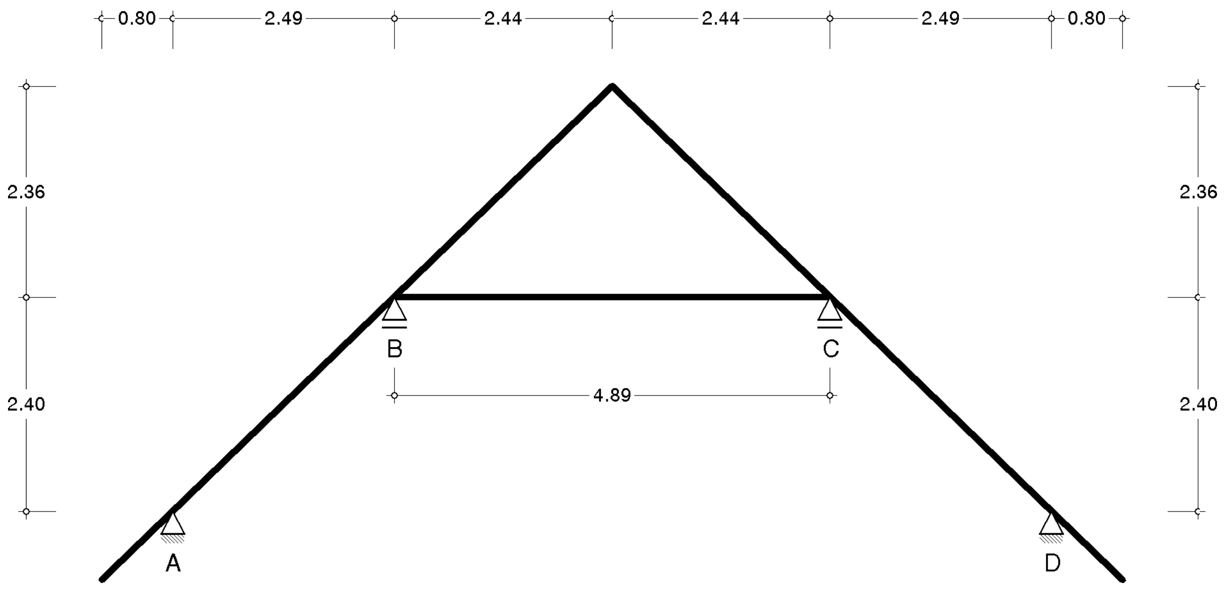
1. System description

1.1. structural system



1.2. system parameters

roof pitches:	left side: 44.00° - right side: 44.00°
apex point:	rafters are hinged connected at apex.
collar beam:	collar beam and rafter are hinged connected.
rafter distan.:	0.850 m
material:	BSH (EC): GL24h
cross-sections:	b/h in cm: left rafter 8.0/18.0, right rafter 8.0/18.0, collar b. 12.0/22.0
notches:	at the support points: A: 3.0cm, B: 3.0cm, C: 3.0cm, D: 3.0cm
roof overhangs:	left side: 0.80 m, right side: 0.80 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.)	7.966	11500	18.0	8.0	144.0	3888.0	432.0
rafter (right s.)	7.966	11500	18.0	8.0	144.0	3888.0	432.0
collar beam	4.888	11500	22.0	12.0	264.0	10648.0	968.0

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)

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**
4: live load	additive (live load on collar beam)
wind loads	**wind loads**
5: wind from left side (1)	alternative (centre area (pressure,pressure))
6: wind from left side (2)	alternative (centre area (pressure,suction))
7: wind from left side (3)	alternative (centre area (suction,suction))
8: wind from left side (4)	alternative (centre area (suction,pressure))
9: wind from right side (1)	alternative (centre area (pressure, pressure))
10: wind from right side (2)	alternative (centre area (suction, pressure))
11: wind from right side (3)	alternative (centre area (suction, suction))
12: wind from right side (4)	alternative (centre area (pressure, suction))
13: wind on gable (1)	alternative (edge region)
14: wind on gable (2)	alternative (centre area)
15: wind on gable (3)	alternative (backside area)
snow loads	**locations up to NN+1000m**
16: snow fully	alternative
17: drift left side	alternative
18: 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 = **0.807 kN/m**
(on both rafters)

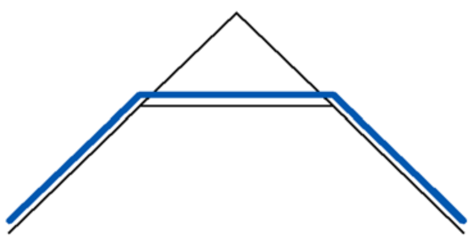
description	value
Interl. ti	0.550 kN/m²
formwork / lathing	0.150 kN/m²
solar- / photovoltaic	0.250 kN/m²
load sum :	**0.950 kN/m²**

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 = **0.297 kN/m**
(load arrangement: see adjacent sketch)

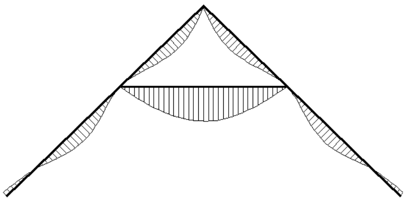


description	value
insulation and facing	0.350 kN/m²
load sum :	**0.350 kN/m²**

2.4. Extremal from action effect permanent loads

extremal deflections

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



(max w = 2.6 mm, min w = -0.4 mm)

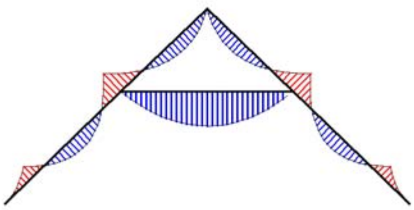
extremal support reactions

sum of all permanent loads in kN

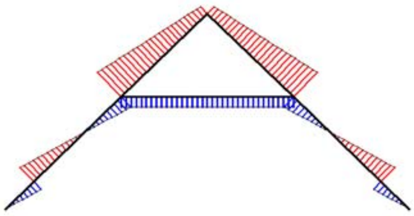
supp.	H	V
A	-0.26	3.42
B	-	5.99
C	-	5.99
D	0.26	3.42

extremal internal forces

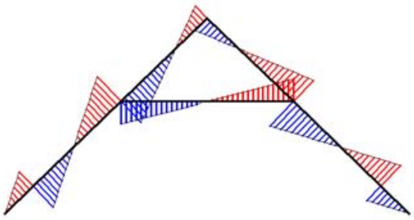
sum of all permanent loads



max M = 1.28 kNm, min M = -0.96 kNm



max N = 1.17 kN, min N = -2.90 kN

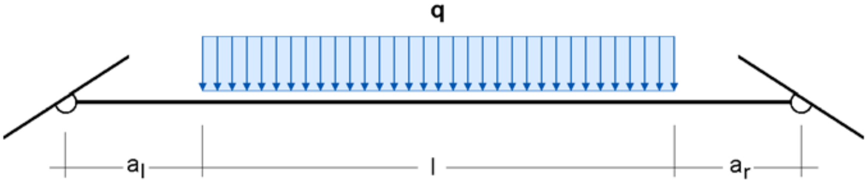


max V = 1.59 kN, min V = -1.59 kN

3. Collar beam live loads

3.1. Action effect from collar beam live loads

The collar beam live loads are arranged as shown in the following sketch.
load value: q = 2.00 kN/m² * distance between rafters = 1.700 kN/m
distances: a1 = 1.00 m, l = 2.89 m, a2 = 1.00 m



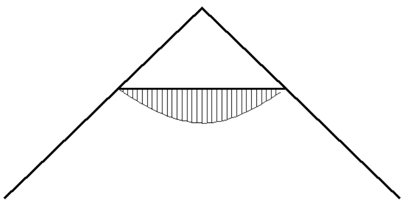
The following additive load cases are analysed.

LF	description
4	live load

3.2. 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 = 8.3 mm, min w = 0.0 mm)

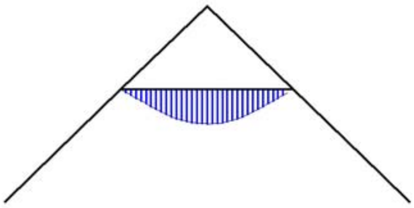
extremal support reactions

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

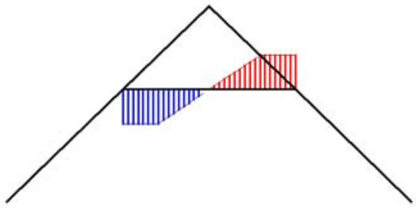
supp.	H		V	
	min	max	min	max
A	0.00	0.00	0.00	0.00
B	-	-	0.00	2.45
C	-	-	0.00	2.45
D	0.00	0.00	0.00	0.00

extremal internal forces

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



max M = 4.23 kNm, min M = 0.00 kNm



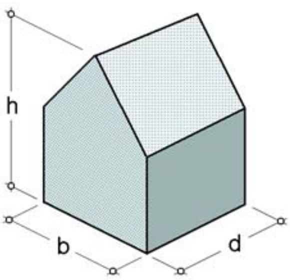
max V = 2.45 kN, min V = -2.45 kN

4. wind loads

4.1. Action effect of wind loads

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

wind zone:	2
h + NN:	67 m
factor:	1.0000
qref:	0.39 kN/m²
h:	8.90 m
b:	9.85 m
d:	10.92 m
⇒ **q(h):**	**0.64 kN/m²**



The following alternative load cases are analysed.

