REV.	PŘEDMĚT ZMĚN	DATUM	PODPIS

2296-CPR-739

VŠEOBECNÉ POKYNY NA

A-CZ1621-01

VYPRACOVAL	KONTROLOVAL	ZODP. PROJEKTANT
Ing. Kamil Patrman	Ing. Stanislav Tóth	Ing. Stanislav Tóth
-	-	-
-	-	-

LLENTAB

LLENTAB, spol. s r. o.
Přátelství 1509/13
104 00 Praha 10 - Uhříněves
IČ: 25079891
DIČ: CZ25079891
www.llentab.cz

MÍSTO STAVBY: LOUČKY U ZÁTORA

O. Ú.: KRNOV KRAJ: MORAVSKOSLEZSKÝ

AKCE:

SKLADOVACÍ HALA, ZÁTOR-LOUČKY

-

-

SMLUVNÍ GARANT	-	
DATUM	FORMÁT	MĚŘÍTKO
25.1.2022	.	.
ČÍS. ZAKÁZKY	OBJEKT	STUPEŇ P. D.
CZ1621	-	DPS

OBSAH: D.1.2 - ČÁST OCELOVÁ KONSTRUKCE

STATICKÝ VÝPOČET

ČÍSLO VÝKRESU:			REVIZE:
S	CZ1621	-	.

Obsah

Informace o projektu

Projekt:

Č. projektu:	CZ1621
Název projektu:	Hala Zator
Místo výstavby:	Loučky u Zátoru
Datum:	25.1.2022
Norma projektu:	ČSN EN

Kontakt na projektanta:

Vypracoval: Kamil Patrman
Tel: +420 720 934 868
Mail: kamil.patrman@llentab.cz

Kontakt na kontrolora:

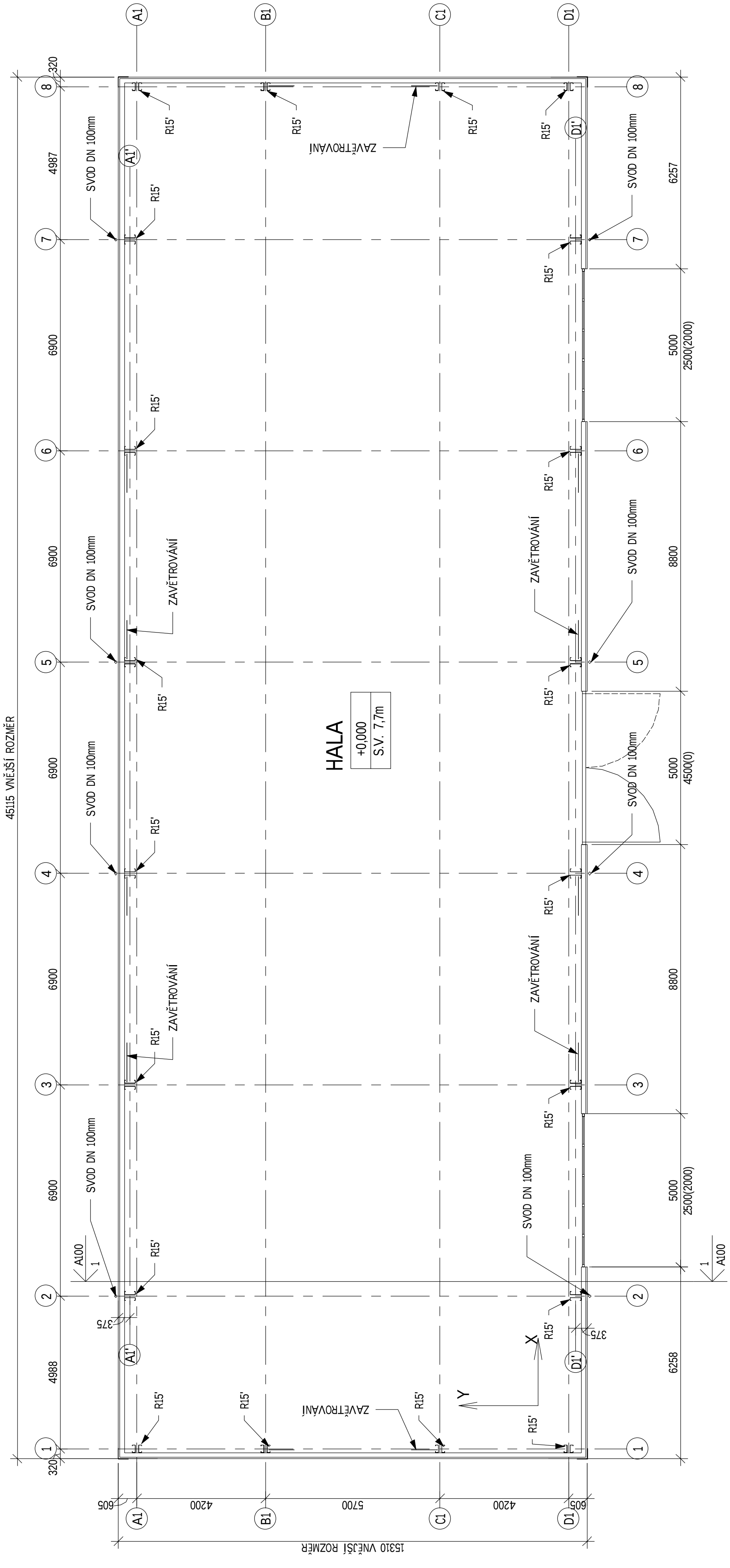
Kontroloval: Stanislav Tóth
Tel: +420 601 387 853
Mail: stanislav.toth@llentab.cz

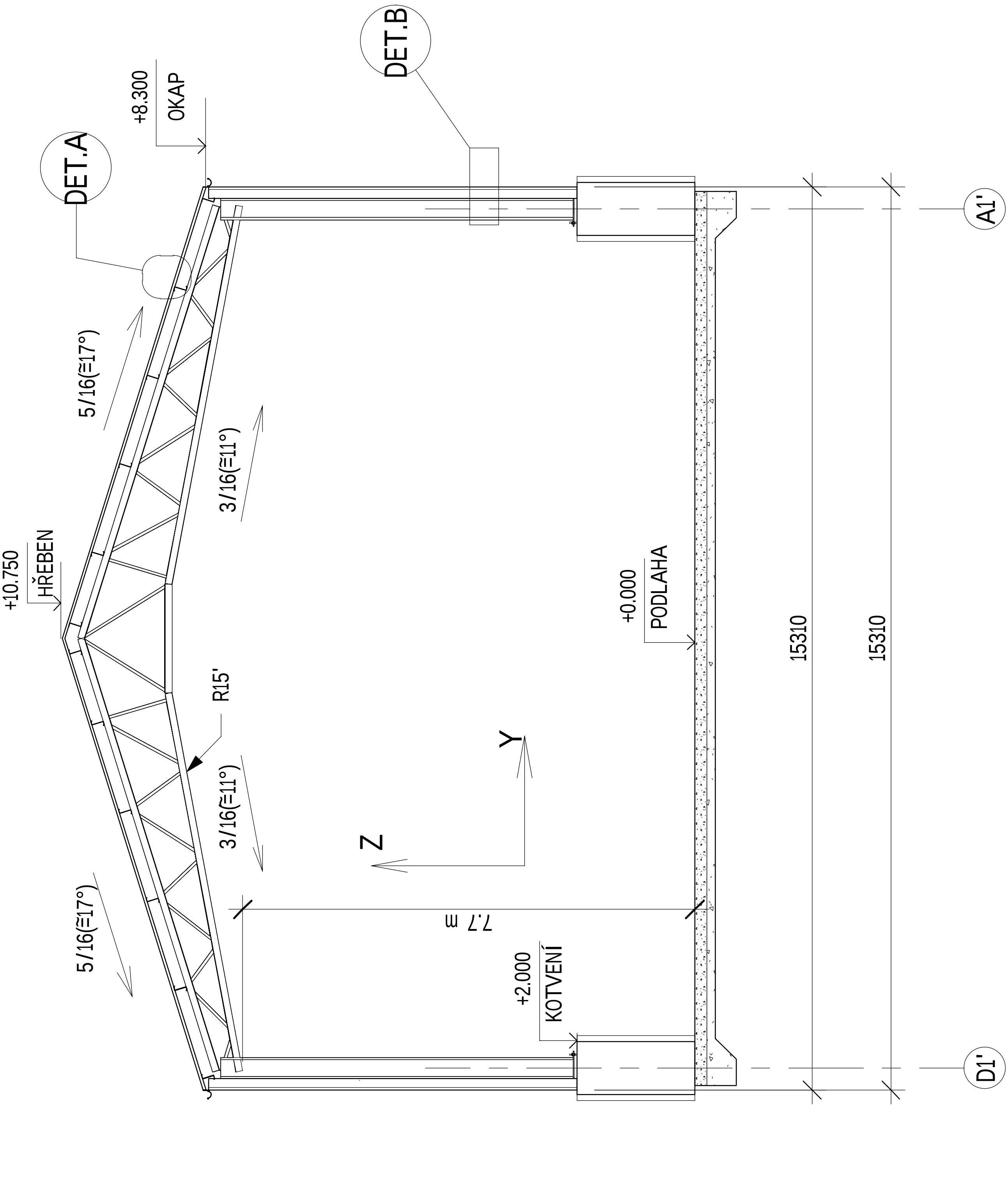
A.

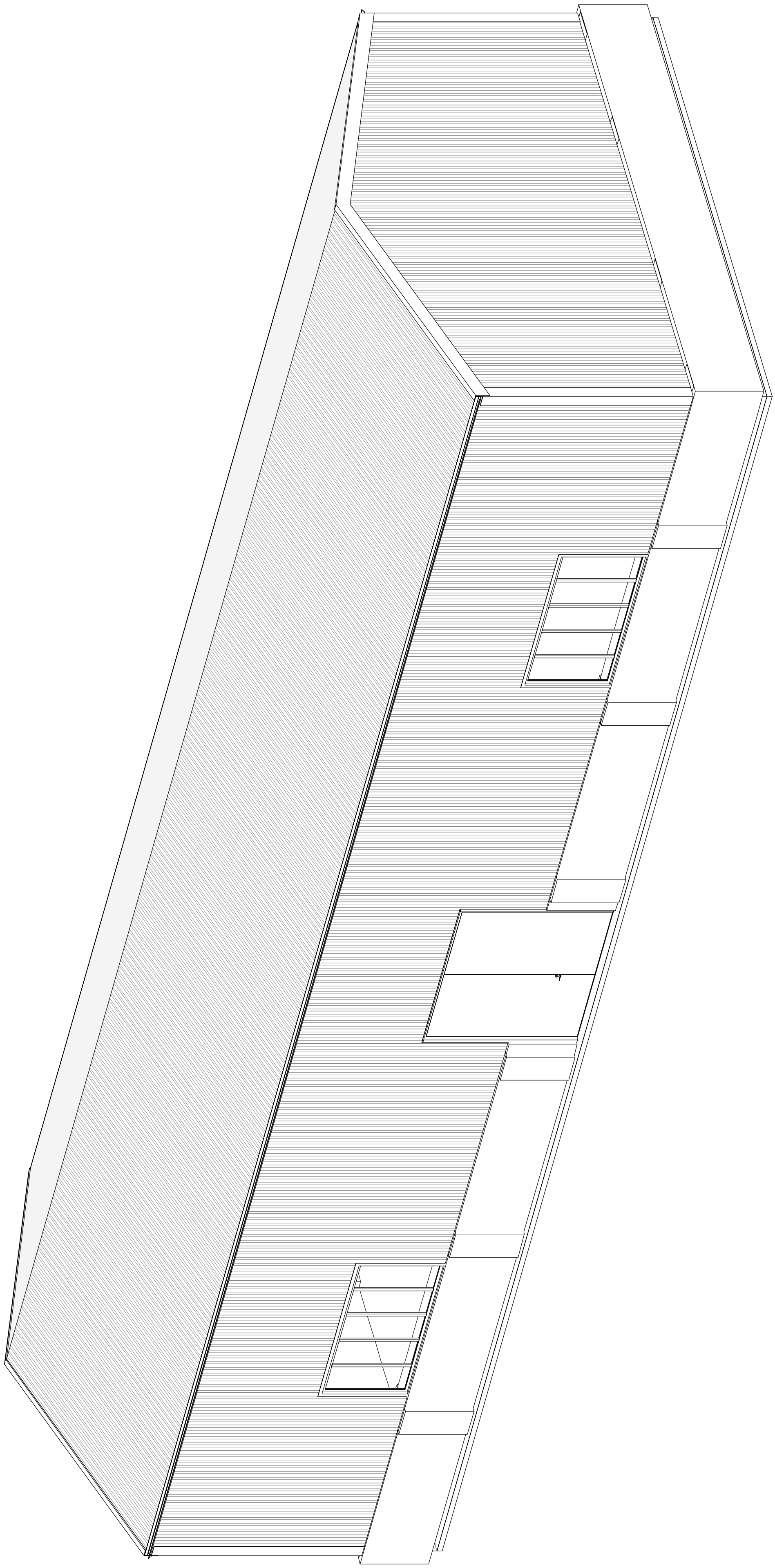
Geometrie

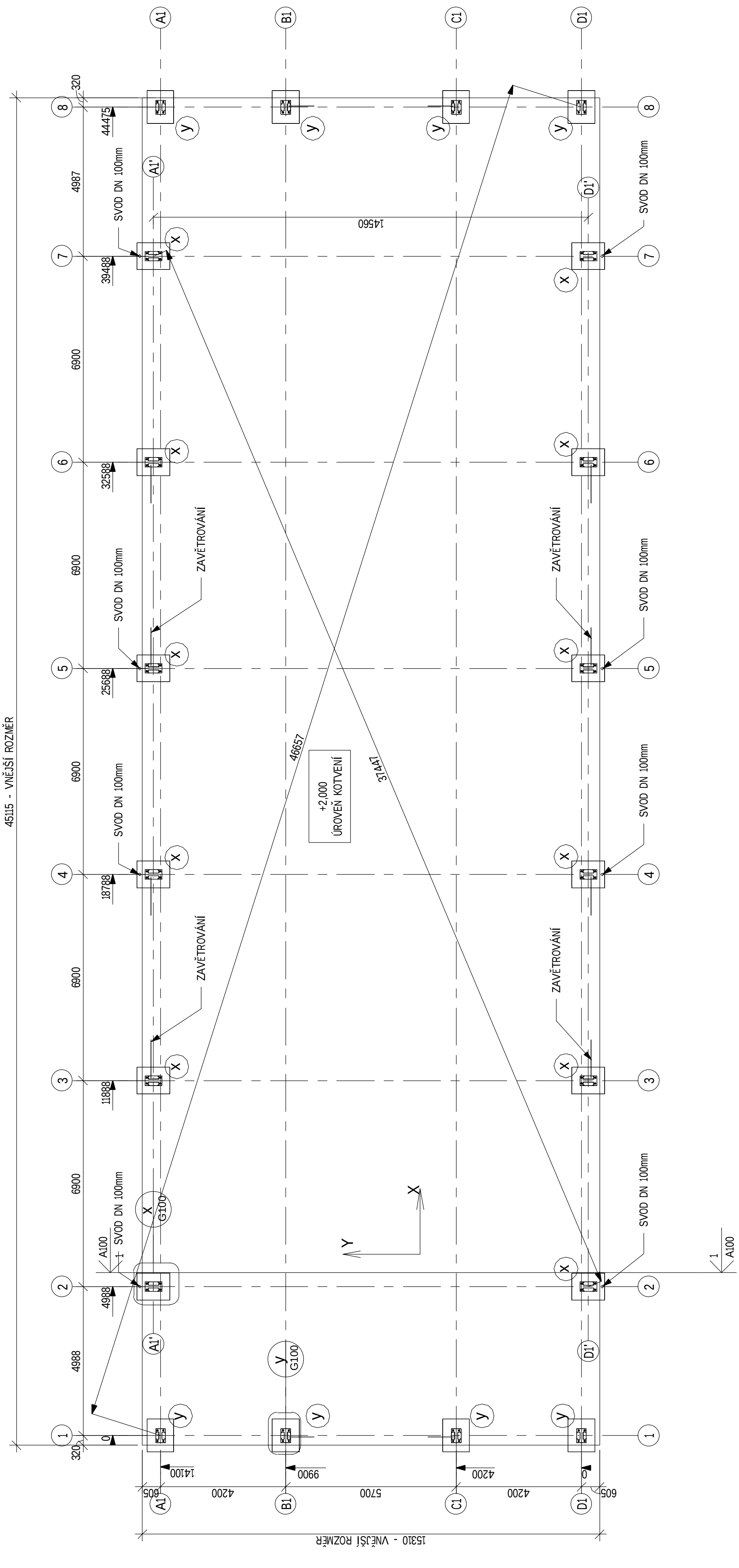
Geometrie:

Typ haly =	S17S14R
Základní modul =	6.900 m
Typ střechy =	Unisolated-Z150
Šířka haly =	15.310 m
Délka haly =	45.115 m
Výška okapu =	8.30 m
Výška hřebene =	10.69 m
Úhel horního pásu =	17.35 °
Úhel dolního pásu =	14.00 °


HALA
+0,000
S.V. 7,7m
45115 VNĚJŠÍ ROZMĚR
15310 VNĚJŠÍ ROZMĚR
SVOD DN 100mm
ZAVĚTROVÁNÍ
R15'
A1
B1
C1
D1
A1'
D1'
1
2
3
4
5
6
7
8
A100
320
4987
6900
4988
6257
5000
2500(2000)
8800
4500(0)
6258
375
605
4200
5700
X
Y


DET.A
DET.B
+8.300
OKAP
+10.750
HŘEBEN
5/16(≅17°)
3/16(≅11°)
R15'
3/16(≅11°)
5/16(≅17°)
Y
Z
+0.000
PODLAHA
15310
15310
7.7 m
+2.000
KOTVENÍ
A1'
D1'




45115 - VNĚJŠÍ ROZMĚR
15310 - VNĚJŠÍ ROZMĚR
SVOD DN 100mm
ZAVĚTROVÁNÍ
+2,000
ÚROVEŇ KOTVENÍ
46657
37447
14560
A1
B1
C1
D1
A1'
D1'
6900
4988
4987
4200
5700
605
320
44475
39488
32588
25688
18788
11888
4988
0
14100
9900
4200
0
G100
A100

Zatížení

Poznámky:
Zatížení větrem, sněhem, vlastní váhou, požárem R15.

B1. Zatížení (ČSN EN 1991)

Norma statického výpočtu:

Norma ČSN (CZ)/ STN (SK): **ČSN EN** **CZ**

Základní informace projektu:

Typ haly: **S** ☐ PULT ☐ HALA S ATIKOU ☐ VÍCE HŘEBENŮ

Typ Izolace : **Unisolated-Z150**

Šířka haly:	**15.310**	m	Horní pás:	**17.35**	°	Výška u žlabu:	**8.30**	m
Délka haly:	**45.115**	m	Dolní pás:	**10.62**	°	Výška hřebene:	10.69	m

Stálé zatížení: ČSN EN 1991 ()

Gravitační tíhu zadávat se znaménkem "-"

Tíha střechy R =	**-0.13**	kN/m^2	
Extra přitížení E =	**-0.10**	kN/m^2	
Tíha podhledu C =	**0.00**	kN/m^2	☐ Extra zatížení zahrnout do podhledu
Přitížení od kapotáže vazníku F =	**0.00**	kN/m	☐ Počítat s přitížením od kapotáže vazníku

Zatížení sněhem ČSN EN 1991-1-3 ()

	Charakteristická hodnota s_k =	**-1.50**	kN/m^2	Tvarový součinitel:
	Tvarový součinitel μ_i =	0.8		Platí pro sklony 0° ≤ α ≤ 30°
l_c= 25.42 m	Součinitel expozice c_e =	1.00		μ_1 = 0.80
	Tepelný součinitel c_t =	1.0		μ_2 = 1.26
	$s = s_k \cdot \mu_i \cdot c_e \cdot c_t$ =	**-1.20**	kN/m^2	Horní hodnota m_2 = 1,6

Zatížení větrem ČSN EN 1991-1-4 ()

Charakteristická hodnota rychlosti větru v_{bo} = **25** m/s

Kategorie terénu: II. Oblasti s nízkou vegetací jako je tráva nebo izolované překážky (stromy, budovy)

Základní rychlost větru v_b =	25	m/s	Parametr drsnosti terénu z_0 =	0.05	m
Součinitel terénu k_r =	0.190		Minimální výška $z_{min.}$ =	2	m
Součinitel drsnosti terénu $c_r(z)$ =	1.019		Výška nad zemí z =	10.69	m
Střední rychlost větru $v_m(z)$ =	25.5	m/s	Součinitel směru =	1.0	
Základní dynamický tlak větru $q_b(z)$ =	405.9	N/m^2	Součinitel období =	1.0	
			Součinitel orografie $c_o(z)$ =	1.0	
Součinitel expozice $c_e(z)$ =	2.305		Součinitel turbulence k_I =	1.0	

Maximální dynamický tlak $q_p(z)$ = **0.936** kN/m^2

B2. Zatížení větrem (ČSN EN 1991-1-4)

Šířka haly:	15.31	m	Horní pás:	17.35	°	Výška u žlabu:	8.30	m
Délka haly:	45.12	m	Dolní pás:	10.62	°	Výška hřebene:	10.69	m
				17				

Zatížení větrem:

ČSN EN 1991-1-4 ()

Typ střechy: Sedlová 1

Příčný vítr

Pro stěny platí e = min(b,2h), kde b je rozměr kolmý na směr větru, d je rozměr ve směru větru

b = 45.12 m d = 15.31 m e = 21.38 m

Stěny	Oblast	A	B	C	D	E	
	Délka [m]	4.3	11.0	0.0	45.1	45.1	délka oblasti působení
	Cpe	-1.20	-0.80	-0.50	0.76	-0.42	tlak/sání na stěnu

Střecha	Oblast	F	G	H	I	J	
	Délka [m]	2.1	2.1	5.5	5.5	2.1	délka oblasti působení
	Cpe sání	-0.85	-0.76	-0.29	-0.40	-0.93	sání na střechu
	Cpe tlak	0.27	0.27	0.23	0.00	0.00	tlak na střechu
	-	-	-	-	-	-	-

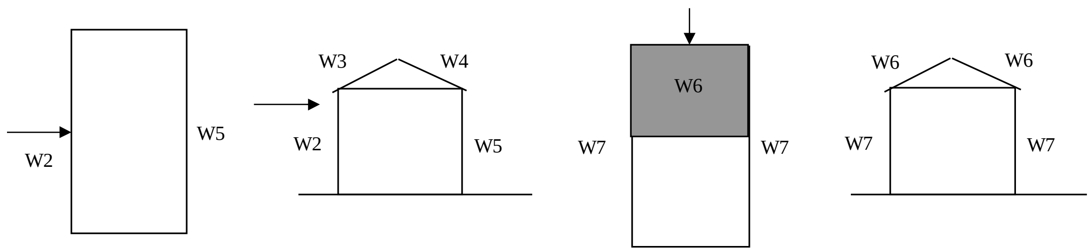
Podélný vítr

Pro stěny platí e = min(b,2h), kde b je rozměr kolmý na směr větru, d je rozměr ve směru větru

b = 15.31 m d = 45.12 m e = 15.31 m

Stěny	Oblast	A	B	C	D	E	
	Délka [m]	3.1	12.2	29.8	15.3	15.3	délka oblasti působení
	Cpe	-1.20	-0.80	-0.50	0.70	-0.30	tlak/sání na stěnu

Střecha	Oblast	F	-	G	H	I	
	Délka [m]	1.5	-	1.5	6.1	37.5	délka oblasti působení
	Cpe sání	-1.27	-	-1.31	-0.63	-0.50	sání na střechu
	-	-	-	-	-	-	-



$w_e = q_p(z)*c_{pe}$

W2	c_{pe} =	0.76	w_e =	0.71	kN/m^2	Příčný vítr	Stěna
W3	c_{pe} =	0.29	w_e =	0.27	kN/m^2	Příčný vítr	Střecha sání
W3a	c_{pe} =	-0.23	w_e =	-0.21	kN/m^2	Příčný vítr	Střecha tlak
W4	c_{pe} =	0.40	w_e =	0.37	kN/m^2	Příčný vítr	Střecha sání
W4a	c_{pe} =	0.00	w_e =	0.00	kN/m^2	Příčný vítr	Střecha tlak
W5	c_{pe} =	0.42	w_e =	0.39	kN/m^2	Příčný vítr	Stěna závětrná
W6	c_{pe} =	0.63	w_e =	0.59	kN/m^2	Podélný vítr	Střecha
W7	c_{pe} =	0.80	w_e =	0.75	kN/m^2	Podélný vítr	Stěna
W8	c_{pe} =	1.30	w_e =	1.22	kN/m^3	Atika	
W9	c_{pe} =		w_e =	0.00	kN/m^4	Mezistřešní pole	

B3. Sedlová střecha - Součinitelé vnějšího tlaku c_{pe}

Sedlová střecha — ČSN EN 1991-1-4 (7.2.5)

Výpočet zatížení větrem v závislosti na dané geometrii haly (šířka, výška haly, sklon střechy)

Závislost dynamického tlaku na výšce stavby: **OK je splněno h<b** — ČSN EN 1991-1-4 (7.2.2) Obrázek 7.4

Příčný vítr

Schéma oblastí ve štítové stěně: **e > d --> Oblast A, B** — ČSN EN 1991-1-4 (7.2.2) Obrázek 7.5

Platí e = min(b,2h), kde b je rozměr kolmý na směr větru, d je rozměr ve směru větru

b = 45.1 m	e/10 = 2.1 m	e/4 = 5.3 m	Poměr h/b = 0.70	0.70
d = 15.3 m	e/5 = 4.3 m	e/2 = 10.7 m	e = 21.4 m	

	Oblast	A	B	C	D	E	
Stěny	Délka [m]	4.3	11.0	0.0	45.1	45.1	délka oblasti působení
	C_{pe}	-1.20	-0.80	-0.50	0.76	-0.42	tlak/sání na stěnu

ČSN EN 1991-1-4 (7.2.5) Tabulka 7.4a 7.4b

	Oblast	F	G	H	I	J	
Střecha	Délka [m]	2.1	2.1	5.5	5.5	2.1	délka oblasti působení
	C_{pe} sání	-0.85	-0.76	-0.29	-0.40	-0.93	sání na střechu
	C_{pe} tlak	0.27	0.27	0.23	0.00	0.00	tlak na střechu
	-	-	-	-	-	-	-

Podélný vítr

Schéma oblastí v podélné stěně: **e < d --> Oblast A, B, C** — ČSN EN 1991-1-4 (7.2.2) Obrázek 7.5

Platí e = min(b,2h), kde b je rozměr kolmý na směr větru, d je rozměr ve směru větru

b = 15.3 m	e/10 = 1.5 m	e/4 = 3.8 m	Poměr h/b = 0.24	0.25
d = 45.1 m	e/5 = 3.1 m	e/2 = 7.7 m	e = 15.3 m	

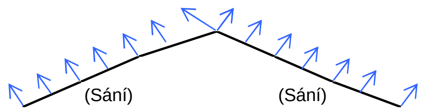
	Oblast	A	B	C	D	E	
Stěny	Délka [m]	3.1	12.2	29.8	15.3	15.3	délka oblasti působení
	C_{pe}	-1.20	-0.80	-0.50	0.70	-0.30	tlak/sání na stěnu

ČSN EN 1991-1-4 (7.2.5) Tabulka 7.4a 7.4b

	Oblast	F	-	G	H	I	
Střecha	Délka [m]	1.5	-	1.5	6.1	37.5	délka oblasti působení
	C_{pe} sání	-1.27	-	-1.31	-0.63	-0.50	sání na střechu
	-	-	-	-	-	-	-

Příčný vítr

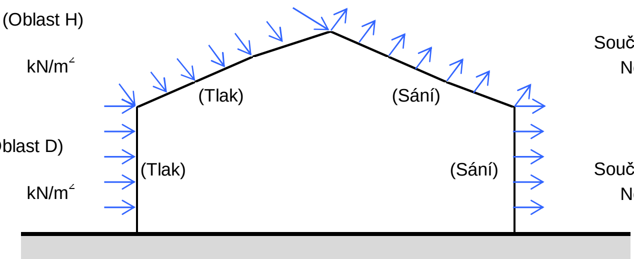
Střecha - Návětrná strana (Oblast H)
Součinitel vněj. tlaku c_{pe} = **-0.29**
Normové zatížení w_e = **-0.27** kN/m²



Střecha - Závětrná strana (Oblast I)
Součinitel vněj. tlaku c_{pe} = **-0.40**
Normové zatížení w_e = **-0.37** kN/m²

Střecha - Návětrná strana (Oblast H)
Součinitel vněj. tlaku c_{pe} = **0.23**
Normové zatížení w_e = **0.21** kN/m²

Stěna - Návětrná strana (Oblast D)
Součinitel vněj. tlaku c_{pe} = **0.76**
Normové zatížení w_e = **0.71** kN/m²



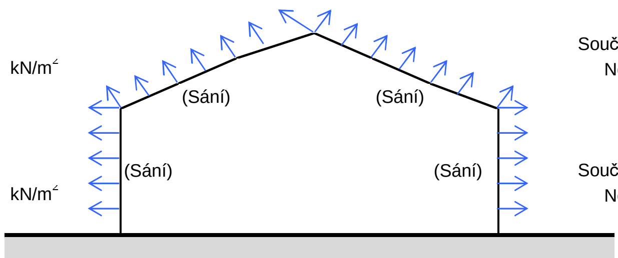
Střecha - Závětrná strana (Oblast I)
Součinitel vněj. tlaku c_{pe} = **0.00**
Normové zatížení w_e = **0.00** kN/m²

Stěna - Závětrná strana (Oblast E)
Součinitel vněj. tlaku c_{pe} = **-0.42**
Normové zatížení w_e = **-0.39** kN/m²

Podélný vítr

Střecha (Oblast H)
Součinitel vněj. tlaku c_{pe} = **-0.63**
Normové zatížení w_e = **-0.59** kN/m²

Stěna (Oblast B)
Součinitel vněj. tlaku c_{pe} = **-0.80**
Normové zatížení w_e = **-0.75** kN/m²



Střecha (Oblast H)
Součinitel vněj. tlaku c_{pe} = **-0.63**
Normové zatížení w_e = **-0.59** kN/m²

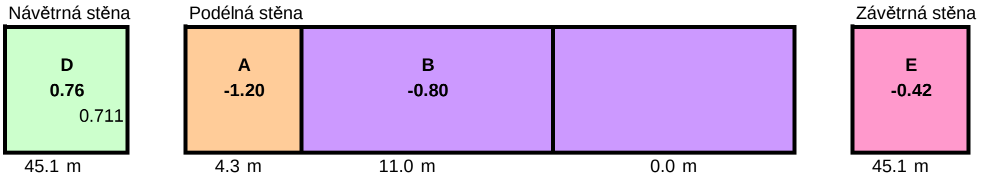
Stěna (Oblast B)
Součinitel vněj. tlaku c_{pe} = **-0.80**
Normové zatížení w_e = **-0.75** kN/m²

B3. Sedlová střecha - Schéma zatížení c_{pe}

Stěny

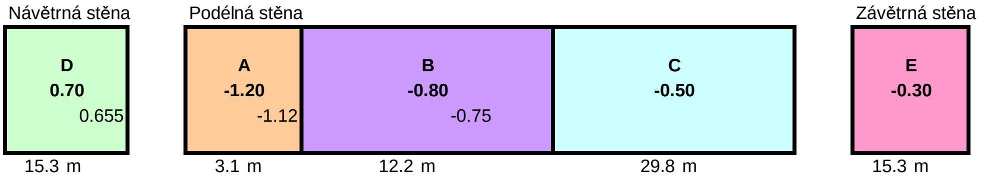
Příčný vítr

ČSN EN 1991-1-4 (7.2.2)
Obrázek 7.5



Podélný vítr

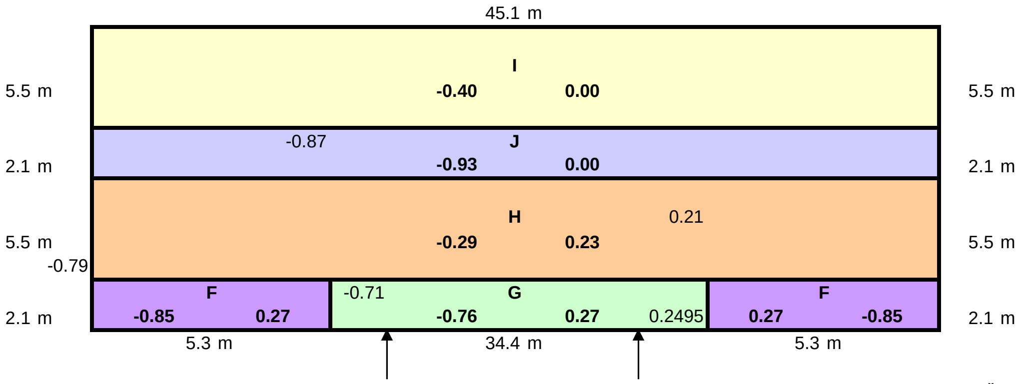
ČSN EN 1991-1-4 (7.2.2)
Obrázek 7.5



Střecha

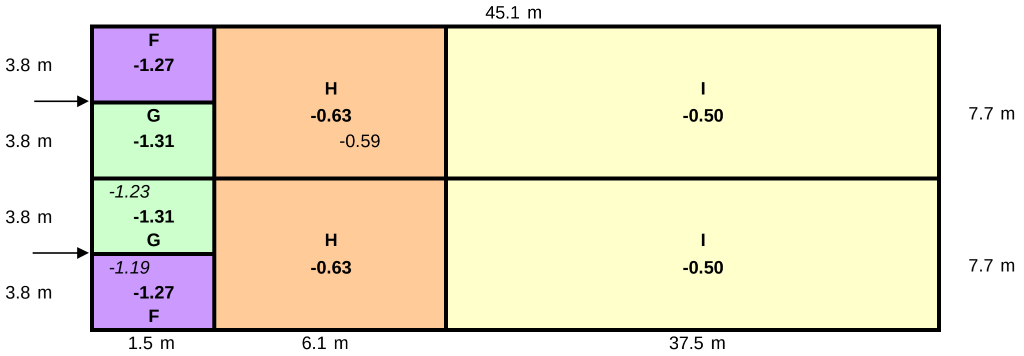
Příčný vítr

ČSN EN 1991-1-4 (7.2.5)
Obrázek 7.8

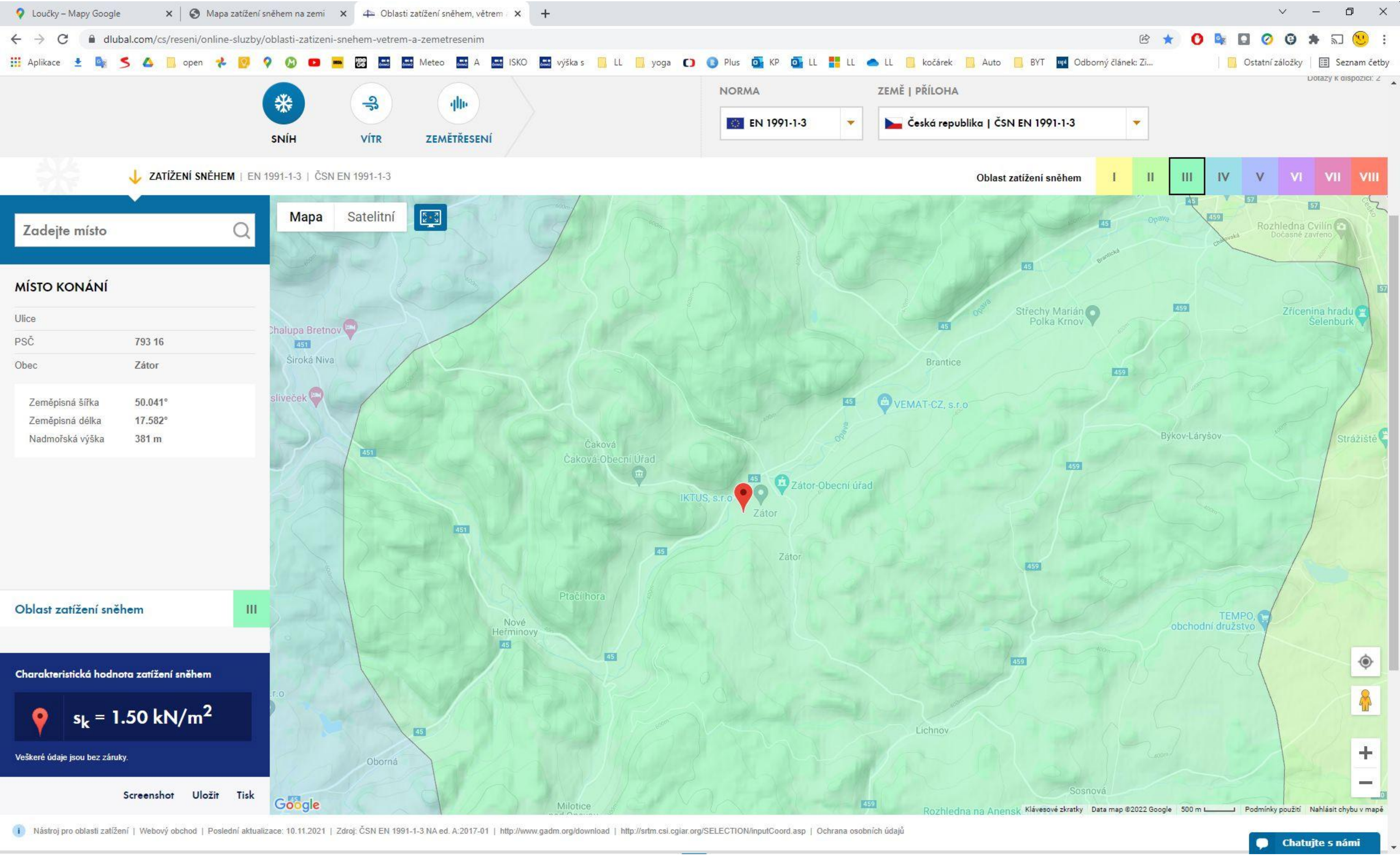


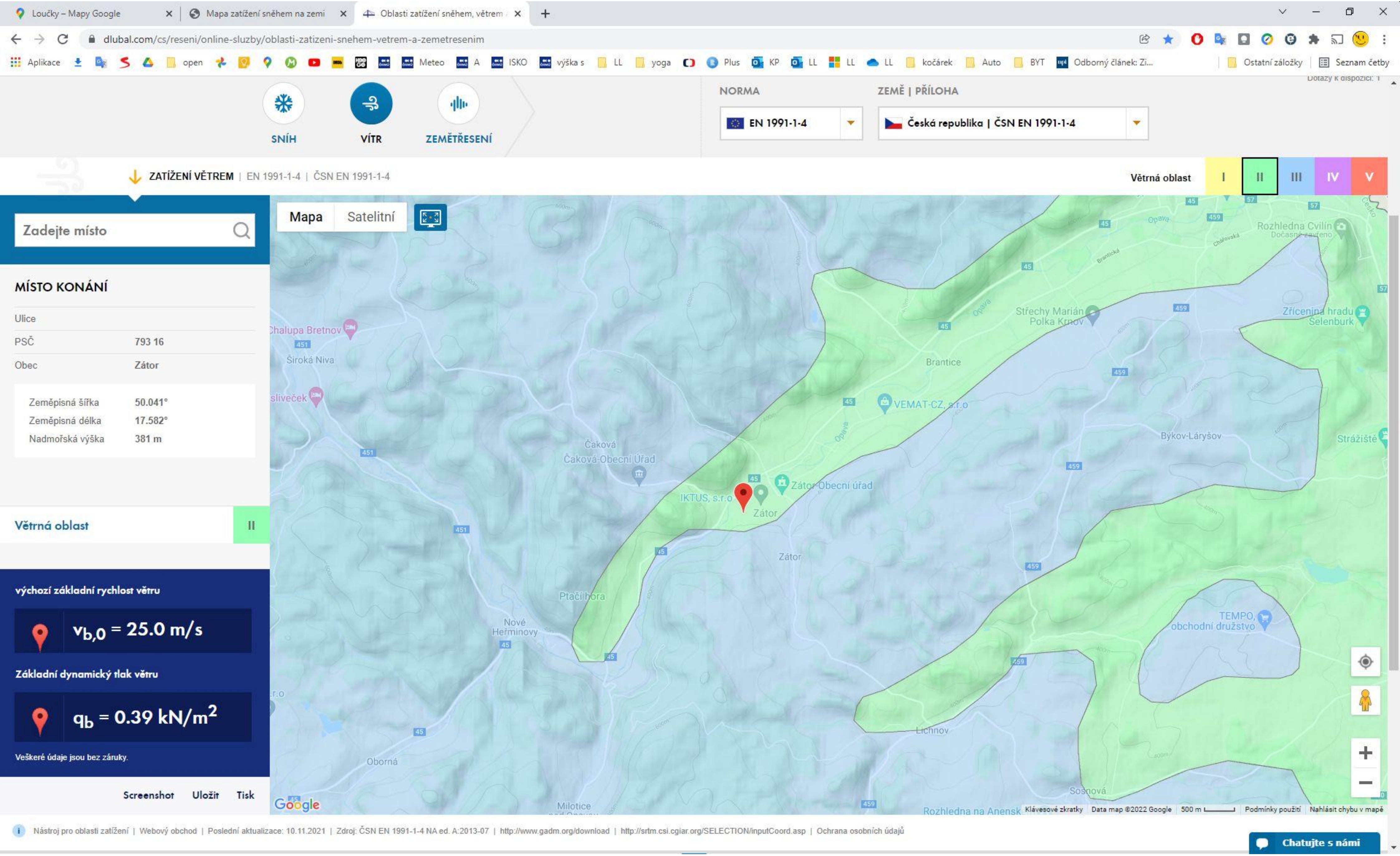
Podélný vítr

ČSN EN 1991-1-4 (7.2.5)
Obrázek 7.8

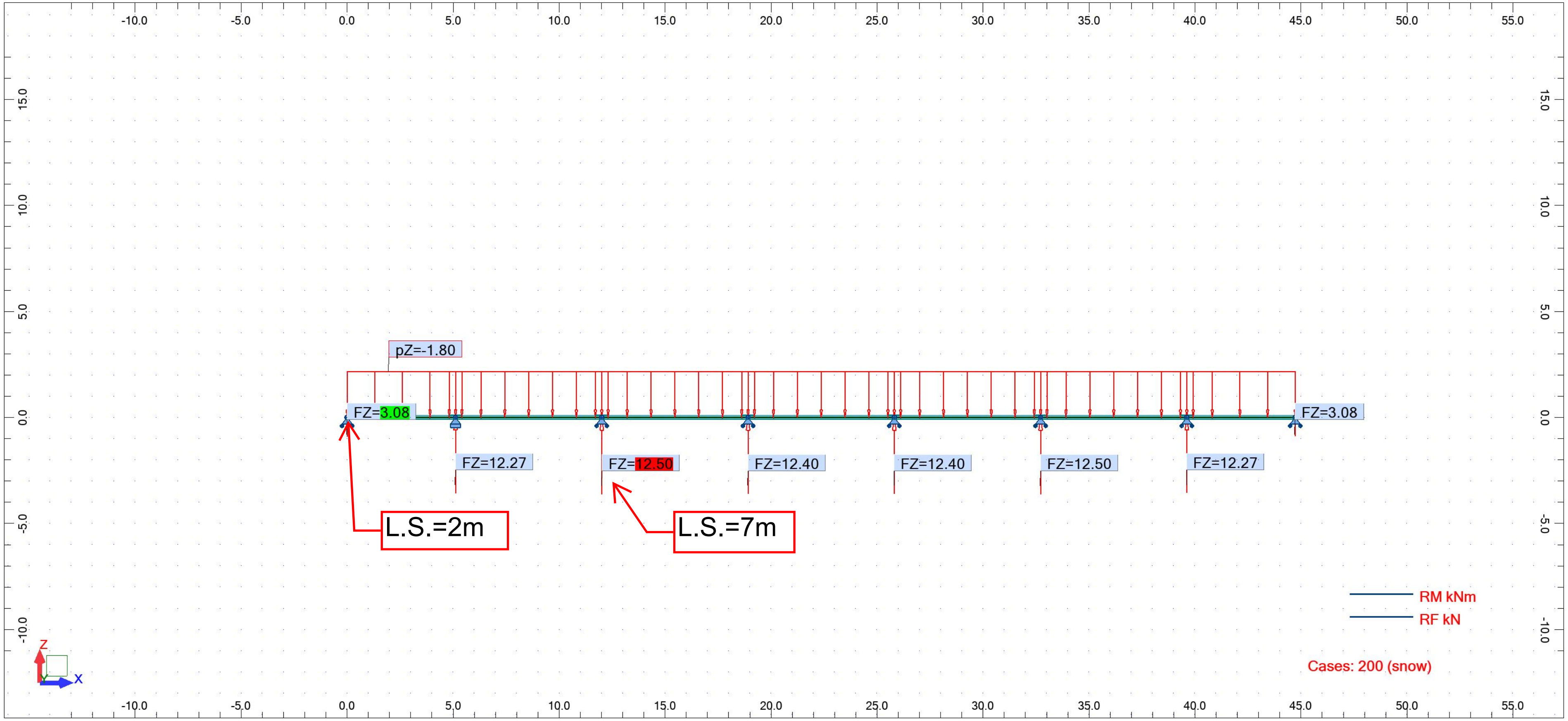


0.936


Loučky – Mapy Google
Mapa zatížení sněhem na zemi
Oblasti zatížení sněhem, větrem
dlubal.com/cs/reseni/online-sluzby/oblasti-zatizeni-snehem-vetrem-a-zemetresenim
SNÍH
VÍTR
ZEMĚTŘESENÍ
NORMA
EN 1991-1-3
ZEMĚ | PŘÍLOHA
Česká republika | ČSN EN 1991-1-3
ZATÍŽENÍ SNĚHEM | EN 1991-1-3 | ČSN EN 1991-1-3
Oblast zatížení sněhem
I II III IV V VI VII VIII
Zadejte místo
MÍSTO KONÁNÍ
Ulice
PSČ 793 16
Obec Zátor
Zeměpisná šířka 50.041°
Zeměpisná délka 17.582°
Nadmořská výška 381 m
Oblast zatížení sněhem III
Charakteristická hodnota zatížení sněhem
s_k = 1.50 kN/m^2
Veškeré údaje jsou bez záruky.
Screenshot
Uložit
Tisk
Mapa
Satelitní
Zátor
Brantice
Nové Heřminovy
Lichnov
Nástroj pro oblasti zatížení | Webový obchod | Poslední aktualizace: 10.11.2021 | Zdroj: ČSN EN 1991-1-3 NA ed. A:2017-01 | http://www.gadm.org/download | http://srtm.csi.cgiar.org/SELECTION/inputCoord.asp | Ochrana osobních údajů
Chatujte s námi


dlubal.com/cs/reseni/online-sluzby/oblasti-zatizeni-snehem-vetrem-a-zemetresenim
SNÍH
VÍTR
ZEMĚTŘESENÍ
NORMA
EN 1991-1-4
ZEMĚ | PŘÍLOHA
Česká republika | ČSN EN 1991-1-4
ZATÍŽENÍ VĚTREM | EN 1991-1-4 | ČSN EN 1991-1-4
Větrná oblast
I
II
III
IV
V
Zadejte místo
MÍSTO KONÁNÍ
Ulice
PSČ 793 16
Obec Zátor
Zeměpisná šířka 50.041°
Zeměpisná délka 17.582°
Nadmořská výška 381 m
Větrná oblast II
výchozí základní rychlost větru
$v_{b,0}$ = 25.0 m/s
Základní dynamický tlak větru
q_b = 0.39 kN/m^2
Veškeré údaje jsou bez záruky.
Screenshot
Uložit
Tisk
Mapa
Satelitní
Zátor
Brantice
Čaková
Nové Heřminovy
Lichnov
Bykov-Lárýšov
Široká Niva
Oborná
Nástroj pro oblasti zatížení | Webový obchod | Poslední aktualizace: 10.11.2021 | Zdroj: ČSN EN 1991-1-4 NA ed. A:2013-07 | http://www.gadm.org/download | http://srtm.csi.cgiar.org/SELECTION/inputCoord.asp | Ochrana osobních údajů
Chatujte s námi

View - Reaction forces(kN);Reaction moments(kN*m); Cases: 200 (snow)



Normy

Poznámky:
Seznam použitých norem.

ČSN EN 1990 Zásady navrhování konstrukcí
ČSN EN 1991-1-1 Zatížení konstrukcí - Část 1-1: Obecná zatížení - Objemové tíhy, vlastní tíha a užitná zatížení pozemních staveb
ČSN EN 1991-1-2 Zatížení konstrukcí - Část 1-2: Obecná zatížení - Zatížení konstrukcí vystavených účinkům požáru
ČSN EN 1991-1-3 Zatížení konstrukcí - Část 1-3: Obecná zatížení - Zatížení sněhem
ČSN EN 1991-1-4 Zatížení konstrukcí - Část 1-4: Obecná zatížení - Zatížení větrem
ČSN EN 1991-1-5 Zatížení konstrukcí - Část 1-5: Obecná zatížení - Zatížení teplotou
ČSN EN 1993-1-1 Navrhování ocelových konstrukcí - Část 1-1: Obecná pravidla a pravidla pro pozemní stavby
ČSN EN 1993-1-2 Navrhování ocelových konstrukcí - Část 1-2: Obecná pravidla - Navrhování konstrukcí na účinky požáru
ČSN EN 1993-1-3 Navrhování ocelových konstrukcí - Část 1-3: Obecná pravidla - Doplňující pravidla pro tenkostěnné za studena tvarované prvky a plošné profily
ČSN EN 1993-1-8 Navrhování ocelových konstrukcí - Část 1-8: Navrhování styčníků
ČSN EN 1993-1-9 Navrhování ocelových konstrukcí - Část 1-9: Únava
ČSN EN 1993-1-12 Navrhování ocelových konstrukcí - Část 1-12: Doplňující pravidla pro oceli vysoké pevnosti do třídy S 700
ČSN EN 1090-1 Provádění ocelových konstrukcí a hliníkových konstrukcí - Část 1: Požadavky na posouzení shody konstrukčních dílců
ČSN EN 1090-1+A1 Provádění ocelových konstrukcí a hliníkových konstrukcí - Část 1: Požadavky na posouzení shody konstrukčních dílců
ČSN EN 1090-2+A1 Provádění ocelových konstrukcí a hliníkových konstrukcí - Část 2: Technické požadavky na ocelové konstrukce
ČSN EN 24015 Šrouby se šestihrannou hlavou s redukovanou hladkou částí dříku

C.

Část konstrukce: CL1 Plechy

Poznámky:
Typ použitého plechu pro střechu. TP46/0.63

Posouzení únosnosti plechů

			Plech		***TP46/0.63***	
			Mom. únosnost v poli	$M_{Rd}=$	1.22	kNm
Rozpětí	1500	mm	Mom. únosnost v poli	$M_{Rd.p}=$	1.46	kNm
Návrhové zatížení spojité	2.9	kN/m^2	Borcení kraj	$R_{Rd.k}=$	6.55	kN
Návrhové zatížení bodové	1.5	kN	Borcení střed	$R_{Rd.s}=$	10.60	kN
$\gamma_{M0}=$	1		Smyková únosnost	$V_{Rd}=$	21.53	kN
			Efekt. Moment setrvačnosti	$I_{eff}=$	131300.00	mm^4

	Vnitřní síly											
	Okrajová reakce R_k [kN]	Středová reakce R_s [kN]	Maximální smyková síla V_s [kN]	Maximální podporový moment M_p [kNm]	Maximální momentv poli M [kNm]	Deformace f [mm]	$M/M_{Rd}<1$	$R_k/R_{Rd.k}<1$	$R_s/R_{Rd.s}<1$	$M_p/M_{Rd.p}<1$	$M_p/M_{Rd.p}+R_s/R_{Rd.s}<1.25$	$(M_p/M_{Rd.p})^2+(V_s/V_{Rd})^2<1.0$
Prostý nosník	2.18		4.35		0.82	6.93	0.669	0.332	0.000	0.000	0.000	0.000
Spojitý n. 2 pole	1.63	5.44	2.72	0.82	0.46	2.88	0.376	0.249	0.513	0.559	1.072	0.376
Spojitý n. 3 pole	1.74	4.79	2.61	0.65	0.52		0.428	0.266	0.451	0.447	0.898	0.249
Spojitý n. 4 pole	1.71	4.97	2.64	0.70	0.50		0.413	0.261	0.469	0.479	0.948	0.282

1/200 $f_{lim}=$ 7.5 mm

Bodová síla uprostřed rozpětí
v krajním poli

	Vnitřní síly											
	Okrajová reakce R_k [kN]	Středová reakce R_s [kN]	Maximální smyková síla V_s [kN]	Maximální podporový moment M_p [kNm]	Maximální momentv poli M [kNm]	Deformace f [mm]	$M/M_{Rd}<1$	$R_k/R_{Rd.k}<1$	$R_s/R_{Rd.s}<1$	$M_p/M_{Rd.p}<1$	$M_p/M_{Rd.p}+R_s/R_{Rd.s}<1.25$	$(M_p/M_{Rd.p})^2+(V_s/V_{Rd})^2<1.0$
Prostý nosník	0.75		0.75		0.56	3.83	0.461	0.115	0.000	0.000	0.000	0.000
Spojitý n. 2 pole	0.61	1.03	0.89	0.21	0.46		0.375	0.093	0.097	0.144	0.242	0.023
Spojitý n. 3 pole	0.60	1.09	0.90	0.23	0.45		0.369	0.092	0.103	0.154	0.257	0.026
Spojitý n. 4 pole	0.60	1.09	0.90	0.23	0.45		0.368	0.091	0.103	0.155	0.258	0.027

1/200 $f_{lim}=$ 7.5 mm

D.

Část konstrukce: PU1 Vaznice standard

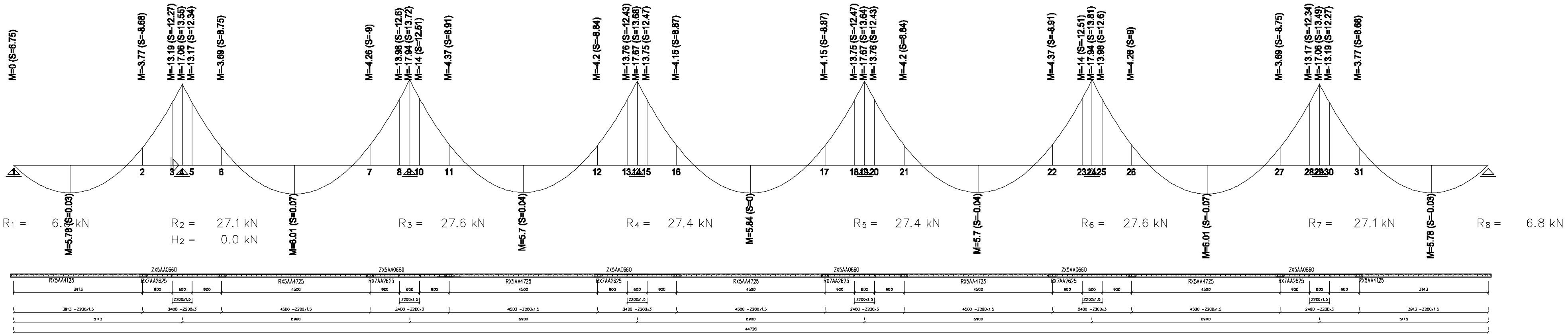
Poznámky:
Zatěžovací šířka vaznice Lw=1.5m. Standardní vaznice je navržena na kombinaci sněhu a tlaku od příčného větru a na maximální sání vyvozené podélným větrem.

PU1 Vaznice standard

Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|D - Vaznice|CZ1621 PU1.purx1/27/2022 7:36 by kamilpat
Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|D - Vaznice|CZ1621 PU1.rtd (1/27/2022 time.7:36:21 AM)
Max V = -17.4mm, bar nr 6, comb.PurlinComb3(7)
av.weight = 5.89 [kg/m]

(107)=-0.423 x0.900
(301)=-0.315 x0.900
(200)=-1.800 x1.500
(104)=-0.150 x1.350
(102)=-0.240 x1.350

cc=1500mm



Loads applied to the purlin

Distributed loads

name	(no.)	f1 (↓)	f2 (↑)	val.[kPa]
roof	(102)	1.35	1.00	-0.160
extra	(104)	1.35	0	-0.100
snow	(200)	1.50	0	-1.200
windSuc	(300)	0	1.50	0.870
windPre	(301)	0.90	0	-0.210
windIntS	(107)	0.90	0	-0.282
windIntP	(108)	0	1.50	0.188

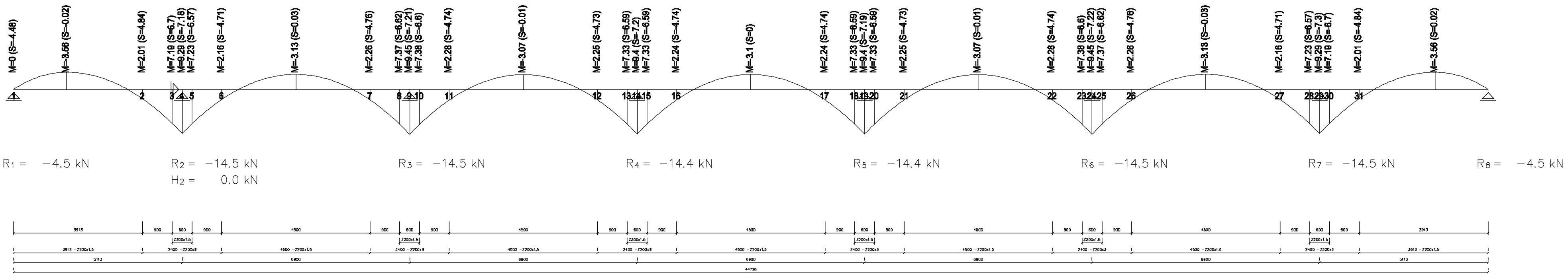
Trapezoidal loads

name	(no.)	f1 (↓)	f2 (↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
FL	(201)	0	1.50	0.320	0.320	1.50	0
FR	(202)	0	1.50	0.320	0.320	1.50	43.23

Combinations

No.	Name and definition of combination
1	PurlinComb1(5) 1.35x(101+102+104)+1.5x(200)+0.9x(301+107)
2	PurlinComb2(6) 1x(101+102)+1.5x(300+108+201+202)

(201)=0.480 x1.500
(108)=0.282 x1.500
(300)=1.305 x1.500
(102)=-0.240 x1.000
(202)=0.480



Geometry of the purlin.

Nodes		
No.	x [m]	type
1	0.00	sup.
2	3.91	-
3	4.81	-
4	5.11	sup.
5	5.41	-
6	6.31	-
7	10.81	-
8	11.71	-
9	12.01	sup.
10	12.31	-
11	13.21	-
12	17.71	-
13	18.61	-
14	18.91	sup.
15	19.21	-
16	20.11	-
17	24.61	-
18	25.51	-
19	25.81	sup.
20	26.11	-
21	27.01	-
22	31.51	-
23	32.41	-
24	32.71	sup.
25	33.01	-
26	33.91	-
27	38.41	-
28	39.31	-
29	39.61	sup.
30	39.91	-
31	40.81	-
32	44.73	sup.

Bars			
start	end	section	len.[m]
1	2	Z 200x55x1.5	3.91
2	3	Z 200x55x3	0.90
3	5	ZZ 200x55x4.5	0.60
5	6	Z 200x55x3	0.90
6	7	Z 200x55x1.5	4.50
7	8	Z 200x55x3	0.90
8	10	ZZ 200x55x4.5	0.60
10	11	Z 200x55x3	0.90
11	12	Z 200x55x1.5	4.50
12	13	Z 200x55x3	0.90
13	15	ZZ 200x55x4.5	0.60
15	16	Z 200x55x3	0.90
16	17	Z 200x55x1.5	4.50
17	18	Z 200x55x3	0.90
18	20	ZZ 200x55x4.5	0.60
20	21	Z 200x55x3	0.90
21	22	Z 200x55x1.5	4.50
22	23	Z 200x55x3	0.90
23	25	ZZ 200x55x4.5	0.60
25	26	Z 200x55x3	0.90
26	27	Z 200x55x1.5	4.50
27	28	Z 200x55x3	0.90
28	30	ZZ 200x55x4.5	0.60
30	31	Z 200x55x3	0.90
31	32	Z 200x55x1.5	3.91

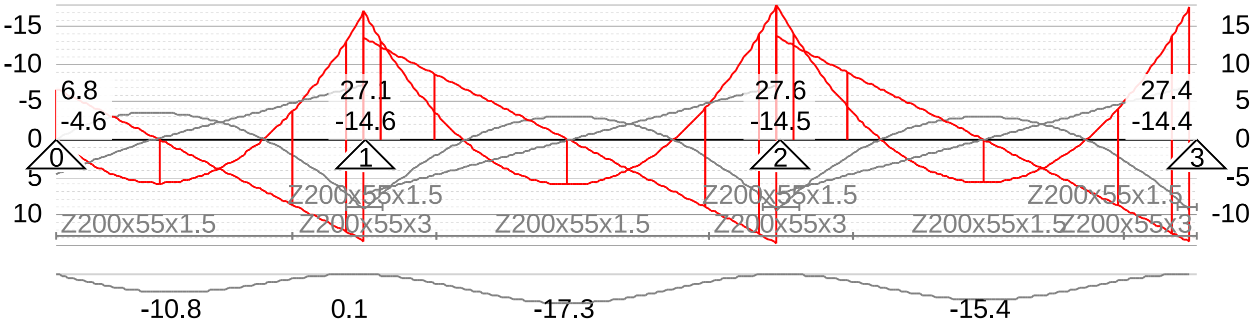
Profiles used in the construction.

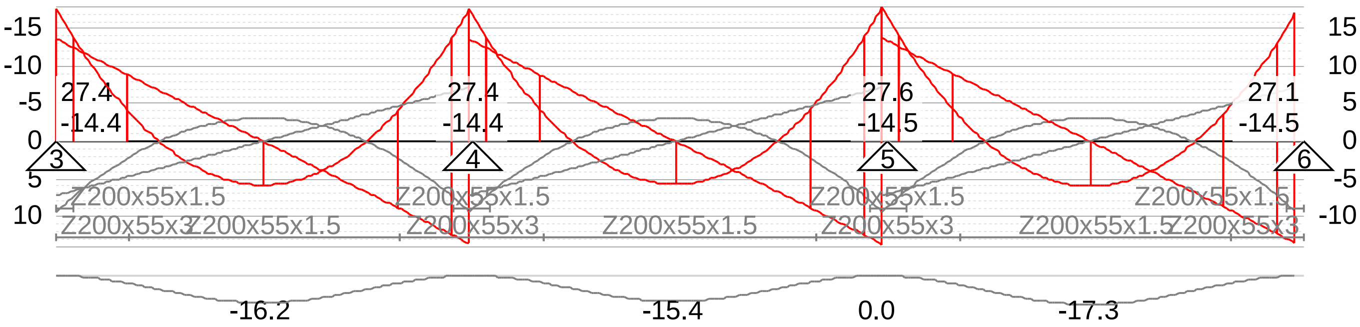
section	length [m]	weight [kg/m]	Iz [mm4]
Z 200x55x1.5	30.3	4.0	3.268 E05
Z 200x55x3	10.8	7.9	6.954 E05
ZZ 200x55x4.5	3.6	11.8	1.022 E06

av.weight = 5.53 [kg/m]

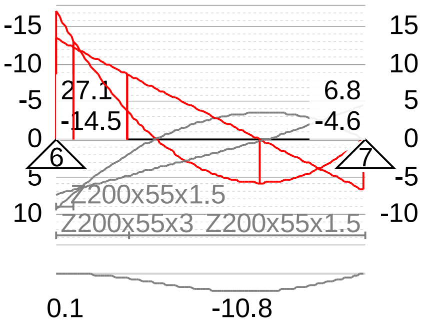
av.weight = 5.89 [kg/m] (inc.overlaps)

Bending moment, shear force, displacement and support reactions in purlin





dlim=6900/200=34mm
VYHOVUJE



dlim=5113/200=25mm
VYHOVUJE

Loads applied to the purlin

Distributed loads

name	(no.)	f1(↓)	f2(↑)	val.[kPa]
Self w.	(101)	1.35	1	-
roof	(102)	1.35	1	-0.16
extra	(104)	1.35	0	-0.1
snow	(200)	1.5	0	-1.2
windSuc	(300)	0	1.5	0.87
windPre	(301)	0.9	0	-0.21
windIntS	(108)	0.9	0	-0.282
windIntP	(109)	0	1.5	0.188

Trapezoidal loads

name	(no.)	f1(↓)	f2(↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
FL	(201)	0	1.5	0.360	0.360	1.50	0
FR	(202)	0	1.5	0.330	0.330	1.50	43.23

Combinations

No.	Name and definition of combination
1	PurlinComb1(5) (ULS)
	1.35x(101+102+104)+1.5x(200)+0.9x(301+107)
2	PurlinComb2(6) (ULS)
	1x(101+102)+1.5x(300+108+201+202)
3	PurlinComb3(7) (SLS)
	1x(101+102+104+200)+0.6x(301+107)

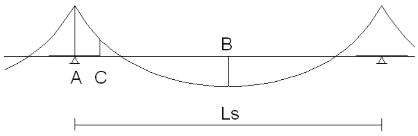
(107)=-0.423	x0.9
(301)=-0.315	x0.9
(200)=-1.800	x1.5
(104)=-0.150	x1.35
(102)=-0.240	x1.35

cc=1500mm

(202)=0.540	x1.5
(201)=0.540	x1.5
(108)=0.282	x1.5
(300)=1.305	x1.5
(102)=-0.240	x1

(107)=-0.423	x0.6
(301)=-0.315	x0.6
(200)=-1.800	x1
(104)=-0.150	x1
(102)=-0.240	x1

Control of purlin Z-200 according to EN 1993-1-3



Global values: all measures in mm

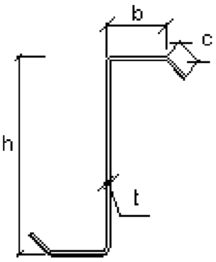
t.rp = 0.58   thickness roof plate   hw.rp = 45   height roof plate

b_roof = 14560   width of roof (hall) $\gamma M0=1$ $\gamma M1=1$ $\gamma M2=1.25$

Stresses on roof purlin   qedg = max gravity load (case 1); qedu = max uplift load (case 2)

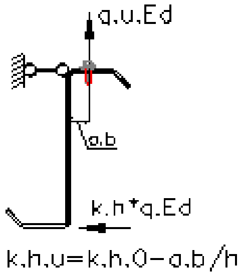
Situation	B1	B2
Section	Z 200x55x1.5	Z 200x55x1.5
cc_pur.[mm]	1500	1500
qedg [kN/m]	3.89	3.89
qedu [kN/m]	3.19	2.38
Ls [m]	5.11	6.90
Gable	Yes	No
NEd g/u [kN]	0.00/0.00	0.00/0.00

Sections' dimensions



	Z 200x55x1.5			**Z 200x55x3**	
MyRk	= 7.58	kNm	MyRk	= 19.59	kNm
VbhRk	= 18.42	kN	VbhRk	= 102.52	kN
RwRk	= 16.47	kN	RwRk	= 51.90	kN
Wyeff	= 2.166 E04	mm3	Wyeff	= 5.596 E04	mm3
Iy	= 2.879 E06	mm4	Iy	= 5.725 E06	mm4
IT	= 3.525 E02	mm4	IT	= 2.882 E03	mm4
$I\omega$	= 2.257 E09	mm6	$I\omega$	= 4.769 E09	mm6

Lateral coefficient EN 1993-1-3: 10.1.4.1 (4): for uplift loading



$$kh0 = \frac{\mathrm{hc} * \mathrm{t} * \left(\mathrm{bc}^2 + 2 * \mathrm{bc} * \mathrm{cc} - 2 * \mathrm{cc}^2 \frac{\mathrm{bc}}{\mathrm{hc}}\right)}{4 * \mathrm{Iy}}$$

*k_hu = k_h0-ab/hc; q_hEdu = qEdu*k_hu*

Z 200x55x1.5

a_b	= 26.8	mm
k_h0	= 0.118	-
k_hu	= -0.017	-
q_hEdu	= -0.04	kN/m

Rotational restraint given by sheeting EN 1993-13 10.1.5

Rotional spring stiffness K, EN 1993-1-3: 10.1.5.2

Rotational stiffness corresponding to flexural stiffness of sheeting EN 1993-1-3: 10.1.5.2 (4):

tension in upper flange: I_effTP46_t = 125,401 mm4/m   C_DC_t = 6*E*I_effTP46_t/cc_pur

compression in upper flange: I_effTP46_c = 130,984 mm4/m   C_DC_c = 6*E*I_effTP46_c/cc_pur

Rotational stiffness of connection between sheeting and purlin ktg = (trp/(0.75*1mm))^1.5 = 0.687

for uplift load   pin in every trough C_100cc150_u = 2.6kN; pin in alternate trough C_100cc300_u = 1.7kN; b=55mm

C_DAcc150_u = C_100cc150_u*(b/1mm/100)^2*ktg = 0.5kN

C_DAcc300_u = C_100cc300_u*(b/1mm/100)^2*ktg = 0.4kN

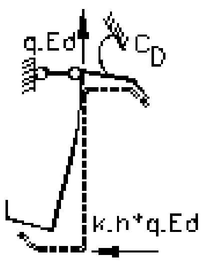
Rotational stiffness

C_Dcc150_u = [1/C_DAcc150_u+1/C_DC_t]^-1 <=> pin in every trough

C_Dcc300_u = [1/C_DAcc300_u+1/C_DC_t]^-1 <=> pin in alternate trough

For uplift load EN 1993-1-3: 10.1.5.1(4): *b_mod_u* = | *2*a_b+b* <=> *q_hEd_u < 0*
| *a_b* <=> *Else*

$$K\_Bu = \frac{\mathrm{E} * \mathrm{t}^3}{4 * (1 - \upsilon^2) * h^2 * (h + b\_mod\_u)}$$



Situation	B1	B2
b_mod_u [mm]	108.6	108.6
K_Bu [N/mm2]	0.016	0.016
1/K_Bu [mm2/N]	63.392	63.392
C_Dcc150_u [kN]	0.538	0.538
C_Dcc300_u [kN]	0.352	0.352

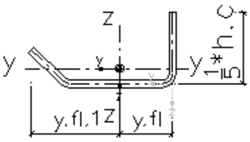
Lateral spring stiffnes per unit length:

pin in every trough: $$K\_ucc150 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc150\_u}}\right)^{-1}$$

pin in alternate trough: $$K\_ucc300 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc300\_u}}\right)^{-1}$$

Situation	B1	B2
K_ucc150 [N/mm2]	0.0073	0.0073
K_ucc300 [N/mm2]	0.0057	0.0057

Gross properties of the free flange EN 1993-1-3: 10.1.4.1



Z 200x55x1.5

t	= 1.5	mm
I_zf1	= 82,869	mm4
W_zf1	= 1,917.1	mm3
i_zf1	= 22.86	kmm

Lateral bending moment for free flanges in compression EN 1993-1-3:10.1.4.1 (5)-(7):

Coefficient R of the spring support EN 1993-1-3: 10.1.4.1 (7):

$$R\_rp = \frac{\mathrm{K\_ucc300} * \mathrm{L\_sp}^4}{\pi^4 * \mathrm{E} * \mathrm{I\_zf1}}$$

Situation	B1	B2
R_rp [-]	2.28	7.56

Correction factor K_R and initial moment M0_fzEd acc. to table 10.1 EN 1993-1-3: 10.1.4.1

M_0fzED = | *1*|q_hEd_u|*L_s^2 /24 <=> no gable*
 | *9*|q_hEd_u|*L_s^2 /128 <=> gable*

κ_Rm = | *(1-0.0125*R_rp)/(1+0.198*R_rp) <=> no gable*
 | *(1-0.0141*R_rp)/(1+0.416*R_rp) <=> gable*

*M_fzED = κ_Rm * M_0fzED*

Situation	B1(gable)	B2
M_0fzEd [Nm]	99.52	80.14
κ_Rm [-]	0.50	0.36
M_fzEd [Nm]	49.45	29.07

Stresses due to gravity load

Combined bending moment and support reaction EN 1993-1-3: 6.1.11. The web rotation is prevented.

