrinėjimai ir bandymai. Gruntų atpažintis ir klasifikavimas. 2 dalis. Klasifikavimo principai.
6. LST EN ISO 22476-1. Geotechniniai tyrinėjimai ir bandymai. Lauko bandymai. 1 dalis. Įspaudimo bandymas, naudojant elektrinį ir pjezoelektrinį kūgį.
7. LST EN 1997-2. Eurokodas 7. Geotechninis projektavimas. 2 dalis. Pagrindo tyrinėjimai ir bandymai.

Interneto adresai

8. www.lgt.lt (ŽGR, GEOLIS informacija)
9. www.meteo.lt (klimato duomenys)
10. www.maps.lt (interneto žemėlapių informacija)
11. www.geoportal.lt (kartografiniai duomenys)

TEKSTINIAI PRIEDAI

UAB Synergy Solutions

TECHININĖ UŽDUOTIS

2022-06-22

Dokumento data

Dokumento registracijos numeris

IGG tyrimų stadija (pabraukti): žvalgybiniai, projektiniai, papildomi, kontroliniai.

Tyrimų objekto pavadinimas: sandėliavimo paskirties ir kitos paskirties statiniai – stoginės

Tyrimų objekto adresas (savivaldybė, seniūnija, gyvenvietė, gatvė, statinio numeris):

Lakūnų g. 3, Šiaulių m.

Užsakovo duomenys (pavadinimas (v. pavardė), adresas, telefono ryšio Nr., el. pašto adresas):

Infrastruktūros valdymo agentūra, įm. k. 188743887, atstovaujanti UAB Synergy Solutions,

įm.k.302781077, Daugėlišio g. 32-206, Vilnius, info@ss-exp.com, tel. +37061260550

Projektuotojo duomenys (pavadinimas (v. pavardė), adresas, telefono ryšio Nr., el. pašto adresas)

UAB Synergy Solutions, įm. k. 302781077 Daugėlišio g. 32-206, Vilnius 09300, info@ss-exp.com,
t. +370 612 60550

Statybos rūšis (pabraukti): nauja statyba, rekonstrukcija, kapitalinis remontas, kita

Statinio paskirtis: kita

Statinio kategorija (pabraukti): ypatingasis, neypatingasis, nesudėtingasis

Nekilnojamųjų kultūros vertybių registro kodas (jei yra): į projektuojamą zoną nepatenka.....

Geotechninė kategorija (projektiniuose tyrimuose) (pabraukti): pirma, antra, trečia

Duomenys apie statinio parametrus (ilgis, plotis, aukštis, gylis, plotas): sandėliavimo – 25x19m, h-
9,20 m., stoginė – 32x18,6 m, h – 7,45 m., stoginė 31x15,6 m, h – 7,45 m.

Perduodamos į pagrindą apkrovos ir jų intensyvumas poliniai arba giliaji pamatai; didžiausia
vertikali apkrova, veikianti kolonos pamatą, F – 1000 kN.

Statybvietės centro koordinatės (LKS-94): 6195261, 461515

Tyrimų ploto ribų koordinatės:

Numeris	X	Y
1	6195278	461562
2	6195235	461604
3	6195253	461623
4	6195241	461634
5	6195261	461656
6	6195313	461600

Papildomai nustatomi geotechniniai parametrai ir kiti reikalavimai:

1.

Sąrašas normatyvinių dokumentų, kuriais vadovaujantis atliekami tyrimai:

STR 1.04.02:2011 „Inžineriniai geologiniai ir geotechniniai tyrimai“

Anksčiau sklype atlikti geologiniai tyrimai: nėra duomenų

Užsakovas UAB Synergy Solutions, direktorė Ieva Čirūnaitė.....
vardas, pavardė, parašas, data

2022-06-22

Projekto vadovas Tomas Kazlauskas
vardas, pavardė, parašas, data

2022-06-22

Tyrimų vadovas (užduotį gavau): UAB „IGEO“ direktorė Rūta Pranavičiūtė

2022-06-22



Lietuvos geologijos tarnybos prie
Aplinkos ministerijos direktoriaus
2020 m. gegužės 14 d. įsakymo Nr. 1-
priedas



**LIETUVOS GEOLOGIJOS TARNYBA
PRIE APLINKOS MINISTERIJOS**

LEIDIMAS

TIRTI ŽEMĖS GELMES

2020-04-14 Nr. 1764351
(data)

Vadovaujantis Lietuvos Respublikos žemės gelmių įstatymu, **l e i d ž i a m a :**

UAB Igeo

(kodas 300112034, buveinė Šiauliai, Tilžės g. 170-334)

nuo 2020-04-14
(leidimo įsigaliojimo data)

a t l i k t i :

inžinerinį geologinį (geotechninį) tyrimą.

Direktorius

A.V.

(parašas)

Giedrius Giparas
(vardas ir pavardė)

Tyrimų vietų geodezinių koordinačių LKS-94 ir altitudžių žiniaraštis

Eil. Nr.	Tyrimo vietos Nr.	LKS koordinačių sistema		Žemės paviršiaus altitudė, m abs.a.
		Y	X	
1	Gr./CPT-1	461576	6195279	129,84
2	Gr./CPT-2	461599	6195265	129,84
3	Gr./CPT-3	461617	6195255	130,07
4	Gr./CPT-4	461636	6195261	130,34
5	Gr./CPT-5	461653	6195261	130,43
6	Gr./CPT-6	461635	6195243	130,19
7	Gr./CPT-7	461600	6195242	130,80
8	Gr./CPT-8	461584	6195260	129,84

Metrologinė patikra

**KALIBRAVIMO LIUDIJIMAS Nr. VMC-KN-K-002701**

Užsakovas	UAB Igso, įm.k. 300112034
Kalibruojamas objektas	Tenozoondas CPT Nr. GL0370 Kūgio spaudimo jėgos matavimo ribos: (0 ... 100) kN (plotas 10 cm²; 100 kN atitinka 100 MPa) Šoninės trinties jėgos matavimo ribos: (0 ... 15) kN (plotas 150 cm²; 15 kN atitinka 1 MPa) Indikatorius GRL 1503
Objekto gavimo data	2021-08-19
Objekto būklė	MP suturi mechaninių ar kitokių požymių, visi įrešai siūlomi įkeitomi
Užsakovo pateikti dokumentai	-
Kalibravimo metodas	Kalibravimo procedūra KM M 2001 09 (2014-03-17)
Kalibravimo atliko	Kauno regiono laboratorija, E. Oželiškienės g. 25, LT-44254 Kaunas Tel. 8 5 233 3393. El. paštas kaunas@vmc.lt
Kalibravimo atlikimo vieta	Tauragė, Ganyklų g. 15
Aplinkos sąlygos	Aplinkos oro temperatūra 21,7 °C Santykinė drėgmė 44,6 %
Kalibravimo protokolo Nr., data Sieta	LIZ-63313-1-4 2021-08-19 Matavimai buvo atlikti su šiais kalibravimo būdu įrešiais etalonais: dinamometras Z4A/50 kN, Nr. 184930037 dinamometras C18/500 kN, Nr. 002874TY
Kalibravimo liudijimo išdavimo data	2021-08-19
Vyresysis inžinierius metrologas	Tadas Kleveckas
Vyresysis inžinierius metrologas	Tadas Kleveckas

KALIBRAVIMO LIUDIJIMAS Nr. VMC-KN-K-002701**KALIBRAVIMO REZULTATAI**

Tenzozondas CPT Nr. GL0370

Etalono apkrova, kN	Zondo rodinys, kN	Paklaida, kN	Patiesa, kN	Išplėstinė neapibrėžtis, kN
Šoninė trintis				
1,50	1,51	+0,01	-0,01	±0,46
3,00	3,02	+0,02	-0,02	±0,27
6,00	6,02	+0,02	-0,02	±0,21
9,00	9,02	+0,02	-0,02	±0,12
15,00	15,03	+0,03	-0,03	±0,07
Kūgis				
5,00	5,01	+0,01	-0,01	±0,17
10,00	10,02	+0,02	-0,02	±0,09
20,00	20,02	+0,02	-0,02	±0,05
30,00	30,02	+0,02	-0,02	±0,04
40,00	39,97	-0,03	+0,03	±0,02
50,00	49,68	-0,32	+0,32	±0,02
60,00	59,53	-0,47	+0,47	±0,09
70,00	69,38	-0,62	+0,62	±0,05

Išplėstinė neapibrėžtis apskaičiuota suminę standartinę neapibrėžtį paduginus iš aprėpties daugiklio $k=2$, kuris, esant normaliajam skirstiniui, apytikriai atitinka 95 % pasididėjimo lygmenį. Standartinė neapibrėžtis paskaičiuota pagal EA-4/02M.