LF	description	explanation
5	wind from left side (1)	centre area (pressure,pressure)
6	wind from left side (2)	centre area (pressure,suction)
7	wind from left side (3)	centre area (suction,suction)
8	wind from left side (4)	centre area (suction,pressure)
9	wind from right side (1)	centre area (pressure, pressure)
10	wind from right side (2)	centre area (suction, pressure)
11	wind from right side (3)	centre area (suction, suction)
12	wind from right side (4)	centre area (pressure, suction)
13	wind on gable (1)	edge region
14	wind on gable (2)	centre area
15	wind on gable (3)	backside area

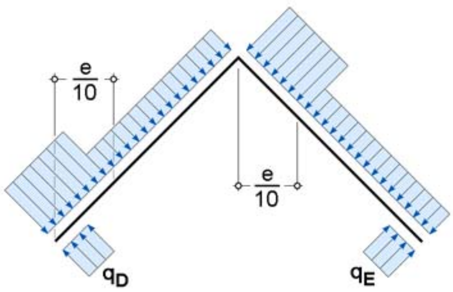
wind from left side

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.

$e = \min(d,2h) = 10.92 \quad \Rightarrow \quad \frac{e}{10} = 1.09$

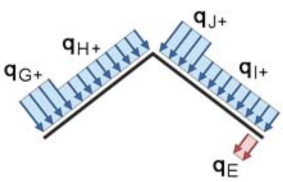
EC 1-1-4	Tabel 7.4a					Tabel 7.1	
Input value	α = 44.00°			α = 44.00°		h/b = 0.90	
Zone	F	G	H	I	J	D	E
$c_{pe.10}$ (-)	-0.03	-0.03	-0.01	-0.21	-0.31	+0.79	-0.47
q()- kN/m	**-0.02**	**-0.02**	**-0.01**	**-0.12**	**-0.17**	**+0.42**	**-0.26**
$c_{pe.10}$ (+)	+0.70	+0.70	+0.59	+0.00	+0.00		
q()+ kN/m	**+0.38**	**+0.38**	**+0.32**	+0.00	+0.00		

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

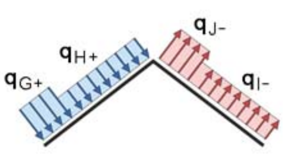


q_D and q_E are acting only in the case of roof overhangs if they increase the local effect unfavorably

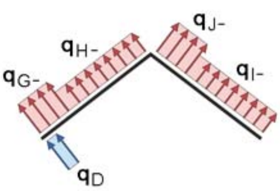
considered load cases (wind from left side)



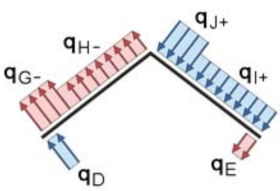
wind from left side (1)
centre area (pressure,pressure)



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)

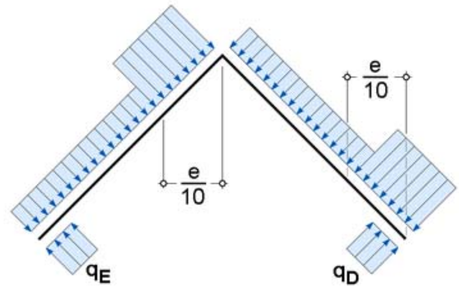
wind from right side

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.

$e = \min(d,2h) = 10.92 \quad \Rightarrow \quad \frac{e}{10} = 1.09$

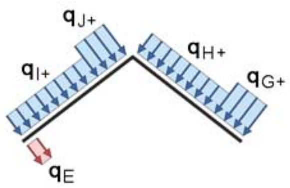
EC 1-1-4	Tabel 7.4a					Tabel 7.1	
Input value	α = 44.00°			α = 44.00°		h/b = 0.90	
Zone	F	G	H	I	J	D	E
$c_{pe.10}$ (-)	-0.03	-0.03	-0.01	-0.21	-0.31	+0.79	-0.47
$q_{()-}$ kN/m	**-0.02**	**-0.02**	**-0.01**	**-0.12**	**-0.17**	**+0.42**	**-0.26**
$c_{pe.10}$ (+)	+0.70	+0.70	+0.59	+0.00	+0.00		
$q_{()+}$ kN/m	**+0.38**	**+0.38**	**+0.32**	+0.00	+0.00		

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

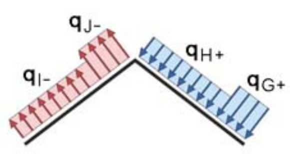


q_D and q_E are acting only in the case of roof overhangs, if they increase the local effect unfavorably

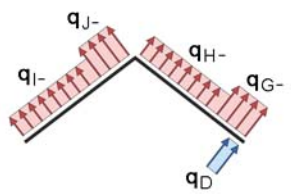
considered load cases (wind from right side)



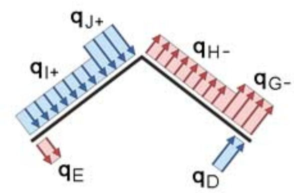
wind from right side (1)
centre area (pressure, pressure)



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



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



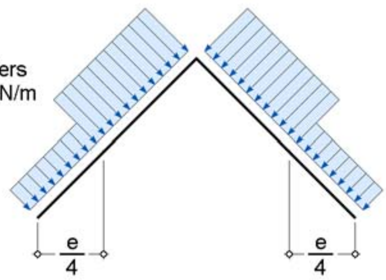
wind from right side (4)
centre area (pressure, suction)

wind on gable

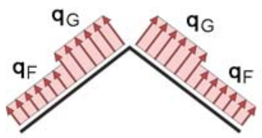
The arrows in the adjacent sketch represent positive load directions (pressure).
In case of negative q-values (suction) the load acts in the reverse direction.

$e = \min(d,2h) = 9.85$ m $\quad \Rightarrow \quad \frac{e}{4} = 2.46$ m $\qquad q = c_{pe.10} * q(h) *$ distance between rafters in kN/m

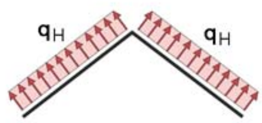
EC 1-1-4	Tabel 7.4b							
Input value	left rafter α = 44.00°				right rafter α = 44.00°			
Zone	F	G	H	I	F	G	H	I
$c_{pe.10}$	-1.10	-1.40	-0.89	-0.50	-1.10	-1.40	-0.89	-0.50
q kN/m	**-0.59**	**-0.76**	**-0.48**	**-0.27**	**-0.59**	**-0.76**	**-0.48**	**-0.27**



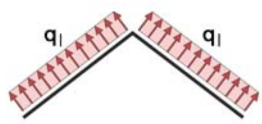
considered load cases (wind on gable)



wind on gable (1)
edge region



wind on gable (2)
centre area

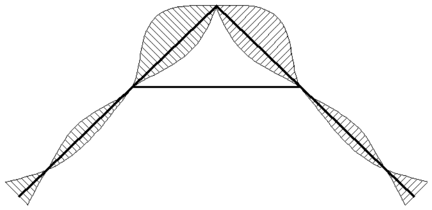


wind on gable (3)
backside area

4.2. 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 = 0.6 mm, min w = -1.5 mm)

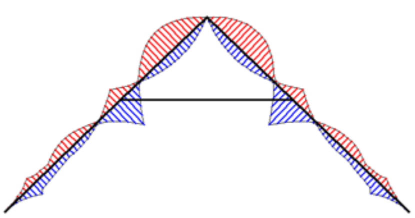
extremal support reactions

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

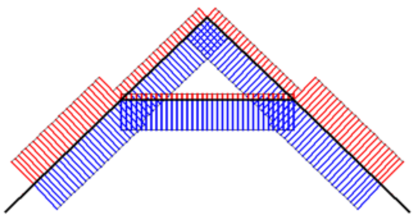
supp.	H		V	
	min	max	min	max
A	-1.29	1.51	-0.91	0.79
B	-	-	-3.02	1.59
C	-	-	-3.02	1.59
D	-1.51	1.29	-0.91	0.79

extremal internal forces

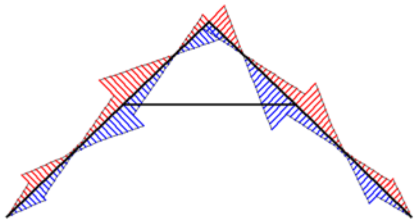
Extremal from all load cases of the action effect wind loads



max M = 0.93 kNm, min M = -0.67 kNm



max N = 1.19 kN, min N = -1.10 kN



max V = 1.56 kN, min V = -1.56 kN

5. snow loads

5.1. Action effect of snow loads

snow load zone:	2
h + NN:	67 m
$\Rightarrow s_k$:	0.85 kN/m²
building model:	free-standing

The following alternative load cases are analysed.