*Exertion = {[M_yEd**$\gamma M0$ */MyRk + R_Ed**$\gamma M1$ */RwRk]}/1.25 < 1*

Situation	A1
Section	ZZ 200x55x4.5
Sup.width [m]	0.1
M_yEd [kNm]	17.93
M_yRk [kNm]	27.17
R_Ed [kN]	27.52
R_wRk [kN]	68.38
Exertion	**0.85**

Combined bending moment and compression force: EN 1993-1-3:10.1.4.1 eqv. 10.3a

*Exertion = [M_yEd/Wyeff + N_Ed/Aeff]**$\gamma M1$ */f_yb < 1*

Situation	A1-gravity
M_yEd [kNm]	17.93
Wyeff [mm3]	7.763 E04
N_Ed [kN]	0.0
Aeff [mm2]	716.9
Exertion	**0.66**

Bending moment and shear force:

*Relation = 2*V_Ed**$\gamma M0$ */VbhRk <= 1*

Situation	A1-gravity
V_Ed [kN]	13.80
VbhRk [kN]	120.94
Relation	**0.23**

If Relation <= 1 then combination of bending moment and shear force does not need to be checked.

Single profile at end of supporting profile:

Exertion = [|M_Ls|/Wyeff + N_Ed/Aeff]$\gamma M0$ /f_yb <= 1*

Situation	C1-gravity	C2-gravity
Section	Z 200x55x1.5	Z 200x55x3
M_Ls [kNm]	4.36	14.00
Wyeff [mm3]	2.166 E04	5.596 E04
Aeff [mm2]	225.9	716.9
N_Ed [kN]	0.00	0.00
Exertion	**0.58**	**0.71**

Combined bending moment and compression force in span

Exertion = [M_yEd/Wyeff + N_Ed/Aeff]$\gamma M1$ /f_yb < 1*

Situation	B1	B2
Section	Z 200x55x1.5	Z 200x55x1.5
M_yEd [kNm]	5.78	6.01
Wyeff [mm3]	2.166 E04	2.166 E04
N_Ed [kN]	0.0	0.0
Aeff [mm2]	225.9	225.9
Exertion	**0.76**	**0.79**

Stresses due to Uplift load:

Non-dim. slenderness: $\lambda\_1 = \sqrt{\frac{E}{f\_yb}}$ $\lambda\_1 = 76.95$

Coefficients from table 10.2b:

gable <=> $\eta\_1 = 0.515$ $\eta\_2 = 1.260$ $\eta\_3 = 0.868$ $\eta\_4 = -0.242$

no gable<=> $\eta\_1 = 0.306$ $\eta\_2 = 0.232$ $\eta\_3 = 0.742$ $\eta\_4 = -0.279$

Buckling lenght for free flange in compression EN 1993-1-3: $L\_fz = \eta\_1 * L\_sp * (1 + \eta\_2 * R\_rp^{\eta\_3})^{\eta\_4}$

Relative slenderness for flexural buckling of free flange: $\lambda\_rfz = \frac{L\_fz}{i\_zf1 * \lambda\_1}$

Reduction factor for lateral torsional buckling: Acc. to 10.1.4.2 (1): use values given in EN 1993-1-1: 6.3.2.3

$\alpha\_LT = 0.34$ $\lambda\_rLT = 0.4$ $\beta = 0.75$ $\phi\_LT = 0.5 * [1 + \alpha\_LT * (\lambda\_rfz - \lambda\_rLT) + \beta * \lambda^2]$

$$\chi\_LT = min\left(\frac{1}{\phi\_LT + \sqrt{\phi\_LT^2 + \beta * \lambda\_rfz^2}}, 1, \frac{1}{\lambda\_rfz^2}\right)$$

Reduction factor for flexural buckling: According to EN 1993-1-1: 6.3.1:

$$i\_y = \sqrt{\frac{I\_yrc}{Agrc}} \quad \lambda\_ryFB = \frac{L\_sp}{i\_y} * \sqrt{\frac{Aeff}{Agrc}} * \frac{1}{\lambda\_1} \quad N\_cry = \frac{\pi^2 * E * I\_yrc}{L\_sp^2}$$

$$\phi\_yFB = 0.5 * [1 + \alpha\_yFB * (\lambda\_ryFB - 0.2) + \lambda\_ryFB^2]$$

Reduction factor for span:

$$\chi\_yFB = \min\left(\frac{1}{\phi\_yFB + \sqrt{\phi\_yFB^2 - \lambda\_ryFB^2}}, 1\right)$$

Single profile in span uplift load:

$$Exertion = \left(\frac{1}{\chi\_LT} * \frac{M\_yEd}{W\_yeff} + \frac{1}{\min(\chi\_LT, \chi\_yFB)} * \frac{N\_Ed}{Aeff}\right) * \frac{\gamma M1}{f\_yb} + \frac{|M_{fzEd}| * \gamma M1}{W\_zf1 * f\_yb} < 1.0$$

Situation	B1	B2
Section	Z 200x55x1.5	Z 200x55x1.5
L_sp [m]	5.11	6.90
M_yEd [kNm]	3.62	3.12
M_fzEd [Nm]	49.45	29.07
L_fz [m]	1.93	1.73
λ_rfz	1.10	0.98
Φ_LT	1.07	0.96
χ_LT	0.64	0.71
λ_ryFB	0.59	0.79
N_Ed [kN]	0.0	0.0
Φ_yFB	0.74	0.92
χ_yFB	0.84	0.73
Exertion	**0.82**	**0.63**

Check joints between purlins:

$$Exertion = \frac{|M\_S| * \gamma M2}{4 * F\_bRk * h/2} + \frac{N\_Ed * \gamma M2}{4 * F\_bRk} < 1.0$$

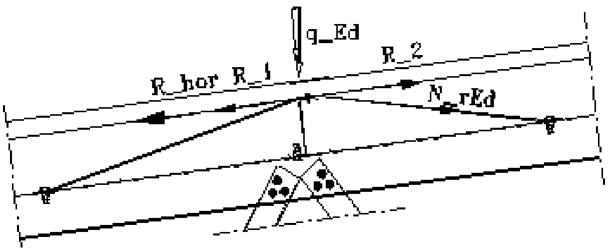
Situation	S1
Section	Z 200x55x1.5
F_bRk [kN]	18.9
M_S [kNm]	-4.36
NEd [kN]	0.0
Exertion	**0.72**

Bracing of Z-roof purlin

Type of hall = S-hall => broof_1 = broof = 14.56m Loadig width frame LW = 7m

Roofslope α_roof = 17° Rigde flashing = No

Max reaction Rg = 27.52kN Rg_slope = Rg*cos(α_roof) = 26.27kN Lateral coeff. k_hg = 0.118



from roof: R1 = sin(α_roof)*Rg
R1 = 8.21kN

from profile: R2 = Rg_slope*k_gh
R2 = 3.10kN

adjust value from profile: R2a = 0.5*R2 <=> α_roof > atan(1/10)
R2a = 1.55kN

Rhor = R1 - R2a
Rhor = 6.66kN

Force acting in the roof plane for one half of the building: P_roof = 0.5*broof1*Rhor/cc_pur
P_roof = 32.32kN

Shear resistance of screws:

– *Φ4.8:* F_4.8_sRd = 3.33kN/γM2 = 2.7kN – *Φ6.3:* F_6.3_sRd = 6.24kN/γM2 = 5.0kN

Overlap screw for use in ridge connection *(Φ4.8):*

d_oscr = 4.8mm t_ridgeplat = 0.5mm ultimate strength ridge plate: f_urp = 330N/mm2

α_oscr = 3.2*√(t_ridgeplat/d_oscr) = 1.03 F_oscr_bRd1 = (α_oscr*f_urp*d_oscr*t_ridgeplat)/γM2 = 0.654kN

F_oscr_bRd = min(F_oscr_bRd1, F_4.8_sRd) = 0.654kN Bearing resistance plate to plate or shear capacity of screw

Plate screw for use in fastening of roofbrace to purlin *(Φ6.3):*

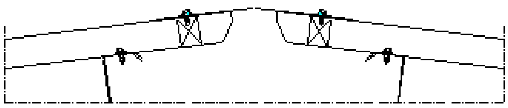
d_dscr = 6.3mm dw_dscr = 19.0mm thickness of z-support = 1.5mm thickness of plate t_bra = 0.58mm

α = 3.2*√(t_bra/d_dscr) = 0.974 α_1 = min(α, 2.1) = 0.974 α2 = (2.1-α_1)*(t_1/t_bra-1)/(2.5-1)+α_1 = 2.15

t_bra ≥ 1mm => α_q = 2.1 α_qq = (α_q-α_1)*(t_1/t_bra-1)/(2.5-1)+α_1 = 0.97

t_bra = t_1 => α_dscrEN = α_1 F_dscr_brdEN = (α_dscrEN*f_u*d_dscr*t_bra)/γM2 = 4.33kN FplabRd=5.6kN

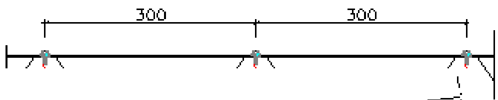
F_dscr_brd = min(F_6.3_sRd,F_dscr_brdEN,FplabRd)=4.33kN



Part of force in plane of roof which is taken by ridge flashing:

Ridge flashing = No => LW_ridge = 0m

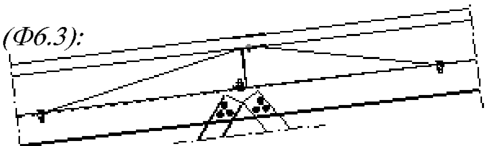
P_Rd_ridge = LW_ridge*F_oscr_bRd/0.3m = 0.00kN



Number of braces needed for each half of the roof: (FIRST CHOICE)

Design resistance for one brace 38x1.5 mm fastened with platescrew: *(Φ6.3):*

N_B = max{1, ceil[(|P_roof|-P_Rd_ridge)/F_dscr_brd]} **N_B = 8**



<u>If using C-profile:</u>

d_dscr = 6.3mm dw_dscr = 19.0mm thickness of c-profile = 2.0mm thickness of plate t_rp = 0.58mm

α = 3.2*√(t_rp/d_dscr) = 0.974 α_1 = min(α, 2.1) = 0.974 α2 = (2.1-α_1)*(t_1/t_rp-1)/(2.5-1)+α_1 = 2.793

t_rp < 1mm => α_q = α_1 α_qq = (α_q-α_1)*(t_1/t_rp-1)/(2.5-1)+α_1 = 0.974 futp=0.390kN/mm2

t_1 ≥ 2.5*t_rp and t_rp < 1mm => α_dscrEN = α_1 F_dscr_brdEN = (α_dscrEN*futp*d_dscr*t_rp)/γM2 = 1.12kN

F_dscr_brd = min(F_6.3_sRd, F_dscr_brdEN) = 1.12kN

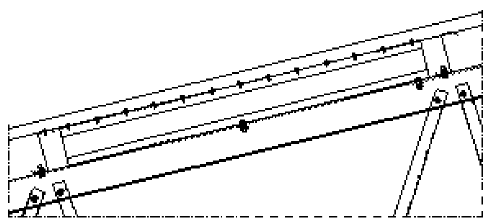
Design resistance for one C-profile (t≥2mm):

P_Rd_Cprof = 10*F_dscr_bRd = 11.2kN

Number of C-profiles needed for each half of the roof:

N_C = max{1, ceil[(|P_roof|-P_Rd_ridge)/P_Rd_Cprof]}

min N_C = 3, min total amount of screws 29



Část konstrukce: PU2 Vaznice zavětrovaná

Poznámky:

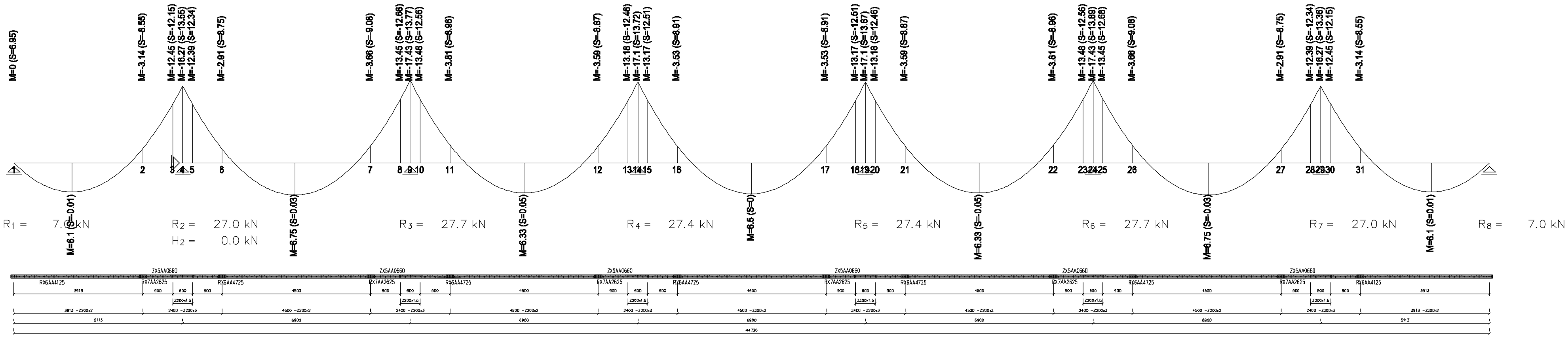
Zatěžovací šířka vaznice Lw=1.5m. Zavětrovaná vaznice je navržena na kombinaci sněhu a tlaku od příčného větru a na maximální sání vyvozené podélným větrem a osovou tlakovou sílu do vaznic ze střešního zavětrování. Uvažovaná osová tlaková síla NEd=30kN.

PU2 Vaznice wind

Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|D - Vaznice|CZ1621 PU2 wind.purx1/27/2022 7:50 by kamilpat
Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|D - Vaznice|CZ1621 PU2 wind.rtd (1/27/2022 time.7:50:06 AM)
Max V = -15.7mm, bar nr 6, comb.PurlinComb3(7)
av.weight = 6.80 [kg/m]

(107)=-0.423 x0.900
(301)=-0.315 x0.900
(200)=-1.800 x1.500
(104)=-0.150 x1.350
(102)=-0.240 x1.350

cc=1500mm



Loads applied to the purlin

Distributed loads

name	(no.)	f1 (↓)	f2 (↑)	val.[kPa]
roof	(102)	1.35	1.00	-0.160
extra	(104)	1.35	0	-0.100
snow	(200)	1.50	0	-1.200
windSuc	(300)	0	1.50	0.590
windPre	(301)	0.90	0	-0.210
windIntS	(107)	0.90	0	-0.282
windIntP	(108)	0	1.50	0.188

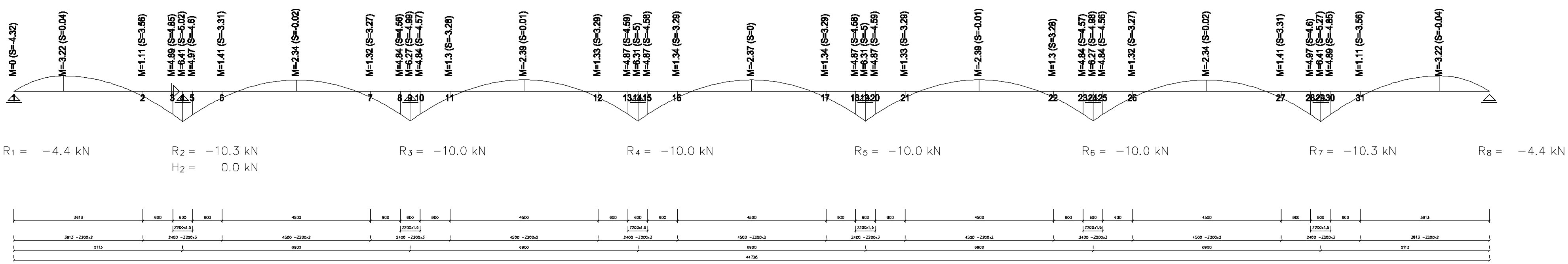
Trapezoidal loads

name	(no.)	f1 (↓)	f2 (↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
FL	(201)	0	1.50	0.640	0.640	1.50	0
FR	(202)	0	1.50	0.640	0.640	1.50	43.23

Combinations

No.	Name and definition of combination
1	PurlinComb1(5)
	1.35x(101+102+104)+1.5x(200)+0.9x(301+107)
2	PurlinComb2(6)
	1x(101+102)+1.5x(300+108+201+202)

(201)=0.960 x1.500
(108)=0.282 x1.500
(300)=0.885 x1.500
(102)=-0.240 x1.000



Geometry of the purlin.

Nodes		
No.	x [m]	type
1	0.00	sup.
2	3.91	-
3	4.81	-
4	5.11	sup.
5	5.41	-
6	6.31	-
7	10.81	-
8	11.71	-
9	12.01	sup.
10	12.31	-
11	13.21	-
12	17.71	-
13	18.61	-
14	18.91	sup.
15	19.21	-
16	20.11	-
17	24.61	-
18	25.51	-
19	25.81	sup.
20	26.11	-
21	27.01	-
22	31.51	-
23	32.41	-
24	32.71	sup.
25	33.01	-
26	33.91	-
27	38.41	-
28	39.31	-
29	39.61	sup.
30	39.91	-
31	40.81	-
32	44.73	sup.

Bars			
start	end	section	len.[m]
1	2	Z 200x55x2	3.91
2	3	Z 200x55x3	0.90
3	5	ZZ 200x55x4.5	0.60
5	6	Z 200x55x3	0.90
6	7	Z 200x55x2	4.50
7	8	Z 200x55x3	0.90
8	10	ZZ 200x55x4.5	0.60
10	11	Z 200x55x3	0.90
11	12	Z 200x55x2	4.50
12	13	Z 200x55x3	0.90
13	15	ZZ 200x55x4.5	0.60
15	16	Z 200x55x3	0.90
16	17	Z 200x55x2	4.50
17	18	Z 200x55x3	0.90
18	20	ZZ 200x55x4.5	0.60
20	21	Z 200x55x3	0.90
21	22	Z 200x55x2	4.50
22	23	Z 200x55x3	0.90
23	25	ZZ 200x55x4.5	0.60
25	26	Z 200x55x3	0.90
26	27	Z 200x55x2	4.50
27	28	Z 200x55x3	0.90
28	30	ZZ 200x55x4.5	0.60
30	31	Z 200x55x3	0.90
31	32	Z 200x55x2	3.91

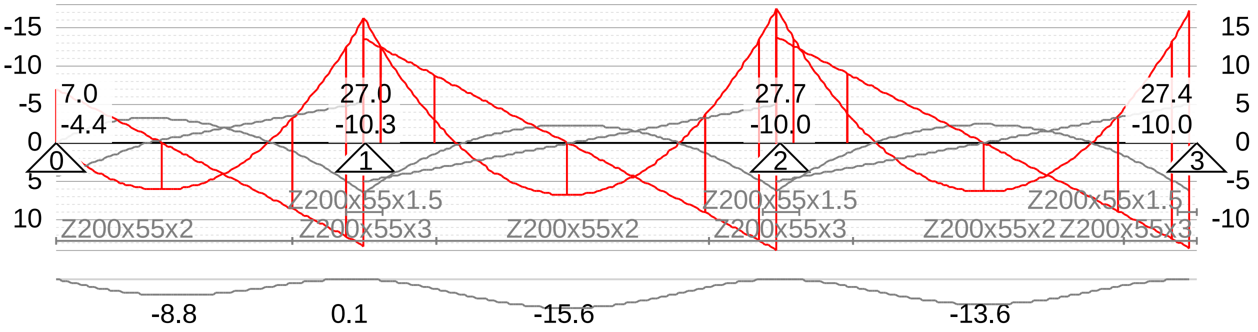
Profiles used in the construction.

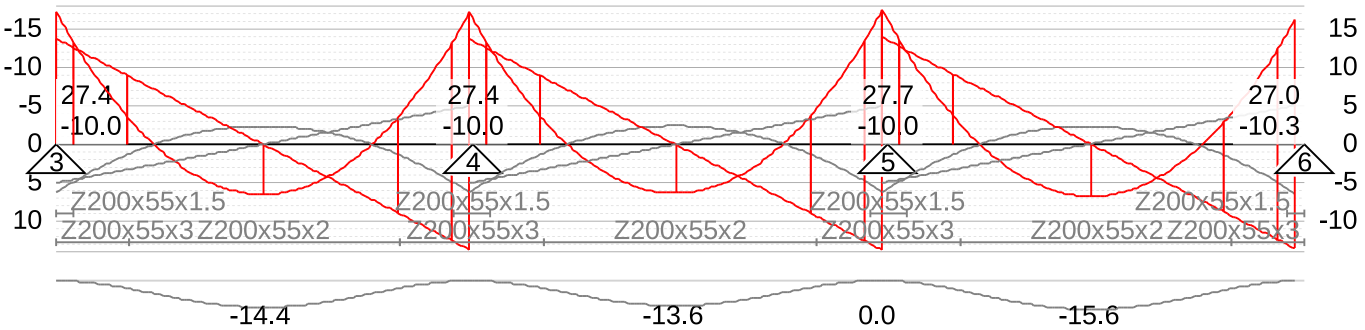
section	length [m]	weight [kg/m]	Iz [mm4]
Z 200x55x2	30.3	5.3	4.471 E05
Z 200x55x3	10.8	7.9	6.954 E05
ZZ 200x55x4.5	3.6	11.8	1.022 E06

av.weight = 6.42 [kg/m]

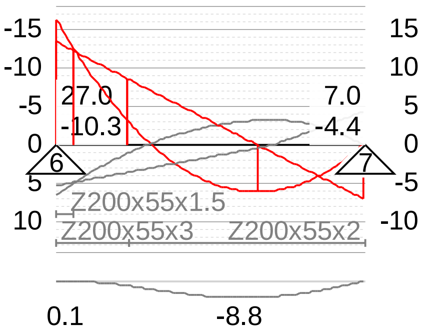
av.weight = 6.80 [kg/m] (inc.overlaps)

Bending moment, shear force, displacement and support reactions in purlin





dlim=6900/200=34mm
VYHOVUJE



dlim=5113/200=25mm
VYHOVUJE

Loads applied to the purlin

Distributed loads

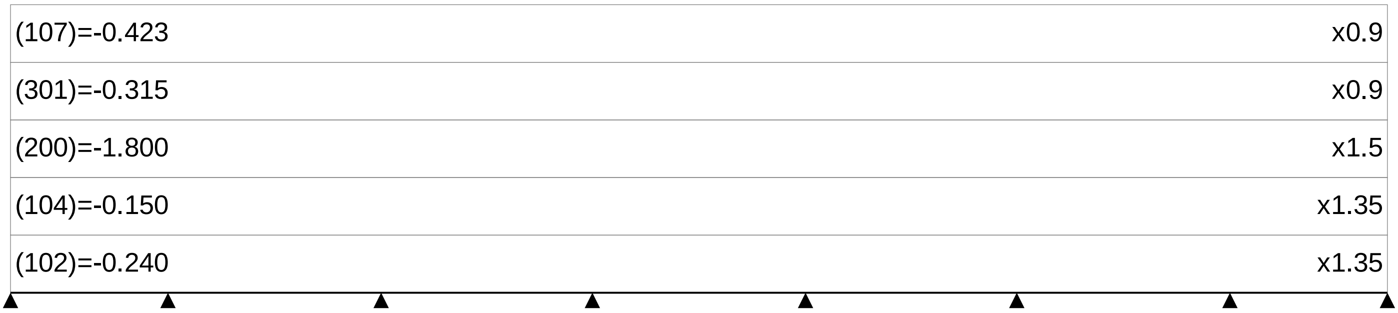
name	(no.)	f1(↓)	f2(↑)	val.[kPa]
Self w.	(101)	1.35	1	-
roof	(102)	1.35	1	-0.16
extra	(104)	1.35	0	-0.1
snow	(200)	1.5	0	-1.2
windSuc	(300)	0	1.5	0.59
windPre	(301)	0.9	0	-0.21
windIntS	(108)	0.9	0	-0.282
windIntP	(109)	0	1.5	0.188

Trapezoidal loads

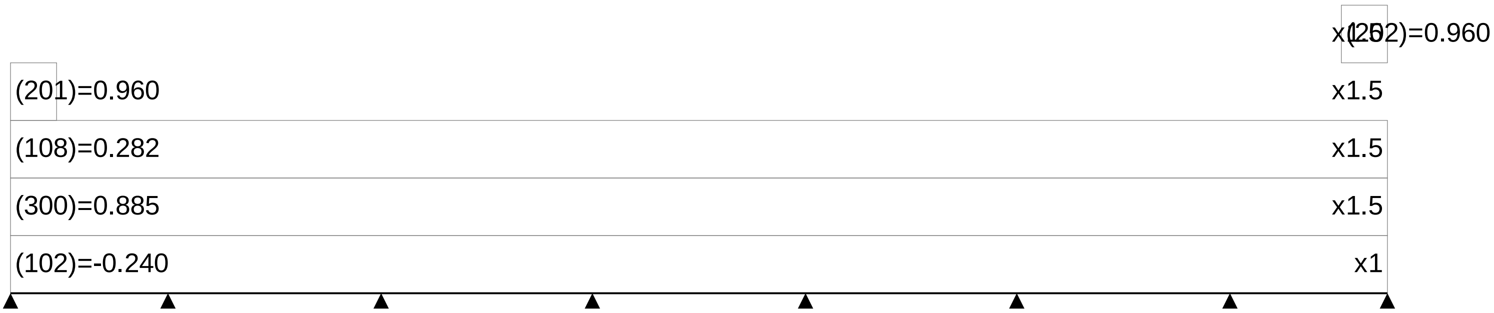
name	(no.)	f1(↓)	f2(↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
FL	(201)	0	1.5	0.640	0.640	1.50	0
FR	(202)	0	1.5	0.640	0.640	1.50	43.23

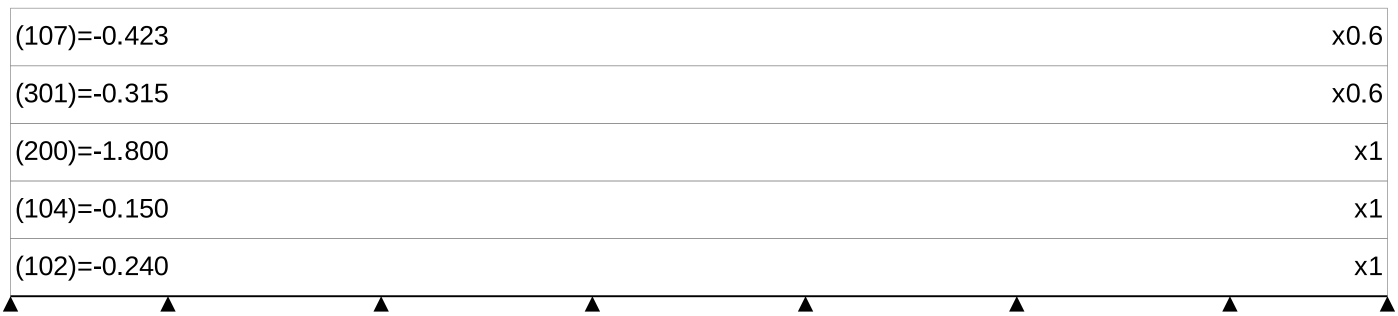
Combinations

No.	Name and definition of combination
1	PurlinComb1(5) (ULS)
	1.35x(101+102+104)+1.5x(200)+0.9x(301+107)
2	PurlinComb2(6) (ULS)
	1x(101+102)+1.5x(300+108+201+202)
3	PurlinComb3(7) (SLS)
	1x(101+102+104+200)+0.6x(301+107)

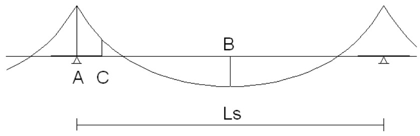


cc=1500mm





Control of purlin Z-200 according to EN 1993-1-3



Global values: all measures in mm

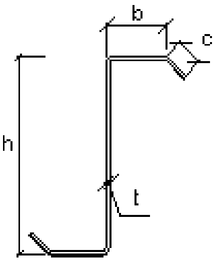
t.rp = 0.58 thickness roof plate hw.rp = 45 height roof plate

b_roof = 15000 width of roof (hall) $\gamma M0=1$ $\gamma M1=1$ $\gamma M2=1.25$

Stresses on roof purlin qedg = max gravity load (case 1); qedu = max uplift load (case 2)

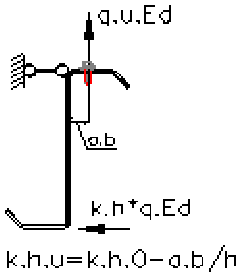
Situation	B1	B2
Section	Z 200x55x2	Z 200x55x2
cc_pur.[mm]	1500	1500
qedg [kN/m]	3.89	3.89
qedu [kN/m]	3.19	1.75
Ls [m]	5.11	6.90
Gable	Yes	No
NEd g/u [kN]	30.00/30.00	30.00/30.00

Sections' dimensions



	Z 200x55x1.5		**Z 200x55x2**		**Z 200x55x3**	
MyRk	= 7.58	kNm	= 12.17	kNm	= 19.59	kNm
VbhRk	= 18.42	kN	= 44.68	kN	= 102.52	kN
RwRk	= 16.47	kN	= 26.27	kN	= 51.90	kN
Wyeff	= 2.166 E04	mm3	= 3.476 E04	mm3	= 5.596 E04	mm3
Iy	= 2.879 E06	mm4	= 3.85 E06	mm4	= 5.725 E06	mm4
IT	= 3.525 E02	mm4	= 8.544 E02	mm4	= 2.882 E03	mm4
$I\omega$	= 2.257 E09	mm6	= 3.08 E09	mm6	= 4.769 E09	mm6

Lateral coefficient EN 1993-1-3: 10.1.4.1 (4): for uplift loading



$$kh0 = \frac{\mathrm{hc} * \mathrm{t} * \left(\mathrm{bc}^2 + 2 * \mathrm{bc} * \mathrm{cc} - 2 * \mathrm{cc}^2 \frac{\mathrm{bc}}{\mathrm{hc}}\right)}{4 * \mathrm{Iy}}$$

*k_hu = k_h0-ab/hc; q_hEdu = qEdu*k_hu*

Z 200x55x2

a_b	= 27.6	mm
k_h0	= 0.118	-
k_hu	= -0.021	-
q_hEdu	= -0.04	kN/m

Rotational restraint given by sheeting EN 1993-13 10.1.5

Rotional spring stiffness K, EN 1993-1-3: 10.1.5.2

Rotational stiffness corresponding to flexural stiffness of sheeting EN 1993-1-3: 10.1.5.2 (4):

tension in upper flange: I_effTP46_t = 125,401 mm4/m C_DC_t = 6*E*I_effTP46_t/cc_pur

compression in upper flange: I_effTP46_c = 130,984 mm4/m C_DC_c = 6*E*I_effTP46_c/cc_pur

Rotational stiffness of connection between sheeting and purlin ktg = (trp/(0.75*1mm))^1.5 = 0.687

for uplift load pin in every trough C_100cc150_u = 2.6kN; pin in alternate trough C_100cc300_u = 1.7kN; b=55mm

C_DAcc150_u = C_100cc150_u*(b/1mm/100)^2*ktg = 0.5kN

C_DAcc300_u = C_100cc300_u*(b/1mm/100)^2*ktg = 0.4kN

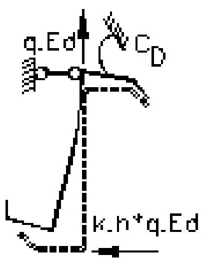
Rotational stiffness

C_Dcc150_u = [1/C_DAcc150_u+1/C_DC_t]^-1 <=> pin in every trough

C_Dcc300_u = [1/C_DAcc300_u+1/C_DC_t]^-1 <=> pin in alternate trough

For uplift load EN 1993-1-3: 10.1.5.1(4): *b_mod_u* = | *2*a_b+b* <=> *q_hEd_u* < *0*
| *a_b* <=> *Else*

$$K\_Bu = \frac{\mathrm{E} * \mathrm{t}^3}{4 * (1 - \upsilon^2) * h^2 * (h + b\_mod\_u)}$$



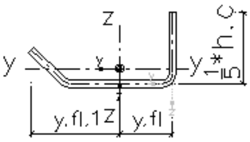
Situation	B1	B2
b_mod_u [mm]	110.2	110.2
K_Bu [N/mm2]	0.037	0.037
1/K_Bu [mm2/N]	26.882	26.882
C_Dcc150_u [kN]	0.538	0.538
C_Dcc300_u [kN]	0.352	0.352

Lateral spring stiffnes per unit length:

pin in every trough: $K\_ucc150 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc150\_u}}\right)^{-1}$ pin in alternate trough: $K\_ucc300 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc300\_u}}\right)^{-1}$

Situation	B1	B2
K_ucc150 [N/mm2]	0.0099	0.0099
K_ucc300 [N/mm2]	0.0071	0.0071

Gross properties of the free flange EN 1993-1-3: 10.1.4.1



Z 200x55x2

t	= 2.0	mm
I_zf1	= 112,773	mm4
W_zf1	= 2,606.5	mm3
i_zf1	= 22.98	kmm

Lateral bending moment for free flanges in compression EN 1993-1-3:10.1.4.1 (5)-(7):

Coefficient R of the spring support EN 1993-1-3: 10.1.4.1 (7):

$$R\_rp = \frac{\mathrm{K\_ucc300} * \mathrm{L\_sp}^4}{\pi^4 * \mathrm{E} * \mathrm{I\_zf1}}$$

Situation	B1	B2
R_rp [-]	2.11	7.00

Correction factor K_R and initial moment M0_fzEd acc. to table 10.1 EN 1993-1-3: 10.1.4.1

M_0fzED = | *1*|q_hEd_u|*L_s^2 /24 <=> no gable*
| *9*|q_hEd_u|*L_s^2 /128 <=> gable*

κ_Rm = | *(1-0.0125*R_rp)/(1+0.198*R_rp) <=> no gable*
| *(1-0.0141*R_rp)/(1+0.416*R_rp) <=> gable*

*M_fzED = κ_Rm * M_0fzED*

Situation	B1(gable)	B2
M_0fzEd [Nm]	124.53	73.74
κ_Rm [-]	0.52	0.38
M_fzEd [Nm]	64.35	28.21

Stresses due to gravity load

Combined bending moment and support reaction EN 1993-1-3: 6.1.11. The web rotation is prevented.

Exertion = {[M_yEd$\gamma M0$ /MyRk + R_Ed*$\gamma M1$ /RwRk]}/1.25 < 1*

Situation	A1
Section	ZZ 200x55x4.5
Sup.width [m]	0.1
M_yEd [kNm]	17.43
M_yRk [kNm]	27.17
R_Ed [kN]	27.65
R_wRk [kN]	68.38
Exertion	**0.84**

Combined bending moment and compression force: EN 1993-1-3:10.1.4.1 eqv. 10.3a

Exertion = [M_yEd/Wyeff + N_Ed/Aeff]$\gamma M1$ /f_yb < 1*

Situation	A1-gravity
M_yEd [kNm]	17.43
Wyeff [mm3]	7.763 E04
N_Ed [kN]	30.0
Aeff [mm2]	716.9
Exertion	**0.76**

Bending moment and shear force:

*Relation = 2*V_Ed*$\gamma M0$ /VbhRk <= 1*

Situation	A1-gravity
V_Ed [kN]	13.88
VbhRk [kN]	120.94
Relation	**0.23**

If Relation <= 1 then combination of bending moment and shear force does not need to be checked.

Single profile at end of supporting profile:

*Exertion = [|M_Ls|/Wyeff + N_Ed/Aeff]** $\gamma M0$ */f_yb <= 1*

Situation	C1-gravity	C2-gravity
Section	Z 200x55x2	Z 200x55x3
M_Ls [kNm]	3.80	13.48
Wyeff [mm3]	3.476 E04	5.596 E04
Aeff [mm2]	378.5	716.9
N_Ed [kN]	30.00	30.00
Exertion	**0.54**	**0.81**

Combined bending moment and compression force in span

*Exertion = [M_yEd/Wyeff + N_Ed/Aeff]** $\gamma M1$ */f_yb < 1*

Situation	B1	B2
Section	Z 200x55x2	Z 200x55x2
M_yEd [kNm]	6.09	6.75
Wyeff [mm3]	3.476 E04	3.476 E04
N_Ed [kN]	30.0	30.0
Aeff [mm2]	378.5	378.5
Exertion	**0.73**	**0.78**

Stresses due to Uplift load:

Non-dim. slenderness: $\lambda\_1 = \sqrt{\frac{E}{f\_yb}}$ $\lambda\_1 = 76.95$

Coefficients from table 10.2b:

gable <=> $\eta\_1 = 0.515$ $\eta\_2 = 1.260$ $\eta\_3 = 0.868$ $\eta\_4 = -0.242$

no gable<=> $\eta\_1 = 0.306$ $\eta\_2 = 0.232$ $\eta\_3 = 0.742$ $\eta\_4 = -0.279$

Buckling lenght for free flange in compression EN 1993-1-3: $L\_fz = \eta\_1 * L\_sp * (1 + \eta\_2 * R\_rp^{\eta\_3})^{\eta\_4}$

Relative slenderness for flexural buckling of free flange: $\lambda\_rfz = \frac{L\_fz}{i\_zf1 * \lambda\_1}$

Reduction factor for lateral torsional buckling: Acc. to 10.1.4.2 (1): use values given in EN 1993-1-1: 6.3.2.3

$\alpha\_LT = 0.34$ $\lambda\_rLT = 0.4$ $\beta = 0.75$ $\phi\_LT = 0.5 * [1 + \alpha\_LT * (\lambda\_rfz - \lambda\_rLT) + \beta * \lambda^2]$

$$\chi\_LT = min\left(\frac{1}{\phi\_LT + \sqrt{\phi\_LT^2 + \beta * \lambda\_rfz^2}}, 1, \frac{1}{\lambda\_rfz^2}\right)$$

Reduction factor for flexural buckling: According to EN 1993-1-1: 6.3.1:

$$i\_y = \sqrt{\frac{I\_yrc}{Agrc}} \quad \lambda\_ryFB = \frac{L\_sp}{i\_y} * \sqrt{\frac{Aeff}{Agrc}} * \frac{1}{\lambda\_1} \quad N\_cry = \frac{\pi^2 * E * I\_yrc}{L\_sp^2}$$

$$\phi\_yFB = 0.5 * [1 + \alpha\_yFB * (\lambda\_ryFB - 0.2) + \lambda\_ryFB^2]$$

Reduction factor for span:

$$\chi\_yFB = \min\left(\frac{1}{\phi\_yFB + \sqrt{\phi\_yFB^2 - \lambda\_ryFB^2}}, 1\right)$$

Single profile in span uplift load:

$$Exertion = \left(\frac{1}{\chi\_LT} * \frac{M\_yEd}{W\_yeff} + \frac{1}{\min(\chi\_LT, \chi\_yFB)} * \frac{N\_Ed}{Aeff}\right) * \frac{\gamma M1}{f\_yb} + \frac{|M_{fzEd}| * \gamma M1}{W\_zf1 * f\_yb} < \quad 1.0$$

Situation	B1	B2
Section	Z 200x55x2	Z 200x55x2
L_sp [m]	5.11	6.90
M_yEd [kNm]	3.22	2.39
M_fzEd [Nm]	64.35	28.21
L_fz [m]	1.96	1.74
λ_rfz	1.11	0.99
Φ_LT	1.08	0.96
χ_LT	0.63	0.71
λ_ryFB	0.66	0.89
N_Ed [kN]	30.0	30.0
Φ_yFB	0.79	1.01
χ_yFB	0.81	0.67
Exertion	**0.85**	**0.65**

Check joints between purlins:

$$Exertion = \frac{|M\_S| * \gamma M2}{4 * F\_bRk * h/2} + \frac{N\_Ed * \gamma M2}{4 * F\_bRk} < 1.0$$

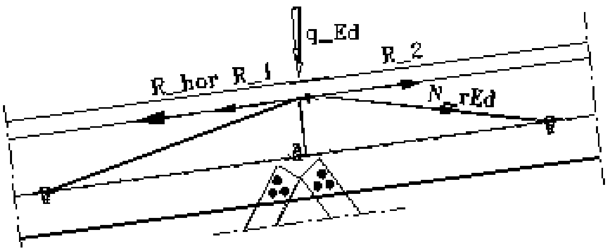
Situation	S1
Section	Z 200x55x2
F_bRk [kN]	25.2
M_S [kNm]	-3.80
NEd [kN]	30.0
Exertion	**0.84**

Bracing of Z-roof purlin

Type of hall = S-hall => broof_1 = broof = 15.00m Loadig width frame LW = 7m

Roofslope α_roof = 17° Rigde flashing = No

Max reaction Rg = 27.65kN Rg_slope = Rg*cos(α_roof) = 26.39kN Lateral coeff. k_hg = 0.118



from roof: R1 = sin(α_roof)*Rg
R1 = 8.25kN

from profile: R2 = Rg_slope*k_gh
R2 = 3.12kN

adjust value from profile: R2a = 0.5*R2 <=> α_roof > atan(1/10)
R2a = 1.56kN

Rhor = R1 - R2a
Rhor = 6.69kN

Force acting in the roof plane for one half of the building: P_roof = 0.5*broof1*Rhor/cc_pur
P_roof = 33.44kN

Shear resistance of screws:

– *Φ4.8:* F_4.8_sRd = 3.33kN/γM2 = 2.7kN – *Φ6.3:* F_6.3_sRd = 6.24kN/γM2 = 5.0kN

Overlap screw for use in ridge connection *(Φ4.8):*

d_oscr = 4.8mm t_ridgeplat = 0.5mm ultimate strength ridge plate: f_urp = 330N/mm2

α_oscr = 3.2*√(t_ridgeplat/d_oscr) = 1.03 F_oscr_bRd1 = (α_oscr*f_urp*d_oscr*t_ridgeplat)/γM2 = 0.654kN

F_oscr_bRd = min(F_oscr_bRd1, F_4.8_sRd) = 0.654kN Bearing resistance plate to plate or shear capacity of screw

Plate screw for use in fastening of roofbrace to purlin *(Φ6.3):*

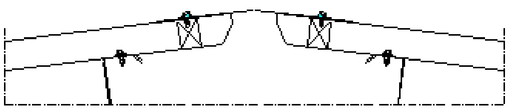
d_dscr = 6.3mm dw_dscr = 19.0mm thickness of z-support = 1.5mm thickness of plate t_bra = 0.58mm

α = 3.2*√(t_bra/d_dscr) = 0.974 α_1 = min(α, 2.1) = 0.974 α2 = (2.1-α_1)*(t_1/t_bra-1)/(2.5-1)+α_1 = 2.15

t_bra ≥ 1mm => α_q = 2.1 α_qq = (α_q-α_1)*(t_1/t_bra-1)/(2.5-1)+α_1 = 0.97

t_bra = t_1 => α_dscrEN = α_1 F_dscr_brdEN = (α_dscrEN*f_u*d_dscr*t_bra)/γM2 = 4.33kN FplabRd=5.6kN

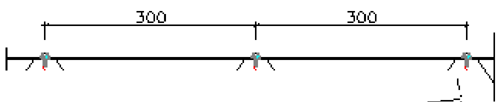
F_dscr_brd = min(F_6.3_sRd,F_dscr_brdEN,FplabRd)=4.33kN



Part of force in plane of roof which is taken by ridge flashing:

Ridge flashing = No => LW_ridge = 0m

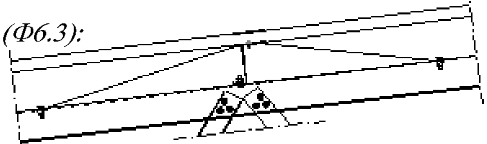
P_Rd_ridge = LW_ridge*F_oscr_bRd/0.3m = 0.00kN



Number of braces needed for each half of the roof: (FIRST CHOICE)

Design resistance for one brace 38x1.5 mm fastened with platescrew: *(Φ6.3):*

N_B = max{1, ceil[(|P_roof|-P_Rd_ridge)/F_dscr_brd]} **N_B = 8**



<u>If using C-profile:</u>

d_dscr = 6.3mm dw_dscr = 19.0mm thickness of c-profile = 2.0mm thickness of plate t_rp = 0.58mm

α = 3.2*√(t_rp/d_dscr) = 0.974 α_1 = min(α, 2.1) = 0.974 α2 = (2.1-α_1)*(t_1/t_rp-1)/(2.5-1)+α_1 = 2.793

t_rp < 1mm => α_q = α_1 α_qq = (α_q-α_1)*(t_1/t_rp-1)/(2.5-1)+α_1 = 0.974 futp=0.390kN/mm2

t_1 ≥ 2.5*t_rp and t_rp < 1mm => α_dscrEN = α_1 F_dscr_brdEN = (α_dscrEN*futp*d_dscr*t_rp)/γM2 = 1.12kN

F_dscr_brd = min(F_6.3_sRd, F_dscr_brdEN) = 1.12kN

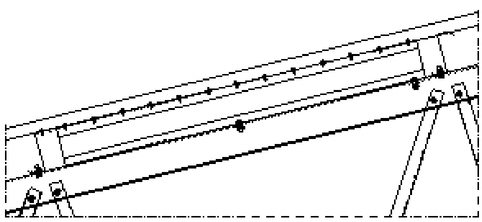
Design resistance for one C-profile (t≥2mm):

P_Rd_Cprof = 10*F_dscr_bRd = 11.2kN

Number of C-profiles needed for each half of the roof:

N_C = max{1, ceil[(|P_roof|-P_Rd_ridge)/P_Rd_Cprof]}

min N_C = 3, min total amount of screws 30

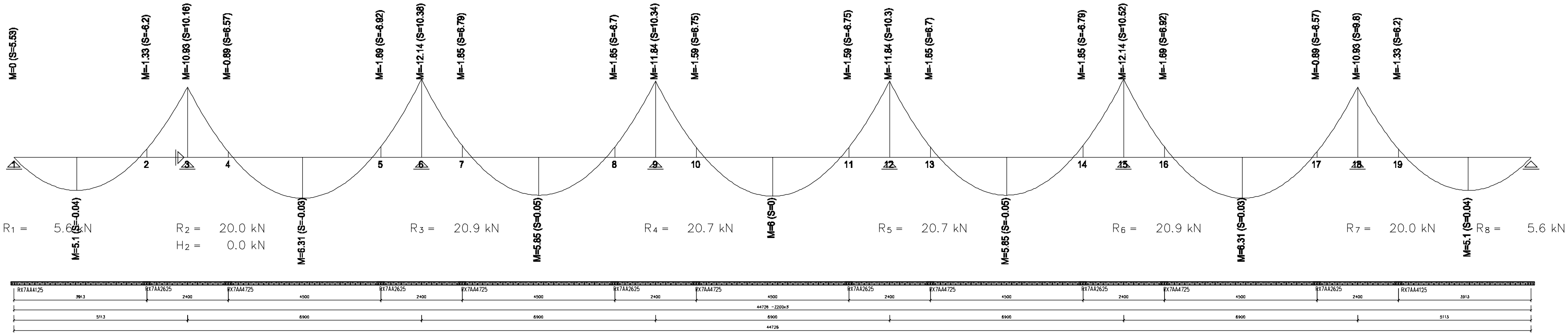


Část konstrukce: PU3 Vaznice okap

Poznámky:
Zatěžovací šířka vaznice Lw=1.1m. Zavětrovaná vaznice je navržena na kombinaci sněhu a tlaku od příčného větru a na maximální sání vyvozené podélným větrem a osovou tlakovou sílu do vaznic ze střešního zavětrování. Uvažovaná osová tlaková síla NEd=68kN.

PU2 Vaznice okap

Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|D - Vaznice|CZ1621 PU3 eaves.purx1/27/2022 8:5 by kamilpat
Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|D - Vaznice|CZ1621 PU3 eaves.rtd (1/27/2022 time.8:05:16 AM)
Max V = -11.2mm, bar nr 16, comb.PurlinComb3(7)
av.weight = 8.20 [kg/m]



Loads applied to the purlin

Distributed loads

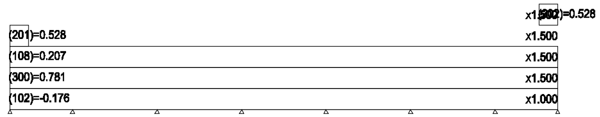
name	(no.)	f1 (↓)	f2 (↑)	val.[kPa]
roof	(102)	1.35	1.00	-0.160
extra	(104)	1.35	0	-0.100
snow	(200)	1.50	0	-1.200
windSuc	(300)	0	1.50	0.710
windPre	(301)	0.90	0	-0.250
windIntS	(107)	0.90	0	-0.282
windIntP	(108)	0	1.50	0.188

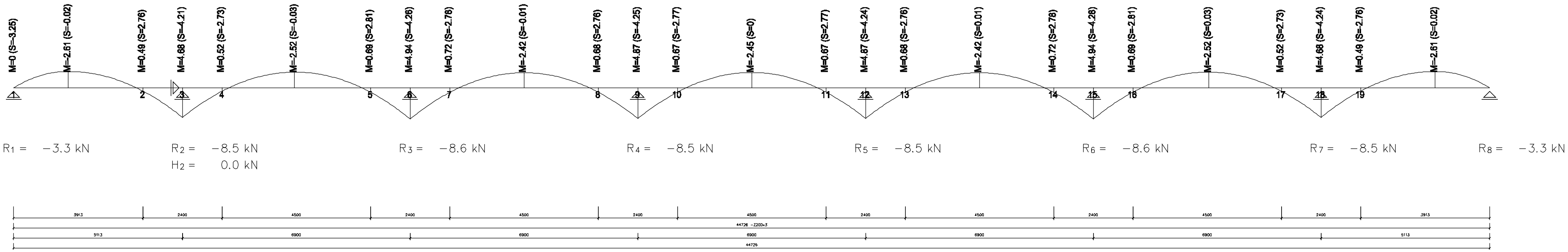
Trapezoidal loads

name	(no.)	f1 (↓)	f2 (↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
FL	(201)	0	1.50	0.480	0.480	1.50	0
FR	(202)	0	1.50	0.480	0.480	1.50	43.23

Combinations

No.	Name and definition of combination
1	PurlinComb1(5) 1.35x(101+102+104)+1.5x(200)+0.9x(301+107)
2	PurlinComb2(6) 1x(101+102)+1.5x(300+108+201+202)





Geometry of the purlin.

Nodes		
No.	x [m]	type
1	0.00	sup.
2	3.91	-
3	5.11	sup.
4	6.31	-
5	10.81	-
6	12.01	sup.
7	13.21	-
8	17.71	-
9	18.91	sup.
10	20.11	-
11	24.61	-
12	25.81	sup.
13	27.01	-
14	31.51	-
15	32.71	sup.
16	33.91	-
17	38.41	-
18	39.61	sup.
19	40.81	-
20	44.73	sup.

Bars			
start	end	section	len.[m]
1	20	Z 200x55x3	44.73

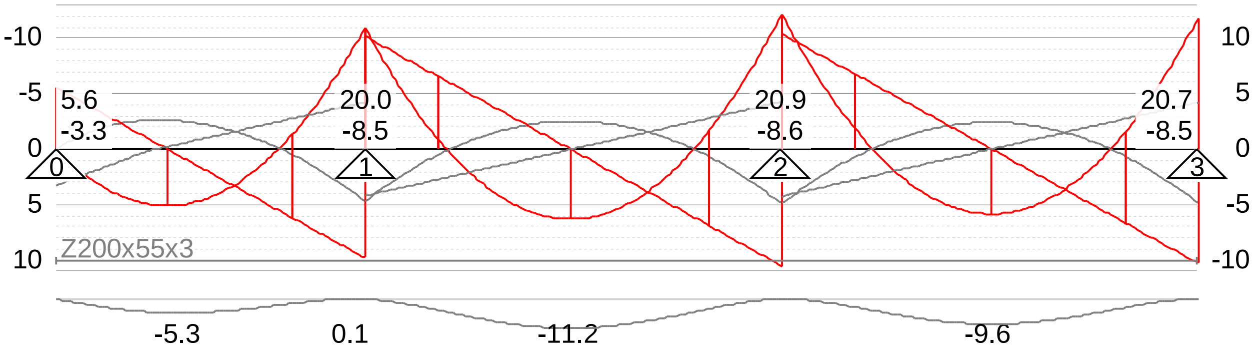
Profiles used in the construction.

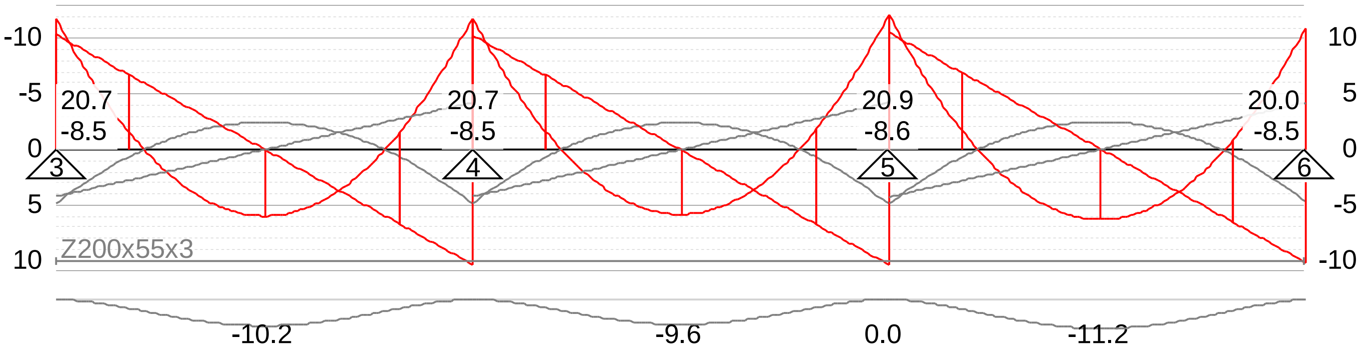
section	length [m]	weight [kg/m]	Iz [mm4]
Z 200x55x3	44.7	7.9	6.954 E05

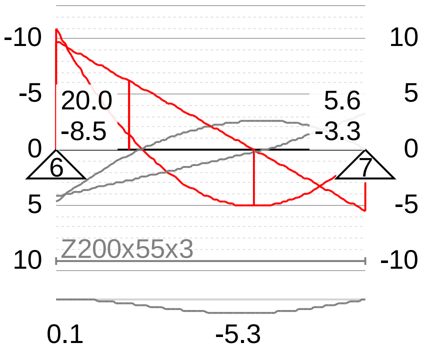
av.weight = 7.88 [kg/m]

av.weight = 8.20 [kg/m] (inc.overlaps)

Bending moment, shear force, displacement and support reactions in purlin







dlim=6900/200=34mm
VYHOVUJE

dlim=5113/200=25mm
VYHOVUJE

Loads applied to the purlin

Distributed loads

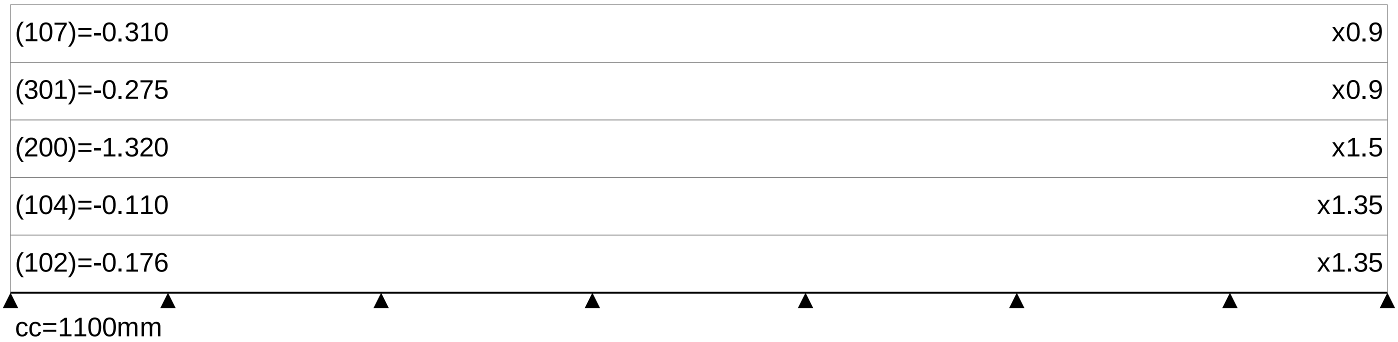
name	(no.)	f1(↓)	f2(↑)	val.[kPa]
Self w.	(101)	1.35	1	-
roof	(102)	1.35	1	-0.16
extra	(104)	1.35	0	-0.1
snow	(200)	1.5	0	-1.2
windSuc	(300)	0	1.5	0.71
windPre	(301)	0.9	0	-0.25
windIntS	(108)	0.9	0	-0.282
windIntP	(109)	0	1.5	0.188

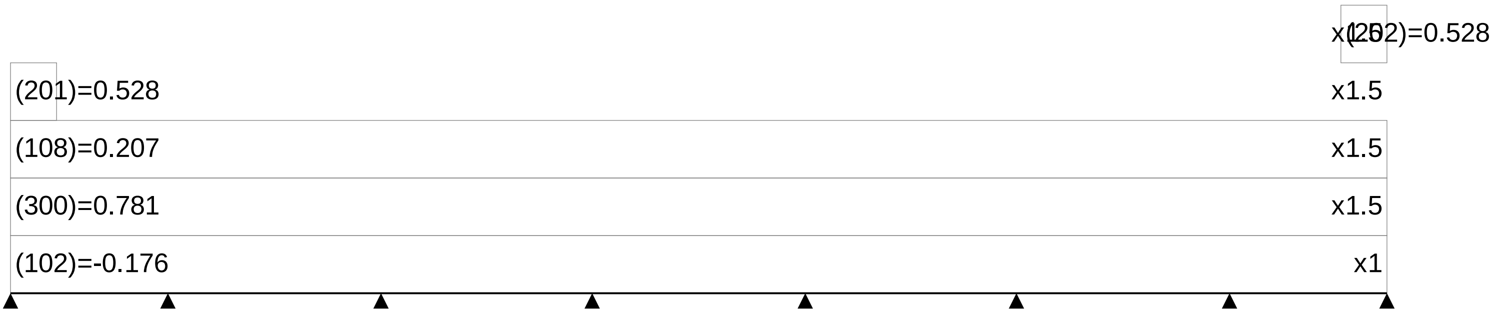
Trapezoidal loads

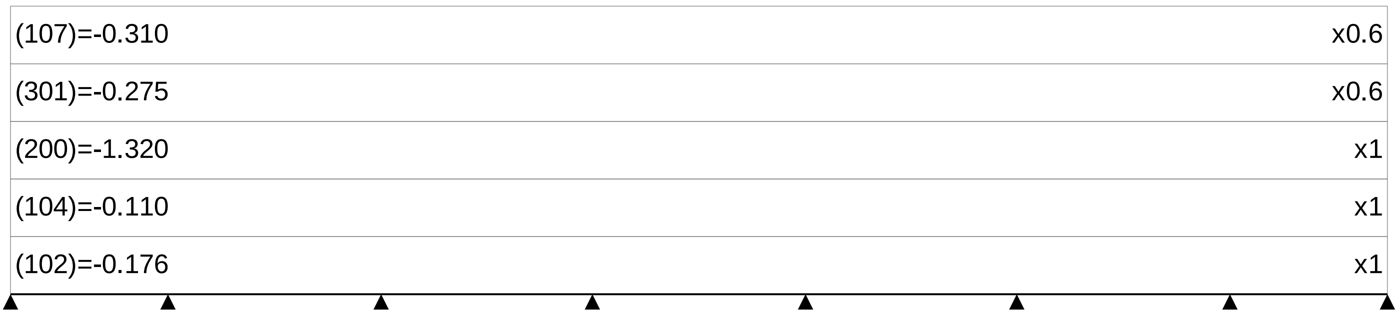
name	(no.)	f1(↓)	f2(↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
FL	(201)	0	1.5	0.480	0.480	1.50	0
FR	(202)	0	1.5	0.480	0.480	1.50	43.23

Combinations

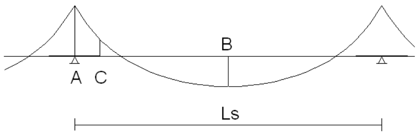
No.	Name and definition of combination
1	PurlinComb1(5) (ULS)
	1.35x(101+102+104)+1.5x(200)+0.9x(301+107)
2	PurlinComb2(6) (ULS)
	1x(101+102)+1.5x(300+108+201+202)
3	PurlinComb3(7) (SLS)
	1x(101+102+104+200)+0.6x(301+107)







Control of purlin Z-200 according to EN 1993-1-3



Global values: all measures in mm

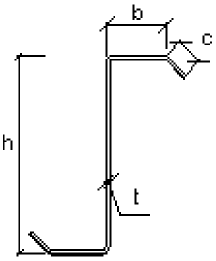
t.rp = 0.58   thickness roof plate   hw.rp = 45   height roof plate

b_roof = 14560   width of roof (hall) $\gamma M0=1$ $\gamma M1=1$ $\gamma M2=1.25$

Stresses on roof purlin   qedg = max gravity load (case 1); qedu = max uplift load (case 2)

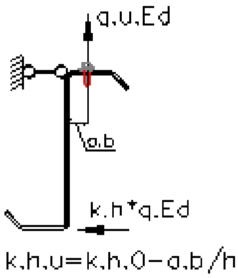
Situation	B1	B2
Section	Z 200x55x3	Z 200x55x3
cc_pur.[mm]	1100	1100
qedg [kN/m]	2.89	2.89
qedu [kN/m]	2.27	1.48
Ls [m]	5.11	6.90
Gable	Yes	No
NEd g/u [kN]	68.00/68.00	68.00/68.00

Sections' dimensions



	Z 200x55x1.5			Z 200x55x2			Z 200x55x3	
MyRk	= 7.58	kNm	MyRk	= 12.17	kNm	MyRk	= 19.59	kNm
VbhRk	= 18.42	kN	VbhRk	= 44.68	kN	VbhRk	= 102.52	kN
RwRk	= 16.47	kN	RwRk	= 26.27	kN	RwRk	= 51.90	kN
Wyeff	= 2.166 E04	mm3	Wyeff	= 3.476 E04	mm3	Wyeff	= 5.596 E04	mm3
Iy	= 2.879 E06	mm4	Iy	= 3.85 E06	mm4	Iy	= 5.725 E06	mm4
IT	= 3.525 E02	mm4	IT	= 8.544 E02	mm4	IT	= 2.882 E03	mm4
$I\omega$	= 2.257 E09	mm6	$I\omega$	= 3.08 E09	mm6	$I\omega$	= 4.769 E09	mm6

Lateral coefficient EN 1993-1-3: 10.1.4.1 (4): for uplift loading



$$kh0 = \frac{\mathrm{hc} * \mathrm{t} * \left(\mathrm{bc}^2 + 2 * \mathrm{bc} * \mathrm{cc} - 2 * \mathrm{cc}^2 \frac{\mathrm{bc}}{\mathrm{hc}}\right)}{4 * \mathrm{Iy}}$$

*k_hu = k_h0-ab/hc; q_hEdu = qEdu*k_hu*

Z 200x55x3

a_b	= 29.2	mm
k_h0	= 0.120	-
k_hu	= -0.028	-
q_hEdu	= -0.04	kN/m

Rotational restraint given by sheeting EN 1993-13 10.1.5

Rotional spring stiffness K, EN 1993-1-3: 10.1.5.2

Rotational stiffness corresponding to flexural stiffness of sheeting EN 1993-1-3: 10.1.5.2 (4):

tension in upper flange: I_effTP46_t = 125,401 mm4/m   C_DC_t = 6*E*I_effTP46_t/cc_pur

compression in upper flange: I_effTP46_c = 130,984 mm4/m   C_DC_c = 6*E*I_effTP46_c/cc_pur

Rotational stiffness of connection between sheeting and purlin ktg = (trp/(0.75*1mm))^1.5 = 0.687

for uplift load   pin in every trough C_100cc150_u = 2.6kN; pin in alternate trough C_100cc300_u = 1.7kN; b=55mm

C_DAcc150_u = C_100cc150_u*(b/1mm/100)^2*ktg = 0.5kN

C_DAcc300_u = C_100cc300_u*(b/1mm/100)^2*ktg = 0.4kN

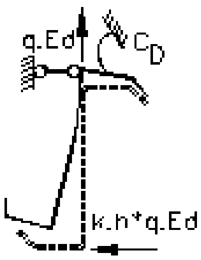
Rotational stiffness

C_Dcc150_u = [1/C_DAcc150_u+1/C_DC_t]^-1 <=> pin in every trough

C_Dcc300_u = [1/C_DAcc300_u+1/C_DC_t]^-1 <=> pin in alternate trough

For uplift load EN 1993-1-3: 10.1.5.1(4): *b_mod_u* = | *2*a_b+b* <=> *q_hEd_u < 0*
| *a_b* <=> *Else*

$$K\_Bu = \frac{\mathrm{E} * \mathrm{t}^3}{4 * (1 - \upsilon^2) * h^2 * (h + b\_mod\_u)}$$



Situation	B1	B2
b_mod_u [mm]	113.4	113.4
K_Bu [N/mm2]	0.124	0.124
1/K_Bu [mm2/N]	8.047	8.047
C_Dcc150_u [kN]	0.539	0.539
C_Dcc300_u [kN]	0.353	0.353

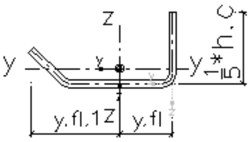
Lateral spring stiffnes per unit length:

pin in every trough: $$K\_ucc150 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc150\_u}}\right)^{-1}$$

pin in alternate trough: $$K\_ucc300 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc300\_u}}\right)^{-1}$$

Situation	B1	B2
K_ucc150 [N/mm2]	0.0121	0.0121
K_ucc300 [N/mm2]	0.0082	0.0082

<u>Gross properties of the free flange EN 1993-1-3: 10.1.4.1</u>



Z 200x55x3

t	= 3.0	mm
I_zf1	= 173,567	mm4
W_zf1	= 4,003.2	mm3
i_zf1	= 23.23	kmm

Lateral bending moment for free flanges in compression EN 1993-1-3:10.1.4.1 (5)-(7):

Coefficient R of the spring support EN 1993-1-3: 10.1.4.1 (7):

$$R\_rp = \frac{\mathrm{K\_ucc300} * \mathrm{L\_sp}^4}{\pi^4 * \mathrm{E} * \mathrm{I\_zf1}}$$

Situation	B1	B2
R_rp [-]	1.58	5.26

Correction factor K_R and initial moment M0_fzEd acc. to table 10.1 EN 1993-1-3: 10.1.4.1

M_0fzED = | 1|q_hEd_u|*L_s^2 /24 <=> no gable*
| 9|q_hEd_u|*L_s^2 /128 <=> gable*

*κ_Rm = | (1-0.0125*R_rp)/(1+0.198*R_rp) <=> no gable*
*| (1-0.0141*R_rp)/(1+0.416*R_rp) <=> gable*

*M_fzED = κ_Rm * M_0fzED*

Situation	B1(gable)	B2
M_0fzEd [Nm]	117.55	82.67
κ_Rm [-]	0.59	0.46
M_fzEd [Nm]	69.26	37.85

Stresses due to gravity load

<u>Combined bending moment and support reaction EN 1993-1-3: 6.1.11. The web rotation is prevented.</u>

Exertion = {[M_yEdγ_{M0} /MyRk + R_Ed*γ_{M1} /RwRk]}/1.25 < 1*

Situation	A1
Section	Z 200x55x3
Sup.width [m]	0.1
M_yEd [kNm]	12.14
M_yRk [kNm]	19.59
R_Ed [kN]	20.89
R_wRk [kN]	51.90
Exertion	**0.82**

<u>Combined bending moment and compression force: EN 1993-1-3:10.1.4.1 eqv. 10.3a</u>

Exertion = [M_yEd/Wyeff + N_Ed/Aeff]γ_{M1} /f_yb < 1*

Situation	A1-gravity
M_yEd [kNm]	12.14
Wyeff [mm3]	5.596 E04
N_Ed [kN]	68.0
Aeff [mm2]	716.9
Exertion	**0.89**

<u>Bending moment and shear force:</u>

*Relation = 2*V_Ed*γ_{M0} /VbhRk <= 1*

Situation	A1-gravity
V_Ed [kN]	10.51
VbhRk [kN]	102.52
Relation	**0.21**

If Relation <= 1 then combination of bending moment and shear force does not need to be checked.

