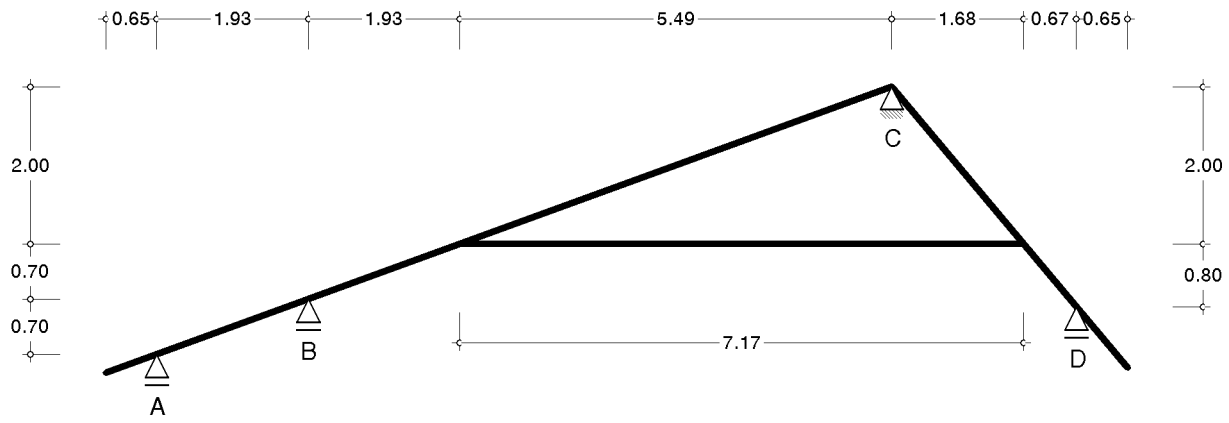
1. System description

1.1. structural system



1.2. system parameters

roof pitches:	left side: 20.00° - right side: 50.00°
apex point:	rafters are hinged connected at apex.
collar beam:	collar beam and rafter are hinged connected.
rafter distan.:	0.700 m
material:	BSH (EC): GL24h
cross-sections:	b/h in cm: left rafter 8.0/18.0, right rafter 8.0/14.0, collar b. 12.0/16.0
notches:	at the support points: A: 3.0cm, B: 3.0cm, C: 4.0cm, D: 4.0cm
roof overhangs:	left side: 0.65 m, right side: 0.65 m
design codes:	Eurocode: EN 1990 (load factors), EN 1991 (wind and snow loads), EN 1995 (timber constr.)
nat. Annex:	NA-DE (Deutschland)

1.3. elastic design values

beam	length	$E_{0,mean}$	h	b	A	I	W
-	m	N/mm²	cm	cm	cm²	cm⁴	cm³
rafter (left s.)	10.642	11500	18.0	8.0	144.0	3888.0	432.0
rafter (right s.)	4.667	11500	14.0	8.0	112.0	1829.3	261.3
collar beam	7.173	11500	16.0	12.0	192.0	4096.0	512.0

1.4. Loading structure

On the left-hand side, the relationship between the actions effects and load cases are shown in a tree structure. The right-hand side shows the characteristics of the superposition to the associated objects on the left-hand.

used symbols: action effect load case

permanent loads	**permanent**
1: dead load	additive (dead load of supporting structure)
2: outer skin	additive (dead load of outer skin)
3: interior finish work(1)	additive (dead load of interior finish work)
man loads	**category H: roofs**
4: man load(1)	alternative (on protruding roof (left side))
5: man load(2)	alternative (on left rafter span 1)
6: man load(3)	alternative (on left rafter span 2)
7: man load(4)	alternative (on left rafter span 3)
8: man load(5)	alternative (on right rafter span 1)
9: man load(6)	alternative (on right rafter span 2)
10: man load(7)	alternative (on protruding roof (right side))

On the left-hand side, the relationship between the actions effects and load cases are shown in a tree structure. The right-hand side shows the characteristics of the superposition to the associated objects on the left-hand.

collar beam live load	**category A: housing and rest rooms**
11: live load	additive (live load on collar beam)
wind loads	**wind loads**
12: wind from left side (1)	alternative (centre area (pressure,pressure))
13: wind from left side (2)	alternative (centre area (pressure,suction))
14: wind from left side (3)	alternative (centre area (suction,suction))
15: wind from left side (4)	alternative (centre area (suction,pressure))
16: wind from left side (7)	alternative (edge region (suction,suction))
17: wind from left side (8)	alternative (edge region (suction,pressure))
18: wind from right side (1)	alternative (centre area (pressure, pressure))
19: wind from right side (2)	alternative (centre area (suction, pressure))
20: wind from right side (3)	alternative (centre area (suction, suction))
21: wind on gable (1)	alternative (edge region)
22: wind on gable (2)	alternative (centre area)
23: wind on gable (3)	alternative (backside area)
snow loads	**locations up to NN+1000m**
24: snow fully	alternative
25: drift left side	alternative
26: drift right side	alternative

2. permanent loads

2.1. load case 1: dead load

dead load of supporting structure

densitiy γ = **5.00 kN/m³** (for all members)

2.2. load case 2: outer skin

dead load of outer skin

load value: q = load sum * distance between rafters = **0.441 kN/m**
(on both rafters)

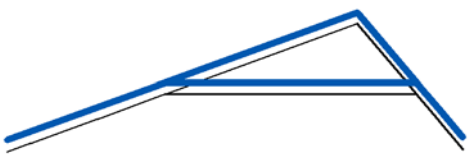
description	value
int. tiles acc. to DIN 456 incl. lathing	0.550 kN/m²
plastic sheet vapour barrier	0.020 kN/m²
6 cm fibre ins. mat. acc. DIN 18 165	0.060 kN/m²
load sum :	**0.630 kN/m²**

2.3. load case 3: interior finish work (1)

load case 3: interior finish work (1)

dead load of interior finish work

load value: q = load sum * distance between rafters = **0.091 kN/m**
(load arrangement: see adjacent sketch)

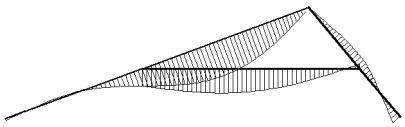


description	value
3/5 lathing	0.030 kN/m²
1,3 cm particleb. DIN 68 763	0.100 kN/m²
load sum :	**0.130 kN/m²**

2.4. Extremal from action effect permanent loads

extremal deflections

deformations perpendicular to the member centre-line
sum of all permanent loads



(max w = 28.5 mm, min w = -7.0 mm)

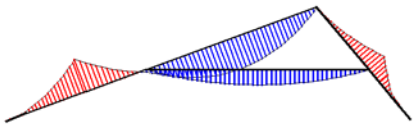
extremal support reactions

sum of all permanent loads in kN

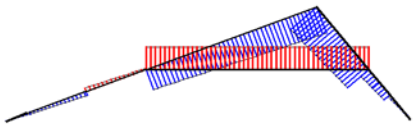
supp.	H	V
A	-	-0.32
B	-	4.11
C	0.00	7.99
D	-	-1.28

extremal internal forces

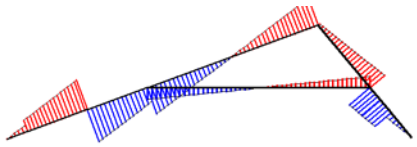
sum of all permanent loads



max M = 2.52 kNm, min M = -2.75 kNm



max N = 6.24 kN, min N = -4.12 kN



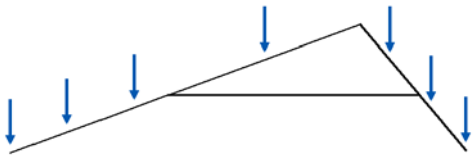
max V = 2.01 kN, min V = -1.85 kN

3. man loads

3.1. Action effect of man loads

Man loads are arranged in the middle of the considered section,
resp. at the cantilevers end. load value: P = 1.00 kN.
The following alternative load cases are analysed.

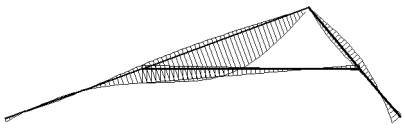
LF	description	explanation
4	man load(1)	on protruding roof (left side)
5	man load(2)	on left rafter span 1
6	man load(3)	on left rafter span 2
7	man load(4)	on left rafter span 3
8	man load(5)	on right rafter span 1
9	man load(6)	on right rafter span 2
10	man load(7)	on protruding roof (right side)



3.2. Extremal from action effect man loads

extremal deflections

deformations perpendicular to the member centre-line
Extremal from all load cases of the action effect man loads



(max w = 10.2 mm, min w = -2.2 mm)

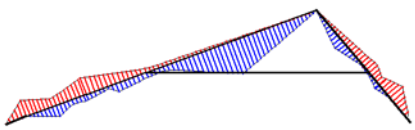
extremal support reactions

Extremal from all load cases of the action effect man loads in kN

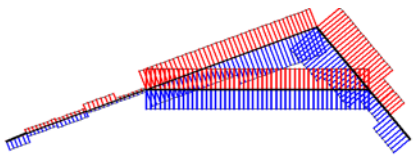
supp.	H		V	
	min	max	min	max
A	-	-	-0.35	1.38
B	-	-	-0.42	1.12
C	0.00	0.00	-1.09	1.39
D	-	-	-0.65	1.87

extremal internal forces

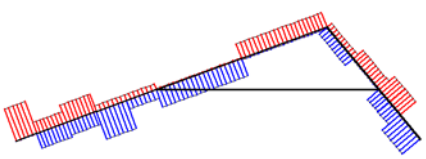
Extremal from all load cases of the action effect man loads



max M = 1.29 kNm, min M = -0.67 kNm



max N = 1.25 kN, min N = -1.11 kN

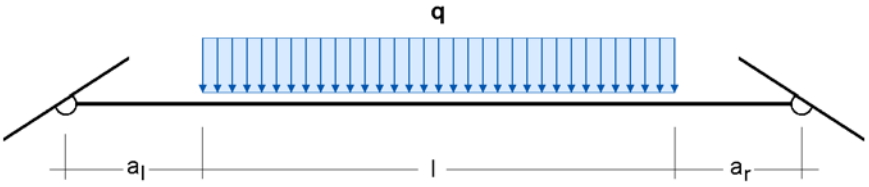


max V = 0.80 kN, min V = -0.94 kN

4. Collar beam live loads

4.1. Action effect from collar beam live loads

The collar beam live loads are arranged as shown in the following sketch.
load value: q = 1.00 kN/m² * distance between rafters = 0.700 kN/m
distances: a1 = 0.00 m, l = 7.17 m, a2 = 0.00 m



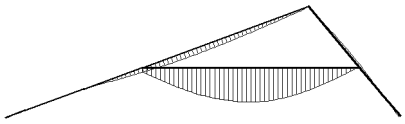
The following additive load cases are analysed.

LF	description
11	live load

4.2. Extremal from action effect collar beam live load

extremal deflections

deformations perpendicular to the member centre-line
Extremal from all load c. of the action eff. collar beam live load



(max w = 53.7 mm, min w = -3.1 mm)

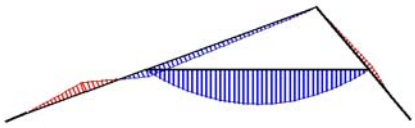
extremal support reactions

Extremal from all load c. of the action eff. collar beam live load in kN

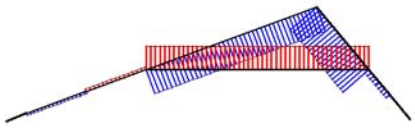
supp.	H		V	
	min	max	min	max
A	-	-	-0.69	0.00
B	-	-	0.00	1.88
C	0.00	0.00	0.00	4.74
D	-	-	-0.90	0.00

extremal internal forces

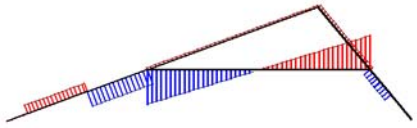
Extremal from all load c. of the action eff. collar beam live load



max M = 4.50 kNm, min M = -1.34 kNm



max N = 4.65 kN, min N = -3.17 kN



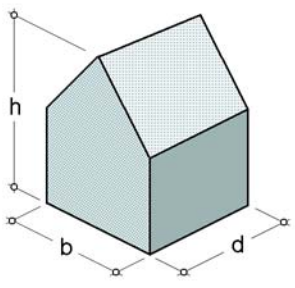
max V = 2.51 kN, min V = -2.51 kN

5. wind loads

5.1. Action effect of wind loads

ground roughness profile acc. to EC 1-1-4\NA-DE: inland

wind zone:	2
h + NN:	60 m
factor:	1.0000
qref:	0.39 kN/m²
h:	5.00 m
b:	13.00 m
d:	12.00 m
⇒ q(h):	**0.58 kN/m²**



The following alternative load cases are analysed.

LF	description	explanation
12	wind from left side (1)	centre area (pressure,pressure)
13	wind from left side (2)	centre area (pressure,suction)
14	wind from left side (3)	centre area (suction,suction)
15	wind from left side (4)	centre area (suction,pressure)
16	wind from left side (7)	edge region (suction,suction)
17	wind from left side (8)	edge region (suction,pressure)
18	wind from right side (1)	centre area (pressure, pressure)
19	wind from right side (2)	centre area (suction, pressure)
20	wind from right side (3)	centre area (suction, suction)
21	wind on gable (1)	edge region
22	wind on gable (2)	centre area
23	wind on gable (3)	backside area

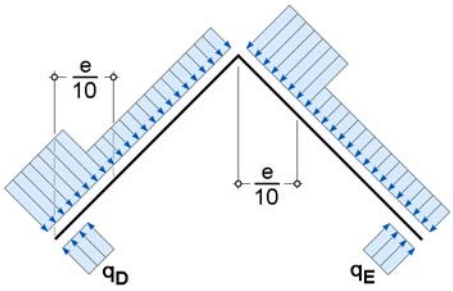
wind from left side

The arrows in the adjacent sketch represent positive load directions (pressure).
In case of negative q-values (suction) the load acts in the reverse direction.

$e = \min(d, 2h) = 10.00 \quad \Rightarrow \quad \frac{e}{10} = 1.00$

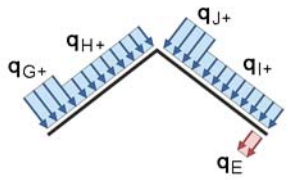
EC 1-1-4	Tabel 7.4a					Tabel 7.1	
Input value	α = 20.00°			α = 50.00°		h/b = 0.38	
Zone	F	G	H	I	J	D	E
$c_{pe.10}$ (-)	-0.77	-0.70	-0.27	-0.20	-0.30	+0.72	-0.34
q()- kN/m	**-0.31**	**-0.29**	**-0.11**	**-0.08**	**-0.12**	**+0.29**	**-0.14**
$c_{pe.10}$ (+)	+0.37	+0.37	+0.27	+0.00	+0.00		
q()+ kN/m	**+0.15**	**+0.15**	**+0.11**	+0.00	+0.00		

$q = c_{pe.10} * q(h) *$ distance between rafters in kN/m

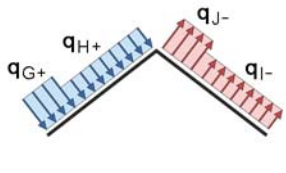


q_D and q_E are acting only in the case of roof overhangs if they increase the local effect unfavorably

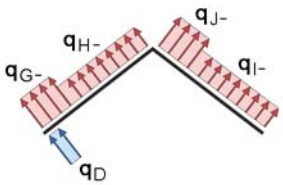
considered load cases (wind from left side)



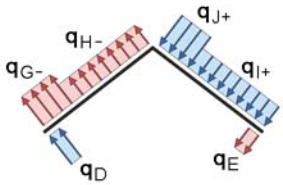
wind from left side (1)
centre area (pressure,pressure)



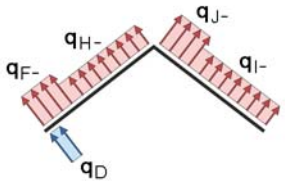
wind from left side (2)
centre area (pressure,suction)



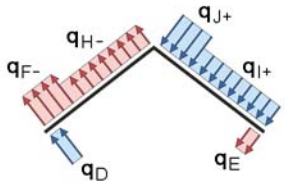
wind from left side (3)
centre area (suction,suction)



wind from left side (4)
centre area (suction,pressure)



wind from left side (7)
edge region (suction,suction)



wind from left side (8)
edge region (suction,pressure)

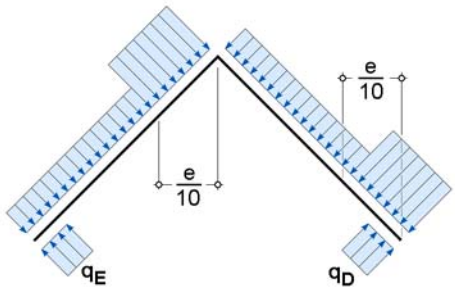
wind from right side

The arrows in the adjacent sketch represent positive load directions (pressure).
In case of negative q-values (suction) the load acts in the reverse direction.

$e = \min(d, 2h) = 10.00 \quad \Rightarrow \quad \frac{e}{10} = 1.00$

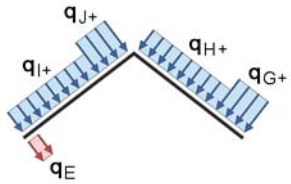
EC 1-1-4	Tabel 7.4a					Tabel 7.1	
Input value	α = 50.00°			α = 20.00°		h/b = 0.38	
Zone	F	G	H	I	J	D	E
$c_{pe.10}$ (-)	+0.00	+0.00	+0.00	-0.40	-0.83	+0.72	-0.34
q()- kN/m	+0.00	+0.00	+0.00	**-0.16**	**-0.34**	**+0.29**	**-0.14**
$c_{pe.10}$ (+)	+0.70	+0.70	+0.63	+0.00	+0.00		
q()+ kN/m	**+0.29**	**+0.29**	**+0.26**	+0.00	+0.00		

$q = c_{pe.10} * q(h) *$ distance between rafters in kN/m

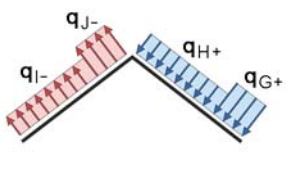


q_D and q_E are acting only in the case of roof overhangs, if they increase the local effect unfavorably

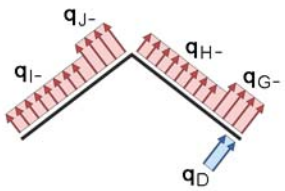
considered load cases (wind from right side)



wind from right side (1)
centre area (pressure, pressure)



wind from right side (2)
centre area (suction, pressure)

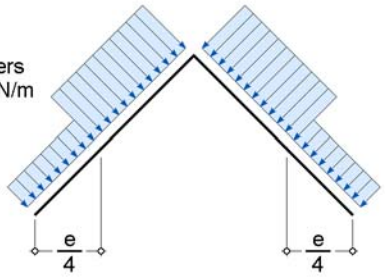


wind from right side (3)
centre area (suction, suction)

wind on gable

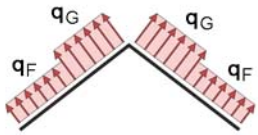
The arrows in the adjacent sketch represent positive load directions (pressure).
In case of negative q-values (suction) the load acts in the reverse direction.

e = min(d,2h) = 10.00 m ⇒ $\frac{e}{4}$ = 2.50 m q = $c_{pe.10}$ * q(h) * distance between rafters in kN/m

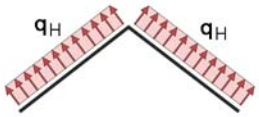


EC 1-1-4	Tabel 7.4b							
Input value	left rafter α = 20.00°				right rafter α = 50.00°			
Zone	F	G	H	I	F	G	H	I
$c_{pe.10}$	-1.23	-1.33	-0.67	-0.50	-1.10	-1.33	-0.87	-0.50
q kN/m	-0.51	-0.55	-0.27	-0.20	-0.45	-0.55	-0.35	-0.20

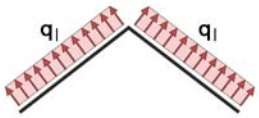
considered load cases (wind on gable)



wind on gable (1)
edge region



wind on gable (2)
centre area

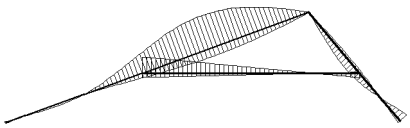


wind on gable (3)
backside area

5.2. Extremal from action effect wind loads

extremal deflections

deformations perpendicular to the member centre-line
Extremal from all load cases of the action effect wind loads



(max w = 5.8 mm, min w = -24.7 mm)

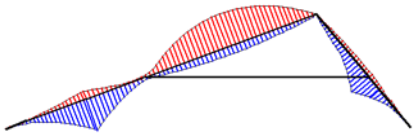
extremal support reactions

Extremal from all load cases of the action effect wind loads in kN

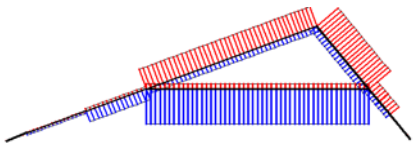
supp.	H		V	
	min	max	min	max
A	-	-	-0.50	0.22
B	-	-	-3.32	0.86
C	-1.62	0.75	-4.38	0.67
D	-	-	-0.65	1.49

extremal internal forces

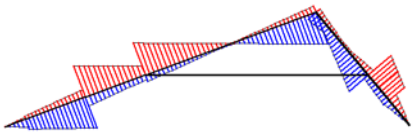
Extremal from all load cases of the action effect wind loads



max M = 2.21 kNm, min M = -2.27 kNm



max N = 3.39 kN, min N = -2.78 kN



max V = 1.58 kN, min V = -1.62 kN

6. snow loads

6.1. Action effect of snow loads

snow load zone:	1
h + NN:	60 m
⇒ s_k:	0.65 kN/m²
building model:	free-standing

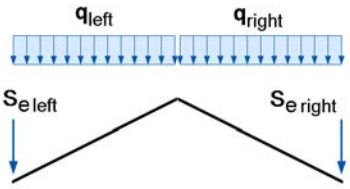
The following alternative load cases are analysed.