LF	description
16	snow fully
17	drift left side
18	drift right side

5.2. load case 16: snow fully

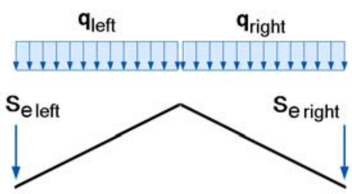
left side: $\alpha = 44°\ \rightarrow\ \mu_1(\alpha) = 0.43\ \rightarrow\ q_{left} = a\ \mu_1(\alpha)\ s_k =$ 0.31 kN/m

right side: $\alpha = 44°\ \rightarrow\ \mu_1(\alpha) = 0.43\ \rightarrow\ q_{right} = a\ \mu_1(\alpha)\ s_k =$ 0.31 kN/m

point loads
(only in case of roof overhangs)

$S_{e\,left} = 0.4\,a\,(\mu_1(\alpha)\,s_k)^2/\gamma =$ 0.01 kN

$S_{e\,right} = 0.4\,a\,(\mu_1(\alpha)\,s_k)^2/\gamma =$ 0.01 kN



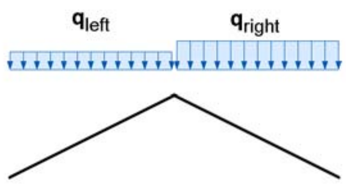
a = distance between rafters, γ = 3.0 kN/m³

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)

5.3. load case 17: drift left side

left side: $\alpha = 44°\ \rightarrow\ \mu_1(\alpha) = 0.43\ \rightarrow\ q_{left} = \frac{1}{2}\,a\ \mu_1(\alpha)\ s_k =$ 0.15 kN/m

right side: $\alpha = 44°\ \rightarrow\ \mu_1(\alpha) = 0.43\ \rightarrow\ q_{right} = a\ \mu_1(\alpha)\ s_k =$ 0.31 kN/m



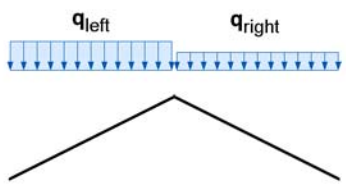
a = distance between rafters

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

5.4. load case 18: drift right side

left side: $\alpha = 44°\ \rightarrow\ \mu_1(\alpha) = 0.43\ \rightarrow\ q_{left} = a\ \mu_1(\alpha)\ s_k =$ 0.31 kN/m

right side: $\alpha = 44°\ \rightarrow\ \mu_1(\alpha) = 0.43\ \rightarrow\ q_{right} = \frac{1}{2}\,a\ \mu_1(\alpha)\ s_k =$ 0.15 kN/m



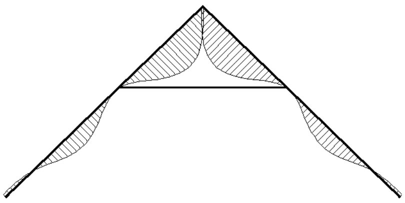
a = distance between rafters

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

5.5. 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 = 0.3 mm, min w = 0.0 mm)

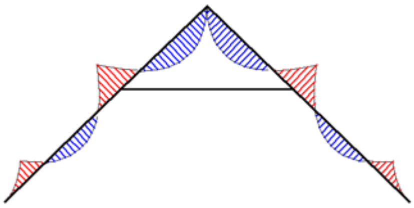
extremal support reactions

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

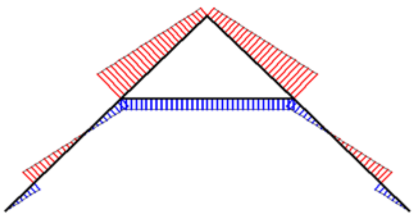
supp.	H		V	
	min	max	min	max
A	-0.07	0.00	0.00	0.67
B	-	-	0.00	1.11
C	-	-	0.00	1.11
D	0.00	0.07	0.00	0.67

extremal internal forces

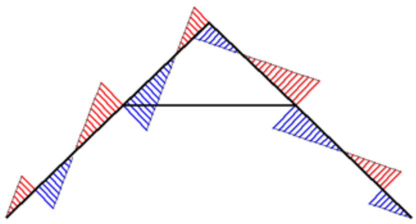
Extremal from all load cases of the action effect snow loads



max M = 0.14 kNm, min M = -0.21 kNm



max N = 0.24 kN, min N = -0.74 kN



max V = 0.33 kN, min V = -0.33 kN

6. Verifications

6.1. Verification of ultimate limit state

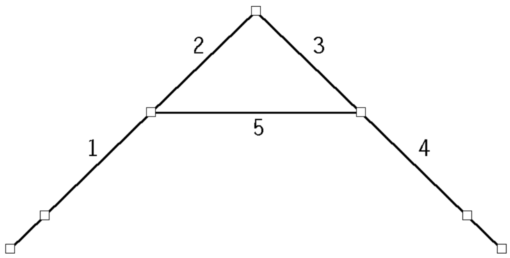
6.1.1. stability

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

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}	$\Rightarrow k_c$
1	3.46	1.00	3.46	0.7183
2	3.40	1.00	3.40	0.7336
3	3.40	1.00	3.40	0.7336
4	3.46	1.00	3.46	0.7183
5	4.89	1.00	4.89	0.5802

sections



6.1.2. main verification

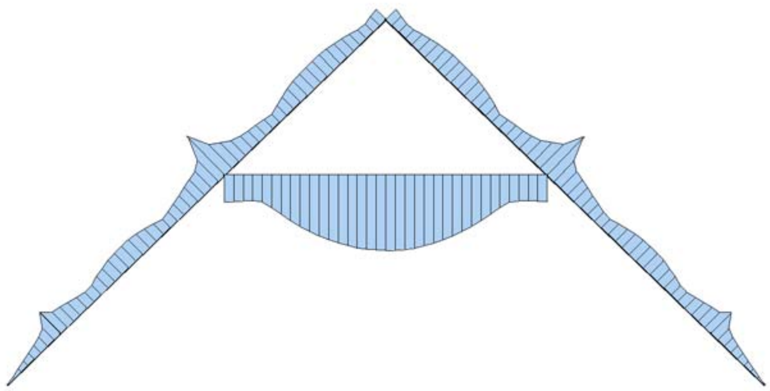
verification of load-carrying capacity for permanent and transient design situations

service class of building	2
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
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

6.1.2.1. maximal utilization



beam	max U
left rafter	0.41
right rafter	0.41
collar b.	0.57

6.1.3. special verification "Norddeutsche Tiefebene"

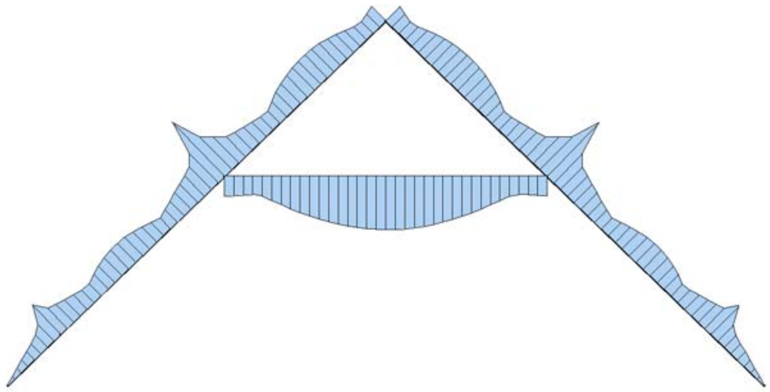
verification of load-carrying capacity for accidental design situations

service class of building	2
material safety factor	1.00
net cross-sections	notches considered
combination of internal forces	acc. to Eurocode EN 1990

safety and combination coefficients, classes of duration of load

action effect	γ_{sup}	γ_{inf}	Ψ_{dom}	Ψ_{sub}	KLED	k_{mod}
permanent loads	1.00	1.00	1.00	1.00	permanent	0.60
collar b. live ld.	1.00	0.00	0.50	0.30	med.-term	0.80
wind loads	1.00	0.00	0.20	0.00	sh.-v.sh.	1.00
snow loads	2.30	2.30	1.00	1.00	sh.-term	0.90

6.1.3.1. maximal utilization



beam	max U
left rafter	0.23
right rafter	0.23
collar b.	0.16

6.1.4. 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	30 minutes
combustion depth	2.40 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)

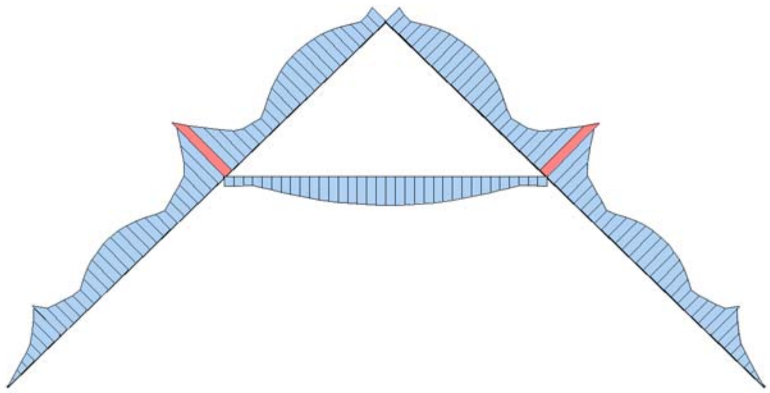
	h_i	b_i	$f_{m,d}$	$f_{c0,d}$	$f_{t,d}$
	cm	cm	N/mm²	N/mm²	N/mm²
rafter (left s)	11.80	1.80	27.60	27.60	22.08
rafter (right s)	11.80	1.80	27.60	27.60	22.08
collar b.	15.80	5.80	27.60	27.60	22.08

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
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.