Single profile at end of supporting profile:

*Exertion = [|M_Ls|/Wyeff + N_Ed/Aeff]**$\gamma M0$ */f_yb <= 1*

Situation	
Section	
M_Ls [kNm]	
Wyeff [mm3]	
Aeff [mm2]	
N_Ed [kN]	
Exertion	

Combined bending moment and compression force in span

*Exertion = [M_yEd/Wyeff + N_Ed/Aeff]**$\gamma M1$ */f_yb < 1*

Situation	B1	B2
Section	Z 200x55x3	Z 200x55x3
M_yEd [kNm]	5.09	6.30
Wyeff [mm3]	5.596 E04	5.596 E04
N_Ed [kN]	68.0	68.0
Aeff [mm2]	716.9	716.9
Exertion	**0.53**	**0.59**

Stresses due to Uplift load:

Non-dim. slenderness: $\lambda\_1 = \sqrt{\frac{E}{f\_yb}}$ $\lambda\_1 = 76.95$

Coefficients from table 10.2b:

gable <=> $\eta\_1 = 0.515$ $\eta\_2 = 1.260$ $\eta\_3 = 0.868$ $\eta\_4 = -0.242$

no gable<=> $\eta\_1 = 0.306$ $\eta\_2 = 0.232$ $\eta\_3 = 0.742$ $\eta\_4 = -0.279$

Buckling lenght for free flange in compression EN 1993-1-3: $L\_fz = \eta\_1 * L\_sp * (1 + \eta\_2 * R\_rp^{\eta\_3})^{\eta\_4}$

Relative slenderness for flexural buckling of free flange: $\lambda\_rfz = \frac{L\_fz}{i\_zf1 * \lambda\_1}$

Reduction factor for lateral torsional buckling: Acc. to 10.1.4.2 (1): use values given in EN 1993-1-1: 6.3.2.3

$\alpha\_LT = 0.34$ $\lambda\_rLT = 0.4$ $\beta = 0.75$ $\phi\_LT = 0.5 * [1+\alpha\_LT*(\lambda\_rfz-\lambda\_rLT)+\beta*\lambda^2]$

$$\chi\_LT = min\left(\frac{1}{\phi\_LT + \sqrt{\phi\_LT^2 + \beta * \lambda\_rfz^2}}, 1, \frac{1}{\lambda\_rfz^2}\right)$$

Reduction factor for flexural buckling: According to EN 1993-1-1: 6.3.1:

$$i\_y = \sqrt{\frac{I\_yrc}{Agrc}} \quad \lambda\_ryFB = \frac{L\_sp}{i\_y} * \sqrt{\frac{Aeff}{Agrc}} * \frac{1}{\lambda\_1} \quad N\_cry = \frac{\pi^2 * E * I\_yrc}{L\_sp^2}$$

$$\phi\_yFB = 0.5 * [1 + \alpha\_yFB * (\lambda\_ryFB - 0.2) + \lambda\_ryFB^2]$$

Reduction factor for span:

$$\chi\_yFB = \min\left(\frac{1}{\phi\_yFB + \sqrt{\phi\_yFB^2 - \lambda\_ryFB^2}}, 1\right)$$

Single profile in span uplift load:

$$Exertion = \left(\frac{1}{\chi\_LT} * \frac{M\_yEd}{W\_yeff} + \frac{1}{\min(\chi\_LT, \chi\_yFB)} * \frac{N\_Ed}{Aeff}\right) * \frac{\gamma M1}{f\_yb} + \frac{|M_{fzEd}| * \gamma M1}{W\_zf1 * f\_yb} < \quad 1.0$$

Situation	B1	B2
Section	Z 200x55x3	Z 200x55x3
L_sp [m]	5.11	6.90
M_yEd [kNm]	2.61	2.52
M_fzEd [Nm]	69.26	37.85
L_fz [m]	2.04	1.79
λ_rfz	1.14	1.00
Φ_LT	1.11	0.98
χ_LT	0.61	0.70
λ_ryFB	0.74	1.00
N_Ed [kN]	68.0	68.0
Φ_yFB	0.87	1.14
χ_yFB	0.76	0.59
Exertion	**0.71**	**0.67**

Check joints between purlins:

$$Exertion = \frac{|M\_S| * \gamma M2}{4 * F\_bRk * h/2} + \frac{N\_Ed * \gamma M2}{4 * F\_bRk} < 1.0$$

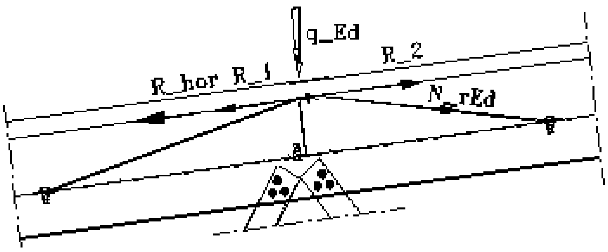
Situation	S1
Section	Z 200x55x3
F_bRk [kN]	37.8
M_S [kNm]	-1.84
NEd [kN]	68.0
Exertion	**0.71**

Bracing of Z-roof purlin

Type of hall = S-hall => broof_1 = broof = 14.56m Loadig width frame LW = 7m

Roofslope α_roof = 17° Rigde flashing = No

Max reaction Rg = 20.89kN Rg_slope = Rg*cos(α_roof) = 19.94kN Lateral coeff. k_hg = 0.120



from roof: R1 = sin(α_roof)*Rg
R1 = 6.23kN

from profile: R2 = Rg_slope*k_gh
R2 = 2.39kN

adjust value from profile: R2a = 0.5*R2 <=> α_roof > atan(1/10)
R2a = 1.20kN

Rhor = R1 - R2a
Rhor = 5.04kN

Force acting in the roof plane for one half of the building: P_roof = 0.5*broof1*Rhor/cc_pur
P_roof = 33.32kN

Shear resistance of screws:

– *Φ4.8:* F_4.8_sRd = 3.33kN/*γM2* = 2.7kN – *Φ6.3:* F_6.3_sRd = 6.24kN/*γM2* = 5.0kN

Overlap screw for use in ridge connection *(Φ4.8):*

d_oscr = 4.8mm t_ridgeplat = 0.5mm ultimate strength ridge plate: f_urp = 330N/mm2

α_oscr = 3.2*√(t_ridgeplat/d_oscr) = 1.03 F_oscr_bRd1 = (α_oscr*f_urp*d_oscr*t_ridgeplat)/*γM2* = 0.654kN

F_oscr_bRd = min(F_oscr_bRd1, F_4.8_sRd) = 0.654kN Bearing resistance plate to plate or shear capacity of screw

Plate screw for use in fastening of roofbrace to purlin *(Φ6.3):*

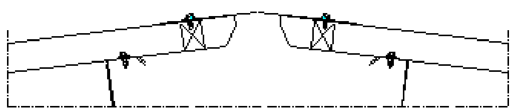
d_dscr = 6.3mm dw_dscr = 19.0mm thickness of z-support = 3.0mm thickness of plate t_bra = 0.58mm

α = 3.2*√(t_bra/d_dscr) = 0.974 α_1 = min(α, 2.1) = 0.974 α2 = (2.1-α_1)*(t_1/t_bra-1)/(2.5-1)+α_1 = 4.08

t_bra ≥ 1mm => α_q = 2.1 α_qq = (α_q-α_1)*(t_1/t_bra-1)/(2.5-1)+α_1 = 0.97

t_bra < t_1 < 2.5*t_bra => α_dscrEN = α_qq F_dscr_brdEN = (α_dscrEN*f_u*d_dscr*t_bra)/γM2 = 4.83kN FplabRd=

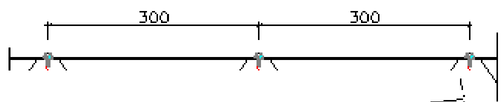
F_dscr_brd = min(F_6.3_sRd,F_dscr_brdEN,FplabRd)=4.83kN



Part of force in plane of roof which is taken by ridge flashing:

Ridge flashing = No => LW_ridge = 0m

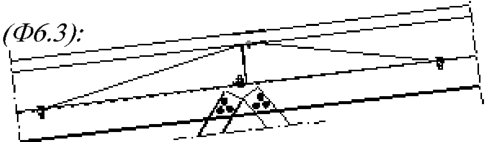
P_Rd_ridge = LW_ridge*F_oscr_bRd/0.3m = 0.00kN



Number of braces needed for each half of the roof: (FIRST CHOICE)

Design resistance for one brace 38x1.5 mm fastened with platescrew: *(Φ6.3):*

N_B = max{1, ceil[(|P_roof|-P_Rd_ridge)/F_dscr_brd]} **N_B = 7**



<u>If using C-profile:</u>

d_dscr = 6.3mm dw_dscr = 19.0mm thickness of c-profile = 2.0mm thickness of plate t_rp = 0.58mm

α = 3.2*√(t_rp/d_dscr) = 0.974 α_1 = min(α, 2.1) = 0.974 α2 = (2.1-α_1)*(t_1/t_rp-1)/(2.5-1)+α_1 = 2.793

t_rp < 1mm => α_q = α_1 α_qq = (α_q-α_1)*(t_1/t_rp-1)/(2.5-1)+α_1 = 0.974 futp=0.390kN/mm2

t_1 ≥ 2.5*t_rp and t_rp < 1mm => α_dscrEN = α_1 F_dscr_brdEN = (α_dscrEN*futp*d_dscr*t_rp)/γM2 = 1.12kN

F_dscr_brd = min(F_6.3_sRd, F_dscr_brdEN) = 1.12kN

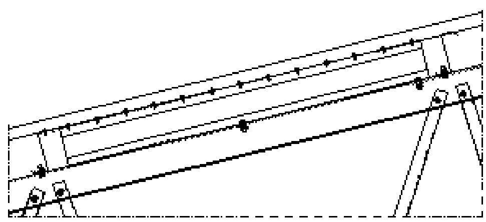
Design resistance for one C-profile (t≥2mm):

P_Rd_Cprof = 10*F_dscr_bRd = 11.2kN

Number of C-profiles needed for each half of the roof:

N_C = max{1, ceil[(|P_roof|-P_Rd_ridge)/P_Rd_Cprof]}

min N_C = 3, min total amount of screws 30



E.

Část konstrukce: RA1 Paždík stěna

Poznámky:
Paždík v podélné stěně. Zatěžovací šířka paždíku Lw=1.5m.

RA1 Paždík stěna

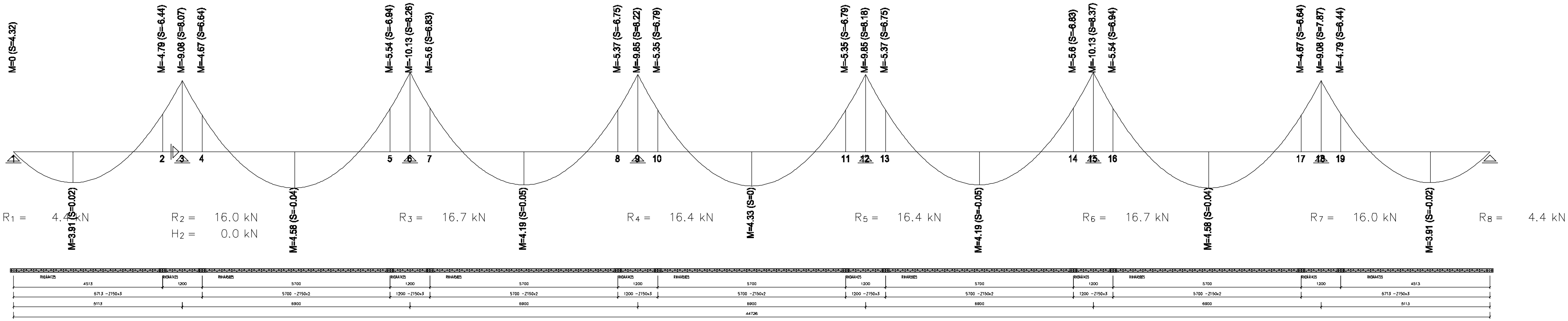
Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|E - Pazdiky|CZ1621 RA1.purx2/4/2022 14:23 by kamilpat
Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|E - Pazdiky|CZ1621 RA1.rtd (2/4/2022 time.2:23:20 PM)
Max V = -23.1mm, bar nr 16, comb.PurlinComb3(7)
av.weight = 5.18 [kg/m]

(104)=-0.451 x1.500
(301)=-1.136 x1.500
cc=1600mm

Loads applied to the purlin

Distributed loads

name	(no.)	f1 (↓)	f2 (↑)	val.[kPa]
roof		1.35	1.00	0
extra		1.35	0	0
snow		1.50	0	0
windSuc	(300)	0	1.50	0.750
windPre	(301)	1.50	0	-0.710
windIntS	(104)	1.50	0	-0.282
windIntP	(105)	0	1.50	0.188



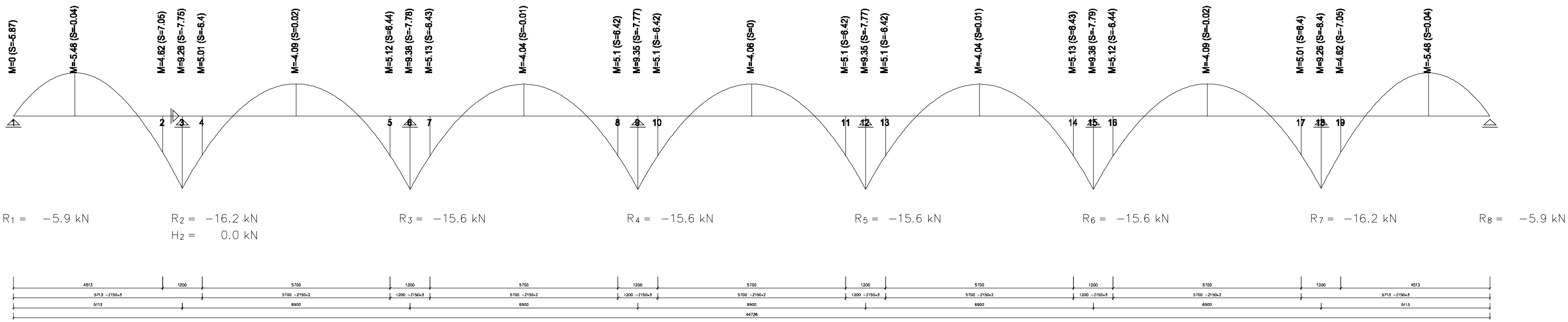
Trapezoidal loads

name	(no.)	f1 (↓)	f2 (↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
LA	(201)	0	1.50	0.370	0.370	3.10	0
RB	(202)	0	1.50	0.370	0.370	3.10	41.63

Combinations

No.	Name and definition of combination
1	PurlinComb1(5)
	1.5x(301+104)
2	PurlinComb2(6)
	1.5x(300+105+201+202)

(201)=0.592
(202)=0.592
(105)=0.301 x1.500
(300)=1.200 x1.500
x1.500



Geometry of the purlin.

Nodes		
No.	x [m]	type
1	0.00	sup.
2	4.51	-
3	5.11	sup.
4	5.71	-
5	11.41	-
6	12.01	sup.
7	12.61	-
8	18.31	-
9	18.91	sup.
10	19.51	-
11	25.21	-
12	25.81	sup.
13	26.41	-
14	32.11	-
15	32.71	sup.
16	33.31	-
17	39.01	-
18	39.61	sup.
19	40.21	-
20	44.73	sup.

Bars			
start	end	section	len.[m]
1	4	Z 150x46x3	5.71
4	5	Z 150x46x2	5.70
5	7	Z 150x46x3	1.20
7	8	Z 150x46x2	5.70
8	10	Z 150x46x3	1.20
10	11	Z 150x46x2	5.70
11	13	Z 150x46x3	1.20
13	14	Z 150x46x2	5.70
14	16	Z 150x46x3	1.20
16	17	Z 150x46x2	5.70
17	20	Z 150x46x3	5.71

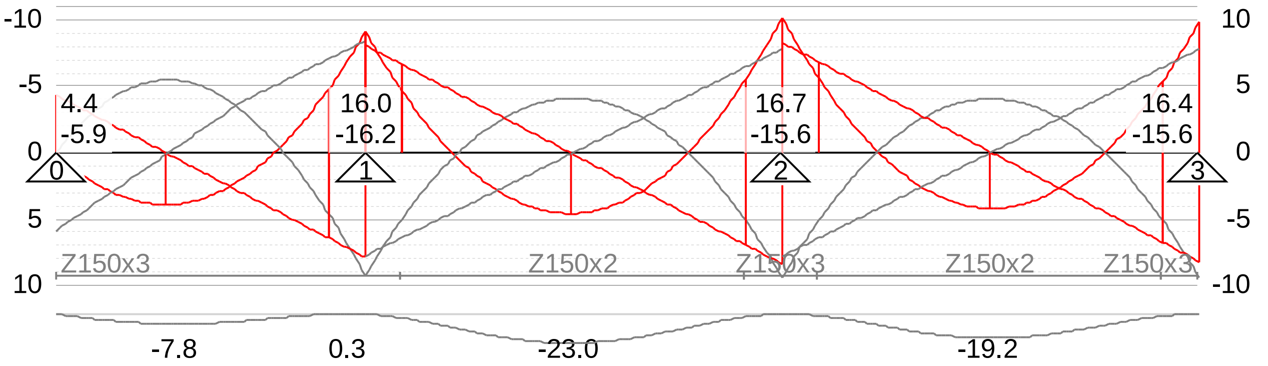
Profiles used in the construction.

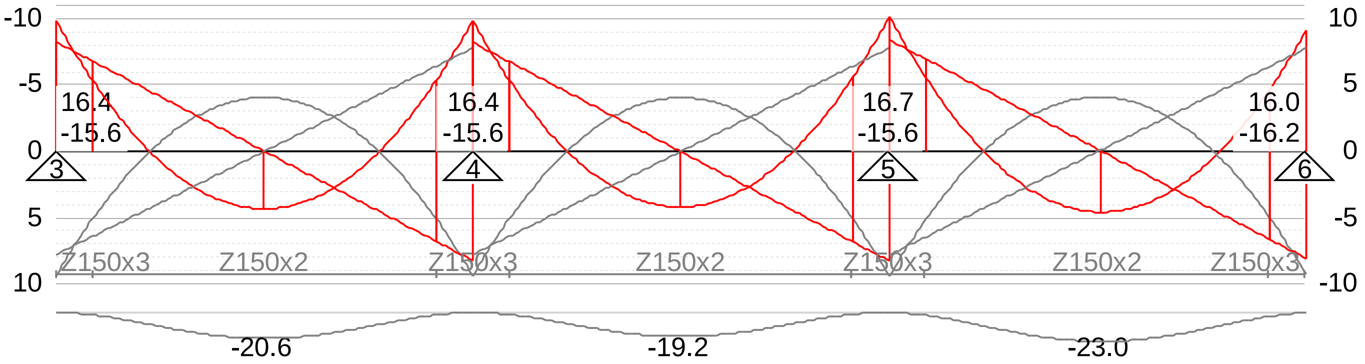
section	length [m]	weight [kg/m]	Iz [mm4]
Z 150x46x3	16.2	6.3	4.883 E17
Z 150x46x2	28.5	4.2	2.994 E17

av.weight = 4.97 [kg/m]

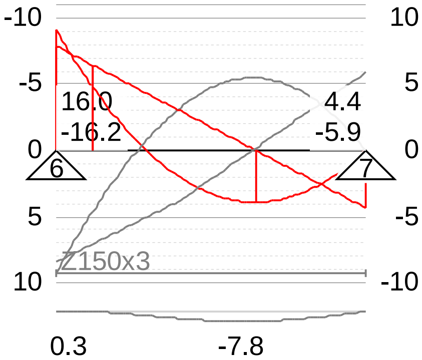
av.weight = 5.18 [kg/m] (inc.overlaps)

Bending moment, shear force, displacement and support reactions in purlin





dlim=6950/250=27mm
VYHOVUJE



dlim=5113/250=20mm
VYHOVUJE

Loads applied to the purlin

Distributed loads

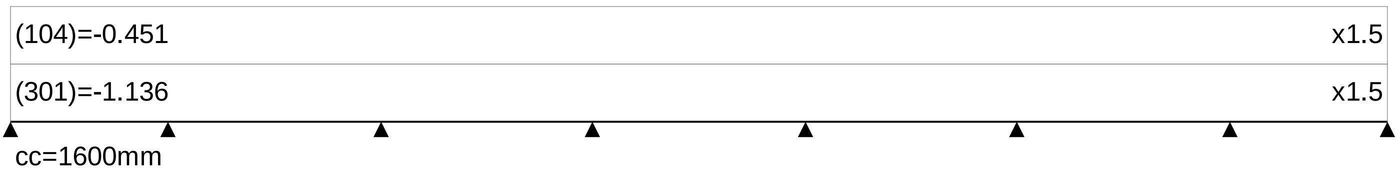
name (no.)	f1(↓)	f2(↑)	val.[kPa]
Self w. (101)	0	0	-
roof	1.35	1	0
extra	1.35	0	0
snow	1.5	0	0
windSuc (300)	0	1.5	0.75
windPre (301)	1.5	0	-0.71
windIntS (105)	1.5	0	-0.282
windIntP (106)	0	1.5	0.188

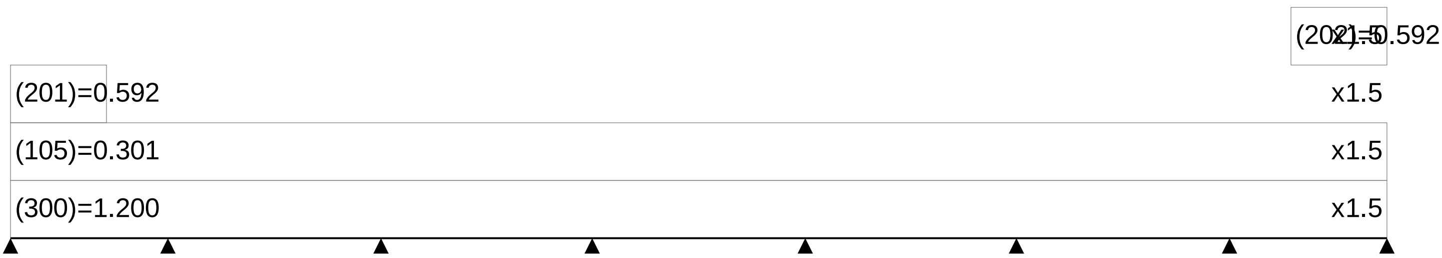
Trapezoidal loads

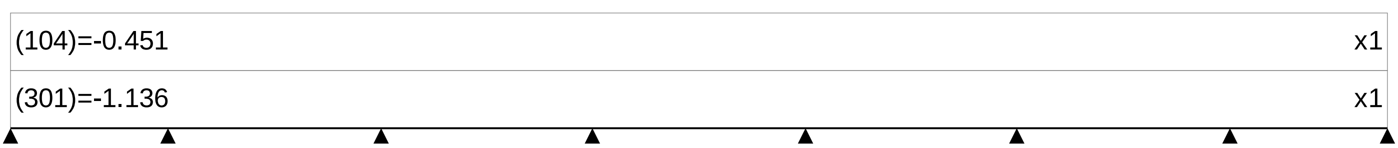
name (no.)	f1(↓)	f2(↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
LA (201)	0	1.5	0.370	0.370	3.10	0
RB (202)	0	1.5	0.370	0.370	3.10	41.63

Combinations

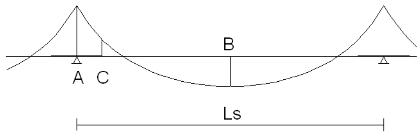
No.	Name and definition of combination
1	PurlinComb1(5) (ULS)
	1.5x(301+104)
2	PurlinComb2(6) (ULS)
	1.5x(300+105+201+202)
3	PurlinComb3(7) (SLS)
	1x(301+104)







Control of purlin Z-150 according to EN 1993-1-3



Global values: all measures in mm

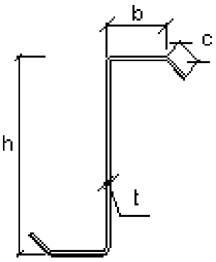
t.rp = 0.58 thickness wall plate hw.rp = 45 height wall plate

$\gamma M0=1$ $\gamma M1=1$ $\gamma M2=1.25$

Stresses on roof purlin qedg = max gravity load (case 1); qedu = max uplift load (case 2)

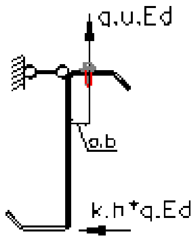
Situation	B1	B2
Section	Z 150x46x3	Z 150x46x2
cc_pur.[mm]	1600	1600
qedg [kN/m]	2.38	2.38
qedu [kN/m]	3.14	2.25
Ls [m]	5.11	6.90
Gable	Yes	No
NEd g/u [kN]	0.00/0.00	0.00/0.00

Sections' dimensions



	Z 150x46x2			**Z 150x46x3**	
MyRk	= 8.07	kNm	MyRk	= 12.79	kNm
VbhRk	= 47.57	kN	VbhRk	= 89.11	kN
RwRk	= 30.46	kN	RwRk	= 59.67/50.8	kN
Wyeff	= 2.305 E04	mm3	Wyeff	= 3.655 E04	mm3
Iy	= 1.799 E06	mm4	Iy	= 2.687 E06	mm4
IT	= 7.263 E02	mm4	IT	= 2.48 E03	mm4
$I\omega$	= 1.125 E09	mm6	$I\omega$	= 1.83 E09	mm6

Lateral coefficient EN 1993-1-3: 10.1.4.1 (4): for uplift loading



$$kh0 = \frac{\mathrm{hc} * \mathrm{t} * \left(\mathrm{bc}^2 + 2 * \mathrm{bc} * \mathrm{cc} - 2 * \mathrm{cc}^2 \frac{\mathrm{bc}}{\mathrm{hc}}\right)}{4 * \mathrm{Iy}}$$

*k_hu = k_h0-ab/hc; q_hEdu = qEdu*k_hu*

	Z 150x46x2			**Z 150x46x3**	
a_b	= 27.6	mm	a_b	= 29.2	mm
k_h0	= 0.139	-	k_h0	= 0.142	-
k_hu	= -0.048	-	k_hu	= -0.056	-
q_hEdu	= -0.11	kN/m	q_hEdu	= -0.18	kN/m

k.h.u=k.h.0-a.b/h

Rotational restraint given by sheeting EN 1993-13 10.1.5

Rotional spring stiffness K, EN 1993-1-3: 10.1.5.2

Rotational stiffness corresponding to flexural stiffness of sheeting EN 1993-1-3: 10.1.5.2 (4):

tension in upper flange: I_effTP46_t = 125,401 mm4/m C_DC_t = 6*E*I_effTP46_t/cc_pur

compression in upper flange: I_effTP46_c = 130,984 mm4/m C_DC_c = 6*E*I_effTP46_c/cc_pur

Rotational stiffness of connection between sheeting and purlin ktg = (trp/(0.75*1mm))^1.5 = 0.687

for uplift load pin in every trough C_100cc150_u = 2.6kN; pin in alternate trough C_100cc300_u = 1.7kN; b=46mm

C_DAcc150_u = C_100cc150_u*(b/1mm/100)^2*ktg = 0.4kN

C_DAcc300_u = C_100cc300_u*(b/1mm/100)^2*ktg = 0.2kN

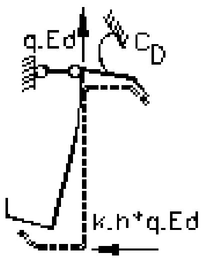
Rotational stiffness

C_Dcc150_u = [1/C_DAcc150_u+1/C_DC_t]^-1 <=> pin in every trough

C_Dcc300_u = [1/C_DAcc300_u+1/C_DC_t]^-1 <=> pin in alternate trough

For uplift load EN 1993-1-3: 10.1.5.1(4): *b_mod_u* = | *2*a_b+b* <=> *q_hEd_u* < *0*
| *a_b* <=> *Else*

$$K\_Bu = \frac{\mathrm{E} * \mathrm{t}^3}{4 * (1 - \upsilon^2) * h^2 * (h + b\_mod\_u)}$$



Situation	B1	B2
b_mod_u [mm]	104.4	101.2
K_Bu [N/mm2]	0.272	0.082
1/K_Bu [mm2/N]	3.674	12.245
C_Dcc150_u [kN]	0.377	0.377
C_Dcc300_u [kN]	0.247	0.247

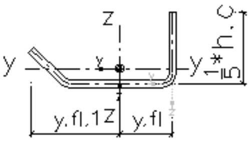
Lateral spring stiffnes per unit length:

pin in every trough: $$K\_ucc150 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc150\_u}}\right)^{-1}$$

pin in alternate trough: $$K\_ucc300 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc300\_u}}\right)^{-1}$$

Situation	B1	B2
K_ucc150 [N/mm2]	0.0158	0.0139
K_ucc300 [N/mm2]	0.0105	0.0097

Gross properties of the free flange EN 1993-1-3: 10.1.4.1



	Z 150x46x2			**Z 150x46x3**	
t	= 2.0	mm	t	= 3.0	mm
I_zf1	= 70,968	mm4	I_zf1	= 113,474	mm4
W_zf1	= 1,965.9	mm3	W_zf1	= 3,110.4	mm3
i_zf1	= 19.87	kmm	i_zf1	= 20.33	kmm

Lateral bending moment for free flanges in compression EN 1993-1-3:10.1.4.1 (5)-(7):

Coefficient R of the spring support EN 1993-1-3: 10.1.4.1 (7):

$$R\_rp = \frac{\mathrm{K\_ucc300} * \mathrm{L\_sp}^4}{\pi^4 * \mathrm{E} * \mathrm{I\_zf1}}$$

Situation	B1	B2
R_rp [-]	3.10	15.09

Correction factor K_R and initial moment M0_fzEd acc. to table 10.1 EN 1993-1-3: 10.1.4.1

M_0fzED = | 1|q_hEd_u|*L_s^2 /24 <=> no gable*
| 9|q_hEd_u|*L_s^2 /128 <=> gable*

*κ_Rm = | (1-0.0125*R_rp)/(1+0.198*R_rp) <=> no gable*
*| (1-0.0141*R_rp)/(1+0.416*R_rp) <=> gable*

*M_fzED = κ_Rm * M_0fzED*

Situation	B1(gable)	B2
M_0fzEd [Nm]	325.14	213.43
κ_Rm [-]	0.42	0.20
M_fzEd [Nm]	135.72	43.42

Stresses due to gravity load

Combined bending moment and support reaction EN 1993-1-3: 6.1.11. The web rotation is prevented.

Exertion = {[M_yEdγ_{M0} /MyRk + R_Ed*γ_{M1} /RwRk]}/1.25 < 1*

Situation	A1	A2
Section	Z 150x46x3	Z 150x46x3
Sup.width [m]	0.2	0.1
M_yEd [kNm]	10.12	10.12
M_yRk [kNm]	12.79	12.79
R_Ed [kN]	16.62	16.62
R_wRk [kN]	59.67	50.84
Exertion	**0.86**	**0.89**

Combined bending moment and compression force: EN 1993-1-3:10.1.4.1 eqv. 10.3a

Exertion = [M_yEd/Wyeff + N_Ed/Aeff]γ_{M1} /f_yb < 1*

Situation	A1-gravity	A2-gravity
M_yEd [kNm]	10.12	10.12
Wyeff [mm3]	3.655 E04	3.655 E04
N_Ed [kN]	0.0	0.0
Aeff [mm2]	700.6	700.6
Exertion	**0.79**	**0.79**

Bending moment and shear force:

*Relation = 2*V_Ed*γ_{M0} /VbhRk <= 1*

Situation	A1-uplift	A2-uplift
V_Ed [kN]	8.40	8.40
VbhRk [kN]	89.11	89.11
Relation	**0.19**	**0.19**

If Relation <= 1 then combination of bending moment and shear force does not need to be checked.

Single profile at end of supporting profile:

Exertion = [|M_Ls|/Wyeff + N_Ed/Aeff]$\gamma M0$ /f_yb <= 1*

Situation	C1-gravity
Section	Z 150x46x2
M_Ls [kNm]	5.60
Wyeff [mm3]	2.305 E04
Aeff [mm2]	375.2
N_Ed [kN]	0.00
Exertion	**0.69**

Combined bending moment and compression force in span

Exertion = [M_yEd/Wyeff + N_Ed/Aeff]$\gamma M1$ /f_yb < 1*

Situation	B1	B2
Section	Z 150x46x3	Z 150x46x2
M_yEd [kNm]	3.90	4.58
Wyeff [mm3]	3.655 E04	2.305 E04
N_Ed [kN]	0.0	0.0
Aeff [mm2]	700.6	375.2
Exertion	**0.31**	**0.57**

Stresses due to Uplift load:

Non-dim. slenderness: $\lambda\_1 = \sqrt{\frac{E}{f\_yb}}$ $\lambda\_1 = 76.95$

Coefficients from table 10.2b:

gable <=> $\eta\_1 = 0.515$ $\eta\_2 = 1.260$ $\eta\_3 = 0.868$ $\eta\_4 = -0.242$

no gable<=> $\eta\_1 = 0.306$ $\eta\_2 = 0.232$ $\eta\_3 = 0.742$ $\eta\_4 = -0.279$

Buckling lenght for free flange in compression EN 1993-1-3: $L\_fz = \eta\_1 * L\_sp * (1 + \eta\_2 * R\_rp^{\eta\_3})^{\eta\_4}$

Relative slenderness for flexural buckling of free flange: $\lambda\_rfz = \frac{L\_fz}{i\_zf1 * \lambda\_1}$

Reduction factor for lateral torsional buckling: Acc. to 10.1.4.2 (1): use values given in EN 1993-1-1: 6.3.2.3

$\alpha\_LT = 0.34$ $\lambda\_rLT = 0.4$ $\beta = 0.75$ $\phi\_LT = 0.5 * [1+\alpha\_LT*(\lambda\_rfz-\lambda\_rLT)+\beta*\lambda^2]$

$$\chi\_LT = min\left(\frac{1}{\phi\_LT + \sqrt{\phi\_LT^2 + \beta * \lambda\_rfz^2}}, 1, \frac{1}{\lambda\_rfz^2}\right)$$

Reduction factor for flexural buckling: According to EN 1993-1-1: 6.3.1:

$$i\_y = \sqrt{\frac{I\_yrc}{Agrc}} \quad \lambda\_ryFB = \frac{L\_sp}{i\_y} * \sqrt{\frac{Aeff}{Agrc}} * \frac{1}{\lambda\_1} \quad N\_cry = \frac{\pi^2 * E * I\_yrc}{L\_sp^2}$$

$$\phi\_yFB = 0.5 * [1 + \alpha\_yFB * (\lambda\_ryFB - 0.2) + \lambda\_ryFB^2]$$

Reduction factor for span:

$$\chi\_yFB = \min\left(\frac{1}{\phi\_yFB + \sqrt{\phi\_yFB^2 - \lambda\_ryFB^2}}, 1\right)$$

Single profile in span uplift load:

$$Exertion = \left(\frac{1}{\chi\_LT} * \frac{M\_yEd}{W\_yeff} + \frac{1}{\min(\chi\_LT, \chi\_yFB)} * \frac{N\_Ed}{Aeff}\right) * \frac{\gamma M1}{f\_yb} + \frac{|M_{fzEd}| * \gamma M1}{W\_zf1 * f\_yb} < \quad 1.0$$

Situation	B1	B2
Section	Z 150x46x3	Z 150x46x2
L_sp [m]	5.11	6.90
M_yEd [kNm]	5.48	4.09
M_fzEd [Nm]	135.72	43.42
L_fz [m]	1.84	1.59
λ_rfz	1.18	1.04
Φ_LT	1.15	1.02
χ_LT	0.59	0.67
λ_ryFB	1.07	1.29
N_Ed [kN]	0.0	0.0
Φ_yFB	1.22	1.52
χ_yFB	0.55	0.43
Exertion	**0.85**	**0.82**

Check joints between purlins:

$$Exertion = \frac{|M\_S| * \gamma M2}{4 * F\_bRk * h/2} + \frac{N\_Ed * \gamma M2}{4 * F\_bRk} < 1.0$$

Situation	S1	S2	S3
Section	Z 150x46x3	Z 150x46x2	Z 150x46x2
F_bRk [kN]	37.8	25.2	25.2
M_S [kNm]	-4.79	5.01	-5.60
NEd [kN]	0.0	0.0	0.0
Exertion	**0.53**	**0.83**	**0.93**

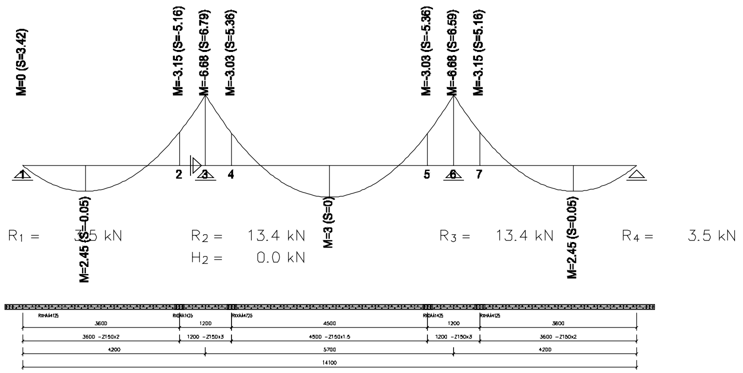
Část konstrukce: RA2 Paždík štít

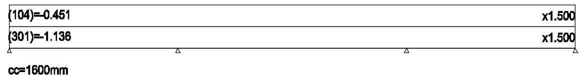
Poznámky:

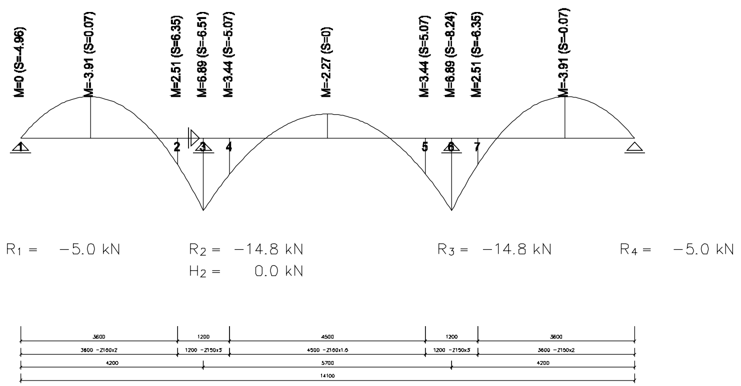
Paždík ve štítu. Zatěžovací šířka paždíku Lw=1.5m.

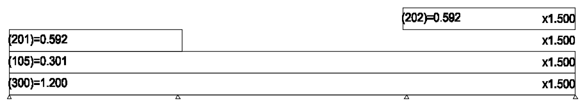
RA2 Paždík štít

Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|E - Pazdiky|CZ1621 RA2 štít.purx2/2/2022 12:30 by kamilpat
Q:|Praha|005_Vykresy|CZ16xx|CZ001621 Hala Zator|20 staticke vypocty|E - Pazdiky|CZ1621 RA2 štít.rtd (2/2/2022 time.12:30:01 PM)
Max V = -11.1mm, bar nr 4, comb.PurlinComb3(7)
av.weight = 4.42 [kg/m]









Loads applied to the purlin

Distributed loads

name	(no.)	f1 (↓)	f2 (↑)	val.[kPa]
roof		1.35	1.00	0
extra		1.35	0	0
snow		1.50	0	0
windSuc	(300)	0	1.50	0.750
windPre	(301)	1.50	0	-0.710
windIntS	(104)	1.50	0	-0.282
windIntP	(105)	0	1.50	0.188

Trapezoidal loads

name	(no.)	f1 (↓)	f2 (↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
LA	(201)	0	1.50	0.370	0.370	4.30	0
RB	(202)	0	1.50	0.370	0.370	4.30	9.80

Combinations

No.	Name and definition of combination
1	PurlinComb1(5)
	1.5x(301+104)
2	PurlinComb2(6)
	1.5x(300+105+201+202)

Geometry of the purlin.

Nodes		
No.	x [m]	type
1	0.00	sup.
2	3.60	-
3	4.20	sup.
4	4.80	-
5	9.30	-
6	9.90	sup.
7	10.50	-
8	14.10	sup.

Bars			
start	end	section	len.[m]
1	2	Z 150x46x2	3.60
2	4	Z 150x46x3	1.20
4	5	Z 150x46x1.5	4.50
5	7	Z 150x46x3	1.20
7	8	Z 150x46x2	3.60

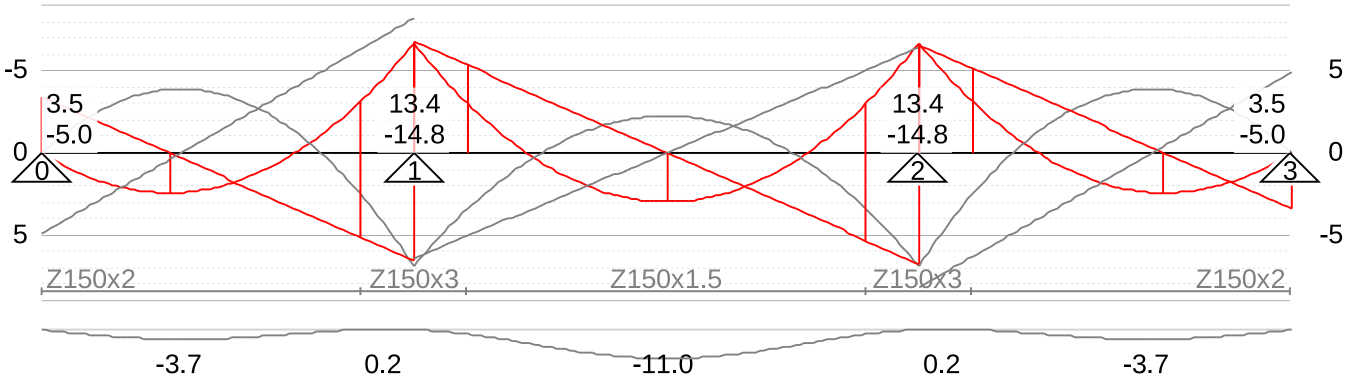
Profiles used in the construction.

section	length [m]	weight [kg/m]	Iz [mm4]
Z 150x46x2	7.2	4.2	2.994 E17
Z 150x46x3	2.4	6.3	4.883 E17
Z 150x46x1.5	4.5	3.2	2.198 E17

av.weight = 4.23 [kg/m]

av.weight = 4.42 [kg/m] (inc.overlaps)

Bending moment, shear force, displacement and support reactions in purlin



dlim=4200/200=16mm
VYHOVUJE

dlim=5700/250=22mm
VYHOVUJE

Loads applied to the purlin

Distributed loads

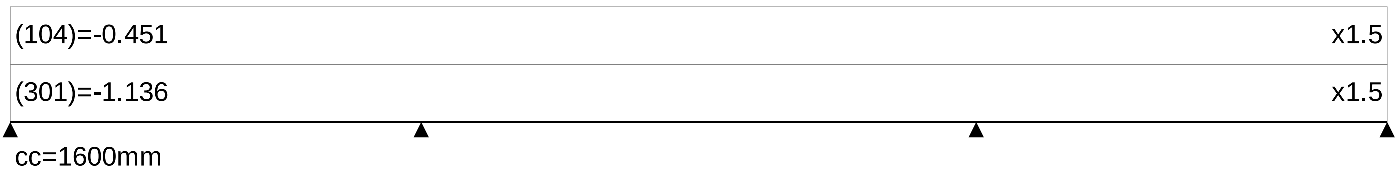
name (no.)	f1(↓)	f2(↑)	val.[kPa]
Self w. (101)	0	0	-
roof	1.35	1	0
extra	1.35	0	0
snow	1.5	0	0
windSuc (300)	0	1.5	0.75
windPre (301)	1.5	0	-0.71
windIntS (105)	1.5	0	-0.282
windIntP (106)	0	1.5	0.188

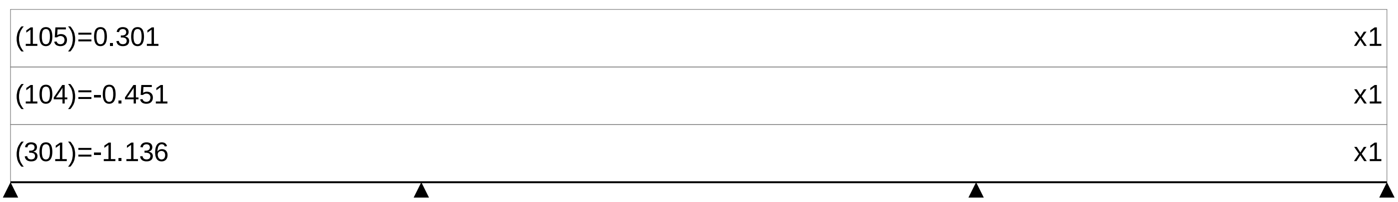
Trapezoidal loads

name (no.)	f1(↓)	f2(↑)	v1[kPa]	v2[kPa]	L[m]	x[m]
LA (201)	0	1.5	0.370	0.370	4.30	0
RB (202)	0	1.5	0.370	0.370	4.30	9.80

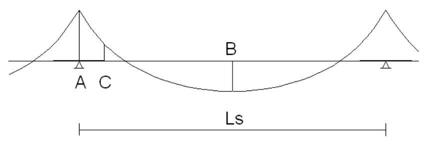
Combinations

No.	Name and definition of combination
1	PurlinComb1(5) (ULS)
	1.5x(301+104)
2	PurlinComb2(6) (ULS)
	1.5x(300+105+201+202)
3	PurlinComb3(7) (SLS)
	1x(301+104+105)





Control of purlin Z-150 according to EN 1993-1-3



Global values: all measures in mm

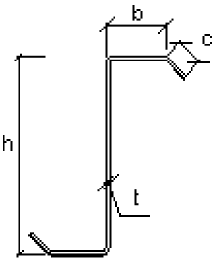
t.rp = 0.58 thickness wall plate hw.rp = 45 height wall plate

$\gamma M0=1$ $\gamma M1=1$ $\gamma M2=1.25$

Stresses on roof purlin qedg = max gravity load (case 1); qedu = max uplift load (case 2)

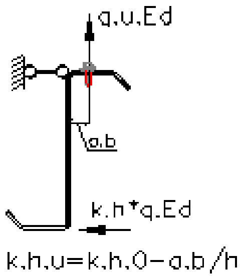
Situation	B1	B2
Section	Z 150x46x2	Z 150x46x1.5
cc_pur.[mm]	1600	1600
qedg [kN/m]	2.38	2.38
qedu [kN/m]	3.14	2.25
Ls [m]	4.20	5.70
Gable	Yes	No
NEd g/u [kN]	0.00/0.00	0.00/0.00

Sections' dimensions



	Z 150x46x1.5		Z 150x46x2		Z 150x46x3	
MyRk	= 5.76	kNm	= 8.07	kNm	= 12.79	kNm
VbhRk	= 26.76	kN	= 47.57	kN	= 89.11	kN
RwRk	= 19.17	kN	= 30.46	kN	= 59.67	kN
Wyeff	= 1.647 E04	mm3	= 2.305 E04	mm3	= 3.655 E04	mm3
Iy	= 1.357 E06	mm4	= 1.799 E06	mm4	= 2.687 E06	mm4
IT	= 3.057 E02	mm4	= 7.263 E02	mm4	= 2.48 E03	mm4
$I\omega$	= 8.281 E08	mm6	= 1.125 E09	mm6	= 1.83 E09	mm6

Lateral coefficient EN 1993-1-3: 10.1.4.1 (4): for uplift loading



$$kh0 = \frac{hc * t * \left(bc^2 + 2 * bc * cc - 2 * cc^2 \frac{bc}{hc}\right)}{4 * Iy}$$

*k_hu = k_h0-ab/hc; q_hEdu = qEdu*k_hu*

	Z 150x46x1.5		Z 150x46x2	
a_b	= 26.8	mm	= 27.6	mm
k_h0	= 0.137	-	= 0.139	-
k_hu	= -0.043	-	= -0.048	-
q_hEdu	= -0.10	kN/m	= -0.15	kN/m

Rotational restraint given by sheeting EN 1993-13 10.1.5

Rotional spring stiffness K, EN 1993-1-3: 10.1.5.2

Rotational stiffness corresponding to flexural stiffness of sheeting EN 1993-1-3: 10.1.5.2 (4):

tension in upper flange: I_effTP46_t = 125,401 mm4/m C_DC_t = 6*E*I_effTP46_t/cc_pur

compression in upper flange: I_effTP46_c = 130,984 mm4/m C_DC_c = 6*E*I_effTP46_c/cc_pur

Rotational stiffness of connection between sheeting and purlin ktg = (trp/(0.75*1mm))^1.5 = 0.687

for uplift load pin in every trough C_100cc150_u = 2.6kN; pin in alternate trough C_100cc300_u = 1.7kN; b=46mm

C_DAcc150_u = C_100cc150_u*(b/1mm/100)^2*ktg = 0.4kN

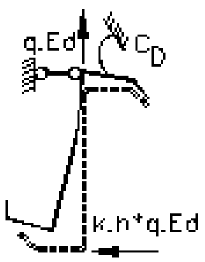
C_DAcc300_u = C_100cc300_u*(b/1mm/100)^2*ktg = 0.2kN

Rotational stiffness

C_Dcc150_u = [1/C_DAcc150_u+1/C_DC_t]^-1 <=> pin in every trough

C_Dcc300_u = [1/C_DAcc300_u+1/C_DC_t]^-1 <=> pin in alternate trough

For uplift load EN 1993-1-3: 10.1.5.1(4): $b\_mod\_u = \begin{cases} 2*a\_b+b & \Leftrightarrow q\_hEd\_u < 0 \\ a\_b & \Leftrightarrow Else \end{cases}$ $K\_Bu = \frac{E * t^3}{4 * (1-\upsilon^2) * h^2 * (h + b\_mod\_u)}$



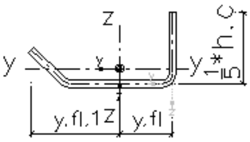
Situation	B1	B2
b_mod_u [mm]	101.2	99.6
K_Bu [N/mm2]	0.082	0.035
1/K_Bu [mm2/N]	12.245	28.840
C_Dcc150_u [kN]	0.377	0.377
C_Dcc300_u [kN]	0.247	0.247

Lateral spring stiffnes per unit length:

pin in every trough: $K\_ucc150 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc150\_u}}\right)^{-1}$ pin in alternate trough: $K\_ucc300 = \left(\frac{1}{\mathrm{K\_Bu}} + \frac{\mathrm{h}^2}{\mathrm{c\_Dcc300\_u}}\right)^{-1}$

Situation	B1	B2
K_ucc150 [N/mm2]	0.0139	0.0113
K_ucc300 [N/mm2]	0.0097	0.0083

Gross properties of the free flange EN 1993-1-3: 10.1.4.1



Z 150x46x1.5			**Z 150x46x2**		
t	= 1.5	mm	t	= 2.0	mm
I_zf1	= 52,456	mm4	I_zf1	= 70,968	mm4
W_zf1	= 1,454.6	mm3	W_zf1	= 1,965.9	mm3
i_zf1	= 19.76	kmm	i_zf1	= 19.87	kmm

Lateral bending moment for free flanges in compression EN 1993-1-3:10.1.4.1 (5)-(7):

Coefficient R of the spring support EN 1993-1-3: 10.1.4.1 (7):

$$R\_rp = \frac{\mathrm{K\_ucc300} * \mathrm{L\_sp}^4}{\pi^4 * \mathrm{E} * \mathrm{I\_zf1}}$$

Situation	B1	B2
R_rp [-]	2.07	8.19

Correction factor K_R and initial moment M0_fzEd acc. to table 10.1 EN 1993-1-3: 10.1.4.1

M_0fzED = | *1*|q_hEd_u|*L_s^2 /24 <=> no gable* | *κ_Rm =* | *(1-0.0125*R_rp)/(1+0.198*R_rp) <=> no gable*

| *9*|q_hEd_u|*L_s^2 /128 <=> gable* | | *(1-0.0141*R_rp)/(1+0.416*R_rp) <=> gable*

*M_fzED = κ_Rm * M_0fzED*

Situation	B1(gable)	B2
M_0fzEd [Nm]	186.08	131.26
κ_Rm [-]	0.52	0.34
M_fzEd [Nm]	97.02	44.93

Stresses due to gravity load

Combined bending moment and support reaction EN 1993-1-3: 6.1.11. The web rotation is prevented.

Exertion = {[M_yEd$\gamma M0$ /MyRk + R_Ed*$\gamma M1$ /RwRk]}/1.25 < 1*

Situation	A1
Section	Z 150x46x3
Sup.width [m]	0.2
M_yEd [kNm]	6.67
M_yRk [kNm]	12.79
R_Ed [kN]	13.37
R_wRk [kN]	59.67
Exertion	**0.60**

Combined bending moment and compression force: EN 1993-1-3:10.1.4.1 eqv. 10.3a

Exertion = [M_yEd/Wyeff + N_Ed/Aeff]$\gamma M1$ /f_yb < 1*

Situation	A1-uplift
M_yEd [kNm]	6.88
Wyeff [mm3]	3.655 E04
N_Ed [kN]	0.0
Aeff [mm2]	700.6
Exertion	**0.54**

Bending moment and shear force:

*Relation = 2*V_Ed*$\gamma M0$ /VbhRk <= 1*

Situation	A1-uplift
V_Ed [kN]	8.23
VbhRk [kN]	89.11
Relation	**0.18**

If Relation <= 1 then combination of bending moment and shear force does not need to be checked.

Single profile at end of supporting profile:

Exertion = [|M_Ls|/Wyeff + N_Ed/Aeff]$\gamma M0$ /f_yb <= 1*

Situation	C1-gravity	C2-uplift
Section	Z 150x46x2	Z 150x46x1.5
M_Ls [kNm]	3.15	3.43
Wyeff [mm3]	2.305 E04	1.647 E04
Aeff [mm2]	375.2	239.7
N_Ed [kN]	0.00	0.00
Exertion	**0.39**	**0.60**

Combined bending moment and compression force in span

Exertion = [M_yEd/Wyeff + N_Ed/Aeff]$\gamma M1$ /f_yb < 1*

Situation	B1	B2
Section	Z 150x46x2	Z 150x46x1.5
M_yEd [kNm]	2.44	3.00
Wyeff [mm3]	2.305 E04	1.647 E04
N_Ed [kN]	0.0	0.0
Aeff [mm2]	375.2	239.7
Exertion	**0.30**	**0.52**

Stresses due to Uplift load:

Non-dim. slenderness: $\lambda\_1 = \sqrt{\frac{E}{f\_yb}}$ $\lambda\_1 = 76.95$

Coefficients from table 10.2b:

gable <=> $\eta\_1 = 0.515$ $\eta\_2 = 1.260$ $\eta\_3 = 0.868$ $\eta\_4 = -0.242$

no gable<=> $\eta\_1 = 0.306$ $\eta\_2 = 0.232$ $\eta\_3 = 0.742$ $\eta\_4 = -0.279$

Buckling lenght for free flange in compression EN 1993-1-3: $L\_fz = \eta\_1 * L\_sp * (1 + \eta\_2 * R\_rp^{\eta\_3})^{\eta\_4}$

Relative slenderness for flexural buckling of free flange: $\lambda\_rfz = \frac{L\_fz}{i\_zf1 * \lambda\_1}$

Reduction factor for lateral torsional buckling: Acc. to 10.1.4.2 (1): use values given in EN 1993-1-1: 6.3.2.3

$\alpha\_LT = 0.34$ $\lambda\_rLT = 0.4$ $\beta = 0.75$ $\phi\_LT = 0.5 * [1+\alpha\_LT*(\lambda\_rfz-\lambda\_rLT)+\beta*\lambda^2]$

$$\chi\_LT = min\left(\frac{1}{\phi\_LT + \sqrt{\phi\_LT^2 + \beta * \lambda\_rfz^2}}, 1, \frac{1}{\lambda\_rfz^2}\right)$$

Reduction factor for flexural buckling: According to EN 1993-1-1: 6.3.1:

$$i\_y = \sqrt{\frac{I\_yrc}{Agrc}} \quad \lambda\_ryFB = \frac{L\_sp}{i\_y} * \sqrt{\frac{Aeff}{Agrc}} * \frac{1}{\lambda\_1} \quad N\_cry = \frac{\pi^2 * E * I\_yrc}{L\_sp^2}$$

$$\phi\_yFB = 0.5 * [1 + \alpha\_yFB * (\lambda\_ryFB - 0.2) + \lambda\_ryFB^2]$$

Reduction factor for span:

$$\chi\_yFB = \min\left(\frac{1}{\phi\_yFB + \sqrt{\phi\_yFB^2 - \lambda\_ryFB^2}}, 1\right)$$

Single profile in span uplift load:

$$Exertion = \left(\frac{1}{\chi\_LT} * \frac{M\_yEd}{W\_yeff} + \frac{1}{\min(\chi\_LT, \chi\_yFB)} * \frac{N\_Ed}{Aeff}\right) * \frac{\gamma M1}{f\_yb} + \frac{|M_{fzEd}| * \gamma M1}{W\_zf1 * f\_yb} < \quad 1.0$$

Situation	B1	B2
Section	Z 150x46x2	Z 150x46x1.5
L_sp [m]	4.20	5.70
M_yEd [kNm]	3.91	2.27
M_fzEd [Nm]	97.02	44.93
L_fz [m]	1.61	1.42
λ_rfz	1.05	0.93
Φ_LT	1.03	0.92
χ_LT	0.67	0.74
λ_ryFB	0.79	0.98
N_Ed [kN]	0.0	0.0
Φ_yFB	0.91	1.12
χ_yFB	0.73	0.61
Exertion	**0.87**	**0.62**

Check joints between purlins:

$$Exertion = \frac{|M\_S| * \gamma M2}{4 * F\_bRk * h/2} + \frac{N\_Ed * \gamma M2}{4 * F\_bRk} < 1.0$$

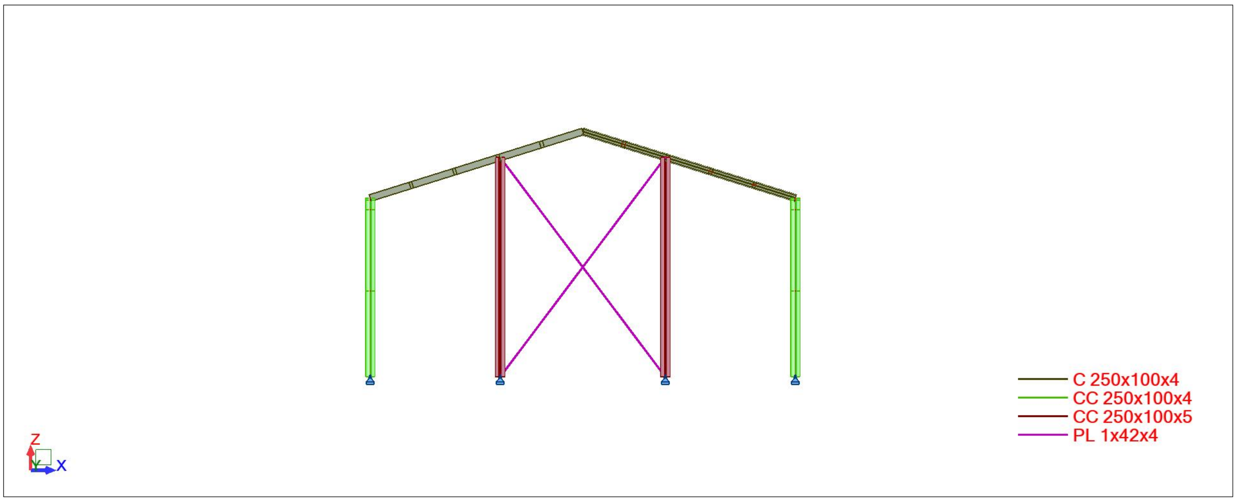
Situation	S1	S2
Section	Z 150x46x2	Z 150x46x1.5
F_bRk [kN]	25.2	18.9
M_S [kNm]	-3.15	3.43
NEd [kN]	0.0	0.0
Exertion	**0.52**	**0.76**

G.

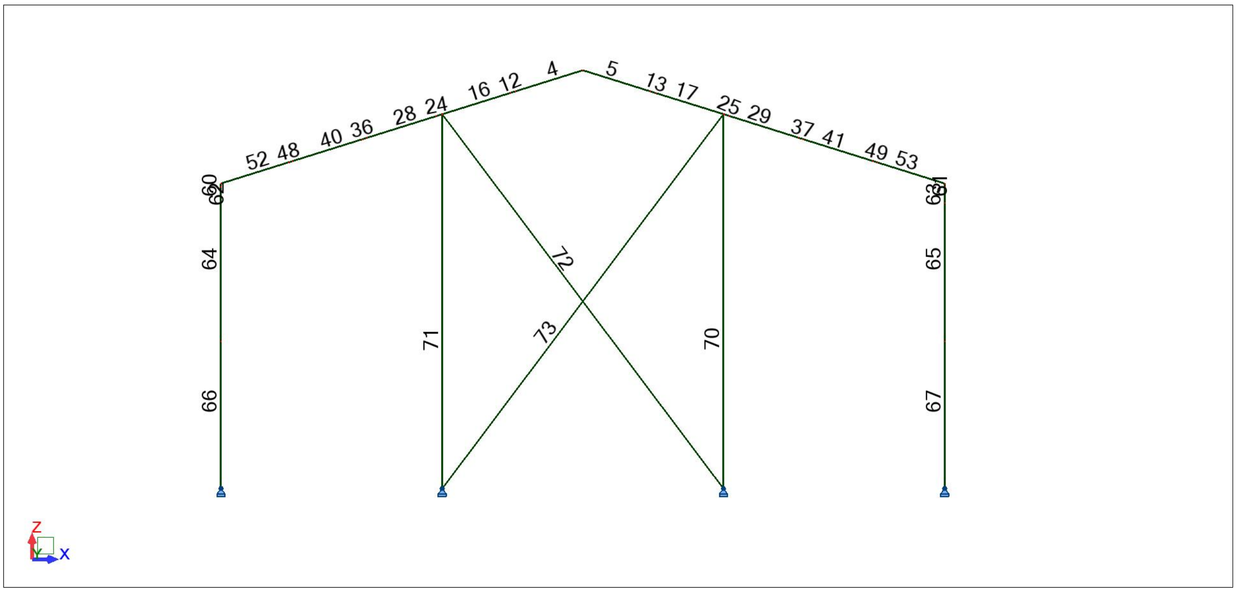
Část konstrukce: GA1 Štít vnější

Poznámky:
Zatěžovací šířka štítu Lw=2m.