LF	description
24	snow fully
25	drift left side
26	drift right side

6.2. load case 24: snow fully

left side: $\alpha = 20° \rightarrow \mu_1(\alpha) = 0.80 \rightarrow q_{left} = a\ \mu_1(\alpha)\ s_k = 0.36$ kN/m

right side: $\alpha = 50° \rightarrow \mu_1(\alpha) = 0.27 \rightarrow q_{right} = a\ \mu_1(\alpha)\ s_k = 0.12$ kN/m



point loads (only in case of roof overhangs)

$S_{e\,left} = 0.4\ a\ (\mu_1(\alpha)\ s_k)^2/\gamma = 0.03$ kN

$S_{e\,right} = 0.4\ a\ (\mu_1(\alpha)\ s_k)^2/\gamma = 0.00$ kN

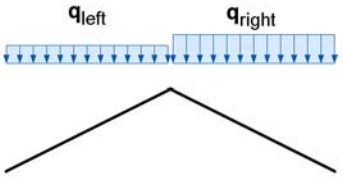
a = distance between rafters, $\gamma = 3.0$ kN/m³

load determination acc. to DIN 1055-5, par. 4.2.3 and 5.1, and Musterliste der techn. Baubestimmungen Feb. 2007 as well as EC 1-1-3 (/NA-DE)

6.3. load case 25: drift left side

left side: $\alpha = 20° \rightarrow \mu_1(\alpha) = 0.80 \rightarrow q_{left} = \frac{1}{2}\ a\ \mu_1(\alpha)\ s_k = 0.18$ kN/m

right side: $\alpha = 50° \rightarrow \mu_1(\alpha) = 0.27 \rightarrow q_{right} = a\ \mu_1(\alpha)\ s_k = 0.12$ kN/m



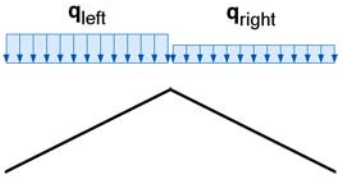
a = distance between rafters

load determination acc. to DIN 1055-5, par. 4.2.3, as well as EC 1-1-3 (/NA-DE)

6.4. load case 26: drift right side

left side: $\alpha = 20° \rightarrow \mu_1(\alpha) = 0.80 \rightarrow q_{left} = a\ \mu_1(\alpha)\ s_k = 0.36$ kN/m

right side: $\alpha = 50° \rightarrow \mu_1(\alpha) = 0.27 \rightarrow q_{right} = \frac{1}{2}\ a\ \mu_1(\alpha)\ s_k = 0.06$ kN/m



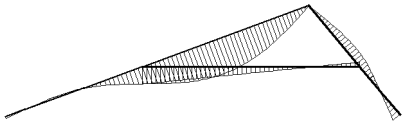
a = distance between rafters

load determination acc. to DIN 1055-5, par. 4.2.3, as well as EC 1-1-3 (/NA-DE)

6.5. Extremal from action effect snow loads

extremal deflections

deformations perpendicular to the member centre-line
Extremal from all load cases of the action effect snow loads



(max w = 15.7 mm, min w = -4.1 mm)

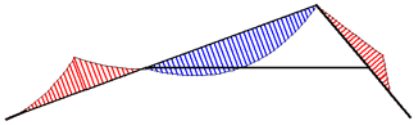
extremal support reactions

Extremal from all load cases of the action effect snow loads in kN

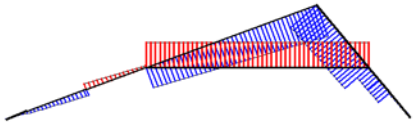
supp.	H		V	
	min	max	min	max
A	-	-	-0.14	0.00
B	-	-	0.00	2.23
C	0.00	0.00	0.00	3.00
D	-	-	-1.14	0.00

extremal internal forces

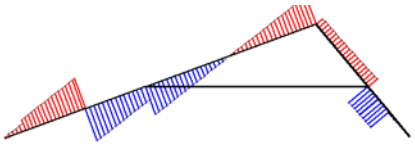
Extremal from all load cases of the action effect snow loads



max M = 1.40 kNm, min M = -1.49 kNm



max N = 2.08 kN, min N = -1.53 kN



max V = 1.08 kN, min V = -1.02 kN

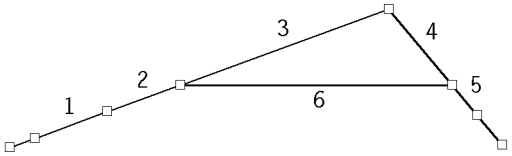
7. Verifications

7.1. Verification of ultimate limit state

7.1.1. stability

System stability is calculated within the verification of load-carrying capacity using the method of fictitious bars
(not in the range of notches)

sections



β = coeff. of eff. column length, l_{ef} = fict. bar length, k_c = instab. factor acc. to EC5(1) 6.3.2.

section	length	β	l_{ef}	⇒ k_c
1	2.05	1.00	2.05	0.9500
2	2.05	1.00	2.05	0.9500
3	5.85	1.00	5.85	0.2920
4	2.61	1.00	2.61	0.7441
5	1.05	1.00	1.05	0.9867
6	7.17	1.00	7.17	0.1568

7.1.2. main verification

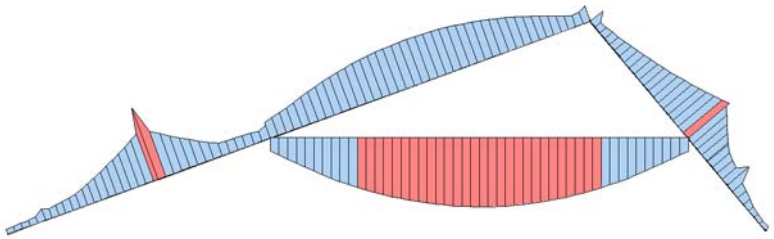
verification of load-carrying capacity for permanent and transient design situations

service class of building	1
material safety factor	1.30
net cross-sections	notches considered
combination of internal forces	acc. to Eurocode

safety and combination coefficients, classes of duration of load

action effect	γ_{sup}	γ_{inf}	Ψ_{dom}	Ψ_{sub}	KLED	k_{mod}
permanent loads	1.35	1.00	1.00	1.00	permanent	0.60
man loads	1.50	0.00	1.00	0.00	sh.-term	0.90
collar b. live ld.	1.50	0.00	1.00	0.70	med.-term	0.80
wind loads	1.50	0.00	1.00	0.60	sh.-v.sh.	1.00
snow loads	1.50	0.00	1.00	0.50	sh.-term	0.90

7.1.2.1. maximal utilization



beam	max U
left rafter	1.48
right rafter	1.05
collar b.	1.34

7.1.2.2. formal proofs of the maximum utilisation

formal verification of maximal utilization in left rafter

decisive point at ξ = 2.74 m (ξ runs from bottom left side to top right side)
decisive internal forces and moments: min N, zug M, zug V
decisive state of stress: minimal normal stress
leading traffic effect: snow loads

action effect	γ	Ψ	factor	N_k	M_k	V_k	N_d	M_d	V_d
permanent loads	1.35	1.00	1.35	-0.73	-2.75	2.01	-0.99	-3.71	2.72
man loads	0.00	0.00	0.00	-0.29	-0.51	0.80	0.00	0.00	0.00
collar beam live load	1.50	0.70	1.05	-0.41	-1.34	1.11	-0.43	-1.41	1.17
wind loads	0.00	0.60	0.00	-0.28	-0.58	0.43	0.00	0.00	0.00
snow loads	1.50	1.00	1.50	-0.39	-1.49	1.08	-0.59	-2.23	1.62
sum							-2.01	-7.34	5.51
moment due to eccentr. of engraving c = 3.0 cm: $\Delta M_d = N_d$ c/2 =								-0.03	
design internal forces and moments in kN, kNm:							-2.01	-7.37	5.51

section properties: h = 18.0 cm, b = 8.0 cm — A = 0.0120000 m²
notch: c = 3.0 cm, h_{net} = 15.0 cm — W = 0.0003000 m³

stresses:
$\sigma_{c0,d} = N_d / A = -0.17$ N/mm²
$\sigma_{m,d} = M_d / W = -24.57$ N/mm²
$\tau_d = 1.5\ V_d / A = 0.69$ N/mm²

characteristic strengths: $f_{c0,k}$ = 24.00, $f_{m,k}$ = 24.00 N/mm²
design values with γ_M = 1.30 and k_{mod} = 0.90: $f_{c0,d}$ = 16.62, $f_{m,d}$ = 16.62 N/mm²

max utilization $U = \left(\frac{\sigma_{c0,d}}{f_{c0,d}}\right)^2 + \frac{|\sigma_{m,d}|}{f_{m,d}} = \underline{\underline{1.48 > 1.00}}$

formal verification of maximal utilization in right rafter

decisive point at ξ = 2.61 m (ξ runs from top left side to bottom right side)
decisive internal forces and moments: max N, zug M, zug V
decisive state of stress: maximal normal stress
leading traffic effect: snow loads

action effect	γ	Ψ	factor	N_k	M_k	V_k	N_d	M_d	V_d
permanent loads	1.35	1.00	1.35	5.07	-1.66	-1.13	6.84	-2.24	-1.52
man loads	0.00	0.00	0.00	0.99	-0.44	-0.17	0.00	0.00	0.00
collar beam live load	1.50	0.70	1.05	4.65	-0.61	-0.23	4.88	-0.64	-0.24
wind loads	0.00	0.60	0.00	0.68	-0.26	0.03	0.00	0.00	0.00
snow loads	1.50	1.00	1.50	1.93	-0.83	-0.38	2.89	-1.24	-0.57
sum (= design internal forces and moments in kN, kNm)							14.61	-4.11	-2.34

section properties: h = 14.0 cm, b = 8.0 cm — A = 0.0112000 m², W = 0.0002613 m³

stresses:
$\sigma_{t0,d} = N_d / A = 1.30$ N/mm²
$\sigma_{m,d} = M_d / W = -15.73$ N/mm²
$\tau_d = 1.5\ V_d / A = -0.31$ N/mm²

characteristic strengths: $f_{t0,k}$ = 19.20, $f_{m,k}$ = 24.00 N/mm²
design values with γ_M = 1.30 and k_{mod} = 0.90: $f_{t0,d}$ = 13.29, $f_{m,d}$ = 16.62 N/mm²

max utilization $U = \frac{\sigma_{t0,d}}{f_{t0,d}} + \frac{|\sigma_{m,d}|}{f_{m,d}} = \underline{\underline{1.05 > 1.00}}$

formal verification of maximal utilization in collar beam

decisive point at ξ = 3.59 m (ξ runs from left side to right side)
decisive internal forces and moments: min N, zug M, zug V
decisive state of stress: minimal normal stress
leading traffic effect: collar beam live load

action effect	γ	Ψ	factor	N_k	M_k	V_k	N_d	M_d	V_d
permanent loads	1.35	1.00	1.35	-4.12	1.20	0.00	-5.56	1.62	0.00
man loads	0.00	0.00	0.00	-0.76	0.00	0.00	0.00	0.00	0.00
collar beam live load	1.50	1.00	1.50	-3.17	4.50	0.00	-4.75	6.75	0.00
wind loads	0.00	0.60	0.00	-0.52	0.00	0.00	0.00	0.00	0.00
snow loads	0.00	0.50	0.00	-1.53	0.00	0.00	0.00	0.00	0.00
sum	(= design internal forces and moments in kN, kNm)						-10.31	8.38	0.00

section properties: h = 16.0 cm, b = 12.0 cm
A = 0.0192000 m²
W = 0.0005120 m³

stresses:
$\sigma_{c0,d} = N_d / A = -0.54$ N/mm²
$\sigma_{m,d} = M_d / W = 16.36$ N/mm²
$\tau_d = 1.5\ V_d / A = 0.00$ N/mm²

characteristic strengths $f_{c0,k} = 24.00$, $f_{m,k} = 24.00$ N/mm²
design values with $\gamma_M = 1.30$ and $k_{mod} = 0.80$: $f_{c0,d} = 14.77$, $f_{m,d} = 14.77$ N/mm²

max utilization $U = \frac{|\sigma_{c0,d}|}{k_c\ f_{c0,d}} + \frac{|\sigma_{m,d}|}{f_{m,d}} = 1.34 > 1.00$ (k_c = 0.1568)

7.1.3. Verification of fire protection

verification of fire protection

fire load for rafters	four-sided
fire load for collar beam	four-sided
formation of the collar beam	single-section
required fire resistance period	25 minutes
combustion depth	2.00 cm
verification method	with reduced cross-section acc. to EC5 1-2 (4.2.2)
material safety factor	1.00
combination of internal forces	acc. to Eurocode EN 1990

imaginary cross-sections and strength values (simplified method)

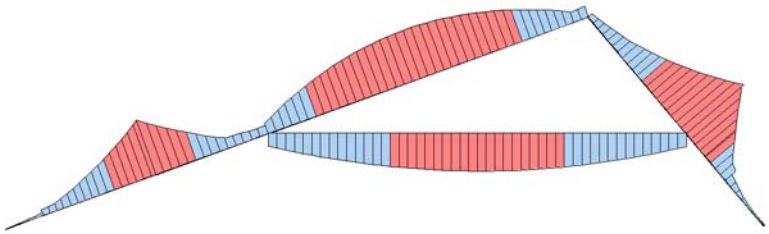
	h_i cm	b_i cm	$f_{m,d}$ N/mm²	$f_{c0,d}$ N/mm²	$f_{t,d}$ N/mm²
rafter (left s)	12.60	2.60	27.60	27.60	22.08
rafter (right s)	8.60	2.60	27.60	27.60	22.08
collar b.	10.60	6.60	27.60	27.60	22.08

partial safety factors and combination coefficients

action effect	γ_{sup}	γ_{inf}	Ψ_{dom}	Ψ_{sub}
permanent loads	1.00	1.00	1.00	1.00
man loads	1.00	0.00	0.00	0.00
collar b. live ld.	1.00	0.00	0.30	0.30
wind loads	1.00	0.00	0.20	0.00
snow loads	1.00	0.00	0.00	0.00

The buckling coefficients k_c according to DIN1052 10.3 are determined for the fire protection verification from the remaining cross-section according to DIN 4102-22 5.5.2.2, taking into account the reduced strength and stiffness parameters. The substitute bar length is assumed to be the same as for the main proof.

7.1.3.1. maximal utilization



beam	max U
left rafter	1.73
right rafter	2.28
collar b.	1.13

7.1.3.2. formal proofs of the maximum utilisation

formal verification of maximal utilization in left rafter

decisive point at ξ = 2.74 m (ξ runs from bottom left side to top right side)
decisive internal forces and moments: min N, zug M, zug V
decisive state of stress: minimal normal stress
leading traffic effect: wind loads

action effect	γ	Ψ	factor	N_k	M_k	V_k	N_d	M_d	V_d
permanent loads	1.00	1.00	1.00	-0.73	-2.75	2.01	-0.73	-2.75	2.01
man loads	0.00	0.00	0.00	-0.29	-0.51	0.80	0.00	0.00	0.00
collar beam live load	1.00	0.30	0.30	-0.41	-1.34	1.11	-0.12	-0.40	0.33
wind loads	1.00	0.20	0.20	-0.28	-0.58	0.43	-0.06	-0.12	0.09
snow loads	0.00	0.00	0.00	-0.39	-1.49	1.08	0.00	0.00	0.00
sum	(= design internal forces and moments in kN, kNm)						-0.91	-3.26	2.43

section properties: h = 12.6 cm, b = 2.6 cm
A = 0.0032760 m²
W = 0.0000688 m³

stresses:
$\sigma_{c0,d} = N_d / A = -0.28$ N/mm²
$\sigma_{m,d} = M_d / W = -47.43$ N/mm²
$\tau_d = 1.5\ V_d / A = 1.11$ N/mm²

max utilization $U = \frac{|\sigma_{c0,d}|}{k_c\ f_{c0,d}} + \frac{|\sigma_{m,d}|}{f_{m,d}} = \underline{\underline{1.73 > 1.00}}$ (k_c = 0.8425)

formal verification of maximal utilization in right rafter

decisive point at ξ = 2.61 m (ξ runs from top left side to bottom right side)
decisive internal forces and moments: min M, zug N, zug V
decisive state of stress: minimal normal stress
leading traffic effect: wind loads

action effect	γ	Ψ	factor	N_k	M_k	V_k	N_d	M_d	V_d
permanent loads	1.00	1.00	1.00	5.07	-1.66	-1.13	5.07	-1.66	-1.13
man loads	0.00	0.00	0.00	0.99	-0.44	-0.17	0.00	0.00	0.00
collar beam live load	1.00	0.30	0.30	4.65	-0.61	-0.23	1.40	-0.18	-0.07
wind loads	1.00	0.20	0.20	0.35	-0.29	-0.11	0.07	-0.06	-0.02
snow loads	0.00	0.00	0.00	1.93	-0.83	-0.38	0.00	0.00	0.00
sum	(= design internal forces and moments in kN, kNm)						6.53	-1.90	-1.22

section properties: h = 8.6 cm, b = 2.6 cm
A = 0.0022360 m²
W = 0.0000320 m³

stresses:
$\sigma_{t0,d} = N_d / A = 2.92$ N/mm²
$\sigma_{m,d} = M_d / W = -59.16$ N/mm²
$\tau_d = 1.5\ V_d / A = -0.82$ N/mm²

max utilization $U = \frac{\sigma_{t0,d}}{f_{t0,d}} + \frac{|\sigma_{m,d}|}{f_{m,d}} = \underline{\underline{2.28 > 1.00}}$

formal verification of maximal utilization in collar beam

decisive point at ξ = 3.59 m (ξ runs from left side to right side)
decisive internal forces and moments: min N, zug M, zug V
decisive state of stress: minimal normal stress
leading traffic effect: wind loads

action effect	γ	Ψ	factor	N_k	M_k	V_k	N_d	M_d	V_d
permanent loads	1.00	1.00	1.00	-4.12	1.20	0.00	-4.12	1.20	0.00
man loads	0.00	0.00	0.00	-0.76	0.00	0.00	0.00	0.00	0.00
collar beam live load	1.00	0.30	0.30	-3.17	4.50	0.00	-0.95	1.35	0.00
wind loads	1.00	0.20	0.20	-0.52	0.00	0.00	-0.10	0.00	0.00
snow loads	0.00	0.00	0.00	-1.53	0.00	0.00	0.00	0.00	0.00
sum	(= design internal forces and moments in kN, kNm)						-5.17	2.55	0.00

section properties: h = 10.6 cm, b = 6.6 cm
A = 0.0069960 m²
W = 0.0001236 m³

stresses:
$\sigma_{c0,d} = N_d / A = -0.74$ N/mm²
$\sigma_{m,d} = M_d / W = 20.66$ N/mm²
$\tau_d = 1.5\ V_d / A = 0.00$ N/mm²

max utilization $U = \frac{|\sigma_{c0,d}|}{k_c\ f_{c0,d}} + \frac{|\sigma_{m,d}|}{f_{m,d}} = \underline{\underline{1.13 > 1.00}}$ (k_c = 0.0699)