6.1.4.1. maximal utilization



beam	max U
left rafter	1.04
right rafter	1.04
collar b.	0.39

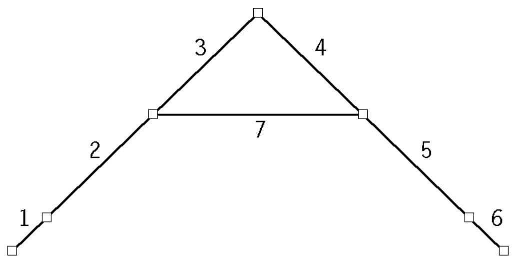
6.2. verifications of serviceability limit states

6.2.1. comparative lengths

for calculation of degree of utilization

section	length	l_v
-	m	m
1	1.11	1.11
2	3.46	3.46
3	3.40	3.40
4	3.40	3.40
5	3.46	3.46
6	1.11	1.11
7	4.89	4.89

sections



6.2.2. limit values

deformation	(in span)	(at cantilever)
w_{inst}	$l_v/300$	$l_v/150$
w_{fin}	$l_v/200$	$l_v/100$
$w_{net,fin}$	$l_v/300$	$l_v/150$

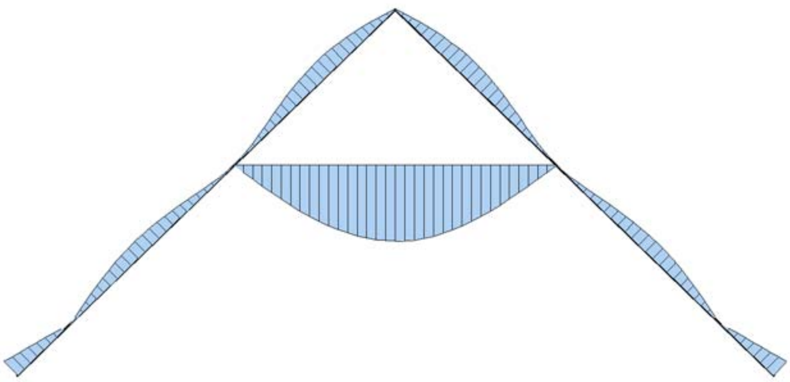
6.2.3. design situation w_{inst}

combination coefficients

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

service class 2
⇒ k_{def} = 0.80

6.2.3.1. maximal utilization



beam	max U
left rafter	0.17
right rafter	0.17
collar b.	0.67

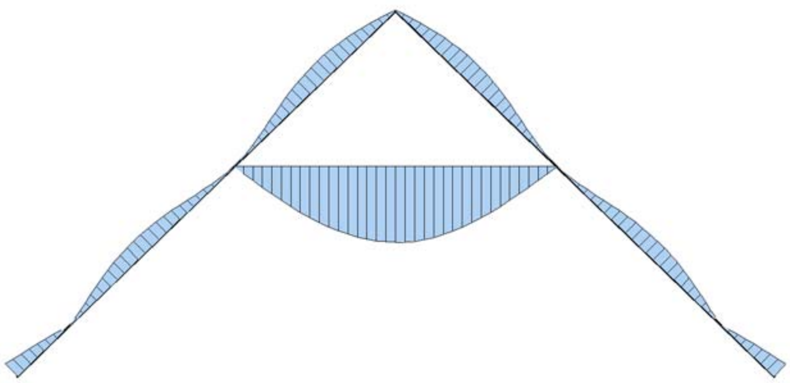
6.2.4. design situation w_{fin}

combination coefficients

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

service class 2
⇒ k_{def} = 0.80

6.2.4.1. maximal utilization



beam	max U
left rafter	0.15
right rafter	0.15
collar b.	0.61

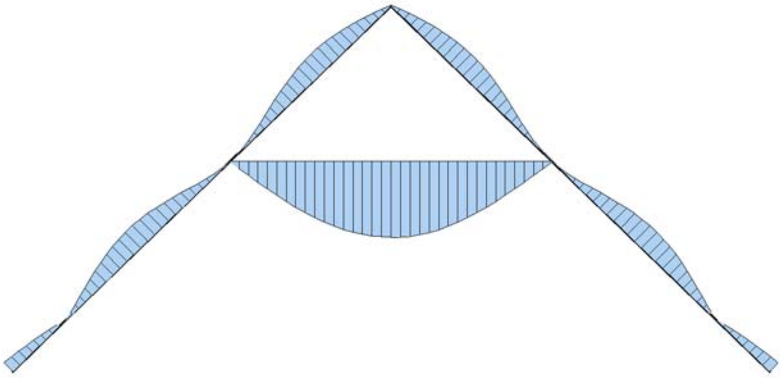
6.2.5. design situation $w_{net,fin}$

combination coefficients

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

service class 2
⇒ k_{def} = 0.80

6.2.5.1. maximal utilization



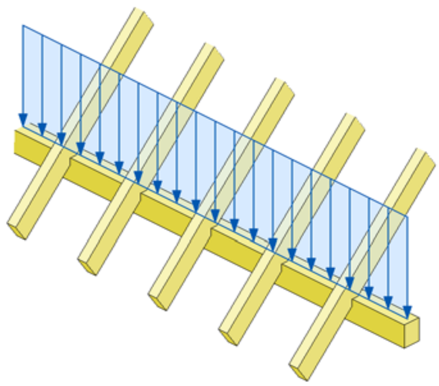
beam	max U
left rafter	0.16
right rafter	0.16
collar b.	0.56

7. 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.

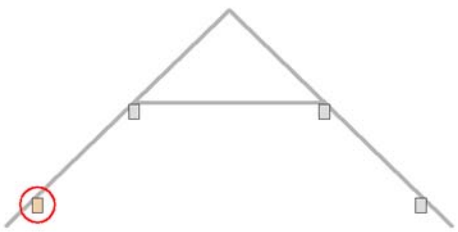
	G	Q		G+Q	
		H	V	H	V
	kN/m	kN/m	kN/m	kN/m	kN/m
support A					
min A_H	-0.31	-1.60	-0.29	-1.90	3.74
max A_H	-0.31	1.78	-0.18	1.47	3.85
min A_V	4.02	-1.52	-1.08	-1.83	2.95
max A_V	4.02	-0.46	1.71	-0.77	5.74
support B					
min B_V	7.05	0.00	-3.55	0.00	3.50
max B_V	7.05	0.00	6.07	0.00	13.12
support C					
min C_V	7.05	0.00	-3.55	0.00	3.50
max C_V	7.05	0.00	6.07	0.00	13.12
support D					
min D_H	0.31	-1.78	-0.18	-1.47	3.85
max D_H	0.31	1.60	-0.29	1.90	3.74
min D_V	4.02	1.52	-1.08	1.83	2.95
max D_V	4.02	0.46	1.71	0.77	5.74



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.

8. purlin A

8.1. position, characteristic values, explanations



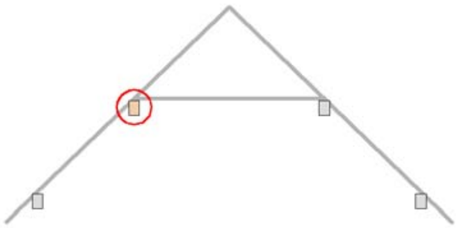
material: coniferous timber: C24

cross-section: b = 18.0 cm, h = 18.0 cm

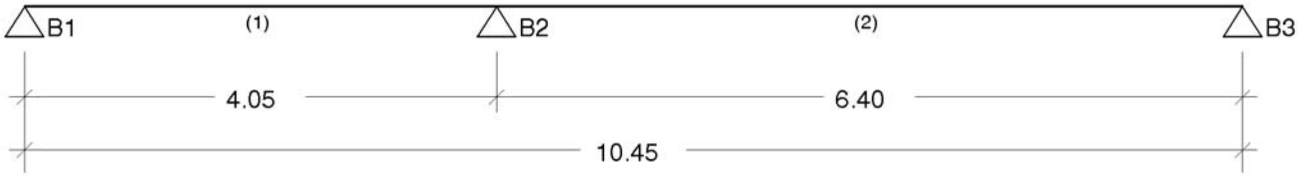
The purlin is continuously supported on the reinforced concrete jamb. To secure the position and prevent lifting the purlin is secured at regular intervals ≤ 1.50 m by an ABC brand anchor.

9. purlin B

9.1. position, characteristic values and structural system



material: BSH (EC): GL24h
cross-section: b = 20.0 cm, h = 36.0 cm
char.valu: EI = 8942 kN/m², W = 4320.0 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



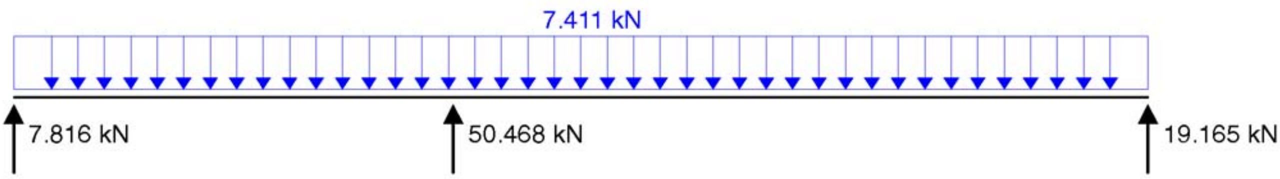
9.2. Loading

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 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.

9.2.1. permanent loads

System, loading + support reactions from vertical loads

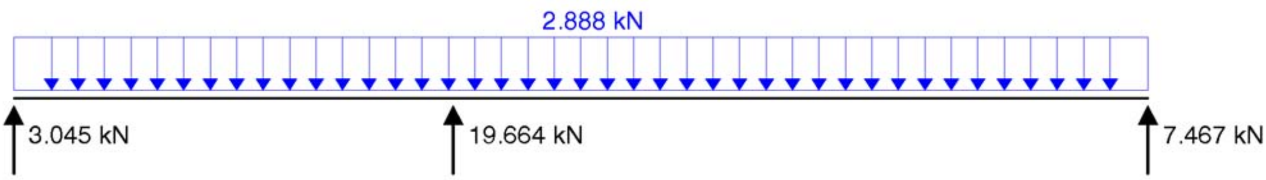
from support force (s B) from rafter load c. dead load	0.685 kN / 0.850 m	0.805 kN/m
from support force (s B) from rafter l. c. outer skin	4.060 kN / 0.850 m	4.777 kN/m
from support force (s B) from rafter l.c. int. finish w(1)	1.249 kN / 0.850 m	1.469 kN/m
dead load purlin (5.000 kN/m³ * 0.360 m * 0.200 m)		0.360 kN/m
sum permanent loads (vertical)		**7.411 kN/m**



9.2.2. live load (1)

System, loading + support reactions from vertical loads

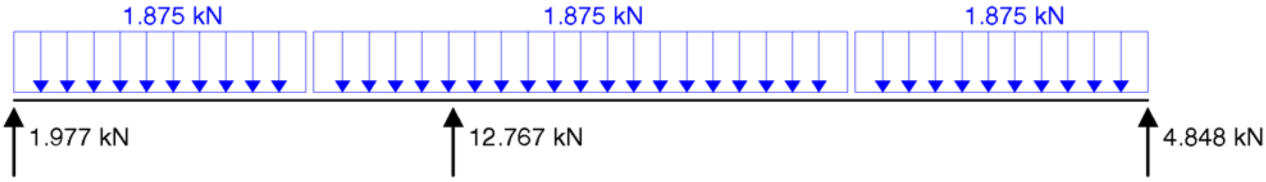
from support force (s B) from rafter l. c. live load	2.455 kN / 0.850 m	**2.888 kN/m**