Kalibravimo rezultatai susiję tik su kalibruojamu objektu.

Nurodytos vertės taikomos tenzozondo būklei kalibravimo metu.

Kalibravimo liudijimas gali būti dauginamas tik visas.

Vyresnysis inžinierius metrologas

Tadas Kleveckas



KLAIPĖDOS UNIVERSITETO JŪROS TYRIMŲ INSTITUTAS

Viešoji įstaiga, Herkaus Manto g. 84, 92294 Klaipėda, tel.: (8 46) 398 846, faks.: (8 46) 398 999, el. p. info@apc.ku.lt
Duomenys kaupiami ir saugomi Juridinių asmenų registre, kodas 211951150

Gruntų laboratorinių tyrimų protokolas Nr. TP-0851-2022

Data 2022-09-09

Užsakovas: UAB "Igeo", Vilniaus g. 274A, LT-76308 Šiauliai

Projektas: Sandėliavimo paskirties ir kitos paskirties inžineriniai statiniai- stoginės Lakūnų g. 3, Šiaulių m

Objektas: Gruntas

Gruntų pridavimo data: 2022-08-26

Grunto bandinių kiekis: 9

Tyrimai atlikti pagal:

* LST EN ISO 14688-1:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikavimas. 1 dalis. Identifikavimas ir aprašymas (ISO 14688-1:2017)

* LST EN ISO 14688-2:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikavimas. 2 dalis. Klasifikavimo principai (ISO 14688-2:2017)

* LST EN ISO 17892-1:2015 Geotechniniai tyrinėjimai ir bandymai. Laboratoriniai grunto bandymai. 1 dalis. Vandens kiekio nustatymas (ISO 17892-1:2014)

* LST EN ISO 17892-2:2015 Geotechniniai tyrinėjimai ir bandymai. Laboratoriniai grunto bandymai. 2 dalis. Tūrinio tankio nustatymas (ISO 17892-2:2014)

* LST EN ISO 17892-3:2016 Geotechniniai tyrinėjimai ir bandymai. Laboratoriniai grunto bandymai. 3 dalis. Dalelių tankio nustatymas (ISO 17892-3:2015)

* LST EN ISO 17892-4:2017 Geotechniniai tyrinėjimai ir bandymai. Laboratoriniai grunto bandymai. 4 dalis. Granulimetrinės sudėties nustatymas (ISO 17892-4:2016)

* LST EN ISO 17892-12:2018 Geotechniniai tyrinėjimai ir bandymai. Laboratoriniai grunto bandymai. 12 dalis. Takumo ir plastiškumo ribų nustatymas (ISO 17892-12:2018)

Protokolo priedai: 1. Laboratorinių tyrimų rezultatai - 1 lapas
2. Granulimetrinės sudėties kreivės - 2 lapai

Parengė:



LIETUVOS GEOLOGIJOS TARNYBA PRIE APLINKOS MINISTERIJOS

LEIDIMAS TIRTI ŽEMĖS GELMES

2021-06-16 Nr. 32

Vilnius

Viešajai įstaigai Klaipėdos universitetui

(juridinio asmens duomenys kaupiami ir saugomi Juridinių asmenų registre, kodas 211951150,
adresas Klaipėda, H. Manto g. 84)

leidžiama atlikti:

hidrogeologinį žemės gelmių kartografavimą,
geocheminį žemės gelmių kartografavimą,
ekogeologinį žemės gelmių kartografavimą,
inžinerinį geologinį žemės gelmių kartografavimą,
ekogeologinį tyrimą.

Direktorius
(pareigų pavadinimas)

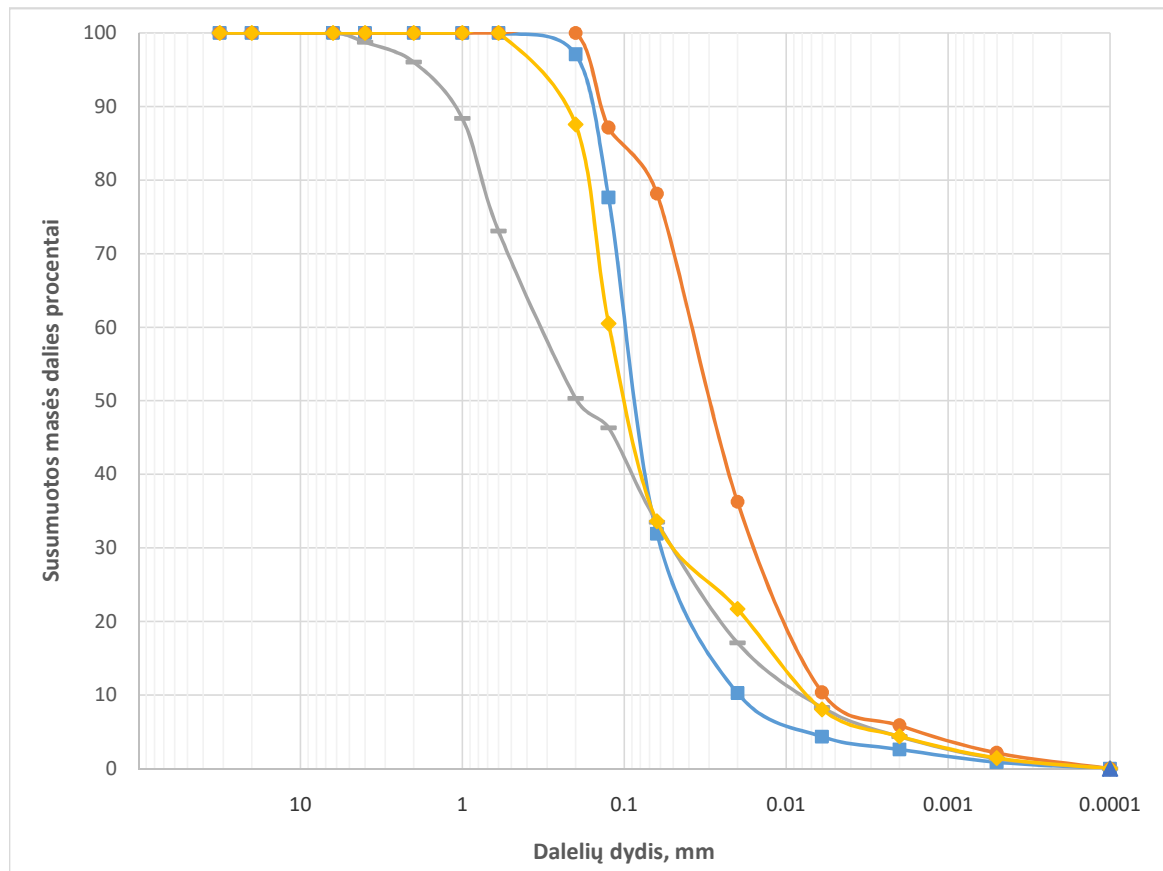
A.V.