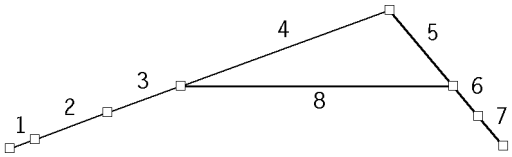
7.2. verifications of serviceability limit states

7.2.1. comparative lengths

for calculation of degree of utilization

section	length	l_v
-	m	m
1	0.69	0.69
2	2.05	2.05
3	2.05	2.05
4	5.85	5.85
5	2.61	2.61
6	1.05	1.05
7	1.01	1.01
8	7.17	7.17

sections



7.2.2. limit values

deformation	(in span)	(at cantilever)
Winst	l_v/300	l_v/150
Wfin	l_v/200	l_v/100
Wnet,fin	l_v/300	l_v/150

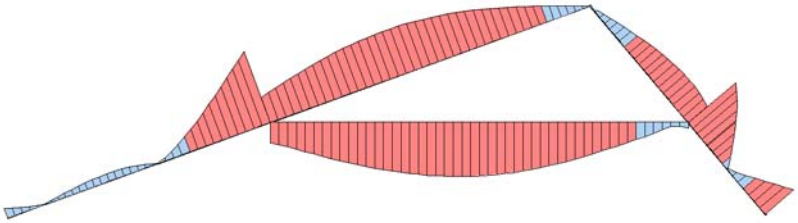
7.2.3. design situation Winst

combination coefficients

action effect	Ψ_0
man loads	0.00
collar b. live ld.	0.70
wind loads	0.60
snow loads	0.50

service class 1
$\Rightarrow k_{def}$ = 0.60

7.2.3.1. maximal utilization



beam	max U
left rafter	4.42
right rafter	3.62
collar b.	3.16

7.2.3.2. formal proofs of the maximum utilisation

formal verification of maximal utilization in left rafter

decisive point at ξ = 4.79 m (ξ runs from bottom left side to top right side)
decisive deformation: max w
decisive comparative length: 2.05 m
leading traffic effect: snow loads

action effect	γ	Ψ	F	w_k	w_{inst}
permanent loads	1.00	1.00	1.00	15.21	15.21
man loads	0.00	0.00	0.00	4.46	0.00
collar beam live load	1.00	0.70	0.70	6.87	4.81
wind loads	1.00	0.60	0.60	3.22	1.93
snow loads	1.00	1.00	1.00	8.27	8.27
sum					30.23

$$\text{max utilization U} = \frac{w_{inst}}{l_v / 300} = \underline{\underline{4.42 > 1.00}}$$

formal verification of maximal utilization in right rafter

decisive point at ξ = 2.61 m (ξ runs from top left side to bottom right side)
decisive deformation: min w
decisive comparative length: 1.05 m
leading traffic effect: snow loads

action effect	γ	Ψ	F	w_k	w_{inst}
permanent loads	1.00	1.00	1.00	-6.33	-6.33
man loads	0.00	0.00	0.00	-1.91	0.00
collar beam live load	1.00	0.70	0.70	-2.71	-1.90
wind loads	1.00	0.60	0.60	-1.41	-0.85
snow loads	1.00	1.00	1.00	-3.53	-3.53
sum					-12.60

$$\text{max utilization U} = \frac{w_{inst}}{l_v / 300} = \underline{\underline{3.62 > 1.00}}$$

formal verification of maximal utilization in collar beam

decisive point at ξ = 3.14 m (ξ runs from left side to right side)
decisive deformation: max w
decisive comparative length: 7.17 m
leading traffic effect: collar beam live load

action effect	γ	Ψ	F	w_k	w_{inst}
permanent loads	1.00	1.00	1.00	19.76	19.76
man loads	0.00	0.00	0.00	1.83	0.00
collar beam live load	1.00	1.00	1.00	53.20	53.20
wind loads	1.00	0.60	0.60	1.31	0.78
snow loads	1.00	0.50	0.50	3.40	1.70
sum					75.44

max utilization U = $\frac{w_{inst}}{l_v / 300}$ = 3.16 > 1.00

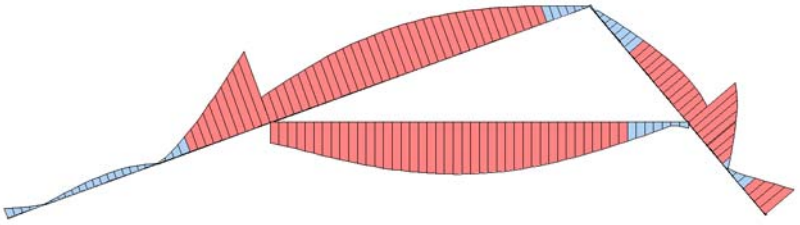
7.2.4. design situation w_{fin}

combination coefficients

action effect	Ψ_0	Ψ_2
man loads	0.00	0.00
collar b. live ld.	0.70	0.30
wind loads	0.60	0.00
snow loads	0.50	0.00

service class 1
$\Rightarrow k_{def}$ = 0.60

7.2.4.1. maximal utilization



beam	max U
left rafter	3.96
right rafter	3.23
collar b.	2.70

7.2.4.2. formal proofs of the maximum utilisation

formal verification of maximal utilization in left rafter

decisive point at ξ = 4.79 m (ξ runs from bottom left side to top right side)
decisive deformation: max w
decisive comparative length: 2.05 m
leading traffic effect: snow loads

Explanation of the table w_k = characteristic deflection
$F = \gamma(\Psi + \Psi_2 k_{def})$, $w_{fin} = F\, w_k$ (all deflections in mm)

action effect	γ	Ψ	$\Psi_2 k_{def}$	F	w_k	w_{fin}
permanent loads	1.00	1.00	0.60	1.60	15.21	24.34
man loads	0.00	0.00	0.00	0.00	4.46	0.00
collar beam live load	1.00	0.70	0.18	0.88	6.87	6.05
wind loads	1.00	0.60	0.00	0.60	3.22	1.93
snow loads	1.00	1.00	0.00	1.00	8.27	8.27
sum						40.60

max utilization U = $\frac{w_{fin}}{l_v / 200}$ = 3.96 > 1.00

formal verification of maximal utilization in right rafter

decisive point at ξ = 2.61 m (ξ runs from top left side to bottom right side)
decisive deformation: min w
decisive comparative length: 1.05 m
leading traffic effect: snow loads

Explanation of the table w_k = characteristic deflection
$F = \gamma(\Psi+\Psi_2 k_{def})$, $w_{fin} = F\, w_k$ (all deflections in mm)

action effect	γ	Ψ	$\Psi_2 k_{def}$	F	w_k	w_{fin}
permanent loads	1.00	1.00	0.60	1.60	-6.33	-10.13
man loads	0.00	0.00	0.00	0.00	-1.91	0.00
collar beam live load	1.00	0.70	0.18	0.88	-2.71	-2.38
wind loads	1.00	0.60	0.00	0.60	-1.41	-0.85
snow loads	1.00	1.00	0.00	1.00	-3.53	-3.53
sum						-16.89

max utilization $U = \dfrac{w_{fin}}{l_v / 200} = \underline{\underline{3.23 > 1.00}}$

formal verification of maximal utilization in collar beam

decisive point at ξ = 3.14 m (ξ runs from left side to right side)
decisive deformation: max w
decisive comparative length: 7.17 m
leading traffic effect: collar beam live load

Explanation of the table w_k = characteristic deflection
$F = \gamma(\Psi+\Psi_2 k_{def})$, $w_{fin} = F\, w_k$ (all deflections in mm)

action effect	γ	Ψ	$\Psi_2 k_{def}$	F	w_k	w_{fin}
permanent loads	1.00	1.00	0.60	1.60	19.76	31.61
man loads	0.00	0.00	0.00	0.00	1.83	0.00
collar beam live load	1.00	1.00	0.18	1.18	53.20	62.77
wind loads	1.00	0.60	0.00	0.60	1.31	0.78
snow loads	1.00	0.50	0.00	0.50	3.40	1.70
sum						96.88

max utilization $U = \dfrac{w_{fin}}{l_v / 200} = \underline{\underline{2.70 > 1.00}}$

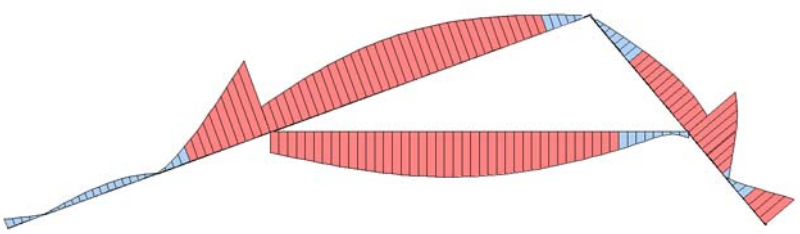
7.2.5. design situation $w_{net,fin}$

combination coefficients

action effect	Ψ_2
man loads	0.00
collar b. live ld.	0.30
wind loads	0.00
snow loads	0.00

service class 1
$\Rightarrow k_{def}$ = 0.60

7.2.5.1. maximal utilization



beam	max U
left rafter	4.04
right rafter	3.28
collar b.	2.39

7.2.5.2. formal proofs of the maximum utilisation

formal verification of maximal utilization in left rafter

decisive point at ξ = 4.79 m (ξ runs from bottom left side to top right side)
decisive deformation: max w
decisive comparative length: 2.05 m

Explanation of the table w_k = characteristic deflection
$w_{net,fin} = \Psi_2 (1 + k_{def})\, w_k$ (all deflections in mm)

action effect	$\Psi_2(1+k_{def})$	w_k	$w_{net,fin}$
permanent loads	1.60	15.21	24.34
collar beam live load	0.48	6.87	3.30
sum			27.64

max utilization U = $\dfrac{w_{net,fin}}{l_v / 300}$ = 4.04 > 1.00

formal verification of maximal utilization in right rafter

decisive point at ξ = 2.61 m (ξ runs from top left side to bottom right side)
decisive deformation: min w
decisive comparative length: 1.05 m

Explanation of the table w_k = characteristic deflection
$w_{net,fin} = \Psi_2 (1 + k_{def})\, w_k$ (all deflections in mm)

action effect	$\Psi_2(1+k_{def})$	w_k	$w_{net,fin}$
permanent loads	1.60	-6.33	-10.13
collar beam live load	0.48	-2.71	-1.30
sum			-11.43

max utilization U = $\dfrac{w_{net,fin}}{l_v / 300}$ = 3.28 > 1.00

formal verification of maximal utilization in collar beam

decisive point at ξ = 2.99 m (ξ runs from left side to right side)
decisive deformation: max w
decisive comparative length: 7.17 m

Explanation of the table w_k = characteristic deflection
$w_{net,fin} = \Psi_2 (1 + k_{def})\, w_k$ (all deflections in mm)

action effect	$\Psi_2(1+k_{def})$	w_k	$w_{net,fin}$
permanent loads	1.60	19.94	31.91
collar beam live load	0.48	52.63	25.26
sum			57.17

max utilization U = $\dfrac{w_{net,fin}}{l_v / 300}$ = 2.39 > 1.00

8. Supportsreaktionen and internal forces to be connected

extremal support reactions

on characteristic load level

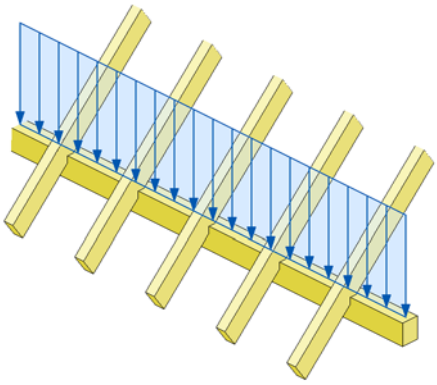
Positive vertical reaction forces (V) are acting from bottom to top.
Positive horizontal reaction forces (H) are acting from right side to left side.

	G	Q		G+Q	
		H	V	H	V
	kN/m	kN/m	kN/m	kN/m	kN/m
support A					
min A_V	-0.45	0.00	-1.91	0.00	-2.36
max A_V	-0.45	0.00	0.31	0.00	-0.14
support B					
min B_V	5.88	0.00	-4.74	0.00	1.14
max B_V	5.88	0.00	7.11	0.00	12.99
support C					
min C_H	0.00	-2.32	2.49	-2.32	13.90

extremal support reactions
on characteristic load level

Positive vertical reaction forces (V) are acting from bottom to top.
Positive horizontal reaction forces (H) are acting from right side to left side.

	G	Q		G+Q	
		H	V	H	V
	kN/m	kN/m	kN/m	kN/m	kN/m
max C_H	0.00	1.08	11.83	1.08	23.25
min C_V	11.42	-0.40	-6.26	-0.40	5.15
max C_V	11.42	1.08	12.01	1.08	23.42
support D					
min D_V	-1.82	0.00	-3.85	0.00	-5.67
max D_V	-1.82	0.00	2.13	0.00	0.31

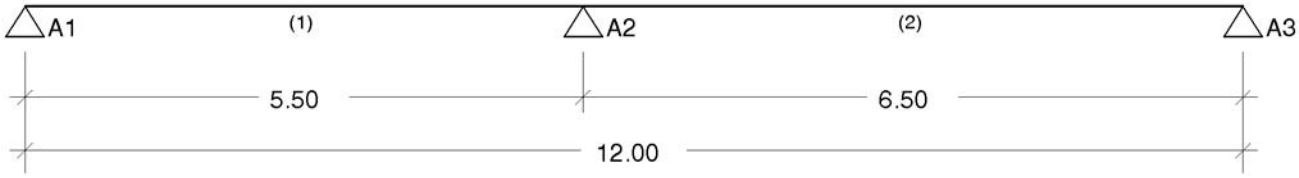


The values describe a line load in purlin running direction. Man loads that are to be applied only once per roof are not taken into account here.

9. purlin A

9.1. position, characteristic values and structural system

material: BSH (EC): GL24h
cross-section: b = 14.0 cm, h = 28.0 cm
char.valu: EI = 2945 kN/m², W = 1829.3 cm³, γ = 5.00 kN/m³
fm,k = 24.0 N/mm², fv,k = 3.5 N/mm²
design codes: Eurocode: EN 1990, EN 1991, EN 1995



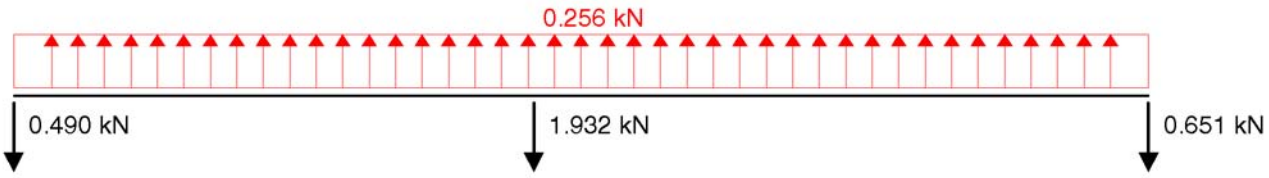
9.2. Loading

The structure of the load corresponds to that of the rafter calculation. Exception: The permanent loads are combined into one load case.The loads are essentially recruited from the bearing reaction forces (support A) of the rafter calculation. Exception: man loads.
Only the relevant load cases are logged here that make a significant contribution to the extrema of the assigned action.

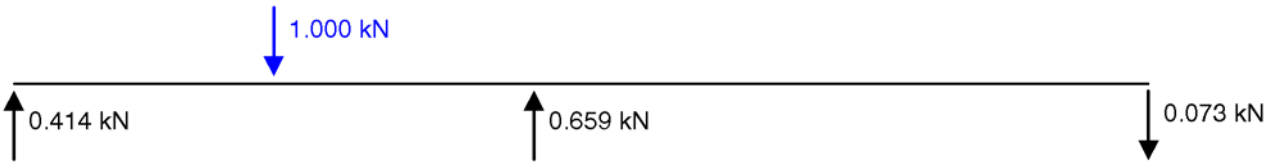
9.2.1. permanent loads

System, loading + support reactions from vertical loads

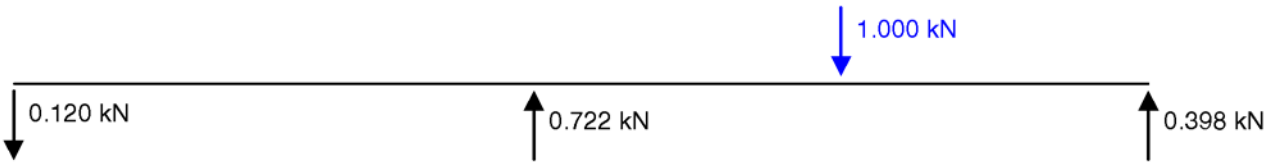
from support force (s A)	from rafter load c. dead load	-0.114 kN / 0.700 m	-0.163 kN/m
from support force (s A)	from rafter l. c. outer skin	-0.093 kN / 0.700 m	-0.133 kN/m
from support force (s A)	from rafter l.c. int. finish w(1)	-0.109 kN / 0.700 m	-0.156 kN/m
dead load purlin	(5.000 kN/m³ * 0.280 m * 0.140 m)		0.196 kN/m
sum permanent loads (vertical)			**-0.256 kN/m**



9.2.2. man load position 1



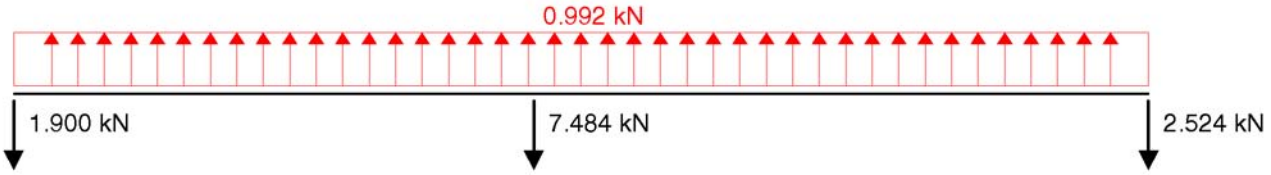
9.2.3. man load position 2



9.2.4. live load (1)

System, loading + support reactions from vertical loads

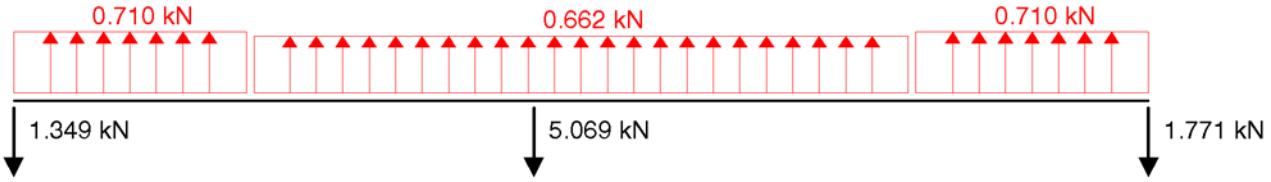
from support force (s A) from rafter l. c. live load	-0.695 kN / 0.700 m	**-0.992 kN/m**



9.2.5. wind from left side (3)

System, loading + support reactions from vertical loads

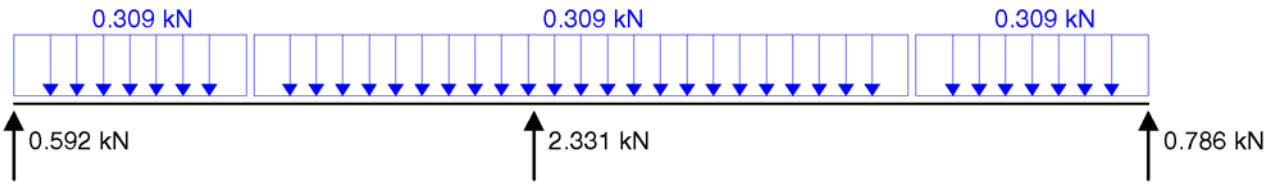
q1 from support force (s A) from rafter l.c. wind from left s (7)	-0.497 kN / 0.700 m	**-0.710 kN/m**
q2 from support force (s A) from rafter l.c. wind from left s (3)	-0.464 kN / 0.700 m	**-0.662 kN/m**
q3 from support force (s A) from rafter l.c. wind from left s (7)	-0.497 kN / 0.700 m	**-0.710 kN/m**



