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Comparative Evaluation of Steel Profiles in Roof Trusses

Topology Optimization and Structural Analysis

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Contents

1 Introduction

The purpose of this document is to provide a comprehensive evaluation of different types of member profiles in roof trusses. Two groups of profiles are compared:

1. Chord profiles are HEA/HEB, and brace profiles are circular hollow sections (CHS).

2. Both chords and braces are of SHS.

The comparison is performed by employing topology optimization for given spans and truss heights. By using topology optimization, the best possible truss configurations for each combination of profiles is determined. This implies that the results are not biased by predetermined truss types that could be in favor of certain profile combination. In topology optimization, the combined manufacturing and material cost is minimized using a simplified cost function, with data provided by Ruukki.

Optimization is performed by assuming a pin-jointed structure, where bending is taken into account by a simplified heuristic approach. Moreover, joint strength checks are not included in optimization. As a post-processing step, the resulting trusses are evaluated by more accurate structural models and appropriate joint strength rules. The results of these evaluations are also reported.

2 Problem Setting

2.1 Overview

The trusses considered in are simply supported single-span roof trusses. The design domain is shown in Figure 2.1. The span of the truss is varied such that $L = 24\,\text{m}$, or $L = 36\,\text{m}$. For both spans, the height h can take the values $h = L/10$, and $h = L/20$. Thus, the geometry of the design domain implies 4 different cases.

For each variation of design domain geometry, two cross-section scenarios are considered:

1. Chords are HEA/HEB (S460), and the braces are CHS (S355);
2. Chords are SHS (S420), and the braces are SHS (S355).

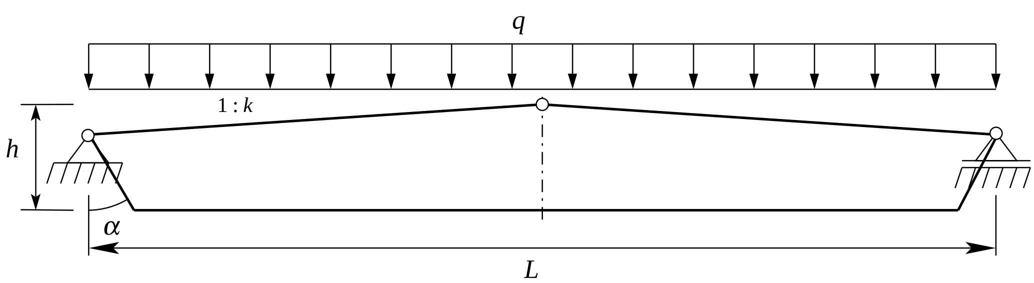


Figure 2.1: Design domain of the roof trusses.

The following loads are employed:

Dead load of roofing	$0.5\,\text{kN/m}^2$
Self-weight of truss	$0.16\,\text{kN/m}^2$
Snow	$0.8 \cdot 2.5\,\text{kN/m}^2 = 2.0\,\text{kN/m}^2$

The distance between trusses is c/c = 6 m.

The design load is determined based on the Swedish National Annex of (EN 1990 2002):

$$\gamma_\text{d} \cdot 1.35 G_{kj,sup} \otimes \gamma_\text{d} \cdot 1.5 \psi_{0,1} Q_{k,1} \tag{2.1}$$

$$\gamma_\text{d} \cdot 0.89 \cdot 1.35 G_{kj,sup} \otimes \gamma_\text{d} \cdot 1.5 Q_{k,1} \tag{2.2}$$

With $\gamma_\text{d} = 1.0$, and $G_{kj,sup} = (0.5 + 0.16)\,\text{kN/m}^2 \cdot 6\,\text{m} = 3.96\,\text{kN/m}$, $Q_{k,1} = 2.0\,\text{kN/m}^2 \cdot 6\,\text{m} = 12\,\text{kN/m}$, and $\psi_{0,1} = 0.7$ for the snow load, the following is obtained:

$$q_1 = 1.0 \cdot 1.35 \cdot 3.96\,\text{kN/m} + 1.0 \cdot 1.5 \cdot 0.7 \cdot 12\,\text{kN/m} = 17.95\,\text{kN/m} \tag{2.3}$$

$$q_2 = 1.0 \cdot 0.89 \cdot 1.35 \cdot 3.96\,\text{kN/m} + 1.0 \cdot 1.5 \cdot 12\,\text{kN/m} = 22.78\,\text{kN/m} \tag{2.4}$$

Thus, the load employed is $q = \max\{q_1, q_2\} = 22.78\,\text{kN/m}$.

2.2 Cross-section alternatives

2.2.1 HEA/HEB chords and CHS braces

The rules for welded joints between I-profile chords and hollow section braces state that the web height of the I-profiles must not exceed 400 mm (EN 1993–1–8 2005, Table 7.20). Furthermore, the cross-section must be in class 1 or 2 (with respect to compression). The largest applicable HEA profile is HEA 450, and the largest HEB profile is HEB 450. Consequently, there are 6 HEA and 16 HEB alternatives. The 22 available HEA/HEB profiles are given in Table 2.1.

Table 2.1: Available HEA/HEB cross-sections.

i	H [mm]	B [mm]	S [mm]	T [mm]	R [mm]	A [$10^2\,\text{mm}^2$]	A_u [$10^3\,\text{mm}^2/\text{mm}$]
1	96	100	5.0	8.0	12	21.20	0.56
2	114	120	5.0	8.0	12	25.30	0.68
3	133	140	5.5	8.5	12	31.40	0.79
4	152	160	6.0	9.0	15	38.80	0.91
5	350	300	10.0	17.5	27	142.80	1.83
6	390	300	11.0	19.0	27	159.00	1.91
7	100	100	6.0	10.0	12	26.00	0.57
8	120	120	6.5	11.0	12	34.00	0.69
9	140	140	7.0	12.0	12	43.00	0.81
10	160	160	8.0	13.0	15	54.30	0.92
11	180	180	8.5	14.0	15	65.30	1.04
12	200	200	9.0	15.0	18	78.10	1.15
13	220	220	9.5	16.0	18	91.00	1.27
14	240	240	10.0	17.0	21	106.00	1.38
15	260	260	10.0	17.5	24	118.40	1.50
16	280	280	10.5	18.0	24	131.40	1.62
17	300	300	11.0	19.0	27	149.10	1.73
18	320	300	11.5	20.5	27	161.30	1.77
19	340	300	12.0	21.5	27	470.90	1.81
20	360	300	12.5	22.5	27	180.60	1.85
21	400	300	13.5	24.0	27	197.80	1.93
22	450	300	14.0	26.0	27	218.00	2.03

For the CHS profiles, the following conditions are employed:

- Wall thickness must be at least 3 mm, which is according to Ruukki's recommendation ((EN 1993–1–8 2005, Clause 7.1.1(5)) requires a 2.5 mm wall thickness)
- Compression members must be in class 1.
- The ratio of the diameter, d_i, and the wall thickness, t_i, must satisfy

$$\frac{d_i}{t_i} \leq 50$$

  For S355, the limit of this ratio on cross-section class 3 is $90\,\varepsilon^2 = 90 \cdot (235/355) = 59.5775$. Thus, only some profiles of class 3 and none of the profiles of class 4 can be employed.
- The diameter of the profile must not exceed 300 mm. This is the maximum flange width of the available HEA/HEB profiles.

Requiring that the profiles belong to Ruukki's 'recommended series', 43 CHS alternatives are obtained. These are listed in Table 2.2. Of the 43 alternatives, only 3 belong to class 2, whereas the others are in class 1. For simplicity, these three alternatives (26, 35, 40) are removed from the profile set. Thus all CHS profiles belong to class 1.

2.2.2 Square hollow section chords and braces

The selection of square hollow sections (SHS) is chosen from the "recommended series" of Ruukki's profiles. The upper chord must be in class 1 (S420), whereas the lower chord must be in class 1 or 2 (S420). Compression braces must be in class 1 or 2, but for braces in tension, no limits for cross-section class are imposed. Nevertheless only class 1 and 2 profiles (S355) are chosen for all braces.

In addition to the restrictions to cross-section class, the following rules are employed in the cross-section selection:

1. Chord width is between 100 mm and 180 mm.
2. Minimum brace width is $0.35 \cdot 100\,\text{mm} = 35\,\text{mm}$. This requirement is derived from design rules for N-, K- and KT-joints. For other joint types, a similar but less stringent design rule is required, but here the more severe rule is used regardless of joint type.
3. Maximum brace width is $0.85 \cdot 180\,\text{mm} = 153\,\text{mm}$.
4. Wall thickness must be at least 3.0 mm.