View - Cases: 101 (Self weight frame)



View - Cases: 101 (Self weight frame) 1



Combinations

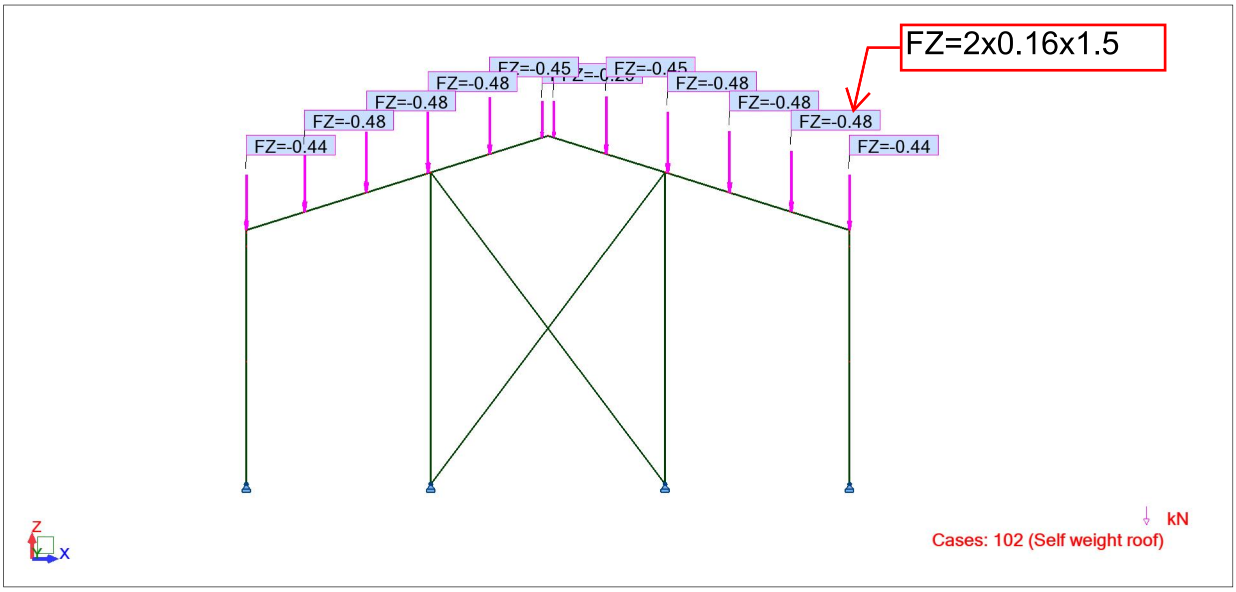
- Cases: 10 11 20 30to32 40to42 50 80to84 90to93

Combinations	Name	Analysis type	Combi nation	Case nature
10	Snow	Nonlin. Combinat	ULS	snow
11	Snow 0.5 right	Nonlin. Combinat	ULS	snow
20	Wind Cpe max	Nonlin. Combinat	ULS	wind
30	Snow + reduced	Nonlin. Combinat	ULS	snow
31	Snow 0.5 left + r	Nonlin. Combinat	ULS	snow
32	Snow 0.5 right +	Nonlin. Combinat	ULS	snow
40	Wind Cpe min +	Nonlin. Combinat	ULS	snow

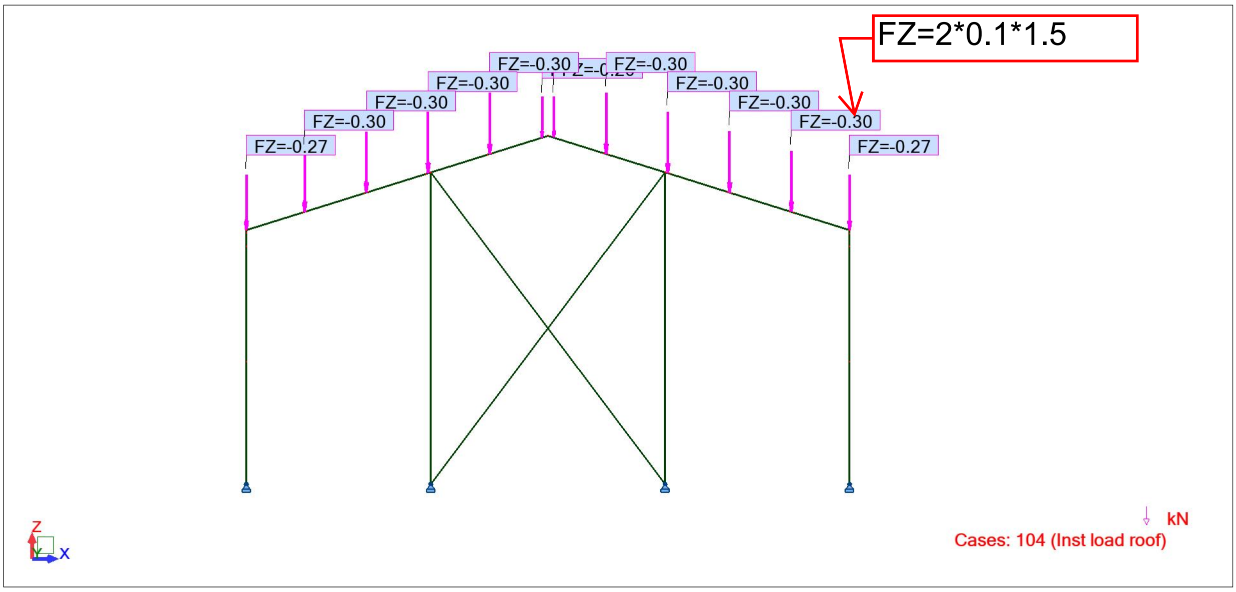
Combinations	Name	Analysis type	Combination	Case nature
41	Wind Cpe min +	Nonlin. Combinat	ULS	snow
42	Wind Cpe min +	Nonlin. Combinat	ULS	snow
50	Wind Cpe max fr	Nonlin. Combinat	ULS	snow
80	R15 Snow	Nonlin. Combinat	ULS	snow
81	R15 Snow 0.5 rig	Nonlin. Combinat	ULS	snow
82	R15 Wind Cpe m	Nonlin. Combinat	ULS	wind
83	R15 Wind Cpe m	Nonlin. Combinat	ULS	snow
84	R15 Wind Cpe m	Nonlin. Combinat	ULS	snow
90	SLS Characterist	Nonlin. Combinat	ULS	snow
91	SLS Characterist	Nonlin. Combinat	ULS	snow
92	SLS Characterist	Nonlin. Combinat	ULS	snow
93	SLS Characterist	Nonlin. Combinat	ULS	snow

Combinations	Definition
10	(101+102+104)*1.35+(201+202)*1.50
11	(101+102+104)*1.35+201*1.50+202*0.75
20	(101+102)*1.00+(300+321)*1.50
30	(101+102+104)*1.35+(201+202)*1.50+(301+320)*
31	(101+102+104)*1.35+201*0.75+202*1.50+(301+3
32	(101+102+104)*1.35+201*1.50+202*0.75+(301+3
40	(101+102+104)*1.35+(201+202)*0.75+(301+320)*
41	(101+102+104)*1.35+201*0.38+202*0.75+(301+3
42	(101+102+104)*1.35+201*0.75+202*0.38+(301+3
50	(101+102)*1.00+(310+321)*1.50
80	(101+102+104)*1.00+(201+202)*0.20
81	(101+102+104)*1.00+201*0.20+202*0.10
82	(101+102)*1.00+(300+321)*0.20
83	(101+102+104)*1.00+(301+320)*0.20
84	(101+102)*1.00+(310+321)*0.20
90	(101+102+104+201+202)*1.00
91	(101+102+300+321)*1.00
92	(101+102+201+202)*1.00+(301+320)*0.60
93	(101+102+301+320)*1.00+(201+202)*0.50

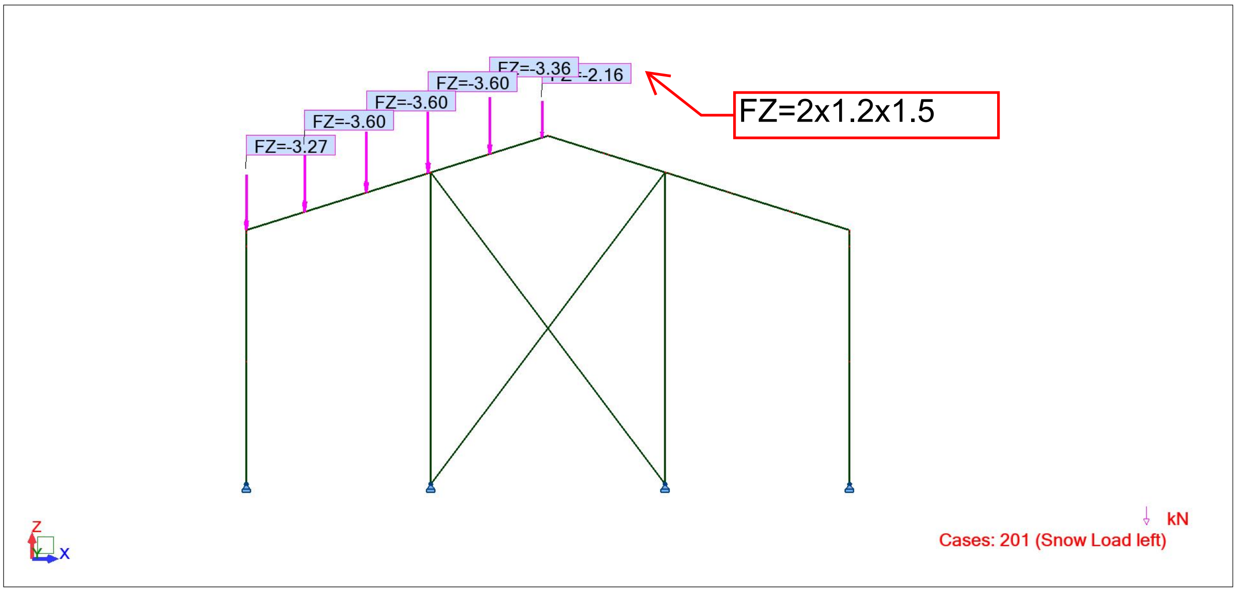
View - Cases: 102 (Self weight roof)



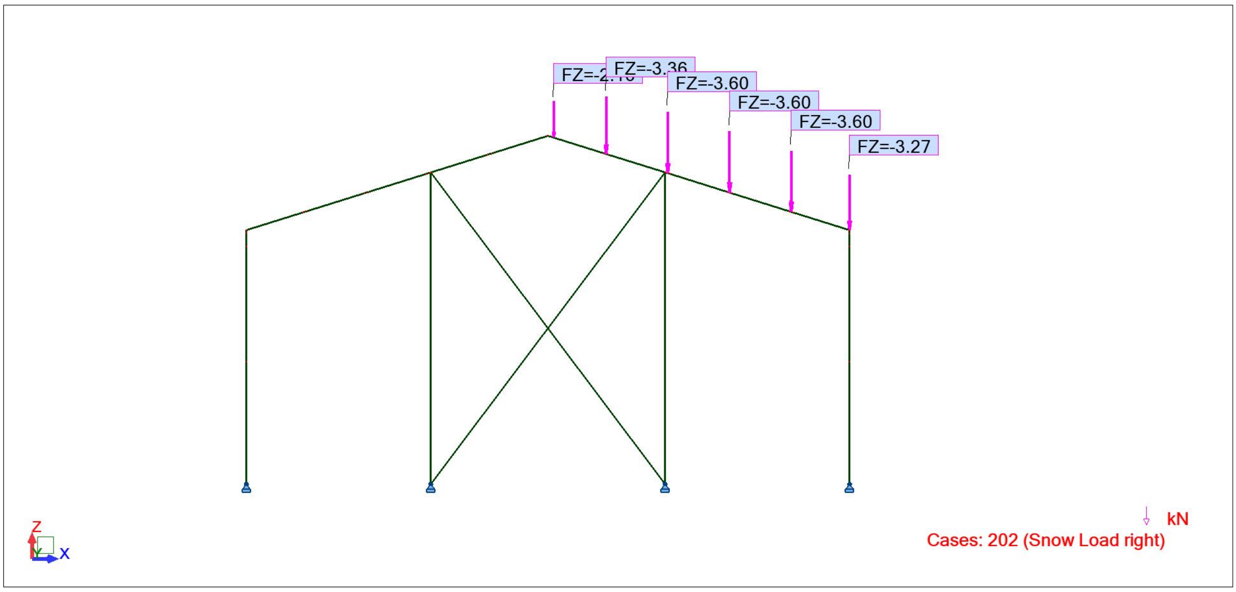
View - Cases: 104 (Inst load roof)



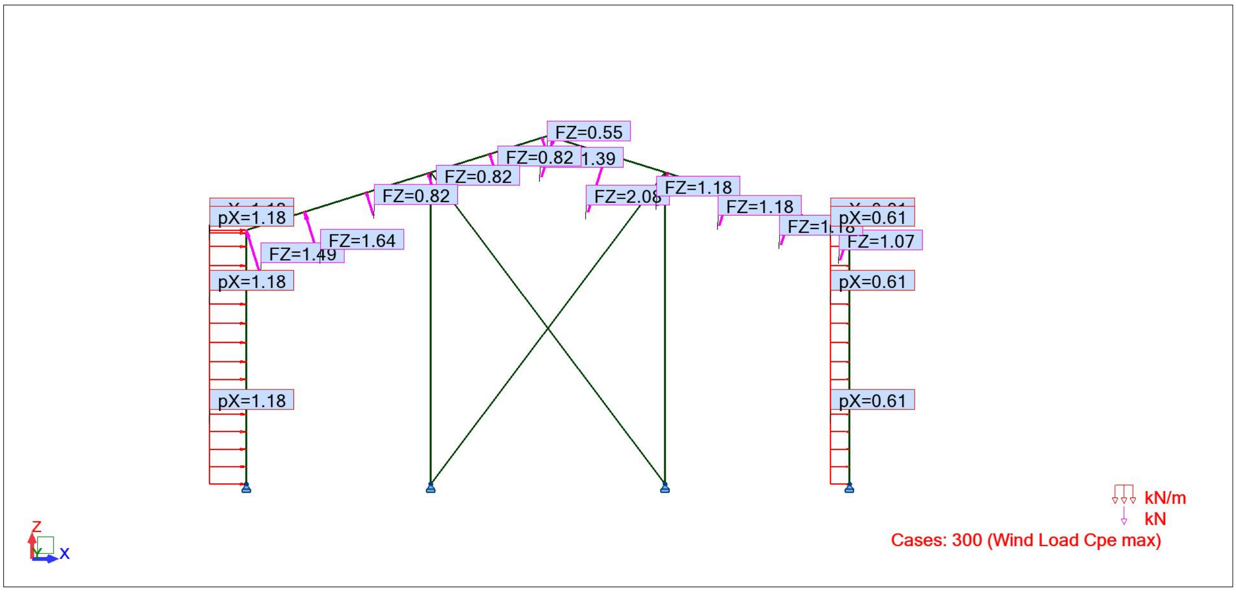
View - Cases: 201 (Snow Load left)



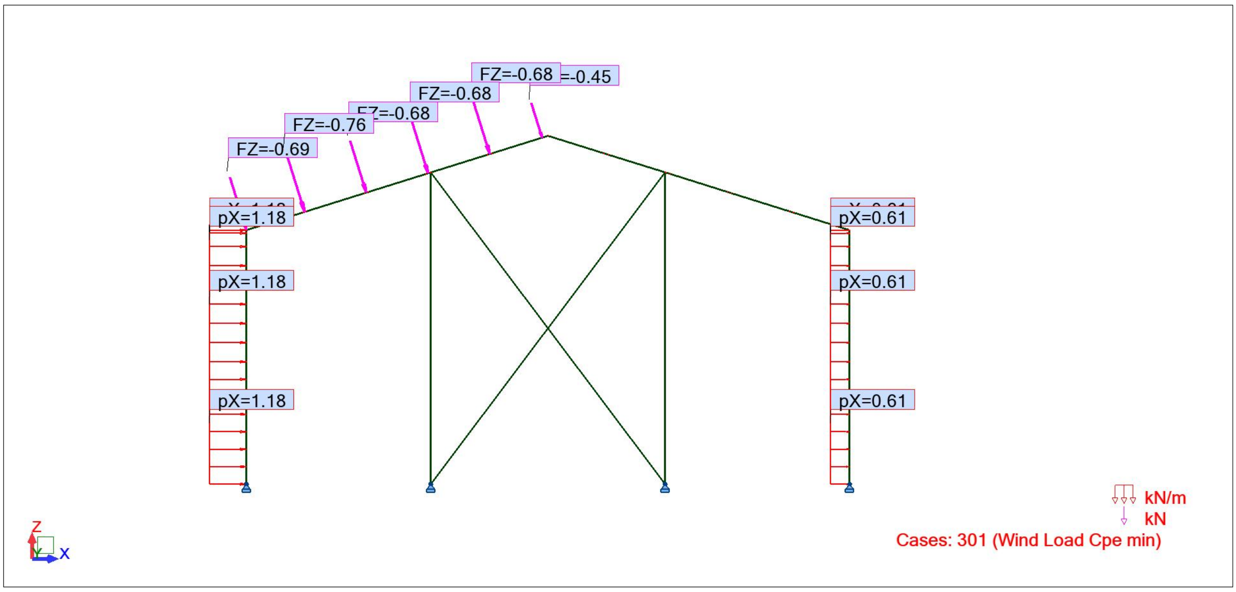
View - Cases: 202 (Snow Load right)



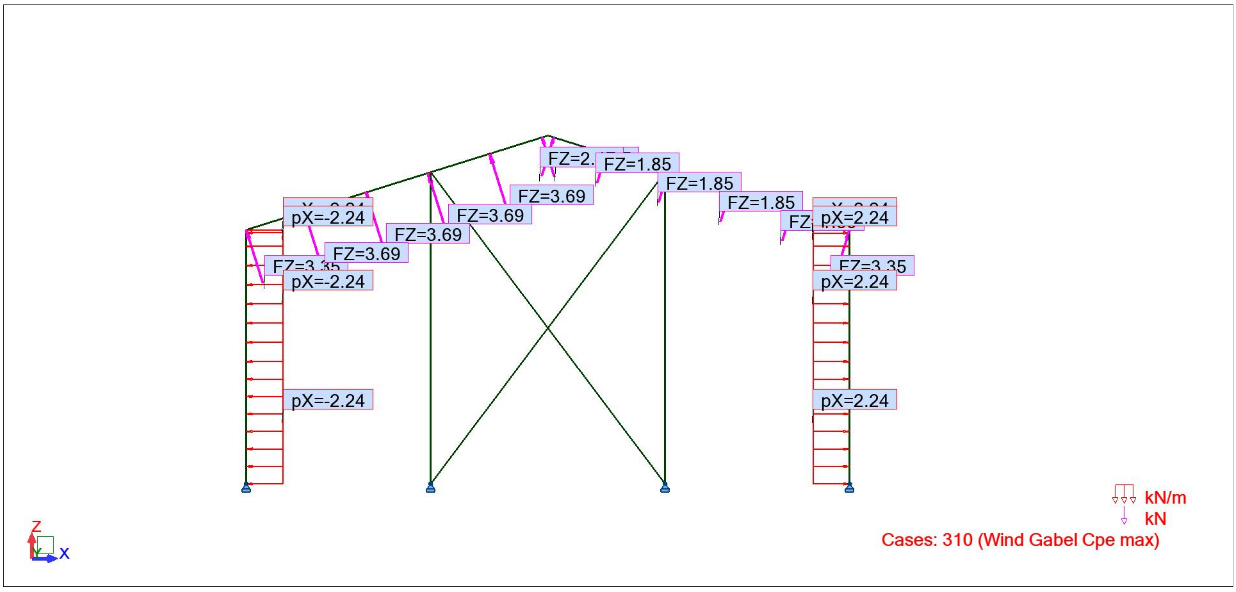
View - Cases: 300 (Wind Load Cpe max)



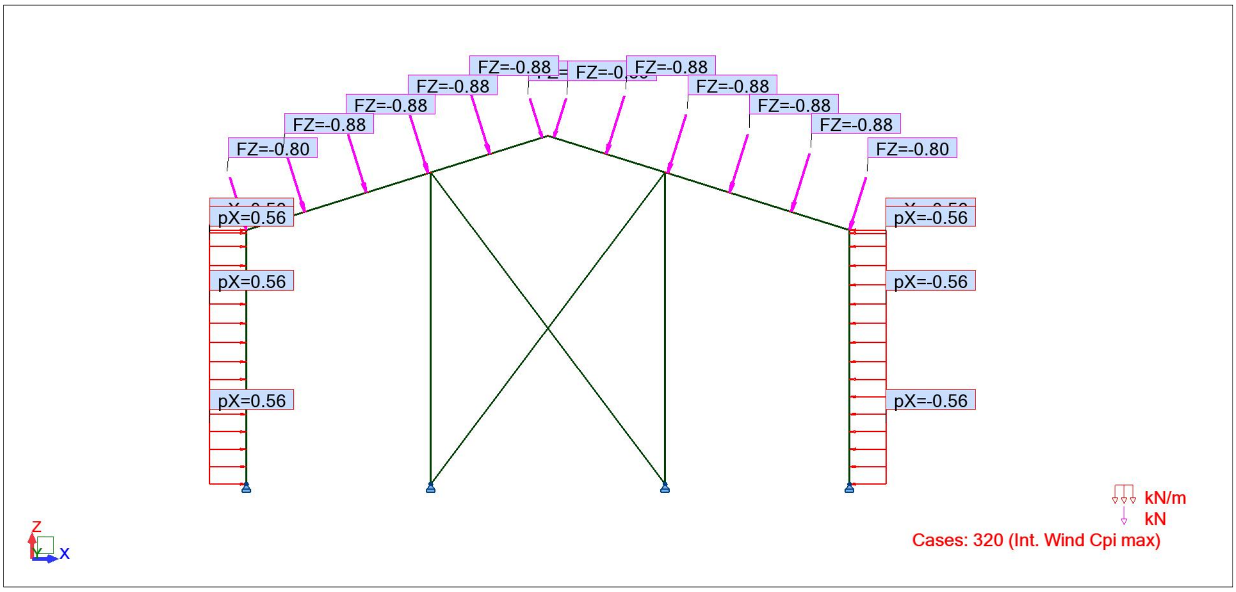
View - Cases: 301 (Wind Load Cpe min)



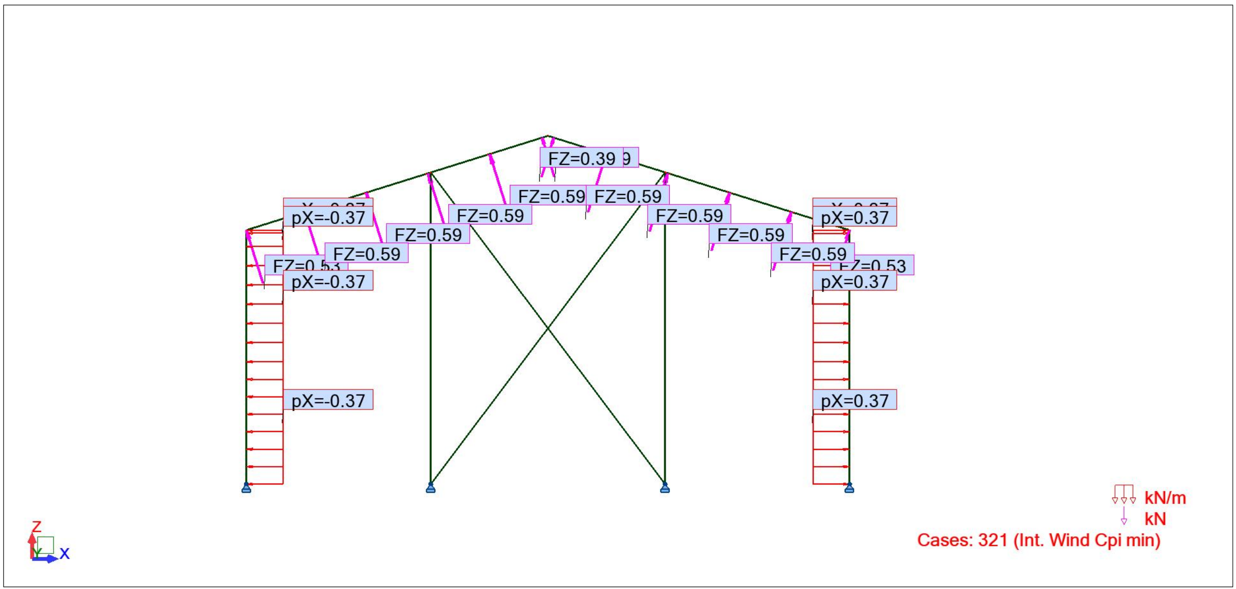
View - Cases: 310 (Wind Gabel Cpe max)



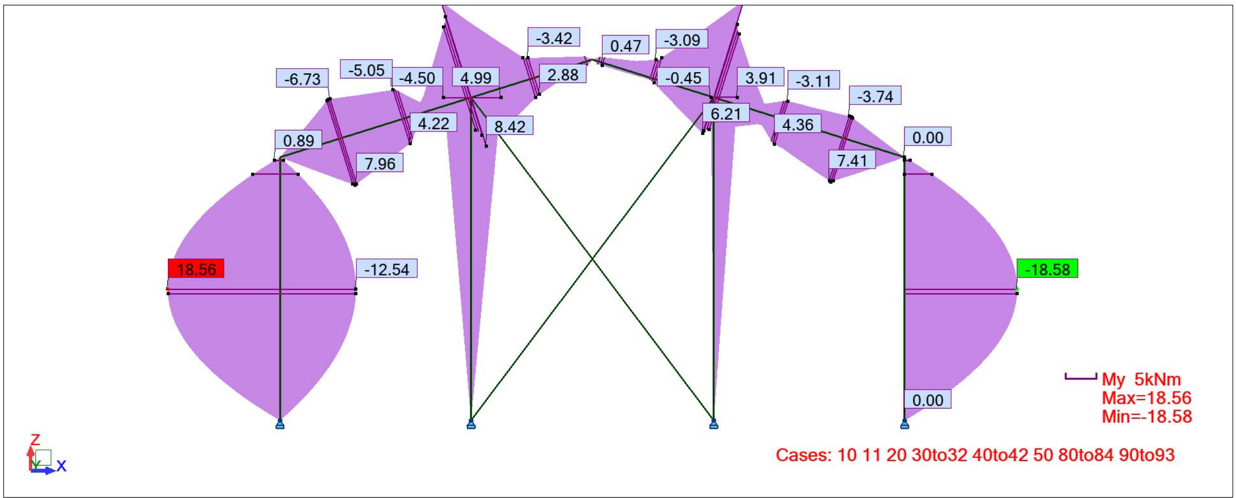
View - Cases: 320 (Int. Wind Cpi max)



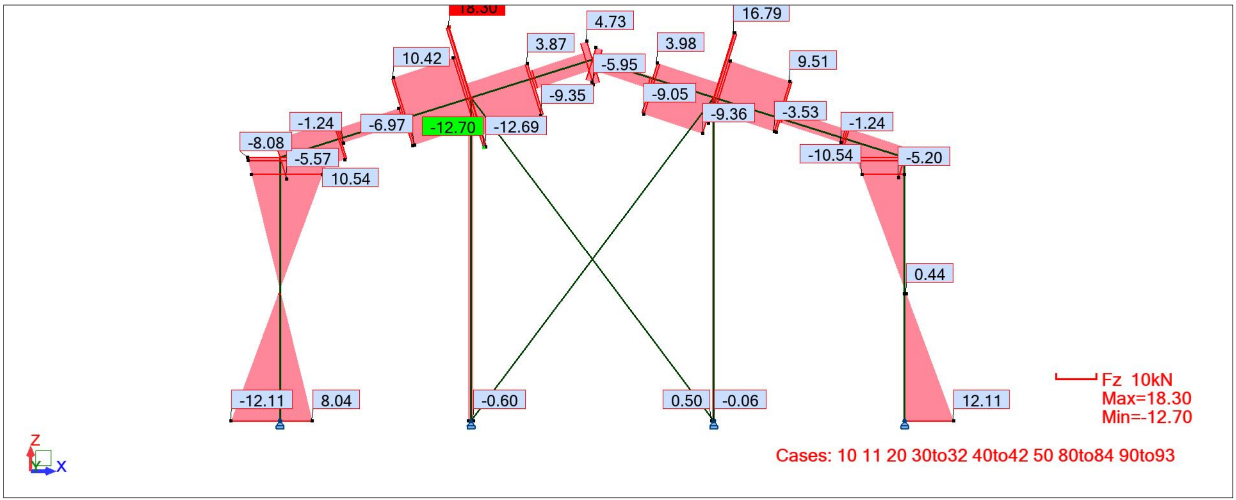
View - Cases: 321 (Int. Wind Cpi min)



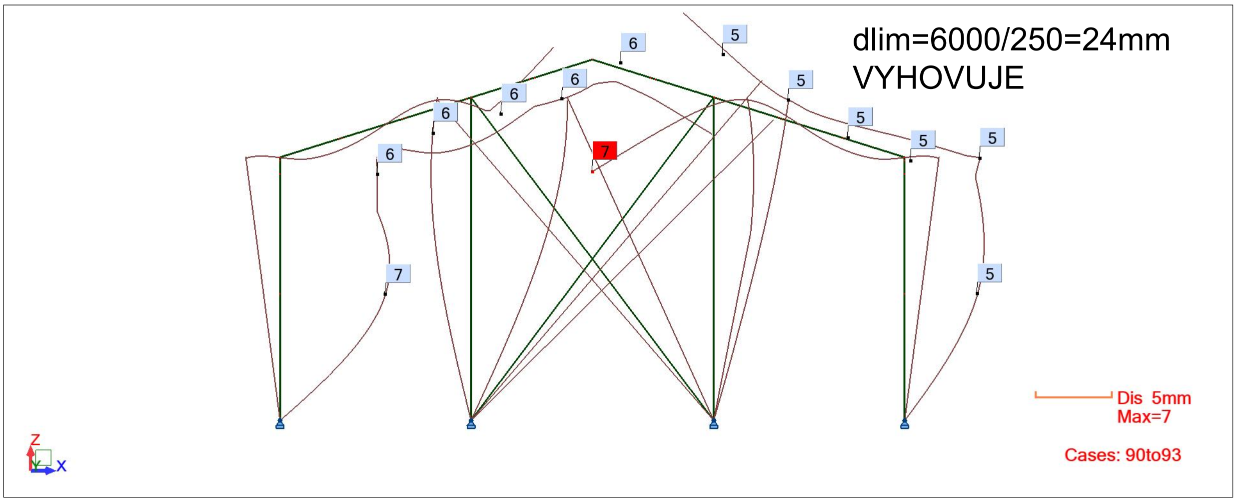
View - MY; Cases: 10 11 20 30to32 40to42 50 80to84 90to93



View - FZ; Cases: 10 11 20 30to32 40to42 50 80to84 90to93



View - Deformation; Cases: 90to93



Gablebeam Single C-profile

$h \equiv 250 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 36 \cdot mm$ $t \equiv 4 \cdot mm$ $\gamma_{M0} \equiv 1.0$ $\gamma_{M1} \equiv 1.0$

Stresses and global geometry:

$f_{yb} = 420 \frac{N}{mm^2}$ $f_u = 480 \frac{N}{mm^2}$ $N_{c.Rk} = 630.1\,kN$

$M_{ycRk} = 62.6\,kN \cdot m$

Buckling lengths:

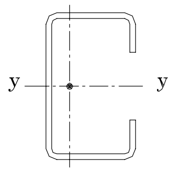
$L := 4.7 \cdot m$ Length

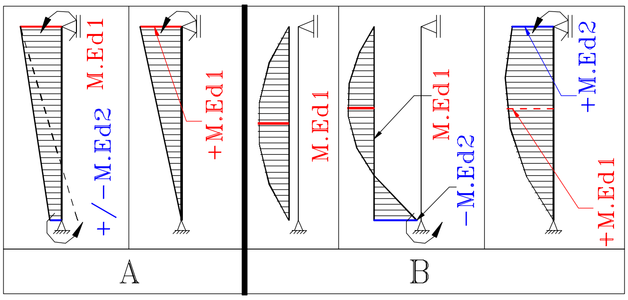
$L_y := 1L$ Flexural buckling axis y-y

$L_z := 1L$ Flexural buckling axis z-z

$L_{LT} := 1L$ Distance flange bracings - LTB innerflange in compression

$L_T := 1L$ Torsional buckling





stresses:

$M_{Ed.1} := 8 \cdot kN \cdot m$

$M_{Ed.2} := -17 \cdot kN \cdot m$

$M_{Ed.z} := 0 \cdot kN \cdot m$

$N_{Ed} := 18 \cdot kN$

$V_{Ed} := 18 \cdot kN$

$Moment\_dis := "B"$ The moment is distributed according to frame modelling like shown in figure:

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = -0.09\,kN \cdot m$

$M_{z.Ed} := |\Delta M_{z.shift}| + M_{Ed.z}$

Combined bending an axial compression EN 1993-1-3: 6.2.2 and 6.2.3

Design buckling resistance for buckling mode -flexural buckling-: y-y

$\lambda_1 = 70.25$ $\lambda_{r.y.FB}(L_y) = 0.6$ $N_{cr.y.C}(L_y) = 1776\,kN$ $\alpha_{y.FB} = 0.34$ $\phi_{y.FB}(L_y) = 0.74$ $\chi_{y.FB}(L_y) = 0.84$

$N_{b.Rk.y.FB}(L_y) = 528.8\,kN$ $N_{b.Rd.y.FB}(L_y) = 528.8\,kN$

Design buckling resistance for buckling mode -flexural buckling-: z-z

$\lambda_1 = 70.25$ $\lambda_{r.z.FB}(L_z) = 1.55$ $N_{cr.z.C}(L_z) = 260.9\,kN$ $\alpha_{z.FB} = 0.34$ $\phi_{z.FB}(L_z) = 1.94$ $\chi_{z.FB}(L_z) = 0.32$

$N_{b.Rk.z.FB}(L_z) = 203.55\,kN$ $N_{b.Rd.z.FB}(L_z) = 203.55\,kN$

Design buckling resistance for buckling mode -torsional or torsional-flexural buckling-:

$I_T = 1.08 \times 10^4\,mm^4$ $3.72 \times 10^{10}\,mm^6$ $y_0 = 78.64\,mm$ $z_0 = 0\,mm$ $i_0 = 131.04\,mm$ $_{TF} = 0.64$ $N_{cr.TF}(L_T, L_y) = 240.3\,kN$

$\lambda_{r.TF}(L_T, L_y) = 1.62$ $\alpha_{TF} = 0.34$ $\phi_{TF}(L_T, L_y) = 2.05$ $\chi_{TF}(L_T, L_y) = 0.3$ $N_{b.Rd.TF}(L_T, L_y) = 190.18\,kN$

$N_{b.Rk.TF}(L_T, L_y) = 190.18\,kN$

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := max(|M_{Ed.1}|, |M_{Ed.2}|)$ $M_{y.max.C} = 17\,kN \cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25 \cdot L_{LT}$ $M_{y.AA} := M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = 1.75\,kN \cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5 \cdot L_{LT}$ $M_{y.BA} := M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = -4.5\,kN \cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75 \cdot L_{LT}$ $M_{y.CA} := M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = -10.75\,kN \cdot m$

$$C_{bA} := \frac{12.5 \cdot M_{y.max.C}}{2.5 \cdot M_{y.max.C} + 3 \cdot |M_{y.AA}| + 4 \cdot |M_{y.BA}| + 3 \cdot |M_{y.CA}|}$$

$y_{0.cr} := (y_M + e_{1c.rc}) \cdot -1$ $z_{0.cr} := 0 \cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2 \cdot E}{\left(\frac{1 \cdot L_{LT}}{i_z}\right)^2}$$ $\sigma_{ez} = 132.02 \frac{N}{mm^2}$

$$\sigma_T := \frac{1}{A_g \cdot r_0^2} \cdot \left(G \cdot I_T + \frac{\pi^2 \cdot E \cdot I_\omega}{L_T^2}\right)$$ $\sigma_T = 128.47 \frac{N}{mm^2}$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry:

$M_{e.cr.C\_A} := C_{bA} \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T}$ $M_{e.cr.C\_A} = 73.1\,kN \cdot m$

$b = 100\,mm \quad h = 250\,mm \quad c = 36\,mm \quad t = 4\,mm \qquad f_{yb} = 420\,\frac{N}{mm^2} \quad f_u = 480\,\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Relative slenderness:

$$\lambda_{rLT.C\_A} := \begin{cases} \sqrt{\dfrac{W_{eff.y.1} \cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0 \\ 0.2 & \text{if } M_{e.cr.C\_A} = 0 \end{cases} \qquad \lambda_{rLT.C\_A} = 0.93$$

Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$$\phi_{LT.C\_A} := 0.5 \cdot \left[1 + \alpha_{LT.C} \cdot \left(\lambda_{rLT.C\_A} - 0.2\right) + \lambda_{rLT.C\_A}^{\,2}\right] \quad \chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^{\,2} - \lambda_{rLT.C\_A}^{\,2}}}, 1\right) \quad \chi_{LT.C\_A} = 0.65$$

For moment distribution accord. to modell B: $q_{Ed.M} := \dfrac{\left|M_{Ed.1}\right| \cdot 8}{L^2} \qquad M_{y.i}(x_i) := \dfrac{q_{Ed.M}}{2} \cdot \left(L \cdot x_i - x_i^{\,2}\right)$

Moment at quater point of unbraced segment:

$$x_{1\_4} := \begin{cases} \dfrac{L_{LT}}{4} & \text{if } L_{LT} = 0.5 \cdot L \\ \dfrac{L - L_{LT}}{2} + \dfrac{1}{4} \cdot L_{LT} & \text{otherwise} \end{cases} \qquad x_{1\_4} = 1.18\,m \quad M_{y.i}(x_{1\_4}) = 6\,kN \cdot m \quad M_{y.AB} := M_{y.i}(x_{1\_4}) \quad M_{y.AB} = 6\,kN \cdot m$$

Moment at centerline of unbraced segment:

$$x_{1\_2} := \begin{cases} \dfrac{2 \cdot L_{LT}}{4} & \text{if } L_{LT} = 0.5 \cdot L \\ \dfrac{L - L_{LT}}{2} + \dfrac{1}{2} \cdot L_{LT} & \text{otherwise} \end{cases} \qquad x_{1\_2} = 2.35\,m \quad M_{y.i}(x_{1\_2}) = 8\,kN \cdot m \quad M_{y.BB} := M_{y.i}(x_{1\_2}) \quad M_{y.BB} = 8\,kN \cdot m$$

Moment at 3/4-point of unbraced segment:

$$x_{3\_4} := \begin{cases} \dfrac{3 \cdot L_{LT}}{4} & \text{if } L_{LT} = 0.5 \cdot L \\ \dfrac{L - L_{LT}}{2} + \dfrac{3}{4} \cdot L_{LT} & \text{otherwise} \end{cases} \qquad x_{3\_4} = 3.53\,m \quad M_{y.i}(x_{3\_4}) = 6\,kN \cdot m \quad M_{y.CB} := M_{y.i}(x_{3\_4}) \quad M_{y.CB} = 6\,kN \cdot m$$

$$C_{bB} := \frac{12.5 \cdot \left(\left|M_{Ed.1}\right|\right)}{2.5 \cdot \left(\left|M_{Ed.1}\right|\right) + 3 \cdot M_{y.AB} + 4 \cdot M_{y.BB} + 3 \cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry:

$$M_{e.cr.C\_B} := C_{bB} \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T} \qquad M_{e.cr.C\_B} = 38.3\,kN \cdot m$$

Relative slenderness:

$$\lambda_{rLT.C\_B} := \begin{cases} \sqrt{\dfrac{W_{eff.y.1} \cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0 \\ 0.2 & \text{if } M_{e.cr.C\_B} = 0 \end{cases} \qquad \lambda_{rLT.C\_B} = 1.28$$

$$\phi_{LT.C\_B} := 0.5 \cdot \left[1 + \alpha_{LT.C} \cdot \left(\lambda_{rLT.C\_B} - 0.2\right) + \lambda_{rLT.C\_B}^{\,2}\right] \quad \chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^{\,2} - \lambda_{rLT.C\_B}^{\,2}}}, 1\right) \quad \chi_{LT.C\_B} = 0.44$$

$\phi_{LT.C\_B} = 1.5$

Moment distribution: $Moment\_dis = \text{"B"}$

$$\chi_{LT} := \begin{cases} \chi_{LT.C\_A} & \text{if } Moment\_dis = \text{"A"} \\ \chi_{LT.C\_B} & \text{if } Moment\_dis = \text{"B"} \end{cases} \qquad \chi_{LT} = 0.44$$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):

The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3

Moment distribution like shown on figure: $Moment\_dis = \text{"B"}$

$$\psi_{md.A} := \begin{cases} \dfrac{M_{Ed.2}}{M_{Ed.1}} & \text{if } \left|M_{Ed.1}\right| \geq \left|M_{Ed.2}\right| \\ \dfrac{M_{Ed.1}}{M_{Ed.2}} & \text{if } \left|M_{Ed.1}\right| < \left|M_{Ed.2}\right| \end{cases} \qquad \psi_{md.A} = -0.47 \qquad \psi_{m.B} := 0$$

$$h.2 := \begin{cases} \dfrac{M_{Ed.2}}{M_{Ed.1}} & \text{if } \left|M_{Ed.1}\right| \geq \left|M_{Ed.2}\right| \\ 0 & \text{otherwise} \end{cases} \qquad \alpha_{h.2} = 0$$

$\psi_{m.B} := 0$

$$\alpha_{s.2} := \begin{cases} \dfrac{M_{Ed.1}}{M_{Ed.2}} & \text{if } \left|M_{Ed.1}\right| < \left|M_{Ed.2}\right| \\ 0 & \text{otherwise} \end{cases} \qquad \alpha_{s.2} = -0.47$$

$\psi_{md.z} := 0 \quad \alpha_{h.2.z} := 0$

$b = 100\,mm \quad h = 250\,mm \quad c = 36\,mm \quad t = 4\,mm \qquad f_{yb} = 420\,\frac{N}{mm^2} \quad f_u = 480\,\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

$C_{my.2.A} := \max(0.6 + 0.4\cdot\psi_{md.A}, 0.4) \quad C_{my.2.A} = 0.41$

$C_{my.2.B.s} := \begin{cases} \max\left[(0.2 + 0.8\cdot\alpha_{s.2}), 0.4\right] & \text{if } 0 \le \alpha_{s.2} \le 1 \\ \max\left[(0.1 - 0.8\cdot\alpha_{s.2}), 0.4\right] & \text{if } -1 \le \alpha_{s.2} < 0 \end{cases} \quad C_{my.2.B.s} = 0.48$

$C_{my.2.B.h} := 0.95 + 0.05\cdot\alpha_{h.2} \quad C_{my.2.B.h} = 0.95$

$C_{my.2.B} := \begin{cases} C_{my.2.B.h} & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ C_{my.2.B.s} & \text{otherwise} \end{cases} \quad C_{my.2.B} = 0.48$

$C_{mz.2} := 0.95 + 0.05\cdot\alpha_{h.2.z} \quad C_{mz.2} = 0.95$

$C_{my.2} := \begin{cases} C_{my.2.A} & \text{if Moment\_dis} = \text{"A"} \\ C_{my.2.B} & \text{if Moment\_dis} = \text{"B"} \end{cases} \quad C_{my.2} = 0.48 \quad C_{mLT.2} := C_{my.2} \quad C_{mLT.2} = 0.48$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$n_{y.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(L_y)\cdot N_{c.Rk}} \quad n_{y.2} = 0.03 \quad n_{z.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(L_z)\cdot N_{c.Rk}} \quad n_{z.2} = 0.09$

$k_{yy} := \min\left[C_{my.2}\cdot\left(1 + 0.6\cdot\lambda_{r.y.FB}(L_y)\cdot n_{y.2}\right), C_{my.2}\cdot\left(1 + 0.6\cdot n_{y.2}\right)\right] \quad k_{yy} = 0.48$

$k_{zy} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FB}(L_z)}{C_{mLT.2} - 0.25}\cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25}\cdot n_{z.2}\right) \quad k_{zy} = 0.98$

$k_{zz} := \min\left[C_{mz.2}\cdot\left(1 + 0.6\cdot\lambda_{r.z.FB}(L_z)\cdot n_{z.2}\right), C_{mz.2}\cdot\left(1 + 0.6\cdot n_{z.2}\right)\right] \quad k_{zz} = 1 \quad k_{yz} := k_{zz}$

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$\frac{N_{Ed}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M0}}{M_{ycRk}} + \frac{|M_{z.Ed}|\cdot\gamma_{M0}}{M_{z.Rk}} = 0.31$ <1.0

Axial compression EN 1993-1-1: 6.3.1:

with relevant buckling mode:

$\frac{N_{Ed}\cdot\gamma_{M1}}{\min(\chi_{y.FB}(L_y), \chi_{z.FB}(L_z), \chi_{TF}(L_T, L_y))\cdot N_{c.Rk}} = 0.09$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y: $\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(L_y)\cdot N_{c.Rk}} + k_{yy}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{ycRk}} + k_{yz}\cdot\frac{|M_{z.Ed}|\cdot\gamma_{M1}}{M_{z.Rk}} = 0.34$ <1.0

Evading in z-z $\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(L_z)\cdot N_{c.Rk}} + k_{zy}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{ycRk}} + k_{zz}\cdot\frac{|M_{z.Ed}|\cdot\gamma_{M1}}{M_{z.Rk}} = 0.70$ <1.0

Shear force EN 1993-1-3: 6.1.10:

$\frac{V_{Ed}\cdot\gamma_{M1}\cdot 2}{V_{bh.Rk}} = 0.17$ < 1.0
if not met, check formula below:

$M\_N\_V_{EC} = 0$ <1.0

G. Štítové sloupy A1/1, 8 D1/1, 8

Geometrie:

Zatěžovací šířka sloupku L_w= 3.51 m
Rozpon L= 6.30 m

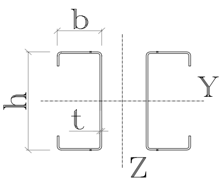
Zatížení větrem: ČSN EN 1991-1-4 ()

Kategorie terénu: II. Oblasti s nízkou vegetací jako je tráva nebo izolované překážky (stromy, budovy)
Charakteristická hodnota rychlosti větru: v_{bo}= 25.0 m/s
Maximální dynamický tlak q_p (z)= 0.936 kN/m²

Součinitel vnějšího tlaku (tlak) C_{pe}=	0.96		Součinitel vnějšího tlaku (sání) C_{pe}=	1.50	
Zatížení větrem (tlak) w_e=	0.90	kN/m²	Zatížení větrem (sání) w_e=	1.40	kN/m²
Normové zatížení větrem (tlak) q_n=	**3.15**	kN/m	Normové zatížení větrem (sání) q_n=	**4.93**	kN/m
Součinitel zatížení g_w=	1.5		Součinitel zatížení g_w=	1.5	
Výpočtové zatížení větrem (tlak) q_d=	**4.73**	kN/m	Výpočtové zatížení větrem (sání) q_d=	**7.39**	kN/m

Charakteristika profilu:

Průřez profilem = **Dvojitý** Profil = **C250X4**



Výška profilu h=	250	mm	Momentová únosnost M_y=	60.48	kNm
Šířka profilu b=	100	mm	Smyková únosnost V_{wy}=	208.46	kN
Tloučťka t=	4	mm	Smyk v podpoře R_{wy}=	85.15	kN
Mez kluzu oceli f_u=	480	MPa	Efekt. moment setrvačnosti I_{yeff}=	18380000	mm4
Mez pevnosti oceli f_b=	420	MPa	Moment setrvačnosti I_y=	19000000	mm4

Součinitelé materiálu:

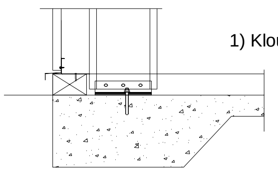
Country = **Czech**
Únosnost průřezu γ_{M0}= 1.00
Únosnost při vzpěru γ_{M1}= 1.00
Únosnost oslabeného průřezu γ_{M2}= 1.25

Deformace:

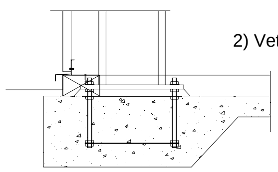
Limit deformace 1/ **250**
Skutečný poměr 481

Kotvení do spodní stavby:

Vetknuté kotvení



1) Kloubové kotvení



2) Vetknuté kotvení

Předběžné posouzení:

Vetknutý sloup

Sání pro C_{pe}= 1.50 Tlak pro C_{pe}= 0.96

Ohybový moment ve vetknutí								
M_{Sd}=1/8.q.L^2	$M_{Sd,s}$=	**36.66**	kNm	$M_{Sd,s}$=	**23.46**	kNm	My =	**120.96** kNm
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				
Ohybový moment v poli								
M_{Sd}=9/128.q.L^2	$M_{Sd,s}$=	**20.62**	kNm	$M_{Sd,s}$=	**13.20**	kNm	My =	**120.96** kNm
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				
Podporová reakce								
R=5/8.q.L	$R_{Sd,s}$=	**29.09**	kN	$R_{Sd,s}$=	**18.62**	kN	Ry =	**170.30** kN
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				
Deformace								
w = q.L4/185.E.Iy	w_{max} =	**5.43**	mm	w_{max} =	**3.48**	mm	w_{lim} =	**25.20** mm
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				

Double gable pillar
A1/1, 8 D1/1, 8

$h \equiv 250 \cdot mm$ $\quad b \equiv 100 \cdot mm$ $\quad c \equiv 36 \cdot mm$ $\quad t \equiv 4 \cdot mm$ $\quad \gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $\quad cc_{batt} := 1000 \cdot mm$ distance between battens

$b_{batt} := 200 \cdot mm$ $\quad t_{batt} := 4 \cdot mm$ $\quad \gamma_{M1} \equiv 1.0$

$h_{batt} := 200 \cdot mm$

$A_{gg} := 2 \cdot A_g$ $\qquad A_{gg} = 3.952 \times 10^3 \, mm^2$

$I_{zz} := 2 \cdot \left[I_z + A_g \cdot \left(e_1 + \frac{d}{2} \right)^2 \right]$ $\qquad I_{zz} = 3.26 \times 10^7 \, mm^4$

$M_{yy.cRk} := 2 \cdot M_{ycRk}$

$M_{yy.cRk} = 125.14 \, kN \cdot m$

$f_{yb} = 420 \frac{N}{mm^2}$

$f_u = 480 \frac{N}{mm^2}$

$I_{yy} := 2 \cdot I_y$ $\qquad I_{yy} = 3.79 \times 10^7 \, mm^4$

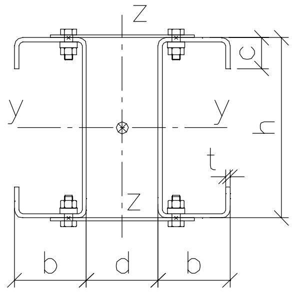
single C: $M_{ycRk} = 62.57 \, kN \cdot m$

$W_{yy} := 2 \cdot W_y$ $\qquad W_{yy} = 3.08 \times 10^5 \, mm^3$

$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $\qquad W_{yy.eff} = 2.98 \times 10^5 \, mm^3$

$N_{cc.Rk} = 1260.25 \, kN$



$W_{zz} := \frac{I_{zz}}{b + \frac{d}{2}}$ $\qquad W_{zz} = 2.17 \times 10^5 \, mm^3$

$i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $\qquad i_{yy} = 97.88 \, mm$ $\qquad i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $\qquad i_{zz} = 90.81 \, mm$

$I_{TT} = 21589.33 \, mm^4$ $\qquad$ torsion_plate_pillar = "NO"

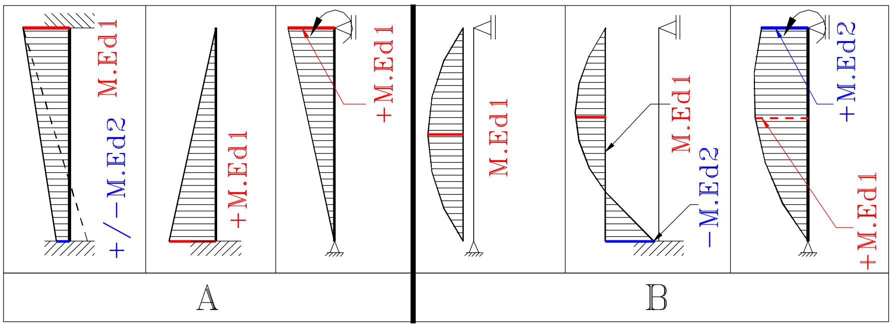
Stresses, global geometry and buckling lengths:

$M_{Ed.1} := 14 \cdot kN \cdot m$ Moment y-y

$M_{Ed.suc.1} := 21 \cdot kN \cdot m$ Moment y-y acting as suction

$M_{Ed.1.z} := 19 \cdot kN \cdot m$ Moment z-z

$N_{Ed} := 15 \cdot kN$



$V_{Ed} := 30 \cdot kN$ > $\quad n_{cross} := 4$ =Numbers of holes in **ONE** web

$d_0 := 12.5 \cdot mm$

Moment_dis := "B" $\qquad M_{Ed.2} := 37 \cdot kN \cdot m$

shift of neutral axis for member in compression: $\qquad \Delta e_N = -5.24 \, mm$

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\qquad \Delta M_{z.shift} = -0.08 \, kN \cdot m$ $\quad {}_{z.Ed} := \Delta M_{z.shift}$

Buckling lengths:

$L := 6.3 \cdot m$ Length of pillar $\qquad cc_{wallZ} := L$ = Distance between wall-Z-profile

$L_y := 1.0 \cdot L$ Flexural buckling axis y-y

$L_z := cc_{wallZ}$ Flexural buckling axis z-z (= Distance between wall-Z-profile)

$L_{LT} := cc_{wallZ}$ Distance flange bracings - LTB outer flange in compression for wind acting as pressure

$L_{LT.suc} := 1.0 \cdot L$ Distance flange bracings - LTB inner flange in compression for wind acting as suction

$L_T := cc_{batt}$ Torsional buckling single C-profile between battens.

$b = 100\,mm$ $\quad h = 250\,mm$ $\quad c = 36\,mm$ $\quad t = 4\,mm$ $\quad f_{yb} = 420\,\frac{N}{mm^2}$ $\quad f_u = 480\,\frac{N}{mm^2}$ $\quad \gamma_{M0} = 1.00$ $\quad \gamma_{M1} = 1.00$

Flexural buckling resistance - axis y-y and z-z EN 1993-1-3: 6.2.2 and EN 1993-1-1: 6.3.1:

Buckling curve for double C-section: EN 1993-1-3 table 6.3 about:

y-y: a EN1993-1-1,table 6.1: α =0,21 z-z: b EN1993-1-1,table 6.1: α =0,34

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about y-y

$$\lambda_1 := \pi\cdot\sqrt{\frac{E}{f_{yb}}} \qquad \lambda_1 = 70.25 \qquad \lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \qquad \lambda_{r.y.FBcc} = 0.8$$

Imperfection factor α relating to bucking curve a $\quad \alpha_y := 0.21$

$$\phi_{y.FB} := 0.5\cdot\left[1 + \alpha_y\cdot\left(\lambda_{r.y.FBcc} - 0.2\right) + \lambda_{r.y.FBcc}^{\;2}\right] \qquad \phi_{y.FB} = 0.88$$

$$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^{\;2} - \lambda_{r.y.FBcc}^{\;2}}}, 1\right) \qquad \chi_{y.FBcc} = 0.8$$

Design buckling resistance for buckling mode -<u>flexural buckling</u>-: y-y

$$N_{b.Rd.y.FBcc} := \frac{\chi_{y.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \qquad N_{b.Rd.y.FBcc} = 1003.9\,kN$$

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about z-z

$$\lambda_1 = 70.25 \qquad \lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \qquad \lambda_{r.z.FBcc} = 0.86$$

Imperfection factor α relating to bucking curve b $\quad \alpha_z := 0.34$

$$\phi_{z.FB} := 0.5\cdot\left[1 + \alpha_z\cdot\left(\lambda_{r.z.FBcc} - 0.2\right) + \lambda_{r.z.FBcc}^{\;2}\right] \qquad \phi_{z.FB} = 0.98$$

$$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^{\;2} - \lambda_{r.z.FBcc}^{\;2}}}, 1\right) \qquad \chi_{z.FBcc} = 0.69$$

Design buckling resistance for buckling mode -<u>flexural buckling</u>-: z-z

$$N_{b.Rd.z.FBcc} := \frac{\chi_{z.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \qquad N_{b.Rd.z.FBcc} = 865.1\,kN$$

Lateral-torsional buckling resistance for uniform member in bending EN 1993-1-3: 6.2.4 and EN 1993-1-1: 6.3.2:

Procedure to calculate the elastic critical buckling moment M_{cr}, based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints, is given neither in EN 1993-1-1 nor in EN 1993-1-3. Any appropriate calculation method can be used, here the procedure given in German ENV 1993-1-1 Annex F.

Moment_dis = "B" $\quad L_{LT} = 6.3\,m$ $\quad k_{M.cr} := 1.0$ (hinged at ends) $\quad k_{w.M.cr} := 1.0$ (no special wrap restraints at ends)

For pillar with end moments: according to picture "A":

$$\psi_{md.A} := \begin{cases} \dfrac{M_{Ed.2}}{M_{Ed.1}} & \text{if } \left|M_{Ed.1}\right| \ge \left|M_{Ed.2}\right| \\ \dfrac{M_{Ed.1}}{M_{Ed.2}} & \text{if } \left|M_{Ed.1}\right| < \left|M_{Ed.2}\right| \end{cases} \qquad \psi_{md.A} = 0.38$$

$$C_{1.A} := \min\left(1.88 - 1.40\cdot\psi_{md.A} + 0.52\cdot\psi_{md.A}^{\;2}, 2.7\right) \qquad C_{1.A} = 1.42$$

$$M_{cr.A} := C_{1.A}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{\left(k_{M.cr}\cdot L_{LT}\right)^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr}\cdot L_{LT}\right)^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.A} = 149.23\,kN\cdot m$$

For pillar with moment distribution according to picture "B": $\quad C_{1.B} := 1.132$

$$M_{cr.B} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{\left(k_{M.cr}\cdot L_{LT}\right)^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr}\cdot L_{LT}\right)^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.B} = 118.57\,kN\cdot m$$

<u>Elastic critical moment for lateral-torsional buckling</u>

based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints for double C-section:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if Moment\_dis} = \text{"A"} \\ M_{cr.B} & \text{if Moment\_dis} = \text{"B"} \end{cases} \qquad M_{cr} = 119\,kN\cdot m$$

Relative slenderness: $\lambda_{rLT} := \sqrt{\dfrac{2\cdot W_{eff.y.1}\cdot f_{yb}}{M_{cr}}}$ $\quad \lambda_{rLT} = 1.03$ $\quad \alpha_{LT} := 0.34$ $\quad \phi_{LT} := 0.5\cdot\left[1 + \alpha_{LT}\cdot\left(\lambda_{rLT} - 0.2\right) + \lambda_{rLT}^{\;2}\right]$ $\quad \phi_{LT} = 1.17$

Reduction buckling factor: $\chi_{LT} := \min\left(\dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{\;2} - \lambda_{rLT}^{\;2}}}, 1\right)$ $\quad \chi_{LT} = 0.58$

Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

Desing buckling moment resistance double profile based on effective section modulus W_{eff}.

$$M_{b.Rd} := \frac{\chi_{LT}\cdot M_{yy.cRk}}{\gamma_{M1}} \qquad M_{b.Rd} = 72.55\,kN\cdot m$$

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

For pillar when wind acting as suction (moment from line load):

$$M_{cr.suc} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT.suc})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}}+\frac{(k_{M.cr}\cdot L_{LT.suc})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.suc} = 118.57\,kN\cdot m$$

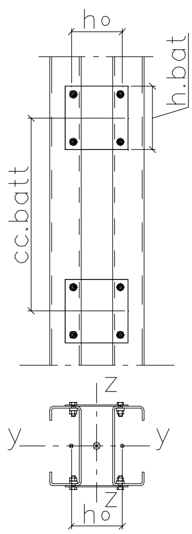
Relative slenderness: $\lambda_{rLT.suc} := \sqrt{\frac{2\cdot W_{eff.y.1}\cdot f_{yb}}{M_{cr.suc}}}$ $\lambda_{rLT.suc} = 1.03$ $\phi_{LT.suc} := 0.5\cdot\left[1+\alpha_{LT}\cdot(\lambda_{rLT.suc}-0.2)+\lambda_{rLT.suc}{}^2\right]$ $\phi_{LT.suc} = 1.17$

Reduction buckling factor: $\chi_{LT.suc} := \min\left(\frac{1}{\phi_{LT.suc}+\sqrt{\phi_{LT.suc}{}^2-\lambda_{rLT.suc}{}^2}},1\right)$ $\chi_{LT.suc} = 0.58$

<u>Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:</u>

Desing buckling moment resistance double profile for wind acting as suction $M_{b.Rd.suc} := \frac{\chi_{LT.suc}\cdot M_{yy.cRk}}{\gamma_{M1}}$ $M_{b.Rd.suc} = 72.55\,kN\cdot m$

Check Uniform built-up member EN 1993-1-1: 6.4



bow imperfection: $e_0 := \frac{L}{500}$ $\text{if}\left(\frac{cc_{batt}}{i_z}\le 70,\text{"OK"},\text{"Shorter-cc.bat"}\right) = \text{"OK"}$

non-dimensional slenderness: $\lambda_1 = 70.25$

<u>Effective second moment of area of battened built-up member:</u> $A_{ch} := A_g$: area of one chord

$h_0 := d + 2\cdot e_1$ $h_0 = 165.4\,mm$ distance centroids of chords $I_{ch} := I_z$: I of one chord

$I_{l.CC} := 0.5\cdot h_0{}^2\cdot A_{ch} + 2\cdot I_{ch}$ I built-up member $i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2\cdot A_{ch}}}$ $\lambda_{CC} := \frac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 69.37$

EN 1993-1-1: table 6.8
Efficiency factor:

$$\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \ge 150 \\ \left(2-\frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \le 75 \end{cases}$$

$I_{l.CC} = 3.26\times10^7\,mm^4$ $\mu_{CC} = 1$ $r_{CC} := 2$

$I_{eff} := 0.5\cdot h_0{}^2\cdot A_{ch} + 2\cdot\mu_{CC}\cdot I_{ch}$ $I_{eff} = 3.26\times10^7\,mm^4$ effective I of built-up member

<u>Shear stiffness EN 1993-1-1:6.4.3.1:(2)</u> $I_b := \frac{t_{batt}\cdot h_{batt}{}^3}{12}$ I of batten $n_{batt} := \frac{L}{cc_{batt}}$ $n_{batt} = 6.3$ umber of planes of lacings

$$S_v := \min\left[\left[\frac{24\cdot E\cdot I_{ch}}{cc_{batt}{}^2\cdot\left(1+\frac{2\cdot I_{ch}}{n_{batt}\cdot I_b}\cdot\frac{h_0}{cc_{batt}}\right)}\right],\left(\frac{2\cdot\pi^2\cdot E\cdot I_{ch}}{cc_{batt}{}^2}\right)\right] \qquad S_v = 11525\,kN$$

effective critical force of built-up member:

$$N_{cr.CC} := \frac{\pi^2\cdot E\cdot I_{eff}}{L_z{}^2} \qquad N_{cr.CC} = 1701.9\,kN$$

Maximum moment in middle of built-up member: EN 1993-1-1:6.4.1 (6):

$M_{z.Ed.I} := |M_{Ed.1.z}|$
moment z-z without second order effects

moment with second order effects $M_{z.Ed.C} := \frac{N_{Ed}\cdot e_0 + M_{z.Ed.I}}{1-\frac{N_{Ed}\cdot\gamma_{M1}}{N_{cr.CC}}-\frac{N_{Ed}\cdot\gamma_{M1}}{S_v}}$ $M_{z.Ed.C} = 19.39\,kN\cdot m$

Compression force in one member: EN 1993-1-1:6.4.1 (6) Compression force in built-up member taking accout to second order effects:

$$X_d := \begin{cases} 1 & \text{if } d \ge 8\cdot mm \\ 0 & \text{otherwise} \end{cases} \qquad N_{ch.Ed} := \frac{N_{Ed}}{r_{CC}} + \frac{|M_{z.Ed.C}|\cdot h_0\cdot A_{ch}\cdot X_d}{2\cdot I_{eff}} \qquad N_{ch.Ed} = 104.7\,kN$$

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):
The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3
Moment distribution like shown on figure: $Moment\_dis = "B"$ $\psi_{md.A} = 0.38$ $\psi_{m.B} := 0$

$\alpha_{h.2} = 0$ $\alpha_{s.2} = 0.38$ $\psi_{md.z} := 0$ $\alpha_{h.2.z} = 0$ $\psi_{md.suc} := 0$ $\alpha_{h.2.suc} = 0$

$C_{my.2} = 0.5$ $C_{my.2.suc} = 0.95$ $C_{mz.2} = 0.95$ $C_{mLT.2} = 0.5$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}}$ $n_{y.2} = 0.01$ $n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}}$ $n_{z.2} = 0.02$

$k_{yy.2} := \min\left[C_{my.2} \cdot \left(1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}\right), C_{my.2} \cdot \left(1 + 0.6 \cdot n_{y.2}\right)\right]$ $k_{yy.2} = 0.51$

$k_{zy.2} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FBcc}}{C_{mLT.2} - 0.25} \cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25} \cdot n_{z.2}\right)$ $k_{zy.2} = 1$

$k_{zz.2} := \min\left[C_{mz.2} \cdot \left(1 + 0.6 \cdot \lambda_{r.z.FBcc} \cdot n_{z.2}\right), C_{mz.2} \cdot \left(1 + 0.6 \cdot n_{z.2}\right)\right]$ $k_{zz.2} = 0.96$ $k_{yz.2} := k_{zz.2}$

$k_{yy.2.suc} := \min\left[C_{my.2.suc} \cdot \left(1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}\right), C_{my.2.suc} \cdot \left(1 + 0.6 \cdot n_{y.2}\right)\right]$ $k_{yy.2.suc} = 0.96$

$I_{zz.eff} := 2 \cdot I_z + A_g \cdot \left(e_1 + \frac{d}{2}\right)^2 + A_{eff} \cdot \left(e_N + \frac{d}{2}\right)^2$ $W_{zz.eff} := \frac{I_{zz.eff}}{b + \frac{d}{2}}$ $I_{zz.eff} = 3.02 \times 10^7\,mm^4$ $W_{zz.eff} = 2.01 \times 10^5\,mm^3$ $M_{zz.cRk} := W_{zz.eff} \cdot f_{yb}$

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{Ed} \cdot \gamma_{M0}}{2 \cdot N_{c.Rk}} + \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.suc.1}\right|\right) \cdot \gamma_{M0}}{2 \cdot M_{ycRk}} + \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M0}}{M_{zz.cRk}} = 0.54$$ <1.0

Double CC-profile: Combined M + N + V acc to EN 1993-1-3: 6.1.10

$\frac{V_{Ed} \cdot \gamma_{M1}}{2 \cdot V_{bh.Rk.pillar}} = 0.09$ < 1.0 according to EN 1993-1-3: 6.1.10 $M\_N\_V_{EC} = 0$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as pressure

Evading in y-y: $\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}} + k_{yy.2} \cdot \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{yz.2} \cdot \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.49$ <1.0

Evading in z-z: $\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}} + k_{zy.2} \cdot \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{zz.2} \cdot \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.75$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as suction

Evading in y-y: $\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}} + k_{yy.2.suc} \cdot \frac{\left|M_{Ed.suc.1}\right| \cdot \gamma_{M1}}{\chi_{LT.suc} \cdot M_{yy.cRk}} + k_{yz.2} \cdot \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.51$ <1.0

Evading in z-z: $\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}} + k_{zy.2} \cdot \frac{\left|M_{Ed.suc.1}\right| \cdot \gamma_{M1}}{\chi_{LT.suc} \cdot M_{yy.cRk}} + k_{zz.2} \cdot \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.53$ <1.0

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords and battens are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1).