9.2.6. wind from right side (1)

System, loading + support reactions from vertical loads

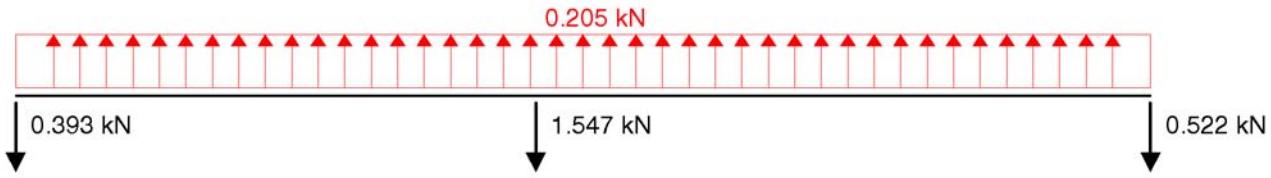
q1 from support force (s A) from rafter l.c. wind from right s (1)	0.216 kN / 0.700 m	**0.309 kN/m**
q2 from support force (s A) from rafter l.c. wind from right s (1)	0.216 kN / 0.700 m	**0.309 kN/m**
q3 from support force (s A) from rafter l.c. wind from right s (1)	0.216 kN / 0.700 m	**0.309 kN/m**



9.2.7. snow (3)

System, loading + support reactions from vertical loads

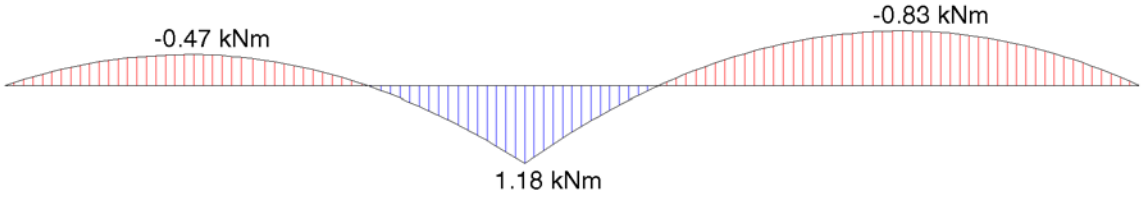
from support force (s A) from rafter l. c. drift right side -0.144 kN / 0.700 m **-0.205 kN/m**



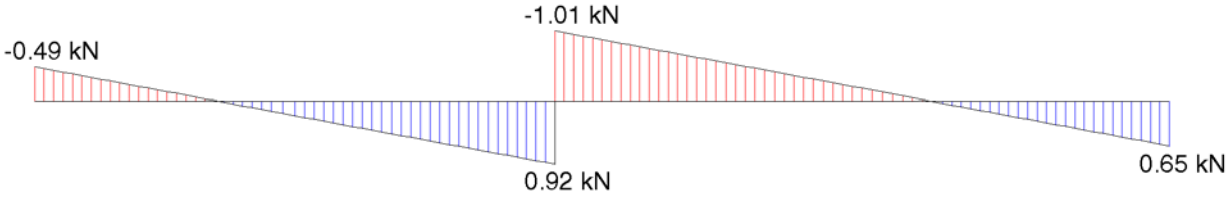
9.3. Extremal from action effects

9.3.1. permanent loads

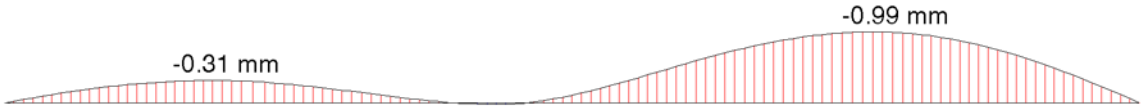
extremal flectural moments from vertical loads (permanent loads)



extremal shear forces from vertical loads (permanent loads)

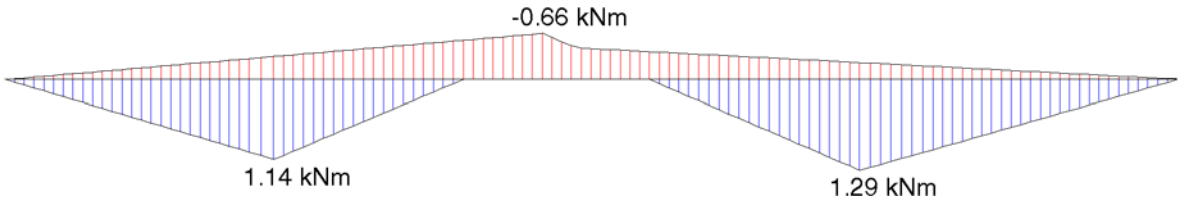


extremal deformations from vertical loads (permanent loads)

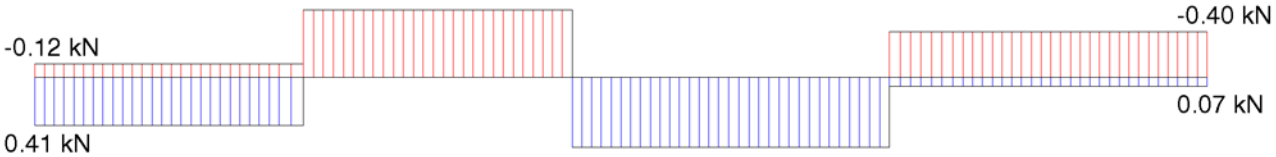


9.3.2. extremal man loads

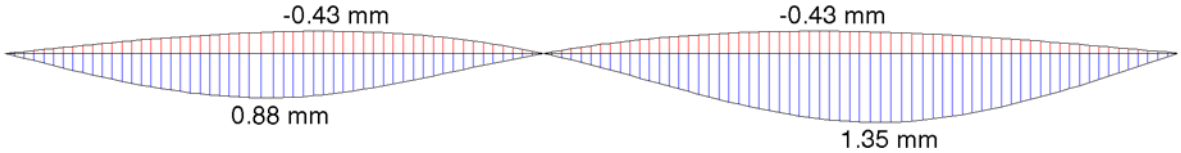
extremal flectural moments from vertical loads (man loads)



extremal shear forces from vertical loads (man loads)

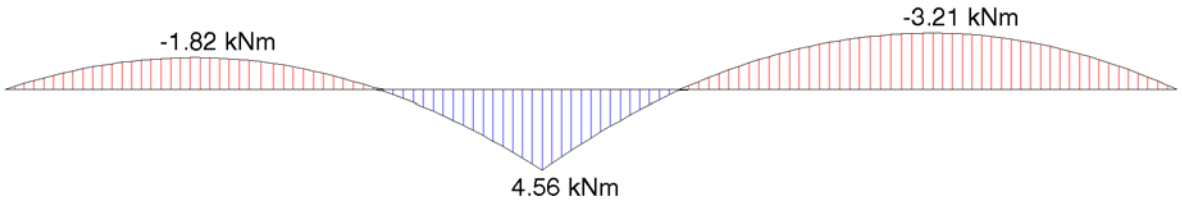


extremal deformations from vertical loads (man loads)

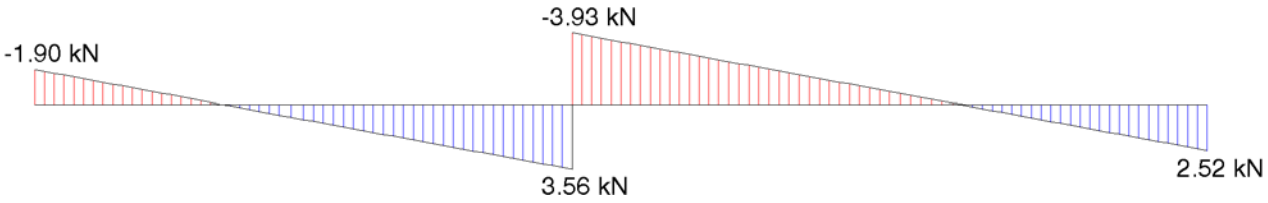


9.3.3. extremal live loads

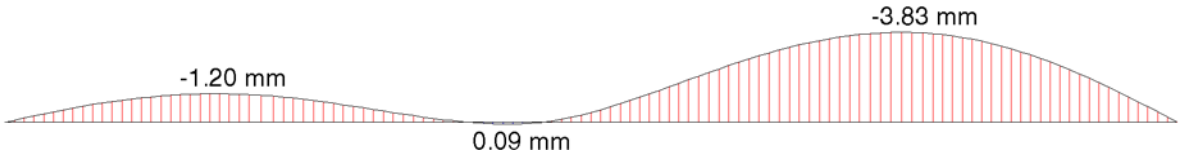
extremal flectural moments from vertical loads (live loads)



extremal shear forces from vertical loads (live loads)

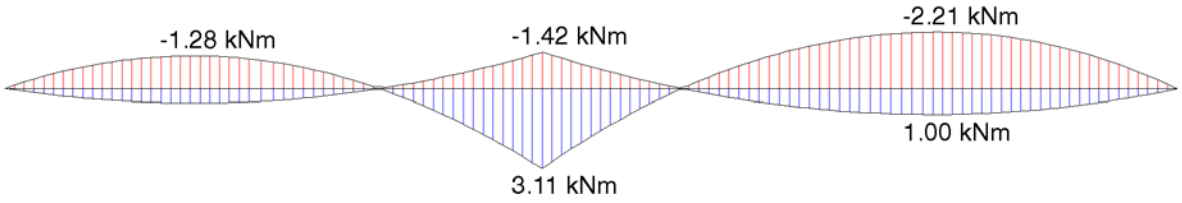


extremal deformations from vertical loads (live loads)

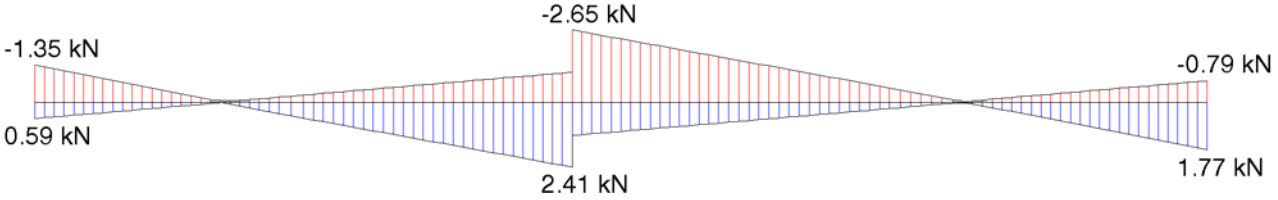


9.3.4. extremal wind loads

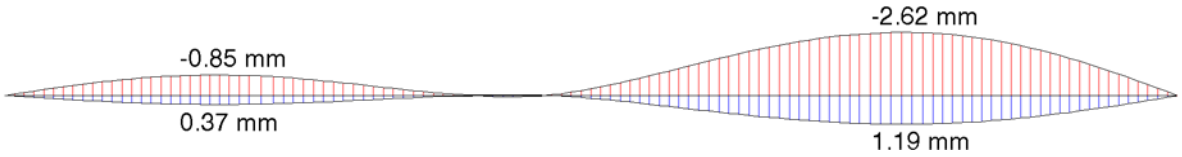
extremal flectural moments from vertical loads (wind)



extremal shear forces from vertical loads (wind)

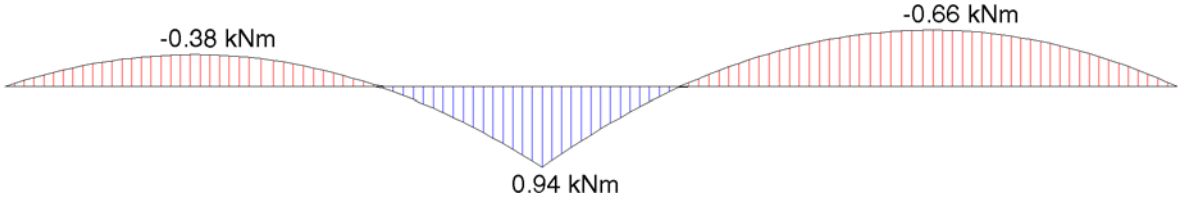


extremal deformations from vertical loads (wind)

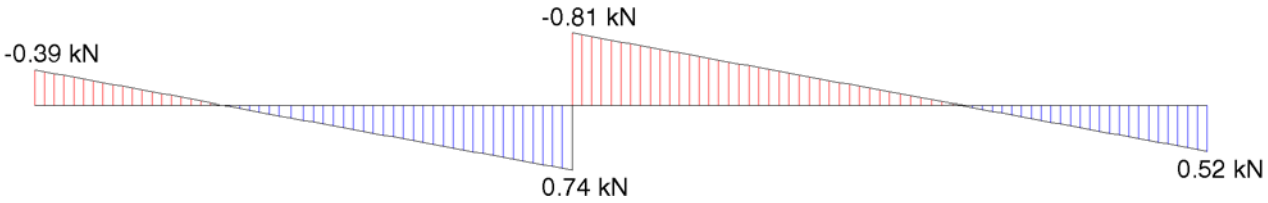


9.3.5. extremal snow loads

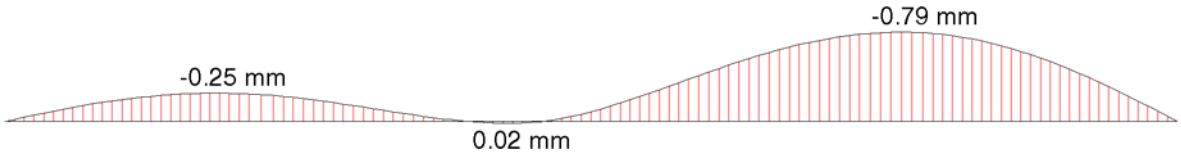
extremal flectural moments from vertical loads (snow)



extremal shear forces from vertical loads (snow)



extremal deformations from vertical loads (snow)



9.4. Verifications

9.4.1. degree of utilizations in the permanent and transienten design situation

The determination of the extreme values for the permanent and temporary design situation is carried out kmod-group-wise under variation of the guiding action with the partial safety and combination coefficients shown in the rafter calculation. For details see chapter 7.1.2. The utilisation rates are determined acc. to EC5 (6.1.6 bending and 6.1.7 thrust) with γ_M = 1.30. They result as shown below

max U = 35% ⇒ verification successful.

9.4.2. Utilisation rates for fire protection verification

verification method, required fire resistance period, partial safety factors and combination coefficients see chapter 7.1.3.
Fire exposure: four-sided. h_{red} = 22.6 cm, b_{red} = 8.6 cm, k_{φ} = 1.15, k_{mod} = 1.00, f_{md} = 27.6 N/mm²
The utilisation rates are determined acc. to EC5 6.1.6 with γ_M = 1.00. They result as shown below.

max U = 16% ⇒ verification successful.

9.4.3. Serviceability utilisation rates without creep influence (w_{inst})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below.

max U = 31% ⇒ verification successful.

9.4.4. Serviceability utilisation rates with creep influence (w_{fin})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below with k_{def} = 0.60

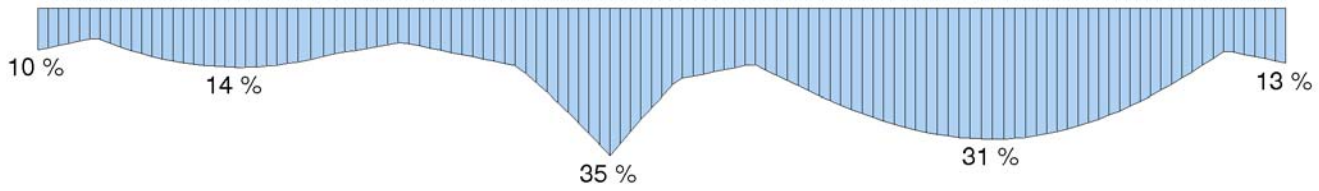
max U = 25% ⇒ verification successful.

9.4.5. Serviceability utilisation rates with creep influence ($w_{fin,net}$)

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the quasi-permanent combination. They result as shown below with k_{def} = 0.60

max U = 16% ⇒ verification successful.

9.4.6. maximum utilization of all verifications



max U = 35% ⇒ all verifications successful.

9.5. extremal support reactions

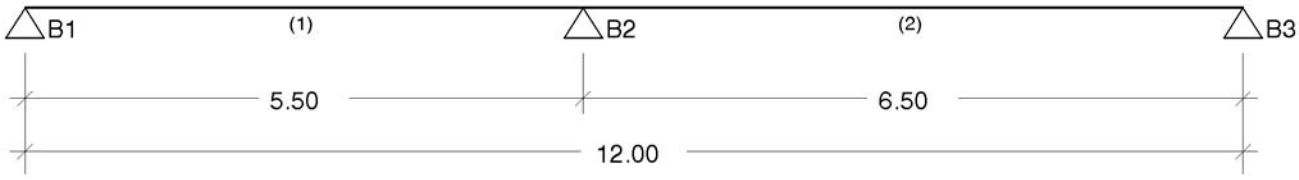
Abbreviations: G: permanent loads, M: man loads, N: live loads, W: wind loads, S: snow loads

support	G kN	M kN	N kN	W kN	S kN	Σ kN
maximal vertical:						
A1	-0.490	0.414	0.000	0.592	0.000	0.515
A2	-1.932	0.722	0.000	2.331	0.000	1.121
A3	-0.651	0.398	0.000	0.786	0.000	0.533
minimal vertical:						
A1	-0.490	-0.120	-1.900	-1.349	-0.393	-4.252
A2	-1.932	0.000	-7.484	-5.069	-1.547	-16.032
A3	-0.651	-0.073	-2.524	-1.771	-0.522	-5.541

10. purlin B

10.1. position, characteristic values and structural system

material: BSH (EC): GL24h
cross-section: b = 16.0 cm, h = 36.0 cm
char.valu: EI = 7154 kN/m², W = 3456.0 cm³, γ = 5.00 kN/m³
fm,k = 24.0 N/mm², fv,k = 3.5 N/mm²
design codes: Eurocode: EN 1990, EN 1991, EN 1995



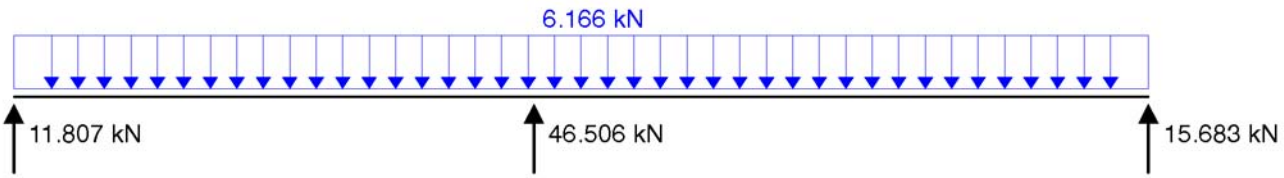
10.2. Loading

The structure of the load corresponds to that of the rafter calculation. Exception: The permanent loads are combined into one load case.The loads are essentially recruited from the bearing reaction forces (support B) of the rafter calculation. Exception: man loads.
Only the relevant load cases are logged here that make a significant contribution to the extrema of the assigned action.