(parašas)

Giedrius Giparas
(vardas ir pavardė)



KLAIPĖDOS UNIVERSITETO
JŪROS TYRIMŲ INSTITUTAS

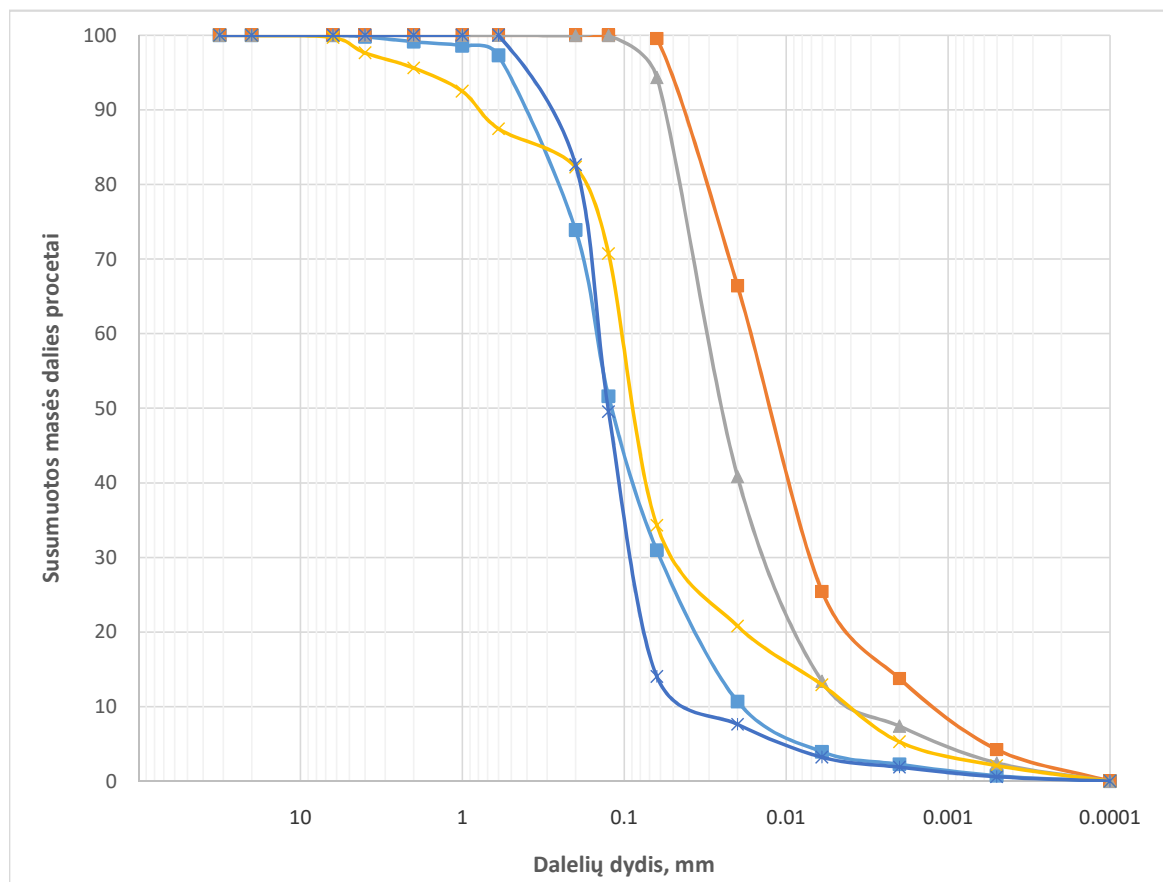


Sandėliavimo paskirties ir kitos paskirties inžineriniai statiniai- stoginės Lakūnų g. 3, Šiaulių m										
Eil. Nr.	Simbolis	Gr. Nr.	Pvz. Nr	Gylis, m	D10%, mm	D30%, mm	D50%, mm	D60%, mm	Cu	Cc
1	—■—	1	1	4,8-5,0	0.0190	0.0569	0.083	0.096	5.06	1.78
2	—●—	1	2	9,6-9,8	0.0055	0.0149	0.029	0.038	6.92	1.05
3	—+—	2	1	1,2-1,4	0.0076	0.0494	0.193	0.319	42.13	1.01
4	—◆—	2	2	7,4-7,6	0.0071	0.0445	0.096	0.123	17.32	2.25

Atliko: j.m.d. Mindaugas Kazbaris



KLAIPĖDOS UNIVERSITETO
JŪROS TYRIMŲ INSTITUTAS



Sandėliavimo paskirties ir kitos paskirties inžineriniai statiniai- stoginės Lakūnų g. 3, Šiaulių m										
Eil. Nr.	Simbolis	Gr. Nr.	Pvz. Nr.	Gylis, m	D10%, mm	D30%, mm	D50%, mm	D60%, mm	Cu	Cc
6		5	1	4,0-4,2	0.0176	0.0595	0.118	0.149	8.45	1.35
7		5	2	9,6-9,8	0.0011	0.0069	0.012	0.017	14.41	2.47
8		6	1	9,6-9,8	0.0032	0.0124	0.024	0.030	9.33	1.59
9		7	1	0,6-0,8	0.0039	0.0436	0.085	0.102	25.86	4.72
10		7	2	7,4-7,6	0.0305	0.0857	0.126	0.145	4.76	1.66