With these rules, 44 profiles are included in the set of alternatives. They are listed in Table 2.3. For the braces, 40 profile alternatives are available. For the top and bottom chords, the number of profile alternatives are 22 and 24, respectively.

2.3 Cost Factors

In this study, the cost function includes the cost of material, blasting, and painting. The corresponding unit data is given in Table 2.4.

Table 2.2: Available CHS cross-sections.

i	D [mm]	T [mm]	A [10^2 mm^2]	A_u [10^3 mm^2/mm]
1	33.7	3.2	3.07	0.11
2	48.3	3.0	4.27	0.15
3	48.3	4.0	5.57	0.15
4	60.3	3.0	5.40	0.19
5	60.3	4.0	7.07	0.19
6	60.3	5.0	8.69	0.19
7	76.1	4.0	9.06	0.24
8	76.1	6.3	13.81	0.24
9	88.9	4.0	10.67	0.28
10	88.9	5.0	13.18	0.28
11	88.9	6.3	16.35	0.28
12	101.6	3.6	11.08	0.32
13	101.6	5.0	15.17	0.32
14	108.0	3.6	11.81	0.34
15	108.0	5.0	16.18	0.34
16	114.3	3.6	12.52	0.36
17	114.3	5.0	17.17	0.36
18	114.3	6.3	21.38	0.36
19	127.0	4.0	15.46	0.40
20	127.0	5.0	19.16	0.40
21	139.7	4.0	17.05	0.44
22	139.7	5.0	21.16	0.44
23	139.7	6.3	26.40	0.44
24	139.7	8.0	33.10	0.44
25	139.7	10.0	40.75	0.44
26	168.3	4.0	20.65	0.53
27	168.3	4.5	23.16	0.53
28	168.3	5.0	25.65	0.53
29	168.3	6.0	30.59	0.53
30	168.3	8.0	40.29	0.53
31	168.3	10.0	49.73	0.53
32	193.7	5.0	29.64	0.61
33	193.7	6.3	37.09	0.61
34	193.7	10.0	57.71	0.61
35	219.1	4.5	30.34	0.69
36	219.1	6.0	40.17	0.69
37	219.1	8.0	53.06	0.69
38	219.1	10.0	65.69	0.69
39	219.1	12.5	81.13	0.69
40	273.0	6.0	50.33	0.86
41	273.0	8.0	66.60	0.86
42	273.0	10.0	82.62	0.86
43	273.0	12.5	102.30	0.86

Table 2.3: Available SHS cross-sections. Profiles 1–40 are available for braces, profiles 21–44 can be chosen for the bottom chord, and profiles 21–44 except 28 and 36 can be selected for the top chord.

i	H [mm]	T [mm]	A [10^2 mm^2]	A_u [10^3 mm^2/mm]
1	40	3.0	4.21	0.15
2	40	4.0	5.35	0.15
3	50	3.0	5.41	0.19
4	50	4.0	6.95	0.19
5	50	5.0	8.36	0.18
6	60	3.0	6.61	0.23
7	60	4.0	8.55	0.23
8	60	5.0	10.36	0.22
9	70	3.0	7.81	0.27
10	70	4.0	10.15	0.27
11	70	5.0	12.36	0.26
12	80	3.0	9.01	0.31
13	80	4.0	11.75	0.31
14	80	5.0	14.36	0.30
15	80	6.0	16.83	0.30
16	90	3.0	10.21	0.35
17	90	4.0	13.35	0.35
18	90	5.0	16.36	0.34
19	90	6.0	19.23	0.34
20	100	3.0	11.41	0.39
21	100	4.0	14.95	0.39
22	100	5.0	18.36	0.38
23	100	6.0	21.63	0.38
24	100	8.0	27.24	0.37
25	110	4.0	16.55	0.43
26	110	5.0	20.36	0.42
27	110	6.0	24.03	0.42
28	120	4.0	18.15	0.47
29	120	5.0	22.36	0.46
30	120	6.0	26.43	0.46
31	120	8.0	33.64	0.45
32	120	10.0	40.57	0.44
33	140	5.0	26.36	0.54
34	140	6.0	31.23	0.54
35	140	8.0	40.04	0.53
36	150	5.0	28.36	0.58
37	150	6.0	33.63	0.58
38	150	8.0	43.24	0.57
39	150	10.0	52.57	0.56
40	150	12.5	62.04	0.54
41	160	8.0	46.44	0.61
42	160	10.0	56.57	0.60
43	180	8.0	52.84	0.69
44	180	10.0	64.57	0.68

Table 2.4: Cost data.

Factor	Cost	Unit
Blasting	3	€/m^2
Painting, C2	8.5	€/m^2
Intumescent Paint, R30	40	€/m^2
Quality control	20	€/(10^3 kg)
HEA & HEB, S355	750	€/(10^3 kg)
HEA & HEB, S460	800	€/(10^3 kg)
CHS, S355	750	€/(10^3 kg)
SHS, S420/S355	750	€/(10^3 kg)

Material cost. The cost of material is expressed as

$$C_M(\mathbf{x}) = \sum_{i=1}^{n_E} \sum_j c_{M,j} \rho_i L_i \hat{A}_j y_{ij} \tag{2.5}$$

where $c_{M,i}$ is the material cost factor that is specific for profile j.

Blasting cost. The cost of blasting is written as

$$C_B(\mathbf{x}) = \sum_{i=1}^{n_E} \sum_j c_B L_i \hat{A}_{u,j} y_{ij} \tag{2.6}$$

where $c_B = 3 \cdot 10^{-6}$€$/\mathrm{mm}^2$ is the blasting cost factor.

Painting cost. In general, the cost of painting can be expressed as

$$C_P(\mathbf{x}) = \sum_{i=1}^{n_E} \sum_j c_P L_i \hat{A}_{u,j} y_{ij} \tag{2.7}$$

where $c_B = 8.5 \cdot 10^{-6}$€$/\mathrm{mm}^2$ is the unit cost for painting.
The complete cost function is the sum of the above cost factors, i.e

$$C(\mathbf{x}) = C_M(\mathbf{x}) + C_B(\mathbf{x}) + C_P(\mathbf{x}) \tag{2.8}$$

3 Topology Optimization

3.1 Introduction

As a first step in the study, optimum topologies for varying truss spans and heights are determined. In topology optimization, the structural model is based on pin-jointed truss elements, where only axial forces appear. In the basic model, the effects of bending are neglected. However, for the specific application, the chord members are sized for bending by approximating the bending moment and applying the appropriate design rule of Eurocode 3. The framework for topology optimization can be summarized as follows.

- Members are pin-jointed bars.
- Bending of the chords is taken into account by assuming a bending moment of

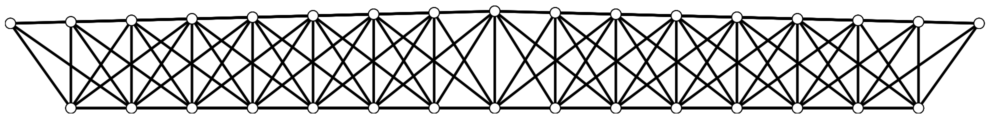
  $$M_i = \frac{qL_i^2}{10}$$

  in the top chord members and

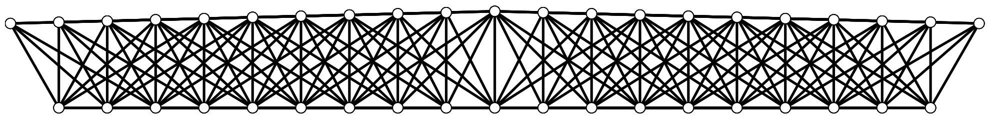
  $$M_i = \frac{qL_i^2}{20}$$

  in the bottom chord members. In the above, q is the design load, and L_i is the length of member i. Members in tension must satisfy the appropriate design rule in (EN 1993–1–1 2005, Clause 6.2.9.1(5)), and members in tension are sized according to the rule (EN 1993–1–1 2005, Clause 6.3.3(4)).
- Joints are not considered, i.e. joint strength and eccentricities are not checked. However, separate constraints ensure that the dimensions of the braces are within the prescribed limits as stated in (EN 1993–1–8 2005, Table 7.20 and Table 7.8). To be more specific, for the SHS profiles, the rules of first column of (EN 1993–1–8 2005, Table 7.8) are implemented. Furthermore, it is ensured that the brace widths do not exceed the width of the chords.
- The trusses are optimized for cost.
- The height of the truss is measured from the center line of the bottom chord to the center line of the top chord at the ridge.