9.2.3. wind from left side (2)

System, loading + support reactions from <u>vertical</u> loads

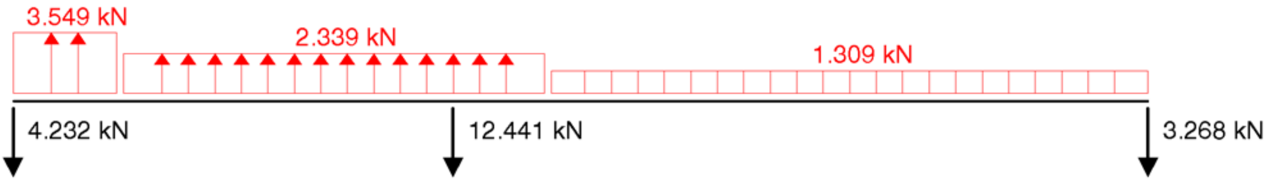
```
q1 from support force (s B) from rafter l.c. wind from left s (2)     1.594 kN / 0.850 m     1.875 kN/m
q2 from support force (s B) from rafter l.c. wind from left s (2)     1.594 kN / 0.850 m     1.875 kN/m
q3 from support force (s B) from rafter l.c. wind from left s (2)     1.594 kN / 0.850 m     1.875 kN/m
```



9.2.4. wind from the front

System, loading + support reactions from <u>vertical</u> loads

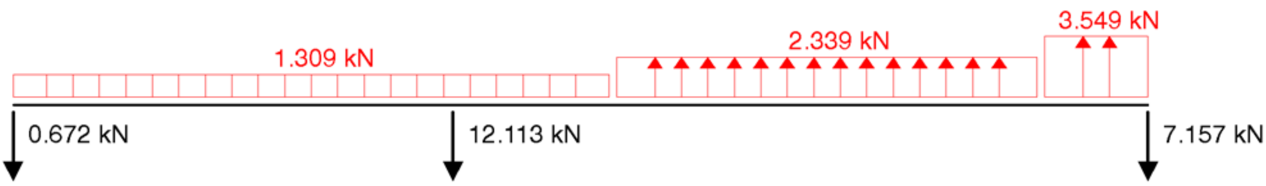
```
q1 from support force (s B) from rafter load c.  wind on gable (1)   -3.017 kN / 0.850 m    -3.549 kN/m
q2 from support force (s B) from rafter load c.  wind on gable (2)   -1.988 kN / 0.850 m    -2.339 kN/m
q3 from support force (s B) from rafter load c.  wind on gable (3)   -1.113 kN / 0.850 m    -1.309 kN/m
```



9.2.5. wind from behind

System, loading + support reactions from <u>vertical</u> loads

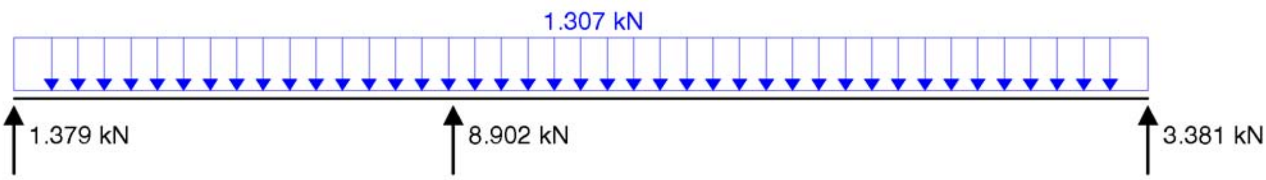
```
q1 from support force (s B) from rafter load c.  wind on gable (3)   -1.113 kN / 0.850 m    -1.309 kN/m
q2 from support force (s B) from rafter load c.  wind on gable (2)   -1.988 kN / 0.850 m    -2.339 kN/m
q3 from support force (s B) from rafter load c.  wind on gable (1)   -3.017 kN / 0.850 m    -3.549 kN/m
```



9.2.6. snow (1)

System, loading + support reactions from <u>vertical</u> loads

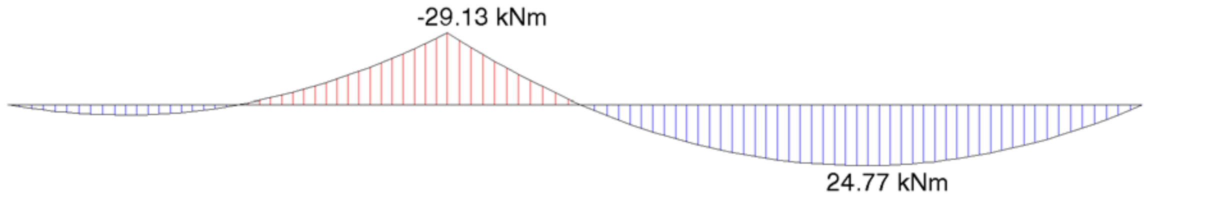
```
from support force (s B) from rafter l. c.  snow fully             1.111 kN / 0.850 m     1.307 kN/m
```



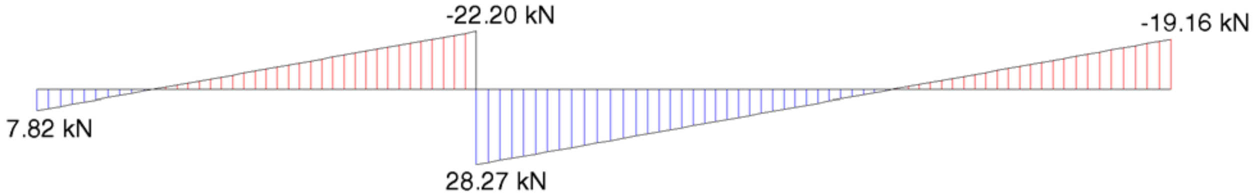
9.3. Extremal from action effects

9.3.1. permanent loads

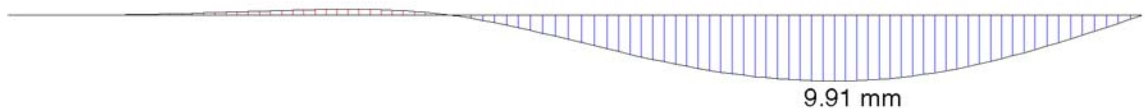
extremal flectural moments from vertical loads (permanent loads)



extremal shear forces from vertical loads (permanent loads)

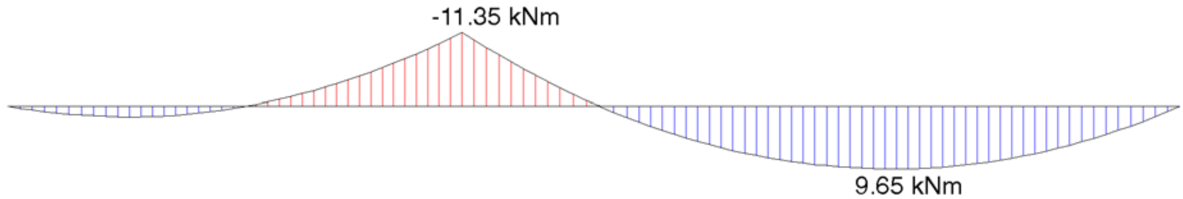


extremal deformations from vertical loads (permanent loads)

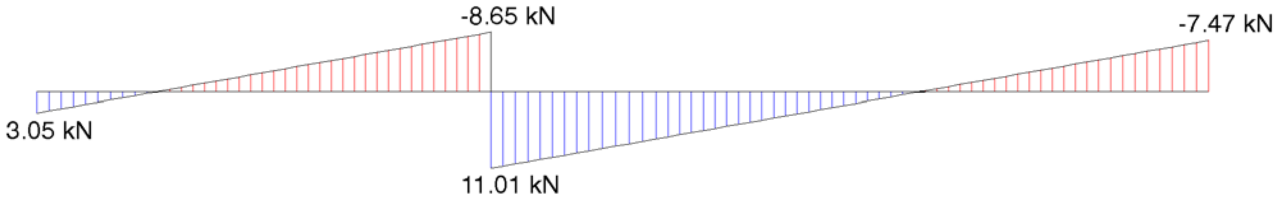


9.3.2. extremal live loads

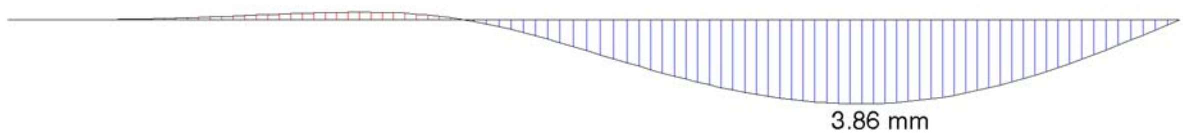
extremal flectural moments from vertical loads (live loads)



extremal shear forces from vertical loads (live loads)

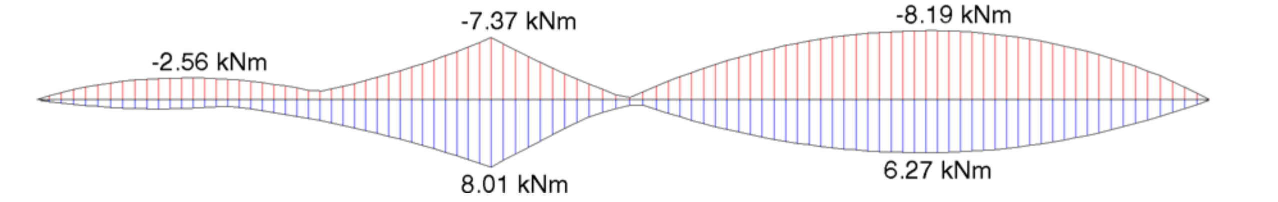


extremal deformations from vertical loads (live loads)