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := \max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.suc.1}\right|\right)\cdot 0.5$ $M_{y.max.C} = 18.5\,kN\cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25\cdot cc_{batt}$ $M_{y.AA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = 17.13\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5\cdot cc_{batt}$ $M_{y.BA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = 17.59\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75\cdot cc_{batt}$ $M_{y.CA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = 18.04\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot M_{y.max.C}}{2.5\cdot M_{y.max.C} + 3\cdot\left|M_{y.AA}\right| + 4\cdot\left|M_{y.BA}\right| + 3\cdot\left|M_{y.CA}\right|}$$

$y_{0.cr} := (y_M + e_{1c.rc})\cdot -1$ $z_{0.cr} := 0\cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot cc_{batt}}{i_z}\right)^2} \qquad \sigma_{ez} = 2916.25\,\frac{N}{mm^2}$$

$$\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{cc_{batt}^2}\right) \qquad \sigma_T = 2295.94\,\frac{N}{mm^2}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_A} := C_{bA}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 697.5\,kN\cdot m$

Relative slenderness:

$$\lambda_{rLT.C\_A} := \begin{cases} \sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0 \\ 0.2 & \text{if } M_{e.cr.C\_A} = 0 \end{cases} \qquad \lambda_{rLT.C\_A} = 0.3$$

Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right] \qquad \chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right) \qquad \chi_{LT.C\_A} = 0.96$$

$\phi_{LT.C\_A} = 0.56$

For moment distribution accord. to modell B: $q_{wind\_pillar} := \frac{\left|M_{Ed.1}\cdot 0.5\right|\cdot 8}{L^2}$ $M_{y.i}(x_i) := \frac{q_{wind\_pillar}}{2}\cdot\left(L\cdot x_i - x_i^2\right)$

Moment at quater point of unbraced segment: $x_{1\_4} := \frac{L - cc_{batt}}{2} + \frac{1}{4}\cdot cc_{batt}$ $x_{1\_4} = 2.9\,m$ $M_{y.i}(x_{1\_4}) = 6.96\,kN\cdot m$ $M_{y.AB} := M_{y.i}(x_{1\_4})$ $M_{y.AB} = 6.96\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 3.15\,m$ $M_{y.i}(x_{1\_2}) = 7\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 7\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := \frac{L - cc_{batt}}{2} + \frac{3}{4}\cdot cc_{batt}$ $x_{3\_4} = 3.4\,m$ $M_{y.i}(x_{3\_4}) = 6.96\,kN\cdot m$ $M_{y.CB} := M_{y.i}(x_{3\_4})$ $M_{y.CB} = 6.96\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot\left(\left|M_{Ed.1}\cdot 0.5\right|\right)}{2.5\cdot\left(\left|M_{Ed.1}\cdot 0.5\right|\right) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := C_{bB}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 672\,kN\cdot m$

Relative slenderness:

$$\lambda_{rLT.C\_B} := \begin{cases} \sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0 \\ 0.2 & \text{if } M_{e.cr.C\_B} = 0 \end{cases} \qquad \lambda_{rLT.C\_B} = 0.31$$

$$\phi_{LT.C\_B} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_B} - 0.2) + \lambda_{rLT.C\_B}^2\right] \qquad \chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right) \qquad \chi_{LT.C\_B} = 0.96$$

$\phi_{LT.C\_B} = 0.56$

Moment distribution: $Moment\_dis = "B"$

$$\chi_{LT.C} := \begin{cases} \chi_{LT.C\_A} & \text{if } Moment\_dis = "A" \\ \chi_{LT.C\_B} & \text{if } Moment\_dis = "B" \end{cases} \qquad \chi_{LT.C} = 0.96$$

Stresses on one member profile at mid-span from above: chord force from above: $N_{ch.Ed} = 104.7\,kN$ <u>*max moment y-y:*</u> $M_{y.max.C} = 18.5\,kN\cdot m$

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Chord at end panel: highest shear force: $r_{CC} = 2$ $V_{Ed.max} := \pi \cdot \frac{M_{z.Ed.C}}{L}$ $V_{Ed.max} = 9.67\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 4.83\,kN$

$V_{bb.Rd} = 186.23\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.026$ << 1,0 The shear force is negligible

Chord at end panel: "corner" moment: $M_{z.ch} := \frac{V_{Ed.max}}{r_{CC}} \cdot \frac{cc_{batt}}{2}$ *maximum moment z-z:* $M_{z.ch} = 2.42\,kN \cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C} \cdot \sin\left(\frac{\pi \cdot cc_{batt}}{L}\right)$ $M_{z.Ed.C.end} = 9.27\,kN \cdot m$ moment due to bow imperfection at end of panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end} \cdot \frac{(h_0 \cdot A_{ch})}{(2 \cdot I_{eff})}$ $N_{ch.end} = 53.99\,kN$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$n_{y.C.2} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(cc_{batt}) \cdot N_{c.Rk}}$ $n_{y.C.2} = 0.09$ $n_{z.C.2} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(cc_{batt}) \cdot N_{c.Rk}}$ $n_{z.C.2} = 0.09$

Equivalent uniform moment factors: $Moment\_dis = "B"$

Moment at start of unbraced segment: $x_{0\_0} := 0 \cdot cc_{batt}$ $M_{y.A0} := 0.5 \cdot M_{y.Ed.ccbatt\_A}(x_{0\_0})$ $M_{y.A0} = 16.67\,kN \cdot m$

Moment at end of unbraced segment: $M_{y.max.C} = 18.5\,kN \cdot m$

$$\psi_{m.C.A} := \frac{M_{y.A0}}{\begin{cases} M_{Ed.1} \cdot 0.5 & \text{if } |M_{Ed.1}| \geq |M_{Ed.2}| \\ M_{Ed.2} \cdot 0.5 & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}}$$ $\psi_{m.C.A} = 0.9$ range for model "A"

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2} \cdot cc_{batt}$ $x_{1\_2} = 3.15\,m$ $M_{y.i}(x_{1\_2}) = 7\,kN \cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 7\,kN \cdot m$

Moment at start of unbraced segment: $x_{0\_0} := \frac{L - cc_{batt}}{2} + 0 \cdot cc_{batt}$ $x_{0\_0} = 2.65\,m$ $M_{y.i}(x_{0\_0}) = 6.82\,kN \cdot m$ $M_{y.B0} := M_{y.i}(x_{0\_0})$ $M_{y.B0} = 6.82\,kN \cdot m$

$\alpha_{h.C.2} := \frac{|M_{y.B0}|}{|M_{y.BB}|}$ $\alpha_{h.C.2} = 0.97$ $\psi_{m.C.B} := 1$ range for model "B"

$$C_{mLT.C.2} := \begin{cases} \max(0.6 + 0.4 \cdot \psi_{m.C.A}, 0.4) & \text{if } Moment\_dis = "A" \\ (0.95 + 0.05 \cdot \alpha_{h.C.2}) & \text{if } Moment\_dis = "B" \end{cases}$$ $C_{mLT.C.2} = 0.999$ $C_{my.C.2} := C_{mLT.C.2}$ $C_{mz.C.2} := 1$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$\lambda_{r.y.FB}(cc_{batt}) = 0.13$ $k_{yy.C.2} := \min\left[C_{my.C.2} \cdot (1 + 0.6 \cdot \lambda_{r.y.FB}(cc_{batt}) \cdot n_{y.C.2}), C_{my.C.2} \cdot (1 + 0.6 \cdot n_{y.C.2})\right]$ $k_{yy.C.2} = 1.01$

$\lambda_{r.z.FB}(cc_{batt}) = 0.33$ $k_{zz.C.2} := \min\left[C_{mz.C.2} \cdot (1 + 0.6 \cdot \lambda_{r.z.FB}(cc_{batt}) \cdot n_{z.C.2}), C_{mz.C.2} \cdot (1 + 0.6 \cdot n_{z.C.2})\right]$ $k_{zz.C.2} = 1.02$

$$k_{zy.C.2} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FB}(cc_{batt})}{C_{my.C.2} - 0.25} \cdot n_{z.C.2}, 1 - \frac{0.05}{C_{mLT.C.2} - 0.25} \cdot n_{z.C.2}\right)$$ $k_{zy.C.2} = 1$ $k_{yz.C.2} := k_{zz.C.2}$

Controll single C-profile between battens:

$M_{y.max.C} = 18.5\,kN \cdot m$ $M_{z.ch} = 2.42\,kN \cdot m$ $\Delta M_{z.shift} := |\Delta e_N \cdot N_{ch.end}|$ $\Delta M_{z.shift} = 0.28\,kN \cdot m$ $N_{ch.end} = 53.99\,kN$

Buckling moment resistance y-y EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C} \cdot M_{ycRk}$ $\chi_{LT.C} = 0.96$ $M_{y.Rk.C} = 60.21\,kN \cdot m$

Buckling resistance moment z-z EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk})$ $M_{z.Rk.C} = 17.43\,kN \cdot m$

Resistance buckling mode flexural buckling-: y-y $cc_{batt} = 1\,m$ $\chi_{y.FB}(cc_{batt}) = 1$ $N_{b.Rk.y.FB}(cc_{batt}) = 630.13\,kN$

Resistance buckling mode flexural buckling-: z-z $cc_{batt} = 1\,m$ $\chi_{z.FB}(cc_{batt}) = 0.95$ $N_{b.Rk.z.FB}(cc_{batt}) = 600.35\,kN$

Resistance buckling mode -torsional or torsional-flexural $cc_{batt} = 1\,m$ $\chi_{TF}(cc_{batt}, cc_{batt}) = 0.93$ $N_{b.Rk.TF}(cc_{batt}, cc_{batt}) = 588.3\,kN$

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

single C-profile: Axial compression EN 1993-1-1: 6.3.1 (3): Buckling about relevant axis in mid-span or en panel of built-up member

$$\frac{\max(N_{ch.Ed}, N_{ch.end})\cdot\gamma_{M1}}{\min(\chi_{y.FB}(cc_{batt}), \chi_{z.FB}(cc_{batt}), \chi_{TF}(cc_{batt}, cc_{batt}))\cdot N_{c.Rk}} = 0.18 \quad <1.0$$

Controll buil-up member: single C-profile (chords) in mid-span of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.Ed}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.48 \quad <1.0$$

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.49 \quad <1.0$$

Evading in z-z:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.5 \quad <1.0$$

Controll buil-up member: single C-profile (chords) at end panel of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.end}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.54 \quad <1.0$$

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.55 \quad <1.0$$

Evading in z-z:

$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.55 \quad <1.0$$

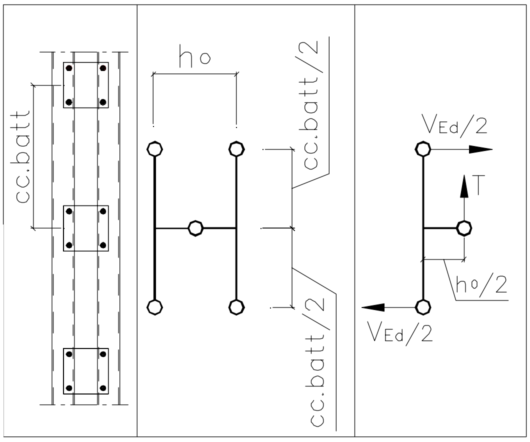
$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Forces on battens: max shear flow in one batten:

$$T_{batt} := \frac{V_{Ed.max} \cdot cc_{batt}}{h_0 \cdot r_{CC}} \qquad T_{batt} = 29.22\,kN$$

max moment in one batten:

$$M_{batt} := \frac{T_{batt} \cdot h_0}{2} \qquad M_{batt} = 2.42\,kN \cdot m$$

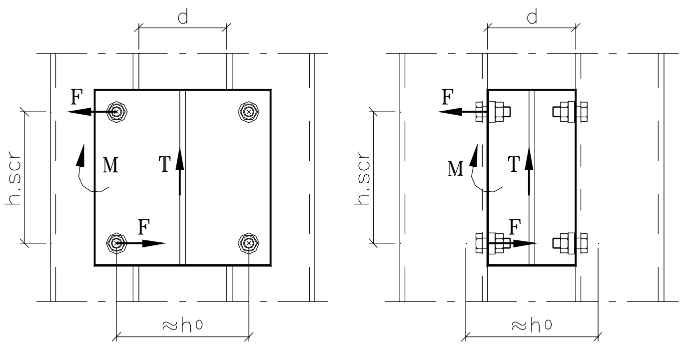


$$\lambda_{w.batt} := 0.346 \cdot \frac{h_{batt}}{t_{batt}} \cdot \sqrt{\frac{f_{yb}}{E}} \qquad f_{vb.batt} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{w.batt} \le 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{w.batt}} & \text{if } 0.83 < \lambda_{w.batt} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{w.batt}^2} & \text{if } \lambda_{w.batt} \ge 1.40 \end{cases}$$

$$V_{bbatt.Rd} := \frac{h_{batt} \cdot t_{batt} \cdot f_{vb.batt}}{\gamma_{M0}} \qquad V_{bbatt.Rd} = 193.99\,kN \qquad \frac{T_{batt}}{V_{bbatt.Rd}} = 0.15 \quad < 1{,}0$$

$$M_{Rd.batt} := \frac{f_{yb}}{\gamma_{M0}} \cdot \frac{t_{batt} \cdot h_{batt}^2}{6} \qquad M_{Rd.batt} = 11.2\,kN \cdot m \qquad \frac{M_{batt}}{M_{Rd.batt}} = 0.22 \quad < 1{,}0$$

Forces on screws:



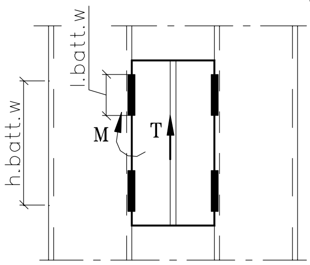
$\boxed{h_{scr} := 150 \cdot mm}$ $\boxed{n_c := 2}$ numbers of screws on one side of one PXK

$$T_{batt} = 29.22\,kN \qquad F := \frac{T_{batt} \cdot h_0}{2 \cdot h_{scr}} \qquad F = 16.11\,kN$$

$$F_{scew} := \sqrt{F^2 + \left(\frac{T_{batt}}{n_c}\right)^2}$$

$F_{scew} = 21.75\,kN$ = force on one screw
< 30 kN

Forces on weld: welding high: $\boxed{h_{batt.w} := 150 \cdot mm}$ welding length: $\boxed{l_{batt.w} := 2 \cdot 40 \cdot mm}$ welding t=: $\boxed{a_{batt.w} := 4 \cdot mm}$



$$\sigma_{batt.w} := \frac{3 \cdot T_{batt} \cdot h_0}{a_{batt.w} \cdot h_{batt.w}^2} \qquad \tau_{batt.w} := \frac{T_{batt}}{l_{batt.w} \cdot a_{batt.w}}$$

$$\sigma_{V.batt.w} := \sqrt{\sigma_{batt.w}^2 + 3 \cdot \tau_{batt.w}^2} \qquad \sigma_{V.batt.w} = 225.77\,\frac{N}{mm^2}$$ = stress in one weld

G. Štítové sloupy B1/1, 8 C1/1, 8

Geometrie:

Zatěžovací šířka sloupku L_w= 5.31 m

Rozpon L= 7.50 m

Zatížení větrem: ČSN EN 1991-1-4 ()

Kategorie terénu: II. Oblasti s nízkou vegetací jako je tráva nebo izolované překážky (stromy, budovy)

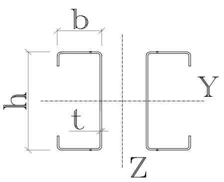
Charakteristická hodnota rychlosti větru: v_{bo}= 25.0 m/s

Maximální dynamický tlak q_p (z)= 0.936 kN/m²

Součinitel vnějšího tlaku (tlak) C_{pe}=	0.96		Součinitel vnějšího tlaku (sání) C_{pe}=	1.10	
Zatížení větrem (tlak) w_e=	0.90	kN/m²	Zatížení větrem (sání) w_e=	1.03	kN/m²
Normové zatížení větrem (tlak) q_n=	**4.77**	kN/m	Normové zatížení větrem (sání) q_n=	**5.46**	kN/m
Součinitel zatížení g_w=	1.5		Součinitel zatížení g_w=	1.5	
Výpočtové zatížení větrem (tlak) q_d=	**7.15**	kN/m	Výpočtové zatížení větrem (sání) q_d=	**8.20**	kN/m

Charakteristika profilu:

Průřez profilem = Dvojitý Profil = C250X4



Výška profilu h=	250	mm	Momentová únosnost M_y=	60.48	kNm
Šířka profilu b=	100	mm	Smyková únosnost V_{wy}=	208.46	kN
Tloušťka t=	4	mm	Smyk v podpoře R_{wy}=	85.15	kN
Mez kluzu oceli f_u=	480	MPa	Efekt. moment setrvačnosti I_{yeff}=	18380000	mm4
Mez pevnosti oceli f_b=	420	MPa	Moment setrvačnosti I_y=	19000000	mm4

Součinitelé materiálu:

Country = Czech

Únosnost průřezu γ_{M0}= 1.00

Únosnost při vzpěru γ_{M1}= 1.00

Únosnost oslabeného průřezu γ_{M2}= 1.25

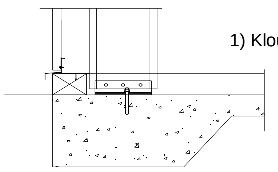
Deformace:

Limit deformace 1/ 250

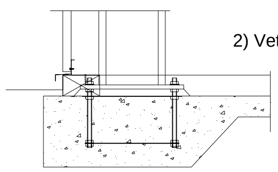
Skutečný poměr 257

Kotvení do spodní stavby:

Vetknuté kotvení



1) Kloubové kotvení



2) Vetknuté kotvení

Předběžné posouzení:

Vetknutý sloup

Sání pro C_{pe}= 1.10 Tlak pro C_{pe}= 0.96

Ohybový moment ve vetknutí M_{Sd}=1/8.q.L²	$M_{Sd,s}$=	57.63	kNm	$M_{Sd,s}$=	50.30	kNm	My =	**120.96** kNm
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				
Ohybový moment v poli M_{Sd}=9/128.q.L²	$M_{Sd,s}$=	32.42	kNm	$M_{Sd,s}$=	28.29	kNm	My =	**120.96** kNm
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				
Podporová reakce R=5/8.q.L	$R_{Sd,s}$=	38.42	kN	$R_{Sd,s}$=	33.53	kN	Ry =	**170.30** kN
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				
Deformace w = q.L4/185.E.ly	w_{max} =	12.11	mm	w_{max} =	10.57	mm	w_{lim} =	**30.00** mm
	PODMÍNKA VYHOVUJE			PODMÍNKA VYHOVUJE				

Double gable pillar B1/1, 8 C1/1, 8

$h \equiv 250 \cdot mm$ $\quad b \equiv 100 \cdot mm$ $\quad c \equiv 39 \cdot mm$ $\quad t \equiv 5 \cdot mm$ $\quad \gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $\quad cc_{batt} := 1500 \cdot mm$ distance between battens

$b_{batt} := 200 \cdot mm$ $\quad t_{batt} := 4 \cdot mm$ $\quad \gamma_{M1} \equiv 1.0$

$h_{batt} := 200 \cdot mm$

$A_{gg} := 2 \cdot A_g$ $\qquad A_{gg} = 4.951 \times 10^3 \, mm^2$

$I_{zz} := 2 \cdot \left[I_z + A_g \cdot \left(e_1 + \frac{d}{2} \right)^2 \right]$ $\qquad I_{zz} = 4.15 \times 10^7 \, mm^4$

$I_{yy} := 2 \cdot I_y$ $\qquad I_{yy} = 4.68 \times 10^7 \, mm^4$

$W_{yy} := 2 \cdot W_y$ $\qquad W_{yy} = 3.82 \times 10^5 \, mm^3$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $\qquad W_{yy.eff} = 3.75 \times 10^5 \, mm^3$

$W_{zz} := \dfrac{I_{zz}}{b + \frac{d}{2}}$ $\qquad W_{zz} = 2.77 \times 10^5 \, mm^3$

$M_{yy.cRk} := 2 \cdot M_{ycRk}$

$M_{yy.cRk} = 187.5 \, kN \cdot m$

single C:

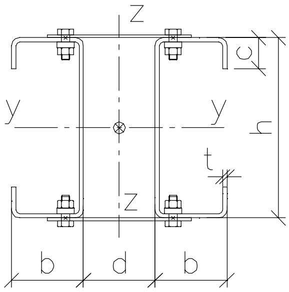
$M_{ycRk} = 93.75 \, kN \cdot m$

$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$N_{cc.Rk} = 1997.23 \, kN$

$f_{yb} = 500 \, \dfrac{N}{mm^2}$

$f_u = 550 \, \dfrac{N}{mm^2}$



$i_{yy} := \sqrt{\dfrac{I_{yy}}{A_{gg}}}$ $\qquad i_{yy} = 97.26 \, mm$ $\qquad i_{zz} := \sqrt{\dfrac{I_{zz}}{A_{gg}}}$ $\qquad i_{zz} = 91.54 \, mm$

$I_{TT} = 42333.33 \, mm^4$ $\qquad$ torsion_plate_pillar = "NO"

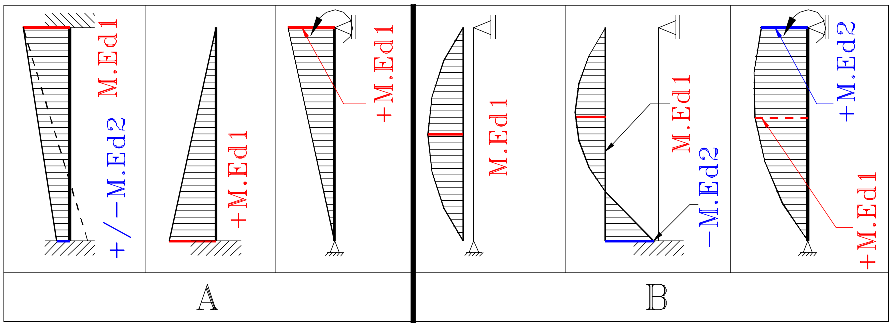
Stresses, global geometry and buckling lengths:

$M_{Ed.1} := 29 \cdot kN \cdot m$ $\quad$ Moment y-y

$M_{Ed.suc.1} := 33 \cdot kN \cdot m$ $\quad$ Moment y-y acting as suction

$M_{Ed.1.z} := 0 \cdot kN \cdot m$ $\quad$ Moment z-z

$N_{Ed} := 43 \cdot kN$



$V_{Ed} := 39 \cdot kN$ > $\quad n_{cross} := 4$ =Numbers of holes in **ONE** web

$d_0 := 12.5 \cdot mm$

Moment_dis := "B" $\qquad M_{Ed.2} := 58 \cdot kN \cdot m$

shift of neutral axis for member in compression: $\qquad \Delta e_N = -4.36 \, mm$

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\qquad \Delta M_{z.shift} = -0.19 \, kN \cdot m$ $\quad M_{z.Ed} := \Delta M_{z.shift}$

Buckling lengths:

$L := 7.5 \cdot m$ $\quad$ Length of pillar $\qquad cc_{wallZ} := L$ $\quad$ = Distance between wall-Z-profile

$L_y := 1.0 \cdot L$ $\quad$ Flexural buckling axis y-y

$L_z := cc_{wallZ}$ $\quad$ Flexural buckling axis z-z (= Distance between wall-Z-profile)

$L_{LT} := cc_{wallZ}$ $\quad$ Distance flange bracings - LTB outer flange in compression for wind acting as pressure

$L_{LT.suc} := 1.0 \cdot L$ $\quad$ Distance flange bracings - LTB inner flange in compression for wind acting as suction

$L_T := cc_{batt}$ $\quad$ Torsional buckling single C-profile between battens.

$b = 100\,mm \quad h = 250\,mm \quad c = 39\,mm \quad t = 5\,mm \quad f_{yb} = 500\,\frac{N}{mm^2} \quad f_u = 550\,\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Flexural buckling resistance - axis y-y and z-z EN 1993-1-3: 6.2.2 and EN 1993-1-1: 6.3.1:

Buckling curve for double C-section: EN 1993-1-3 table 6.3 about:

y-y: a EN1993-1-1,table 6.1: α =0,21 z-z: b EN1993-1-1,table 6.1: α =0,34

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about y-y

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}} \quad \lambda_1 = 64.38 \quad \lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1} \quad \lambda_{r.y.FBcc} = 1.08$

Imperfection factor α relating to bucking curve a $\quad \alpha_y := 0.21$

$\phi_{y.FB} := 0.5 \cdot \left[1 + \alpha_y \cdot \left(\lambda_{r.y.FBcc} - 0.2\right) + \lambda_{r.y.FBcc}^2\right] \quad \phi_{y.FB} = 1.17$

$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^2 - \lambda_{r.y.FBcc}^2}}, 1\right) \quad \chi_{y.FBcc} = 0.61$

Design buckling resistance for buckling mode -flexural buckling-: y-y

$N_{b.Rd.y.FBcc} := \frac{\chi_{y.FBcc} \cdot 2 \cdot A_{eff} \cdot f_{yb}}{\gamma_{M1}} \quad N_{b.Rd.y.FBcc} = 1223.7\,kN$

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about z-z

$\lambda_1 = 64.38 \quad \lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1} \quad \lambda_{r.z.FBcc} = 1.14$

Imperfection factor α relating to bucking curve b $\quad \alpha_z := 0.34$

$\phi_{z.FB} := 0.5 \cdot \left[1 + \alpha_z \cdot \left(\lambda_{r.z.FBcc} - 0.2\right) + \lambda_{r.z.FBcc}^2\right] \quad \phi_{z.FB} = 1.31$

$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^2 - \lambda_{r.z.FBcc}^2}}, 1\right) \quad \chi_{z.FBcc} = 0.51$

Design buckling resistance for buckling mode -flexural buckling-: z-z

$N_{b.Rd.z.FBcc} := \frac{\chi_{z.FBcc} \cdot 2 \cdot A_{eff} \cdot f_{yb}}{\gamma_{M1}} \quad N_{b.Rd.z.FBcc} = 1018.5\,kN$

Lateral-torsional buckling resistance for uniform member in bending EN 1993-1-3: 6.2.4 and EN 1993-1-1: 6.3.2:

Procedure to calculate the elastic critical buckling moment M_{cr}, based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints, is given neither in EN 1993-1-1 nor in EN 1993-1-3. Any appropriate calculation method can be used, here the procedure given in German ENV 1993-1-1 Annex F.

$Moment\_dis = "B" \quad L_{LT} = 7.5\,m \quad k_{M.cr} := 1.0$ (hinged at ends) $\quad k_{w.M.cr} := 1.0$ (no special wrap restraints at ends)

For pillar with end moments: according to picture "A":

$$\psi_{md.A} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases} \quad \psi_{md.A} = 0.5$$

$C_{1.A} := \min\left(1.88 - 1.40 \cdot \psi_{md.A} + 0.52 \cdot \psi_{md.A}^2, 2.7\right)$

$C_{1.A} = 1.31$

$$M_{cr.A} := C_{1.A} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{\left(k_{M.cr} \cdot L_{LT}\right)^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr} \cdot L_{LT}\right)^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}}\right]^{0.5} \quad M_{cr.A} = 138.58\,kN \cdot m$$

For pillar with moment distribution according to picture "B": $\quad C_{1.B} := 1.132$

$$M_{cr.B} := C_{1.B} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{\left(k_{M.cr} \cdot L_{LT}\right)^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr} \cdot L_{LT}\right)^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}}\right]^{0.5} \quad M_{cr.B} = 119.75\,kN \cdot m$$

Elastic critical moment for lateral-torsional buckling

based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints for double C-section:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if } Moment\_dis = "A" \\ M_{cr.B} & \text{if } Moment\_dis = "B" \end{cases} \quad M_{cr} = 120\,kN \cdot m$$

Relative slenderness: $\lambda_{rLT} := \sqrt{\frac{2 \cdot W_{eff.y.1} \cdot f_{yb}}{M_{cr}}} \quad \lambda_{rLT} = 1.25 \quad \alpha_{LT} := 0.34 \quad \phi_{LT} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot \left(\lambda_{rLT} - 0.2\right) + \lambda_{rLT}^2\right] \quad \phi_{LT} = 1.46$

Reduction buckling factor: $\chi_{LT} := \min\left(\frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{rLT}^2}}, 1\right) \quad \chi_{LT} = 0.45$

Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

Desing buckling moment resistance double profile based on effective section modulus W_{eff}.

$M_{b.Rd} := \frac{\chi_{LT} \cdot M_{yy.cRk}}{\gamma_{M1}} \quad M_{b.Rd} = 84.58\,kN \cdot m$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

For pillar when wind acting as suction (moment from line load):

$$M_{cr.suc} := C_{1.B} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{(k_{M.cr} \cdot L_{LT.suc})^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}} \right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr} \cdot L_{LT.suc})^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}} \right]^{0.5} \qquad M_{cr.suc} = 119.75\,kN \cdot m$$

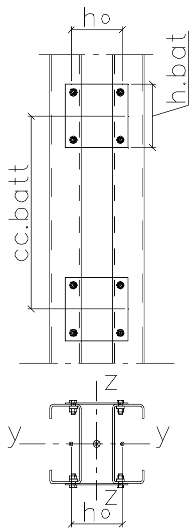
Relative slenderness: $\lambda_{rLT.suc} := \sqrt{\frac{2 \cdot W_{eff.y.1} \cdot f_{yb}}{M_{cr.suc}}}$ $\lambda_{rLT.suc} = 1.25$ $\phi_{LT.suc} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{rLT.suc} - 0.2) + \lambda_{rLT.suc}^2 \right]$ $\phi_{LT.suc} = 1.46$

Reduction buckling factor: $\chi_{LT.suc} := \min\left(\frac{1}{\phi_{LT.suc} + \sqrt{\phi_{LT.suc}^2 - \lambda_{rLT.suc}^2}}, 1 \right)$ $\chi_{LT.suc} = 0.45$

Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

Desing buckling moment resistance double profile for wind acting as suction $M_{b.Rd.suc} := \frac{\chi_{LT.suc} \cdot M_{yy.cRk}}{\gamma_{M1}}$ $M_{b.Rd.suc} = 84.58\,kN \cdot m$

Check Uniform built-up member EN 1993-1-1: 6.4



bow imperfection: $e_0 := \frac{L}{500}$ $\text{if}\left(\frac{cc_{batt}}{i_z} \le 70, \text{"OK"}, \text{"Shorter-cc.bat"} \right) = \text{"OK"}$

non-dimensional slenderness: $\lambda_1 = 64.38$

Effective second moment of area of battened built-up member:

$A_{ch} := A_g$: area of one chord

$h_0 := d + 2 \cdot e_1$ $h_0 = 167\,mm$ distance centroids of chords

$I_{ch} := I_z$: I of one chord

$I_{l.CC} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot I_{ch}$ I built-up member $i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2 \cdot A_{ch}}}$ $\lambda_{CC} := \frac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 81.94$

EN 1993-1-1: table 6.8
Efficiency factor:

$$\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \ge 150 \\ \left(2 - \frac{\lambda_{CC}}{75} \right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \le 75 \end{cases} \qquad \mu_{CC} = 0.91$$

$I_{l.CC} = 4.15 \times 10^7\,mm^4$ $r_{CC} := 2$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu_{CC} \cdot I_{ch}$ $I_{eff} = 4.08 \times 10^7\,mm^4$ effective I of built-up member

Shear stiffness EN 1993-1-1:6.4.3.1:(2) $I_b := \frac{t_{batt} \cdot h_{batt}^3}{12}$ I of batten $n_{batt} := \frac{L}{cc_{batt}}$ $n_{batt} = 5$ number of planes of lacings

$$S_v := \min\left[\left[\frac{24 \cdot E \cdot I_{ch}}{cc_{batt}^2 \cdot \left(1 + \frac{2 \cdot I_{ch}}{n_{batt} \cdot I_b} \cdot \frac{h_0}{cc_{batt}} \right)} \right], \left(\frac{2 \cdot \pi^2 \cdot E \cdot I_{ch}}{cc_{batt}^2} \right) \right] \qquad S_v = 6416\,kN$$

effective critical force of built-up member:

$$N_{cr.CC} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_z^2} \qquad N_{cr.CC} = 1504.85\,kN$$

Maximum moment in middle of built-up member: EN 1993-1-1:6.4.1 (6):

$M_{z.Ed.I} := |M_{Ed.1.z}|$

moment z-z without second order effects

moment with second order effects $M_{z.Ed.C} := \frac{N_{Ed} \cdot e_0 + M_{z.Ed.I}}{1 - \frac{N_{Ed} \cdot \gamma_{M1}}{N_{cr.CC}} - \frac{N_{Ed} \cdot \gamma_{M1}}{S_v}}$ $M_{z.Ed.C} = 0.67\,kN \cdot m$

Compression force in one member: EN 1993-1-1:6.4.1 (6) Compression force in built-up member taking accout to second order effects:

$$X_d := \begin{cases} 1 & \text{if } d \ge 8 \cdot mm \\ 0 & \text{otherwise} \end{cases} \qquad N_{ch.Ed} := \frac{N_{Ed}}{r_{CC}} + \frac{|M_{z.Ed.C}| \cdot h_0 \cdot A_{ch} \cdot X_d}{2 \cdot I_{eff}} \qquad N_{ch.Ed} = 24.88\,kN$$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):
The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3
Moment distribution like shown on figure: Moment_dis = "B" $\psi_{md.A} = 0.5$ $\psi_{m.B} := 0$

$\alpha_{h.2} = 0$ $\alpha_{s.2} = 0.5$ $\psi_{md.z} := 0$ $\alpha_{h.2.z} = 0$ $\psi_{md.suc} := 0$ $\alpha_{h.2.suc} = 0$

$C_{my.2} = 0.6$ $C_{my.2.suc} = 0.95$ $C_{mz.2} = 0.95$ $C_{mLT.2} = 0.6$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$n_{y.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}}$ $n_{y.2} = 0.04$ $n_{z.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}}$ $n_{z.2} = 0.04$

$k_{yy.2} := \min\left[C_{my.2}\cdot(1 + 0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}), C_{my.2}\cdot(1 + 0.6\cdot n_{y.2})\right]$ $k_{yy.2} = 0.61$

$k_{zy.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FBcc}}{C_{mLT.2} - 0.25}\cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25}\cdot n_{z.2}\right)$ $k_{zy.2} = 0.99$

$k_{zz.2} := \min\left[C_{mz.2}\cdot(1 + 0.6\cdot\lambda_{r.z.FBcc}\cdot n_{z.2}), C_{mz.2}\cdot(1 + 0.6\cdot n_{z.2})\right]$ $k_{zz.2} = 0.97$ $k_{yz.2} := k_{zz.2}$

$k_{yy.2.suc} := \min\left[C_{my.2.suc}\cdot(1 + 0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}), C_{my.2.suc}\cdot(1 + 0.6\cdot n_{y.2})\right]$ $k_{yy.2.suc} = 0.97$

$I_{zz.eff} := 2\cdot I_z + A_g\cdot\left(e_1 + \frac{d}{2}\right)^2 + A_{eff}\cdot\left(e_N + \frac{d}{2}\right)^2$ $W_{zz.eff} := \frac{I_{zz.eff}}{b + \frac{d}{2}}$ $I_{zz.eff} = 3.88\times10^7\,mm^4$ $M_{zz.cRk} := W_{zz.eff}\cdot f_{yb}$

$W_{zz.eff} = 2.59\times10^5\,mm^3$

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{Ed}\cdot\gamma_{M0}}{2\cdot N_{c.Rk}} + \frac{\max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.suc.1}|)\cdot\gamma_{M0}}{2\cdot M_{ycRk}} + \frac{|M_{z.Ed.C}|\cdot\gamma_{M0}}{M_{zz.cRk}} = 0.34$$ <1.0

Double CC-profile: Combined M + N + V acc to EN 1993-1-3: 6.1.10

$$\frac{V_{Ed}\cdot\gamma_{M1}}{2\cdot V_{bh.Rk.pillar}} = 0.07$$ < 1.0 according to EN 1993-1-3: 6.1.10 $M\_N\_V_{EC} = 0$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as pressure

Evading in y-y:
$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} + k_{yy.2}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{yz.2}\cdot\frac{|M_{z.Ed.C}|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.46$$ <1.0

Evading in z-z:
$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} + k_{zy.2}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{zz.2}\cdot\frac{|M_{z.Ed.C}|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.73$$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as suction

Evading in y-y:
$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} + k_{yy.2.suc}\cdot\frac{|M_{Ed.suc.1}|\cdot\gamma_{M1}}{\chi_{LT.suc}\cdot M_{yy.cRk}} + k_{yz.2}\cdot\frac{|M_{z.Ed.C}|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.42$$ <1.0

Evading in z-z:
$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} + k_{zy.2}\cdot\frac{|M_{Ed.suc.1}|\cdot\gamma_{M1}}{\chi_{LT.suc}\cdot M_{yy.cRk}} + k_{zz.2}\cdot\frac{|M_{z.Ed.C}|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.44$$ <1.0

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords and battens are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1).

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := \max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.suc.1}|)\cdot 0.5$ $M_{y.max.C} = 29\,kN\cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25\cdot cc_{batt}$ $M_{y.AA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = 26.82\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5\cdot cc_{batt}$ $M_{y.BA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = 27.55\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75\cdot cc_{batt}$ $M_{y.CA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = 28.27\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot M_{y.max.C}}{2.5\cdot M_{y.max.C} + 3\cdot|M_{y.AA}| + 4\cdot|M_{y.BA}| + 3\cdot|M_{y.CA}|}$$

$y_{0.cr} := (y_M + e_{1c.rc})\cdot -1$ $z_{0.cr} := 0\cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot cc_{batt}}{i_z}\right)^2} \qquad \sigma_{ez} = 1295.74\,\frac{N}{mm^2}$$

$$\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{cc_{batt}^2}\right) \qquad \sigma_T = 1074.55\,\frac{N}{mm^2}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_A} := C_{bA}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 398.8\,kN\cdot m$

Relative slenderness:
$$\lambda_{rLT.C\_A} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0\\ 0.2 & \text{if } M_{e.cr.C\_A} = 0\end{cases}$$
$\lambda_{rLT.C\_A} = 0.48$

Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right] \qquad \chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right) \qquad \chi_{LT.C\_A} = 0.89$$

$\phi_{LT.C\_A} = 0.67$

For moment distribution accord. to modell B: $q_{wind\_pillar} := \frac{|M_{Ed.1}\cdot 0.5|\cdot 8}{L^2}$ $M_{y.i}(x_i) := \frac{q_{wind\_pillar}}{2}\cdot(L\cdot x_i - x_i^2)$

Moment at quater point of unbraced segment: $x_{1\_4} := \frac{L - cc_{batt}}{2} + \frac{1}{4}\cdot cc_{batt}$ $x_{1\_4} = 3.38\,m$ $M_{y.i}(x_{1\_4}) = 14.36\,kN\cdot m$ $M_{y.AB} := M_{y.i}(x_{1\_4})$ $M_{y.AB} = 14.36\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 3.75\,m$ $M_{y.i}(x_{1\_2}) = 14.5\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 14.5\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := \frac{L - cc_{batt}}{2} + \frac{3}{4}\cdot cc_{batt}$ $x_{3\_4} = 4.13\,m$ $M_{y.i}(x_{3\_4}) = 14.36\,kN\cdot m$ $M_{y.CB} := M_{y.i}(x_{3\_4})$ $M_{y.CB} = 14.36\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot(|M_{Ed.1}\cdot 0.5|)}{2.5\cdot(|M_{Ed.1}\cdot 0.5|) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := C_{bB}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 384.7\,kN\cdot m$

Relative slenderness:
$$\lambda_{rLT.C\_B} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0\\ 0.2 & \text{if } M_{e.cr.C\_B} = 0\end{cases}$$
$\lambda_{rLT.C\_B} = 0.49$

$$\phi_{LT.C\_B} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_B} - 0.2) + \lambda_{rLT.C\_B}^2\right] \qquad \chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right) \qquad \chi_{LT.C\_B} = 0.89$$

$\phi_{LT.C\_B} = 0.67$

Moment distribution: $Moment\_dis = "B"$
$$\chi_{LT.C} := \begin{cases}\chi_{LT.C\_A} & \text{if } Moment\_dis = "A"\\ \chi_{LT.C\_B} & \text{if } Moment\_dis = "B"\end{cases}$$
$\chi_{LT.C} = 0.89$

Stresses on one member profile at mid-span from above: chord force from above: $N_{ch.Ed} = 24.88\,kN$ *max moment y-y:* $M_{y.max.C} = 29\,kN\cdot m$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Chord at end panel: highest shear force: $r_{CC} = 2$ $V_{Ed.max} := \pi \cdot \frac{M_{z.Ed.C}}{L}$ $V_{Ed.max} = 0.28\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 0.14\,kN$

$V_{bb.Rd} = 274.24\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 5.106 \times 10^{-4}$ The shear force is negligible

Chord at end panel: "corner" moment: $M_{z.ch} := \frac{V_{Ed.max}}{r_{CC}} \cdot \frac{cc_{batt}}{2}$ *maximum moment z-z:* $M_{z.ch} = 0.11\,kN \cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C} \cdot \sin\left(\frac{\pi \cdot cc_{batt}}{L}\right)$ $M_{z.Ed.C.end} = 0.39\,kN \cdot m$ moment due to bow imperfection at end of panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end} \cdot \frac{(h_0 \cdot A_{ch})}{(2 \cdot I_{eff})}$ $N_{ch.end} = 23.49\,kN$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$n_{y.C.2} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(cc_{batt}) \cdot N_{c.Rk}}$ $n_{y.C.2} = 0.02$ $n_{z.C.2} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(cc_{batt}) \cdot N_{c.Rk}}$ $n_{z.C.2} = 0.03$

Equivalent uniform moment factors: Moment_dis = "B"

Moment at start of unbraced segment: $x_{0\_0} := 0 \cdot cc_{batt}$ $M_{y.A0} := 0.5 \cdot M_{y.Ed.ccbatt\_A}(x_{0\_0})$ $M_{y.A0} = 26.1\,kN \cdot m$

Moment at end of unbraced segment: $M_{y.max.C} = 29\,kN \cdot m$

$$\psi_{m.C.A} := \frac{M_{y.A0}}{\begin{cases} M_{Ed.1} \cdot 0.5 & \text{if } |M_{Ed.1}| \geq |M_{Ed.2}| \\ M_{Ed.2} \cdot 0.5 & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}}$$ $\psi_{m.C.A} = 0.9$ range for model "A"

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2} \cdot cc_{batt}$ $x_{1\_2} = 3.75\,m$ $M_{y.i}(x_{1\_2}) = 14.5\,kN \cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 14.5\,kN \cdot m$

Moment at start of unbraced segment: $x_{0\_0} := \frac{L - cc_{batt}}{2} + 0 \cdot cc_{batt}$ $x_{0\_0} = 3\,m$ $M_{y.i}(x_{0\_0}) = 13.92\,kN \cdot m$ $M_{y.B0} := M_{y.i}(x_{0\_0})$ $M_{y.B0} = 13.92\,kN \cdot m$

$\alpha_{h.C.2} := \frac{|M_{y.B0}|}{|M_{y.BB}|}$ $\alpha_{h.C.2} = 0.96$ $\psi_{m.C.B} := 1$ range for model "B"

$$C_{mLT.C.2} := \begin{cases} \max(0.6 + 0.4 \cdot \psi_{m.C.A}, 0.4) & \text{if Moment\_dis = "A"} \\ (0.95 + 0.05 \cdot \alpha_{h.C.2}) & \text{if Moment\_dis = "B"} \end{cases}$$ $C_{mLT.C.2} = 0.998$ $C_{my.C.2} := C_{mLT.C.2}$ $C_{mz.C.2} := 1$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$\lambda_{r.y.FB}(cc_{batt}) = 0.22$ $k_{yy.C.2} := \min\left[C_{my.C.2} \cdot (1 + 0.6 \cdot \lambda_{r.y.FB}(cc_{batt}) \cdot n_{y.C.2}), C_{my.C.2} \cdot (1 + 0.6 \cdot n_{y.C.2})\right]$ $k_{yy.C.2} = 1$

$\lambda_{r.z.FB}(cc_{batt}) = 0.56$ $k_{zz.C.2} := \min\left[C_{mz.C.2} \cdot (1 + 0.6 \cdot \lambda_{r.z.FB}(cc_{batt}) \cdot n_{z.C.2}), C_{mz.C.2} \cdot (1 + 0.6 \cdot n_{z.C.2})\right]$ $k_{zz.C.2} = 1.01$

$$k_{zy.C.2} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FB}(cc_{batt})}{C_{my.C.2} - 0.25} \cdot n_{z.C.2}, 1 - \frac{0.05}{C_{mLT.C.2} - 0.25} \cdot n_{z.C.2}\right)$$ $k_{zy.C.2} = 1$ $k_{yz.C.2} := k_{zz.C.2}$

Controll single C-profile between battens:

$M_{y.max.C} = 29\,kN \cdot m$ $M_{z.ch} = 0.11\,kN \cdot m$ $\Delta M_{z.shift} := |\Delta e_N \cdot N_{ch.end}|$ $\Delta M_{z.shift} = 0.1\,kN \cdot m$ $N_{ch.end} = 23.49\,kN$

Buckling moment resistance y-y EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C} \cdot M_{ycRk}$ $\chi_{LT.C} = 0.89$ $M_{y.Rk.C} = 83.16\,kN \cdot m$

Buckling resistance moment z-z EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk})$ $M_{z.Rk.C} = 26.77\,kN \cdot m$

Resistance buckling mode flexural buckling-: y-y $cc_{batt} = 1.5\,m$ $\chi_{y.FB}(cc_{batt}) = 0.99$ $N_{b.Rk.y.FB}(cc_{batt}) = 993.25\,kN$

Resistance buckling mode flexural buckling-: z-z $cc_{batt} = 1.5\,m$ $\chi_{z.FB}(cc_{batt}) = 0.86$ $N_{b.Rk.z.FB}(cc_{batt}) = 856.42\,kN$

Resistance buckling mode -torsional or torsional-flexural $cc_{batt} = 1.5\,m$ $\chi_{TF}(cc_{batt}, cc_{batt}) = 0.82$ $N_{b.Rk.TF}(cc_{batt}, cc_{batt}) = 821.9\,kN$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

single C-profile: Axial compression EN 1993-1-1: 6.3.1 (3): Buckling about relevant axis in mid-span or en panel of built-up member

$$\frac{\max(N_{ch.Ed}, N_{ch.end})\cdot\gamma_{M1}}{\min(\chi_{y.FB}(cc_{batt}), \chi_{z.FB}(cc_{batt}), \chi_{TF}(cc_{batt}, cc_{batt}))\cdot N_{c.Rk}} = 0.03$$ <1.0

Controll buil-up member: single C-profile (chords) in mid-span of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.Ed}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.34$$ <1.0

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.38$$ <1.0

Evading in z-z:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.38$$ <1.0

Controll buil-up member: single C-profile (chords) at end panel of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.end}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.34$$ <1.0

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.38$$ <1.0

Evading in z-z:

$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.38$$ <1.0

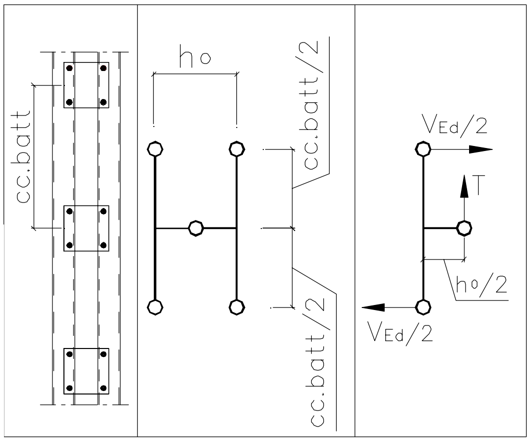
$b = 100\,mm \quad h = 250\,mm \quad c = 39\,mm \quad t = 5\,mm \qquad f_{yb} = 500\,\frac{N}{mm^2} \quad f_u = 550\,\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Forces on battens: max shear flow in one batten:

$$T_{batt} := \frac{V_{Ed.max} \cdot cc_{batt}}{h_0 \cdot r_{CC}} \qquad T_{batt} = 1.26\,kN$$

max moment in one batten:

$$M_{batt} := \frac{T_{batt} \cdot h_0}{2} \qquad M_{batt} = 0.11\,kN \cdot m$$

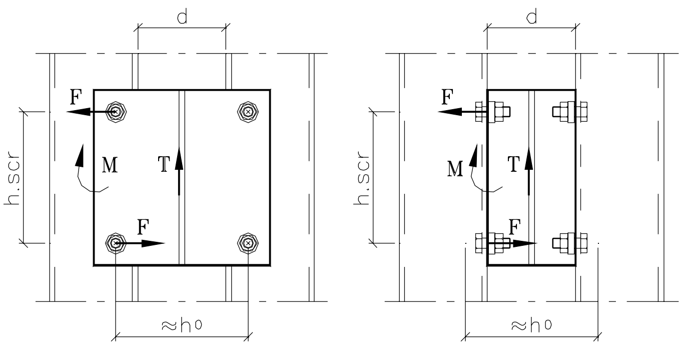


$$\lambda_{w.batt} := 0.346 \cdot \frac{h_{batt}}{t_{batt}} \cdot \sqrt{\frac{f_{yb}}{E}} \qquad f_{vb.batt} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{w.batt} \le 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{w.batt}} & \text{if } 0.83 < \lambda_{w.batt} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{w.batt}^2} & \text{if } \lambda_{w.batt} \ge 1.40 \end{cases}$$

$$V_{bbatt.Rd} := \frac{h_{batt} \cdot t_{batt} \cdot f_{vb.batt}}{\gamma_{M0}} \qquad V_{bbatt.Rd} = 227.45\,kN \qquad \frac{T_{batt}}{V_{bbatt.Rd}} = 0.01 \quad < 1,0$$

$$M_{Rd.batt} := \frac{f_{yb}}{\gamma_{M0}} \cdot \frac{t_{batt} \cdot h_{batt}^2}{6} \qquad M_{Rd.batt} = 13.33\,kN \cdot m \qquad \frac{M_{batt}}{M_{Rd.batt}} = 0.01 \quad < 1,0$$

Forces on screws:



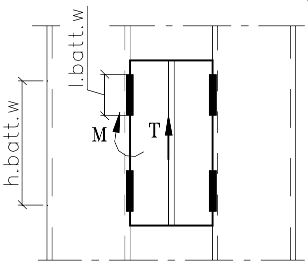
$\boxed{h_{scr} := 150 \cdot mm}$ $\boxed{n_c := 2}$ numbers of screws on one side of one PXK

$$T_{batt} = 1.26\,kN \qquad F := \frac{T_{batt} \cdot h_0}{2 \cdot h_{scr}} \qquad F = 0.7\,kN$$

$$F_{scew} := \sqrt{F^2 + \left(\frac{T_{batt}}{n_c}\right)^2}$$

$F_{scew} = 0.94\,kN$ = force on one screw
< 30 kN

Forces on weld:



welding high: $\boxed{h_{batt.w} := 150 \cdot mm}$ welding length: $\boxed{l_{batt.w} := 2 \cdot 40 \cdot mm}$ welding t=: $\boxed{a_{batt.w} := 4 \cdot mm}$

$$\sigma_{batt.w} := \frac{3 \cdot T_{batt} \cdot h_0}{a_{batt.w} \cdot h_{batt.w}^2} \qquad \tau_{batt.w} := \frac{T_{batt}}{l_{batt.w} \cdot a_{batt.w}}$$

$$\sigma_{V.batt.w} := \sqrt{\sigma_{batt.w}^2 + 3 \cdot \tau_{batt.w}^2} \qquad \sigma_{V.batt.w} = 9.77\,\frac{N}{mm^2}$$

= stress in one weld

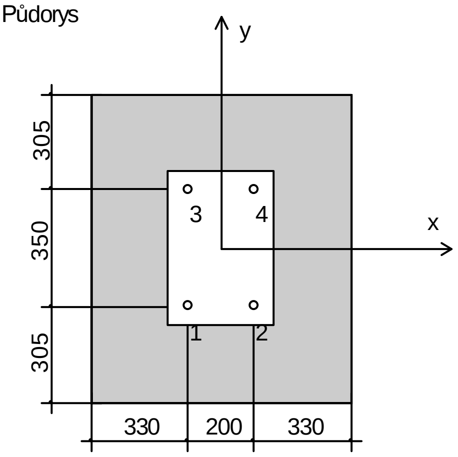
HILTI	Firma: LLENTAB	Strana 1 z 1
	Vypracoval: Kamil Patrman	Zákazník: Hala Zátor
Aplikace	Adresa:	Projekt: CZ1621
PROFIS kotvy 1.8.0	Telefon/Fax: - / -	Kontaktní osoba:
http://www.hilti.com/	E-mail: kamil.patrman@llentab.cz	Datum: - / 04.02.2022

Poznámky:

Typ a rozměr kotvy:	**HIT-RE 500 + HAS-E (8.8)-M27**
Efektivní hloubka kotvení:	h_{ef}= 240 mm; součinitel hloubky osazení = 1.800
Materiál:	8.8
Certfikát:	
Platnost:	- / -
Zkouška:	Návrh podle SOFA - po ETAG zkoušce
Distanční montáž:	s upevnění na povrchu; plně podlití (kotevní deska); e_b = 50 mm; t = 20 mm
Kotevní deska:	S355 (ST52); ; l_x x l_y x t = 320 x 460 x 20 mm
Základní materiál:	netrhlinový Beton C20/25, f_{cc} = 25.00 N/mm$_²$; h = 10000 mm
Výztuž:	vzdálenost výztuže >= 150 mm
	s okrajovou podélnou výztuží d >= 12 mm + uzavřená výztuž třmínkového typu s <= 100 mm

Geometrie [mm]

Půdorys

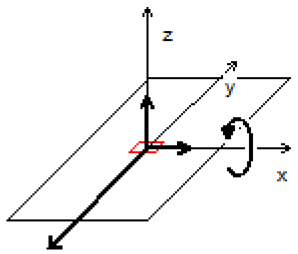


Zatížení [kN]

Výslené zatížení [kN, kNm]

N = 13.00 V_y = -35.00

M_z = 0.00 M_y = 0.00



Exentricita [mm]

e_x = 0 V_x = 10.00

e_y = 0 M_x = 53.00

Posouzení/Úroveň (Upravené případy)

		Výpočtová hodnota [kN]		Využití [%]	
Zatížení	Zkouška	Zatížení	Kapacita	β_N/β_V	Status
Tah	Betonový kužel	151.37	324.89	47 / -	OK
Smyk	Únosnost oceli (distanční montáž)	9.10	21.29	- / 43	OK

Zatížení	β_N	β_V	α	Využití $\beta_{N,V}$[%]	Status
Interakce	0.466	0.428	1.5	60	OK

Upozornění

Při použití HILTI dynamického setu se smykové zatížení distribuuje do kotev rovnoměrně
Za kompaktibilitu se současnými normami (např.EC3) zodpovídá uživatel
Předpokládá se suchá díra a standardní vyčištění! Vliv teploty je zanedbán!
Toto je nestandardní výška upevnění. Prosím kontaktuje poradce Hilti a zjistěte možnost dodání.

Upevnění je bezpečné!

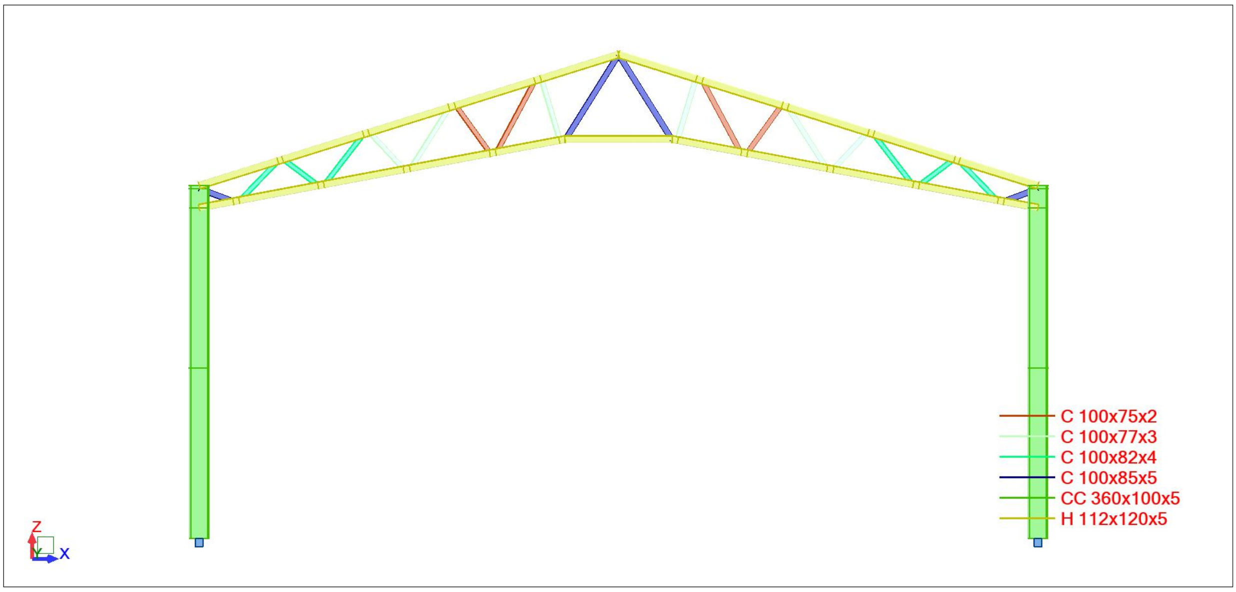
Vložené údaje překontrolujte jestli odpovídají skutečným podmínkám a záměru, pro které je chcete použít.

J.

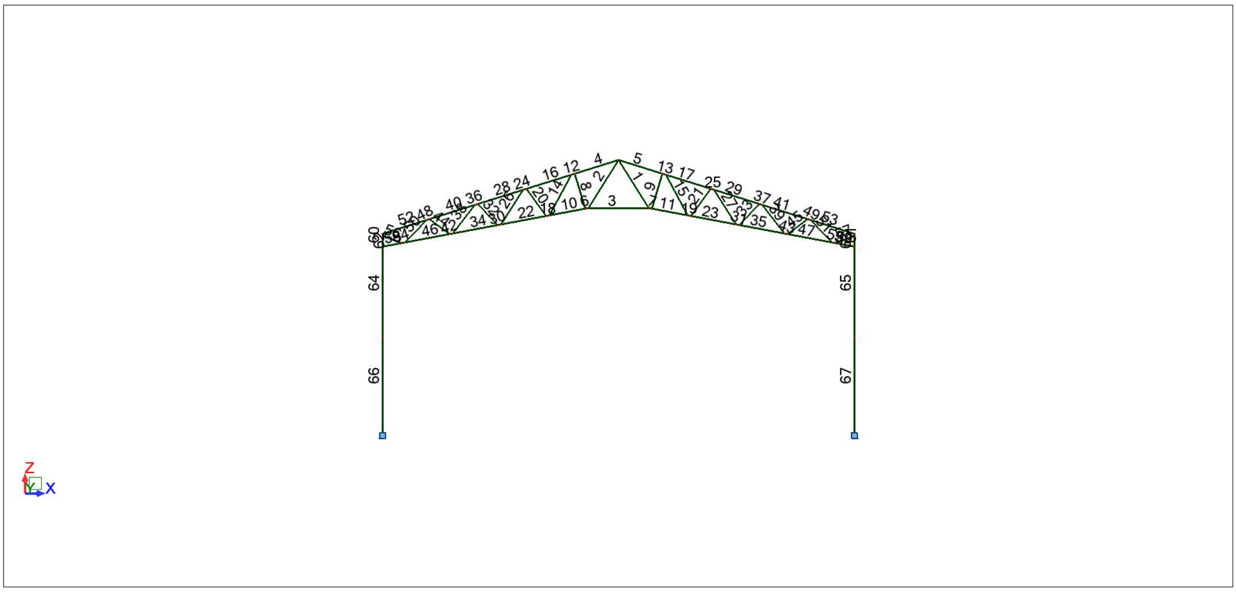
Část konstrukce: FR1 Rám standard

Poznámky:
Zatěžovací šířka rámu Lw=7m.

View - Cases: 101 (Self weight frame)



View - Cases: 101 (Self weight frame) 1



Combinations

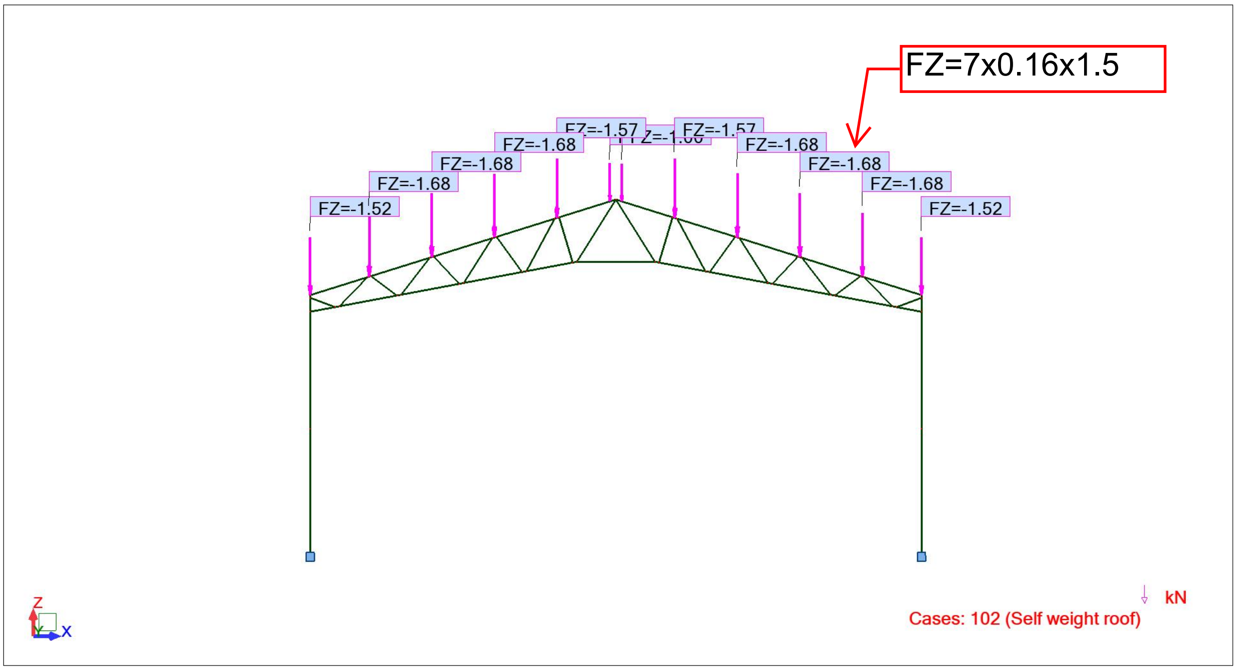
- Cases: 10 11 20 30to32 40to42 50 90to93

Combinations	Name	Analysis type	Combination	Case nature
10	Snow	Nonlin. Combinat	ULS	snow
11	Snow 0.5 right	Nonlin. Combinat	ULS	snow
20	Wind Cpe max	Nonlin. Combinat	ULS	wind
30	Snow + reduced	Nonlin. Combinat	ULS	snow

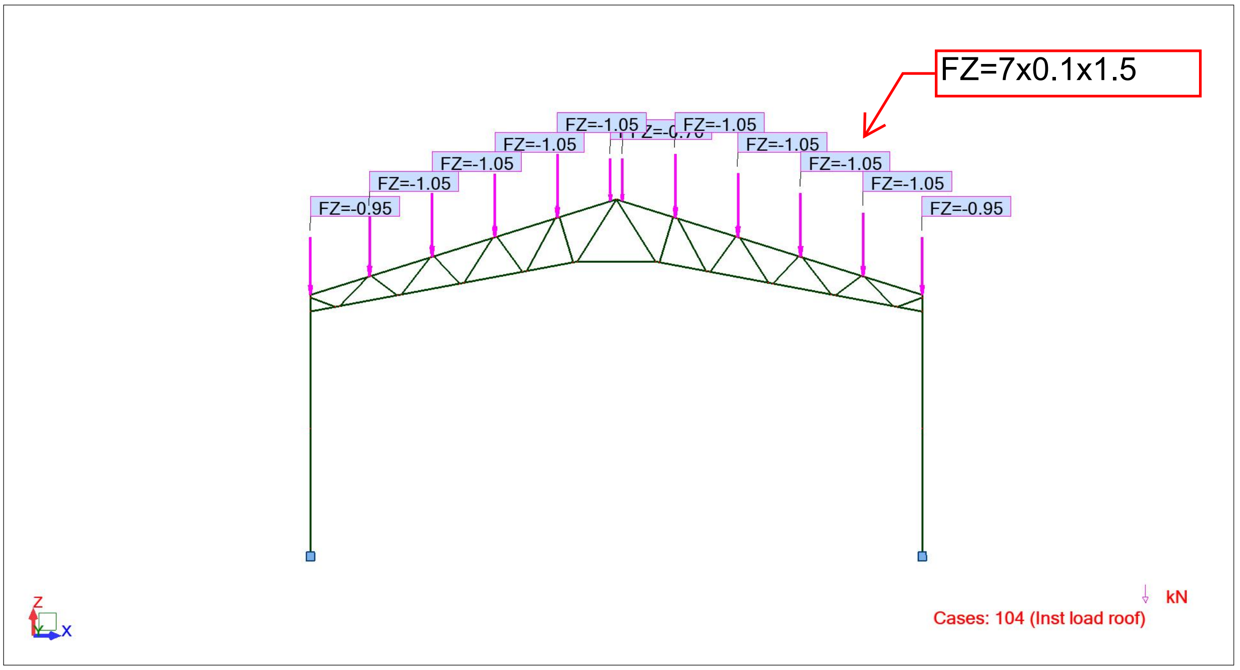
Combinations	Name	Analysis type	Combination	Case nature
31	Snow 0.5 left + r	Nonlin. Combinat	ULS	snow
32	Snow 0.5 right +	Nonlin. Combinat	ULS	snow
40	Wind Cpe min +	Nonlin. Combinat	ULS	snow
41	Wind Cpe min +	Nonlin. Combinat	ULS	snow
42	Wind Cpe min +	Nonlin. Combinat	ULS	snow
50	Wind Cpe max fr	Nonlin. Combinat	ULS	snow
90	SLS Characterist	Nonlin. Combinat	ULS	snow
91	SLS Characterist	Nonlin. Combinat	ULS	snow
92	SLS Characterist	Nonlin. Combinat	ULS	snow
93	SLS Characterist	Nonlin. Combinat	ULS	snow

Combinations	Definition
10	(101+102+104)*1.35+(201+202)*1.50
11	(101+102+104)*1.35+201*1.50+202*0.75
20	(101+102)*1.00+(300+321)*1.50
30	(101+102+104)*1.35+(201+202)*1.50+(301+320)*
31	(101+102+104)*1.35+201*0.75+202*1.50+(301+3
32	(101+102+104)*1.35+201*1.50+202*0.75+(301+3
40	(101+102+104)*1.35+(201+202)*0.75+(301+320)*
41	(101+102+104)*1.35+201*0.38+202*0.75+(301+3
42	(101+102+104)*1.35+201*0.75+202*0.38+(301+3
50	(101+102)*1.00+(310+321)*1.50
90	(101+102+104+201+202)*1.00
91	(101+102+300+321)*1.00
92	(101+102+201+202)*1.00+(301+320)*0.60
93	(101+102+301+320)*1.00+(201+202)*0.50

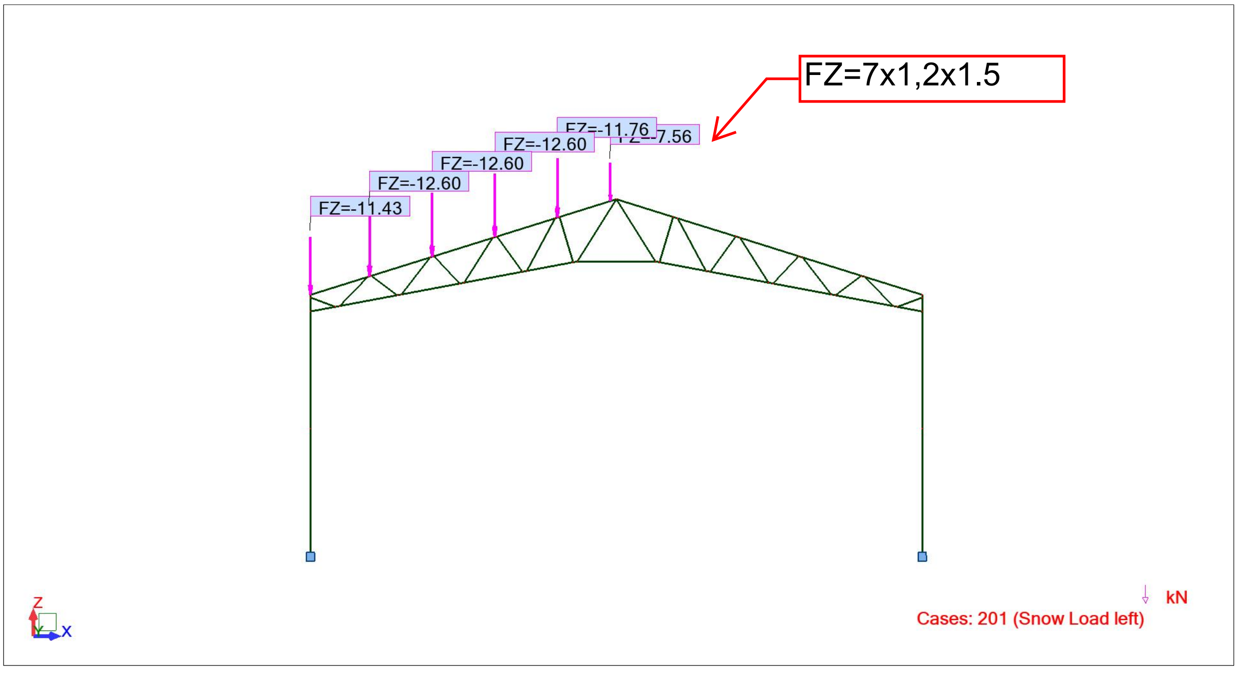
View - Cases: 102 (Self weight roof)



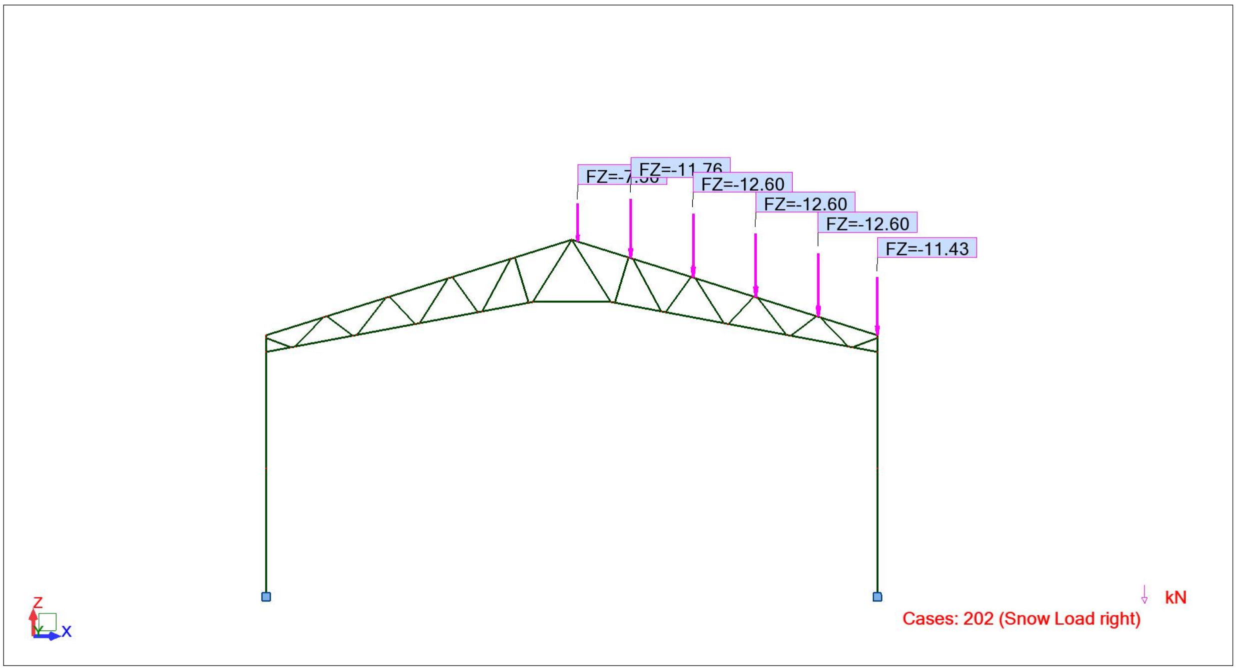
View - Cases: 104 (Inst load roof)



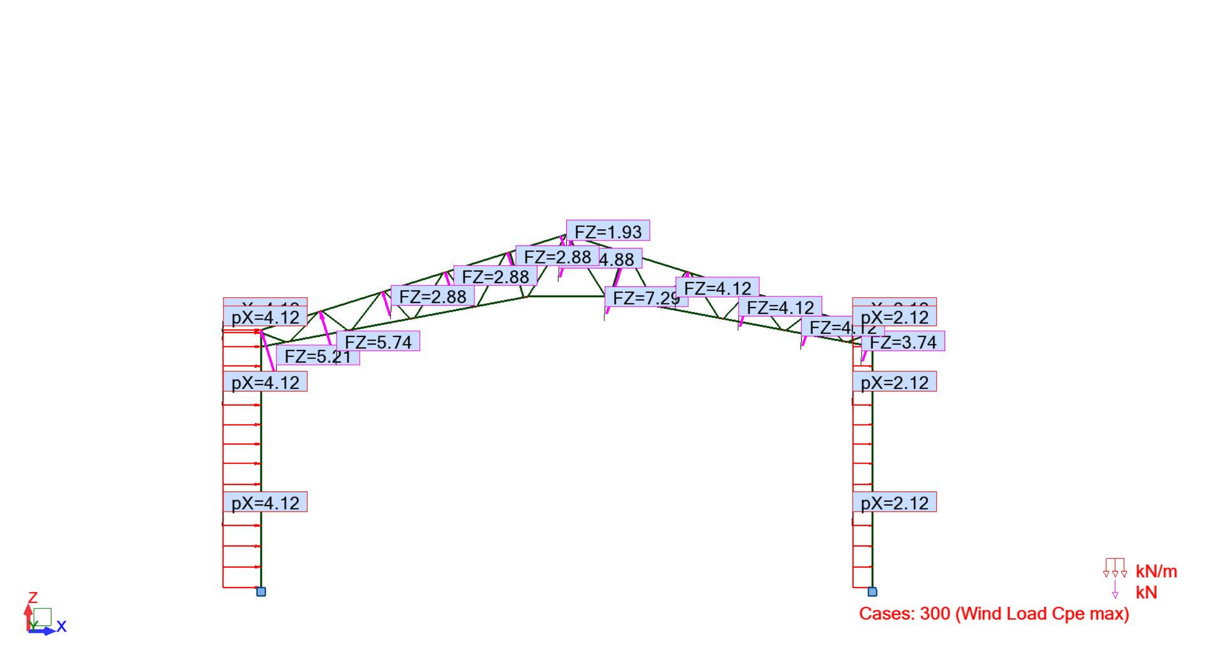
View - Cases: 201 (Snow Load left)



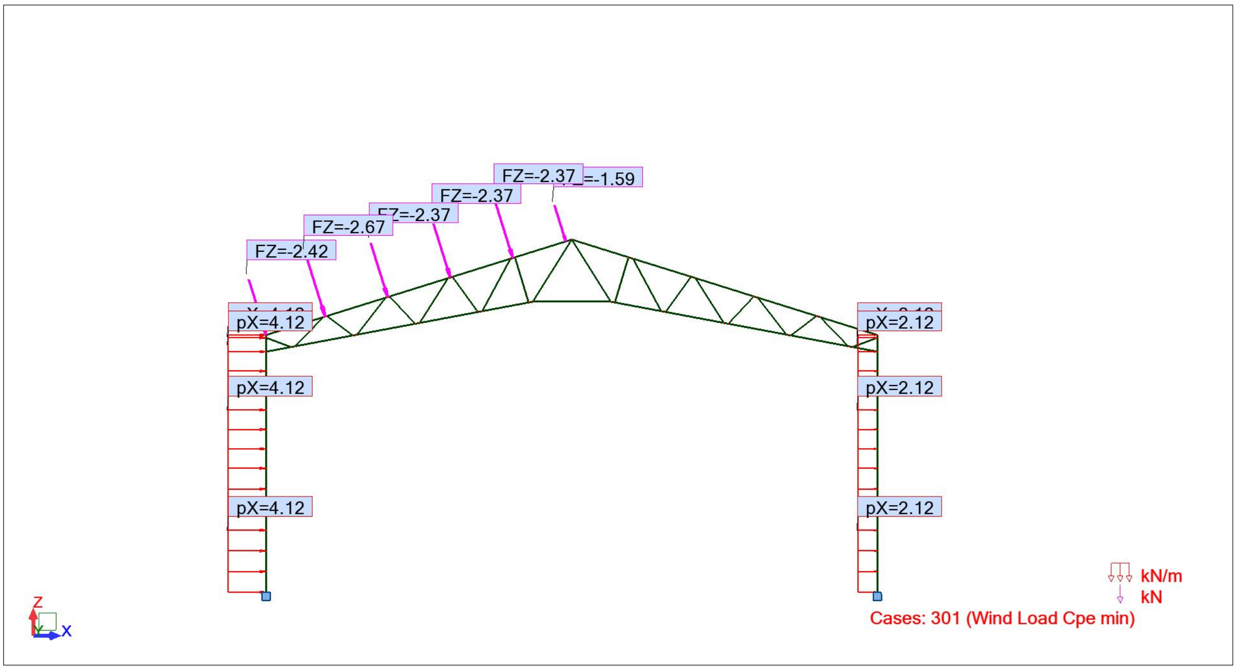
View - Cases: 202 (Snow Load right)