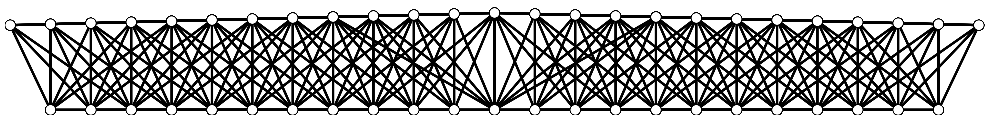
For each combination of span and height, 3 ground structures are considered. One half of the design domain is divided into 8, 10, or 12 equally spaced intervals. Braces are added to the ground structure, if they meet both chords in an angle greater than 30°. The maximum member length is limited to $L/5$. The ground structures are depicted in Figure 3.1.



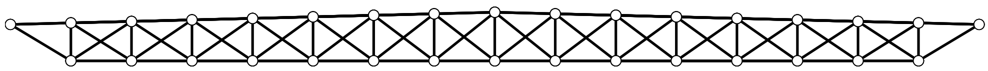
(a) Ground structure with 8 intervals on one half of the span, $h = L/10$.



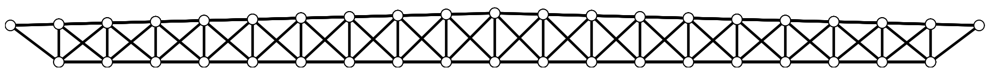
(b) Ground structure with 10 intervals on one half of the span, $h = L/10$.



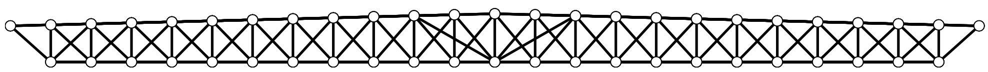
(c) Ground structure with 12 intervals on one half of the span, $h = L/10$.



(d) Ground structure with 8 intervals on one half of the span, $h = L/20$.



(e) Ground structure with 10 intervals on one half of the span, $h = L/20$.



(f) Ground structure with 12 intervals on one half of the span, $h = L/20$.

Figure 3.1: Ground structures for topology optimization, $L = 24\,\text{m}$.

3.2 HEA/HEB chords and CHS braces

For HEA/HEB profiles, there are 22 available alternatives that belong to cross-section class 1 or 2 (S460) and have diameter at most 400 mm. For CHS braces belonging to class 1 (S355), 40 alternatives are available. The number of profile alternatives has a direct effect on the optimization problem size. In Table 3.1, the different problem instances are shown.

The results of topology optimization are summarized in Table 3.2. In the table, the initial and minimum cost are reported. For given span and height, the minimum cost among the three ground structures is in bold face. It can be seen that depending on the span and height, any ground structure can provide the minimum cost.

The optimum designs are illustrated in Figures 3.2–3.5, and member details are shown in Tables 3.3–3.6. For each member, the profile, cost, axial force, design value of bending moment, and utilization ratios for strength and buckling design rules are given. Note that due to symmetry, only half of the members are displayed in the tables. Note that the utilization ratios are given with respect to normal force resistances, also for chord members.

Table 3.1: Problem instances for topology optimization of HEA/HEB/CHS trusses.

i	q [kN/m]	L [m]	L/h	k	n_x	N_x	N_y	$n^{\leq}$	$n^{=}$	n_E
1	22.78	24	10	40	8	7268	2457	21617	275	150
2	22.78	24	10	40	10	11651	3923	35042	394	237
3	22.78	24	10	40	12	15103	5071	45958	502	313
4	22.78	24	20	40	8	5669	1924	17139	249	124
5	22.78	24	20	40	10	7715	2611	23676	330	173
6	22.78	24	20	40	12	10183	3431	31688	422	233
7	22.78	36	10	40	8	7268	2457	21617	275	150
8	22.78	36	10	40	10	11651	3923	35042	394	237
9	22.78	36	10	40	12	15103	5071	45958	502	313
10	22.78	36	20	40	8	5669	1924	17139	249	124
11	22.78	36	20	40	10	7715	2611	23676	330	173
12	22.78	36	20	40	12	10183	3431	31688	422	233

Table 3.2: Results of topology optimization for HEA/HEB/CHS trusses. C_0 = initial cost; C^* = minimum cost obtained; t^* = runtime when the optimum was found; t_{fin} = runtime at termination (time limit 21600 s); G_0 = initial optimality gap; G^* = optimality gap, when optimum was found; G_{fin} = final optimality gap.

i	C_0 [10^2 €]	C^* [10^2 €]	$\frac{C_0 - C^*}{C^*}$ [%]	t^* [s]	t_{fin}	G_0 [%]	G^* [%]	G_{fin} [%]
1	13.60	**13.37**	1.69	32	248	28.90	13.40	1.98
2	14.73	14.32	2.86	429	666	33.00	12.30	1.67
3	15.24	14.01	8.79	1476	3180	37.10	15.40	1.44
4	17.10	17.07	0.16	194	257	26.20	11.70	1.81
5	17.28	17.14	0.79	465	731	29.20	13.80	1.05
6	16.53	**16.36**	1.01	1078	3109	25.90	12.00	2.00
7	29.89	**29.73**	0.55	564	603	33.10	7.04	1.88
8	36.13	35.10	2.94	238	890	45.20	30.10	1.84
9	34.14	32.56	4.86	7922	7944	42.70	2.22	1.61
10	38.93	38.75	0.46	247	249	29.90	4.35	1.20
11	36.14	**35.99**	0.44	785	1367	27.70	13.50	1.39
12	36.40	36.22	0.50	692	1340	29.00	18.60	1.04

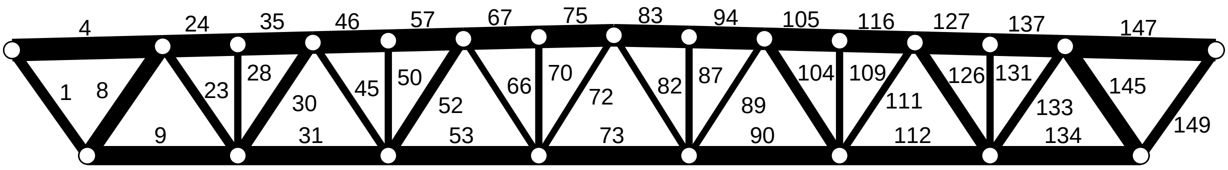


Figure 3.2: Optimum topology for HEA/HEB/CHS profiles. $L = 24\,\text{m}$, $h = L/10$. $C^* = 1337$€.

Table 3.3: Optimum design for $L = 24\,\text{m}$, $h = L/10$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	CHS 60.3x5.0	38.33	288.78	0.00	0.94	0.00
4	HEB 100	139.58	-167.91	20.51	0.23	0.97
8	CHS 88.9x6.3	69.18	-285.46	0.00	0.49	0.97
9	HEA 100	120.59	329.92	20.50	0.44	0.00
23	CHS 60.3x4.0	34.07	214.58	0.00	0.85	0.00
24	HEB 100	69.79	-451.88	5.13	0.42	0.56
28	CHS 48.3x3.0	19.16	-34.17	0.00	0.23	0.81
30	CHS 88.9x4.0	52.23	-171.23	0.00	0.45	0.89
31	HEA 100	120.59	546.72	20.50	0.73	0.00
35	HEB 100	69.79	-451.88	5.13	0.42	0.56
45	CHS 48.3x3.0	23.41	125.22	0.00	0.83	0.00
46	HEB 100	69.79	-616.37	5.13	0.57	0.77
50	CHS 48.3x3.0	19.81	-34.17	0.00	0.23	0.86
52	CHS 76.1x4.0	45.51	-83.33	0.00	0.26	0.65
53	HEA 100	120.59	661.35	20.50	0.89	0.00
57	HEB 100	69.79	-616.37	5.13	0.57	0.77
66	CHS 33.7x3.2	17.01	40.66	0.00	0.37	0.00
67	HEB 100	69.79	-683.61	5.13	0.63	0.85
70	CHS 48.3x3.0	20.45	-34.17	0.00	0.23	0.90
72	CHS 33.7x3.2	17.40	-0.00	0.00	0.00	0.00
73	HEA 100	60.29	683.40	20.50	0.92	0.00
75	HEB 100	69.79	-683.61	5.13	0.63	0.85
Total Cost		1336.96				

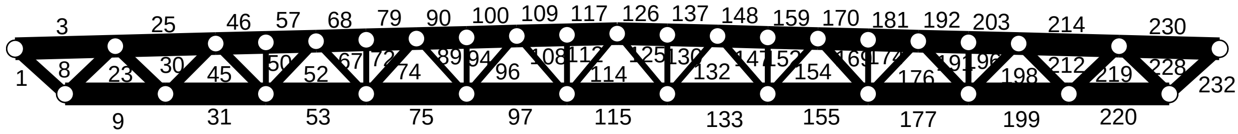


Figure 3.3: Optimum topology for HEA/HEB/CHS profiles. $L = 24\,\mathrm{m}$, $h = L/20$. $C^* = 1636$ €.