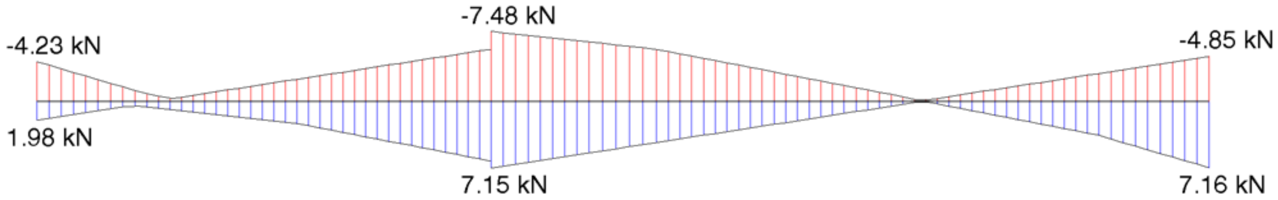


9.3.3. extremal wind loads

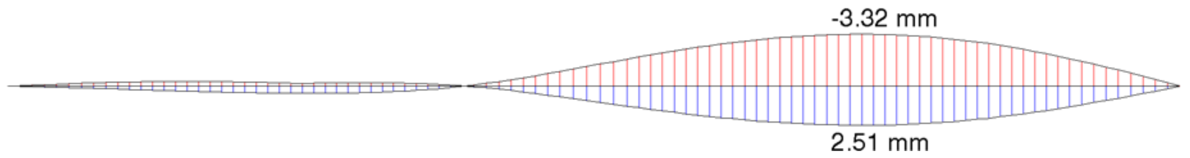
extremal flectural moments from vertical loads (wind)



extremal shear forces from vertical loads (wind)

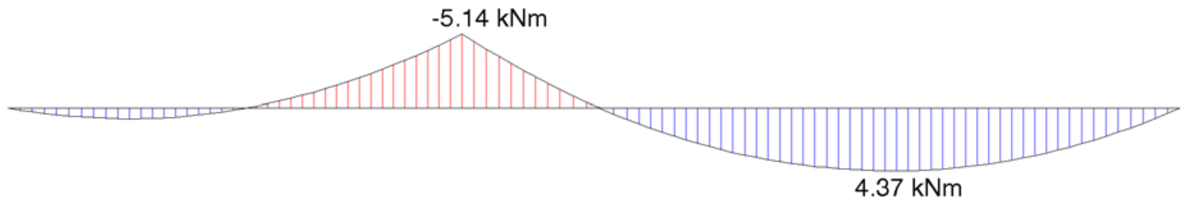


extremal deformations from vertical loads (wind)

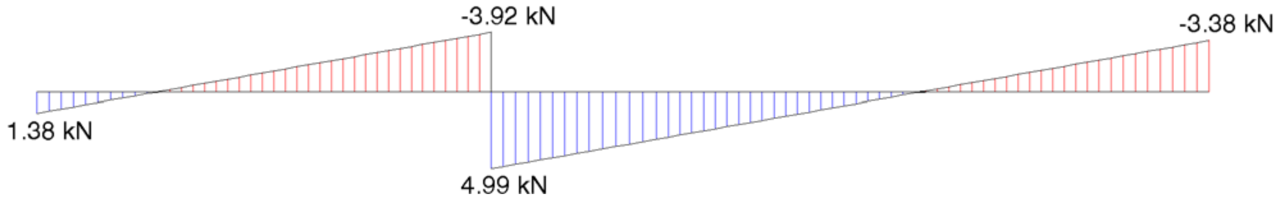


9.3.4. extremal snow loads

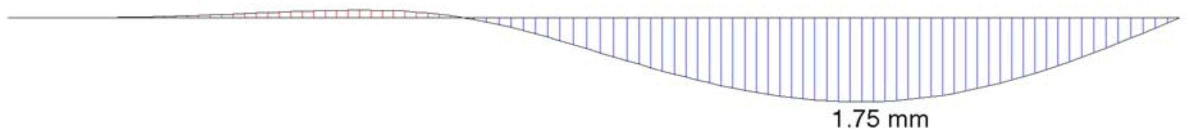
extremal flectural moments from vertical loads (snow)



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 6.1.2. 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 = 88% $\Rightarrow$ verification successful.

9.4.2. Utilisation rates in the exceptional design situation "Nordeutsche Tiefebene"
The determination of the extreme values for the exceptional 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 6.1.3. The utilisation rates are determined acc. to EC5 (6.1.6 bending and 6.1.7 thrust) with γ_M = 1.00. They result as shown below

max U = 50% $\Rightarrow$ verification successful.

9.4.3. Utilisation rates for fire protection verification
verification method, required fire resistance period, partial safety factors and combination coefficients see chapter 6.1.4. Fire exposure: four-sided. h_{red} = 29.8 cm, b_{red} = 13.8 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 = 60% $\Rightarrow$ verification successful.

9.4.4. Serviceability utilisation rates without creep influence (w_{inst})
Comparison lengths = field lengths. For limit values and combination coefficients see chapter 6.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below.

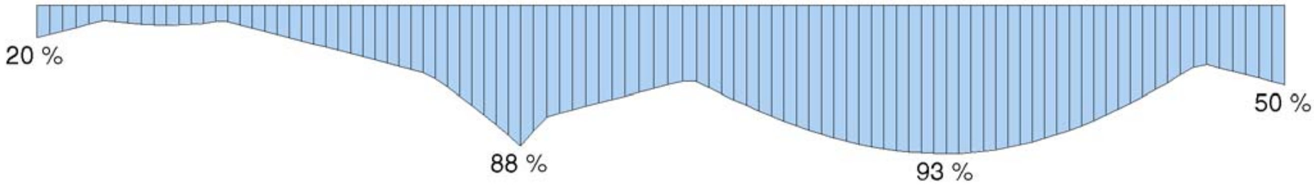
max U = 76% $\Rightarrow$ verification successful.

9.4.5. Serviceability utilisation rates with creep influence (w_{fin})
Comparison lengths = field lengths. For limit values and combination coefficients see chapter 6.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below with k_{def} = 0.80

max U = 78% $\Rightarrow$ verification successful.

9.4.6. Serviceability utilisation rates with creep influence ($w_{fin,net}$)
Comparison lengths = field lengths. For limit values and combination coefficients see chapter 6.2.3. The utilisation rates are determined for the quasi-permanent combination. They result as shown below with k_{def} = 0.80

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

9.4.7. maximum utilization of all verifications


max U = 93% $\Rightarrow$ all verifications successful.

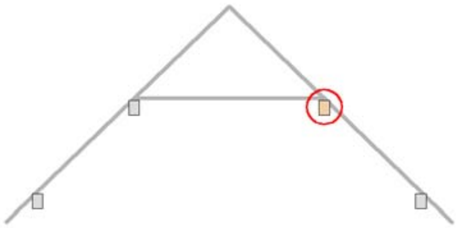
9.5. extremal support reactions

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

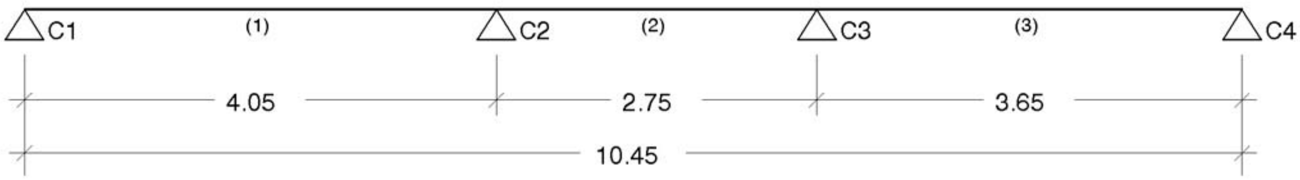
support	G kN	M kN	N kN	W kN	S kN	Σ kN
maximal vertical:						
B1	7.816	---	3.045	1.977	1.379	14.217
B2	50.468	---	19.664	12.767	8.902	91.801
B3	19.165	---	7.467	4.848	3.381	34.861
minimal vertical:						
B1	7.816	---	0.000	-4.232	0.000	3.584
B2	50.468	---	0.000	-12.441	0.000	38.027
B3	19.165	---	0.000	-7.157	0.000	12.008

10. purlin C

10.1. position, characteristic values and structural system



material:	BSH (EC): GL24h
cross-section:	b = 20.0 cm, h = 24.0 cm
char.valu:	EI = 2650 kN/m², W = 1920.0 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



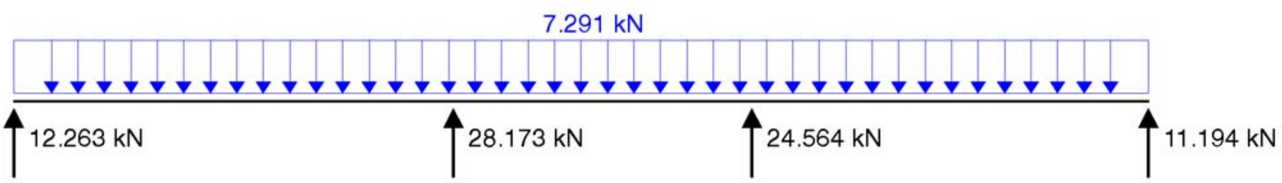
10.2. Loading

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 C) 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