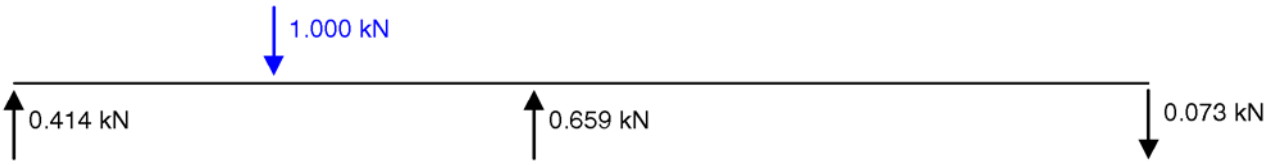
10.2.1. permanent loads

System, loading + support reactions from vertical loads

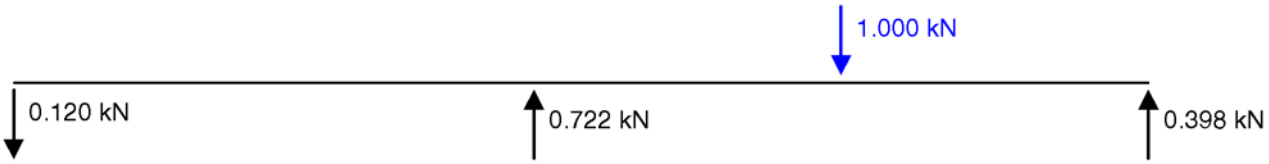
from support force (s B) from rafter load c. dead load	0.698 kN / 0.700 m	0.996 kN/m
from support force (s B) from rafter l. c. outer skin	2.630 kN / 0.700 m	3.757 kN/m
from support force (s B) from rafter l.c. int. finish w(1)	0.787 kN / 0.700 m	1.125 kN/m
dead load purlin (5.000 kN/m³ * 0.360 m * 0.160 m)		0.288 kN/m
sum permanent loads (vertical)		**6.166 kN/m**



10.2.2. man load position 1



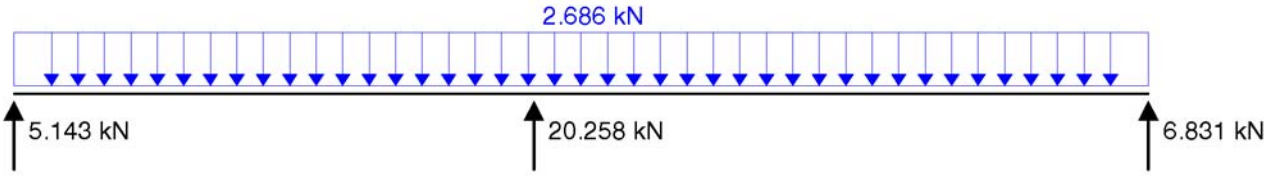
10.2.3. man load position 2



10.2.4. live load (1)

System, loading + support reactions from vertical loads

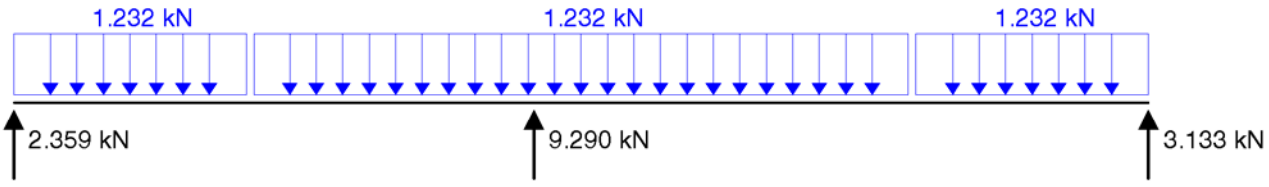
from support force (s B) from rafter l. c. live load	1.880 kN / 0.700 m	**2.686 kN/m**



10.2.5. wind from left side (2)

System, loading + support reactions from vertical loads

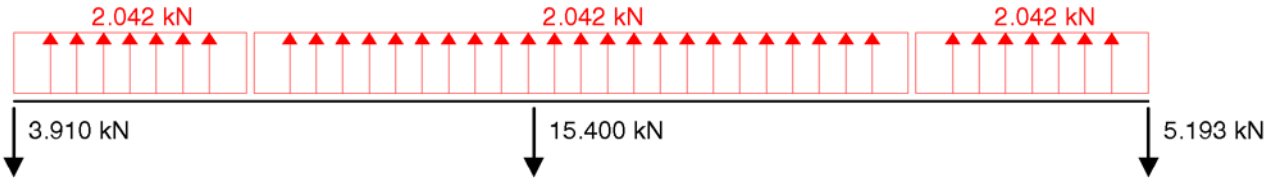
q1 from support force (s B) from rafter l.c. wind from left s (2)	0.862 kN / 0.700 m	**1.232 kN/m**
q2 from support force (s B) from rafter l.c. wind from left s (2)	0.862 kN / 0.700 m	**1.232 kN/m**
q3 from support force (s B) from rafter l.c. wind from left s (2)	0.862 kN / 0.700 m	**1.232 kN/m**



10.2.6. wind from right side (2)

System, loading + support reactions from vertical loads

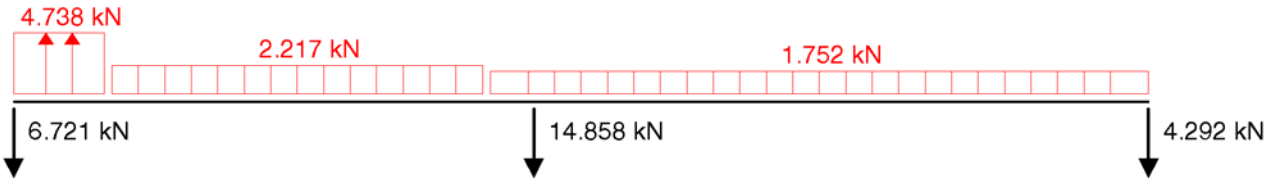
q1 from support force (s B) from rafter l.c. wind from right s (2)	-1.429 kN / 0.700 m	**-2.042 kN/m**
q2 from support force (s B) from rafter l.c. wind from right s (2)	-1.429 kN / 0.700 m	**-2.042 kN/m**
q3 from support force (s B) from rafter l.c. wind from right s (2)	-1.429 kN / 0.700 m	**-2.042 kN/m**



10.2.7. wind from the front

System, loading + support reactions from vertical loads

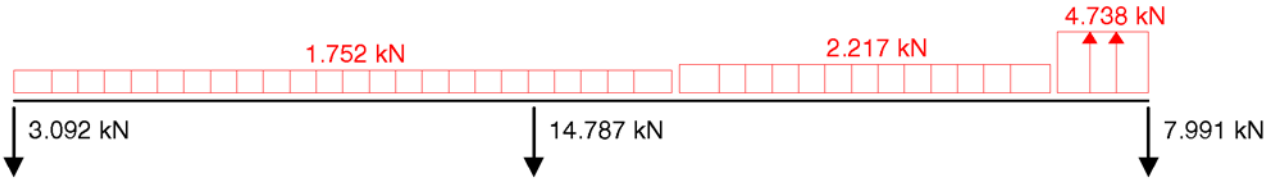
q1 from support force (s B) from rafter load c.	wind on gable (1)	-3.316 kN / 0.700 m	**-4.738 kN/m**
q2 from support force (s B) from rafter load c.	wind on gable (2)	-1.552 kN / 0.700 m	**-2.217 kN/m**
q3 from support force (s B) from rafter load c.	wind on gable (3)	-1.226 kN / 0.700 m	**-1.752 kN/m**



10.2.8. wind from behind

System, loading + support reactions from vertical loads

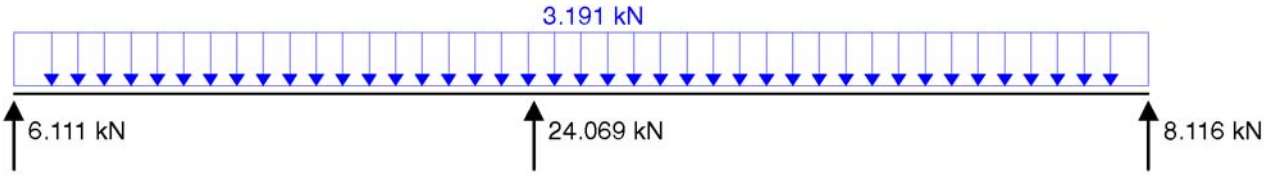
q1 from support force (s B) from rafter load c.	wind on gable (3)	-1.226 kN / 0.700 m	**-1.752 kN/m**
q2 from support force (s B) from rafter load c.	wind on gable (2)	-1.552 kN / 0.700 m	**-2.217 kN/m**
q3 from support force (s B) from rafter load c.	wind on gable (1)	-3.316 kN / 0.700 m	**-4.738 kN/m**



10.2.9. snow (3)

System, loading + support reactions from vertical loads

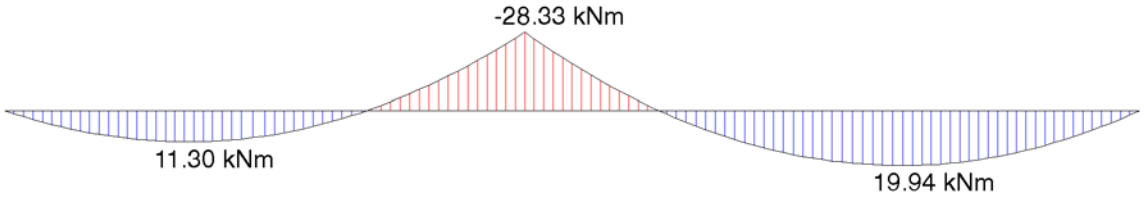
from support force (s B) from rafter l. c.	drift right side	2.234 kN / 0.700 m	**3.191 kN/m**



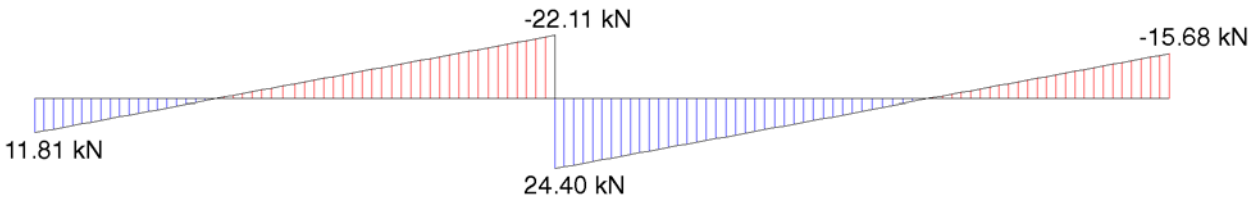
10.3. Extremal from action effects

10.3.1. permanent loads

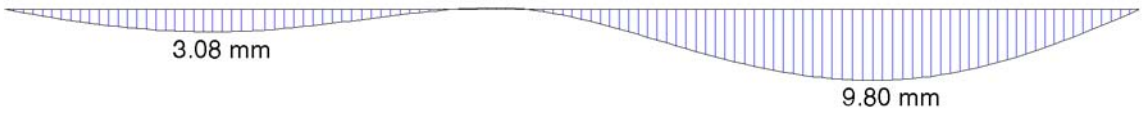
extremal flectural moments from vertical loads (permanent loads)



extremal shear forces from vertical loads (permanent loads)

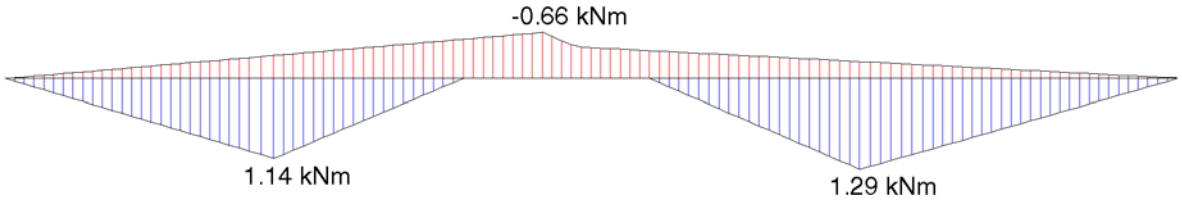


extremal deformations from vertical loads (permanent loads)

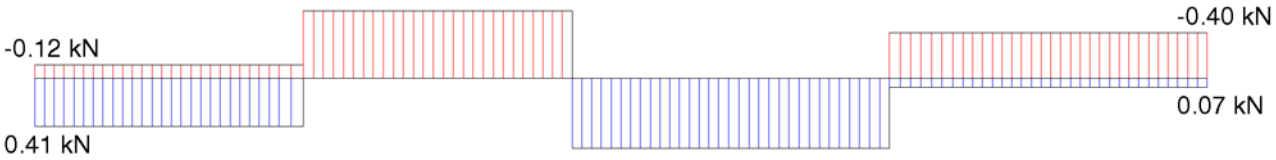


10.3.2. extremal man loads

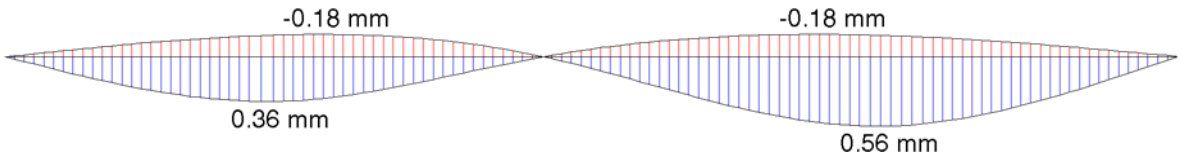
extremal flectural moments from vertical loads (man loads)



extremal shear forces from vertical loads (man loads)

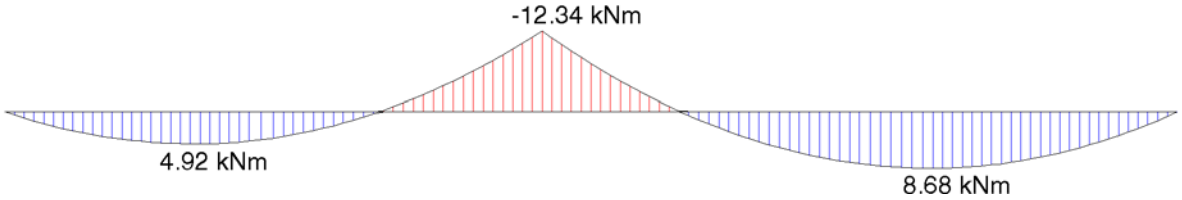


extremal deformations from vertical loads (man loads)

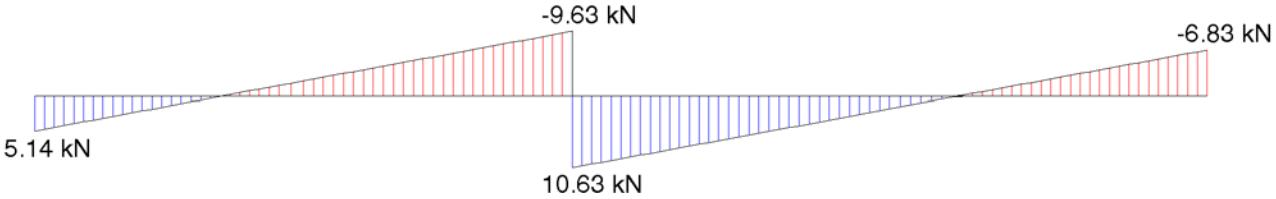


10.3.3. extremal live loads

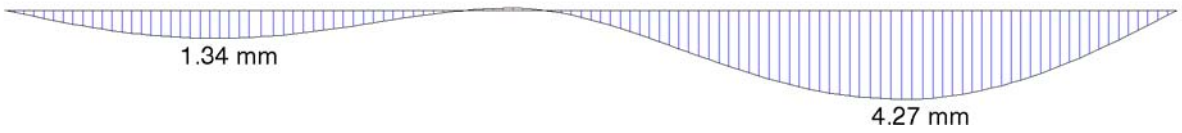
extremal flectural moments from vertical loads (live loads)



extremal shear forces from vertical loads (live loads)

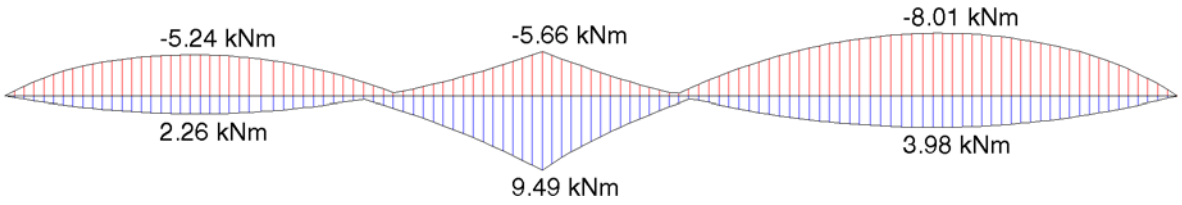


extremal deformations from vertical loads (live loads)

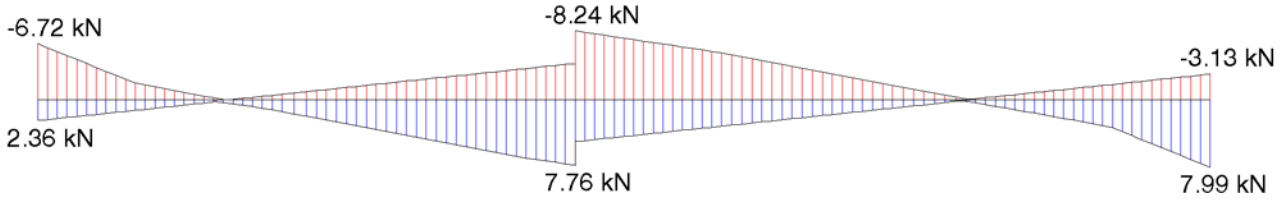


10.3.4. extremal wind loads

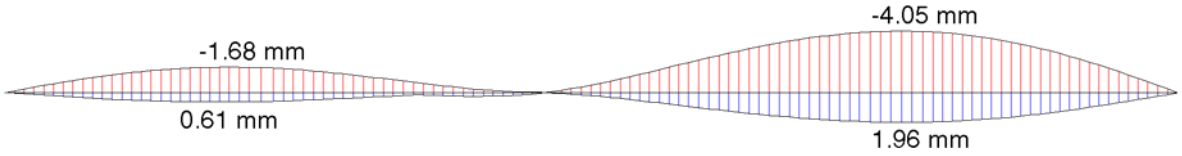
extremal flectural moments from vertical loads (wind)



extremal shear forces from vertical loads (wind)

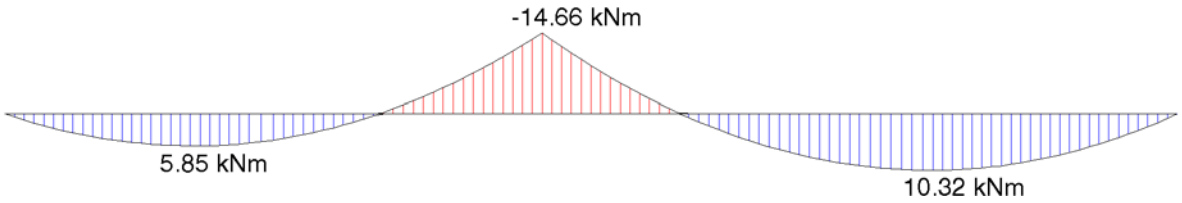


extremal deformations from vertical loads (wind)

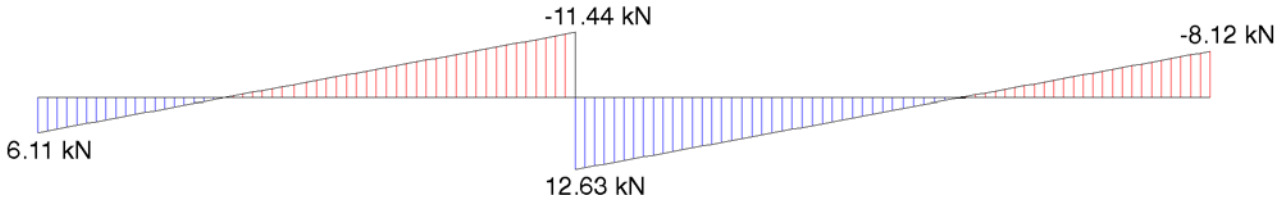


10.3.5. extremal snow loads

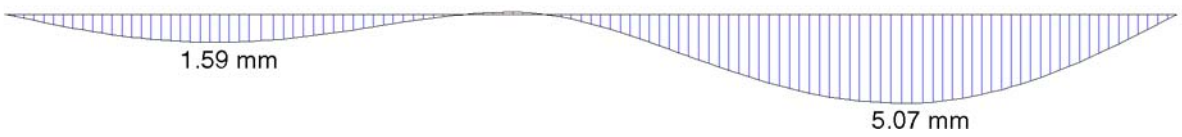
extremal flectural moments from vertical loads (snow)



extremal shear forces from vertical loads (snow)



extremal deformations from vertical loads (snow)



10.4. Verifications

10.4.1. degree of utilizations in the permanent and transienten design situation

The determination of the extreme values for the permanent and temporary design situation is carried out kmod-group-wise under variation of the guiding action with the partial safety and combination coefficients shown in the rafter calculation. For details see chapter 7.1.2. The utilisation rates are determined acc. to EC5 (6.1.6 bending and 6.1.7 thrust) with γ_M = 1.30. They result as shown below

max U = 127% ⇒ verification not successful.