Table 3.4: Optimum design for $L = 24\,\mathrm{m}$, $h = L/20$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	CHS 76.1x6.3	29.86	364.46	0.00	0.74	0.00
3	HEB 120	119.14	-270.98	9.12	0.19	0.28
8	CHS 76.1x6.3	30.61	-353.99	0.00	0.72	0.96
9	HEB 120	119.10	527.54	9.11	0.36	0.00
23	CHS 76.1x6.3	30.61	271.38	0.00	0.55	0.00
25	HEB 120	119.14	-724.51	9.12	0.52	0.76
30	CHS 88.9x4.0	27.32	-264.33	0.00	0.70	0.86
31	HEB 120	119.10	911.20	9.11	0.62	0.00
45	CHS 60.3x5.0	21.00	204.30	0.00	0.66	0.00
46	HEB 120	59.57	-1055.99	2.28	0.69	0.74
50	CHS 33.7x3.2	6.30	-22.78	0.00	0.21	0.44
52	CHS 60.3x5.0	21.54	-168.03	0.00	0.54	0.88
53	HEB 120	119.10	1171.54	9.11	0.79	0.00
57	HEB 120	59.57	-1055.99	2.28	0.69	0.74
67	CHS 48.3x4.0	14.83	129.49	0.00	0.65	0.00
68	HEB 120	59.57	-1261.24	2.28	0.83	0.89
72	CHS 33.7x3.2	6.61	-22.78	0.00	0.21	0.47
74	CHS 60.3x5.0	22.08	-95.94	0.00	0.31	0.51
75	HEB 120	119.10	1325.38	9.11	0.90	0.00
79	HEB 120	59.57	-1261.24	2.28	0.83	0.89
89	CHS 33.7x3.2	9.14	61.57	0.00	0.56	0.00
90	HEB 120	59.57	-1367.23	2.28	0.90	0.96
94	CHS 33.7x3.2	6.92	-22.78	0.00	0.21	0.49
96	CHS 48.3x3.0	13.19	-30.19	0.00	0.20	0.42
97	HEB 120	119.10	1386.61	9.11	0.94	0.00
100	HEB 120	59.57	-1367.23	2.28	0.90	0.96
108	CHS 33.7x3.2	9.37	-0.64	0.00	0.01	0.02
109	HEB 120	59.57	-1386.62	2.28	0.91	0.98
112	CHS 33.7x3.2	7.23	-22.78	0.00	0.21	0.52
114	CHS 33.7x3.2	9.61	30.28	0.00	0.28	0.00
115	HEB 120	59.55	1366.80	9.11	0.92	0.00
117	HEB 120	59.57	-1386.62	2.28	0.91	0.98
Total Cost		1636.08				

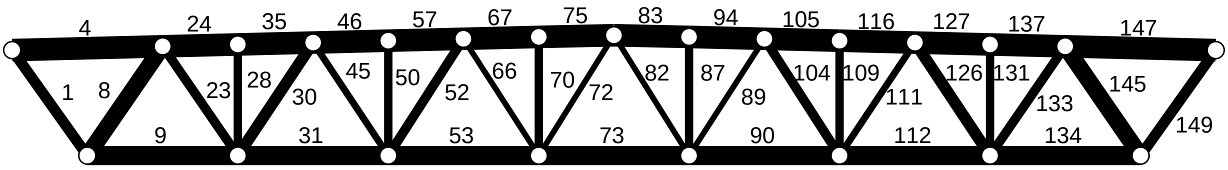


Figure 3.4: Optimum topology for HEA/HEB/CHS profiles. $L = 36\,\text{m}$, $h = L/10$. $C^* = 2973$€.

Table 3.5: Optimum design for $L = 36\,\text{m}$, $h = L/10$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	CHS 76.1x6.3	85.91	433.18	0.00	0.88	0.00
4	HEA 160	318.65	-251.86	46.16	0.22	0.87
8	CHS 139.7x6.3	166.50	-428.19	0.00	0.46	0.84
9	HEA 140	264.09	494.88	46.13	0.46	0.00
23	CHS 88.9x4.0	76.55	321.86	0.00	0.85	0.00
24	HEA 160	159.33	-677.82	11.54	0.42	0.55
28	CHS 60.3x5.0	49.29	-51.25	0.00	0.17	0.84
30	CHS 127.0x4.0	113.03	-256.84	0.00	0.47	0.96
31	HEA 140	264.09	820.08	46.13	0.76	0.00
35	HEA 160	159.33	-677.82	11.54	0.42	0.55
45	CHS 48.3x4.0	41.49	187.83	0.00	0.95	0.00
46	HEA 160	159.33	-924.56	11.54	0.57	0.74
50	CHS 60.3x5.0	50.96	-51.25	0.00	0.17	0.89
52	CHS 101.6x3.6	86.04	-124.99	0.00	0.32	0.92
53	HEA 140	264.09	992.03	46.13	0.92	0.00
57	HEA 160	159.33	-924.56	11.54	0.57	0.74
66	CHS 33.7x3.2	25.52	61.00	0.00	0.56	0.00
67	HEA 160	159.33	-1025.42	11.54	0.63	0.83
70	CHS 60.3x5.0	52.63	-51.26	0.00	0.17	0.94
72	CHS 33.7x3.2	26.11	0.00	0.00	0.00	0.00
73	HEA 140	132.04	1025.10	46.13	0.95	0.00
75	HEA 160	159.33	-1025.42	11.54	0.63	0.83
Total Cost		2972.95				

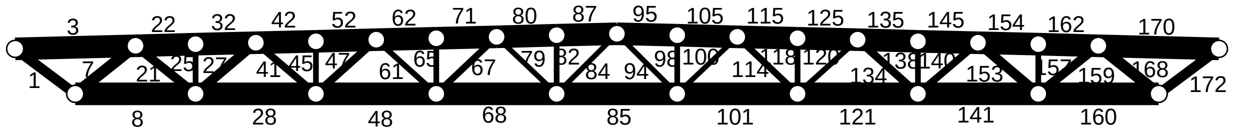


Figure 3.5: Optimum topology for HEA/HEB/CHS profiles. $L = 36\,\text{m}$, $h = L/20$. $C^* = 3599$ €.

Table 3.6: Optimum design for $L = 36\,\text{m}$, $h = L/20$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	CHS 114.3x5.0	65.28	595.22	0.00	0.98	0.00
3	HEB 160	327.77	-476.32	29.54	0.23	0.50
7	CHS 139.7x5.0	82.24	-571.69	0.00	0.76	0.95
8	HEB 160	327.67	922.59	29.52	0.40	0.00
21	CHS 76.1x6.3	51.16	441.57	0.00	0.90	0.00
22	HEB 160	163.89	-1267.79	7.39	0.53	0.61
25	CHS 48.3x3.0	12.86	-41.00	0.00	0.27	0.55
27	CHS 108.0x5.0	64.63	-362.61	0.00	0.63	0.92
28	HEB 160	327.67	1543.68	29.52	0.67	0.00
32	HEB 160	163.89	-1267.79	7.39	0.53	0.61
41	CHS 60.3x5.0	35.09	280.38	0.00	0.91	0.00
42	HEB 160	163.89	-1757.86	7.39	0.73	0.84
45	CHS 48.3x3.0	13.64	-41.00	0.00	0.27	0.59
47	CHS 88.9x4.0	46.78	-210.15	0.00	0.55	0.97
48	HEB 160	327.67	1913.52	29.52	0.83	0.00
52	HEB 160	163.89	-1757.86	7.39	0.73	0.84
61	CHS 48.3x3.0	20.97	139.16	0.00	0.92	0.00
62	HEB 160	163.89	-2017.58	7.39	0.84	0.97
65	CHS 48.3x3.0	14.42	-41.00	0.00	0.27	0.64
67	CHS 60.3x4.0	32.01	-75.62	0.00	0.30	0.92
68	HEB 160	327.67	2071.78	29.52	0.90	0.00
71	HEB 160	163.89	-2017.58	7.39	0.84	0.97
79	CHS 33.7x3.2	15.27	13.74	0.00	0.13	0.00
80	HEB 160	163.89	-2082.39	7.39	0.87	1.00
82	CHS 48.3x3.0	15.19	-41.00	0.00	0.27	0.69
84	CHS 33.7x3.2	15.65	44.61	0.00	0.41	0.00
85	HEB 160	163.84	2050.20	29.52	0.89	0.00
87	HEB 160	163.89	-2082.39	7.39	0.87	1.00
Total Cost		3598.56				

3.3 SHS chord and SHS braces

For SHS trusses, same ground structures are used as for the HEA/HEB/CHS trusses (see Figure 3.1). Problem sizes for each instance are given in Table 3.7.