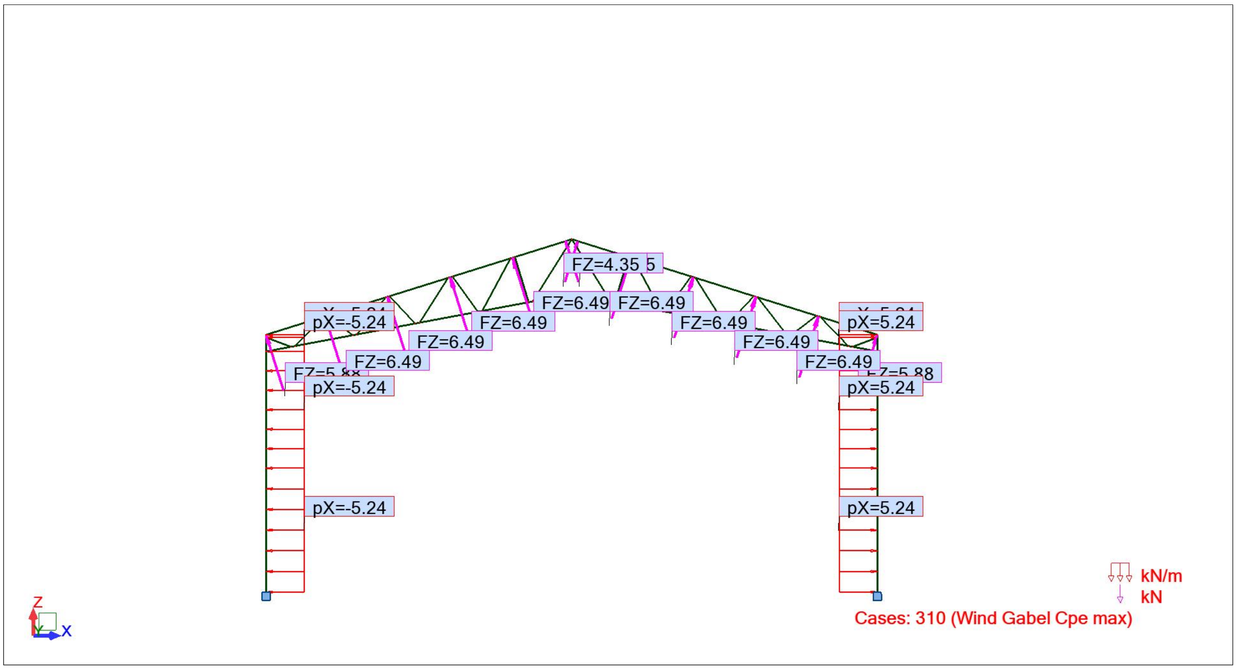
View - Cases: 300 (Wind Load Cpe max)



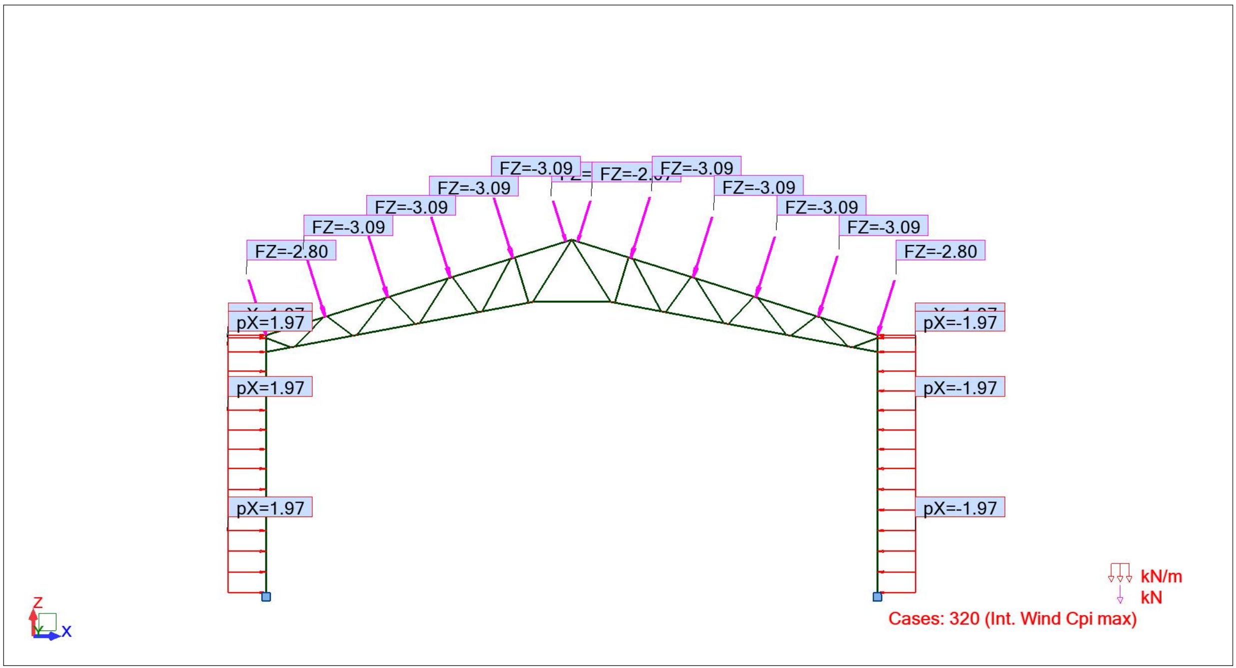
View - Cases: 301 (Wind Load Cpe min)



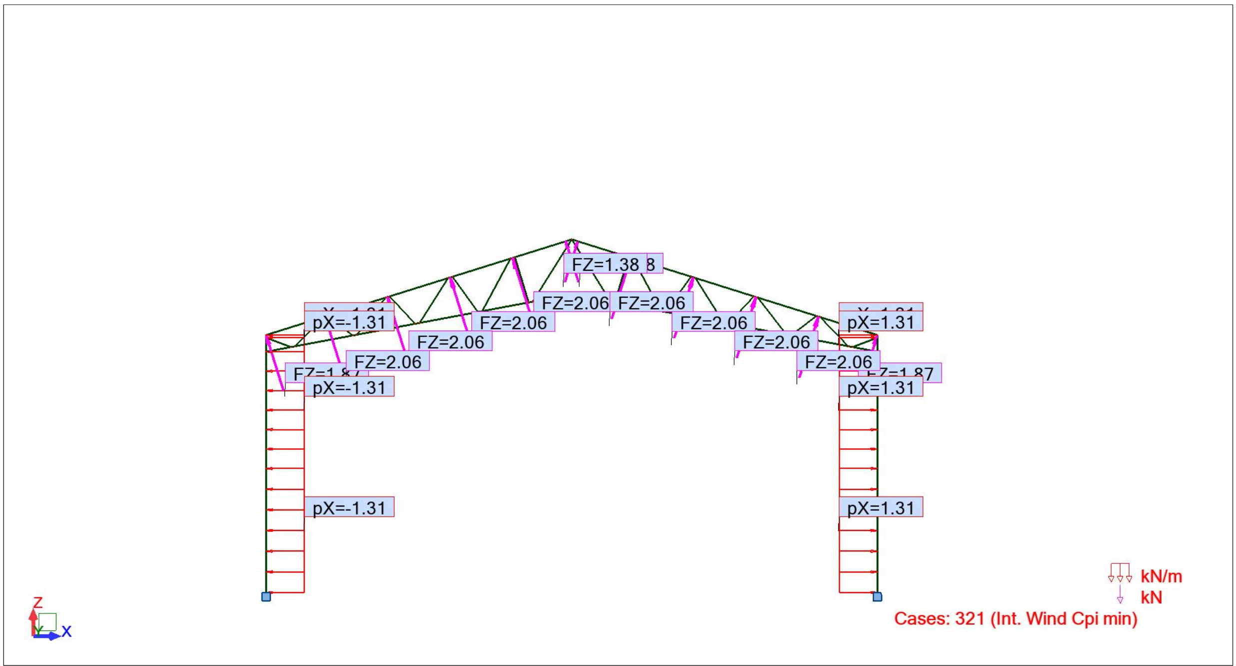
View - Cases: 310 (Wind Gabel Cpe max)



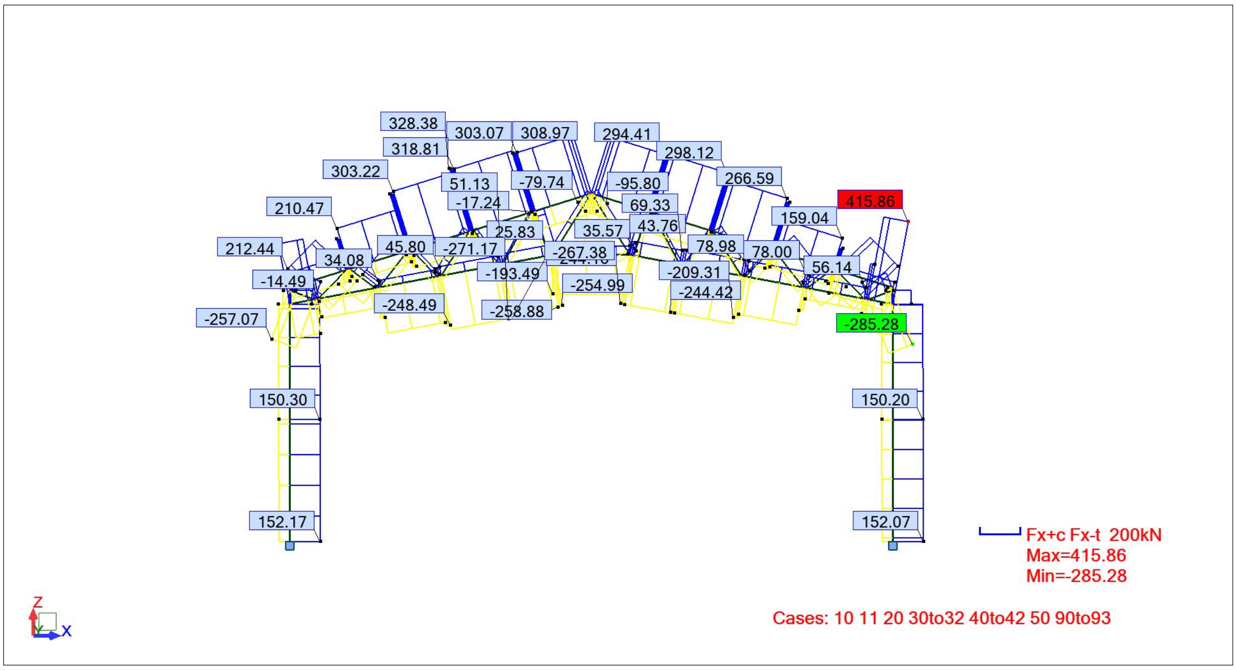
View - Cases: 320 (Int. Wind Cpi max)



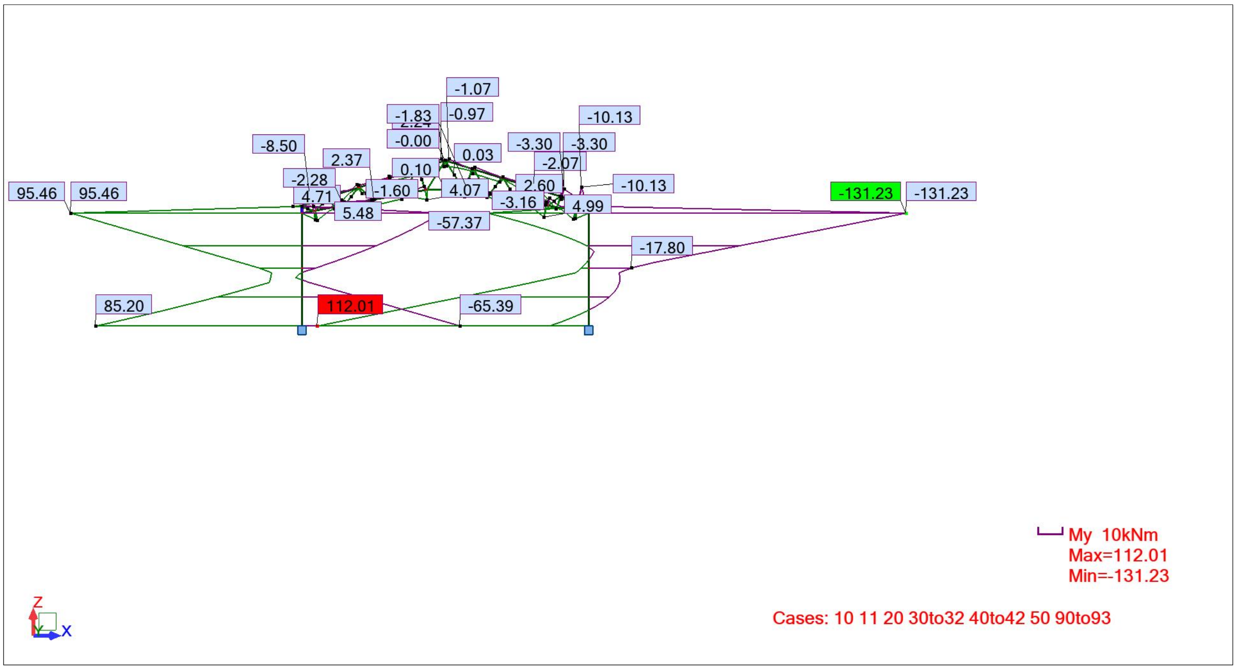
View - Cases: 321 (Int. Wind Cpi min)



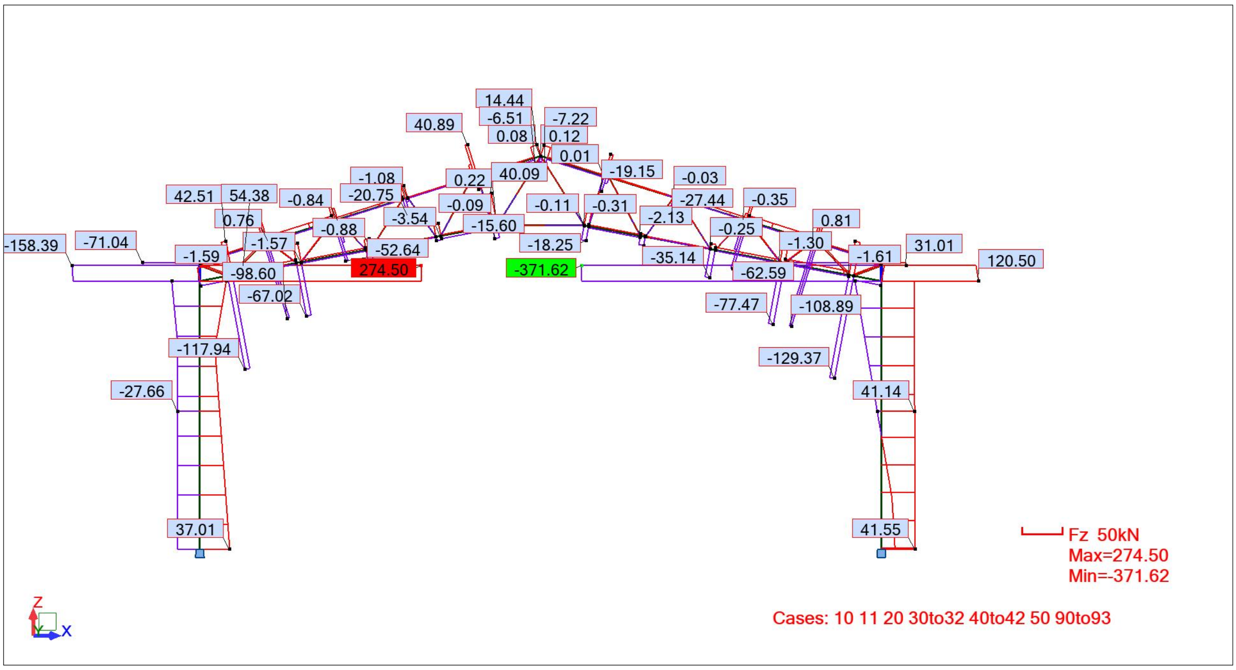
View - FX; Cases: 10 11 20 30to32 40to42 50 90to93



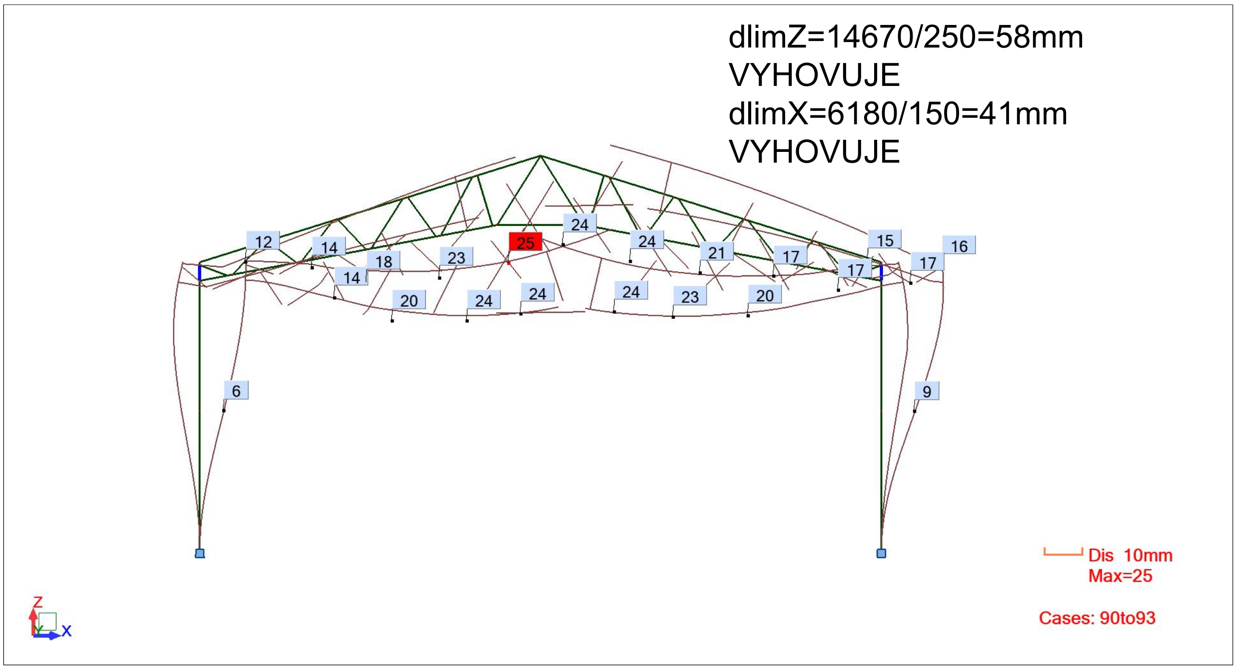
View - MY; Cases: 10 11 20 30to32 40to42 50 90to93



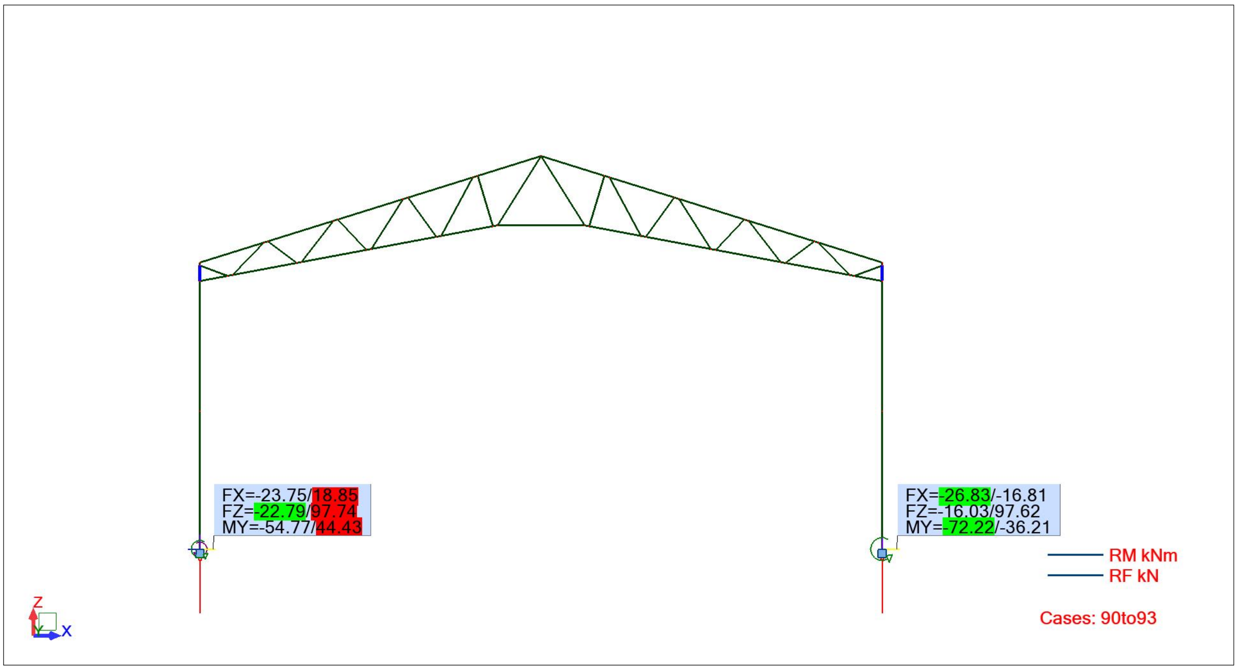
View - FZ; Cases: 10 11 20 30to32 40to42 50 90to93



View - Deformation; Cases: 90to93



View - Reaction forces(kN);Reaction moments(kN*m); Cases: 90to93



Vazník FR1 diagonály

Popis:

Poznámky:

Datum: 2022.01.18 @ 14:17:22

Importováno ze souboru: CZ1621 FR1.rtd

:

Diagonály

57 66 % Spoj: Střih 30 Snow + reduced wind Cpe min

Parametry posouzení prvků:

Typ	Popis	L_y	Koef.	L_y,koef	Křivka Y	L_z	Koef.	L_z,koef	Křivka Z	L_ft	L_t	Tenkost.
Bottom	Bottom beam 6.0m	6	False	1	b	0.7	True	1	b	True	True	True
Strut	Strut 2 screw	1	True	1	b	0.7	True	1	b	True	False	True
Top	Top beam	1	True	1	b	0.7	True	1	b	True	True	True
Other	Simple member	1	True	0	auto	1	True	0	auto	False	False	False
Strut	Strut 1 screw	1	True	1	b	0.7	True	1	b	True	False	True
Other	Frame distributed	0.33	False	1	b	0.5	True	1	b	True	False	True
Top	Top beam SPH	0.33	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 1.5m	1.5	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 3.0m	3	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 4.5m	4.5	False	1	b	0.7	True	1	b	True	True	True

Materiál:

Prvek	Typ	Member type	Profil	Délka [m]	Materiál	Hmotnost [kg]	A [m²]	A_eff [m²]	Komb. N_Ed
1	Strut	Strut 2 screw	C 100x85x5	1.762	HX500LAD	11.45	1580	1580	30
2	Strut	Strut 2 screw	C 100x85x5	1.761	HX500LAD	11.45	1580	1580	30
8	Strut	Strut 2 screw	C 100x77x3	1.134	S350GD	6.21	840	840	32
9	Strut	Strut 2 screw	C 100x77x3	1.134	S350GD	6.21	840	840	31
14	Strut	Strut 1 screw	C 100x75x2	1.475	S350GD	4.14	552	552	42
15	Strut	Strut 1 screw	C 100x75x2	1.475	S350GD	4.14	552	472.35	32
20	Strut	Strut 1 screw	C 100x75x2	1.059	S350GD	4.14	552	472.35	42
21	Strut	Strut 1 screw	C 100x75x2	1.059	S350GD	4.14	552	552	32
26	Strut	Strut 2 screw	C 100x77x3	1.274	S350GD	6.21	840	840	10
27	Strut	Strut 2 screw	C 100x77x3	1.271	S350GD	6.21	840	840	30
32	Strut	Strut 2 screw	C 100x77x3	0.898	S350GD	6.21	840	840	10
33	Strut	Strut 2 screw	C 100x77x3	0.899	S350GD	6.21	840	840	30
38	Strut	Strut 2 screw	C 100x82x4	1.123	HX420LAD	8.99	1166.8	1166.8	30
39	Strut	Strut 2 screw	C 100x82x4	1.122	HX420LAD	8.99	1166.8	1166.8	30
44	Strut	Strut 2 screw	C 100x82x4	0.776	HX420LAD	8.99	1166.8	1166.8	30
45	Strut	Strut 2 screw	C 100x82x4	0.775	HX420LAD	8.99	1166.8	1166.8	30
50	Strut	Strut 2 screw	C 100x82x4	0.945	HX420LAD	8.99	1166.8	1166.8	30
51	Strut	Strut 2 screw	C 100x82x4	0.95	HX420LAD	8.99	1166.8	1166.8	30
56	Strut	Strut 2 screw	C 100x85x5	0.671	HX500LAD	11.45	1580	1580	30

57	Strut	Strut 2 screw	C 100x85x5	0.668	HX500LAD	11.45	1580	1580	30

Profily:

Prvek	Profil	N_c [%]	N_t [%]	NcMz [%]	NcMz2 [%]	V_y [%]	N_t [%]	N_b [%]	NyMy Mz [%]	NzMy Mz [%]	Profil [%]	Rozh. profil	Komb. N_Ed
1	C 100x85x5	0	10	10	10	0	0	0	0	0	10	Prostý tah	30
2	C 100x85x5	0	12	12	11	0	0	0	0	0	12	Prostý tah	30
8	C 100x77x3	15	0	0	0	0	13	19	0	0	19	Vzpěr	32
9	C 100x77x3	10	0	0	0	0	13	13	0	0	13	Štíhlost	31
14	C 100x75x2	0	12	12	12	0	0	0	0	0	12	Prostý tah	42
15	C 100x75x2	9	0	9	8	0	17	12	10	10	12	Štíhlost	32
20	C 100x75x2	11	0	11	10	0	12	13	12	12	13	Vzpěr	42
21	C 100x75x2	0	6	7	6	0	0	0	0	0	7	Tah a ohyb	32
26	C 100x77x3	13	0	0	0	0	15	17	0	0	17	Vzpěr	10
27	C 100x77x3	16	0	0	0	0	15	22	0	0	22	Vzpěr	30
32	C 100x77x3	0	11	11	11	0	0	0	0	0	11	Prostý tah	10
33	C 100x77x3	0	15	15	14	0	0	0	0	0	15	Prostý tah	30
38	C 100x82x4	21	0	0	0	0	13	27	0	0	27	Vzpěr	30
39	C 100x82x4	25	0	0	0	1	13	32	0	0	32	Vzpěr	30
44	C 100x82x4	0	18	19	18	1	0	0	0	0	19	Tah a ohyb	30
45	C 100x82x4	0	21	22	21	1	0	0	0	0	22	Tah a ohyb	30
50	C 100x82x4	43	0	0	0	1	12	52	0	0	52	Vzpěr	30
51	C 100x82x4	48	0	0	0	1	12	58	0	0	58	Vzpěr	30
56	C 100x85x5	0	31	31	31	1	0	0	0	0	31	Prostý tah	30
57	C 100x85x5	0	35	35	35	1	0	0	0	0	35	Prostý tah	30

Spoje:

Prvek	Profil	Spoj	N_t,Ed [kN]	N_c,Ed [kN]	F_v [%]	F_b,c [%]	F_b,t [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
1	C 100x85x5	C54	-79.74	35.57	46	13	30	16	31	46	Střih
2	C 100x85x5	C54	-95.8	25.83	55	10	36	19	37	55	Střih
8	C 100x77x3	C36v2	-17.24	43.61	17	26	12	9	11	26	Otlačení
9	C 100x77x3	C36v2	-20.16	30.28	12	18	14	10	13	18	Otlačení
14	C 100x75x2	C26v2	-24.23	7.89	9	7	25	19	23	25	Otlačení
15	C 100x75x2	C26v2	-6.45	14.09	5	13	7	5	6	13	Otlačení
20	C 100x75x2	C24v2	-5.44	17.64	10	22	7	4	6	22	Otlačení
21	C 100x75x2	C24v2	-13	2.96	7	4	16	10	15	16	Otlačení
26	C 100x77x3	C36v2	-19.05	36.79	14	22	13	10	12	22	Otlačení
27	C 100x77x3	C36v2	-9.63	48.15	18	29	7	5	6	29	Otlačení
32	C 100x77x3	C36v2	-34.99	17.13	13	10	24	18	22	24	Otlačení
33	C 100x77x3	C36v2	-45.42	9.28	17	6	31	23	29	31	Otlačení
38	C 100x82x4	C46v2	-37.85	104.34	40	42	17	11	15	42	Otlačení
39	C 100x82x4	C46v2	-35.88	121.84	47	49	16	11	14	49	Otlačení
44	C 100x82x4	C46v2	-95.41	33.56	37	14	43	28	38	43	Otlačení
45	C 100x82x4	C46v2	-110.94	32.38	43	13	50	33	44	50	Otlačení
50	C 100x82x4	C410v2	-75.9	212.44	49	54	21	23	21	54	Otlačení

51	C 100x82x4	C410v2	-75.9	233.91	54	60	21	23	21	60	Otlačení
56	C 100x85x5	C510v2	-257.07	93.53	59	16	49	51	49	59	Střih
57	C 100x85x5	C510v2	-285.28	91.99	66	16	54	57	55	66	Střih

Souhrn:

Prvek	Member type	Profil	Materiál	Komb. N_Ed	Profil [%]	Spoj [%]	Vyhodnoceni	Posouzení
1	Strut 2 screw	C 100x85x5	HX500LAD	30	10	46	Střih	Vyhovuje
2	Strut 2 screw	C 100x85x5	HX500LAD	30	12	55	Střih	Vyhovuje
8	Strut 2 screw	C 100x77x3	S350GD	32	19	26	Otlačení	Vyhovuje
9	Strut 2 screw	C 100x77x3	S350GD	31	13	18	Otlačení	Vyhovuje
14	Strut 1 screw	C 100x75x2	S350GD	42	12	25	Otlačení	Vyhovuje
15	Strut 1 screw	C 100x75x2	S350GD	32	12	13	Otlačení	Vyhovuje
20	Strut 1 screw	C 100x75x2	S350GD	42	13	22	Otlačení	Vyhovuje
21	Strut 1 screw	C 100x75x2	S350GD	32	7	16	Otlačení	Vyhovuje
26	Strut 2 screw	C 100x77x3	S350GD	10	17	22	Otlačení	Vyhovuje
27	Strut 2 screw	C 100x77x3	S350GD	30	22	29	Otlačení	Vyhovuje
32	Strut 2 screw	C 100x77x3	S350GD	10	11	24	Otlačení	Vyhovuje
33	Strut 2 screw	C 100x77x3	S350GD	30	15	31	Otlačení	Vyhovuje
38	Strut 2 screw	C 100x82x4	HX420LAD	30	27	42	Otlačení	Vyhovuje
39	Strut 2 screw	C 100x82x4	HX420LAD	30	32	49	Otlačení	Vyhovuje
44	Strut 2 screw	C 100x82x4	HX420LAD	30	19	43	Otlačení	Vyhovuje
45	Strut 2 screw	C 100x82x4	HX420LAD	30	22	50	Otlačení	Vyhovuje
50	Strut 2 screw	C 100x82x4	HX420LAD	30	52	54	Otlačení	Vyhovuje
51	Strut 2 screw	C 100x82x4	HX420LAD	30	58	60	Otlačení	Vyhovuje
56	Strut 2 screw	C 100x85x5	HX500LAD	30	31	59	Střih	Vyhovuje
57	Strut 2 screw	C 100x85x5	HX500LAD	30	35	66	Střih	Vyhovuje

Vazník FR1 horní pás

Popis:
Poznámky:
Datum: 2022.01.18 @ 14:17:22
Importováno ze souboru: CZ1621 FR1.rtd

Horní pas
16 62 % Profil: Vzpěr 30 Snow + reduced wind Cpe min

Parametry posouzení prvků:

Typ	Popis	L_y	Koef.	L_y,koef	Křivka Y	L_z	Koef.	L_z,koef	Křivka Z	L_ft	L_t	Tenko st.
Bottom	Bottom beam 6.0m	6	False	1	b	0.7	True	1	b	True	True	True
Strut	Strut 2 screw	1	True	1	b	0.7	True	1	b	True	False	True
Top	Top beam	1	True	1	b	0.7	True	1	b	True	True	True
Other	Simple member	1	True	0	auto	1	True	0	auto	False	False	False
Strut	Strut 1 screw	1	True	1	b	0.7	True	1	b	True	False	True
Other	Frame distributed	0.33	False	1	b	0.5	True	1	b	True	False	True
Top	Top beam SPH	0.33	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 1.5m	1.5	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 3.0m	3	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 4.5m	4.5	False	1	b	0.7	True	1	b	True	True	True

Materiál:

Prvek	Typ	Member type	Profil	Délka [m]	Materiál	Hmotnost [kg]	A [m²]	A_eff [m²]	Komb. N_Ed
4	Top	Top beam	H 112x120x5	1.422	HX500LAD	15.39	2090	2090	30
5	Top	Top beam	H 112x120x5	1.422	HX500LAD	15.39	2090	2090	30
12	Top	Top beam	H 112x120x5	0.077	HX500LAD	15.39	2090	2090	30
13	Top	Top beam	H 112x120x5	0.077	HX500LAD	15.39	2090	2090	30
16	Top	Top beam	H 112x120x5	1.521	HX500LAD	15.39	2090	2090	30
17	Top	Top beam	H 112x120x5	1.521	HX500LAD	15.39	2090	2090	30
24	Top	Top beam	H 112x120x5	0.082	HX500LAD	15.39	2090	2090	30
25	Top	Top beam	H 112x120x5	0.082	HX500LAD	15.39	2090	2090	30
28	Top	Top beam	H 112x120x5	1.47	HX500LAD	15.39	2090	2090	30
29	Top	Top beam	H 112x120x5	1.47	HX500LAD	15.39	2090	2090	30
36	Top	Top beam	H 112x120x5	0.105	HX500LAD	15.39	2090	2090	30
37	Top	Top beam	H 112x120x5	0.104	HX500LAD	15.39	2090	2090	30
40	Top	Top beam	H 112x120x5	1.478	HX500LAD	15.39	2090	2090	30
41	Top	Top beam	H 112x120x5	1.478	HX500LAD	15.39	2090	2090	10
48	Top	Top beam	H 112x120x5	0.083	HX500LAD	15.39	2090	2090	30
49	Top	Top beam	H 112x120x5	0.079	HX500LAD	15.39	2090	2090	30
52	Top	Top beam	H 112x120x5	1.445	HX500LAD	15.39	2090	2090	20
53	Top	Top beam	H 112x120x5	1.45	HX500LAD	15.39	2090	2090	40

Profily:

Prvek	Profil	N_c [%]	N_t [%]	NcMz [%]	NcMz2 [%]	V_y [%]	N_t [%]	N_b [%]	NyMy Mz [%]	NzMy Mz [%]	Profil [%]	Rozh. profil	Komb. N_Ed
4	H 112x120x5	28	0	29	28	1	13	57	36	35	57	Vzpěr	30
5	H 112x120x5	28	0	29	26	0	13	55	35	34	55	Vzpěr	30
12	H 112x120x5	29	0	35	22	3	1	30	34	34	35	Tlak a ohyb	30
13	H 112x120x5	28	0	32	23	0	1	29	32	32	32	Tlak a ohyb	30
16	H 112x120x5	30	0	33	26	0	13	62	38	37	62	Vzpěr	30
17	H 112x120x5	28	0	32	23	0	13	58	35	34	58	Vzpěr	30
24	H 112x120x5	31	0	33	29	5	1	31	35	35	35	Vzpěr za ohybu	30
25	H 112x120x5	28	0	32	24	8	1	29	32	32	32	Tlak a ohyb	30
28	H 112x120x5	28	0	35	21	1	13	59	38	37	59	Vzpěr	30
29	H 112x120x5	25	0	33	16	1	13	52	35	34	52	Vzpěr	30
36	H 112x120x5	27	0	34	19	8	1	28	33	33	34	Tlak a ohyb	30
37	H 112x120x5	23	0	31	15	11	1	24	30	30	31	Tlak a ohyb	30
40	H 112x120x5	20	0	33	4	2	13	41	34	34	41	Vzpěr	30
41	H 112x120x5	15	0	27	1	2	13	31	28	27	31	Vzpěr	10
48	H 112x120x5	15	0	21	10	30	1	16	28	28	30	Smyk	30
49	H 112x120x5	9	0	18	1	33	1	9	21	21	33	Smyk	30
52	H 112x120x5	7	0	0	0	0	13	14	0	0	14	Vzpěr	20
53	H 112x120x5	0	11	19	4	0	0	0	0	0	19	Tah a ohyb	40

Spoje:

Prvek	Profil	Spoj	N_t,Ed [kN]	N_c,Ed [kN]	F_v [%]	F_b,c [%]	F_b,t [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
4	H 112x120x5	HH06	-93.51	303.07	58	45	14	12	0	58	Střih
5	H 112x120x5	HH06	-93.53	294.41	56	43	14	12	0	56	Střih
12	H 112x120x5	X02	-92.95	308.97	0	0	0	12	0	12	Osl.průřez
13	H 112x120x5	X02	-92.97	300.46	0	0	0	12	0	12	Osl.průřez
16	H 112x120x5	X02	-98.09	318.81	0	0	0	13	0	13	Osl.průřez
17	H 112x120x5	X02	-97.9	298.12	0	0	0	13	0	13	Osl.průřez
24	H 112x120x5	X02	-99.74	328.38	0	0	0	13	0	13	Osl.průřez
25	H 112x120x5	X02	-99.48	303.37	0	0	0	13	0	13	Osl.průřez
28	H 112x120x5	X02	-91.72	303.22	0	0	0	12	0	12	Osl.průřez
29	H 112x120x5	X02	-97.79	266.59	0	0	0	13	0	13	Osl.průřez
36	H 112x120x5	X02	-87.53	296.3	0	0	0	11	0	11	Osl.průřez
37	H 112x120x5	X02	-97.25	253.84	0	0	0	13	0	13	Osl.průřez
40	H 112x120x5	X02	-57.66	210.47	0	0	0	8	0	8	Osl.průřez
41	H 112x120x5	X02	-83.56	159.04	0	0	0	11	0	11	Osl.průřez
48	H 112x120x5	X02	-38.63	162.11	0	0	0	5	0	5	Osl.průřez
49	H 112x120x5	X02	-74.9	113.54	0	0	0	10	0	10	Osl.průřez
52	H 112x120x5	HC06	-57.38	71.44	14	12	10	8	0	14	Střih
53	H 112x120x5	HC06	-120.97	28.44	23	5	20	16	0	23	Střih

Souhrn:

Prve	Member type	Profil	Materiál	Komb.	Profil	Spoj	Vyhodnoceni	Posouzení

k				N_Ed	[%]	[%]		
4	Top beam	H 112x120x5	HX500LAD	30	57	58	Střih	Vyhovuje
5	Top beam	H 112x120x5	HX500LAD	30	55	56	Střih	Vyhovuje
12	Top beam	H 112x120x5	HX500LAD	30	35	12	Tlak a ohyb	Vyhovuje
13	Top beam	H 112x120x5	HX500LAD	30	32	12	Tlak a ohyb	Vyhovuje
16	Top beam	H 112x120x5	HX500LAD	30	62	13	Vzpěr	Vyhovuje
17	Top beam	H 112x120x5	HX500LAD	30	58	13	Vzpěr	Vyhovuje
24	Top beam	H 112x120x5	HX500LAD	30	35	13	Vzpěr za ohybu	Vyhovuje
25	Top beam	H 112x120x5	HX500LAD	30	32	13	Tlak a ohyb	Vyhovuje
28	Top beam	H 112x120x5	HX500LAD	30	59	12	Vzpěr	Vyhovuje
29	Top beam	H 112x120x5	HX500LAD	30	52	13	Vzpěr	Vyhovuje
36	Top beam	H 112x120x5	HX500LAD	30	34	11	Tlak a ohyb	Vyhovuje
37	Top beam	H 112x120x5	HX500LAD	30	31	13	Tlak a ohyb	Vyhovuje
40	Top beam	H 112x120x5	HX500LAD	30	41	8	Vzpěr	Vyhovuje
41	Top beam	H 112x120x5	HX500LAD	10	31	11	Vzpěr	Vyhovuje
48	Top beam	H 112x120x5	HX500LAD	30	30	5	Smyk	Vyhovuje
49	Top beam	H 112x120x5	HX500LAD	30	33	10	Smyk	Vyhovuje
52	Top beam	H 112x120x5	HX500LAD	20	14	14	Střih	Vyhovuje
53	Top beam	H 112x120x5	HX500LAD	40	19	23	Střih	Vyhovuje

Vazník FR1 dolní pás

Popis:
Poznámky:
Datum: 2022.01.18 @ 14:17:22
Importováno ze souboru: CZ1621 FR1.rtd

Dolní pas
59 81 % Profil: Vzpěr za ohybu 30 Snow + reduced wind Cpe min

Parametry posouzení prvků:

Typ	Popis	L_y	Koef.	L_y,koef	Křivka Y	L_z	Koef.	L_z,koef	Křivka Z	L_ft	L_t	Tenkost.
Bottom	Bottom beam 6.0m	6	False	1	b	0.7	True	1	b	True	True	True
Strut	Strut 2 screw	1	True	1	b	0.7	True	1	b	True	False	True
Top	Top beam	1	True	1	b	0.7	True	1	b	True	True	True
Other	Simple member	1	True	0	auto	1	True	0	auto	False	False	False
Strut	Strut 1 screw	1	True	1	b	0.7	True	1	b	True	False	True
Other	Frame distributed	0.33	False	1	b	0.5	True	1	b	True	False	True
Top	Top beam SPH	0.33	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 1.5m	1.5	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 3.0m	3	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 4.5m	4.5	False	1	b	0.7	True	1	b	True	True	True

Materiál:

Prvek	Typ	Member type	Profil	Délka [m]	Materiál	Hmotnost [kg]	A [m²]	A_eff [m²]	Komb. N_Ed
3	Bottom	Bottom beam 4.5m	H 112x120x5	1.872	HX500LAD	15.39	2090	2090	30
6	Bottom	Bottom beam 4.5m	H 112x120x5	0.096	HX500LAD	15.39	2090	2090	30
7	Bottom	Bottom beam 4.5m	H 112x120x5	0.1	HX500LAD	15.39	2090	2090	30
10	Bottom	Bottom beam 6.0m	H 112x120x5	1.16	HX500LAD	15.39	2090	2090	30
11	Bottom	Bottom beam 3.0m	H 112x120x5	1.155	HX500LAD	15.39	2090	2090	30
18	Bottom	Bottom beam 4.5m	H 112x120x5	0.062	HX500LAD	15.39	2090	2090	30
19	Bottom	Bottom beam 4.5m	H 112x120x5	0.061	HX500LAD	15.39	2090	2090	30
22	Bottom	Bottom beam 4.5m	H 112x120x5	1.44	HX500LAD	15.39	2090	2090	30
23	Bottom	Bottom beam 3.0m	H 112x120x5	1.436	HX500LAD	15.39	2090	2090	30
30	Bottom	Bottom beam 4.5m	H 112x120x5	0.108	HX500LAD	15.39	2090	2090	30
31	Bottom	Bottom beam 4.5m	H 112x120x5	0.111	HX500LAD	15.39	2090	2090	30
34	Bottom	Bottom beam 4.5m	H 112x120x5	1.426	HX500LAD	15.39	2090	2090	30
35	Bottom	Bottom beam 3.0m	H 112x120x5	1.419	HX500LAD	15.39	2090	2090	30
42	Bottom	Bottom beam 4.5m	H 112x120x5	0.105	HX500LAD	15.39	2090	2090	30
43	Bottom	Bottom beam 4.5m	H 112x120x5	0.106	HX500LAD	15.39	2090	2090	30
46	Bottom	Bottom beam 4.5m	H 112x120x5	1.365	HX500LAD	15.39	2090	2090	32
47	Bottom	Bottom beam 4.5m	H 112x120x5	1.365	HX500LAD	15.39	2090	2090	20
54	Bottom	Bottom beam 4.5m	H 112x120x5	0.1	HX500LAD	15.39	2090	2090	10
55	Bottom	Bottom beam 4.5m	H 112x120x5	0.103	HX500LAD	15.39	2090	2090	30

58	Bottom	Bottom beam 3.0m	H 112x120x5	0.65	HX500LAD	15.39	2090	2090	30
59	Bottom	Bottom beam 3.0m	H 112x120x5	0.647	HX500LAD	15.39	2090	2090	30

Profily:

Prvek	Profil	N_c [%]	N_t [%]	NcMz [%]	NcMz2 [%]	V_y [%]	N_t [%]	N_b [%]	NyMy Mz [%]	NzMy Mz [%]	Profil [%]	Rozh. profil	Komb. N_Ed
3	H 112x120x5	0	18	18	18	0	0	0	0	0	18	Prostý tah	30
6	H 112x120x5	0	24	32	14	10	0	0	0	0	32	Tah a ohyb	30
7	H 112x120x5	0	23	28	17	7	0	0	0	0	28	Tah a ohyb	30
10	H 112x120x5	0	24	33	15	1	0	0	0	0	33	Tah a ohyb	30
11	H 112x120x5	0	23	29	17	0	0	0	0	0	29	Tah a ohyb	30
18	H 112x120x5	0	25	29	20	2	0	0	0	0	29	Tah a ohyb	30
19	H 112x120x5	0	23	26	19	1	0	0	0	0	26	Tah a ohyb	30
22	H 112x120x5	0	25	31	19	0	0	0	0	0	31	Tah a ohyb	30
23	H 112x120x5	0	22	29	16	0	0	0	0	0	29	Tah a ohyb	30
30	H 112x120x5	0	23	29	17	7	0	0	0	0	29	Tah a ohyb	30
31	H 112x120x5	0	19	26	13	11	0	0	0	0	26	Tah a ohyb	30
34	H 112x120x5	0	21	33	9	1	0	0	0	0	33	Tah a ohyb	30
35	H 112x120x5	0	17	30	4	1	0	0	0	0	30	Tah a ohyb	30
42	H 112x120x5	0	14	26	2	20	0	0	0	0	26	Tah a ohyb	30
43	H 112x120x5	0	9	18	1	23	0	0	0	0	23	Smyk	30
46	H 112x120x5	0	9	19	1	1	0	0	0	0	19	Tah a ohyb	32
47	H 112x120x5	5	0	8	2	0	40	18	15	8	18	Štíhlost	20
54	H 112x120x5	10	0	16	3	32	40	23	41	28	41	Vzpěr za ohybu	10
55	H 112x120x5	16	0	24	7	39	40	39	61	39	61	Vzpěr za ohybu	30
58	H 112x120x5	29	0	29	29	5	27	47	61	48	61	Vzpěr za ohybu	30
59	H 112x120x5	39	0	39	39	6	27	64	81	63	81	Vzpěr za ohybu	30

Spoje:

Prvek	Profil	Spoj	N_t,Ed [kN]	N_c,Ed [kN]	F_v [%]	F_b,c [%]	F_b,t [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
3	H 112x120x5	HC04	-193.49	43.76	56	10	45	25	0	56	Střih
6	H 112x120x5	X02	-254.99	51.04	0	0	0	33	0	33	Osl.průřez
7	H 112x120x5	X02	-244.16	67.14	0	0	0	32	0	32	Osl.průřez
10	H 112x120x5	X02	-258.88	51.13	0	0	0	34	0	34	Osl.průřez
11	H 112x120x5	X02	-246.62	69.33	0	0	0	32	0	32	Osl.průřez
18	H 112x120x5	X02	-267.38	46.18	0	0	0	35	0	35	Osl.průřez
19	H 112x120x5	X02	-244.42	75.77	0	0	0	32	0	32	Osl.průřez
22	H 112x120x5	X02	-271.17	45.8	0	0	0	36	0	36	Osl.průřez
23	H 112x120x5	X02	-242.35	78.98	0	0	0	32	0	32	Osl.průřez
30	H 112x120x5	X02	-248.49	39.11	0	0	0	33	0	33	Osl.průřez
31	H 112x120x5	X02	-209.31	78.51	0	0	0	27	0	27	Osl.průřez
34	H 112x120x5	X02	-231.53	34.08	0	0	0	30	0	30	Osl.průřez
35	H 112x120x5	X02	-190.53	78	0	0	0	25	0	25	Osl.průřez
42	H 112x120x5	X02	-153.03	7.35	0	0	0	20	0	20	Osl.průřez

43	H 112x120x5	X02	-116.01	66.01	0	0	0	15	0	15	Osl.průřez
46	H 112x120x5	X02	-98.05	-14.49	0	0	0	13	0	13	Osl.průřez
47	H 112x120x5	X02	-55.45	56.14	0	0	0	7	0	7	Osl.průřez
54	H 112x120x5	X02	-91.39	103.64	0	0	0	12	0	12	Osl.průřez
55	H 112x120x5	X02	-76.23	176.14	0	0	0	10	0	10	Osl.průřez
58	H 112x120x5	HH08	-171.49	304.55	44	32	18	22	0	44	Střih
59	H 112x120x5	HH08	-155.04	415.86	60	44	16	20	0	60	Střih

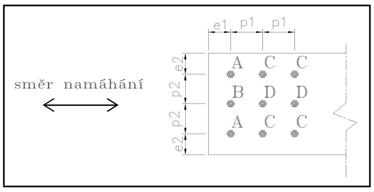
Souhrn:

Prvek	Member type	Profil	Materiál	Komb. N_Ed	Profil [%]	Spoj [%]	Vyhodnoceni	Posouzení
3	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	18	56	Střih	Vyhovuje
6	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	32	33	Osl.průřez	Vyhovuje
7	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	28	32	Osl.průřez	Vyhovuje
10	Bottom beam 6.0m	H 112x120x5	HX500LAD	30	33	34	Osl.průřez	Vyhovuje
11	Bottom beam 3.0m	H 112x120x5	HX500LAD	30	29	32	Osl.průřez	Vyhovuje
18	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	29	35	Osl.průřez	Vyhovuje
19	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	26	32	Osl.průřez	Vyhovuje
22	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	31	36	Osl.průřez	Vyhovuje
23	Bottom beam 3.0m	H 112x120x5	HX500LAD	30	29	32	Osl.průřez	Vyhovuje
30	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	29	33	Osl.průřez	Vyhovuje
31	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	26	27	Osl.průřez	Vyhovuje
34	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	33	30	Tah a ohyb	Vyhovuje
35	Bottom beam 3.0m	H 112x120x5	HX500LAD	30	30	25	Tah a ohyb	Vyhovuje
42	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	26	20	Tah a ohyb	Vyhovuje
43	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	23	15	Smyk	Vyhovuje
46	Bottom beam 4.5m	H 112x120x5	HX500LAD	32	19	13	Tah a ohyb	Vyhovuje
47	Bottom beam 4.5m	H 112x120x5	HX500LAD	20	18	7	Štíhlost	Vyhovuje
54	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	41	12	Vzpěr za ohybu	Vyhovuje
55	Bottom beam 4.5m	H 112x120x5	HX500LAD	30	61	10	Vzpěr za ohybu	Vyhovuje
58	Bottom beam 3.0m	H 112x120x5	HX500LAD	30	61	44	Vzpěr za ohybu	Vyhovuje
59	Bottom beam 3.0m	H 112x120x5	HX500LAD	30	81	60	Vzpěr za ohybu	Vyhovuje

PŘÍPOJ SPODNÍHO PASU UC56 M16

Únosnot šroubového spoje OTLAČENÍ / STŘICH / OSL. PRŮŘEZ / VYTRŽENÍ SKUPINY ŠROUBŮ

Dle: ČSN EN 1993-1-1, ČSN EN 1993-1-3,ČSN EN 1993-1-8



Geometrie

Ideální geometrie pro k_1=2,5 a α_d=1,0

$e1_A$=	30	mm	$e_1=3d_0$=	49.5	mm
$e1_B$=	30	mm	$e_1=3d_0$=	49.5	mm
e_2=	25	mm	$e_2=1{,}5d_0$=	24.8	mm
p_1=	50	mm	$p_1=3{,}75d_0$=	61.9	mm
p_2=	62	mm	$p_2=3d_0$=	49.5	mm

šroub Střih v dříku 1-střižný

Pevnostní třída šroubu 8.8

f_{ub}= 800 MPa

Průřez -Průměr šroubu d= 16 mm => A= 201.0 mm2 (Plocha šroubu)

A_s= 139.0 mm2 (Plocha jádra šroubu)

-Průměr otvoru d_0= 16.5 mm

Počet šroubů n_b= 12

Profil C100x129x5 t_1= 5.0 mm

Ocel HX500LAD

f_{u1}= 550.0 MPa

Zákl. materiál Stejný nebo únosnější

Koeficient bez. γ_{M0}= 1.00 γ_{M1}= 1.00 γ_{M2}= 1.25

Únosnost v otlačení jednotlivých šroubů

(Viz. ČSN EN 1993-1-3: 8.3, ČSN EN 1993-1-8: 3.6.1)

$F_{b,Rd,1}=k_1\alpha_b f_u d t/\gamma_{M2}$

VE SMĚRU ZATÍŽENÍ

Koncové šrouby A: min { $\alpha_d=e_{1A}/3d_0$= 0.61; $\alpha_d=f_{ub}/f_{u1}$=; α_d=1.0 }

Koncové šrouby B: min { $\alpha_d=e_{1B}/3d_0$= 0.61; $\alpha_d=f_{ub}/f_{u1}$=; α_d=1.0 }

Vnitřní šrouby: min { $\alpha_d=(p_1/3d_0)-1/4$= 0.76; $\alpha_d=f_{ub}/f_u$=; α_d=1.0 }

KOLMO NA SMĚR ZATÍŽENÍ

Šrouby u okraje: min { $K_1=(2{,}8e_2/d_0)-1{,}7$= 2.50; $K_1=(1{,}4p_2/d_0)-1{,}7$=; K_1=2,5 }

Vnitřní šrouby: min { $K_1=(1.4p_2/d_0)-1{,}7$= 2.50; K_1=2,5 }

t1= 5 mm; Ocel HX500LAD; fu1= 550 MPa	Poloha šroubu		$F_{b,Rd}$		Počet typu šroubu ve spoji
	Konec/okraj	A	53.3	kN	2
	Konec/vnitřní	B	53.3	kN	2
	Vnitřní/okraj	C	66.9	kN	4
	Vnitřní/vnitřní	D	66.9	kN	4
				SUMA=	12 (kontrola)

Únosnost ve střihu jednoho šroubu / Únosnost ve střihu celého spoje

(Viz. ČSN EN 1993-1-3: 8.3)

α_v= 0.6

n_v= 1 (počet střižných ploch)

$F_{v,R,1}=n_v\alpha_v f_{ub}\{A,A_s\}/\gamma_{M2}$

$F_{v,Rd,1}$= 77.2 kN

$F_{v,Rd}$= 926.2 kN

Únosnost v otlačení/střihu celého spoje

(Viz. ČSN EN 1993-1-8: 3.7)

(Střih) $F_{v,R,1}$= 77.2 kN > (Otlak) $\max(F_{b,Rd,(A,B,C,D)})$= 66.9 kN => Celková únosnost spoje bude rovna součtu únosností jednotlivých spojovacích prostředků v otlačení.

$F_{v/b,Rd}$= 748.4 kN

Únosnost oslabeného průřezu

Plocha dle zadaného profilu

(Viz. ČSN EN 1993-1-3: 8.3, ČSN EN 1993-1-1: 6.2.3)

A_g= 2025.6 mm2

t= 5.0 mm

f_u= 550.0 Mpa

f_y= 500.0 Mpa

Celkový počet šroubů ve spoji n_b= 12

Počet šroubů v řezu n_c= 4

Poměr $r=n_c/n_b$ r= 0.33

$u=\min(2e_2;p_2)$ u= 50 mm

Oslabená plocha profilu A_{net}= 1696 mm2 $A_{net}=A_g-(n_c d_0 t)$

$F_{n,Rd,1}=(1 + 3r(d_0/u - 0{,}3))A_{net}f_u/\gamma_{M2}$

$F_{n,Rd,1}$= 768 kN

$F_{n,Rd,2}=0{,}9A_{net}f_u/\gamma_{M2}$ $F_{pl,Rd}=A_g f_y/\gamma_{M0}$

$F_{n,Rd,2}$= 671 kN $F_{pl,Rd}$= 810 kN

$F_{n,Rd}=\min(F_{n,Rd,1};F_{n,Rd,2};F_{pl,Rd})$

$F_{n,Rd}$= 671 kN

Vytržení skupiny šroubů

(Viz. ČSN EN 1993-1-8: 3.10.2)

L_{nt}= 91 mm => A_{nt}= 455 mm^2

L_{nv}= 360 mm => A_{nv}= 1800 mm^2

f_u= 550.0 Mpa

f_y= 500.0 Mpa

t= 5.0 mm

Osové zatížení

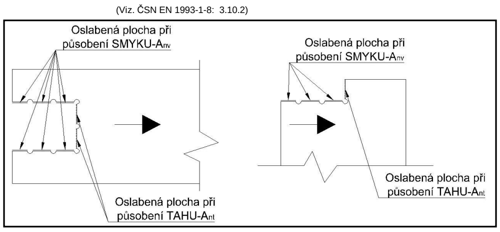
$V_{eff,1,Rd}=f_u A_{nt}/\gamma_{M2} + (1/3^{0.5})\, f_y A_{nv}/\gamma_{M0}$

$V_{eff,1,Rd}$= 720 kN

Excentrické zatížení

$V_{eff,1,Rd}=0{,}5f_u A_{nt}/\gamma_{M2} + (1/3^{0.5})\, f_y A_{nv}/\gamma_{M0}$

$V_{eff,1,Rd}$= 620 kN



Check M, V och N acc. to EC

$\boxed{Country := "CZ"}$

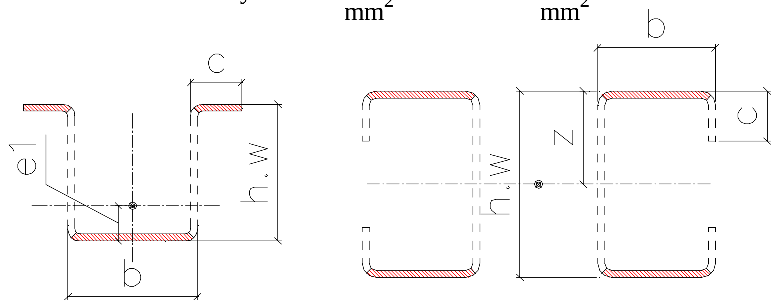
$\boxed{PR \equiv "Hx5"}$

Profile data for a single profile

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

$h_w = 120\,mm$ $b_c = 112\,mm$ $c = 43\,mm$ $t = 5\,mm$ $e_1 = 56.31\,mm$ $A_g = 2025.62\,mm^2$

$N_{cRk} = 981.63\,kN$ $M_{Rk} = 33.76\,kN \cdot m$ $f_{yb} = 500 \dfrac{N}{mm^2}$ $f_u = 550 \dfrac{N}{mm^2}$



The beam is H-profile ("H") or double C-profile ("C")

Hole diameter: $\boxed{d_o := 12.5 \cdot mm}$

Number of bolts at the cross-section in ONE web: $\boxed{n_{cross} := 3}$

Number screws and diameter i FLANGES: $\boxed{X_{h.fl} := 0}$ $\boxed{d_{o.fl} := 0 \cdot mm}$

Momentcap.one profile:

Axialforcecapacity one profile:

Axialforce acting as TENTION : "YES" or "NO" $\boxed{N_{drag} := "YES"}$

Netto area for shearforce:

$$A_w := \left| \begin{array}{ll} \left[\left[(h_w - t) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} = 0 \cdot mm \\ \left[\left[(L_{II} - t) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} \neq 0 \cdot mm \end{array} \right.$$

$A_w = 775.00\,mm^2$

Nettoarea for profiles in tention: $A_{net} := A_g - t \cdot \left(n_{cross} \cdot d_o \cdot \dfrac{2}{X} + X_{h.fl} \cdot d_{o.fl} \right)$

Moment capacity profile: $M_{Rd} := \dfrac{X \cdot M_{Rk}}{\gamma_{M0}}$ $M_{Rd} = 33.76\,kN \cdot m$

Axialforce capacity compresion member: $N_{Rd.1} := \dfrac{N_{cRk} \cdot X}{\gamma_{M0}}$ (6.2.4) $N_{Rd.1} = 981.6\,kN$

Capacity for axial tension:

For material <= 4.0mm : $N_{Rd.t.a} := \min\left(\dfrac{f_{ya} \cdot A_g}{\gamma_{M0}}, \dfrac{A_{net} \cdot f_u}{\gamma_{M2}} \right)$

(EN 1993-1-3: 6.1.2 (6.1) and EN 1993-1-3: 8 (table 8.4))

$N_{Rd.t.a} = 726.27\,kN$

For material >= 5.0mm : $N_{Rd.t.b} := \dfrac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2.t}}$ $N_{Rd.t.b} = 653.6\,kN$ (EN1993-1-1: 6.2.3) $\gamma_{M2.t} = 1.25$

$$N_{Rd.t} := \left(\left| \begin{array}{ll} N_{Rd.t.a} & \text{if } t \le 4 \cdot mm \\ N_{Rd.t.b} & \text{if } t \ge 5 \cdot mm \end{array} \right. \right) \cdot X$$

$N_{Rd.t} = 653.65\,kN$

Normalkraftscapacity:

$$N_{Rd} := \left| \begin{array}{ll} N_{Rd.1} & \text{if } N_{drag} = "NO" \\ \min(N_{Rd.1}, N_{Rd.t}) & \text{if } N_{drag} = "YES" \end{array} \right.$$

$N_{Rd} = 653.65\,kN$

Shearforce capacity:

$\lambda_{wh} := 0.346 \cdot \dfrac{h_w - t}{t} \cdot \sqrt{\dfrac{f_{yb}}{E}}$

$\lambda_{wh} = 0.39$

$$f_{vb} := \left| \begin{array}{ll} \dfrac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{wh} \le 0.83 \\ 0.48 \cdot \dfrac{f_{yb}}{\lambda_{wh}} & \text{if } 0.83 < \lambda_{wh} < 1.40 \\ 0.67 \cdot \dfrac{f_{yb}}{\lambda_{wh}^2} & \text{if } \lambda_{wh} \ge 1.40 \end{array} \right.$$

$f_{vb} = 288.68 \dfrac{N}{mm^2}$

$V_{Rd} := \dfrac{A_w \cdot f_{vb}}{\gamma_{M0}}$

$V_{Rd} = 223.72\,kN$

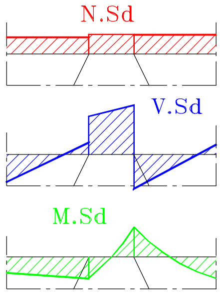
elast. momentcap.: $M_{elRd} := M_{Rd}$

$\theta_{pl} := 1.0$ faktor M_{el} -> M_{pl}

Section value for interaction formula M + N + V:
Momentresistance of a cross-section consisting only flanges: $M_{flRd} = 25.37\,kN \cdot m$

Element Nr: dolní pás

$N_{Sd} := 177 \cdot kN$ $V_{Sd} := 130 \cdot kN$ $M_{Sd} := 11 \cdot kN \cdot m$



$\frac{|V_{Sd}|}{V_{Rd}} = 0.58$ <1.0 (EN 1993-1-1: 6.2.6)

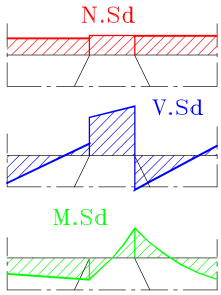
$\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 0.6$ <1.0 (EN 1993-1-3 6.1.8+6.1.9)

Ratio for check
Moment + Axialforce + Shear force

$M\_N\_V_{EC} = 0.6$ <1.0 (EN 1993-1-3 6.1.10)

Element Nr: horní pás

$N_{Sd} := 163 \cdot kN$ $V_{Sd} := 100 \cdot kN$ $M_{Sd} := 6 \cdot kN \cdot m$



$\frac{|V_{Sd}|}{V_{Rd}} = 0.45$ <1.0 (EN 1993-1-1: 6.2.6)

$\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 0.43$ <1.0 (EN 1993-1-3 6.1.8+6.1.9)

Ratio for check
Moment + Axialforce + Shear force

$M\_N\_V_{EC} = 0$ <1.0 (EN 1993-1-3 6.1.10)

Frame Column FR1

$h \equiv 360 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 39 \cdot mm$ $t \equiv 5 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $cc_{batt} := 1500 \cdot mm$ distance between battens $h_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $kb = 599.63\ mm$ $\gamma_{M1} \equiv 1.0$

$b_{batt} := 200 \cdot mm$

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 6.051 \times 10^3\ mm^2$

$I_{zz} := 2 \cdot \left[I_z + A_g \cdot \left(e_1 + \frac{d}{2} \right)^2 \right]$ $I_{zz} = 4.45 \times 10^7\ mm^4$

$I_{yy} := 2 \cdot I_y$ $I_{yy} = 1.12 \times 10^8\ mm^4$ ingle C:

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 6.29 \times 10^5\ mm^3$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 6.1 \times 10^5\ mm^3$

$W_{zz} := \frac{I_{zz}}{b + \frac{d}{2}}$ $W_{zz} = 2.97 \times 10^5\ mm^3$

$I_{TT} = 51500\ mm^4$

$M_{yy.cRk} := 2 \cdot M_{ycRk}$

$M_{yy.cRk} = 305.06\ kN \cdot m$

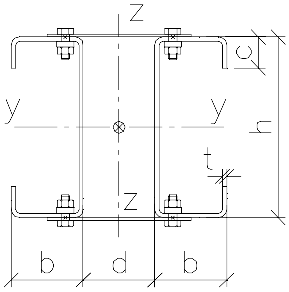
$M_{ycRk} = 152.53\ kN \cdot m$

$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$N_{cc.Rk} = 2014.52\ kN$

$f_{yb} = 500 \frac{N}{mm^2}$

$f_u = 550 \frac{N}{mm^2}$



$i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 135.84\ mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 85.77\ mm$

torsion_plate_pillar = "NO"

forces from:

Loadcase := max

Stresses and buckling length according to first order frame analys

$M_{Ed.1} := 132 \cdot kN \cdot m$ max moment in pillar

The moment is distributed according to frame modelling

Moment_dis := "A"

$M_{Ed.1.z} := 0 \cdot kN \cdot m$ moment about z-z

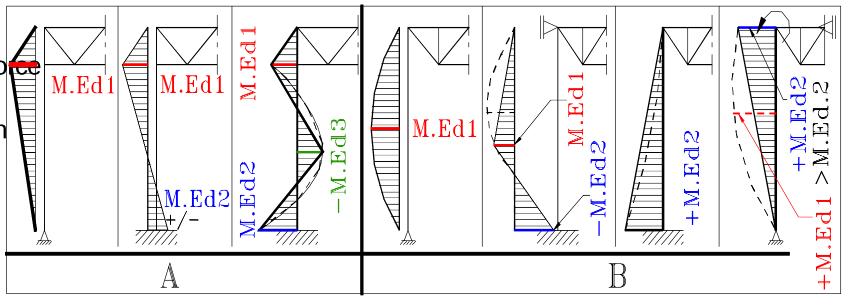
$N_{Ed} := 153 \cdot kN$ axial force in pillar = reaction force

$N_{Ed.1} := 106 \cdot kN$ axial force in element between top and bottom beam

$N_{Ed.2} := 146 \cdot kN$ reaction force in second pillar

$V_{Ed} := 275 \cdot kN$ --> $n_{cross} := 4$ =Numbers of holes in **ONE** web

$d_0 := 12.5 \cdot mm$



$M_{Ed.2} := -113 \cdot kN \cdot m$ $M_{Ed.3} := 0 \cdot kN \cdot m$ $M_{Ed.2.z} := 0 \cdot kN \cdot m$

Global geometry.

$L := 5.8 \cdot m$ length pillar

$L_z := 6.2 \cdot m$ length pillar for buckling z-z

$B_{fr} := 14.64 \cdot m$ width frame

$H_{tr} := 975 \cdot mm$ height truss

$A_{u.chord} := 1616 \cdot mm^2$ Area upper chord

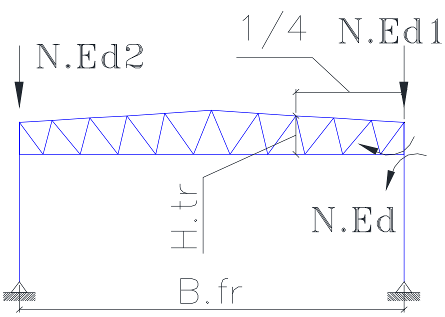
$I_{u.chord} := 3.29 \cdot 10^6 \cdot mm^4$ I upper chord

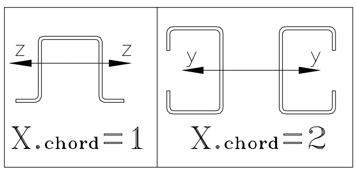
$X_{u.chord} := 1$ number of profiles upper chord

$A_{lo.chord} := 1616 \cdot mm^2$ Area lower chord

$I_{lo.chord} := 3.29 \cdot 10^6 \cdot mm^4$ I lower chord

$X_{lo.chord} := 1$ number of profiles lower chord





Buckling lengths:

The global buckling length in y-y is calculated accounting for the stiffness behavior of the frame. The method from "Stahlbau in Beispielen, Berechnungspraxis nach DIN 18800-1 bis 3", Hünersen, Fritzsche is uesd.