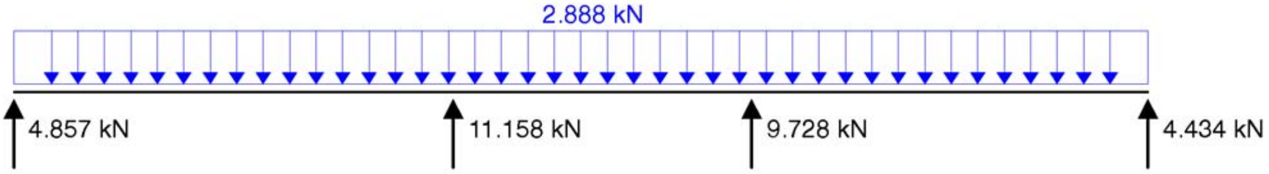
from support force (s C) from rafter load c. dead load	0.685 kN / 0.850 m	0.805 kN/m
from support force (s C) from rafter l. c. outer skin	4.060 kN / 0.850 m	4.777 kN/m
from support force (s C) from rafter l.c. int. finish w(1)	1.249 kN / 0.850 m	1.469 kN/m
dead load purlin (5.000 kN/m³ * 0.240 m * 0.200 m)		0.240 kN/m
sum permanent loads (vertical)		**7.291 kN/m**



10.2.2. live load (1)

System, loading + support reactions from vertical loads

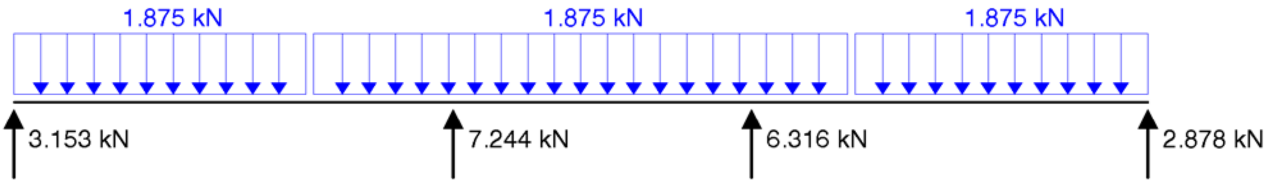
from support force (s C) from rafter l. c. live load 2.455 kN / 0.850 m **2.888 kN/m**



10.2.3. wind from right side (2)

System, loading + support reactions from vertical loads

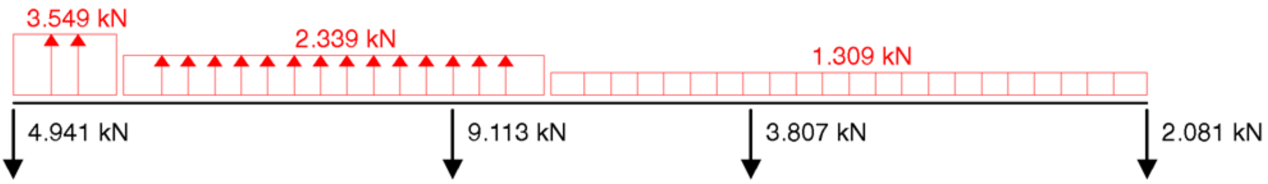
q1 from support force (s C) from rafter l.c. wind from right s (2) 1.594 kN / 0.850 m **1.875 kN/m**
q2 from support force (s C) from rafter l.c. wind from right s (2) 1.594 kN / 0.850 m **1.875 kN/m**
q3 from support force (s C) from rafter l.c. wind from right s (2) 1.594 kN / 0.850 m **1.875 kN/m**



10.2.4. wind from the front

System, loading + support reactions from vertical loads

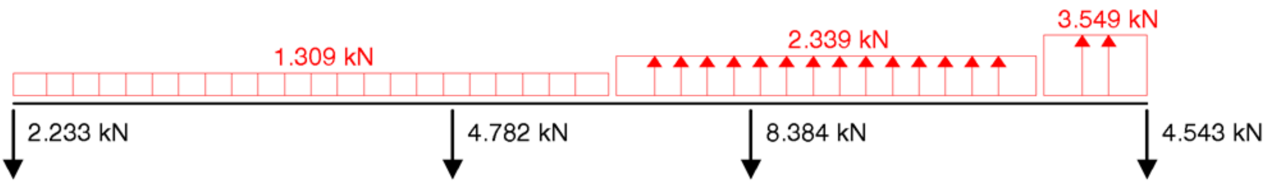
q1 from support force (s C) from rafter load c. wind on gable (1) -3.017 kN / 0.850 m **-3.549 kN/m**
q2 from support force (s C) from rafter load c. wind on gable (2) -1.988 kN / 0.850 m **-2.339 kN/m**
q3 from support force (s C) from rafter load c. wind on gable (3) -1.113 kN / 0.850 m **-1.309 kN/m**



10.2.5. wind from behind

System, loading + support reactions from vertical loads

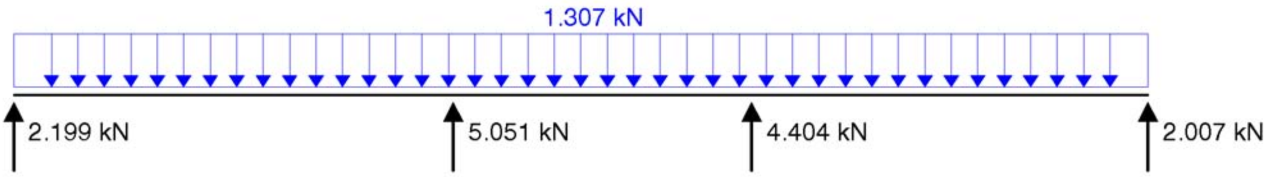
q1 from support force (s C) from rafter load c. wind on gable (3) -1.113 kN / 0.850 m **-1.309 kN/m**
q2 from support force (s C) from rafter load c. wind on gable (2) -1.988 kN / 0.850 m **-2.339 kN/m**
q3 from support force (s C) from rafter load c. wind on gable (1) -3.017 kN / 0.850 m **-3.549 kN/m**



10.2.6. snow (1)

System, loading + support reactions from vertical loads

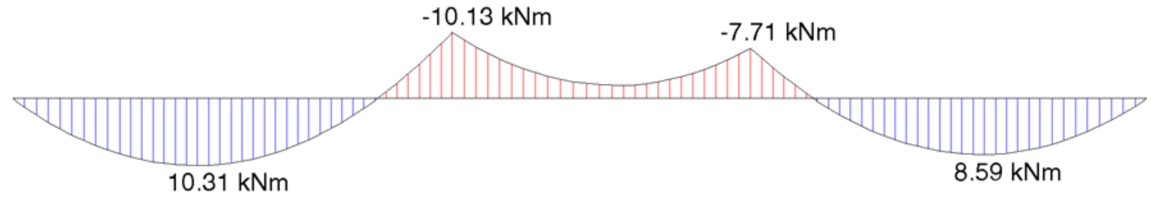
```
from support force (s C) from rafter l. c.  snow fully            1.111 kN / 0.850 m     1.307 kN/m
```



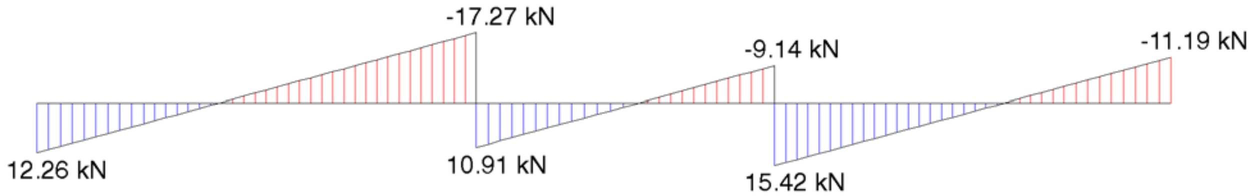
10.3. Extremal from action effects

10.3.1. permanent loads

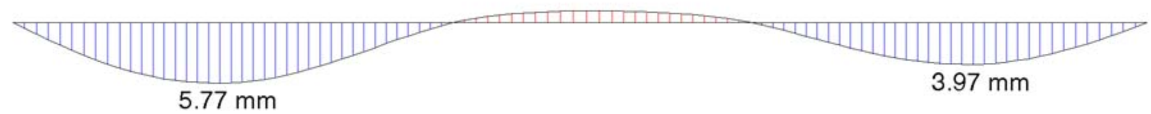
extremal flectural moments from vertical loads (permanent loads)



extremal shear forces from vertical loads (permanent loads)

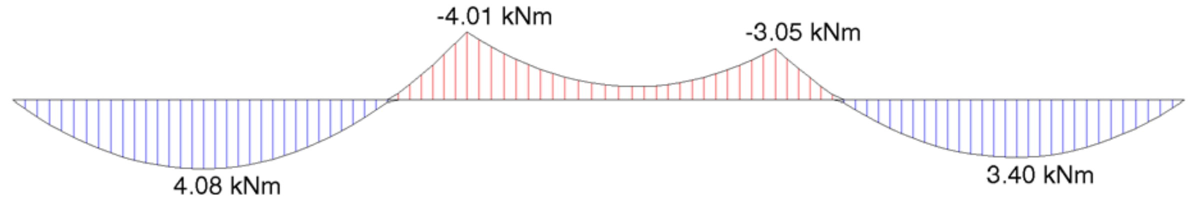


extremal deformations from vertical loads (permanent loads)