Two sets of optimization problems are solved regarding the buckling curve. In the first run, the buckling curve 'c' required by the present Eurocode is employed. Then, due to the implied results of recent measurements, buckling curve 'b' is adopted for the members. The results for these separate instances are reported below.

Table 3.7: Problem instances for topology optimization of SHS trusses.

i	q [kN/m]	L [m]	L/h	k	n_x	N_x	N_y	$n^{\leq}$	$n^{=}$	n_E
1	22.78	24	10	40	8	7384	2497	22131	275	150
2	22.78	24	10	40	10	11843	3989	35906	394	237
3	22.78	24	10	40	12	15343	5153	47042	502	313
4	22.78	24	20	40	8	5785	1964	17601	249	124
5	22.78	24	20	40	10	7907	2677	24412	330	173
6	22.78	24	20	40	12	10423	3513	32612	422	233
7	22.78	36	10	40	8	7384	2497	22131	275	150
8	22.78	36	10	40	10	11843	3989	35906	394	237
9	22.78	36	10	40	12	15343	5153	47042	502	313
10	22.78	36	20	40	8	5785	1964	17601	249	124
11	22.78	36	20	40	10	7907	2677	24412	330	173
12	22.78	36	20	40	12	10423	3513	32612	422	233

3.3.1 Results using present Eurocode

First, the SHS trusses are optimized using present Eurocode. This means that the buckling curve 'c' as indicated in (EN 1993–1–1 2005, Table 6.2) is employed for cold-formed sections. The results of topology optimization are summarized in Table 3.8. It can be seen that the algorithm converged within one hour in all cases, and usually in much less time.

From the results, it can be seen that for both spans the trusses with height $h = L/10$ are substantially more economical. For the 24 m span, the difference is 28 %, and for the span of 36 m, the difference is 20 %.

The optimum topologies are shown in Figures 3.6–3.9, and the corresponding member data are given in Tables 3.9–3.12. In addition to member profiles, the normal forces, bending moments, and utilization ratios are also given.

Table 3.8: Results of topology optimization for SHS trusses. C_0 = initial cost; C^* = minimum cost obtained; t^* = runtime when the optimum was found; t_{fin} = runtime at termination (time limit 21600 s); G_0 = initial optimality gap; G^* = optimality gap, when optimum was found; G_{fin} = final optimality gap.

i	C_0 [10^2 €]	C^* [10^2 €]	$\frac{C_0 - C^*}{C^*}$ [%]	t^* [s]	t_{fin}	G_0 [%]	G^* [%]	G_{fin} [%]
1	12.72	**12.18**	4.47	424	430	28.50	2.33	1.93
2	13.74	13.11	4.81	675	763	32.50	8.59	1.06
3	13.49	12.54	7.58	2963	3311	33.60	6.72	1.93
4	16.39	16.29	0.63	29	278	25.90	16.30	1.95
5	16.48	**15.61**	5.55	644	1103	28.10	9.70	1.94
6	16.88	15.79	6.89	1528	3463	30.30	11.70	2.00
7	28.75	**28.03**	2.59	298	455	31.90	15.20	1.97
8	31.40	30.03	4.57	571	627	34.80	10.50	1.79
9	30.43	28.75	5.87	2081	2484	36.00	7.43	2.00
10	37.06	35.50	4.40	41	50	21.60	6.29	0.59
11	35.28	34.53	2.18	541	541	22.90	1.77	1.67
12	35.32	**33.75**	4.64	1932	1932	26.20	2.15	1.88

3.3.2 Results using buckling curve b

The results of optimization using the buckling curve 'b' of (EN 1993–1–1 2005) are summarized in Table 3.13. When the minimum costs obtained are compared with the minimum costs of the previous Section (see Table 3.8), it can be seen that for the trusses with $h = L/10$, the cost saving is 1.5 % and 2.0 % for the spans 24 m and 36 m, respectively. For the lower trusses (with $h = L/20$), the cost savings for both spans are only 0.3 %.

As in the previous Section, the higher trusses ($h = L/10$) are significantly more economical (30 %, and 22 %).

The optimum topologies are depicted in Figures 3.10–3.13, and the corresponding member data are given in Tables 3.14–3.17.

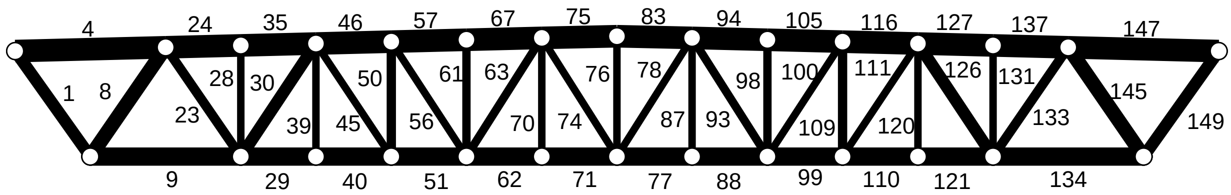


Figure 3.6: Optimum topology for SHS profiles. $L = 24\,\mathrm{m}$, $h = L/10$. $C^* = 1218$€.

Table 3.9: Optimum SHS truss design for $L = 24\,\mathrm{m}$, $h = L/10$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	SHS 90x3.0	52.63	288.78	0.00	0.80	0.00
4	SHS 110x6.0	116.10	-167.91	20.51	0.28	0.82
8	SHS 90x4.0	63.67	-285.46	0.00	0.60	1.00
9	SHS 100x5.0	93.01	329.92	10.25	0.60	0.00
23	SHS 50x4.0	33.50	214.58	0.00	0.87	0.00
24	SHS 110x6.0	58.05	-451.88	5.13	0.50	0.62
28	SHS 40x3.0	18.89	-34.17	0.00	0.23	0.93
29	SHS 100x5.0	46.51	546.72	2.56	0.76	0.00
30	SHS 90x3.0	55.15	-171.23	0.00	0.47	0.79
35	SHS 110x6.0	58.05	-451.88	5.13	0.50	0.62
39	SHS 40x3.0	19.21	-0.00	0.00	0.00	0.00
40	SHS 100x5.0	46.51	546.72	2.56	0.76	0.00
45	SHS 40x3.0	23.09	125.22	0.00	0.84	0.00
46	SHS 110x6.0	58.05	-616.37	5.13	0.68	0.84
50	SHS 60x3.0	30.38	-104.19	0.00	0.44	0.99
51	SHS 100x5.0	46.51	616.18	2.56	0.86	0.00
56	SHS 40x3.0	23.36	82.38	0.00	0.55	0.00
57	SHS 110x6.0	58.05	-661.56	5.13	0.73	0.90
61	SHS 40x4.0	22.84	-34.17	0.00	0.18	0.84
62	SHS 100x5.0	46.51	683.40	2.56	0.96	0.00
63	SHS 50x3.0	30.53	-41.13	0.00	0.21	0.86
67	SHS 110x6.0	58.05	-661.56	5.13	0.73	0.90
70	SHS 40x3.0	20.17	0.00	0.00	0.00	0.00
71	SHS 100x5.0	46.51	683.40	2.56	0.96	0.00
74	SHS 40x3.0	23.90	0.00	0.00	0.00	0.00
75	SHS 110x6.0	58.05	-683.61	5.13	0.75	0.93
76	SHS 40x3.0	10.25	-0.00	0.00	0.00	0.00
Total Cost		1217.51				

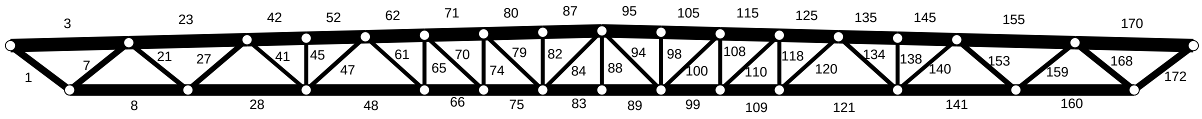


Figure 3.7: Optimum topology for SHS profiles. $L = 24\,\text{m}$, $h = L/20$. $C^* = 1561\,€$.