10.4.2. Utilisation rates for fire protection verification

verification method, required fire resistance period, partial safety factors and combination coefficients see chapter 7.1.3.
Fire exposure: four-sided. h_{red} = 30.6 cm, b_{red} = 10.6 cm, k_φ = 1.15, k_{mod} = 1.00, f_{md} = 27.6 N/mm²
The utilisation rates are determined acc. to EC5 6.1.6 with γ_M = 1.00. They result as shown below.

max U = 73% ⇒ verification successful.

10.4.3. Serviceability utilisation rates without creep influence (w_{inst})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below.

max U = 88% ⇒ verification successful.

10.4.4. Serviceability utilisation rates with creep influence (w_{fin})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below with k_{def} = 0.60

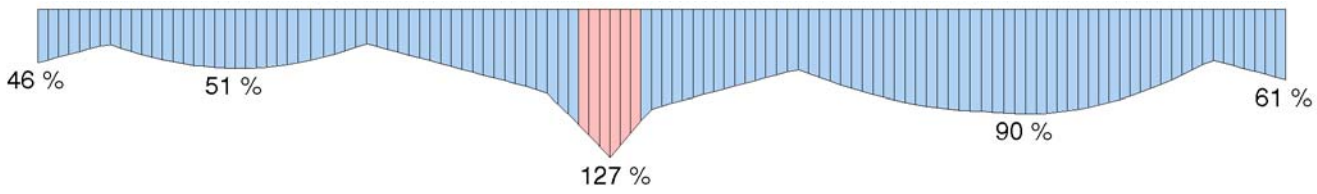
max U = 79% ⇒ verification successful.

10.4.5. Serviceability utilisation rates with creep influence ($w_{fin,net}$)

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the quasi-permanent combination. They result as shown below with k_{def} = 0.60

max U = 82% ⇒ verification successful.

10.4.6. maximum utilization of all verifications



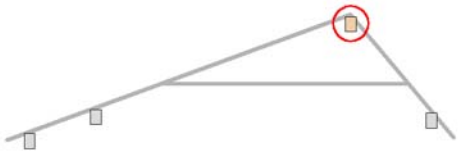
10.5. extremal support reactions

Abbreviations: G: permanent loads, M: man loads, N: live loads, W: wind loads, S: snow loads

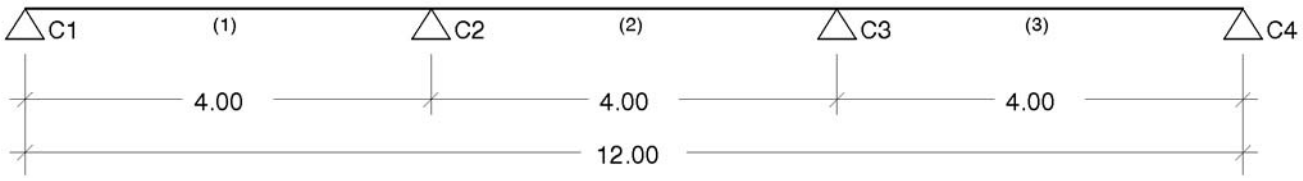
support	G kN	M kN	N kN	W kN	S kN	Σ kN
maximal vertical:						
B1	11.807	0.414	5.143	2.359	6.111	25.833
B2	46.506	0.722	20.258	9.290	24.069	100.844
B3	15.683	0.398	6.831	3.133	8.116	34.161
minimal vertical:						
B1	11.807	-0.120	0.000	-6.721	0.000	4.966
B2	46.506	0.000	0.000	-15.400	0.000	31.107
B3	15.683	-0.073	0.000	-7.991	0.000	7.619

11. purlin C

11.1. position, characteristic values and structural system



material:	coniferous timber: C24
cross-section:	b = 18.0 cm, h = 18.0 cm
char.valu:	EI = 962 kN/m², W = 972.0 cm³, γ = 5.00 kN/m³
	fm,k = 24.0 N/mm², fv,k = 4.0 N/mm²
design codes:	Eurocode: EN 1990, EN 1991, EN 1995



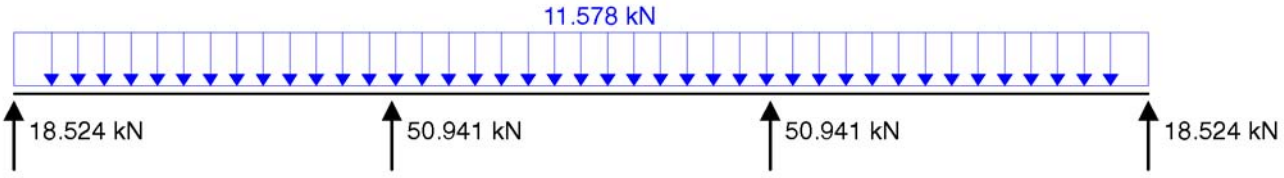
11.2. Loading

The structure of the load corresponds to that of the rafter calculation. Exception: The permanent loads are combined into one load case.The loads are essentially recruited from the bearing reaction forces (support C) of the rafter calculation. Exception: man loads. The purlin is stressed to double bending.
Only the relevant load cases are logged here that make a significant contribution to the extrema of the assigned action.

11.2.1. permanent loads

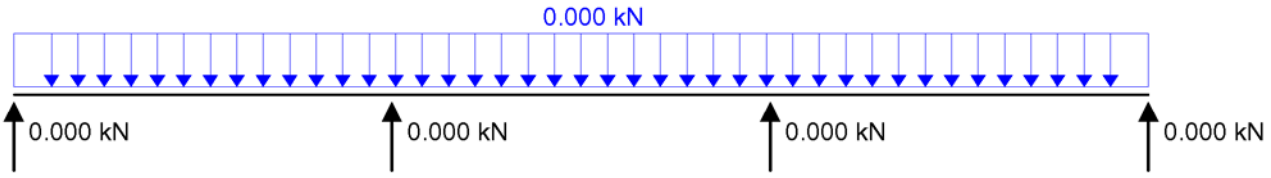
System, loading + support reactions from vertical loads

from support force (s C) from rafter load c. dead load	1.407 kN / 0.700 m	2.010 kN/m
from support force (s C) from rafter l. c. outer skin	4.947 kN / 0.700 m	7.067 kN/m
from support force (s C) from rafter l.c. int. finish w(1)	1.637 kN / 0.700 m	2.338 kN/m
dead load purlin (5.000 kN/m³ * 0.180 m * 0.180 m)		0.162 kN/m
sum permanent loads (vertical)		**11.578 kN/m**

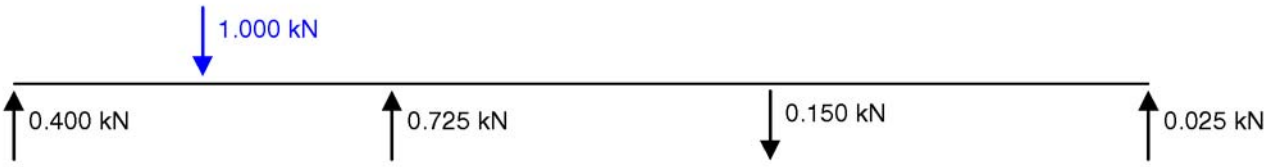


System, loading + support reactions from horizontal loads

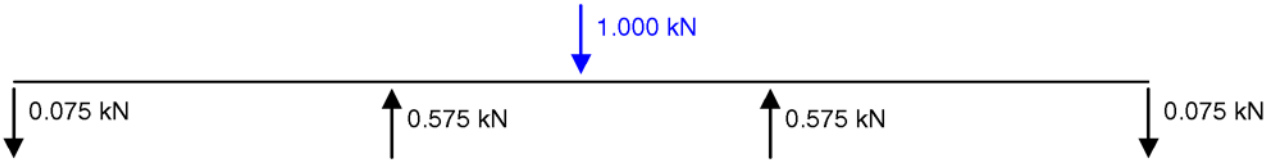
from support force (s C) from rafter load c. dead load	0.000 kN / 0.700 m	0.000 kN/m
from support force (s C) from rafter l. c. outer skin	0.000 kN / 0.700 m	0.000 kN/m
from support force (s C) from rafter l.c. int. finish w(1)	0.000 kN / 0.700 m	0.000 kN/m
sum permanent loads (horizontal)		**0.000 kN/m**



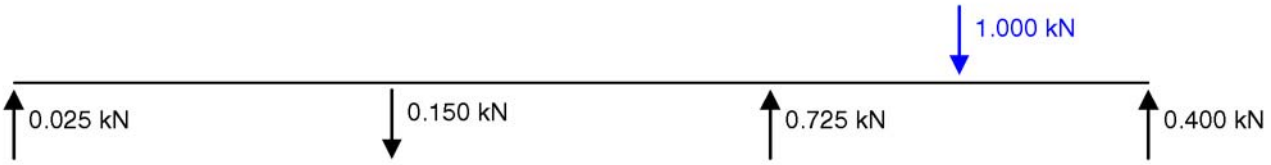
11.2.2. man load position 1



11.2.3. man load position 2



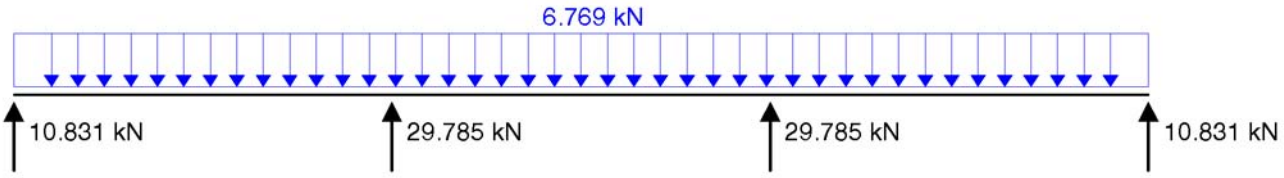
11.2.4. man load position 3



11.2.5. live load (1)

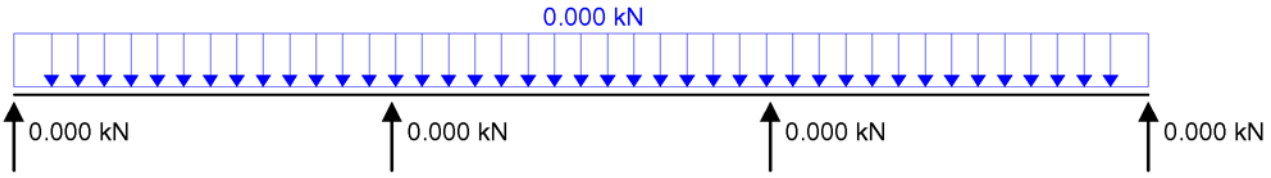
System, loading + support reactions from vertical loads

from support force (s C) from rafter l. c. live load	4.739 kN / 0.700 m	**6.769 kN/m**



System, loading + support reactions from horizontal loads

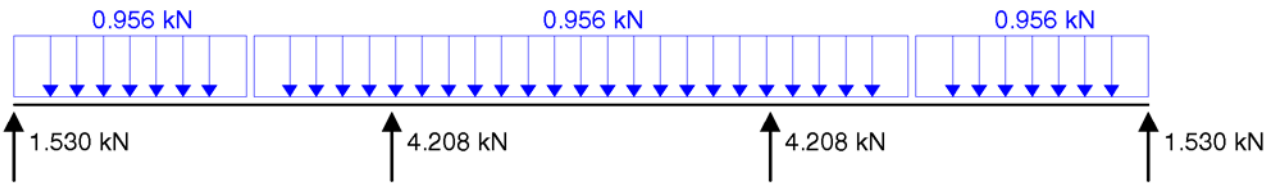
from support force (s C) from rafter l. c. live load	0.000 kN / 0.700 m	**0.000 kN/m**



11.2.6. wind from left side (2)

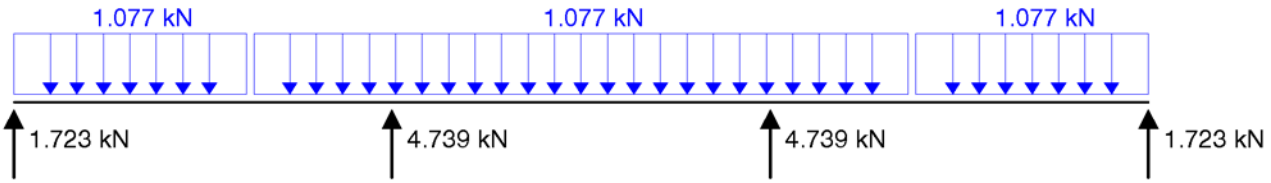
System, loading + support reactions from vertical loads

q1 from support force (s C) from rafter l.c. wind from left s (2)	0.669 kN / 0.700 m	**0.956 kN/m**
q2 from support force (s C) from rafter l.c. wind from left s (2)	0.669 kN / 0.700 m	**0.956 kN/m**
q3 from support force (s C) from rafter l.c. wind from left s (2)	0.669 kN / 0.700 m	**0.956 kN/m**



System, loading + support reactions from horizontal loads

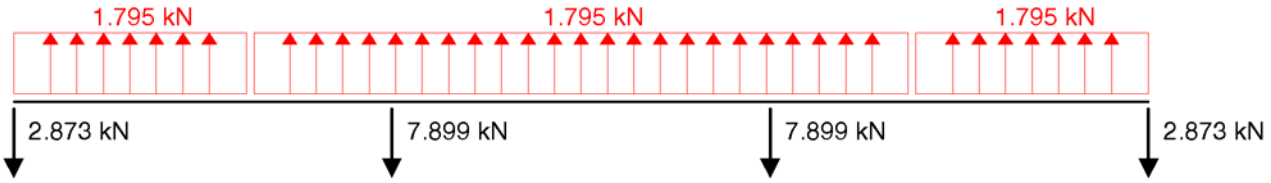
q1 from support force (s C) from rafter l.c. wind from left s (2)	0.754 kN / 0.700 m	**1.077 kN/m**
q2 from support force (s C) from rafter l.c. wind from left s (2)	0.754 kN / 0.700 m	**1.077 kN/m**
q3 from support force (s C) from rafter l.c. wind from left s (2)	0.754 kN / 0.700 m	**1.077 kN/m**



11.2.7. wind from right side (2)

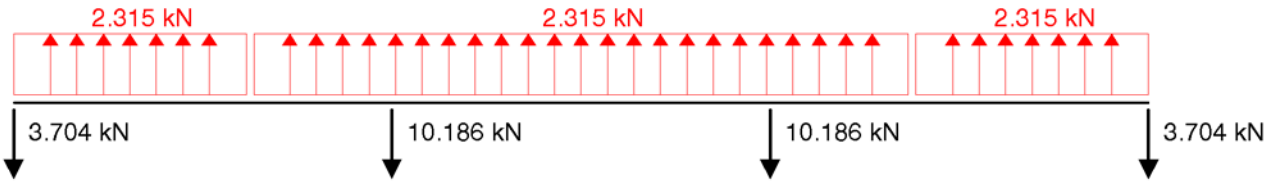
System, loading + support reactions from vertical loads

q1 from support force (s C) from rafter l.c. wind from right s (2)	-1.257 kN / 0.700 m	**-1.795 kN/m**
q2 from support force (s C) from rafter l.c. wind from right s (2)	-1.257 kN / 0.700 m	**-1.795 kN/m**
q3 from support force (s C) from rafter l.c. wind from right s (2)	-1.257 kN / 0.700 m	**-1.795 kN/m**



System, loading + support reactions from horizontal loads

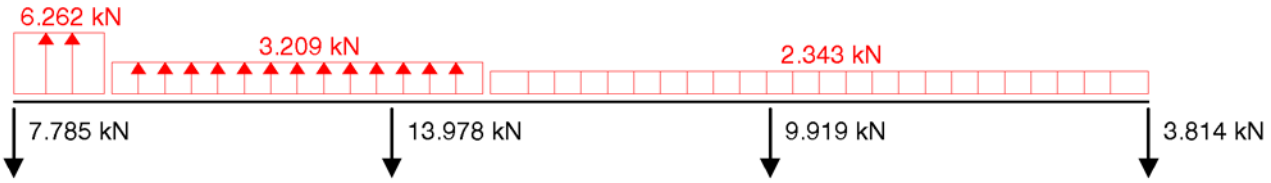
q1 from support force (s C) from rafter l.c. wind from right s (2)	-1.621 kN / 0.700 m	**-2.315 kN/m**
q2 from support force (s C) from rafter l.c. wind from right s (2)	-1.621 kN / 0.700 m	**-2.315 kN/m**
q3 from support force (s C) from rafter l.c. wind from right s (2)	-1.621 kN / 0.700 m	**-2.315 kN/m**



11.2.8. wind from the front

System, loading + support reactions from vertical loads

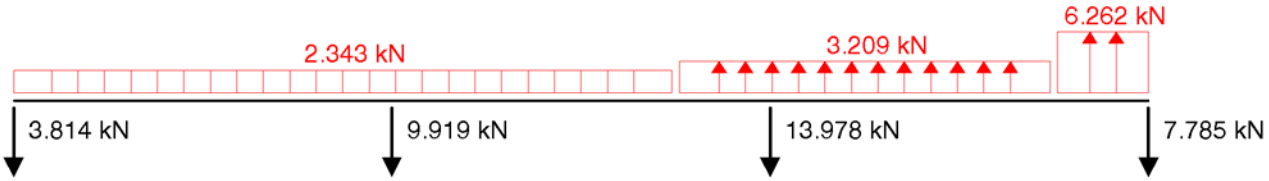
q1 from support force (s C) from rafter load c. wind on gable (1)	-4.384 kN / 0.700 m	**-6.262 kN/m**
q2 from support force (s C) from rafter load c. wind on gable (2)	-2.246 kN / 0.700 m	**-3.209 kN/m**
q3 from support force (s C) from rafter load c. wind on gable (3)	-1.640 kN / 0.700 m	**-2.343 kN/m**



11.2.9. wind from behind

System, loading + support reactions from vertical loads

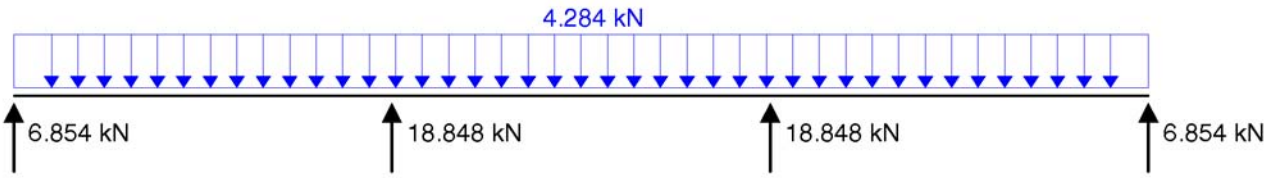
```
q1 from support force (s C) from rafter load c.  wind on gable (3)   -1.640 kN / 0.700 m    -2.343 kN/m
q2 from support force (s C) from rafter load c.  wind on gable (2)   -2.246 kN / 0.700 m    -3.209 kN/m
q3 from support force (s C) from rafter load c.  wind on gable (1)   -4.384 kN / 0.700 m    -6.262 kN/m
```



11.2.10. snow (1)

System, loading + support reactions from vertical loads

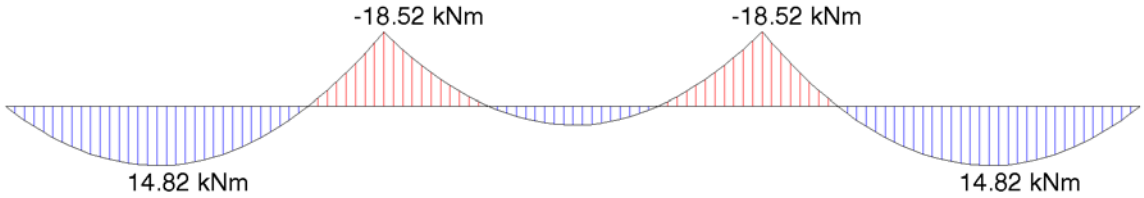
```
from support force (s C) from rafter l. c.  snow fully              2.999 kN / 0.700 m     4.284 kN/m
```