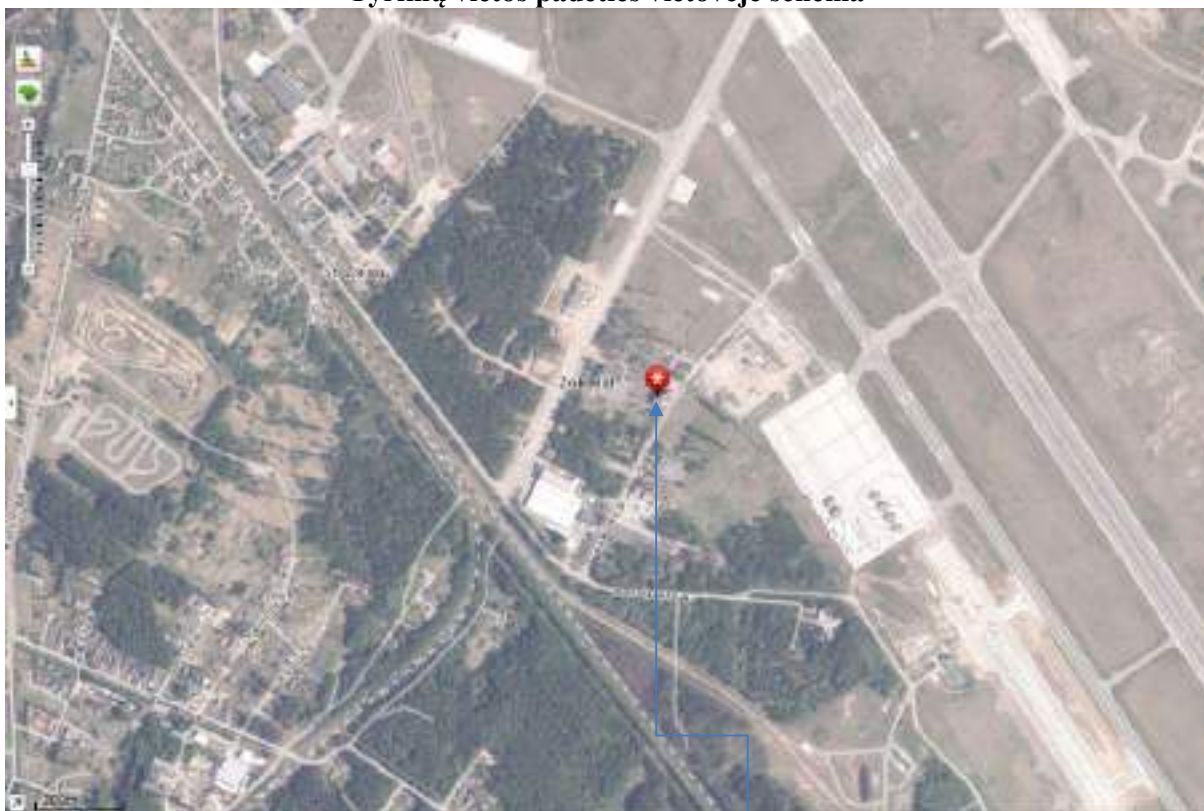
Atliko: j.m.d. Mindaugas Kazbaris

Ataskaitoje naudoti sutrumpinimai, dydžiai, žymenys ir matavimo vienetai

γ – savitasis sunkis, kN/m^3
 γ_w – vandens savitasis sunkis, kN/m^3
 ρ – gamtinis (masės) tankis, Mg /m^3
 ρ_s – kietų dalelių (masės) tankis, Mg /m^3
 e – poringumo koeficientas, vnt.d.
 w – gamtinis drėgnis, %
 w_L – takumo drėgnis, %
 w_p – plastingumo drėgnis, %
 I_p – plastingumo rodiklis, %
 I_L – takumo rodiklis, vnt.d.
 I_D – tankumo rodiklis, vnt.d.
 k – filtracijos koeficientas, m/d
 p_a – atmosferos slėgis, MPa
 σ_{v0} – efektyvus vertikalus įtempis, MPa
 g – laisvojo kritimo pagreitis, m/s^2
 E – Jungo modulis, MPa
 E_0 – deformacijų modulis (visuminės deformacijos modulis), MPa
 G_0 – šlyties modulis (mažų deformacijų zonai), MPa
 c_u – nedrenuotoji sankiba, kPa, MPa
 ϕ' – efektyviosios vidinės trinties kampas, laipsniai
 I_c – konsistencijos rodiklis, vnt.d.
 q_c – kūginis stipris, MPa
 q_t – koreguotas kūginis stipris, MPa
 Q_c – normalizuotas kūginis stipris, įvertinus vertikalų įtempį, vnt.d.
 Q_t – normalizuotas koreguotas kūginis stipris, įvertinus vertikalų įtempį, vnt.d.
 Q_{cn} – normalizuotas kūginis stipris, įvertinus vertikalų įtempį ir jo priklausomybę nuo grunto tipo, vnt. d.
 Q_{tn} – normalizuotas koreguotas kūginis stipris, įvertinus vertikalų įtempį ir jo priklausomybę nuo grunto tipo, vnt.d.
 f_s – šoninės trinties stipris, kPa
 R_f – šoninės trinties stiprio ir kūginio stiprio santykis, %
 $I_{c_{SBT}}$ – SBT (gruntų elgsenos tipo) indeksas, vnt.d.
 Q_C – spūdumo koeficientas
 Q_{OCR} – perkonsoliavimo koeficientas
 Q_A – nuogulų amžiaus koeficientas
 n – imtis
 x – imties vidurkis
 S – standartinis nuokrypis
 $Gr.$ – grėžinys
 IGS – inžinerinis geologinis sluoksnis
 x, y – koordinatės (LKS 94), m
 $Abs.a.$ – absoliutinis aukštis, m
 GVG – gruntinio vandens slūgsojimo gylis, m
 GVL – gruntinio vandens lygis, m abs.a.
 CPT – bandymas kūginiu penetrometru
Pastaba: žymuo su $_k$ raide rodo būdingąją (charakteristinę) vertę.

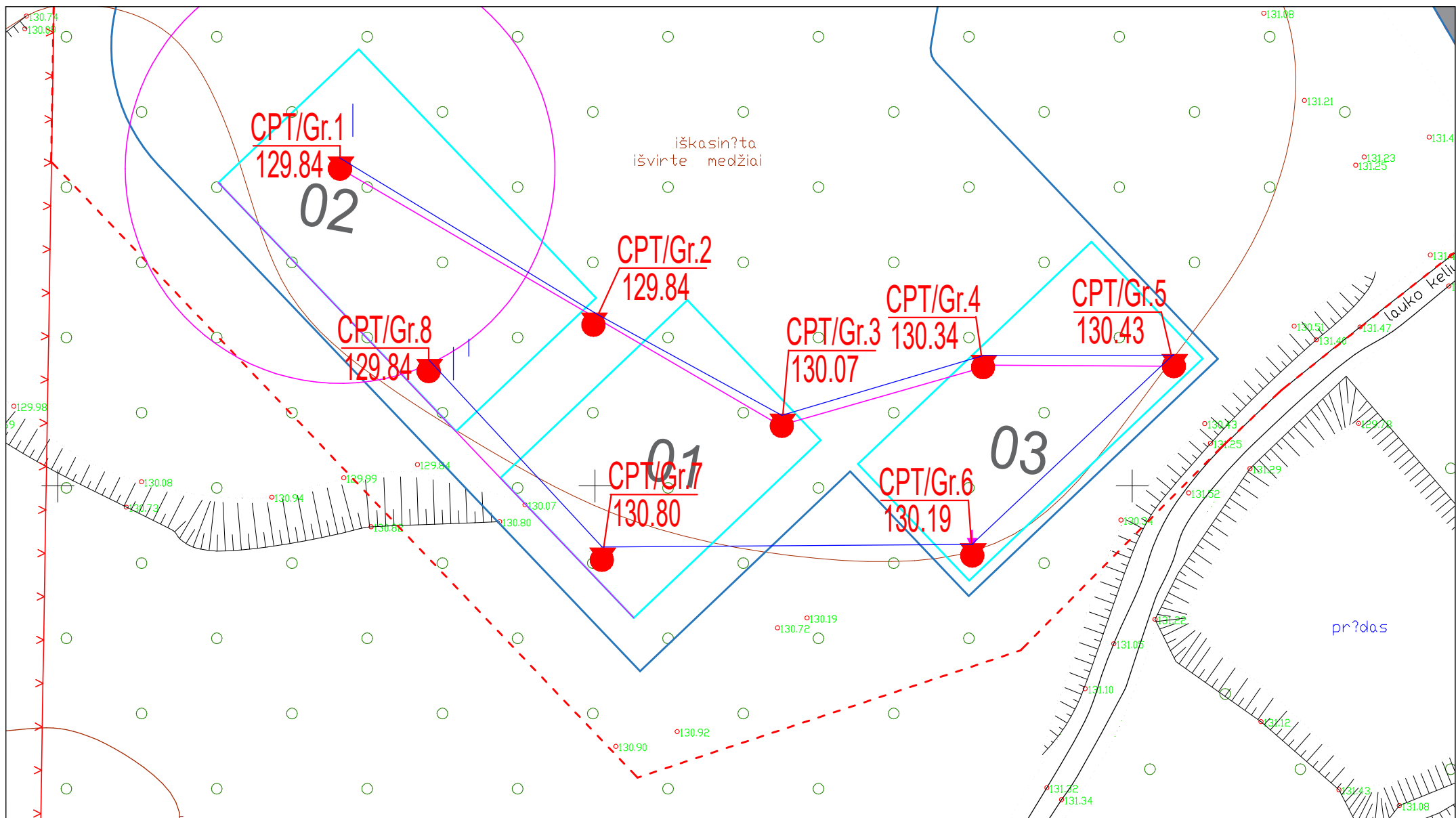
GRAFINIAI PRIEDAI

Tyrimų vietos padėties vietovėje schema



www.maps.lt

Objekto vieta



PLANO SUTARTINIAI ŽENKLAI

— inžinerinis geologinis pjūvis, jo Nr.

Gr.1
132 — gręžinio vieta, jo Nr. ir žiočių altitudė CPT-1
132 — CPT bandymo vieta, jo Nr. ir žiočių altitudė

Pareigos	V.Pavardė	Parašas
Tyrimų vadovas	R.Pranevičiūtė	
Brėžinys: Planas su tyrimų vietomis ir inžinerinio geologinio pjūvio linija		



UAB "IGEO"
tel.: 863482898; el. paštas: uabigeo@gmail.lt
www.i-geo.lt

Užsakovas:		UAB Synergy Solutions		
Objektas:		Projektuojama stoginė ir sandėlis Lakūnų g. 3, Šiaulių m.		
Leidimas	Mastelis	Tyrimų Data	Grafinio Priedo Nr.	
1764351	1:500	2022.08.18	2	

3 grafinis priedas

Gręžinių stulpeliai su geotechninio bandymo CPT kreivėmis

Projektas Projektuojamas sandėliavimo paskirties pastatas ir stoginė Lakūnų g. 3, Šiaulių m.