Table 3.10: Optimum design for $L = 24\,\text{m}$, $h = L/20$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	SHS 110x4.0	44.71	396.81	0.00	0.68	0.00
3	SHS 120x10.0	141.87	-317.55	13.13	0.22	0.35
7	SHS 90x4.0	37.03	-381.13	0.00	0.80	0.97
8	SHS 120x8.0	122.22	615.06	6.56	0.48	0.00
21	SHS 50x5.0	22.00	273.16	0.00	0.92	0.00
23	SHS 120x10.0	141.87	-828.62	13.13	0.57	0.91
27	SHS 90x3.0	32.12	-263.48	0.00	0.73	0.88
28	SHS 120x8.0	122.22	1029.12	6.56	0.80	0.00
41	SHS 50x3.0	17.18	186.92	0.00	0.97	0.00
42	SHS 120x10.0	70.94	-1171.91	3.28	0.72	0.79
45	SHS 50x3.0	11.46	-27.34	0.00	0.14	0.19
47	SHS 60x3.0	21.44	-140.10	0.00	0.60	0.92
48	SHS 120x8.0	122.22	1275.68	6.56	0.99	0.00
52	SHS 120x10.0	70.94	-1171.91	3.28	0.72	0.79
61	SHS 50x3.0	17.61	92.77	0.00	0.48	0.00
62	SHS 120x10.0	70.94	-1345.06	3.28	0.82	0.91
65	SHS 50x3.0	12.11	-62.06	0.00	0.32	0.44
66	SHS 120x8.0	61.11	1344.64	1.64	0.97	0.00
70	SHS 50x3.0	17.83	49.79	0.00	0.26	0.00
71	SHS 120x10.0	70.94	-1381.62	3.28	0.84	0.93
74	SHS 50x3.0	12.44	-33.81	0.00	0.18	0.24
75	SHS 120x8.0	61.11	1381.19	1.64	1.00	0.00
79	SHS 50x3.0	18.06	9.16	0.00	0.05	0.00
80	SHS 120x10.0	70.94	-1388.26	3.28	0.85	0.93
82	SHS 50x3.0	12.76	-27.34	0.00	0.14	0.20
83	SHS 120x8.0	61.11	1366.80	1.64	0.99	0.00
84	SHS 50x3.0	18.52	29.74	0.00	0.15	0.00
87	SHS 120x10.0	70.94	-1388.26	3.28	0.85	0.93
88	SHS 50x3.0	6.55	0.00	0.00	0.00	0.00
Total Cost		1561.18				

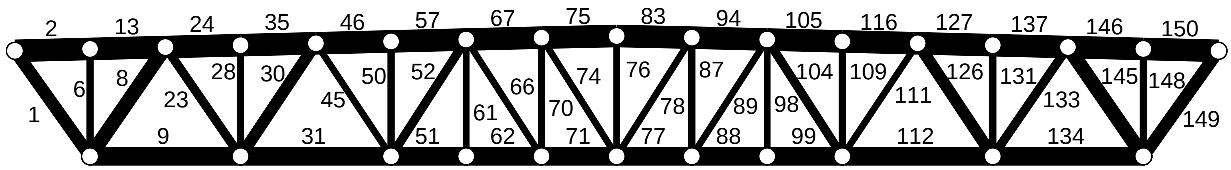


Figure 3.8: Optimum topology for SHS profiles. $L = 36\,\text{m}$, $h = L/10$. $C^* = 2803$€.

Table 3.11: Optimum design for $L = 36\,\text{m}$, $h = L/10$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	SHS 120x4.0	126.43	464.12	0.00	0.72	0.00
2	SHS 140x8.0	136.17	-269.85	11.54	0.18	0.24
6	SHS 60x3.0	42.58	-51.25	0.00	0.22	0.80
8	SHS 120x5.0	149.33	-396.51	0.00	0.50	0.93
9	SHS 140x6.0	225.68	494.88	23.06	0.52	0.00
13	SHS 140x8.0	136.17	-269.85	11.54	0.18	0.24
23	SHS 60x5.0	69.96	321.86	0.00	0.88	0.00
24	SHS 140x8.0	136.17	-677.82	11.54	0.45	0.60
28	SHS 60x3.0	44.08	-51.25	0.00	0.22	0.84
30	SHS 110x4.0	120.90	-256.84	0.00	0.44	0.91
31	SHS 140x6.0	225.68	820.08	23.06	0.86	0.00
35	SHS 140x8.0	136.17	-677.82	11.54	0.45	0.60
45	SHS 50x3.0	44.25	187.83	0.00	0.98	0.00
46	SHS 140x8.0	136.17	-924.56	11.54	0.62	0.82
50	SHS 60x3.0	45.57	-51.25	0.00	0.22	0.89
51	SHS 140x6.0	112.84	992.03	5.77	0.81	0.00
52	SHS 90x3.0	84.64	-124.99	0.00	0.34	0.97
57	SHS 140x8.0	136.17	-924.56	11.54	0.62	0.82
61	SHS 50x3.0	38.05	0.00	0.00	0.00	0.00
62	SHS 140x6.0	112.84	992.03	5.77	0.81	0.00
66	SHS 50x3.0	45.28	61.00	0.00	0.32	0.00
67	SHS 140x8.0	136.17	-1025.42	11.54	0.69	0.91
70	SHS 60x3.0	47.06	-51.25	0.00	0.22	0.94
71	SHS 140x6.0	112.84	1025.10	5.77	0.84	0.00
74	SHS 50x3.0	45.80	-0.00	0.00	0.00	0.00
75	SHS 140x8.0	136.17	-1025.42	11.54	0.69	0.91
76	SHS 50x3.0	19.64	-0.00	0.00	0.00	0.00
Total Cost		2802.82				

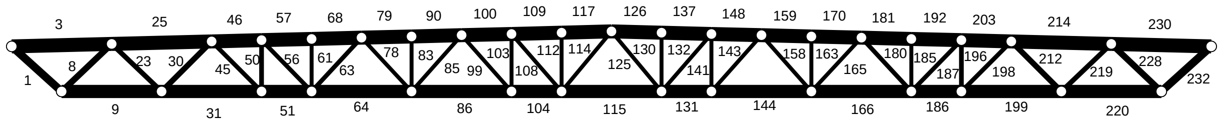


Figure 3.9: Optimum topology for SHS profiles. $L = 36\,\text{m}$, $h = L/20$. $C^* = 3375$€.

Table 3.12: Optimum design for $L = 36\,\text{m}$, $h = L/20$. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	SHS 140x5.0	89.51	546.68	0.00	0.58	0.00
3	SHS 160x10.0	246.43	-406.47	20.51	0.20	0.28
8	SHS 120x4.0	67.57	-530.98	0.00	0.82	0.99
9	SHS 150x10.0	229.09	791.31	10.25	0.39	0.00
23	SHS 80x4.0	43.95	407.07	0.00	0.98	0.00
25	SHS 160x10.0	246.43	-1086.77	20.51	0.52	0.76
30	SHS 100x4.0	57.17	-396.50	0.00	0.75	0.98
31	SHS 150x10.0	229.09	1366.80	10.25	0.67	0.00
45	SHS 80x3.0	38.23	306.44	0.00	0.96	0.00
46	SHS 160x10.0	123.22	-1583.98	5.13	0.69	0.74
50	SHS 60x5.0	27.14	-216.69	0.00	0.59	0.91
51	SHS 150x10.0	114.54	1583.49	2.56	0.73	0.00
56	SHS 60x4.0	33.37	248.92	0.00	0.82	0.00
57	SHS 160x10.0	123.22	-1757.86	5.13	0.76	0.82
61	SHS 60x3.0	20.92	-34.17	0.00	0.15	0.22
63	SHS 80x3.0	39.69	-196.67	0.00	0.61	0.96
64	SHS 150x10.0	229.09	1891.27	10.25	0.92	0.00
68	SHS 160x10.0	123.22	-1757.86	5.13	0.76	0.82
78	SHS 60x3.0	29.25	142.13	0.00	0.61	0.00
79	SHS 160x10.0	123.22	-1988.69	5.13	0.86	0.93
83	SHS 60x3.0	21.91	-34.17	0.00	0.15	0.23
85	SHS 60x3.0	29.99	-93.51	0.00	0.40	0.87
86	SHS 150x10.0	229.09	2050.20	10.25	1.00	0.00
90	SHS 160x10.0	123.22	-1988.69	5.13	0.86	0.93
99	SHS 60x3.0	29.99	44.72	0.00	0.19	0.00
100	SHS 160x10.0	123.22	-2080.56	5.13	0.90	0.98
103	SHS 60x3.0	22.91	-33.43	0.00	0.14	0.23
104	SHS 150x10.0	114.54	2079.91	2.56	0.96	0.00
108	SHS 60x3.0	30.36	-0.96	0.00	0.00	0.01
109	SHS 160x10.0	123.22	-2079.93	5.13	0.90	0.97
112	SHS 60x3.0	23.41	-34.17	0.00	0.15	0.24
114	SHS 60x3.0	31.12	45.43	0.00	0.19	0.00
115	SHS 150x10.0	114.54	2050.20	10.25	1.00	0.00
117	SHS 160x10.0	123.22	-2079.93	5.13	0.90	0.97
Total Cost		3375.06				