$L_y := 2 \cdot L$ $L_y = 11.6\ m$ exural buckling axis y-y: according to modelled system

$L_z = 6.2\ m$ Flexural buckling axis z-z $L_y = 11.6\ m$

$L_{LT} := 1.0 \cdot L$ $L_{LT} = 5.8\ m$ Distance flange bracings - LTB inner flange in compression

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Flexural buckling resistance - axis y-y and z-z EN 1993-1-3: 6.2.2 and EN 1993-1-1: 6.3.1:

Buckling curve for double C-section: EN 1993-1-3 table 6.3 about:

y-y: a EN1993-1-1,table 6.1: α =0,21

z-z: b EN1993-1-1,table 6.1: α =0,34

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about y-y

$$\lambda_1 := \pi\cdot\sqrt{\frac{E}{f_{yb}}} \quad \lambda_1 = 64.38 \quad \lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \quad \lambda_{r.y.FBcc} = 1.08$$

Imperfection factor α relating to bucking curve a $\alpha_y := 0.21$

$$\phi_{y.FB} := 0.5\cdot\left[1 + \alpha_y\cdot(\lambda_{r.y.FBcc} - 0.2) + \lambda_{r.y.FBcc}^2\right] \quad \phi_{y.FB} = 1.18$$

$$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^2 - \lambda_{r.y.FBcc}^2}}, 1\right) \quad \chi_{y.FBcc} = 0.61$$

Design buckling resistance for buckling mode -flexural buckling-: y-y

$$N_{b.Rd.y.FBcc} := \frac{\chi_{y.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \quad N_{b.Rd.y.FBcc} = 1225.2\,kN$$

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about z-z

$$\lambda_1 = 64.38 \quad \lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \quad \lambda_{r.z.FBcc} = 0.92$$

Imperfection factor α relating to bucking curve b $\alpha_z := 0.34$

$$\phi_{z.FB} := 0.5\cdot\left[1 + \alpha_z\cdot(\lambda_{r.z.FBcc} - 0.2) + \lambda_{r.z.FBcc}^2\right] \quad \phi_{z.FB} = 1.04$$

$$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^2 - \lambda_{r.z.FBcc}^2}}, 1\right) \quad \chi_{z.FBcc} = 0.65$$

Design buckling resistance for buckling mode -flexural buckling-: z-z

$$N_{b.Rd.z.FBcc} := \frac{\chi_{z.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \quad N_{b.Rd.z.FBcc} = 1311.1\,kN$$

Lateral-torsional buckling resistance for uniform member in bending EN 1993-1-3: 6.2.4 and EN 1993-1-1: 6.3.2:

Procedure to calculate the elastic critical buckling moment M_{cr}, based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints, is given neither in EN 1993-1-1 nor in EN 1993-1-3. Any appropriate calculation method can be used, here the procedure given in German ENV 1993-1-1 Annex F.

Moment_dis = "A" $L_{LT} = 5.8\,m$ $k_{M.cr} := 1.0$ (hinged at ends) $k_{w.M.cr} := 1.0$ (no special wrap restraints at ends)

For frame pillar with fixed both upper and lower beam (end moments): according to picture "A":

$$\psi_{md.A} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases} \quad \psi_{md.A} = -0.86$$

$$C_{1.A} := \min\left(1.88 - 1.40\cdot\psi_{md.A} + 0.52\cdot\psi_{md.A}^2, 2.7\right) \quad C_{1.A} = 2.7$$

$$M_{cr.A} := C_{1.A}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \quad M_{cr.A} = 600.7\,kN\cdot m$$

For framepillar with moment distribution according to picture "B": $C_{1.B} := 1.132$

$$M_{cr.B} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \quad M_{cr.B} = 251.85\,kN\cdot m$$

Elastic critical moment for lateral-torsional buckling

based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints for double C-section:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if Moment\_dis} = \text{"A"} \\ M_{cr.B} & \text{if Moment\_dis} = \text{"B"} \end{cases} \quad M_{cr} = 601\,kN\cdot m$$

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Relative slenderness: $\lambda_{rLT} := \sqrt{\frac{2 \cdot W_{eff.y.1} \cdot f_{yb}}{M_{cr}}}$ $\lambda_{rLT} = 0.71$ $\alpha_{LT} := 0.34$ $\phi_{LT} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{rLT} - 0.2) + \lambda_{rLT}^2\right]$ $\phi_{LT} = 0.84$

$\chi_{LT.y} := \min\left(\frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{rLT}^2}}, 1\right)$ $\chi_{LT.y} = 0.78$ $\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{yy} \geq I_{zz} \\ 1 & \text{otherwise} \end{cases}$ Reduction buckling factor: $\chi_{LT} = 0.78$

<u>Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:</u>

Desing buckling moment resistance double profile based on effective section modulus W_{eff}. $M_{b.Rd} := \frac{\chi_{LT} \cdot M_{yy.cRk}}{\gamma_{M1}}$ $M_{b.Rd} = 236.89\,kN \cdot m$

<u>Check Uniform built-up member EN 1993-1-1: 6.4</u>

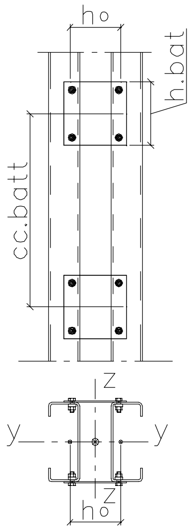
bow imperfection: $e_0 := \frac{L}{500}$ $\text{if}\left(\frac{cc_{batt}}{i_z} \leq 70, \text{"OK"}, \text{"Shorter-cc.bat"}\right) = \text{"OK"}$

non-dimensional slenderness: $\lambda_1 = 64.38$

<u>Effective second moment of area of battened built-up member:</u> $A_{ch} := A_g$: area of one chord

$h_0 := d + 2 \cdot e_1$ $h_0 = 155.7\,mm$ distance centroids of chords $I_{ch} := I_z$: I of one chord

$I_{l.CC} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot I_{ch}$ I built-up member $i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2 \cdot A_{ch}}}$ $\lambda_{CC} := \frac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 72.28$



EN 1993-1-1: table 6.8 Efficiency factor: $\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \geq 150 \\ \left(2 - \frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \leq 75 \end{cases}$ $\mu_{CC} = 1$

$I_{l.CC} = 4.45 \times 10^7\,mm^4$ $r_{CC} := 2$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu_{CC} \cdot I_{ch}$ $I_{eff} = 4.45 \times 10^7\,mm^4$ effective I of built-up member

<u>Shear stiffness EN 1993-1-1:6.4.3.1:(2)</u> $I_b := \frac{t_{batt} \cdot h_{batt}^3}{12}$ I of batten $n_{batt} := 2$ number of planes of lacings

$$S_v := \min\left[\left[\frac{24 \cdot E \cdot I_{ch}}{cc_{batt}^2 \cdot \left(1 + \frac{2 \cdot I_{ch}}{n_{batt} \cdot I_b} \cdot \frac{h_0}{cc_{batt}}\right)}\right], \left(\frac{2 \cdot \pi^2 \cdot E \cdot I_{ch}}{cc_{batt}^2}\right)\right] \quad S_v = 7214\,kN$$

effective critical force of built-up member: $N_{cr.CC} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_z^2}$ $N_{cr.CC} = 2400.4\,kN$

Maximum moment in middle of built-up member: EN 1993-1-1:6.4.1 (6):

$M_{z.Ed.I} := \max\left(\left|M_{Ed.1.z}\right|, \left|M_{Ed.2.z}\right|\right)$ moment with second order effects $M_{z.Ed.C} := \frac{N_{Ed} \cdot e_0 + M_{z.Ed.I}}{1 - \frac{N_{Ed} \cdot \gamma_{M1}}{N_{cr.CC}} - \frac{N_{Ed} \cdot \gamma_{M1}}{S_v}}$ $M_{z.Ed.C} = 1.94\,kN \cdot m$

moment z-z without second order effects

Compression force in one member: EN 1993-1-1:6.4.1 (6) Compression force in built-up member taking accout to second order effects:

$X_d := \begin{cases} 1 & \text{if } d \geq 8 \cdot mm \\ 0 & \text{otherwise} \end{cases}$ $N_{ch.Ed} := \frac{N_{Ed}}{r_{CC}} + \frac{\left|M_{z.Ed.C}\right| \cdot h_0 \cdot A_{ch} \cdot X_d}{2 \cdot I_{eff}}$ $N_{ch.Ed} = 86.76\,kN$

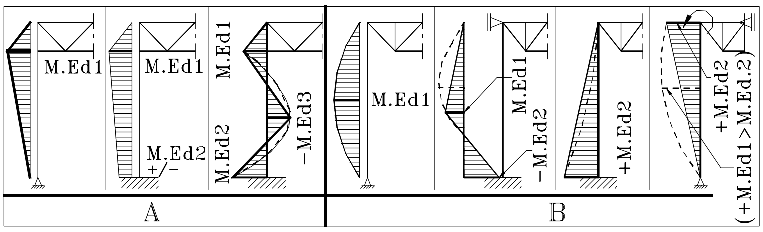
$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500 \frac{N}{mm^2}$ $f_u = 550 \frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):

The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3

Moment distribution like shown on figure: Moment_dis = "A"



$\psi_{md.A} = -0.86$ $\psi_{m.B} = 0$ $\psi_{md.z} = 0$

$\alpha_{h.2} = -0.$ $\alpha_{h.2.z} = 0$ $\alpha_{s.2} = 1$

$C_{my.2.A} := 0.9$ $C_{my.2.B} = 1$

$C_{my.2} = 0.9$ $C_{mz.2} = 0.95$

$C_{mLT.2} = 0.4$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}}$ $n_{y.2} = 0.12$ $n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}}$ $n_{z.2} = 0.12$

$k_{yy.2} := \min\left[C_{my.2} \cdot \left(1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}\right), C_{my.2} \cdot \left(1 + 0.6 \cdot n_{y.2}\right)\right]$ $k_{yy.2} = 0.97$

$k_{zy.2} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FBcc}}{C_{mLT.2} - 0.25} \cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25} \cdot n_{z.2}\right)$ $k_{zy.2} = 0.96$

$k_{zz.2} := \min\left[C_{mz.2} \cdot \left(1 + 0.6 \cdot \lambda_{r.z.FBcc} \cdot n_{z.2}\right), C_{mz.2} \cdot \left(1 + 0.6 \cdot n_{z.2}\right)\right]$ $k_{zz.2} = 1.01$ $k_{yz.2} := k_{zz.2}$

Reduction factor for L-T buckling: $\chi_{LT} = 0.78$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.61$ $\chi_{z.FBcc} = 0.65$

$I_{zz.eff} := 2 \cdot I_z + A_g \cdot \left(e_1 + \frac{d}{2}\right)^2 + A_{eff} \cdot \left(e_N + \frac{d}{2}\right)^2$ $W_{zz.eff} := \frac{I_{zz.eff}}{b + \frac{d}{2}}$ $I_{zz.eff} = 4.02 \times 10^7\,mm^4$ $M_{zz.cRk} := W_{zz.eff} \cdot f_{yb}$

$W_{zz.eff} = 2.68 \times 10^5\,mm^3$

Double CC-profile: Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{\max(N_{Ed}, N_{Ed.1}) \cdot \gamma_{M0}}{2 \cdot N_{c.Rk}} + \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right) \cdot \gamma_{M0}}{2 \cdot M_{ycRk}} + \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M0}}{M_{zz.cRk}} = 0.52$$ <1.0

Double CC-profile: Combined M + N + V acc to EN 1993-1-3: 6.1.10

$\frac{V_{Ed} \cdot \gamma_{M0}}{2 \cdot V_{bh.Rk.pillar}} = 0.45$ < 1.0 according to EN 1993-1-3: 6.1.10 $M\_N\_V_{EC} = 0$ <1.0

Double CC-profile: Combined bending an axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}} + k_{yy.2} \cdot \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{yz.2} \cdot \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.68$$ <1.0

Evading in z-z: $\psi_{c.N} := \frac{N_{Ed}}{N_{Ed.1}}$ $k_{c.N} := \frac{1}{1.33 - 0.33 \cdot \psi_{c.N}}$ $k_{c.N} = 1$ EN 1993-1-1 Tab. 6.6

$$\frac{\max(N_{Ed}, N_{Ed.1}) \cdot k_{c.N} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}} + k_{zy.2} \cdot \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{zz.2} \cdot \frac{\left|M_{z.Ed.C}\right| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.67$$ <1.0

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords and battens are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1).

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := \max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.3}|)\cdot 0.5$ $M_{y.max.C} = 66\,kN\cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25\cdot cc_{batt}$ $M_{y.AA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = 42.24\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5\cdot cc_{batt}$ $M_{y.BA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = 50.16\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75\cdot cc_{batt}$ $M_{y.CA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = 58.08\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot M_{y.max.C}}{2.5\cdot M_{y.max.C} + 3\cdot|M_{y.AA}| + 4\cdot|M_{y.BA}| + 3\cdot|M_{y.CA}|}$$

$y_{0.cr} := (y_M + e_{1c.rc})\cdot -1$ $z_{0.cr} := 0\cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot cc_{batt}}{i_z}\right)^2} \qquad \sigma_{ez} = 1192.21\,\frac{N}{mm^2}$$

$$\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{cc_{batt}^2}\right) \qquad \sigma_T = 1307.05\,\frac{N}{mm^2}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_A} := C_{bA}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 730.8\,kN\cdot m$

Relative slenderness:

$$\lambda_{rLT.C\_A} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0\\ 0.2 & \text{if } M_{e.cr.C\_A} = 0\end{cases}$$

$\lambda_{rLT.C\_A} = 0.46$ Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right] \quad \chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right) \quad \chi_{LT.C\_A} = 0.9$$

$\phi_{LT.C\_A} = 0.65$

For moment distribution accord. to modell B: $q_{wind\_pillar} := \frac{|M_{Ed.1}\cdot 0.5|\cdot 8}{L^2}$ $M_{y.i}(x_i) := \frac{q_{wind\_pillar}}{2}\cdot(L\cdot x_i - x_i^2)$

Moment at quater point of unbraced segment: $x_{1\_4} := \frac{L - cc_{batt}}{2} + \frac{1}{4}\cdot cc_{batt}$ $x_{1\_4} = 2.52\,m$ $M_{y.i}(x_{1\_4}) = 64.9\,kN\cdot m$ $M_{y.AB} := M_{y.i}(x_{1\_4})$ $M_{y.AB} = 64.9\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 2.9\,m$ $M_{y.i}(x_{1\_2}) = 66\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 66\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := \frac{L - cc_{batt}}{2} + \frac{3}{4}\cdot cc_{batt}$ $x_{3\_4} = 3.27\,m$ $M_{y.i}(x_{3\_4}) = 64.9\,kN\cdot m$ $M_{y.CB} := M_{y.i}(x_{3\_4})$ $M_{y.CB} = 64.9\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot(|M_{Ed.1}\cdot 0.5|)}{2.5\cdot(|M_{Ed.1}\cdot 0.5|) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := C_{bB}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 595.3\,kN\cdot m$

Relative slenderness:

$$\lambda_{rLT.C\_B} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0\\ 0.2 & \text{if } M_{e.cr.C\_B} = 0\end{cases}$$

$\phi_{LT.C\_B} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_B} - 0.2) + \lambda_{rLT.C\_B}^2\right]$

$\lambda_{rLT.C\_B} = 0.51$ $\phi_{LT.C\_B} = 0.68$

$$\chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right) \qquad \chi_{LT.C\_B} = 0.88$$

Moment distribution: Moment_dis = "A"

$$\chi_{LT.C} := \begin{cases}\chi_{LT.C\_A} & \text{if Moment\_dis = "A"}\\ \chi_{LT.C\_B} & \text{if Moment\_dis = "B"}\end{cases} \qquad \chi_{LT.C} = 0.9$$

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Stresses on one member profile at mid-span from above: chord force from above: $N_{ch.Ed} = 86.76\,kN$ *max moment y-y:* $M_{y.max.C} = 66\,kN\cdot m$

Chord at end panel: highest shear force: $V_{Ed.max} := \pi\cdot\frac{M_{z.Ed.C}}{L}$ $V_{Ed.max} = 1.05\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 0.53\,kN$

$V_{bb.Rd} = 274.24\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.002$ << 1,0 The shear force is negligible

Chord at end panel: "corner" moment: $M_{z.ch} := \frac{V_{Ed.max}}{r_{CC}}\cdot\frac{cc_{batt}}{2}$ *maximum moment z-z:* $M_{z.ch} = 0.39\,kN\cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C}\cdot\sin\left(\frac{\pi\cdot cc_{batt}}{L}\right)$ $M_{z.Ed.C.end} = 1.41\,kN\cdot m$ moment due to bow imperfection at end of panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end}\cdot\frac{(h_0\cdot A_{ch})}{(2\cdot I_{eff})}$ $N_{ch.end} = 83.95\,kN$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$n_{y.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{y.C.2} = 0.08$ $n_{z.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{z.C.2} = 0.1$ $\chi_{y.FB}(cc_{batt}) = 1$ $\chi_{z.FB}(cc_{batt}) = 0.87$

Equivalent uniform moment factors: Moment_dis = "A"

Moment at start of unbraced segment: $x_{0\_0} := 0\cdot cc_{batt}$ $M_{y.A0} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{0\_0})$ $M_{y.A0} = 34.32\,kN\cdot m$

Moment at end of unbraced segment: $M_{y.max.C} = 66\,kN\cdot m$

$$\psi_{m.C.A} := \frac{M_{y.A0}}{\begin{cases} M_{Ed.1}\cdot 0.5 & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ M_{Ed.2}\cdot 0.5 & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}}$$ $\psi_{m.C.A} = 0.52$ range for model "A"

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 2.9\,m$ $M_{y.i}(x_{1\_2}) = 66\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 66\,kN\cdot m$

Moment at start of unbraced segment: $x_{0\_0} := \frac{L - cc_{batt}}{2} + 0\cdot cc_{batt}$ $x_{0\_0} = 2.15\,m$ $M_{y.i}(x_{0\_0}) = 61.59\,kN\cdot m$ $M_{y.B0} := M_{y.i}(x_{0\_0})$ $M_{y.B0} = 61.59\,kN\cdot m$

$\alpha_{h.C.2} := \frac{|M_{y.B0}|}{|M_{y.BB}|}$ $\alpha_{h.C.2} = 0.93$ $\psi_{m.C.B} := 1$ range for model "B"

$$C_{mLT.C.2} := \begin{cases} \max(0.6 + 0.4\cdot\psi_{m.C.A}, 0.4) & \text{if Moment\_dis = "A"} \\ (0.95 + 0.05\cdot\alpha_{h.C.2}) & \text{if Moment\_dis = "B"} \end{cases}$$ $C_{mLT.C.2} = 0.808$ $C_{my.C.2} := C_{mLT.C.2}$ $C_{mz.C.2} := 1$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$\lambda_{r.y.FB}(cc_{batt}) = 0.14$ $k_{yy.C.2} := \min\left[C_{my.C.2}\cdot\left(1 + 0.6\cdot\lambda_{r.y.FB}(cc_{batt})\cdot n_{y.C.2}\right), C_{my.C.2}\cdot\left(1 + 0.6\cdot n_{y.C.2}\right)\right]$ $k_{yy.C.2} = 0.81$

$\lambda_{r.z.FB}(cc_{batt}) = 0.53$ $k_{zz.C.2} := \min\left[C_{mz.C.2}\cdot\left(1 + 0.6\cdot\lambda_{r.z.FB}(cc_{batt})\cdot n_{z.C.2}\right), C_{mz.C.2}\cdot\left(1 + 0.6\cdot n_{z.C.2}\right)\right]$ $k_{zz.C.2} = 1.03$

$k_{zy.C.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FB}(cc_{batt})}{C_{my.C.2} - 0.25}\cdot n_{z.C.2}, 1 - \frac{0.05}{C_{mLT.C.2} - 0.25}\cdot n_{z.C.2}\right)$ $k_{zy.C.2} = 1$ $k_{yz.C.2} := k_{zz.C.2}$

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Controll single C-profile between battens:

$M_{y.max.C} = 66\,kN \cdot m$ $M_{z.ch} = 0.39\,kN \cdot m$ $\Delta M_{z.shift} := \left|\Delta e_N \cdot N_{ch.end}\right|$ $\Delta M_{z.shift} = 0.69\,kN \cdot m$ $N_{ch.end} = 83.95\,kN$

Buckling moment resistance y-y
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C} \cdot M_{ycRk}$ $\chi_{LT.C} = 0.9$ $M_{y.Rk.C} = 137.71\,kN \cdot m$

Buckling resistance moment z-z
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min\left(M_{z1cRk}, M_{z2cRk}\right)$ $M_{z.Rk.C} = 27.32\,kN \cdot m$

Resistance for one profile buckling mode
flexural buckling-: y-y $cc_{batt} = 1.5\,m$ $\chi_{y.FB}\left(cc_{batt}\right) = 1$ $N_{b.Rk.y.FB}\left(cc_{batt}\right) = 1007.26\,kN$

Resistance for one profile buckling mode
flexural buckling-: z-z $cc_{batt} = 1.5\,m$ $\chi_{z.FB}\left(cc_{batt}\right) = 0.87$ $N_{b.Rk.z.FB}\left(cc_{batt}\right) = 877.73\,kN$

Resistance for one profile buckling mode
-torsional or torsional-flexural $cc_{batt} = 1.5\,m$ $\chi_{TF}\left(cc_{batt}, cc_{batt}\right) = 0.88$ $N_{b.Rk.TF}\left(cc_{batt}, cc_{batt}\right) = 886.8\,kN$

single C-profile: Axial compression EN 1993-1-1: 6.3.1 (3): Buckling about relevant axis in mid-span or en panel of built-up member

$$\frac{\max\left(N_{ch.Ed}, N_{ch.end}\right) \cdot \gamma_{M1}}{\min\left(\chi_{y.FB}\left(cc_{batt}\right), \chi_{z.FB}\left(cc_{batt}\right), \chi_{TF}\left(cc_{batt}, cc_{batt}\right)\right) \cdot N_{c.Rk}} = 0.1 \qquad <1.0$$

Controll buil-up member: single C-profile (chords) in mid-span of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.Ed} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{\left(\left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.54 \qquad <1.0$$

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{y.FB}\left(cc_{batt}\right) \cdot N_{c.Rk}} + k_{yy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C.2} \cdot \frac{\left(\left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.5 \qquad <1.0$$

Evading in z-z:

$$\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{z.FB}\left(cc_{batt}\right) \cdot N_{c.Rk}} + k_{zy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C.2} \cdot \frac{\left(\left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.6 \qquad <1.0$$

Controll buil-up member: single C-profile (chords) at end panel of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.end} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{\left(M_{z.ch} + \left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.56 \qquad <1.0$$

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}\left(cc_{batt}\right) \cdot N_{c.Rk}} + k_{yy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C.2} \cdot \frac{\left(M_{z.ch} + \left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.51 \qquad <1.0$$

Evading in z-z:

$$\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}\left(cc_{batt}\right) \cdot N_{c.Rk}} + k_{zy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C.2} \cdot \frac{\left(M_{z.ch} + \left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.61 \qquad <1.0$$

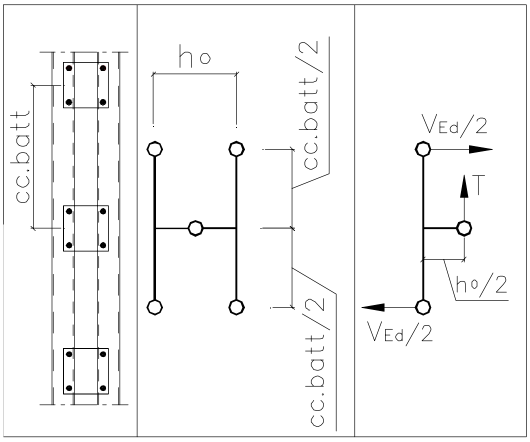
$b = 100\,mm \qquad h = 360\,mm \qquad c = 39\,mm \qquad t = 5\,mm \qquad f_{yb} = 500\,\frac{N}{mm^2} \qquad f_u = 550\,\frac{N}{mm^2} \qquad \gamma_{M0} = 1.00 \qquad \gamma_{M1} = 1.00$

Forces on battens: max shear flow in one batten:

$$T_{batt} := \frac{V_{Ed.max} \cdot cc_{batt}}{h_0 \cdot r_{CC}} \qquad T_{batt} = 5.06\,kN$$

max moment in one batten:

$$M_{batt} := \frac{T_{batt} \cdot h_0}{2} \qquad M_{batt} = 0.39\,kN \cdot m$$

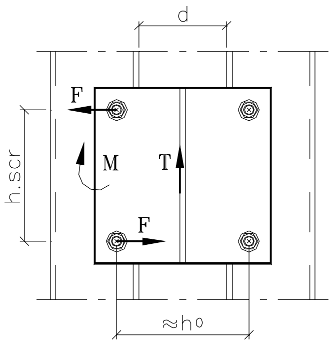


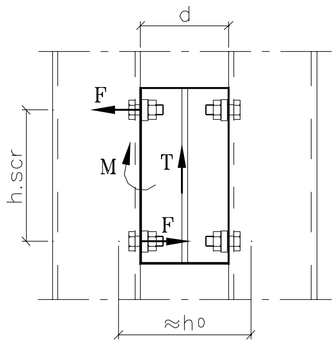
$$\lambda_{w.batt} := 0.346 \cdot \frac{h_{batt}}{t_{batt}} \cdot \sqrt{\frac{f_{yb}}{E}} \qquad f_{vb.batt} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{w.batt} \le 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{w.batt}} & \text{if } 0.83 < \lambda_{w.batt} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{w.batt}^2} & \text{if } \lambda_{w.batt} \ge 1.40 \end{cases}$$

$$V_{bbatt.Rd} := \frac{h_{batt} \cdot t_{batt} \cdot f_{vb.batt}}{\gamma_{M0}} \qquad V_{bbatt.Rd} = 227.45\,kN \qquad \frac{T_{batt}}{V_{bbatt.Rd}} = 0.02 \quad < 1{,}0$$

$$M_{Rd.batt} := \frac{f_{yb}}{\gamma_{M0}} \cdot \frac{t_{batt} \cdot h_{batt}^2}{6} \qquad M_{Rd.batt} = 13.33\,kN \cdot m \qquad \frac{M_{batt}}{M_{Rd.batt}} = 0.03 \quad < 1{,}0$$

Forces on screws:





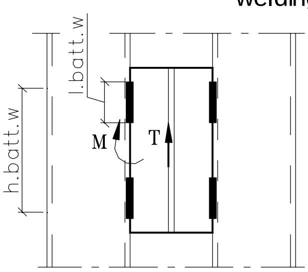
$h_{scr} := 150 \cdot mm \qquad n_c := 2$ numbers of screws on one side of one PXK

$$T_{batt} = 5.06\,kN \qquad F := \frac{T_{batt} \cdot h_0}{2 \cdot h_{scr}} \qquad F = 2.63\,kN$$

$$F_{scew} := \sqrt{F^2 + \left(\frac{T_{batt}}{n_c}\right)^2}$$

$F_{scew} = 3.65\,kN$ = force on one screw
< 30 kN

Forces on weld:



welding high: $h_{batt.w} := 150 \cdot mm$ welding length: $l_{batt.w} := 2 \cdot 40 \cdot mm$ welding t=: $a_{batt.w} := 4 \cdot mm$

$$\sigma_{batt.w} := \frac{3 \cdot T_{batt} \cdot h_0}{a_{batt.w} \cdot h_{batt.w}^2} \qquad \tau_{batt.w} := \frac{T_{batt}}{l_{batt.w} \cdot a_{batt.w}}$$

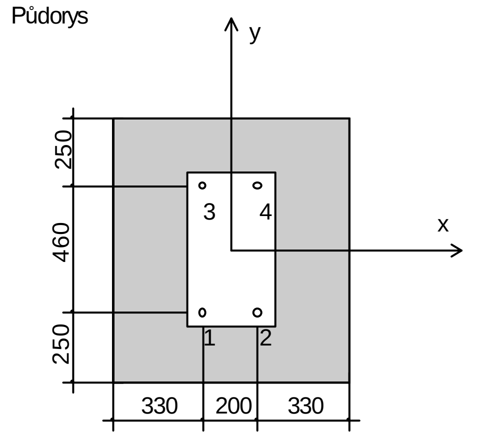
$$\sigma_{V.batt.w} := \sqrt{\sigma_{batt.w}^2 + 3 \cdot \tau_{batt.w}^2} \qquad \sigma_{V.batt.w} = 37.94\,\frac{N}{mm^2}$$ = stress in one weld

Aplikace PROFIS kotvy 1.8.0	Firma: LLENTAB	Strana 1 z 1
	Vypracoval: Kamil Patrman	Zákazník: Hala Zator
	Adresa:	Projekt: CZ1621
	Telefon/Fax: - / -	Kontaktní osoba:
http://www.hilti.com/	E-mail: kamil.patrman@llentab.cz	Datum: - / 04.02.2022

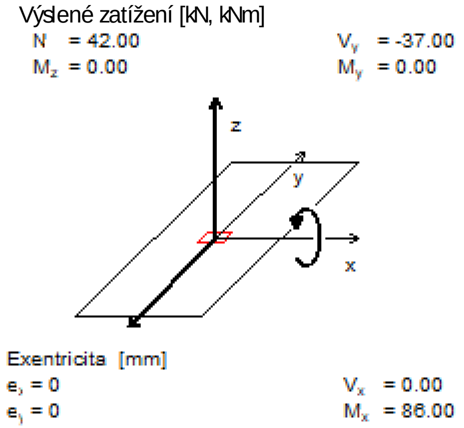
Poznámky:

Typ a rozměr kotvy:	**HIT-RE 500 + HAS-E (8.8)-M30**
Efektivní hloubka kotvení:	h_{ef}= 270 mm; součinitel hloubky osazení = 1.800
Materiál:	8.8
Certfikát:	
Platnost:	- / -
Zkouška:	Návrh podle SOFA - po ETAG zkoušce
Distanční montáž:	s upevnění na povrchu; plně podlití (kotevní deska); e_b = 50 mm; t = 20 mm
Kotevní deska:	S355 (ST52); ; l_x x l_y x t = 320 x 560 x 20 mm
Základní materiál:	netrhlinový Beton C20/25, f_{cc} = 25.00 N/mm$_2$; h = 10000 mm
Výztuž:	vzdálenost výztuže >= 150 mm
	s okrajovou podélnou výztuží d >= 12 mm + uzavřená výztuž třmínkového typu s <= 100 mm

Geometrie [mm]



Zatížení [kN]



Posouzení/Úroveň (Upravené případy)

		Výpočtová hodnota [kN]		Využití [%]	
Zatížení	Zkouška	Zatížení	Kapacita	β_N/β_V	Status
Tah	Betonový kužel	206.26	393.18	52 / -	OK
Smyk	Únosnost oceli (distanční montáž)	9.25	26.77	- / 35	OK

Zatížení	β_N	β_V	α	Využití $\beta_{N,V}$[%]	Status
Interakce	0.525	0.345	1.5	58	OK

Upozornění

Při použití HILTI dynamického setu se smykové zatížení distribuuje do kotev rovnoměrně
Za kompaktibilitu se současnými normami (např.EC3) zodpovídá uživatel
Předpokládá se suchá díra a standardní vyčištění! Vliv teploty je zanedbán!

Upevnění je bezpečné!

L.

Část konstrukce: BR1 Ztužidla podélná

Poznámky:

H. Ztužidla v podélné stěně

Základní informace projektu:

Šířka haly:	15.310	m	Horní pas:	17.35	°	Výška u žlabu:	8.30	m
Délka haly:	45.115	m	Dolní pás:	10.62	°	Výška hřebene:	10.69	m

Modulová vzdálenost rámů: **6.90** m

Síly v podélném stěnovém ztužení

Zatížení větrem ČSN EN 1991-1-4 ()

Kategorie terénu: II. Oblasti s nízkou vegetací jako je tráva nebo izolované překážky (stromy, budovy)

Charakteristická hodnota rychlosti větru v_{bo}= 25 m/s

Maximální dynamický tlak q_p (z) = 0.936 kN/m^2

Rozhodující směr větru: **Podélný**

Rozhoduje součet tlaku a sání z podélného větru na návětrném a závětrném štítu

Způsob výpočtu tlaku do štítu: **Počítat plochu automaticky**

Zatížení je počítáno automaticky z celého čtverce, výpočet na straně bezpečnosti

Automatický výpočet

Návětrná stěna (tlak)			**Závětrná stěna (sání)**		
Součinitel vnějšího tlaku c_{pe} =	0.70		Součinitel vnějšího tlaku c_{pe} =	0.30	
Výpočtová hodnota $w_{e\,(tlak)}$ =	0.65	kN/m^2	Výpočtová hodnota $w_{e\,(sání)}$ =	0.28	kN/m^2
Součinitel zatížení g_w =	1.5		Součinitel zatížení g_w =	1.5	
Normová hodnota $w_{d(tlak)}$ =	0.98	kN/m^2	Normová hodnota $w_{d\,(sání)}$ =	0.42	kN/m^2
Síly na horní část stěny w_{dn} =	**80.40**	kN	Síly na horní část stěny w_{dz} =	**34.46**	kN

Součet sání a tlaku do štítu = **114.85** kN

Tření větru o povrch haly ČSN EN 1991-1-4 ()

Délka na které dochází ke tření (vlna do 60mm) x =	14.50	m
Součinitel tření pro hrubé povrchy c_{fr} =	0.04	
Třecí šířka ve střeše a =	16.04	m
Síla od tření ve střeše w_{fd} =	**13.05**	kN
Třecí šířka ve stěně a =	8.30	m
Síla od tření ve stěně w_{fd} =	**6.75**	kN

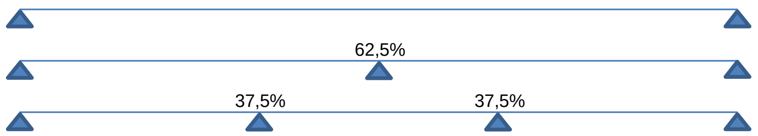
Zatížení jeřábovou dráhou

Podélná brzdná síla B_v = **0.00** kN (pouze je-li jeřábová dráha)

Rozložení sil do ztužidel

Podíl celkové síly do 1 řady ztužidel = **50** %

Statické schéma ztužidel:

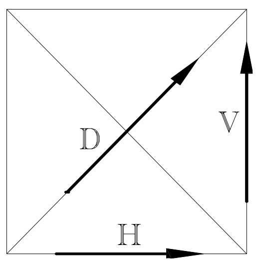


Typ ztužidel: **Jednoduchá ztužidla**

Stěna s jednoduchým ztužidlem

Typ zavětrovacích pásků: **Nové pásky 42x4mm**

Výška ztužidla h =	**8.30**	m
Počet ztužidel v jedné řadě =	**4**	
Síla přenášená do 1 řady ztužidel w_1 =	**67.33**	kN
Sklon ztužidla a =	50.26	°
Smyková síla do kotvení H =	**16.83**	kN
Tah/tlak do základů V =	**20.25**	kN
Síla v zavětrování D =	**26.33**	kN



Posouzení zavětrování

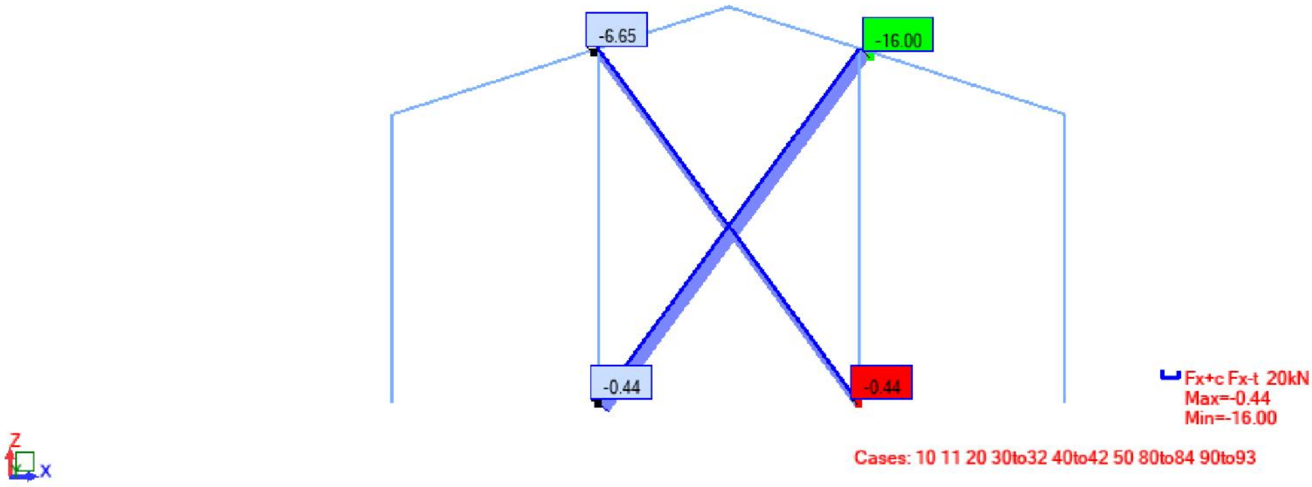
D = **26.33** kN < $R_{d,p}$ = **43.00** kN

ZAVĚTROVÁNÍ VYHOVUJE

Část konstrukce: BR2 Ztužidla štítová

Poznámky:

1xP4x42
Nrd=43kN
VYHOVUJE



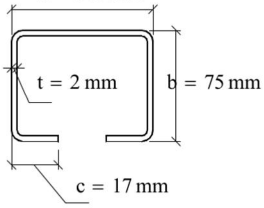
O.

Část konstrukce: DO1 Únosnosti profilů

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section HB

kb = 268mm



$M_{ycRd} = 5.81\,kN\cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 2.89\,kN\cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 3.06\,kN\cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 39.61\,kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 52.54\,kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 59.01\,kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 20.8\,kN$	Reaction force (RF) resistance web, 100mm support (6.1.7)
$R_{w.Rd1.1} = 11.02\,kN$	RF resistance web, 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 28.51\,kN$	RF resistance web, 200mm support (6.1.7)
$R_{w.Rd1.2} = 14.69\,kN$	RF resistance web, 200mm support close to end (6.1.7)
$R_{w.Rd4.1} = 23.91\,kN$	RF resistance restrained web, 100mm supp. (6.1.7)
$R_{w.Rd3.1} = 24.73\,kN$	RF resistance restrained web, 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 193.83\,kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 148.97\,kN$	Axial compression resistance (6.1.3):

$h = 100\,mm$

$b = 75\,mm$

$c = 17\,mm$

$t = 2\,mm$

$A_g = 531.40\,mm^2$

$f_{yb} = 350\,\frac{N}{mm^2}$

$f_u = 420\,\frac{N}{mm^2}$

$e_1 = 28.43\,mm$

$y_M = 37.65\,mm$

$W_y = 1.87 \times 10^4\,mm^3$

$W_{z1} = 1.47 \times 10^4\,mm^3$

$W_{z2} = 8860.64\,mm^3$

$I_y = 9.16 \times 10^5\,mm^4$

$I_z = 4.04 \times 10^5\,mm^4$

$E\cdot I_y = 192325\,N\cdot m^2$

$E\cdot I_z = 84787\,N\cdot m^2$

$i_y = 41.51\,mm$

$i_z = 27.56\,mm$

$I_T = 736\,mm^4$

$I_\omega = 8.67 \times 10^8\,mm^6$

$g_p = 4.16\,\frac{kg}{m}$

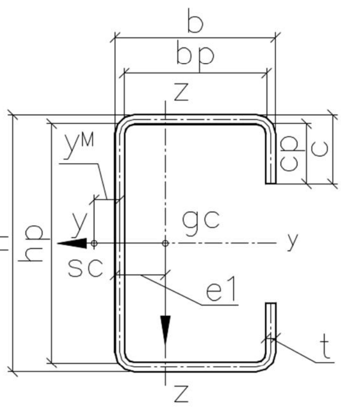
Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 1.52\,mm$

$L_z := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$ $L_y := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)
0.5	148.97	148.39
1	144.70	136.65
1.5	136.75	122.19
2	127.59	103.84
2.5	116.72	83.82
3	104.16	66.10
3.5	90.79	52.28
4	77.92	41.94
4.5	66.49	34.21
5	56.82	28.36
5.5	48.81	23.86
6	42.23	20.33
6.5	36.80	17.52
7	32.31	15.25
7.5	28.57	13.40
8	25.42	11.85
8.5	22.76	10.56
9	20.49	9.47
9.5	18.53	8.54
10	16.84	7.74

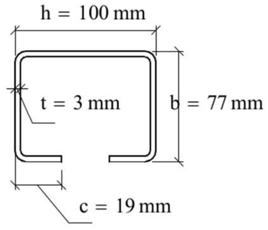
	$N_{TF.Rd}$ (kN)
0.5	138.62
1	110.73
1.5	76.81
2	52.34
2.5	37.81
3	29.02
3.5	23.38
4	19.57
4.5	16.86
5	14.86
5.5	13.33
6	12.13
6.5	11.16
7	10.36
7.5	9.68
8	9.09
8.5	8.58
9	8.13
9.5	7.72
10	7.35



Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Section OB

kb = 268mm



$M_{ycRd} = 9.49\,kN\cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 4.91\,kN\cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 4.91\,kN\cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 58.8\,kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 80.02\,kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 89.72\,kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 46.22\,kN$	Reaction force (RF) resistance web, 100mm support (6.1.7)
$R_{w.Rd1.1} = 26.46\,kN$	RF resistance web, 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 55.59\,kN$	RF resistance web, 200mm support (6.1.7)
$R_{w.Rd1.2} = 33.07\,kN$	RF resistance web, 200mm support close to end (6.1.7)
$R_{w.Rd4.1} = 43.8\,kN$	RF resistance restrained web, 100mm supp. (6.1.7)
$R_{w.Rd3.1} = 49.78\,kN$	RF resistance restrained web, 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 299.92\,kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 269.74\,kN$	Axial compression resistance (6.1.3):

$h = 100\,mm$

$b = 77\,mm$

$c = 19\,mm$

$t = 3\,mm$

$A_g = 806.52\,mm^2$

$f_{yb} = 350\,\frac{N}{mm^2}$

$f_u = 420\,\frac{N}{mm^2}$

$e_1 = 29.97\,mm$

$y_M = 38.94\,mm$

$W_y = 2.8 \times 10^4\,mm^3$

$W_{z1} = 2.24 \times 10^4\,mm^3$

$W_{z2} = 1.4 \times 10^4\,mm^3$

$I_y = 1.36 \times 10^6\,mm^4$

$I_z = 6.39 \times 10^5\,mm^4$

$E\cdot I_y = 285635\,N\cdot m^2$

$E\cdot I_z = 134090\,N\cdot m^2$

$i_y = 41.07\,mm$

$i_z = 28.14\,mm$

$I_T = 2520\,mm^4$

$I_\omega = 1.42 \times 10^9\,mm^6$

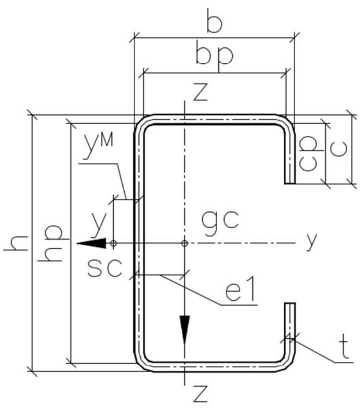
$g_p = 6.28\,\frac{kg}{m}$

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 1.5\,mm$

$L_z := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m \quad L_y := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$

L_z = (m)	$N_{b.Rd.y.FB}(L_y)$ = (kN)	$N_{b.Rd.z.FB}(L_z)$ = (kN)		$N_{TF.Rd}$ (kN)
0.5	269.74	267.27	0.5	247.91
1	259.13	244.14	1	192.65
1.5	242.72	214.83	1.5	131.49
2	223.23	177.84	2	91.39
2.5	199.81	139.81	2.5	68.34
3	173.49	108.32	3	54.45
3.5	147.19	84.79	3.5	45.47
4	123.62	67.61	4	39.27
4.5	103.88	54.94	4.5	34.75
5	87.83	45.44	5	31.29
5.5	74.91	38.15	5.5	28.53
6	64.46	32.47	6	26.26
6.5	55.97	27.96	6.5	24.32
7	49.00	24.32	7	22.64
7.5	43.22	21.34	7.5	21.16
8	38.39	18.87	8	19.82
8.5	34.32	16.81	8.5	18.61
9	30.85	15.07	9	17.51
9.5	27.88	13.58	9.5	16.49
10	25.32	12.31	10	15.55

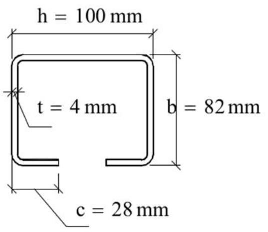


LLENTAB
MORE THAN STEEL BUILDINGS

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Section VB

kb = 290mm



$M_{ycRd} = 16.5\,kN{\cdot}m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 10.31\,kN{\cdot}m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 10.31\,kN{\cdot}m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 93.12\,kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 143.55\,kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 151.31\,kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 90.08\,kN$	Reaction force (RF) resistance web (h), 100mm support (6.1.7)
$R_{w.Rd1.1} = 50.44\,kN$	RF resistance web (h), 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 103.49\,kN$	RF resistance web (h), 200mm support (6.1.7)
$R_{w.Rd1.2} = 60.53\,kN$	RF resistance web (h), 200mm support close to end (6.1.7)
$R_{w.Rd4.1} = 69.69\,kN$	RF resistance restrained web (h), 100mm supp. (6.1.7)
$R_{w.Rd3.1} = 99.17\,kN$	RF resistance restrained web (h), 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 517.41\,kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 488.08\,kN$	Axial compression resistance (6.1.3):

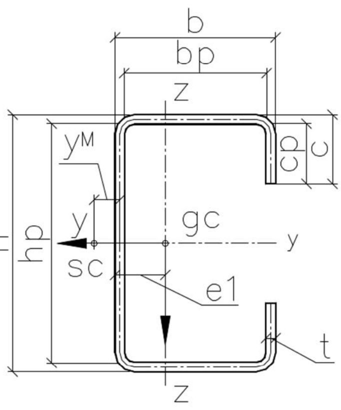
Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 0.16\,mm$

$L_z := 0.5{\cdot}m, 1.0{\cdot}m\,..\,10.0{\cdot}m \quad L_y := 0.5{\cdot}m, 1.0{\cdot}m\,..\,10.0{\cdot}m$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)
0.5	488.08	482.67
1	460.84	439.52
1.5	424.95	384.28
2	380.65	314.93
2.5	327.91	245.31
3	273.05	188.98
3.5	223.82	147.46
4	183.55	117.35
4.5	151.83	95.26
5	127.04	78.71
5.5	107.56	66.06
6	92.09	56.19
6.5	79.65	48.37
7	69.53	42.06
7.5	61.19	36.90
8	54.25	32.63
8.5	48.42	29.06
9	43.48	26.05
9.5	39.25	23.47
10	35.60	21.27

	$N_{TF.Rd}$ (kN)
0.5	447.98
1	347.17
1.5	236.72
2	164.83
2.5	123.46
3	98.37
3.5	81.97
4	70.51
4.5	62.02
5	55.42
5.5	50.08
6	45.63
6.5	41.83
7	38.52
7.5	35.59
8	32.99
8.5	30.65
9	28.53
9.5	26.61
10	24.87



$h = 100\,mm$

$b = 82\,mm$

$c = 28\,mm$

$t = 4\,mm$

$A_g = 1167.93\,mm^2$

$f_{yb} = 420\,\dfrac{N}{mm^2}$

$f_u = 480\,\dfrac{N}{mm^2}$

$e_1 = 35.12\,mm$

$y_M = 44.9\,mm$

$W_y = 3.93 \times 10^4\,mm^3$

$W_{z1} = 3.33 \times 10^4\,mm^3$

$W_{z2} = 2.45 \times 10^4\,mm^3$

$I_y = 1.89 \times 10^6\,mm^4$

$I_z = 1.1 \times 10^6\,mm^4$

$E{\cdot}I_y = 396050\,N{\cdot}m^2$

$E{\cdot}I_z = 231314\,N{\cdot}m^2$

$i_y = 40.18\,mm$

$i_z = 30.71\,mm$

$I_T = 6485.33\,mm^4$

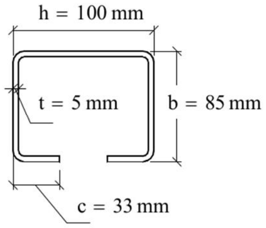
$I_\omega = 3.25 \times 10^9\,mm^6$

$g_p = 9.05\,\dfrac{kg}{m}$

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section FB

kb = 300mm



$M_{ycRd} = 24.8\ kN \cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 17.31\ kN \cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 17.31\ kN \cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 137.12\ kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 225.17\ kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 230.94\ kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 150.46\ kN$	Reaction force (RF) resistance web, 100mm support (6.1.7)
$R_{w.Rd1.1} = 79.34\ kN$	RF resistance web, 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 168.94\ kN$	RF resistance web, 200mm support (6.1.7)
$R_{w.Rd1.2} = 92.56\ kN$	RF resistance web, 200mm support close to end (6.1.7)
$R_{w.Rd4.1} = 110.17\ kN$	RF resistance restrained web, 100mm supp. (6.1.7)
$R_{w.Rd3.1} = 175.84\ kN$	RF resistance restrained web, 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 792.81\ kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 757.81\ kN$	Axial compression resistance (6.1.3):

$h = 100\ mm$

$b = 85\ mm$

$c = 33\ mm$

$t = 5\ mm$

$A_g = 1515.62\ mm^2$

$f_{yb} = 500 \frac{N}{mm^2}$

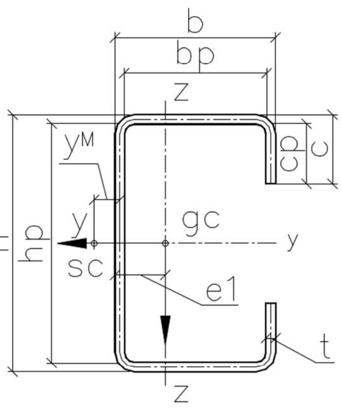
$f_u = 550 \frac{N}{mm^2}$

$L_z := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$ $L_y := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$

Change of centre of gravity in y-direction, due to effective plates under compression:
(**-**) = to the right

$\Delta e_N = 0\ mm$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)	L	$N_{TF.Rd}$ (kN)
0.5	757.81	746.05	0.5	690.97
1	703.60	674.30	1	527
1.5	637.76	580.44	1.5	355.5
2	554.63	464.70	2	248.3
2.5	459.72	354.90	2.5	187.16
3	369.86	270.26	3	149.84
3.5	296.13	209.54	3.5	125.11
4	239.30	166.14	4	107.51
4.5	196.11	134.54	4.5	94.22
5	163.09	110.99	5	83.71
5.5	137.49	93.04	5.5	75.1
6	117.35	79.08	6	67.86
6.5	101.26	68.02	6.5	61.65
7	88.23	59.11	7	56.25
7.5	77.54	51.84	7.5	51.51
8	68.67	45.83	8	47.31
8.5	61.22	40.80	8.5	43.58
9	54.92	36.56	9	40.24
9.5	49.55	32.94	9.5	37.25
10	44.92	29.83	10	34.55



$e_1 = 38.01\ mm$

$y_M = 47.78\ mm$

$W_y = 4.96 \times 10^4\ mm^3$

$W_{z1} = 4.34 \times 10^4\ mm^3$

$W_{z2} = 3.46 \times 10^4\ mm^3$

$I_y = 2.36 \times 10^6\ mm^4$

$I_z = 1.54 \times 10^6\ mm^4$

$E \cdot I_y = 494738\ N \cdot m^2$

$E \cdot I_z = 323343\ N \cdot m^2$

$i_y = 39.43\ mm$

$i_z = 31.87\ mm$

$I_T = 1.32 \times 10^4\ mm^4$

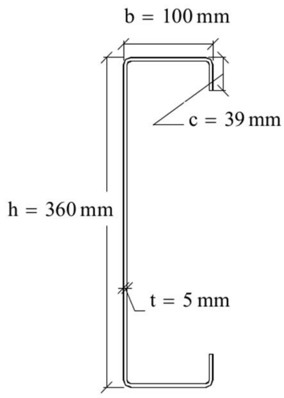
$I_\omega = 5.32 \times 10^9\ mm^6$

$g_p = 11.68 \frac{kg}{m}$

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Section IA

kb = 600mm



$M_{ycRd} = 152.53\,kN\cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 27.71\,kN\cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 27.32\,kN\cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 355.39\,kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 460.75\,kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 274.24\,kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 139.42\,kN$	Reaction force (RF) resistance web (h), 100mm support (6.1.7)
$R_{w.Rd1.1} = 71.45\,kN$	RF resistance web (h), 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 156.54\,kN$	RF resistance web (h), 200mm support (6.1.7)
$R_{w.Rd1.2} = 83.36\,kN$	RF resistance web (h), 200mm support close to end (6.1.7)
$R_{w.Rd4.1} = 110.17\,kN$	RF resistance restrained web (h), 100mm supp. (6.1.7)
$R_{w.Rd3.1} = 187.73\,kN$	RF resistance restrained web (h), 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 1547.81\,kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 1007.26\,kN$	Axial compression resistance (6.1.3):

$h = 360\,mm$

$b = 100\,mm$

$c = 39\,mm$

$t = 5\,mm$

$A_g = 3025.62\,mm^2$

$f_{yb} = 500\,\frac{N}{mm^2}$

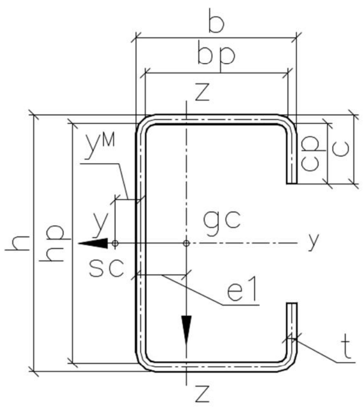
$f_u = 550\,\frac{N}{mm^2}$

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = -8.2\,mm$

$L_z := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m \quad L_y := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)		$N_{TF.Rd}$ (kN)
0.5	1007.26	1007.26	0.5	1007.26
1	1007.26	951.45	1	955.72
1.5	1007.26	877.73	1.5	886.76
2	1007.26	786.79	2	804.71
2.5	995.37	678.45	2.5	709.48
3	978.49	565.49	3	610.02
3.5	961.21	463.88	3.5	518.5
4	943.37	380.60	4	441.55
4.5	924.78	314.93	4.5	379.63
5	905.29	263.56	5	330.53
5.5	884.72	223.18	5.5	291.56
6	862.95	191.10	6	260.4
6.5	839.86	165.30	6.5	235.22
7	815.40	144.30	7	214.65
7.5	789.54	127.00	7.5	197.67
8	762.37	112.61	8	183.49
8.5	734.02	100.50	8.5	171.56
9	704.74	90.24	9	161.41
9.5	674.81	81.46	9.5	152.71
10	644.59	73.90	10	145.19



$e_1 = 27.86\,mm$

$y_M = 43.18\,mm$

$W_y = 3.15 \times 10^5\,mm^3$

$W_{z1} = 1.54 \times 10^5\,mm^3$

$W_{z2} = 5.62 \times 10^4\,mm^3$

$I_y = 5.58 \times 10^7\,mm^4$

$I_z = 3.92 \times 10^6\,mm^4$

$E\cdot I_y = 11724166\,N\cdot m^2$

$E\cdot I_z = 822335\,N\cdot m^2$

$i_y = 135.84\,mm$

$i_z = 35.98\,mm$

$I_T = 2.57 \times 10^4\,mm^4$

$I_\omega = 1.03 \times 10^{11}\,mm^6$

$g_p = 23.54\,\frac{kg}{m}$

LLENTAB
MORE THAN STEEL BUILDINGS

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section AA

kb = 400mm

$M_{yRd} = 30.73\ kN \cdot m$ Bending moment resistance y-y (6.1.4)

$M_{z1Rd} = 33.76\ kN \cdot m$ Bending moment resistance z-z tension in web (6.1.4)

$M_{z2Rd} = 33.77\ kN \cdot m$ Bending moment resistance z-z compression in web (6.1.4)

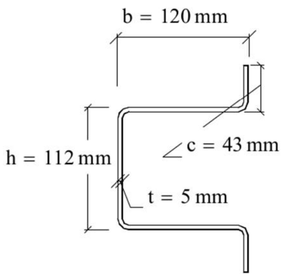
$V_{bh.Rd} = 154.44\ kN$ Shear force resistance perpendicular to y-axis web only (6.1.5)

$V_{bhc.Rd} = 271.35\ kN$ Shear force resistance perpendicular to y-axis web+lips (6.1.5)

$V_{bb.Rd} = 331.98\ kN$ Shear force resistance perpendicular to z-axis (6.1.5)

$N_{t.Rd} = 1047.81\ kN$ Axial tension resistance (6.1.2)

$N_{c.Rd} = 981.63\ kN$ Axial compression resistance (6.1.3):



Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

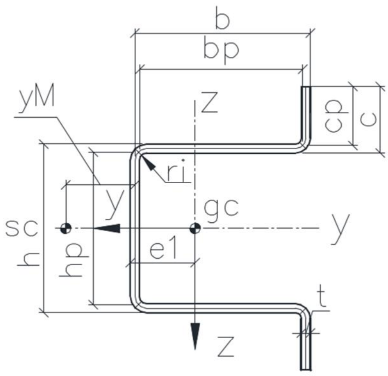
$\Delta e_N = 1.39\ mm$

$L_z := 1.0 \cdot m, 1.5 \cdot m .. 15.0 \cdot n$ $L_y := 1.0 \cdot m, 1.5 \cdot m .. 15.0 \cdot m$

$L_z =$		$N_{b.Rd.y.FB}(L_y)$		$N_{b.Rd.z.FB}(L_z) =$	
1.00	m	952.19	kN	932.35	kN
1.50		898.90		864.56	
2.00		837.28		781.86	
2.50		763.95		682.68	
3.00		679.45		576.35	
3.50		590.13		477.46	
4.00		504.94		394.29	
4.50		429.92		327.62	
5.00		366.81		274.95	
5.50		314.78		233.27	
6.00		272.10		200.01	
6.50		237.03		173.18	
7.00		208.02		151.30	
7.50		183.86		133.25	
8.00		163.57		118.20	
8.50		146.39		105.54	
9.00		131.75		94.80	
9.50		119.16		85.60	
10.00		108.28		77.68	
10.50		98.81		70.80	
11.00		90.53		64.79	
11.50		83.23		59.51	
12.00		76.78		54.86	
12.50		71.05		50.72	
13.00		65.93		47.04	
13.50		61.35		43.74	
14.00		57.22		40.78	
14.50		53.50		38.10	
15.00		50.13		35.68	

$N_{TF.Rd}$

0.5	852.85	kN
1	566.2	
1.5	347.1	
2	238.47	
2.5	181.78	
3	148.85	
3.5	127.89	
4	113.52	
4.5	103.03	
5	94.95	
5.5	88.45	
6	83.01	
6.5	78.31	
7	74.15	
7.5	70.39	
8	66.94	
8.5	63.74	
9	60.74	
9.5	57.93	
10	55.26	
10.5	52.74	
11	50.35	
11.5	48.08	
12	45.93	
12.5	43.88	
13	41.94	
13.5	40.1	
14	38.35	
14.5	36.69	
15	35.12	



$h = 112\ mm$

$b = 120\ mm$

$c = 43\ mm$

$t = 5\ mm$

$A_g = 2025.62\ mm^2$

$f_{yb} = 500 \dfrac{N}{mm^2}$

$f_u = 550 \dfrac{N}{mm^2}$

$e_1 = 56.31\ mm$

$y_M = 49.01\ mm$

$W_y = 6.42 \times 10^4\ mm^3$

$W_{z1} = 7.68 \times 10^4\ mm^3$

$W_{z2} = 6.75 \times 10^4\ mm^3$

$I_y = 5.88 \times 10^6\ mm^4$

$I_z = 4.13 \times 10^6\ mm^4$

$E \cdot I_y = 1234446\ N \cdot m^2$

$E \cdot I_z = 867806\ N \cdot m^2$

$i_y = 53.87\ mm$

$i_z = 45.17\ mm$

$I_T = 1.74 \times 10^4\ mm^4$

$I_\omega = 6.60 \times 10^9\ mm^6$

$g_p = 15.69 \dfrac{kg}{m}$

R.

Část konstrukce: FR1 Rám standard R15

Poznámky:
Zatěžovací šířka rámu Lw=7m.
Rám je posouzen na zatížení při požáru R15.