Projekto Nr. 20322

Gręžimo staklės

Unimog

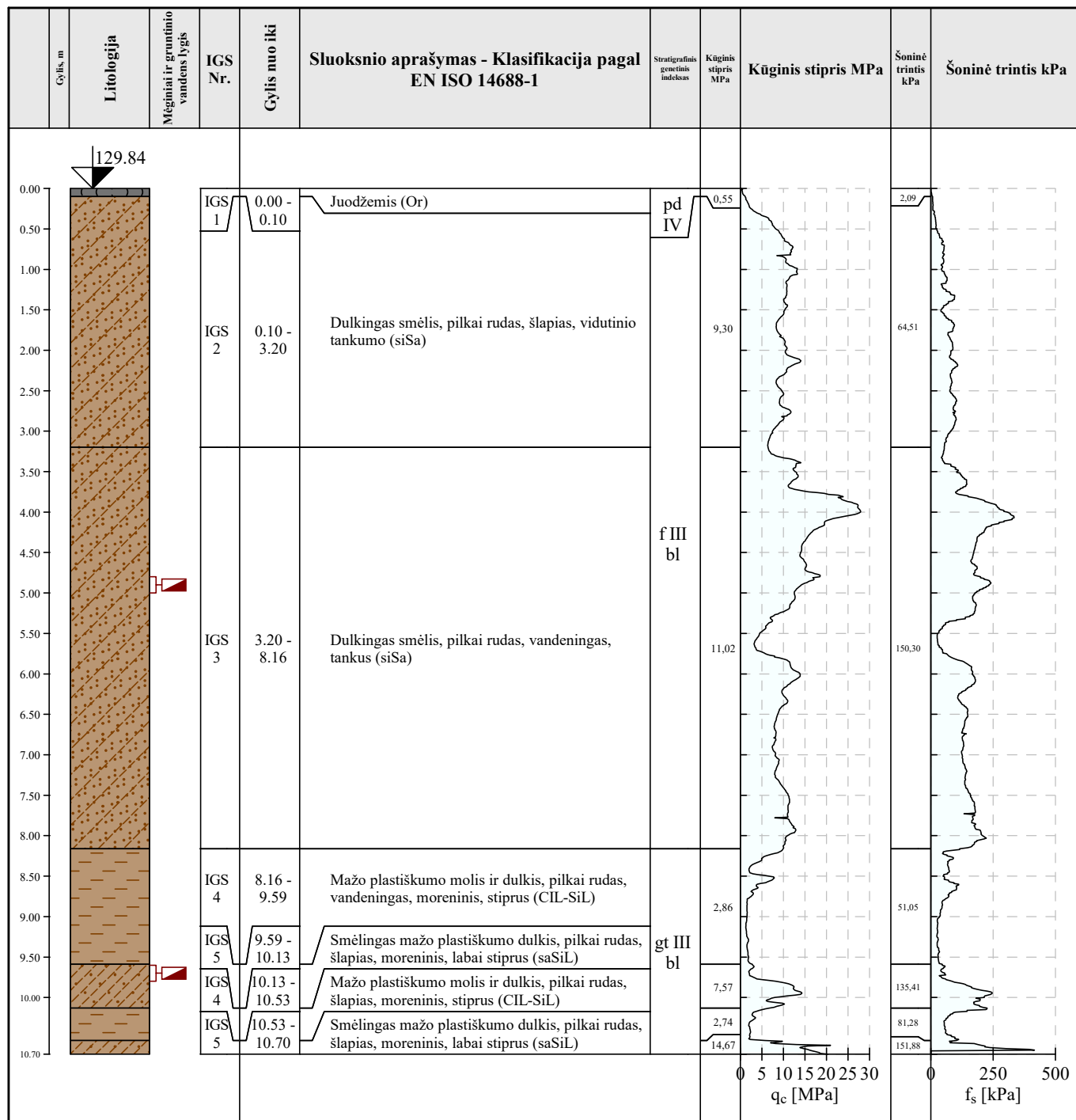
Tyrimo pradžia 8/18/2022

Tyrimo pabaiga 8/18/2022

Koordinatė X 6195279.00

Koordinatė Y 461576.00

Koordinatė Z 129.84 m



Žymėjimas

 Mėginys

Projektas Projektuojamas sandėliavimo paskirties pastatas ir stoginė Lakūnų g. 3, Šiaulių m.