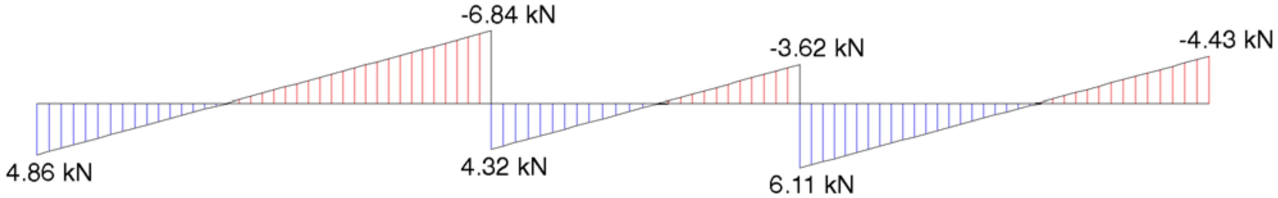


10.3.2. extremal live loads

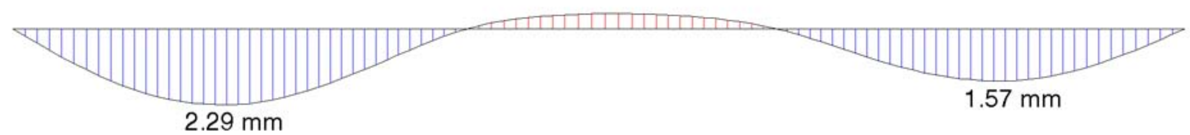
extremal flectural moments from vertical loads (live loads)



extremal shear forces from vertical loads (live loads)

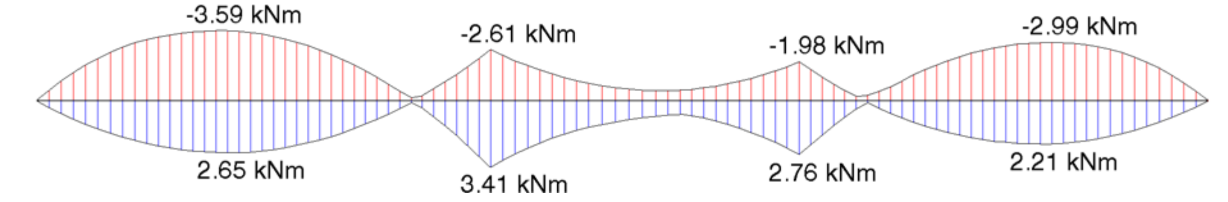


extremal deformations from vertical loads (live loads)

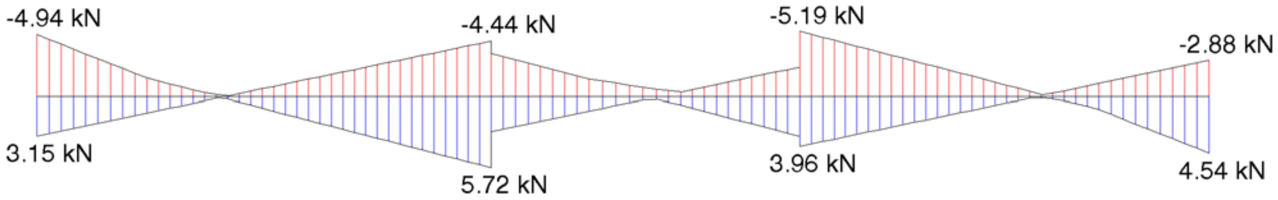


10.3.3. extremal wind loads

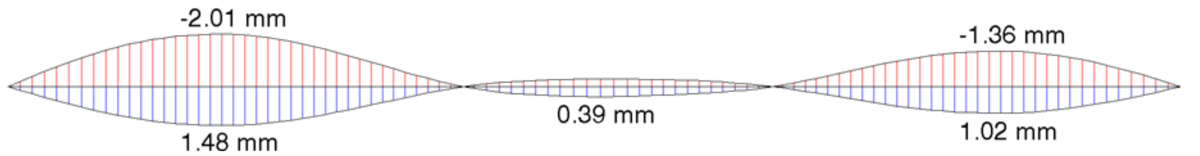
extremal flectural moments from vertical loads (wind)



extremal shear forces from vertical loads (wind)

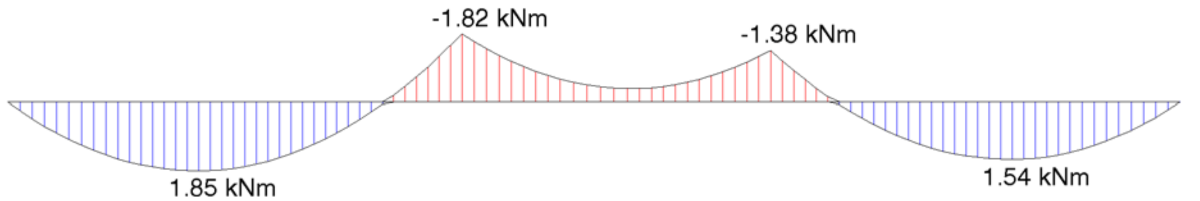


extremal deformations from vertical loads (wind)

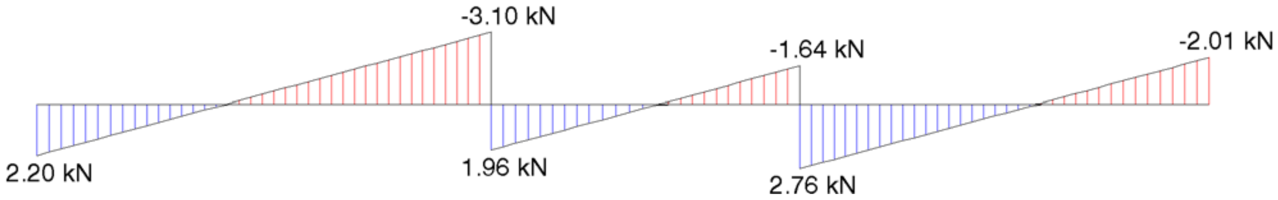


10.3.4. extremal snow loads

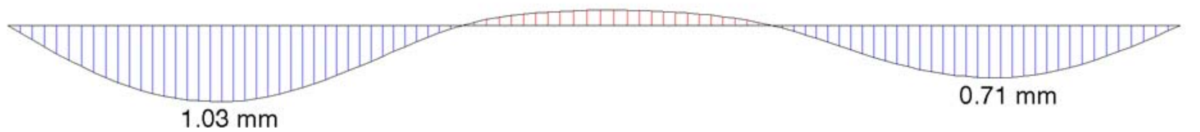
extremal flectural moments from vertical loads (snow)



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 6.1.2. 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 = 71% $\Rightarrow$ verification successful.

10.4.2. Utilisation rates in the exceptional design situation "Nordeutsche Tiefebene"
The determination of the extreme values for the exceptional 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 6.1.3. The utilisation rates are determined acc. to EC5 (6.1.6 bending and 6.1.7 thrust) with γ_M = 1.00. They result as shown below

max U = 40% $\Rightarrow$ verification successful.

10.4.3. Utilisation rates for fire protection verification
verification method, required fire resistance period, partial safety factors and combination coefficients see chapter 6.1.4. Fire exposure: four-sided. h_{red} = 17.8 cm, b_{red} = 13.8 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 = 60% $\Rightarrow$ verification successful.

10.4.4. Serviceability utilisation rates without creep influence (w_{inst})
Comparison lengths = field lengths. For limit values and combination coefficients see chapter 6.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below.

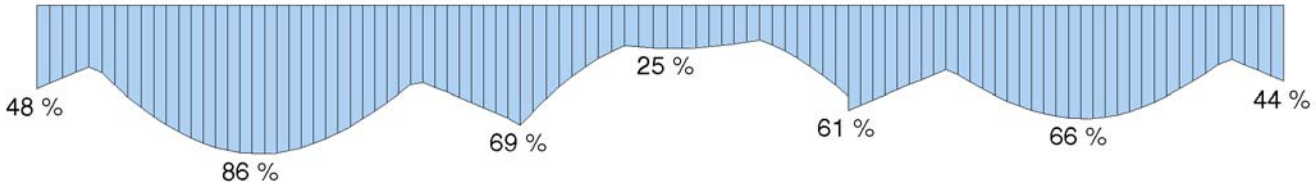
max U = 70% $\Rightarrow$ verification successful.

10.4.5. Serviceability utilisation rates with creep influence (w_{fin})
Comparison lengths = field lengths. For limit values and combination coefficients see chapter 6.2.3. The utilisation rates are determined for the characteristic combination. They result as shown below with k_{def} = 0.80

max U = 72% $\Rightarrow$ verification successful.

10.4.6. Serviceability utilisation rates with creep influence ($w_{fin,net}$)
Comparison lengths = field lengths. For limit values and combination coefficients see chapter 6.2.3. The utilisation rates are determined for the quasi-permanent combination. They result as shown below with k_{def} = 0.80

max U = 86% $\Rightarrow$ verification successful.

10.4.7. maximum utilization of all verifications


max U = 86% $\Rightarrow$ all verifications successful.

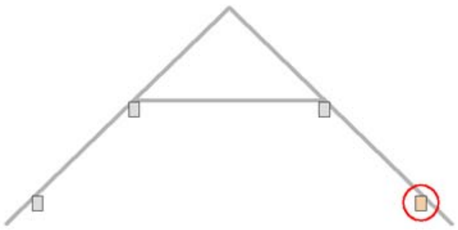
10.5. extremal support reactions

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:						
C1	12.263	---	4.857	3.153	2.199	22.472
C2	28.173	---	11.158	7.244	5.051	51.627
C3	24.564	---	9.728	6.316	4.404	45.012
C4	11.194	---	4.434	2.878	2.007	20.513
minimal vertical:						
C1	12.263	---	0.000	-4.941	0.000	7.322
C2	28.173	---	0.000	-9.113	0.000	19.060
C3	24.564	---	0.000	-8.384	0.000	16.180
C4	11.194	---	0.000	-4.543	0.000	6.652

11. purlin D

11.1. position, characteristic values, explanations



material: coniferous timber: C24

cross-section: b = 18.0 cm, h = 18.0 cm

siehe purlin A