Table 3.13: Results of topology optimization for SHS trusses with buckling curve b. C_0 = initial cost; C^* = minimum cost obtained; t^* = runtime when the optimum was found; t_{fin} = runtime at termination (time limit 21600 s); G_0 = initial optimality gap; G^* = optimality gap, when optimum was found; G_{fin} = final optimality gap.

i	C_0 [10^2 €]	C^* [10^2 €]	$\frac{C_0 - C^*}{C^*}$ [%]	t^* [s]	t_{fin}	G_0 [%]	G^* [%]	G_{fin} [%]
1	12.56	**12.00**	4.66	29	363	28.20	15.00	1.80
2	13.56	13.05	3.91	145	848	32.50	18.40	1.89
3	13.19	12.12	8.81	2922	4872	32.90	8.00	1.84
4	16.14	16.08	0.32	270	339	25.30	6.68	1.50
5	16.44	**15.56**	5.64	566	922	28.50	10.70	1.93
6	16.78	15.74	6.62	2517	7984	30.20	13.60	2.00
7	28.71	27.60	4.01	241	816	32.90	13.80	1.98
8	31.01	29.85	3.89	1301	1302	35.30	1.72	1.70
9	29.20	**27.47**	6.32	3398	3412	34.40	2.25	1.99
10	36.99	35.40	4.50	38	50	23.20	5.87	0.87
11	35.20	34.46	2.15	356	371	24.10	3.71	2.00
12	34.80	**33.64**	3.44	1075	1312	26.00	6.00	1.94

3.4 Summary of Topology Optimization

The results of topology optimization are summarized in Table 3.18. Note that for SHS trusses, the summary is based on the results obtained with buckling curve 'b'. The following observations can be made:

- Tubular trusses are 5–10% more economical than trusses with HEA/HEB chords and CHS braces
- The weight of the minimum cost tubular trusses is 4–8% greater, except for the 24 m span truss with $h = L/10$, where tubular truss is 6% lighter.

Table 3.18: Comparison of minimum cost HEA/HEB/CHS and SHS trusses. For SHS trusses, the results obtained using buckling curve 'b' are reported. Costs and weights are given for cost optimized trusses. In addition, the cost and weight ratios are reported.

L [m]	L/h	C_{HEA} [€]	C_{SHS} [€]	$\frac{C_{SHS}}{C_{HEA}}$	W_{HEA} [10^2 kg]	W_{SHS} [10^2 kg]	$\frac{W_{SHS}}{W_{HEA}}$
24	10	1336.96	1200.22	0.90	11.48	10.78	0.94
24	20	1636.08	1556.48	0.95	14.66	15.79	1.08
36	10	2972.95	2746.61	0.92	25.75	27.04	1.05
36	20	3598.56	3364.29	0.93	33.48	34.90	1.04

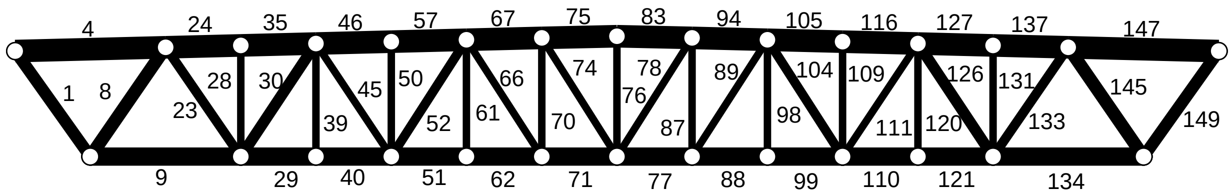


Figure 3.10: Optimum topology for SHS profiles with buckling curve ’b’. $L = 24\,\text{m}$, $h = L/10$. $C^* = 1200$€.

Table 3.14: Optimum SHS truss design for $L = 24\,\text{m}$, $h = L/10$ with buckling curve ’b’. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	SHS 90x3.0	52.63	288.78	0.00	0.80	0.00
4	SHS 110x6.0	116.10	-167.91	20.51	0.28	0.75
8	SHS 100x3.0	60.14	-285.46	0.00	0.70	0.96
9	SHS 100x5.0	93.01	329.92	10.25	0.60	0.00
23	SHS 50x4.0	33.50	214.58	0.00	0.87	0.00
24	SHS 110x6.0	58.05	-451.88	5.13	0.50	0.59
28	SHS 40x3.0	18.89	-34.17	0.00	0.23	0.87
29	SHS 100x5.0	46.51	546.72	2.56	0.76	0.00
30	SHS 80x3.0	48.73	-171.23	0.00	0.54	0.92
35	SHS 110x6.0	58.05	-451.88	5.13	0.50	0.59
39	SHS 40x3.0	19.21	-0.00	0.00	0.00	0.00
40	SHS 100x5.0	46.51	546.72	2.56	0.76	0.00
45	SHS 40x3.0	23.09	125.22	0.00	0.84	0.00
46	SHS 110x6.0	58.05	-616.37	5.13	0.68	0.81
50	SHS 40x3.0	19.53	-34.17	0.00	0.23	0.92
51	SHS 100x5.0	46.51	661.35	2.56	0.93	0.00
52	SHS 60x3.0	36.75	-83.33	0.00	0.36	0.94
57	SHS 110x6.0	58.05	-616.37	5.13	0.68	0.81
61	SHS 40x3.0	19.85	0.00	0.00	0.00	0.00
62	SHS 100x5.0	46.51	661.35	2.56	0.93	0.00
66	SHS 40x3.0	23.63	40.66	0.00	0.27	0.00
67	SHS 110x6.0	58.05	-683.61	5.13	0.75	0.89
70	SHS 40x3.0	20.17	-34.17	0.00	0.23	0.97
71	SHS 100x5.0	46.51	683.40	2.56	0.96	0.00
74	SHS 40x3.0	23.90	-0.00	0.00	0.00	0.00
75	SHS 110x6.0	58.05	-683.61	5.13	0.75	0.89
76	SHS 40x3.0	10.25	-0.00	0.00	0.00	0.00
Total Cost		1200.22				

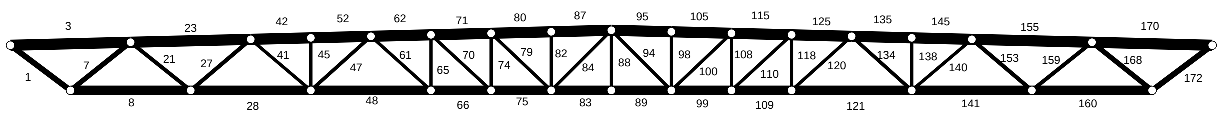


Figure 3.11: Optimum topology for SHS profiles with buckling curve 'b'. $L = 24\,\mathrm{m}$, $h = L/20$. $C^* = 1566$€.