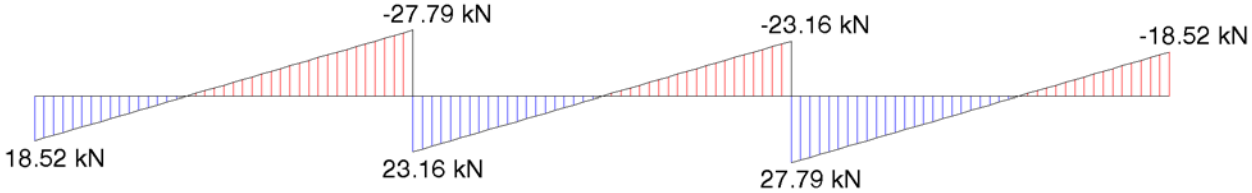
11.3. Extremal from action effects

11.3.1. permanent loads

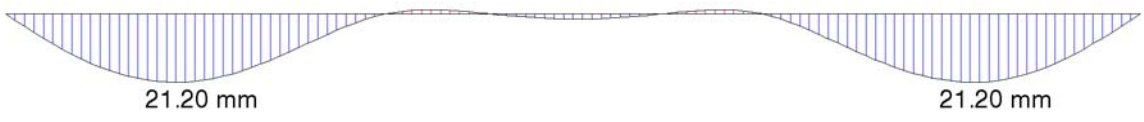
extremal flectural moments from vertical loads (permanent loads)



extremal shear forces from vertical loads (permanent loads)

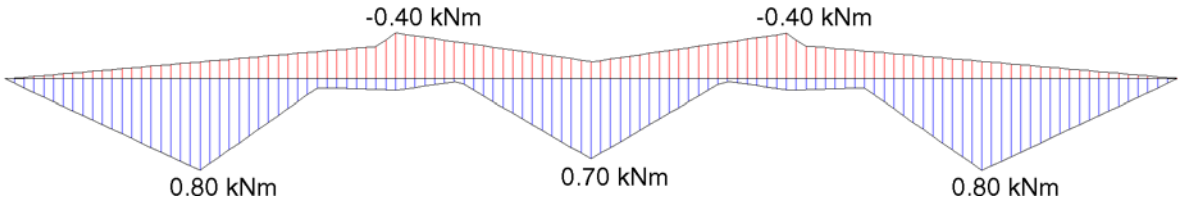


extremal deformations from vertical loads (permanent loads)

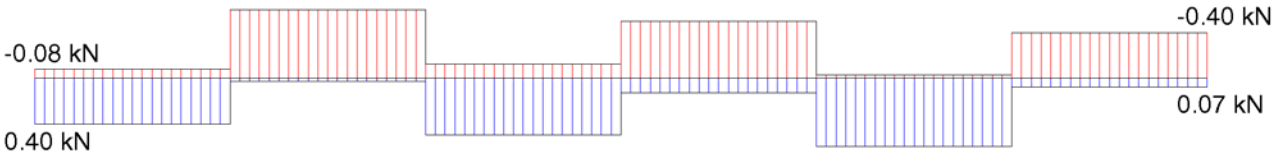


11.3.2. extremal man loads

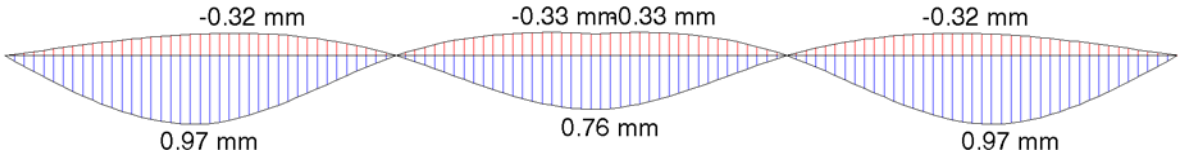
extremal flectural moments from vertical loads (man loads)



extremal shear forces from vertical loads (man loads)

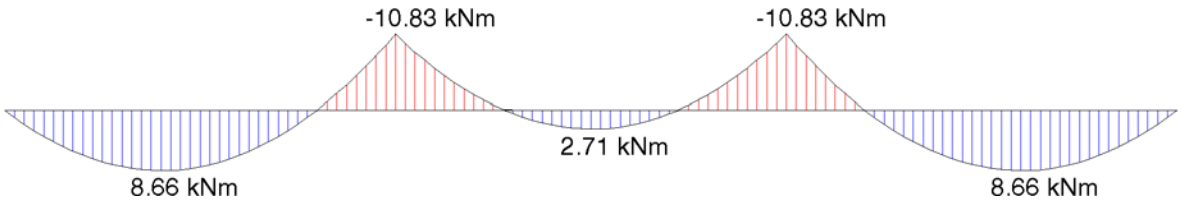


extremal deformations from vertical loads (man loads)

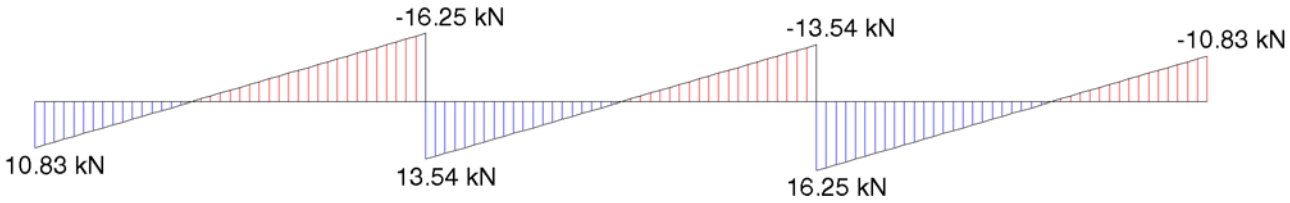


11.3.3. extremal live loads

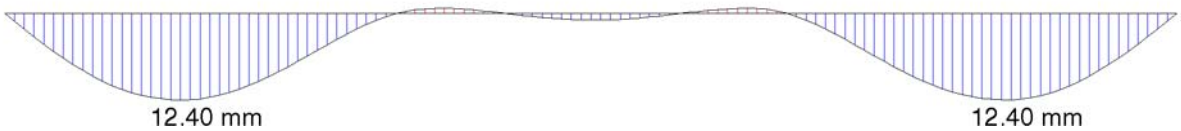
extremal flectural moments from vertical loads (live loads)



extremal shear forces from vertical loads (live loads)

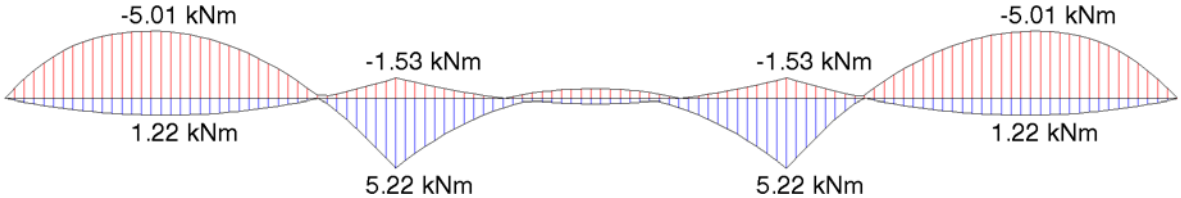


extremal deformations from vertical loads (live loads)

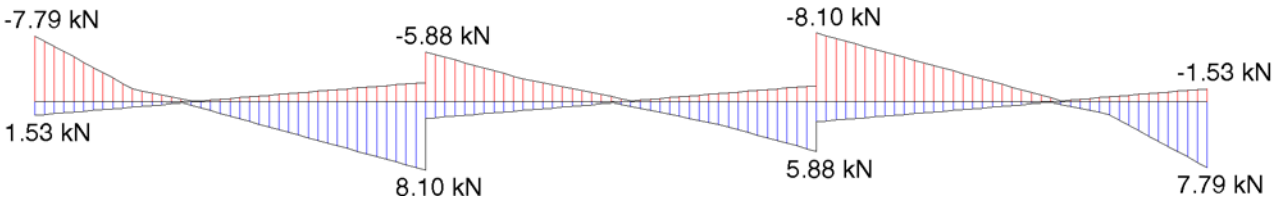


11.3.4. extremal wind loads

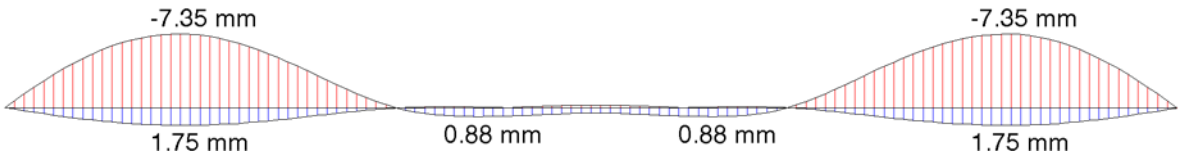
extremal flectural moments from vertical loads (wind)



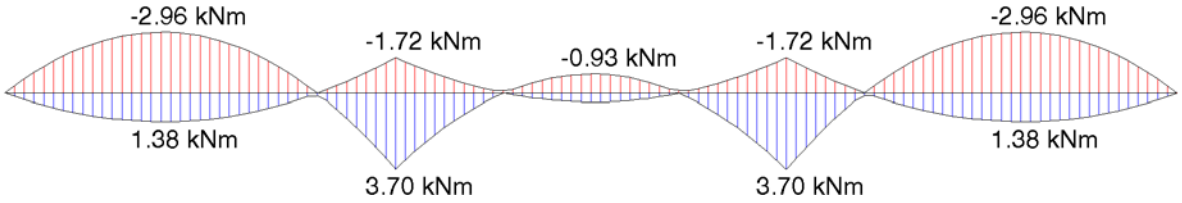
extremal shear forces from vertical loads (wind)



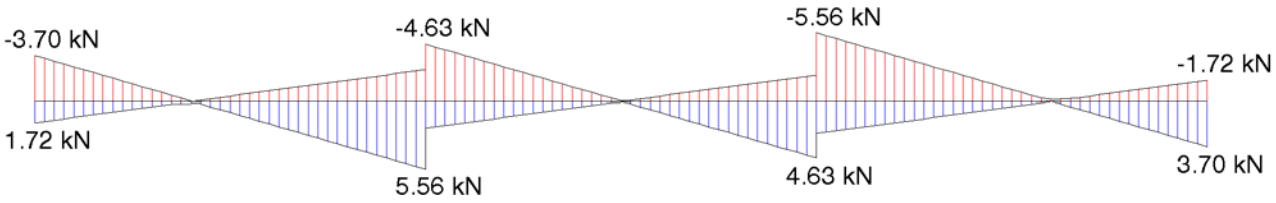
extremal deformations from vertical loads (wind)



extremal flectural moments from horizontal loads (wind)



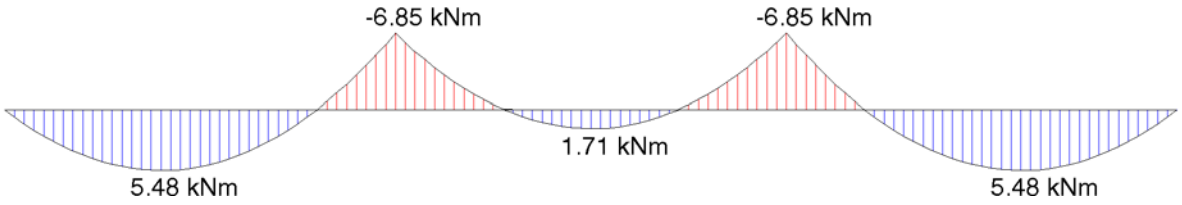
extremal shear forces from horizontal loads (wind)



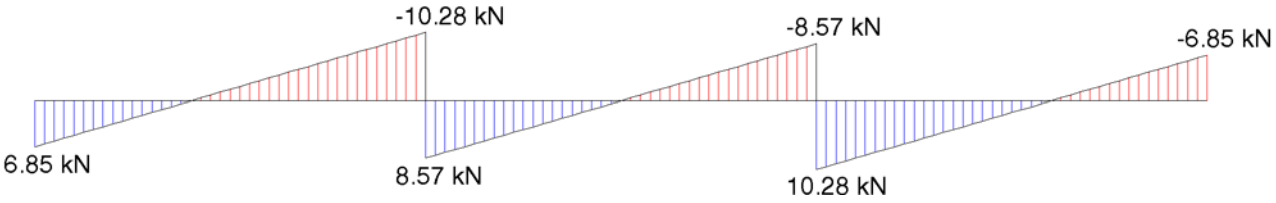
extremal deformations from horizontal loads (wind)

11.3.5. extremal snow loads

extremal flectural moments from vertical loads (snow)



extremal shear forces from vertical loads (snow)



extremal deformations from vertical loads (snow)

11.4. Verifications

11.4.1. degree of utilizations in the permanent and transienten design situation

The determination of the extreme values for the permanent and temporary design situation is carried out kmod-group-wise under variation of the guiding action with the partial safety and combination coefficients shown in the rafter calculation. For details see chapter 7.1.2. The utilisation rates are determined acc. to EC5 (6.1.6 bending and 6.1.7 thrust) with γ_M = 1.30. They result as shown below

max U = 289% ⇒ verification not successful.

11.4.2. Utilisation rates for fire protection verification

verification method, required fire resistance period, partial safety factors and combination coefficients see chapter 7.1.3.
Fire exposure: four-sided. h_{red} = 12.6 cm, b_{red} = 12.6 cm, k_φ = 1.25, k_{mod} = 1.00, f_{md} = 30.0 N/mm²
The utilisation rates are determined acc. to EC5 6.1.6 with γ_M = 1.00. They result as shown below.

max U = 226% ⇒ verification not successful.

11.4.3. Serviceability utilisation rates without creep influence (w_{inst})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below.

max U = 291% ⇒ verification not successful.

11.4.4. Serviceability utilisation rates with creep influence (w_{fin})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below with k_{def} = 0.60

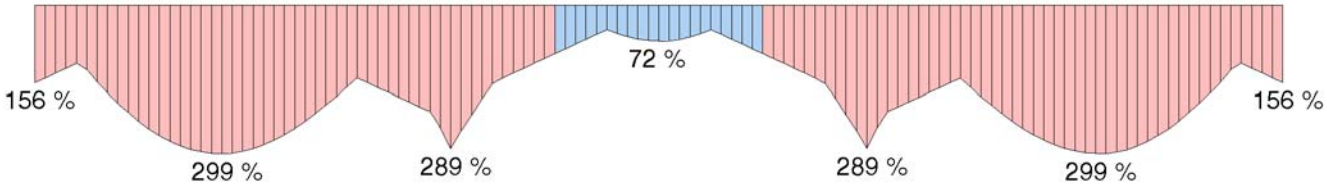
max U = 269% ⇒ verification not successful.

11.4.5. Serviceability utilisation rates with creep influence ($w_{fin,net}$)

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the quasi-permanent combination. They result as shown below with k_{def} = 0.60

max U = 299% ⇒ verification not successful.

11.4.6. maximum utilization of all verifications



11.5. extremal support reactions

Abbreviations: G: permanent loads, M: man loads, N: live loads, W: wind loads, S: snow loads

support	G kN	M kN	N kN	W kN	S kN	Σ kN
maximal vertical:						
C1	18.524	0.400	10.831	1.530	6.854	38.139
C2	50.941	0.725	29.785	4.208	18.848	104.508
C3	50.941	0.725	29.785	4.208	18.848	104.508
C4	18.524	0.400	10.831	1.530	6.854	38.139
minimal vertical:						
C1	18.524	-0.075	0.000	-7.785	0.000	10.664
C2	50.941	-0.150	0.000	-13.978	0.000	36.813
C3	50.941	-0.150	0.000	-13.978	0.000	36.813
C4	18.524	-0.075	0.000	-7.785	0.000	10.664
maximal horizontal:						
C1	0.000	0.000	0.000	1.723	0.000	1.723

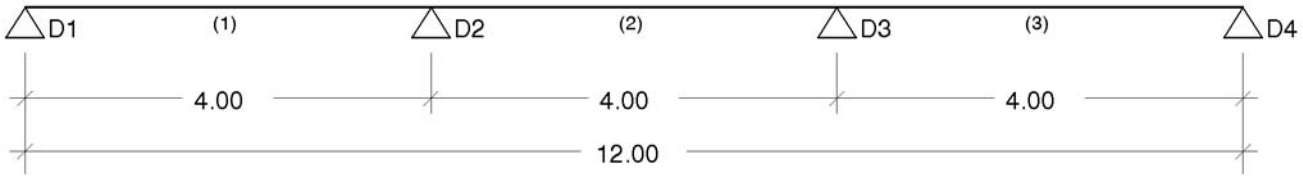
Abbreviations: G: permanent loads, M: man loads, N: live loads, W: wind loads, S: snow loads

support	G kN	M kN	N kN	W kN	S kN	Σ kN
C2	0.000	0.000	0.000	4.739	0.000	4.739
C3	0.000	0.000	0.000	4.739	0.000	4.739
C4	0.000	0.000	0.000	1.723	0.000	1.723
minimal horizontal:						
C1	0.000	0.000	0.000	-3.704	0.000	-3.704
C2	0.000	0.000	0.000	-10.186	0.000	-10.186
C3	0.000	0.000	0.000	-10.186	0.000	-10.186
C4	0.000	0.000	0.000	-3.704	0.000	-3.704

12. purlin D

12.1. position, characteristic values and structural system

material: coniferous timber: C24
cross-section: b = 12.0 cm, h = 18.0 cm
char.valu: EI = 642 kN/m², W = 648.0 cm³, γ = 5.00 kN/m³
fm,k = 24.0 N/mm², fv,k = 4.0 N/mm²
design codes: Eurocode: EN 1990, EN 1991, EN 1995



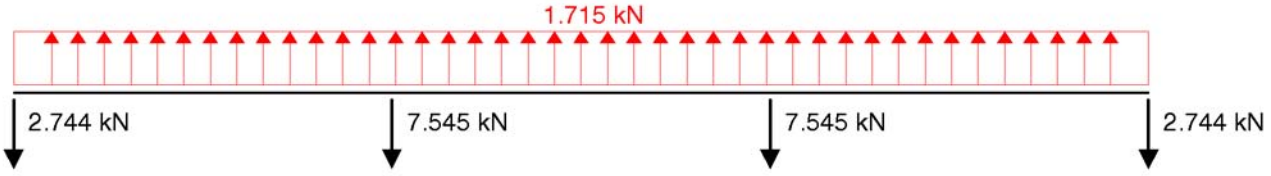
12.2. Loading

The structure of the load corresponds to that of the rafter calculation. Exception: The permanent loads are combined into one load case.The loads are essentially recruited from the bearing reaction forces (support D) of the rafter calculation. Exception: man loads.
Only the relevant load cases are logged here that make a significant contribution to the extrema of the assigned action.