Projekto Nr. 20322

Gręžimo staklės

Unimog

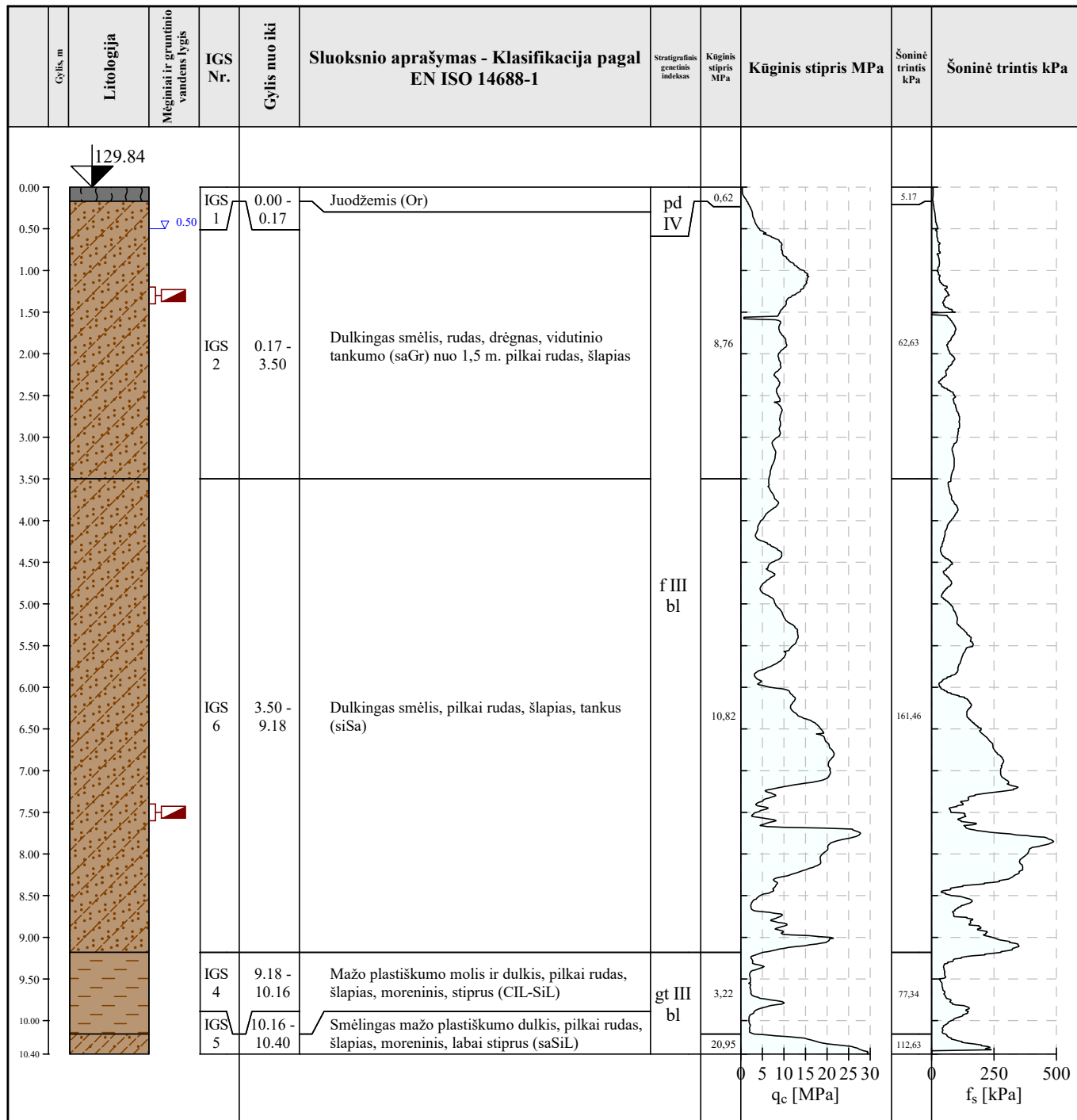
Tyrimo pradžia 8/18/2022

Tyrimo pabaiga 8/18/2022

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Koordinatė Y 461599.00

Koordinatė Z 129.84 m

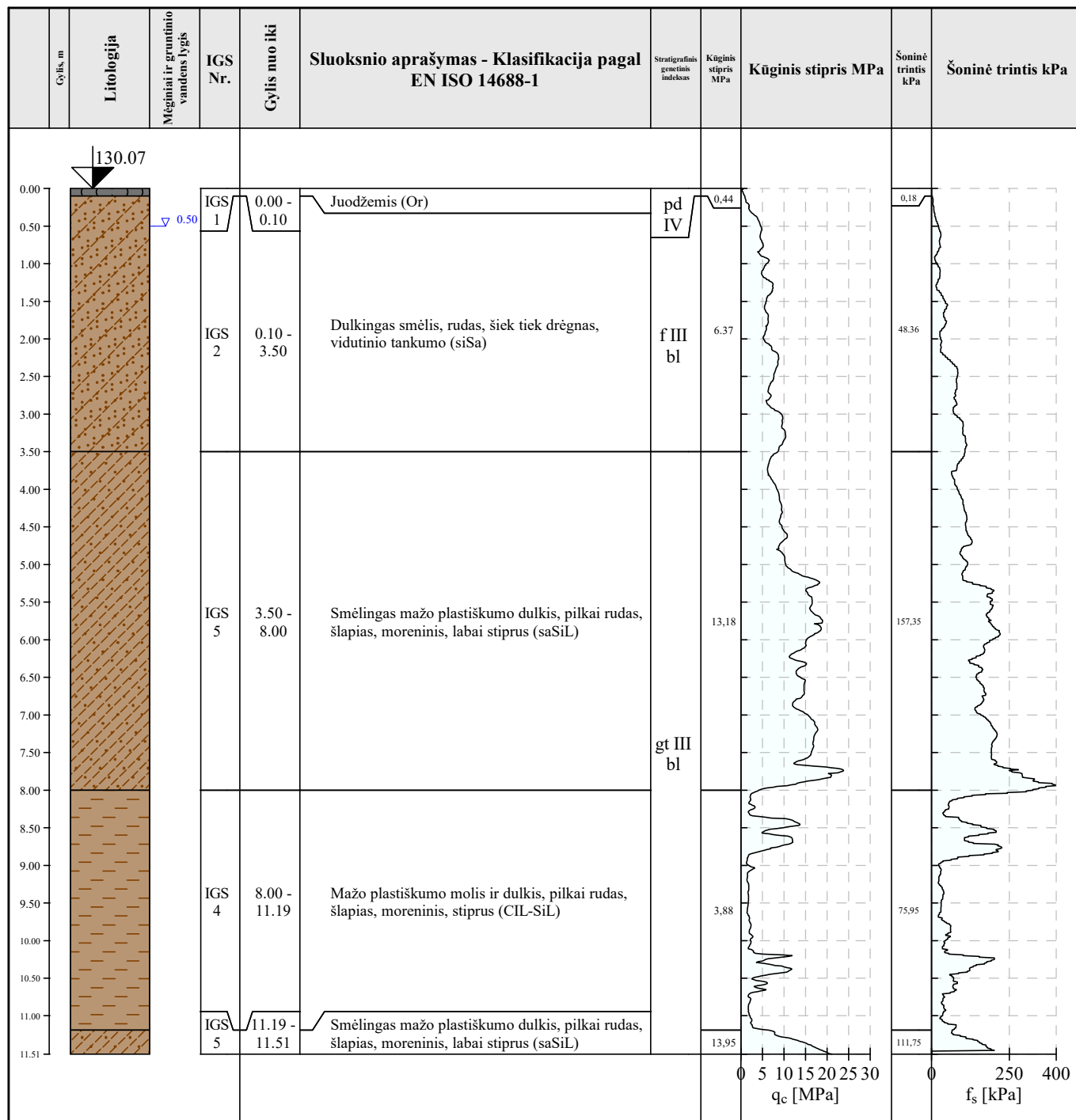


Žymėjimas

Vanduo

Mėginys

Projektas	Projektuojamas sandėliavimo paskirties pastatas ir stoginė Lakūnų g. 3, Šiaulių m.		
Projekto Nr.	20322	Gręžimo staklės	Unimog
Tyrimo pradžia	8/18/2022		Koordinatė X 6195255.00
Tyrimo pabaiga	8/18/2022		Koordinatė Y 461617.00
			Koordinatė Z 130.07 m



Žymėjimas

▽ Vanduo

Projektas Projektuojamas sandėliavimo paskirties pastatas ir stoginė Lakūnų g. 3, Šiaulių m.

Projekto Nr. 20322

Gręžimo staklės

Unimog

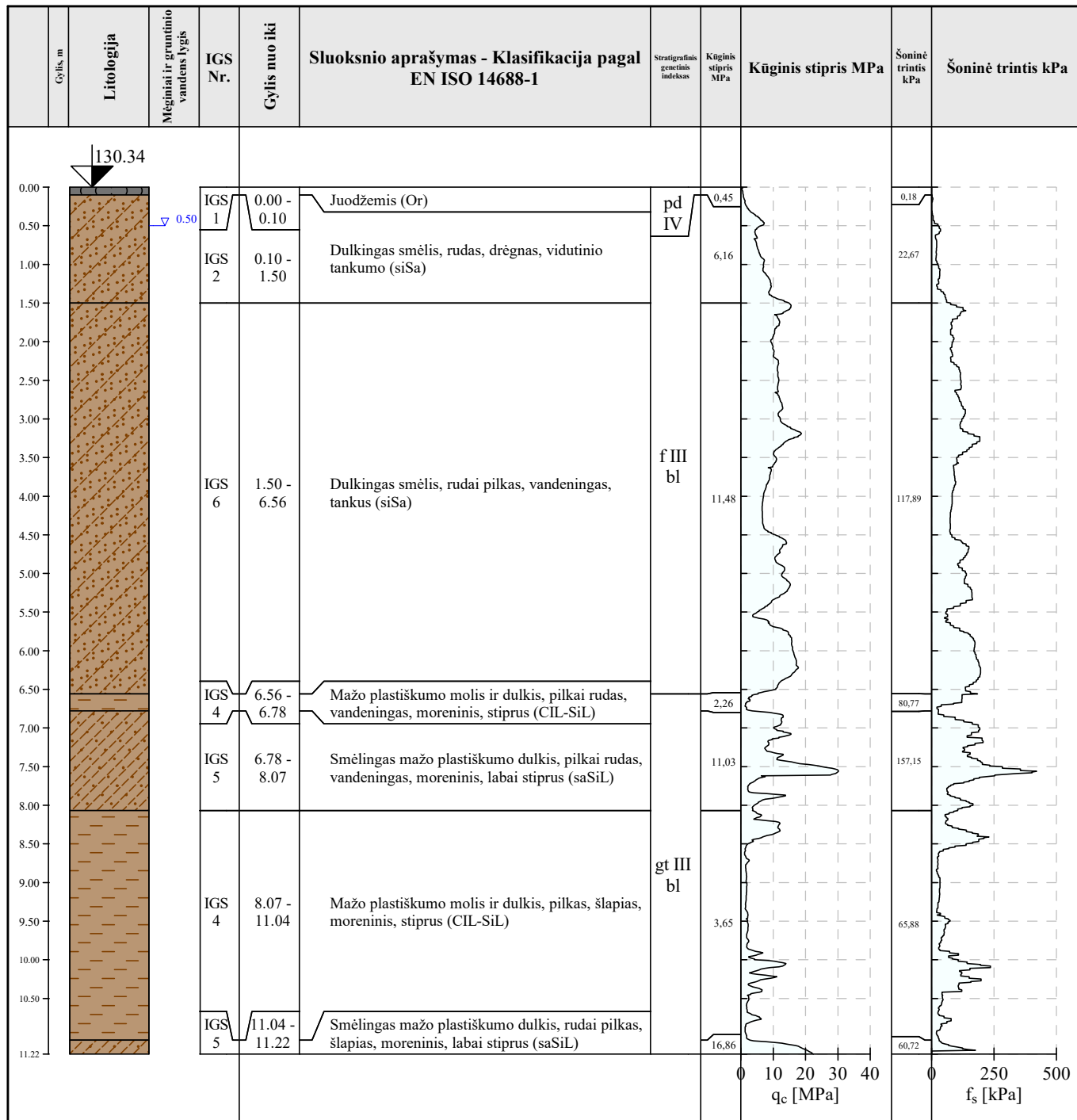
Tyrimo pradžia 8/18/2022

Tyrimo pabaiga 8/18/2022

Koordinatė X 6195261.00

Koordinatė Y 461636.00

Koordinatė Z 130.34 m



Žymėjimas

 Vanduo

Projektas Projektuojamas sandėliavimo paskirties pastatas ir stoginė Lakūnų g. 3, Šiaulių m.