Table 3.15: Optimum SHS truss design for $L = 24\,\mathrm{m}$, $h = L/20$ with buckling curve 'b'. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	SHS 110x4.0	44.71	396.81	0.00	0.68	0.00
3	SHS 120x10.0	141.87	-317.55	13.13	0.22	0.32
7	SHS 90x4.0	37.03	-381.13	0.00	0.80	0.92
8	SHS 120x8.0	122.22	615.06	6.56	0.48	0.00
21	SHS 50x5.0	22.00	273.16	0.00	0.92	0.00
23	SHS 120x10.0	141.87	-828.62	13.13	0.57	0.85
27	SHS 80x3.0	28.38	-263.48	0.00	0.82	0.98
28	SHS 120x8.0	122.22	1029.12	6.56	0.80	0.00
41	SHS 50x3.0	17.18	186.92	0.00	0.97	0.00
42	SHS 120x10.0	70.94	-1171.91	3.28	0.72	0.77
45	SHS 50x3.0	11.46	-27.34	0.00	0.14	0.18
47	SHS 50x4.0	20.47	-140.10	0.00	0.57	0.98
48	SHS 120x8.0	122.22	1275.68	6.56	0.99	0.00
52	SHS 120x10.0	70.94	-1171.91	3.28	0.72	0.77
61	SHS 50x3.0	17.61	92.77	0.00	0.48	0.00
62	SHS 120x10.0	70.94	-1345.06	3.28	0.82	0.88
65	SHS 50x3.0	12.11	-62.06	0.00	0.32	0.41
66	SHS 120x8.0	61.11	1344.64	1.64	0.97	0.00
70	SHS 50x3.0	17.83	49.79	0.00	0.26	0.00
71	SHS 120x10.0	70.94	-1381.62	3.28	0.84	0.91
74	SHS 50x3.0	12.44	-33.81	0.00	0.18	0.23
75	SHS 120x8.0	61.11	1381.19	1.64	1.00	0.00
79	SHS 50x3.0	18.06	9.16	0.00	0.05	0.00
80	SHS 120x10.0	70.94	-1388.26	3.28	0.85	0.91
82	SHS 50x3.0	12.76	-27.34	0.00	0.14	0.19
83	SHS 120x8.0	61.11	1366.80	1.64	0.99	0.00
84	SHS 50x3.0	18.52	29.74	0.00	0.15	0.00
87	SHS 120x10.0	70.94	-1388.26	3.28	0.85	0.91
88	SHS 50x3.0	6.55	0.00	0.00	0.00	0.00
Total Cost		1556.48				

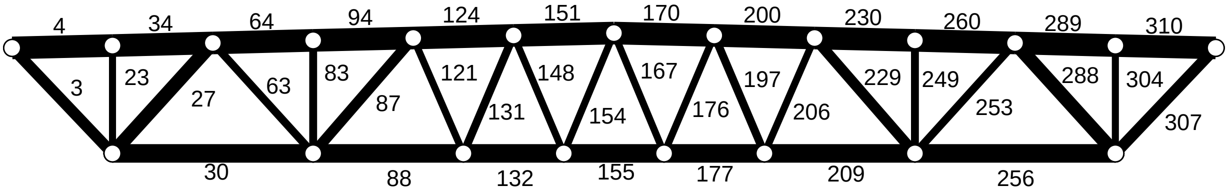


Figure 3.12: Optimum topology for SHS profiles with buckling curve 'b'. $L = 36\,\text{m}$, $h = L/10$. $C^* = 2747$€.

Table 3.16: Optimum SHS truss design for $L = 36\,\text{m}$, $h = L/10$ with buckling curve 'b'. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
3	SHS 110x4.0	129.65	506.99	0.00	0.86	0.00
4	SHS 150x8.0	195.93	-349.76	20.51	0.23	0.34
23	SHS 60x3.0	42.83	-68.34	0.00	0.29	0.99
27	SHS 120x5.0	168.05	-403.80	0.00	0.51	0.98
30	SHS 120x8.0	305.55	621.27	41.00	0.99	0.00
34	SHS 150x8.0	195.93	-349.76	20.51	0.23	0.34
63	SHS 60x4.0	69.28	295.55	0.00	0.97	0.00
64	SHS 150x8.0	195.93	-820.34	20.51	0.54	0.80
83	SHS 60x4.0	52.43	-68.34	0.00	0.23	0.86
87	SHS 100x4.0	123.22	-199.24	0.00	0.38	0.97
88	SHS 120x8.0	229.16	950.82	23.06	0.98	0.00
94	SHS 150x8.0	195.93	-820.34	20.51	0.54	0.80
121	SHS 60x3.0	49.96	84.94	0.00	0.36	0.00
124	SHS 150x8.0	195.93	-984.99	20.51	0.65	0.95
131	SHS 80x3.0	69.04	-84.65	0.00	0.26	0.73
132	SHS 120x8.0	152.78	1017.83	10.25	0.84	0.00
148	SHS 60x3.0	50.88	9.38	0.00	0.04	0.00
151	SHS 150x8.0	195.93	-1021.82	20.51	0.68	0.99
154	SHS 60x3.0	51.80	-9.35	0.00	0.04	0.19
155	SHS 120x8.0	76.39	1025.10	10.25	0.84	0.00
Total Cost		2746.61				

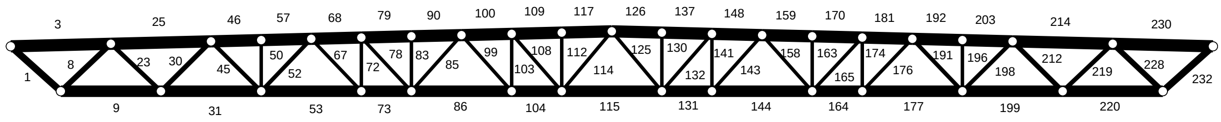


Figure 3.13: Optimum topology for SHS profiles with buckling curve 'b'. $L = 36\,\text{m}$, $h = L/20$. $C^* = 3364$€.

Table 3.17: Optimum SHS truss design for $L = 36\,\text{m}$, $h = L/20$ with buckling curve 'b'. The axial forces and design values of the bending moment are given. Furthermore, the utilization ratio for strength, U_S, and stability, U_B, are reported. Due to symmetry, only the members on the left to the symmetry axis are shown.

i	Profile	C [€]	N [kN]	M_{Ed} [kNm]	U_S	U_B
1	SHS 140x5.0	89.51	546.68	0.00	0.58	0.00
3	SHS 160x10.0	246.43	-406.47	20.51	0.20	0.27
8	SHS 100x5.0	64.15	-530.98	0.00	0.81	0.99
9	SHS 150x10.0	229.09	791.31	10.25	0.39	0.00
23	SHS 70x5.0	43.43	407.07	0.00	0.93	0.00
25	SHS 160x10.0	246.43	-1086.77	20.51	0.52	0.71
30	SHS 100x4.0	57.17	-396.50	0.00	0.75	0.92
31	SHS 150x10.0	229.09	1366.80	10.25	0.67	0.00
45	SHS 60x5.0	37.45	306.44	0.00	0.83	0.00
46	SHS 160x10.0	123.22	-1583.98	5.13	0.69	0.73
50	SHS 60x3.0	20.42	-34.17	0.00	0.15	0.20
52	SHS 90x3.0	44.35	-252.05	0.00	0.70	0.90
53	SHS 150x10.0	229.09	1757.31	10.25	0.86	0.00
57	SHS 160x10.0	123.22	-1583.98	5.13	0.69	0.73
67	SHS 60x3.0	28.89	194.24	0.00	0.83	0.00
68	SHS 160x10.0	123.22	-1891.86	5.13	0.82	0.87
72	SHS 60x3.0	21.42	-140.65	0.00	0.60	0.84
73	SHS 150x10.0	114.54	1891.27	2.56	0.87	0.00
78	SHS 60x3.0	29.25	142.13	0.00	0.61	0.00
79	SHS 160x10.0	123.22	-1988.69	5.13	0.86	0.91
83	SHS 60x3.0	21.91	-34.17	0.00	0.15	0.21
85	SHS 60x3.0	29.99	-93.51	0.00	0.40	0.79
86	SHS 150x10.0	229.09	2050.20	10.25	1.00	0.00
90	SHS 160x10.0	123.22	-1988.69	5.13	0.86	0.91
99	SHS 60x3.0	29.99	44.72	0.00	0.19	0.00
100	SHS 160x10.0	123.22	-2080.56	5.13	0.90	0.96
103	SHS 60x3.0	22.91	-33.43	0.00	0.14	0.21
104	SHS 150x10.0	114.54	2079.91	2.56	0.96	0.00
108	SHS 60x3.0	30.36	-0.96	0.00	0.00	0.01
109	SHS 160x10.0	123.22	-2079.93	5.13	0.90	0.96
112	SHS 60x3.0	23.41	-34.17	0.00	0.15	0.22
114	SHS 60x3.0	31.12	45.43	0.00	0.19	0.00
115	SHS 150x10.0	114.54	2050.20	10.25	1.00	0.00
117	SHS 160x10.0	123.22	-2079.93	5.13	0.90	0.96
Total Cost		3364.29				

References

EN 1990 (2002), *Eurocode – Basis of structural design*, CEN.

EN 1993–1–1 (2005), *Eurocode 3: Design of Steel Structures. Part 1-1: General rules and rules for buildings*, CEN.

EN 1993–1–8 (2005), *Eurocode 3: Design of Steel Structures. Part 1-8: Design of joints*, CEN.