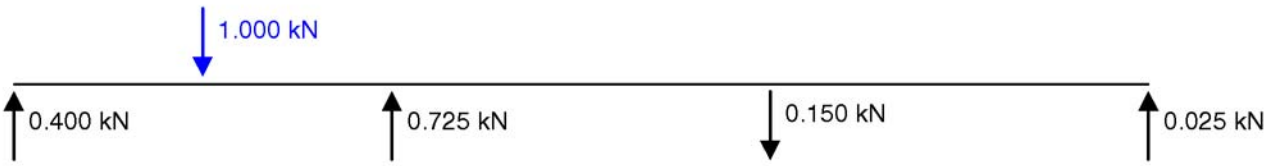
12.2.1. permanent loads

System, loading + support reactions from vertical loads

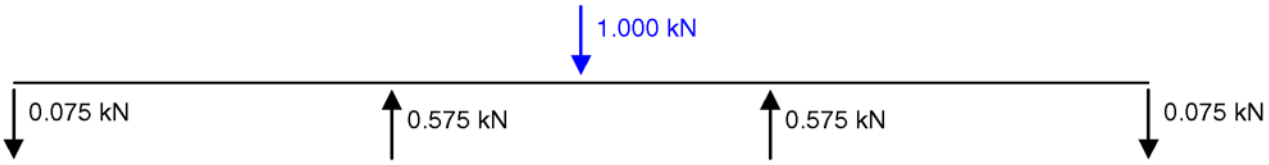
from support force (s D)	from rafter load c. dead load	-0.274 kN / 0.700 m	-0.391 kN/m
from support force (s D)	from rafter l. c. outer skin	-0.733 kN / 0.700 m	-1.048 kN/m
from support force (s D)	from rafter l.c. int. finish w(1)	-0.269 kN / 0.700 m	-0.384 kN/m
dead load purlin	(5.000 kN/m³ * 0.180 m * 0.120 m)		0.108 kN/m
sum permanent loads (vertical)			**-1.715 kN/m**



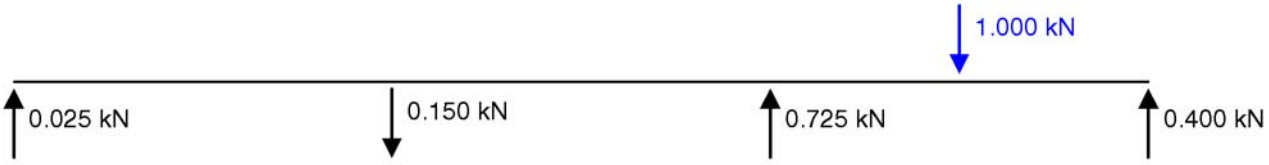
12.2.2. man load position 1



12.2.3. man load position 2



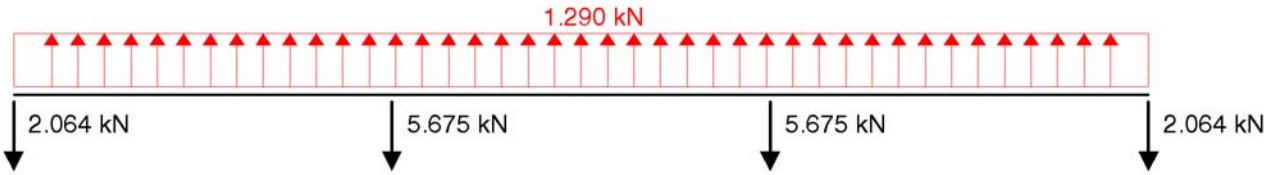
12.2.4. man load position 3



12.2.5. live load (1)

System, loading + support reactions from vertical loads

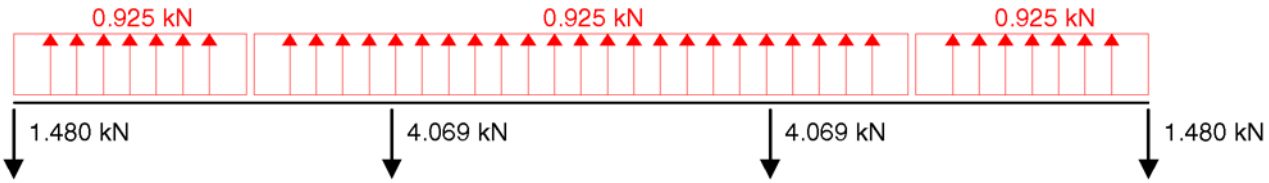
from support force (s D) from rafter l. c. live load	-0.903 kN / 0.700 m	**-1.290 kN/m**



12.2.6. wind from left side (2)

System, loading + support reactions from vertical loads

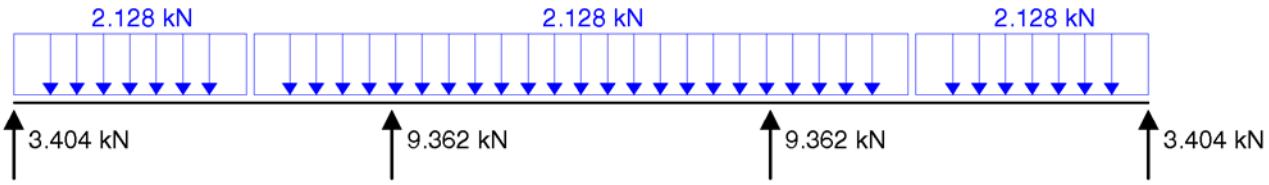
q1 from support force (s D) from rafter l.c. wind from left s (2)	-0.647 kN / 0.700 m	**-0.925 kN/m**
q2 from support force (s D) from rafter l.c. wind from left s (2)	-0.647 kN / 0.700 m	**-0.925 kN/m**
q3 from support force (s D) from rafter l.c. wind from left s (2)	-0.647 kN / 0.700 m	**-0.925 kN/m**



12.2.7. wind from right side (2)

System, loading + support reactions from vertical loads

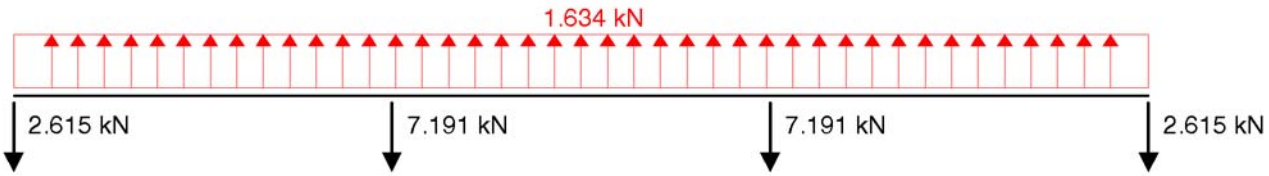
q1 from support force (s D) from rafter l.c. wind from right s (2)	1.489 kN / 0.700 m	**2.128 kN/m**
q2 from support force (s D) from rafter l.c. wind from right s (2)	1.489 kN / 0.700 m	**2.128 kN/m**
q3 from support force (s D) from rafter l.c. wind from right s (2)	1.489 kN / 0.700 m	**2.128 kN/m**



12.2.8. snow (3)

System, loading + support reactions from vertical loads

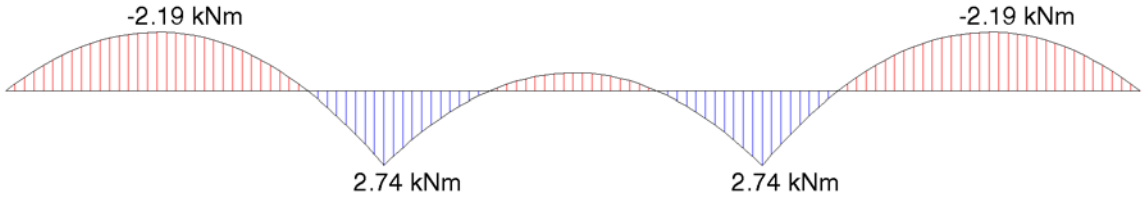
from support force (s D) from rafter l. c. drift right side	-1.144 kN / 0.700 m	**-1.634 kN/m**



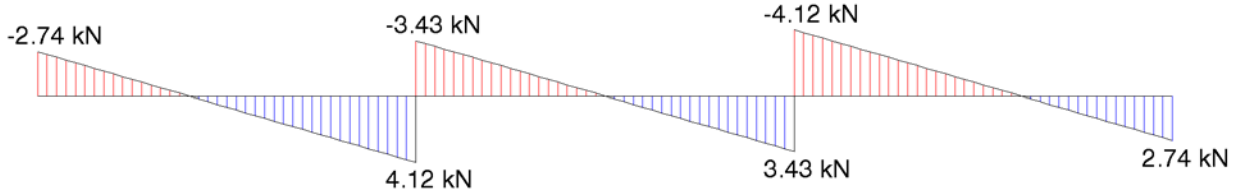
12.3. Extremal from action effects

12.3.1. permanent loads

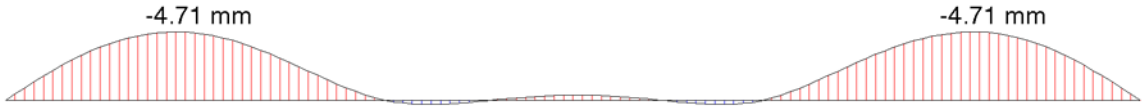
extremal flectural moments from vertical loads (permanent loads)



extremal shear forces from vertical loads (permanent loads)

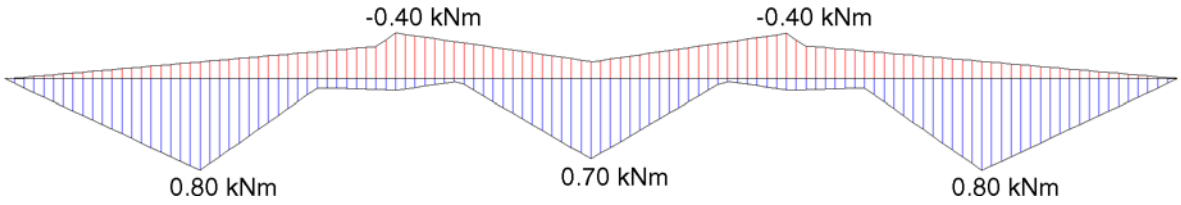


extremal deformations from vertical loads (permanent loads)

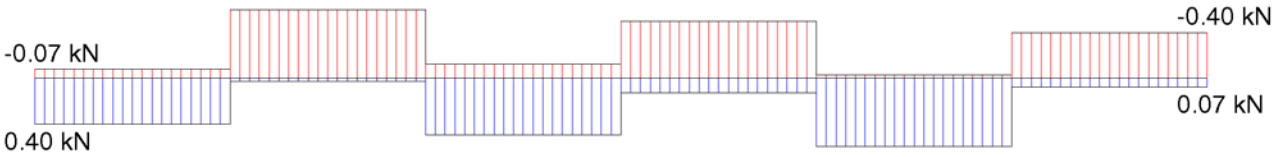


12.3.2. extremal man loads

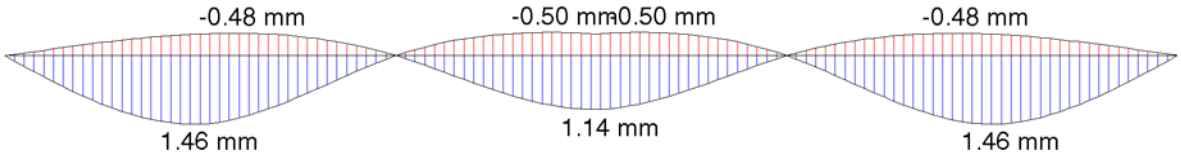
extremal flectural moments from vertical loads (man loads)



extremal shear forces from vertical loads (man loads)

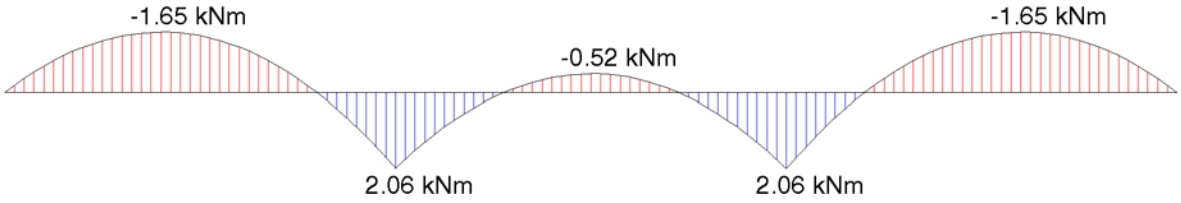


extremal deformations from vertical loads (man loads)

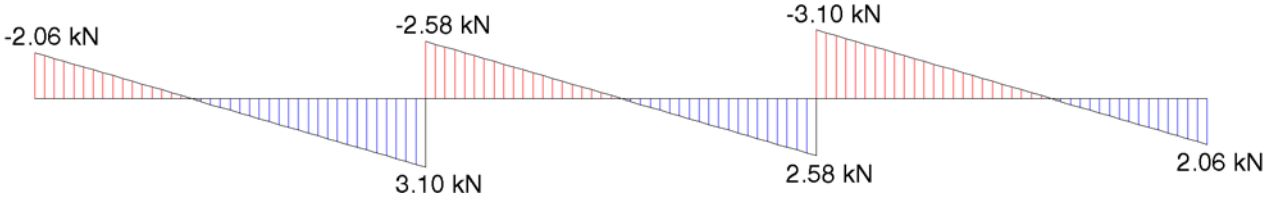


12.3.3. extremal live loads

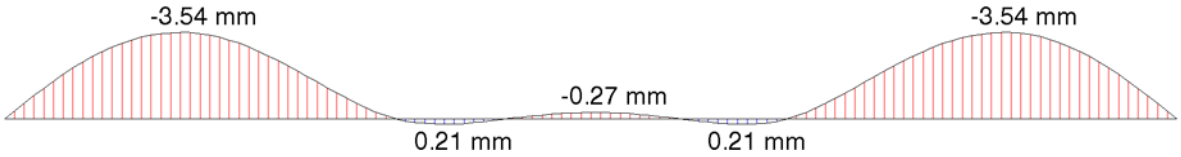
extremal flectural moments from vertical loads (live loads)



extremal shear forces from vertical loads (live loads)

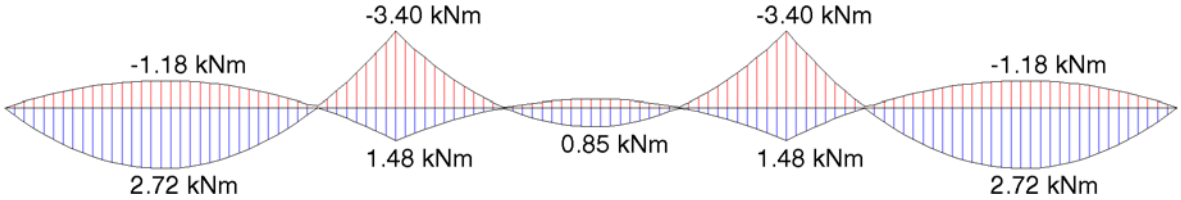


extremal deformations from vertical loads (live loads)

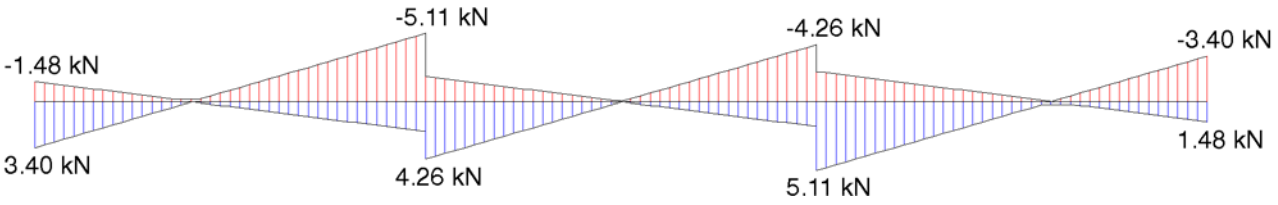


12.3.4. extremal wind loads

extremal flectural moments from vertical loads (wind)



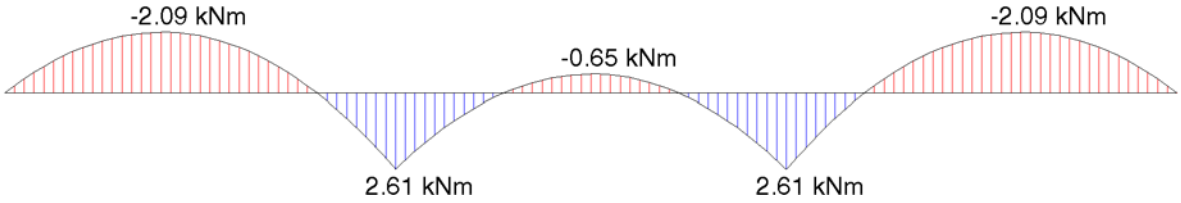
extremal shear forces from vertical loads (wind)



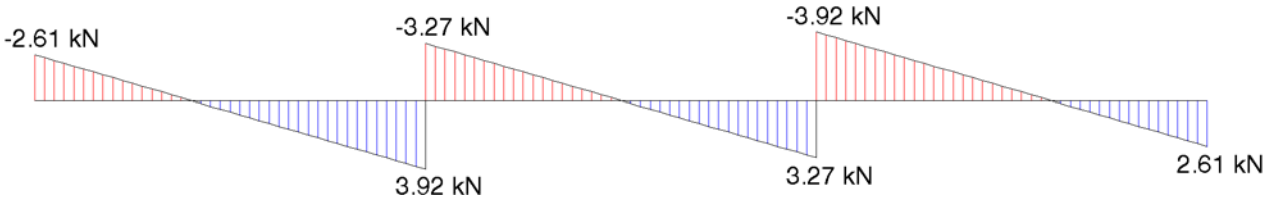
extremal deformations from vertical loads (wind)

12.3.5. extremal snow loads

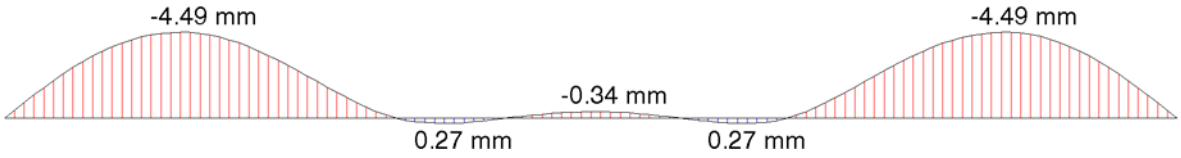
extremal flectural moments from vertical loads (snow)



extremal shear forces from vertical loads (snow)



extremal deformations from vertical loads (snow)



12.4. Verifications

12.4.1. degree of utilizations in the permanent and transienten design situation

The determination of the extreme values for the permanent and temporary design situation is carried out kmod-group-wise under variation of the guiding action with the partial safety and combination coefficients shown in the rafter calculation. For details see chapter 7.1.2. The utilisation rates are determined acc. to EC5 (6.1.6 bending and 6.1.7 thrust) with γ_M = 1.30. They result as shown below

max U = 93% $\Rightarrow$ verification successful.

12.4.2. Utilisation rates for fire protection verification

verification method, required fire resistance period, partial safety factors and combination coefficients see chapter 7.1.3.
Fire exposure: four-sided. h_{red} = 12.6 cm, b_{red} = 6.6 cm, k_{φ} = 1.25, k_{mod} = 1.00, f_{md} = 30.0 N/mm²
The utilisation rates are determined acc. to EC5 6.1.6 with γ_M = 1.00. They result as shown below.

max U = 70% ⇒ verification successful.

12.4.3. Serviceability utilisation rates without creep influence (w_{inst})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below.

max U = 99% ⇒ verification successful.

12.4.4. Serviceability utilisation rates with creep influence (w_{fin})

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below with k_{def} = 0.60

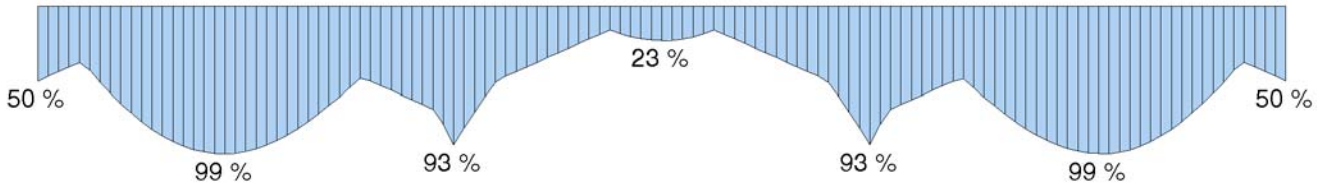
max U = 83% ⇒ verification successful.

12.4.5. Serviceability utilisation rates with creep influence ($w_{fin,net}$)

Comparison lengths = field lengths. For limit values and combination coefficients see chapter 7.2.3. The utilisation rates are determined for the quasi-permanent combination. They result as shown below with k_{def} = 0.60

max U = 69% ⇒ verification successful.

12.4.6. maximum utilization of all verifications



max U = 99% ⇒ all verifications successful.

12.5. extremal support reactions

Abbreviations: G: permanent loads, M: man loads, N: live loads, W: wind loads, S: snow loads

support	G kN	M kN	N kN	W kN	S kN	Σ kN
maximal vertical:						
D1	-2.744	0.400	0.000	3.404	0.000	1.061
D2	-7.545	0.725	0.000	9.362	0.000	2.542
D3	-7.545	0.725	0.000	9.362	0.000	2.542
D4	-2.744	0.400	0.000	3.404	0.000	1.061
minimal vertical:						
D1	-2.744	-0.075	-2.064	-1.480	-2.615	-8.977
D2	-7.545	-0.150	-5.675	-4.069	-7.191	-24.630
D3	-7.545	-0.150	-5.675	-4.069	-7.191	-24.630
D4	-2.744	-0.075	-2.064	-1.480	-2.615	-8.977