Projekto Nr. 20322

Gręžimo staklės

Unimog

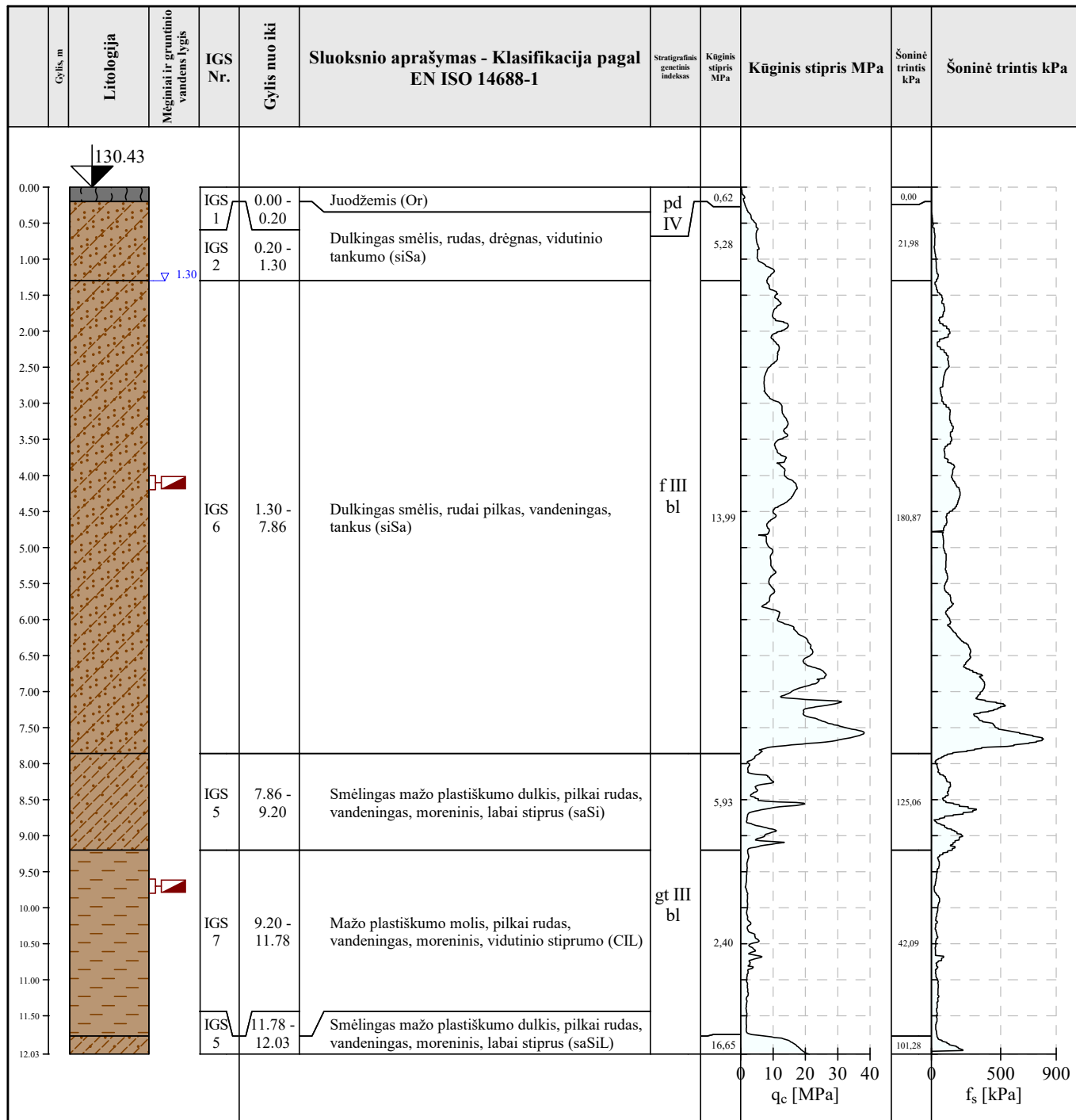
Tyrimo pradžia 8/18/2022

Tyrimo pabaiga 8/18/2022

Koordinatė X 6195261.00

Koordinatė Y 461653.00

Koordinatė Z 130.43 m



Žymėjimas

▽ Vanduo

▬ Mėginys

Projektas Projektuojamas sandėliavimo paskirties pastatas ir stoginė Lakūnų g. 3, Šiaulių m.

Projekto Nr. 20322

Gręžimo staklės

Unimog

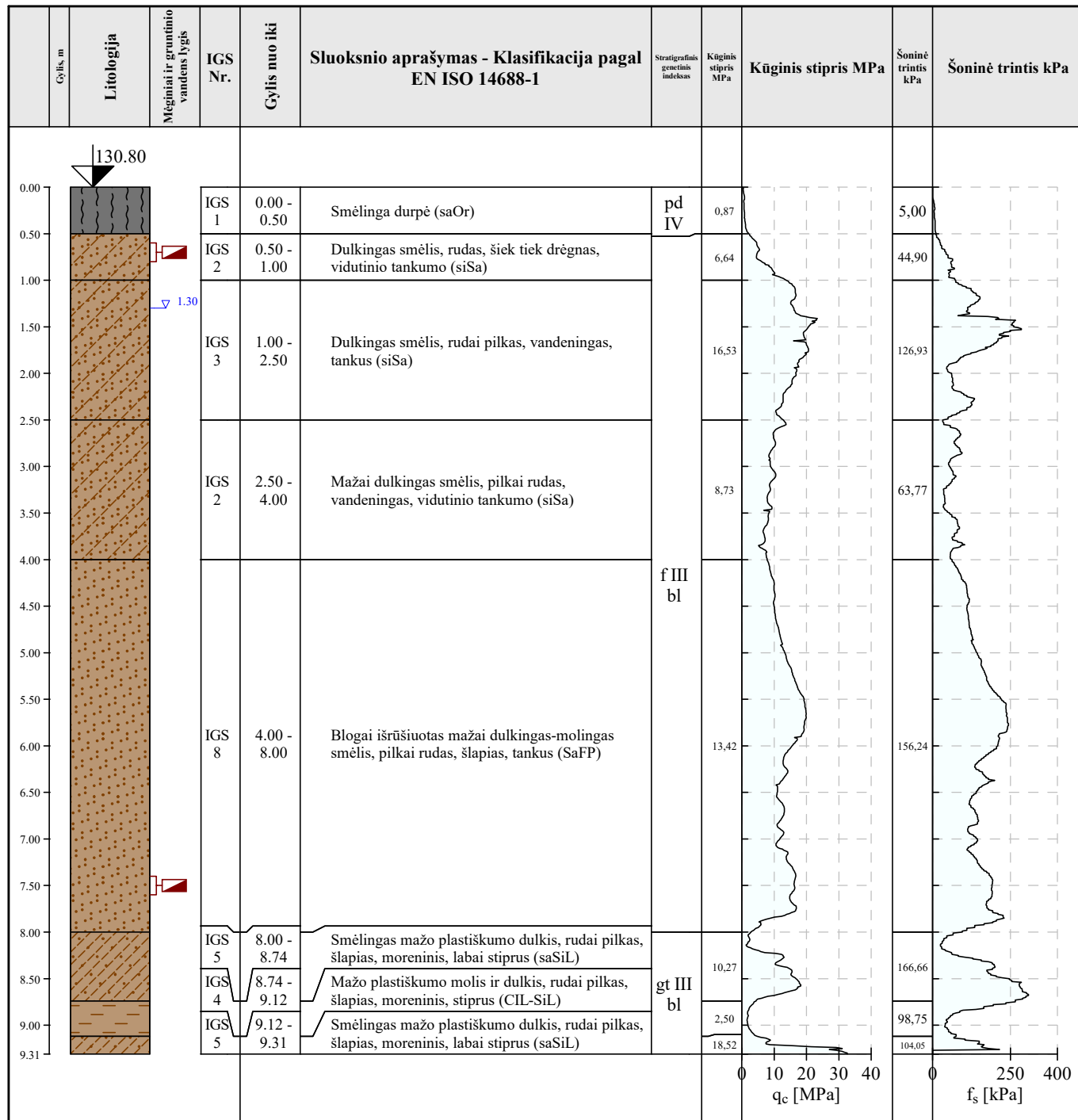
Tyrimo pradžia 8/18/2022

Tyrimo pabaiga 8/18/2022

Koordinatė X 6195242.00

Koordinatė Y 461600.00

Koordinatė Z 130.80 m



Žymėjimas

Vanduo Mėginys

Projektas Projektuojamas sandėliavimo paskirties pastatas ir stoginė Lakūnų g. 3, Šiaulių m.

Projekto Nr. 20322

Gręžimo staklės

Unimog

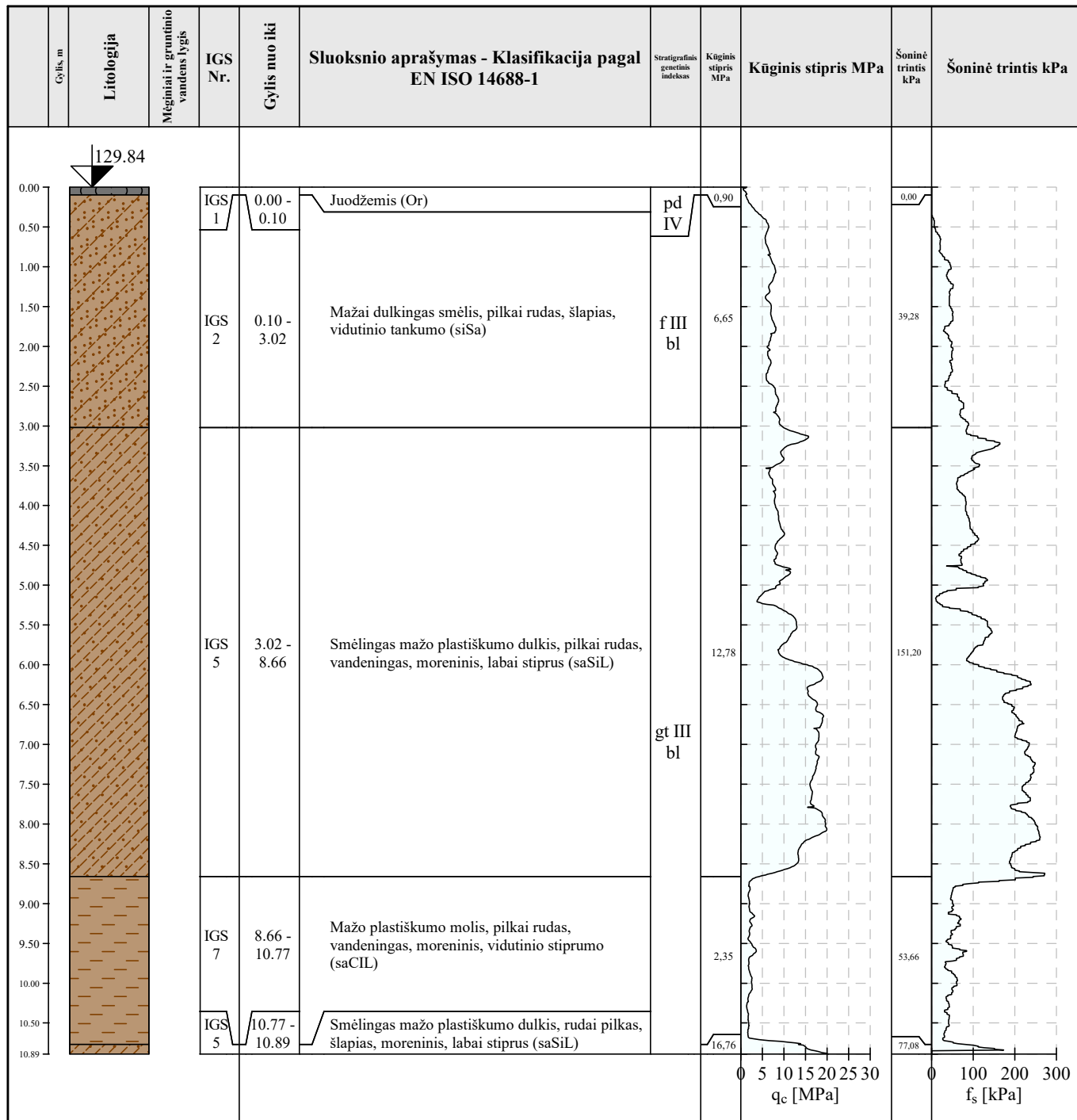
Tyrimo pradžia 8/18/2022

Tyrimo pabaiga 8/18/2022

Koordinatė X 6195260.00

Koordinatė Y 461584.00

Koordinatė Z 129.84 m

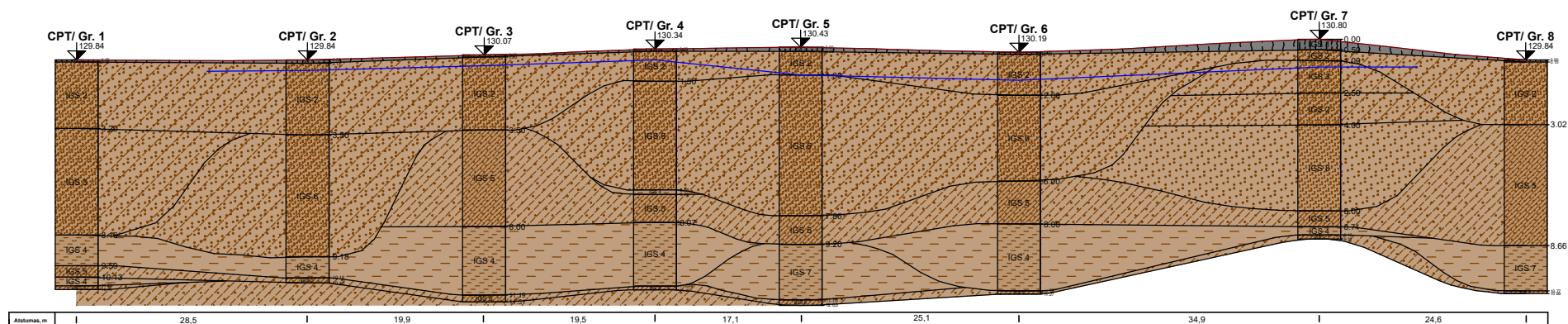


Žymėjimas

4 grafinis priedas

Geologinis pjūvis

GEOLOGINIS-LITOLOGINIS PJŪVIS



GEOLOGICAL SECTION S 1:250/100

[GEO5 - Stratigraphy] version 5.2021.13.0 | hardware key 11043711 | Igeo UAB
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UAB "IGEO"
tel.: 863482898
el. paštas: uabigeo@gmail.lt

Vadovas

R. Pranevičiūtė

Tyrimų Data

Grafinio Priedo Nr.

Parašas

2022.08.18

Nr. 4

DETALŪS METADUOMENYS

Dokumento sudarytojas (-ai)	UAB "Synergy Solutions" 302781077, Daugėlišio g. 32, Vilnius
Dokumento pavadinimas (antraštė)	4.1. SS2209-XX-TP-SK priedai
Dokumento registracijos data ir numeris	–
Dokumento gavimo data ir dokumento gavimo registracijos numeris	–
Dokumento specifikacijos identifikavimo žymuo	ADOC-V1.0
Parašo paskirtis	Pasirašymas
Parašą sukūrusio asmens vardas, pavardė ir pareigos	TOMAS KAZLAUSKAS
Sertifikatas išduotas	TOMAS KAZLAUSKAS LT
Parašo sukūrimo data ir laikas	2023-12-11 13:53:43 (GMT+02:00)
Parašo formatas	XAdES-EPES
Laiko žymoje nurodytas laikas	–
Informacija apie sertifikavimo paslaugų teikėją	RCSC IssuingCA, VI Registru centras - i.k. 124110246 LT
Sertifikato galiojimo laikas	2023-01-13 10:03:41 – 2025-01-12 10:03:41
Informacija apie būdus, naudotus metaduomenų vientisumui užtikrinti	–
Pagrindinio dokumento priedų skaičius	–
Pagrindinio dokumento pridedamų dokumentų skaičius	–
Priedamo dokumento sudarytojas (-ai)	–
Priedamo dokumento pavadinimas (antraštė)	–
Priedamo dokumento registracijos data ir numeris	–
Programinės įrangos, kuria naudojantis sudarytas elektroninis dokumentas, pavadinimas	Signa 2010 (1.3.0.v20231023-11764)
Informacija apie elektroninio dokumento ir elektroninio (-ių) parašo (-ų) tikrinimą (tikrinimo data)	Metaduomuo „Gavimo data“ turi būti nurodytas Metaduomuo „Dokumento gavimo registracijos Nr.“ turi būti nurodytas Metaduomuo „Gavėjas“ turi būti nurodytas Metaduomuo „Priskirtos bylos (tomo) indeksas“ turi būti nurodytas Visi dokumente esantys elektroniniai parašai galioja (2023-12-12 10:35:51)
Paieškos nuoroda	–
Papildomi metaduomenys	Nuorašą suformavo 2023-12-12 10:35:51 Dokumentų valdymo sistema Avilys