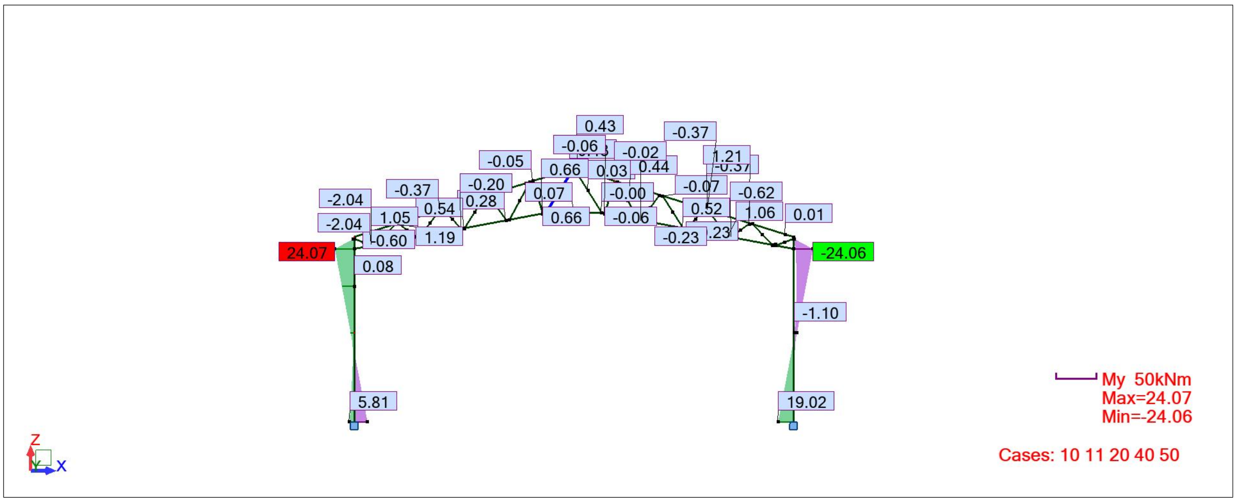
Combinations

- Cases: 10 11 20 40 50

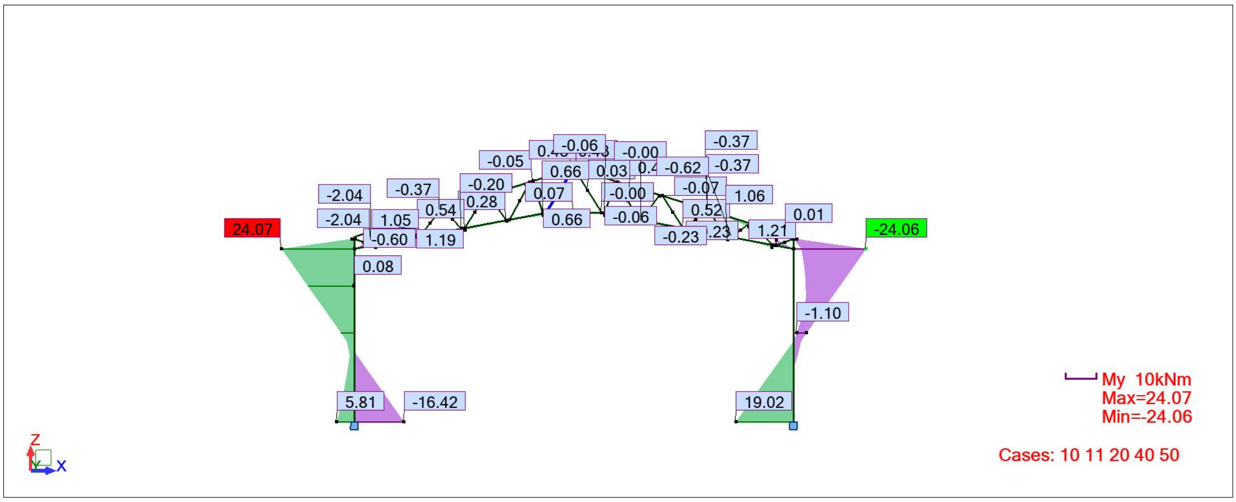
Combinations	Name	Analysis type	Combination	Case nature
10 (C)	R15 Snow	Linear Combinati	ULS	snow
11 (C)	R15 Snow 0.5 rig	Linear Combinati	ULS	snow
20 (C)	R15 Wind Cpe m	Linear Combinati	ULS	wind
40 (C)	R15 Wind Cpe m	Linear Combinati	ULS	snow
50 (C)	R15 Wind Cpe m	Linear Combinati	ULS	snow

Combinations	Definition
10 (C)	(101+102+104)*1.00+(201+202)*0.20
11 (C)	(101+102+104)*1.00+201*0.20+202*0.10
20 (C)	(101+102)*1.00+(300+321)*0.20
40 (C)	(101+102+104)*1.00+(301+320)*0.20
50 (C)	(101+102)*1.00+(310+321)*0.20

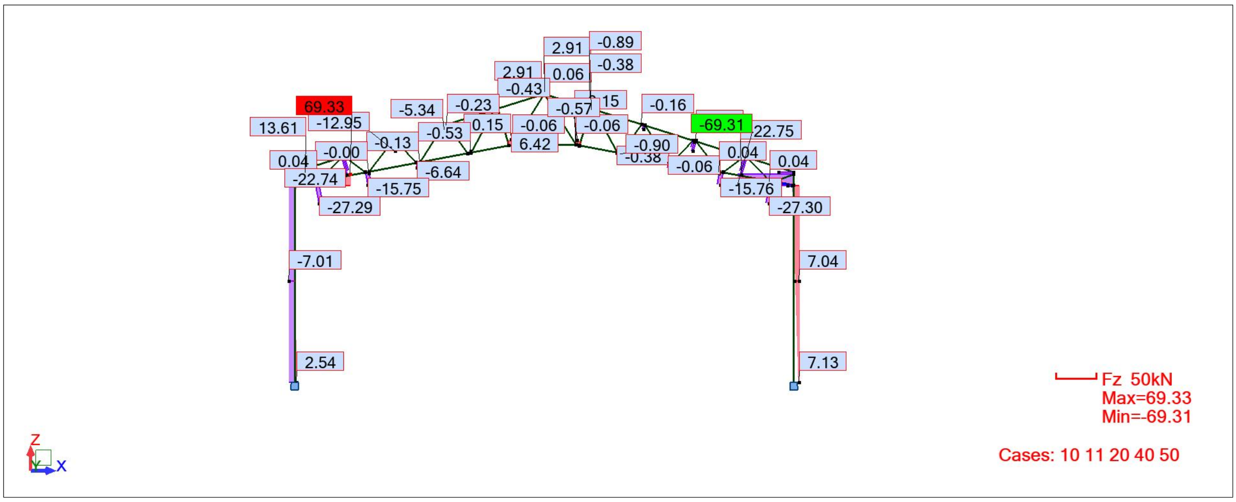
View - MY; Cases: 10 11 20 40 50



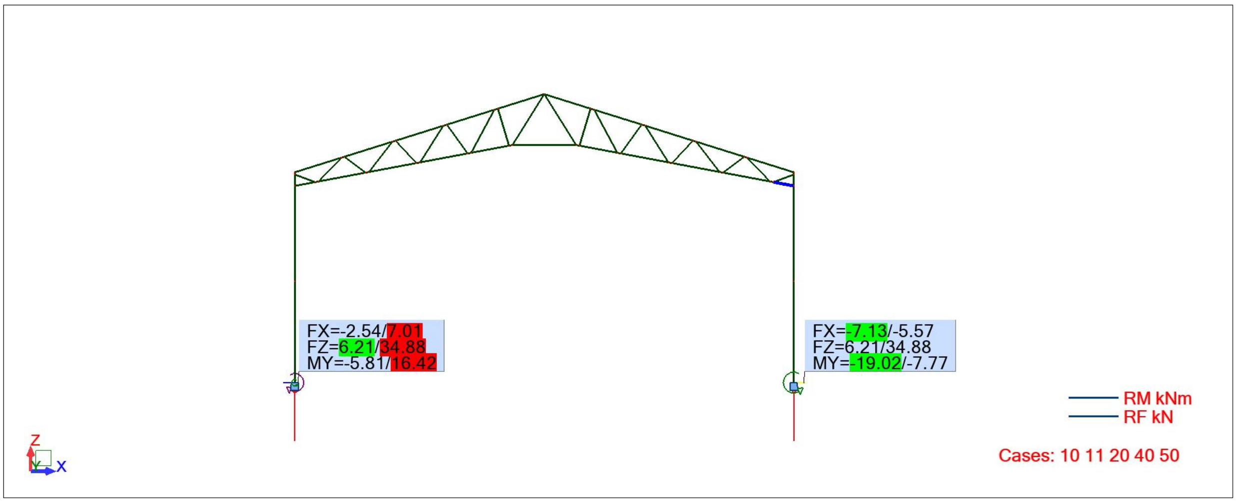
View - MY; Cases: 10 11 20 40 50 1



View - FZ; Cases: 10 11 20 40 50



View - Reaction forces(kN);Reaction moments(kN*m); Cases: 10 11 20 40 50



Vazník FR1 R15 diagonály

Popis:

Poznámky:
Datum: 2022.01.18 @ 13:19:50
Importováno ze souboru: CZ1621 FR1 R15.rtd

:

Diagonály
50 87 % Spoj: Střih 10 R15 Snow

Parametry posouzení prvků:

Typ	Popis	L_y	Koef.	L_y,koef	Křivka Y	L_z	Koef.	L_z,koef	Křivka Z	L_ft	L_t	Tenkost.
Bottom	Bottom beam 6.0m	6	False	1	b	0.7	True	1	b	True	True	True
Strut	Strut 2 screw	1	True	1	b	0.7	True	1	b	True	False	True
Top	Top beam	1	True	1	b	0.7	True	1	b	True	True	True
Other	Simple member	1	True	0	auto	1	True	0	auto	False	False	False
Strut	Strut 1 screw	1	True	1	b	0.7	True	1	b	True	False	True
Other	Frame distributed	0.33	False	1	b	0.5	True	1	b	True	False	True
Top	Top beam SPH	0.33	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 1.5m	1.5	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 3.0m	3	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 4.5m	4.5	False	1	b	0.7	True	1	b	True	True	True

Materiál:

Prvek	Typ	Member type	Profil	Délka [m]	Materiál	Hmotnost [kg]	A [m²]	A_eff [m²]	Komb. N_Ed
1	Strut	Strut 2 screw	C 100x85x5	1.762	HX500LAD	11.45	1580	1580	10
2	Strut	Strut 2 screw	C 100x85x5	1.761	HX500LAD	11.45	1580	1580	10
8	Strut	Strut 2 screw	C 100x77x3	1.134	S350GD	6.21	840	840	11
9	Strut	Strut 2 screw	C 100x77x3	1.134	S350GD	6.21	840	840	10
14	Strut	Strut 1 screw	C 100x75x2	1.475	S350GD	4.14	552	552	40
15	Strut	Strut 1 screw	C 100x75x2	1.475	S350GD	4.14	552	472.35	20
20	Strut	Strut 1 screw	C 100x75x2	1.059	S350GD	4.14	552	472.35	40
21	Strut	Strut 1 screw	C 100x75x2	1.059	S350GD	4.14	552	552	40
26	Strut	Strut 2 screw	C 100x77x3	1.274	S350GD	6.21	840	840	10
27	Strut	Strut 2 screw	C 100x77x3	1.271	S350GD	6.21	840	840	10
32	Strut	Strut 2 screw	C 100x77x3	0.898	S350GD	6.21	840	840	10
33	Strut	Strut 2 screw	C 100x77x3	0.899	S350GD	6.21	840	840	10
38	Strut	Strut 2 screw	C 100x82x4	1.123	HX420LAD	8.99	1166.8	1166.8	10
39	Strut	Strut 2 screw	C 100x82x4	1.122	HX420LAD	8.99	1166.8	1166.8	10
44	Strut	Strut 2 screw	C 100x82x4	0.776	HX420LAD	8.99	1166.8	1166.8	10
45	Strut	Strut 2 screw	C 100x82x4	0.775	HX420LAD	8.99	1166.8	1166.8	10
50	Strut	Strut 2 screw	C 100x82x4	0.945	HX420LAD	8.99	1166.8	1166.8	10
51	Strut	Strut 2 screw	C 100x82x4	0.95	HX420LAD	8.99	1166.8	1166.8	10
56	Strut	Strut 2 screw	C 100x85x5	0.671	HX500LAD	11.45	1580	1580	10

57	Strut	Strut 2 screw	C 100x85x5	0.668	HX500LAD	11.45	1580	1580	10

Profily:

Prvek	Profil	N_c [%]	N_t [%]	NcMz [%]	NcMz2 [%]	V_y [%]	N_t [%]	N_b [%]	NyMy Mz [%]	NzMy Mz [%]	Profil [%]	Rozh. profil	Komb. N_Ed
1	C 100x85x5	0	6	7	6	0	0	0	0	0	7	Tah a ohyb	10
2	C 100x85x5	0	6	7	6	0	0	0	0	0	7	Tah a ohyb	10
8	C 100x77x3	10	0	0	0	0	13	19	0	0	19	Vzpěr	11
9	C 100x77x3	9	0	0	0	0	13	17	0	0	17	Vzpěr	10
14	C 100x75x2	0	7	7	5	0	0	0	0	0	7	Prostý tah	40
15	C 100x75x2	5	0	6	5	0	17	9	7	7	9	Štíhlost	20
20	C 100x75x2	8	0	8	8	0	12	11	10	10	11	Štíhlost	40
21	C 100x75x2	0	3	3	3	0	0	0	0	0	3	Prostý tah	40
26	C 100x77x3	13	0	0	0	0	15	29	0	0	29	Vzpěr	10
27	C 100x77x3	13	0	0	0	0	15	29	0	0	29	Vzpěr	10
32	C 100x77x3	0	13	13	13	0	0	0	0	0	13	Prostý tah	10
33	C 100x77x3	0	12	13	12	0	0	0	0	0	13	Tah a ohyb	10
38	C 100x82x4	18	0	0	0	0	13	32	0	0	32	Vzpěr	10
39	C 100x82x4	18	0	0	0	0	13	32	0	0	32	Vzpěr	10
44	C 100x82x4	0	16	16	15	0	0	0	0	0	16	Prostý tah	10
45	C 100x82x4	0	16	16	15	0	0	0	0	0	16	Prostý tah	10
50	C 100x82x4	35	0	0	0	0	12	57	0	0	57	Vzpěr	10
51	C 100x82x4	35	0	0	0	0	12	57	0	0	57	Vzpěr	10
56	C 100x85x5	0	19	19	19	0	0	0	0	0	19	Prostý tah	10
57	C 100x85x5	0	19	19	19	0	0	0	0	0	19	Prostý tah	10

Spoje:

Prvek	Profil	Spoj	N_t,Ed [kN]	N_c,Ed [kN]	F_v [%]	F_b,c [%]	F_b,t [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
1	C 100x85x5	C54	-19.54	-1.9	68	3	36	3	6	68	Střih
2	C 100x85x5	C54	-19.55	-3.2	68	6	36	3	6	68	Střih
8	C 100x77x3	C36v2	-0.45	6.79	26	32	2	0	0	32	Otlačení
9	C 100x77x3	C36v2	-0.85	5.87	23	28	5	0	0	28	Otlačení
14	C 100x75x2	C26v2	-2.8	0.7	11	5	24	2	2	24	Otlačení
15	C 100x75x2	C26v2	-0.65	1.09	4	8	6	0	0	8	Otlačení
20	C 100x75x2	C24v2	-0.87	1.57	10	16	9	1	1	16	Otlačení
21	C 100x75x2	C24v2	-1.38	-0.27	8	3	14	1	1	14	Otlačení
26	C 100x77x3	C36v2	0.39	9.16	35	43	2	0	0	43	Otlačení
27	C 100x77x3	C36v2	1.59	8.99	34	43	9	1	1	43	Otlačení
32	C 100x77x3	C36v2	-9.1	-0.86	35	4	50	4	5	50	Otlačení
33	C 100x77x3	C36v2	-8.97	-1.87	34	9	50	4	5	50	Otlačení
38	C 100x82x4	C46v2	3.19	24.81	74	62	9	1	1	74	Střih
39	C 100x82x4	C46v2	3.46	24.82	74	62	10	1	1	74	Střih
44	C 100x82x4	C46v2	-22.97	-3.41	68	9	64	5	7	68	Střih
45	C 100x82x4	C48v2	-22.98	-3.58	51	7	48	5	7	51	Střih
50	C 100x82x4	C410v2	6.28	48.68	87	77	11	1	1	87	Střih
51	C 100x82x4	C410v	6.29	48.68	87	77	11	2	1	87	Střih

		2									
56	C 100x85x5	C510v 2	-59.5	-7.92	83	7	54	10	9	83	Střih
57	C 100x85x5	C510v 2	-59.52	-8.13	83	7	54	10	9	83	Střih

Souhrn:

Prvek	Member type	Profil	Materiál	Komb. N_Ed	Profil [%]	Spoj [%]	Vyhodnoceni	Posouzení
1	Strut 2 screw	C 100x85x5	HX500LAD	10	7	68	Střih	Vyhovuje
2	Strut 2 screw	C 100x85x5	HX500LAD	10	7	68	Střih	Vyhovuje
8	Strut 2 screw	C 100x77x3	S350GD	11	19	32	Otlačení	Vyhovuje
9	Strut 2 screw	C 100x77x3	S350GD	10	17	28	Otlačení	Vyhovuje
14	Strut 1 screw	C 100x75x2	S350GD	40	7	24	Otlačení	Vyhovuje
15	Strut 1 screw	C 100x75x2	S350GD	20	9	8	Štíhlost	Vyhovuje
20	Strut 1 screw	C 100x75x2	S350GD	40	11	16	Otlačení	Vyhovuje
21	Strut 1 screw	C 100x75x2	S350GD	40	3	14	Otlačení	Vyhovuje
26	Strut 2 screw	C 100x77x3	S350GD	10	29	43	Otlačení	Vyhovuje
27	Strut 2 screw	C 100x77x3	S350GD	10	29	43	Otlačení	Vyhovuje
32	Strut 2 screw	C 100x77x3	S350GD	10	13	50	Otlačení	Vyhovuje
33	Strut 2 screw	C 100x77x3	S350GD	10	13	50	Otlačení	Vyhovuje
38	Strut 2 screw	C 100x82x4	HX420LAD	10	32	74	Střih	Vyhovuje
39	Strut 2 screw	C 100x82x4	HX420LAD	10	32	74	Střih	Vyhovuje
44	Strut 2 screw	C 100x82x4	HX420LAD	10	16	68	Střih	Vyhovuje
45	Strut 2 screw	C 100x82x4	HX420LAD	10	16	51	Střih	Vyhovuje
50	Strut 2 screw	C 100x82x4	HX420LAD	10	57	87	Střih	Vyhovuje
51	Strut 2 screw	C 100x82x4	HX420LAD	10	57	87	Střih	Vyhovuje
56	Strut 2 screw	C 100x85x5	HX500LAD	10	19	83	Střih	Vyhovuje
57	Strut 2 screw	C 100x85x5	HX500LAD	10	19	83	Střih	Vyhovuje

Vazník FR1 R15 horní pás

Popis:

Poznámky:
Datum: 2022.01.18 @ 13:19:50
Importováno ze souboru: CZ1621 FR1 R15.rtd

Horní pas
4 96 % Spoj: Střih 10 R15 Snow

Parametry posouzení prvků:

Typ	Popis	L_y	Koef.	L_y,koef	Křivka Y	L_z	Koef.	L_z,koef	Křivka Z	L_ft	L_t	Tenkost.
Bottom	Bottom beam 6.0m	6	False	1	b	0.7	True	1	b	True	True	True
Strut	Strut 2 screw	1	True	1	b	0.7	True	1	b	True	False	True
Top	Top beam	1	True	1	b	0.7	True	1	b	True	True	True
Other	Simple member	1	True	0	auto	1	True	0	auto	False	False	False
Strut	Strut 1 screw	1	True	1	b	0.7	True	1	b	True	False	True
Other	Frame distributed	0.33	False	1	b	0.5	True	1	b	True	False	True
Top	Top beam SPH	0.33	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 1.5m	1.5	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 3.0m	3	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 4.5m	4.5	False	1	b	0.7	True	1	b	True	True	True

Materiál:

Prvek	Typ	Member type	Profil	Délka [m]	Materiál	Hmotnost [kg]	A [m²]	A_eff [m²]	Komb. N_Ed
4	Top	Top beam	H 112x120x5	1.422	HX500LAD	15.39	2090	2090	10
5	Top	Top beam	H 112x120x5	1.422	HX500LAD	15.39	2090	2090	10
12	Top	Top beam	H 112x120x5	0.077	HX500LAD	15.39	2090	2090	10
13	Top	Top beam	H 112x120x5	0.077	HX500LAD	15.39	2090	2090	10
16	Top	Top beam	H 112x120x5	1.521	HX500LAD	15.39	2090	2090	10
17	Top	Top beam	H 112x120x5	1.459	HX500LAD	15.39	2090	2090	10
24	Top	Top beam	H 112x120x5	0.082	HX500LAD	15.39	2090	2090	10
25	Top	Top beam	H 112x120x5	0.082	HX500LAD	15.39	2090	2090	10
28	Top	Top beam	H 112x120x5	1.47	HX500LAD	15.39	2090	2090	10
29	Top	Top beam	H 112x120x5	1.47	HX500LAD	15.39	2090	2090	10
36	Top	Top beam	H 112x120x5	0.105	HX500LAD	15.39	2090	2090	10
37	Top	Top beam	H 112x120x5	0.104	HX500LAD	15.39	2090	2090	10
40	Top	Top beam	H 112x120x5	1.478	HX500LAD	15.39	2090	2090	10
41	Top	Top beam	H 112x120x5	1.478	HX500LAD	15.39	2090	2090	10
48	Top	Top beam	H 112x120x5	0.083	HX500LAD	15.39	2090	2090	10
49	Top	Top beam	H 112x120x5	0.079	HX500LAD	15.39	2090	2090	10
52	Top	Top beam	H 112x120x5	1.445	HX500LAD	15.39	2090	2090	10
53	Top	Top beam	H 112x120x5	1.45	HX500LAD	15.39	2090	2090	40

Profily:

Prvek	Profil	N_c [%]	N_t [%]	NcMz [%]	NcMz2 [%]	V_y [%]	N_t [%]	N_b [%]	NyMy Mz [%]	NzMy Mz [%]	Profil [%]	Rozh. profil	Komb. N_Ed
4	H 112x120x5	21	0	21	20	0	13	66	27	26	66	Vzpěr	10
5	H 112x120x5	21	0	21	20	0	13	66	28	26	66	Vzpěr	10
12	H 112x120x5	21	0	24	17	1	1	22	25	25	25	Vzpěr za ohybu	10
13	H 112x120x5	21	0	25	17	1	1	22	25	25	25	Tlak a ohyb	10
16	H 112x120x5	21	0	24	18	0	13	70	32	30	70	Vzpěr	10
17	H 112x120x5	21	0	24	18	0	13	70	31	30	70	Vzpěr	10
24	H 112x120x5	21	0	24	18	0	1	22	25	25	25	Vzpěr za ohybu	10
25	H 112x120x5	22	0	25	18	5	1	22	25	25	25	Tlak a ohyb	10
28	H 112x120x5	20	0	25	13	0	13	65	32	31	65	Vzpěr	10
29	H 112x120x5	20	0	25	13	0	13	65	32	31	65	Vzpěr	10
36	H 112x120x5	18	0	24	12	8	1	19	24	24	24	Tlak a ohyb	10
37	H 112x120x5	18	0	24	12	8	1	19	24	24	24	Tlak a ohyb	10
40	H 112x120x5	13	0	22	1	1	13	42	28	27	42	Vzpěr	10
41	H 112x120x5	13	0	23	1	1	13	42	28	27	42	Vzpěr	10
48	H 112x120x5	9	0	14	4	23	1	9	14	14	23	Smyk	10
49	H 112x120x5	9	0	15	4	23	1	9	15	15	23	Smyk	10
52	H 112x120x5	0	4	9	1	1	0	0	0	0	9	Tah a ohyb	10
53	H 112x120x5	0	6	11	1	0	0	0	0	0	11	Tah a ohyb	40

Spoje:

Prvek	Profil	Spoj	N_t,Ed [kN]	N_c,Ed [kN]	F_v [%]	F_b,c [%]	F_b,t [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
4	H 112x120x5	HH06	9.66	65.95	96	59	9	1	0	96	Střih
5	H 112x120x5	HH06	9.66	65.96	96	59	9	1	0	96	Střih
12	H 112x120x5	X02	10.05	67.39	0	0	0	1	0	1	Osl.průřez
13	H 112x120x5	X02	10.04	67.39	0	0	0	1	0	1	Osl.průřez
16	H 112x120x5	X02	10.05	68.05	0	0	0	1	0	1	Osl.průřez
17	H 112x120x5	X02	10.03	67.92	0	0	0	1	0	1	Osl.průřez
24	H 112x120x5	X02	9.85	69.57	0	0	0	1	0	1	Osl.průřez
25	H 112x120x5	X02	9.83	69.41	0	0	0	1	0	1	Osl.průřez
28	H 112x120x5	X02	9.11	62.69	0	0	0	1	0	1	Osl.průřez
29	H 112x120x5	X02	8.28	62.65	0	0	0	1	0	1	Osl.průřez
36	H 112x120x5	X02	8.36	60.33	0	0	0	1	0	1	Osl.průřez
37	H 112x120x5	X02	7.05	60.34	0	0	0	1	0	1	Osl.průřez
40	H 112x120x5	X02	6.01	40.1	0	0	0	1	0	1	Osl.průřez
41	H 112x120x5	X02	2.52	40.1	0	0	0	0	0	0	
48	H 112x120x5	X02	3.99	28.33	0	0	0	0	0	0	
49	H 112x120x5	X02	-0.89	28.29	0	0	0	0	0	0	
52	H 112x120x5	HC06	-14.42	4.81	21	5	15	2	0	21	Střih
53	H 112x120x5	HC06	-19.56	-0.97	28	1	20	2	0	28	Střih

Souhrn:

Prvek	Member type	Profil	Materiál	Komb. N_Ed	Profil [%]	Spoj [%]	Vyhodnoceni	Posouzení
4	Top beam	H 112x120x5	HX500LAD	10	66	96	Střih	Vyhovuje
5	Top beam	H 112x120x5	HX500LAD	10	66	96	Střih	Vyhovuje
12	Top beam	H 112x120x5	HX500LAD	10	25	1	Vzpěr za ohybu	Vyhovuje
13	Top beam	H 112x120x5	HX500LAD	10	25	1	Tlak a ohyb	Vyhovuje
16	Top beam	H 112x120x5	HX500LAD	10	70	1	Vzpěr	Vyhovuje
17	Top beam	H 112x120x5	HX500LAD	10	70	1	Vzpěr	Vyhovuje
24	Top beam	H 112x120x5	HX500LAD	10	25	1	Vzpěr za ohybu	Vyhovuje
25	Top beam	H 112x120x5	HX500LAD	10	25	1	Tlak a ohyb	Vyhovuje
28	Top beam	H 112x120x5	HX500LAD	10	65	1	Vzpěr	Vyhovuje
29	Top beam	H 112x120x5	HX500LAD	10	65	1	Vzpěr	Vyhovuje
36	Top beam	H 112x120x5	HX500LAD	10	24	1	Tlak a ohyb	Vyhovuje
37	Top beam	H 112x120x5	HX500LAD	10	24	1	Tlak a ohyb	Vyhovuje
40	Top beam	H 112x120x5	HX500LAD	10	42	1	Vzpěr	Vyhovuje
41	Top beam	H 112x120x5	HX500LAD	10	42	0	Vzpěr	Vyhovuje
48	Top beam	H 112x120x5	HX500LAD	10	23	0	Smyk	Vyhovuje
49	Top beam	H 112x120x5	HX500LAD	10	23	0	Smyk	Vyhovuje
52	Top beam	H 112x120x5	HX500LAD	10	9	21	Střih	Vyhovuje
53	Top beam	H 112x120x5	HX500LAD	40	11	28	Střih	Vyhovuje

Vazník FR1 R15 dolní pás

Popis:
Poznámky:
Datum: 2022.01.18 @ 13:19:50
Importováno ze souboru: CZ1621 FR1 R15.rtd

Dolní pas
3 99 % Spoj: Střih 10 R15 Snow

Parametry posouzení prvků:

Typ	Popis	L_y	Koef.	L_y,koef	Křivka Y	L_z	Koef.	L_z,koef	Křivka Z	L_ft	L_t	Tenkost.
Bottom	Bottom beam 6.0m	6	False	1	b	0.7	True	1	b	True	True	True
Strut	Strut 2 screw	1	True	1	b	0.7	True	1	b	True	False	True
Top	Top beam	1	True	1	b	0.7	True	1	b	True	True	True
Other	Simple member	1	True	0	auto	1	True	0	auto	False	False	False
Strut	Strut 1 screw	1	True	1	b	0.7	True	1	b	True	False	True
Other	Frame distributed	0.33	False	1	b	0.5	True	1	b	True	False	True
Top	Top beam SPH	0.33	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 1.5m	1.5	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 3.0m	3	False	1	b	0.7	True	1	b	True	True	True
Bottom	Bottom beam 4.5m	4.5	False	1	b	0.7	True	1	b	True	True	True

Materiál:

Prvek	Typ	Member type	Profil	Délka [m]	Materiál	Hmotnost [kg]	A [m²]	A_eff [m²]	Komb. N_Ed
3	Bottom	Bottom beam 4.5m	H 112x120x5	1.872	HX500LAD	15.39	2090	2090	10
6	Bottom	Bottom beam 4.5m	H 112x120x5	0.096	HX500LAD	15.39	2090	2090	10
7	Bottom	Bottom beam 4.5m	H 112x120x5	0.1	HX500LAD	15.39	2090	2090	10
10	Bottom	Bottom beam 6.0m	H 112x120x5	1.16	HX500LAD	15.39	2090	2090	10
11	Bottom	Bottom beam 3.0m	H 112x120x5	1.155	HX500LAD	15.39	2090	2090	10
18	Bottom	Bottom beam 4.5m	H 112x120x5	0.062	HX500LAD	15.39	2090	2090	10
19	Bottom	Bottom beam 4.5m	H 112x120x5	0.061	HX500LAD	15.39	2090	2090	10
22	Bottom	Bottom beam 4.5m	H 112x120x5	1.44	HX500LAD	15.39	2090	2090	10
23	Bottom	Bottom beam 3.0m	H 112x120x5	1.436	HX500LAD	15.39	2090	2090	10
30	Bottom	Bottom beam 4.5m	H 112x120x5	0.108	HX500LAD	15.39	2090	2090	10
31	Bottom	Bottom beam 4.5m	H 112x120x5	0.111	HX500LAD	15.39	2090	2090	10
34	Bottom	Bottom beam 4.5m	H 112x120x5	1.426	HX500LAD	15.39	2090	2090	10
35	Bottom	Bottom beam 3.0m	H 112x120x5	1.419	HX500LAD	15.39	2090	2090	10
42	Bottom	Bottom beam 4.5m	H 112x120x5	0.105	HX500LAD	15.39	2090	2090	10
43	Bottom	Bottom beam 4.5m	H 112x120x5	0.106	HX500LAD	15.39	2090	2090	10
46	Bottom	Bottom beam 4.5m	H 112x120x5	1.365	HX500LAD	15.39	2090	2090	11
47	Bottom	Bottom beam 4.5m	H 112x120x5	1.365	HX500LAD	15.39	2090	2090	10
54	Bottom	Bottom beam 4.5m	H 112x120x5	0.1	HX500LAD	15.39	2090	2090	10
55	Bottom	Bottom beam 4.5m	H 112x120x5	0.103	HX500LAD	15.39	2090	2090	40
58	Bottom	Bottom beam 3.0m	H 112x120x5	0.606	HX500LAD	15.39	2090	2090	10

59	Bottom	Bottom beam 3.0m	H 112x120x5	0.647	HX500LAD	15.39	2090	2090	10

Profily:

Prvek	Profil	N_c [%]	N_t [%]	NcMz [%]	NcMz2 [%]	V_y [%]	N_t [%]	N_b [%]	NyMy Mz [%]	NzMy Mz [%]	Profil [%]	Rozh. profil	Komb. N_Ed
3	H 112x120x5	0	14	15	13	0	0	0	0	0	15	Tah a ohyb	10
6	H 112x120x5	0	18	23	13	6	0	0	0	0	23	Tah a ohyb	10
7	H 112x120x5	0	18	23	13	6	0	0	0	0	23	Tah a ohyb	10
10	H 112x120x5	0	18	23	13	0	0	0	0	0	23	Tah a ohyb	10
11	H 112x120x5	0	18	23	13	0	0	0	0	0	23	Tah a ohyb	10
18	H 112x120x5	0	18	20	16	0	0	0	0	0	20	Tah a ohyb	10
19	H 112x120x5	0	18	20	16	0	0	0	0	0	20	Tah a ohyb	10
22	H 112x120x5	0	18	23	13	0	0	0	0	0	23	Tah a ohyb	10
23	H 112x120x5	0	18	22	14	0	0	0	0	0	22	Tah a ohyb	10
30	H 112x120x5	0	16	21	12	7	0	0	0	0	21	Tah a ohyb	10
31	H 112x120x5	0	16	21	12	6	0	0	0	0	21	Tah a ohyb	10
34	H 112x120x5	0	15	23	6	1	0	0	0	0	23	Tah a ohyb	10
35	H 112x120x5	0	15	24	5	1	0	0	0	0	24	Tah a ohyb	10
42	H 112x120x5	0	9	18	0	16	0	0	0	0	18	Tah a ohyb	10
43	H 112x120x5	0	9	18	0	16	0	0	0	0	18	Tah a ohyb	10
46	H 112x120x5	0	5	11	2	1	0	0	0	0	11	Tah a ohyb	11
47	H 112x120x5	0	4	11	3	1	0	0	0	0	11	Tah a ohyb	10
54	H 112x120x5	8	0	25	11	27	40	31	47	25	47	Vzpěr za ohybu	10
55	H 112x120x5	9	0	23	6	21	40	36	49	23	49	Vzpěr za ohybu	40
58	H 112x120x5	24	0	41	5	3	27	60	69	44	69	Vzpěr za ohybu	10
59	H 112x120x5	24	0	41	5	3	27	60	69	44	69	Vzpěr za ohybu	10

Spoje:

Prvek	Profil	Spoj	N_t,Ed [kN]	N_c,Ed [kN]	F_v [%]	F_b,c [%]	F_b,t [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
3	H 112x120x5	HC04	-45.49	-9.59	99	13	64	5	0	99	Střih
6	H 112x120x5	X02	-57.86	-12.88	0	0	0	6	0	6	Osl.průřez
7	H 112x120x5	X02	-57.87	-10.73	0	0	0	6	0	6	Osl.průřez
10	H 112x120x5	X02	-58.48	-13.04	0	0	0	6	0	6	Osl.průřez
11	H 112x120x5	X02	-58.48	-10.6	0	0	0	6	0	6	Osl.průřez
18	H 112x120x5	X02	-58.94	-13.94	0	0	0	6	0	6	Osl.průřez
19	H 112x120x5	X02	-58.83	-9.92	0	0	0	6	0	6	Osl.průřez
22	H 112x120x5	X02	-58.87	-13.86	0	0	0	6	0	6	Osl.průřez
23	H 112x120x5	X02	-58.7	-9.38	0	0	0	6	0	6	Osl.průřez
30	H 112x120x5	X02	-52.59	-12.72	0	0	0	6	0	6	Osl.průřez
31	H 112x120x5	X02	-52.54	-7.44	0	0	0	6	0	6	Osl.průřez
34	H 112x120x5	X02	-47.71	-11.64	0	0	0	5	0	5	Osl.průřez
35	H 112x120x5	X02	-47.72	-5.77	0	0	0	5	0	5	Osl.průřez
42	H 112x120x5	X02	-29.14	-9.01	0	0	0	3	0	3	Osl.průřez
43	H 112x120x5	X02	-29.15	-1.16	0	0	0	3	0	3	Osl.průřez

46	H 112x120x5	X02	-14.98	-6.54	0	0	0	2	0	2	Osl.průřez
47	H 112x120x5	X02	-13.66	2.93	0	0	0	1	0	1	Osl.průřez
54	H 112x120x5	X02	-3.45	26.09	0	0	0	0	0	0	
55	H 112x120x5	X02	-1.37	29.93	0	0	0	0	0	0	
58	H 112x120x5		3.35	77.05	0	0	0	0	0	0	
59	H 112x120x5		5.61	77.03	0	0	0	0	0	0	

Souhrn:

Prvek	Member type	Profil	Materiál	Komb. N_Ed	Profil [%]	Spoj [%]	Vyhodnoceni	Posouzení
3	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	15	99	Střih	Vyhovuje
6	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	23	6	Tah a ohyb	Vyhovuje
7	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	23	6	Tah a ohyb	Vyhovuje
10	Bottom beam 6.0m	H 112x120x5	HX500LAD	10	23	6	Tah a ohyb	Vyhovuje
11	Bottom beam 3.0m	H 112x120x5	HX500LAD	10	23	6	Tah a ohyb	Vyhovuje
18	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	20	6	Tah a ohyb	Vyhovuje
19	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	20	6	Tah a ohyb	Vyhovuje
22	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	23	6	Tah a ohyb	Vyhovuje
23	Bottom beam 3.0m	H 112x120x5	HX500LAD	10	22	6	Tah a ohyb	Vyhovuje
30	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	21	6	Tah a ohyb	Vyhovuje
31	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	21	6	Tah a ohyb	Vyhovuje
34	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	23	5	Tah a ohyb	Vyhovuje
35	Bottom beam 3.0m	H 112x120x5	HX500LAD	10	24	5	Tah a ohyb	Vyhovuje
42	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	18	3	Tah a ohyb	Vyhovuje
43	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	18	3	Tah a ohyb	Vyhovuje
46	Bottom beam 4.5m	H 112x120x5	HX500LAD	11	11	2	Tah a ohyb	Vyhovuje
47	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	11	1	Tah a ohyb	Vyhovuje
54	Bottom beam 4.5m	H 112x120x5	HX500LAD	10	47	0	Vzpěr za ohybu	Vyhovuje
55	Bottom beam 4.5m	H 112x120x5	HX500LAD	40	49	0	Vzpěr za ohybu	Vyhovuje
58	Bottom beam 3.0m	H 112x120x5	HX500LAD	10	69	0	Vzpěr za ohybu	Vyhovuje
59	Bottom beam 3.0m	H 112x120x5	HX500LAD	10	69	0	Vzpěr za ohybu	Vyhovuje

Zatížení při požáru

Popis:
Poznámky:
Datum: 2022.01.18 @ 09:13:54

STANOVENÍ NORMOVÉ TEPLOTNÍ KŘIVKY A TEPLOTY OCELI DLE ČSN EN 1991-1-2

Zadání:

Profile =	C 100x53x3	
Výška profilu h =	100 mm	
Šířka profilu b =	53 mm	
Objem prvku na jednotku délky V =	662 mm²	
Šířka ohřívané pásnice b_fl =	0 mm	
Plocha povrchu prvku na jednotku délky A_m =	0.446 m²/m	
Čas vystavení účinkům požáru t =	15 min	
Časový úsek Δ t =	5 s	ČSN EN 1993-1-2 odst. 4.2.5.1 (4)
Uvažovat vliv pozinkování =	True	
Emisivita prvku ε_m =	0.35	
Emisivita požáru ε_f =	1.00	
Polohový faktor φ =	1.00	
Součinitel přestupu tepla α_c =	25.00 W/m²K	ČSN EN 1991-1-2 odst. 3.2.1 (2)
Stefan-Boltzmanova konstanta δ =	5.67E-08 W/m²K^4	ČSN EN 1991-1-2 odst. 3.1 (6)
Objemová hmotnost ρ =	7850 kg/m³	ČSN EN 1993-1-2 odst. 3.2.2 (1)
Uvažovat ohřev jen 3 stran =	False	

Výpočet:

$$Am/V = \frac{A_m}{V} = \frac{0.446}{662 \cdot 0.000001} = 673.716 m^{-1}$$

$$Uvažovat\ ohřev\ jen\ 3\ stran = False => A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l$$

$$A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l = (2 \cdot 53 \cdot 0.001 + 2 \cdot 100 \cdot 0.001) \cdot 1 = 0.306 m^2/m$$

$$(A_m/V)_b = \frac{A_{m,b}}{V} = \frac{0.306}{662 \cdot 0.000001} = 462.236 m^{-1}$$

$$Uvažovat\ vliv\ zastínění = True => k_{sh} = \frac{(A_m/V)_b}{Am/V}$$

$$k_{sh} = \frac{(A_m/V)_b}{Am/V} = \frac{462.236}{673.716} = 0.686$$

$$\Theta_g = 20 + 345 \cdot \log_{10}(8 \cdot t + 1) = 20 + 345 \cdot \log_{10}(8 \cdot 15 + 1) = 738.56\,°C$$

$$\Theta_a, t = k_{s,h} \cdot \left(\frac{Am/V}{c_a \cdot \rho}\right) \cdot h_{net} \cdot \Delta t = 0.686 \cdot \left(\frac{673.716}{1045.28 \cdot 7850}\right) \cdot 3589.28 \cdot 5 = 1.03\,°C$$

$$\Theta_c = \Theta_c + \Theta_a, t = 703.719 + 1.025 = 703.72\,°C$$

$$c_a = 666 + \left(\frac{13002}{738 - o_c}\right) = 666 + \left(\frac{13002}{738 - 703.72}\right) = 1045.28 J/kgK$$

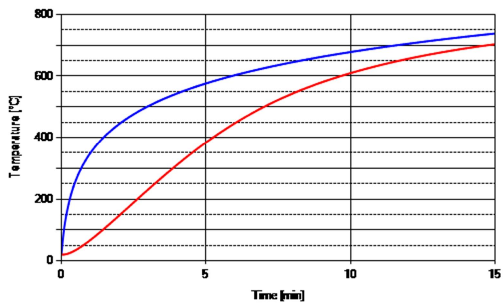
$$h_{net} = h_{net,c} + h_{net,r}$$

$$h_{net,c} = \alpha_c \cdot (\Theta_g - \Theta_c) = 25.00 \cdot (738.56 - 703.72) = 871.04 W/m^2$$

$$h_{net,r} = \varphi \cdot e_m \cdot e_f \cdot \delta \cdot \left((\Theta_g + 273)^4 - (\Theta_c + 273)^4\right)$$
$$= 1.00 \cdot 0.35 \cdot 1.00 \cdot (5.67E-08) \cdot ((738.56 + 273)^4 - (703.72 + 273)^4) = 2718.24 W/m^2$$

$$h_{net} = h_{net,c} + h_{net,r} = 871.04 + 2718.24 = 3589.28 W/m^2$$

Součinitel průřezu nechráněných ocelových prvků A_m/V =	673.716 1/m	
Povrch obdélníka opsaného průřezu A_m,b =	0.306 m²/m	
Součinitel průřezu obdélníka opsaného průřezu (A_m/V)_b =	462.236 1/m	
Uvažovat vliv zastínění =	True	
Opravný součinitel zastínění k_sh =	0.686	ČSN EN 1993-1-2 odst. 4.2.5.1 (2), výraz 4.26b
Normová teplotní křivka Θ_g =	738.56 °C	ČSN EN 1991-1-2 odst. 3.2.1 (1), výraz 3.4
Přírůstek tep. oceli Θ_a,t =	1.03 °C	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Teplota oceli Θ_c =	703.72 °C	
Měrné teplo oceli c_a =	1045.28 J/kgK	ČSN EN 1993-1-2 odst. 3.4.1.2 (1), výraz 3.2
Čistý tepelný tok h_net =	3589.28 W/m²	ČSN EN 1991-1-2 odst. 3.1 (2), výraz 3.1



REDUKČNÍ SOUČINITELÉ OCELI A SPOJŮ PŘI ZVÝŠENÝCH TEPLOTÁCH DLE ČSN EN 1993-1-2

Redukční součinitel	Redukční součinitel
Součinitelé pro průřezy 1-3. třídy	
k_yQ	0.225
k_PQ	0.074
k_EQ	0.128
Součinitelé pro průřezy 4. třídy	
k_PQ	0.128
k_EQ	0.128
Součinitelé pro spoje	
k_bQ	0.099
k_wQ	0.128

Teplota:

Teplota při požáru T =	704 °C	ČSN EN 1993-1-2

Materiál:

Ocel =	S350GD	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez kluzu oceli f_y =	350 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez pevnosti oceli f_u =	420 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Modul pružnosti oceli E =	210 GPa	
Třída průřezu =	4	
Redukovaná mez kluzu oceli f_y,red =	44.800 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaná mez pevnosti oceli f_u,red =	53.760 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaný modul pružnosti oceli E_red =	26.880 GPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25

Zatížení při požáru

Popis:
Poznámky:
Datum: 2022.01.18 @ 13:30:43

STANOVENÍ NORMOVÉ TEPLOTNÍ KŘIVKY A TEPLOTY OCELI DLE ČSN EN 1991-1-2

Zadání:

Profile =	C 100x48x2	
Výška profilu h =	100 mm	
Šířka profilu b =	48 mm	
Objem prvku na jednotku délky V =	423 mm²	
Šířka ohřívané pásnice b_fl =	0 mm	
Plocha povrchu prvku na jednotku délky A_m =	0.434 m²/m	
Čas vystavení účinkům požáru t =	15 min	
Časový úsek Δ t =	5 s	ČSN EN 1993-1-2 odst. 4.2.5.1 (4)
Uvažovat vliv pozinkování =	True	
Emisivita prvku ε_m =	0.35	

Emisivita požáru ε_f =	1.00	
Polohový faktor φ =	1.00	
Součinitel přestupu tepla α_c =	25.00 W/m²K	ČSN EN 1991-1-2 odst. 3.2.1 (2)
Stefan-Boltzmanova konstanta δ =	5.67E-08 W/m²K^4	ČSN EN 1991-1-2 odst. 3.1 (6)
Objemová hmotnost ρ =	7850 kg/m³	ČSN EN 1993-1-2 odst. 3.2.2 (1)
Uvažovat ohřev jen 3 stran =	False	

Výpočet:

$$Am/V = \frac{A_m}{V} = \frac{0.434}{423 \cdot 0.000001} = 1026.005 m^{-1}$$

$$Uvažovat\ ohřev\ jen\ 3\ stran = False => A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l$$

$$A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l = (2 \cdot 48 \cdot 0.001 + 2 \cdot 100 \cdot 0.001) \cdot 1 = 0.296 m^2/m$$

$$(A_m/V)_b = \frac{A_{m,b}}{V} = \frac{0.296}{423 \cdot 0.000001} = 699.764 m^{-1}$$

$$Uvažovat\ vliv\ zastínění = True => k_{sh} = \frac{(A_m/V)_b}{Am/V}$$

$$k_{sh} = \frac{(A_m/V)_b}{Am/V} = \frac{699.764}{1026.005} = 0.682$$

$$\Theta_g = 20 + 345 \cdot \log_{10}(8 \cdot t + 1) = 20 + 345 \cdot \log_{10}(8 \cdot 15 + 1) = 738.56\,°C$$

$$\Theta_a, t = k_{s,h} \cdot \left(\frac{Am/V}{c_a \cdot \rho}\right) \cdot h_{net} \cdot \Delta t = 0.682 \cdot \left(\frac{1026.005}{1271.99 \cdot 7850}\right) \cdot 2301.23 \cdot 5 = 0.82\,°C$$

$$\Theta_c = \Theta_c + \Theta_a, t = 716.544 + 0.819 = 716.54\,°C$$

$$c_a = 666 + \left(\frac{13002}{738 - o_c}\right) = 666 + \left(\frac{13002}{738 - 716.54}\right) = 1271.99 J/kgK$$

$$h_{net} = h_{net,c} + h_{net,r}$$

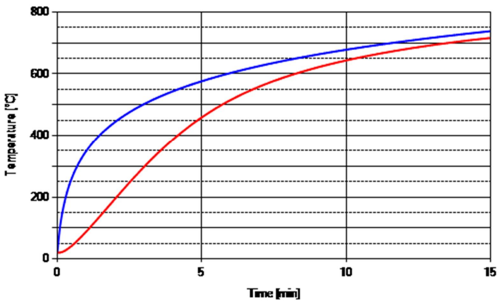
$$h_{net,c} = \alpha_c \cdot (\Theta_g - \Theta_c) = 25.00 \cdot (738.56 - 716.54) = 550.42 W/m^2$$

$$h_{net,r} = \varphi \cdot e_m \cdot e_f \cdot \delta \cdot \left((\Theta_g + 273)^4 - (\Theta_c + 273)^4\right)$$
$$= 1.00 \cdot 0.35 \cdot 1.00 \cdot (5.67E-08) \cdot ((738.56 + 273)^4 - (716.54 + 273)^4) = 1750.81 W/m^2$$

$$h_{net} = h_{net,c} + h_{net,r} = 550.42 + 1750.81 = 2301.23 W/m^2$$

Součinitel průřezu nechráněných ocelových prvků A_m/V =	1026.005 1/m	
Povrch obdélníka opsaného průřezu A_m,b =	0.296 m²/m	
Součinitel průřezu obdélníka opsaného průřezu (A_m/V)_b =	699.764 1/m	
Uvažovat vliv zastínění =	True	
Opravný součinitel zastínění k_sh =	0.682	ČSN EN 1993-1-2 odst. 4.2.5.1 (2), výraz 4.26b

Normová teplotní křivka Θ_g =	738.56 °C	ČSN EN 1991-1-2 odst. 3.2.1 (1), výraz 3.4
Přírůstek tep. oceli Θ_a,t =	0.82 °C	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Teplota oceli Θ_c =	716.54 °C	
Měrné teplo oceli c_a =	1271.99 J/kgK	ČSN EN 1993-1-2 odst. 3.4.1.2 (1), výraz 3.2
Čistý tepelný tok h_net =	2301.23 W/m²	ČSN EN 1991-1-2 odst. 3.1 (2), výraz 3.1



REDUKČNÍ SOUČINITELÉ OCELI A SPOJŮ PŘI ZVÝŠENÝCH TEPLOTÁCH DLE ČSN EN 1993-1-2

Redukční součinitel	Redukční součinitel
Součinitelé pro průřezy 1-3. třídy	
k_yQ	0.21
k_PQ	0.071
k_EQ	0.123
Součinitelé pro průřezy 4. třídy	
k_PQ	0.12
k_EQ	0.123
Součinitelé pro spoje	
k_bQ	0.094
k_wQ	0.12

Teplota:

Teplota při požáru T =	717 °C	ČSN EN 1993-1-2

Materiál:

Ocel =	S350GD	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez kluzu oceli f_y =	350 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez pevnosti oceli f_u =	420 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Modul pružnosti oceli E =	210 GPa	
Třída průřezu =	4	
Redukovaná mez kluzu oceli f_y,red =	42.000 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaná mez pevnosti oceli f_u,red =	50.400 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaný modul pružnosti oceli E_red =	25.830 GPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25

Zatížení při požáru

Popis:
Poznámky:
Datum: 2022.01.18 @ 13:30:43

STANOVENÍ NORMOVÉ TEPLOTNÍ KŘIVKY A TEPLOTY OCELI DLE ČSN EN 1991-1-2

Zadání:

Profile =	C 100x75x2	
Výška profilu h =	100 mm	
Šířka profilu b =	75 mm	
Objem prvku na jednotku délky V =	531 mm²	
Šířka ohřívané pásnice b_fl =	0 mm	
Plocha povrchu prvku na jednotku délky A_m =	0.542 m²/m	
Čas vystavení účinkům požáru t =	15 min	
Časový úsek Δ t =	5 s	ČSN EN 1993-1-2 odst. 4.2.5.1 (4)
Uvažovat vliv pozinkování =	True	
Emisivita prvku ε_m =	0.35	

Emisivita požáru ε_f =	1.00	
Polohový faktor φ =	1.00	
Součinitel přestupu tepla α_c =	25.00 W/m²K	ČSN EN 1991-1-2 odst. 3.2.1 (2)
Stefan-Boltzmanova konstanta δ =	5.67E-08 W/m²K^4	ČSN EN 1991-1-2 odst. 3.1 (6)
Objemová hmotnost ρ =	7850 kg/m³	ČSN EN 1993-1-2 odst. 3.2.2 (1)
Uvažovat ohřev jen 3 stran =	False	

Výpočet:

$$Am/V = \frac{A_m}{V} = \frac{0.542}{531 \cdot 0.000001} = 1020.716 m^{-1}$$

$$Uvažovat\ ohřev\ jen\ 3\ stran = False => A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l$$

$$A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l = (2 \cdot 75 \cdot 0.001 + 2 \cdot 100 \cdot 0.001) \cdot 1 = 0.350 m^2/m$$

$$(A_m/V)_b = \frac{A_{m,b}}{V} = \frac{0.350}{531 \cdot 0.000001} = 659.134 m^{-1}$$

$$Uvažovat\ vliv\ zastínění = True => k_{sh} = \frac{(A_m/V)_b}{Am/V}$$

$$k_{sh} = \frac{(A_m/V)_b}{Am/V} = \frac{659.134}{1020.716} = 0.646$$

$$\Theta_g = 20 + 345 \cdot \log_{10}(8 \cdot t + 1) = 20 + 345 \cdot \log_{10}(8 \cdot 15 + 1) = 738.56\,°C$$

$$\Theta_a, t = k_{s,h} \cdot \left(\frac{Am/V}{c_a \cdot \rho}\right) \cdot h_{net} \cdot \Delta t = 0.646 \cdot \left(\frac{1020.716}{1237.39 \cdot 7850}\right) \cdot 2433.46 \cdot 5 = 0.84\,°C$$

$$\Theta_c = \Theta_c + \Theta_a, t = 715.245 + 0.838 = 715.24\,°C$$

$$c_a = 666 + \left(\frac{13002}{738 - \theta_c}\right) = 666 + \left(\frac{13002}{738 - 715.24}\right) = 1237.39 J/kgK$$

$$h_{net} = h_{net,c} + h_{net,r}$$

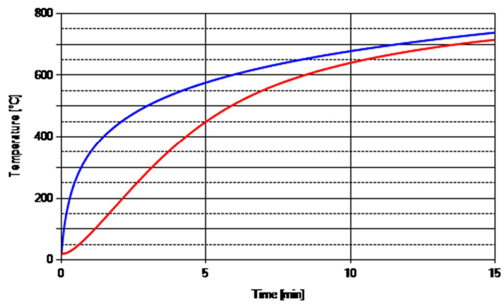
$$h_{net,c} = \alpha_c \cdot (\theta_g - \theta_c) = 25.00 \cdot (738.56 - 715.24) = 582.90 W/m^2$$

$$h_{net,r} = \varphi \cdot e_m \cdot e_f \cdot \delta \cdot \left((\theta_g + 273)^4 - (\theta_c + 273)^4\right)$$
$$= 1.00 \cdot 0.35 \cdot 1.00 \cdot (5.67E-08) \cdot ((738.56 + 273)^4 - (715.24 + 273)^4) = 1850.55 W/m^2$$

$$h_{net} = h_{net,c} + h_{net,r} = 582.90 + 1850.55 = 2433.46 W/m^2$$

Součinitel průřezu nechráněných ocelových prvků A_m/V =	1020.716 1/m	
Povrch obdélníka opsaného průřezu A_m,b =	0.350 m²/m	
Součinitel průřezu obdélníka opsaného průřezu (A_m/V)_b =	659.134 1/m	
Uvažovat vliv zastínění =	True	
Opravný součinitel zastinění k_sh =	0.646	ČSN EN 1993-1-2 odst. 4.2.5.1 (2), výraz 4.26b

Normová teplotní křivka Θ_g =	738.56 °C	ČSN EN 1991-1-2 odst. 3.2.1 (1), výraz 3.4
Přírůstek tep. oceli Θ_a,t =	0.84 °C	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Teplota oceli Θ_c =	715.24 °C	
Měrné teplo oceli c_a =	1237.39 J/kgK	ČSN EN 1993-1-2 odst. 3.4.1.2 (1), výraz 3.2
Čistý tepelný tok h_net =	2433.46 W/m²	ČSN EN 1991-1-2 odst. 3.1 (2), výraz 3.1



REDUKČNÍ SOUČINITELÉ OCELI A SPOJŮ PŘI ZVÝŠENÝCH TEPLOTÁCH DLE ČSN EN 1993-1-2

Redukční součinitel	Redukční součinitel
Součinitelé pro průřezy 1-3. třídy	
k_yQ	0.212
k_PQ	0.071
k_EQ	0.124
Součinitelé pro průřezy 4. třídy	
k_PQ	0.121
k_EQ	0.124
Součinitelé pro spoje	
k_bQ	0.095
k_wQ	0.122

Teplota:

Teplota při požáru T =	715 °C	ČSN EN 1993-1-2

Materiál:

Ocel =	S350GD	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez kluzu oceli f_y =	350 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez pevnosti oceli f_u =	420 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Modul pružnosti oceli E =	210 GPa	
Třída průřezu =	4	
Redukovaná mez kluzu oceli f_y,red =	42.350 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaná mez pevnosti oceli f_u,red =	50.820 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaný modul pružnosti oceli E_red =	26.040 GPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25

Zatížení při požáru

Popis:
Poznámky:
Datum: 2022.01.18 @ 13:30:43

STANOVENÍ NORMOVÉ TEPLOTNÍ KŘIVKY A TEPLOTY OCELI DLE ČSN EN 1991-1-2

Zadání:

Profile =	C 100x77x3	
Výška profilu h =	100 mm	
Šířka profilu b =	77 mm	
Objem prvku na jednotku délky V =	806 mm²	
Šířka ohřívané pásnice b_fl =	0 mm	
Plocha povrchu prvku na jednotku délky A_m =	0.542 m²/m	
Čas vystavení účinkům požáru t =	15 min	
Časový úsek Δ t =	5 s	ČSN EN 1993-1-2 odst. 4.2.5.1 (4)
Uvažovat vliv pozinkování =	True	
Emisivita prvku ε_m =	0.35	

Emisivita požáru ε_f =	1.00	
Polohový faktor φ =	1.00	
Součinitel přestupu tepla α_c =	25.00 W/m²K	ČSN EN 1991-1-2 odst. 3.2.1 (2)
Stefan-Boltzmanova konstanta δ =	5.67E-08 W/m²K^4	ČSN EN 1991-1-2 odst. 3.1 (6)
Objemová hmotnost ρ =	7850 kg/m³	ČSN EN 1993-1-2 odst. 3.2.2 (1)
Uvažovat ohřev jen 3 stran =	False	

Výpočet:

$$Am/V = \frac{A_m}{V} = \frac{0.542}{806 \cdot 0.000001} = 672.457 m^{-1}$$

$$Uvažovat\ ohřev\ jen\ 3\ stran = False => A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l$$

$$A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l = (2 \cdot 77 \cdot 0.001 + 2 \cdot 100 \cdot 0.001) \cdot 1 = 0.354 m^2/m$$

$$(A_m/V)_b = \frac{A_{m,b}}{V} = \frac{0.354}{806 \cdot 0.000001} = 439.206 m^{-1}$$

$$Uvažovat\ vliv\ zastínění = True => k_{sh} = \frac{(A_m/V)_b}{Am/V}$$

$$k_{sh} = \frac{(A_m/V)_b}{Am/V} = \frac{439.206}{672.457} = 0.653$$

$$\Theta_g = 20 + 345 \cdot \log_{10}(8 \cdot t + 1) = 20 + 345 \cdot \log_{10}(8 \cdot 15 + 1) = 738.56\,°C$$

$$\Theta_a, t = k_{s,h} \cdot \left(\frac{Am/V}{c_a \cdot \rho}\right) \cdot h_{net} \cdot \Delta t = 0.653 \cdot \left(\frac{672.457}{1019.89 \cdot 7850}\right) \cdot 3831.99 \cdot 5 = 1.07\,°C$$

$$\Theta_c = \Theta_c + \Theta_a, t = 701.260 + 1.066 = 701.26\,°C$$

$$c_a = 666 + \left(\frac{13002}{738 - o_c}\right) = 666 + \left(\frac{13002}{738 - 701.26}\right) = 1019.89 J/kgK$$

$$h_{net} = h_{net,c} + h_{net,r}$$

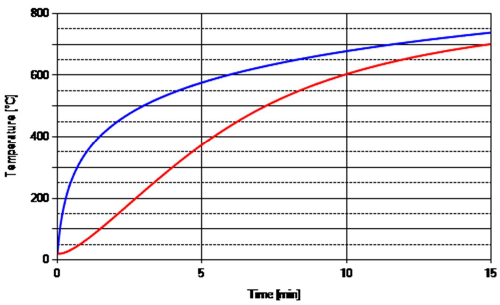
$$h_{net,c} = \alpha_c \cdot (\Theta_g - \Theta_c) = 25.00 \cdot (738.56 - 701.26) = 932.53 W/m^2$$

$$h_{net,r} = \varphi \cdot e_m \cdot e_f \cdot \delta \cdot \left((\Theta_g + 273)^4 - (\Theta_c + 273)^4\right)$$
$$= 1.00 \cdot 0.35 \cdot 1.00 \cdot (5.67E-08) \cdot ((738.56 + 273)^4 - (701.26 + 273)^4) = 2899.46 W/m^2$$

$$h_{net} = h_{net,c} + h_{net,r} = 932.53 + 2899.46 = 3831.99 W/m^2$$

Součinitel průřezu nechráněných ocelových prvků A_m/V =	672.457 1/m	
Povrch obdélníka opsaného průřezu A_m,b =	0.354 m²/m	
Součinitel průřezu obdélníka opsaného průřezu (A_m/V)_b =	439.206 1/m	
Uvažovat vliv zastínění =	True	
Opravný součinitel zastinění k_sh =	0.653	ČSN EN 1993-1-2 odst. 4.2.5.1 (2), výraz 4.26b

Normová teplotní křivka Θ_g =	738.56 °C	ČSN EN 1991-1-2 odst. 3.2.1 (1), výraz 3.4
Přírůstek tep. oceli Θ_a,t =	1.07 °C	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Teplota oceli Θ_c =	701.26 °C	
Měrné teplo oceli c_a =	1019.89 J/kgK	ČSN EN 1993-1-2 odst. 3.4.1.2 (1), výraz 3.2
Čistý tepelný tok h_net =	3831.99 W/m²	ČSN EN 1991-1-2 odst. 3.1 (2), výraz 3.1



REDUKČNÍ SOUČINITELÉ OCELI A SPOJŮ PŘI ZVÝŠENÝCH TEPLOTÁCH DLE ČSN EN 1993-1-2

Redukční součinitel	Redukční součinitel
Součinitelé pro průřezy 1-3. třídy	
k_yQ	0.229
k_PQ	0.075
k_EQ	0.13
Součinitelé pro průřezy 4. třídy	
k_PQ	0.129
k_EQ	0.13
Součinitelé pro spoje	
k_bQ	0.1
k_wQ	0.129

Teplota:

Teplota při požáru T =	701 °C	ČSN EN 1993-1-2

Materiál:

Ocel =	S350GD	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez kluzu oceli f_y =	350 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez pevnosti oceli f_u =	420 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Modul pružnosti oceli E =	210 GPa	
Třída průřezu =	4	
Redukovaná mez kluzu oceli f_y,red =	45.150 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaná mez pevnosti oceli f_u,red =	54.180 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaný modul pružnosti oceli E_red =	27.300 GPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25

Zatížení při požáru

Popis:
Poznámky:
Datum: 2022.01.18 @ 13:30:44

STANOVENÍ NORMOVÉ TEPLOTNÍ KŘIVKY A TEPLOTY OCELI DLE ČSN EN 1991-1-2

Zadání:

Profile =	C 100x82x4	
Výška profilu h =	100 mm	
Šířka profilu b =	82 mm	
Objem prvku na jednotku délky V =	1167 mm²	
Šířka ohřívané pásnice b_fl =	0 mm	
Plocha povrchu prvku na jednotku délky A_m =	0.588 m²/m	
Čas vystavení účinkům požáru t =	15 min	
Časový úsek Δ t =	5 s	ČSN EN 1993-1-2 odst. 4.2.5.1 (4)
Uvažovat vliv pozinkování =	True	
Emisivita prvku ε_m =	0.35	

Emisivita požáru ε_f = 1.00
Polohový faktor φ = 1.00
Součinitel přestupu tepla α_c = 25.00 W/m²K — ČSN EN 1991-1-2 odst. 3.2.1 (2)
Stefan-Boltzmanova konstanta δ = 5.67E-08 W/m²K^4 — ČSN EN 1991-1-2 odst. 3.1 (6)
Objemová hmotnost ρ = 7850 kg/m³ — ČSN EN 1993-1-2 odst. 3.2.2 (1)
Uvažovat ohřev jen 3 stran = False

Výpočet:

$$Am/V = \frac{A_m}{V} = \frac{0.588}{1167 \cdot 0.000001} = 503.856 m^{-1}$$

$$Uvažovat\ ohřev\ jen\ 3\ stran = False => A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l$$

$$A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l = (2 \cdot 82 \cdot 0.001 + 2 \cdot 100 \cdot 0.001) \cdot 1 = 0.364 m^2/m$$

$$(A_m/V)_b = \frac{A_{m,b}}{V} = \frac{0.364}{1167 \cdot 0.000001} = 311.911 m^{-1}$$

$$Uvažovat\ vliv\ zastínění = True => k_{sh} = \frac{(A_m/V)_b}{Am/V}$$

$$k_{sh} = \frac{(A_m/V)_b}{Am/V} = \frac{311.911}{503.856} = 0.619$$

$$\theta_g = 20 + 345 \cdot \log_{10}(8 \cdot t + 1) = 20 + 345 \cdot \log_{10}(8 \cdot 15 + 1) = 738.56\,°C$$

$$\theta_a, t = k_{s,h} \cdot \left(\frac{Am/V}{c_a \cdot \rho}\right) \cdot h_{net} \cdot \Delta t = 0.619 \cdot \left(\frac{503.856}{875.48 \cdot 7850}\right) \cdot 6253.14 \cdot 5 = 1.44\,°C$$

$$\theta_c = \theta_c + \theta_a, t = 675.932 + 1.437 = 675.93\,°C$$

$$c_a = 666 + \left(\frac{13002}{738 - o_c}\right) = 666 + \left(\frac{13002}{738 - 675.93}\right) = 875.48 J/kgK$$

$$h_{net} = h_{net,c} + h_{net,r}$$

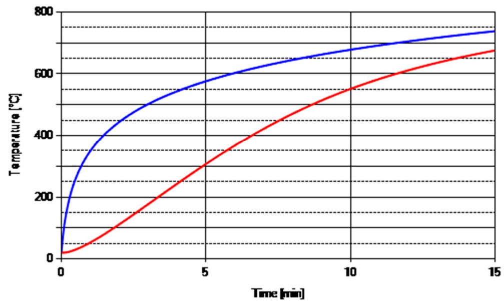
$$h_{net,c} = \alpha_c \cdot (\theta_g - \theta_c) = 25.00 \cdot (738.56 - 675.93) = 1565.72 W/m^2$$

$$h_{net,r} = \varphi \cdot e_m \cdot e_f \cdot \delta \cdot \left((\theta_g + 273)^4 - (\theta_c + 273)^4\right)$$
$$= 1.00 \cdot 0.35 \cdot 1.00 \cdot (5.67E-08) \cdot ((738.56 + 273)^4 - (675.93 + 273)^4) = 4687.42 W/m^2$$

$$h_{net} = h_{net,c} + h_{net,r} = 1565.72 + 4687.42 = 6253.14 W/m^2$$

Součinitel průřezu nechráněných ocelových prvků A_m/V = 503.856 1/m
Povrch obdélníka opsaného průřezu A_m,b = 0.364 m²/m
Součinitel průřezu obdélníka opsaného průřezu (A_m/V)_b = 311.911 1/m
Uvažovat vliv zastínění = True
Opravný součinitel zastinění k_sh = 0.619 — ČSN EN 1993-1-2 odst. 4.2.5.1 (2), výraz 4.26b

Normová teplotní křivka Θ_g = 738.56 °C — ČSN EN 1991-1-2 odst. 3.2.1 (1), výraz 3.4
Přírůstek tep. oceli Θ_a,t = 1.44 °C — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Teplota oceli Θ_c = 675.93 °C
Měrné teplo oceli c_a = 875.48 J/kgK — ČSN EN 1993-1-2 odst. 3.4.1.2 (1), výraz 3.2
Čistý tepelný tok h_net = 6253.14 W/m² — ČSN EN 1991-1-2 odst. 3.1 (2), výraz 3.1



REDUKČNÍ SOUČINITELÉ OCELI A SPOJŮ PŘI ZVÝŠENÝCH TEPLOTÁCH DLE ČSN EN 1993-1-2

Redukční součinitel	Redukční součinitel
Součinitelé pro průřezy 1-3. třídy	
k_yQ	0.288
k_PQ	0.1
k_EQ	0.173
Součinitelé pro průřezy 4. třídy	
k_PQ	0.171
k_EQ	0.173
Součinitelé pro spoje	
k_bQ	0.129
k_wQ	0.19

Teplota:

Teplota při požáru T = 676 °C — ČSN EN 1993-1-2

Materiál:

Ocel = HX420LAD — ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez kluzu oceli f_y = 420 MPa — ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez pevnosti oceli f_u = 480 MPa — ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Modul pružnosti oceli E = 210 GPa
Třída průřezu = 4
Redukovaná mez kluzu oceli f_y,red = 71.820 MPa — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaná mez pevnosti oceli f_u,red = 82.080 MPa — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaný modul pružnosti oceli E_red = 36.330 GPa — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25

Zatížení při požáru

Popis:
Poznámky:
Datum: 2022.01.18 @ 13:30:44

STANOVENÍ NORMOVÉ TEPLOTNÍ KŘIVKY A TEPLOTY OCELI DLE ČSN EN 1991-1-2

Zadání:

Profile = C 100x85x5
Výška profilu h = 100 mm
Šířka profilu b = 85 mm
Objem prvku na jednotku délky V = 1515 mm²
Šířka ohřívané pásnice b_fl = 0 mm
Plocha povrchu prvku na jednotku délky A_m = 0.610 m²/m

Čas vystavení účinkům požáru t = 15 min
Časový úsek Δ t = 5 s — ČSN EN 1993-1-2 odst. 4.2.5.1 (4)
Uvažovat vliv pozinkování = True
Emisivita prvku ε_m = 0.35

Emisivita požáru ε_f = 1.00
Polohový faktor φ = 1.00
Součinitel přestupu tepla α_c = 25.00 W/m²K — ČSN EN 1991-1-2 odst. 3.2.1 (2)
Stefan-Boltzmanova konstanta δ = 5.67E-08 W/m²K^4 — ČSN EN 1991-1-2 odst. 3.1 (6)
Objemová hmotnost ρ = 7850 kg/m³ — ČSN EN 1993-1-2 odst. 3.2.2 (1)
Uvažovat ohřev jen 3 stran = False

Výpočet:

$$Am/V = \frac{A_m}{V} = \frac{0.610}{1515 \cdot 0.000001} = 402.640 m^{-1}$$

$$Uvažovat\ ohřev\ jen\ 3\ stran = False => A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l$$

$$A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l = (2 \cdot 85 \cdot 0.001 + 2 \cdot 100 \cdot 0.001) \cdot 1 = 0.370 m^2/m$$

$$(A_m/V)_b = \frac{A_{m,b}}{V} = \frac{0.370}{1515 \cdot 0.000001} = 244.224 m^{-1}$$

$$Uvažovat\ vliv\ zastínění = True => k_{sh} = \frac{(A_m/V)_b}{Am/V}$$

$$k_{sh} = \frac{(A_m/V)_b}{Am/V} = \frac{244.224}{402.640} = 0.607$$

$$\theta_g = 20 + 345 \cdot \log_{10}(8 \cdot t + 1) = 20 + 345 \cdot \log_{10}(8 \cdot 15 + 1) = 738.56\,°C$$

$$\theta_a, t = k_{s,h} \cdot \left(\frac{Am/V}{c_a \cdot \rho}\right) \cdot h_{net} \cdot \Delta t = 0.607 \cdot \left(\frac{402.640}{806.09 \cdot 7850}\right) \cdot 9008.12 \cdot 5 = 1.76\,°C$$

$$\theta_c = \theta_c + \theta_a, t = 645.186 + 1.756 = 645.19\,°C$$

$$c_a = 666 + \left(\frac{13002}{738 - o_c}\right) = 666 + \left(\frac{13002}{738 - 645.19}\right) = 806.09 J/kgK$$

$$h_{net} = h_{net,c} + h_{net,r}$$

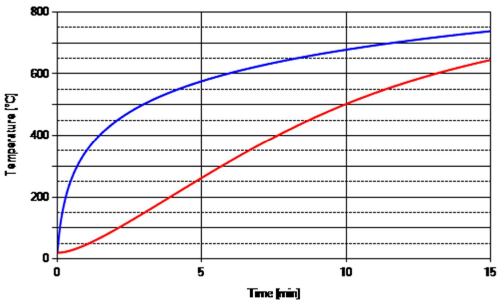
$$h_{net,c} = \alpha_c \cdot (\theta_g - \theta_c) = 25.00 \cdot (738.56 - 645.19) = 2334.38 W/m^2$$

$$h_{net,r} = \varphi \cdot e_m \cdot e_f \cdot \delta \cdot \left((\theta_g + 273)^4 - (\theta_c + 273)^4\right)$$
$$= 1.00 \cdot 0.35 \cdot 1.00 \cdot (5.67E-08) \cdot ((738.56 + 273)^4 - (645.19 + 273)^4) = 6673.74 W/m^2$$

$$h_{net} = h_{net,c} + h_{net,r} = 2334.38 + 6673.74 = 9008.12 W/m^2$$

Součinitel průřezu nechráněných ocelových prvků A_m/V = 402.640 1/m
Povrch obdélníka opsaného průřezu A_m,b = 0.370 m²/m
Součinitel průřezu obdélníka opsaného průřezu (A_m/V)_b = 244.224 1/m
Uvažovat vliv zastínění = True
Opravný součinitel zastinění k_sh = 0.607 — ČSN EN 1993-1-2 odst. 4.2.5.1 (2), výraz 4.26b

Normová teplotní křivka Θ_g = 738.56 °C — ČSN EN 1991-1-2 odst. 3.2.1 (1), výraz 3.4
Přírůstek tep. oceli Θ_a,t = 1.76 °C — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Teplota oceli Θ_c = 645.19 °C
Měrné teplo oceli c_a = 806.09 J/kgK — ČSN EN 1993-1-2 odst. 3.4.1.2 (1), výraz 3.2
Čistý tepelný tok h_net = 9008.12 W/m² — ČSN EN 1991-1-2 odst. 3.1 (2), výraz 3.1



REDUKČNÍ SOUČINITELÉ OCELI A SPOJŮ PŘI ZVÝŠENÝCH TEPLOTÁCH DLE ČSN EN 1993-1-2

Redukční součinitel	Redukční součinitel
Součinitelé pro průřezy 1-3. třídy	
k_yQ	0.362
k_PQ	0.133
k_EQ	0.229
Součinitelé pro průřezy 4. třídy	
k_PQ	0.223
k_EQ	0.229
Součinitelé pro spoje	
k_bQ	0.166
k_wQ	0.266

Teplota:

Teplota při požáru T = 645 °C — ČSN EN 1993-1-2

Materiál:

Ocel = HX500LAD — ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez kluzu oceli f_y = 500 MPa — ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez pevnosti oceli f_u = 550 MPa — ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Modul pružnosti oceli E = 210 GPa
Třída průřezu = 4
Redukovaná mez kluzu oceli f_y,red = 111.500 MPa — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaná mez pevnosti oceli f_u,red = 122.650 MPa — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaný modul pružnosti oceli E_red = 48.090 GPa — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25

Zatížení při požáru

Popis:
Poznámky:
Datum: 2022.01.18 @ 13:30:44

STANOVENÍ NORMOVÉ TEPLOTNÍ KŘIVKY A TEPLOTY OCELI DLE ČSN EN 1991-1-2

Zadání:

Profile = H 112x120x5
Výška profilu h = 188 mm
Šířka profilu b = 120 mm
Objem prvku na jednotku délky V = 2025 mm²
Šířka ohřívané pásnice b_fl = 0 mm
Plocha povrchu prvku na jednotku délky A_m = 0.820 m²/m

Čas vystavení účinkům požáru t = 15 min
Časový úsek Δ t = 5 s — ČSN EN 1993-1-2 odst. 4.2.5.1 (4)
Uvažovat vliv pozinkování = True
Emisivita prvku ε_m = 0.35

Emisivita požáru ε_f = 1.00
Polohový faktor φ = 1.00
Součinitel přestupu tepla α_c = 25.00 W/m²K — ČSN EN 1991-1-2 odst. 3.2.1 (2)
Stefan-Boltzmanova konstanta δ = 5.67E-08 W/m²K^4 — ČSN EN 1991-1-2 odst. 3.1 (6)
Objemová hmotnost ρ = 7850 kg/m³ — ČSN EN 1993-1-2 odst. 3.2.2 (1)
Uvažovat ohřev jen 3 stran = False

Výpočet:

$$Am/V = \frac{A_m}{V} = \frac{0.820}{2025 \cdot 0.000001} = 404.938 m^{-1}$$

$$Uvažovat\ ohřev\ jen\ 3\ stran = False => A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l$$

$$A_{m,b} = (2 \cdot b + 2 \cdot h) \cdot l = (2 \cdot 120 \cdot 0.001 + 2 \cdot 188 \cdot 0.001) \cdot 1 = 0.616 m^2/m$$

$$(A_m/V)_b = \frac{A_{m,b}}{V} = \frac{0.616}{2025 \cdot 0.000001} = 304.198 m^{-1}$$

$$Uvažovat\ vliv\ zastínění = True => k_{sh} = \frac{(A_m/V)_b}{Am/V}$$

$$k_{sh} = \frac{(A_m/V)_b}{Am/V} = \frac{304.198}{404.938} = 0.751$$

$$\Theta_g = 20 + 345 \cdot \log_{10}(8 \cdot t + 1) = 20 + 345 \cdot \log_{10}(8 \cdot 15 + 1) = 738.56\,°C$$

$$\Theta_a, t = k_{s,h} \cdot \left(\frac{Am/V}{c_a \cdot \rho}\right) \cdot h_{net} \cdot \Delta t = 0.751 \cdot \left(\frac{404.938}{867.04 \cdot 7850}\right) \cdot 6494.38 \cdot 5 = 1.47\,°C$$

$$\Theta_c = \Theta_c + \Theta_a, t = 673.326 + 1.469 = 673.33\,°C$$

$$c_a = 666 + \left(\frac{13002}{738 - \theta_c}\right) = 666 + \left(\frac{13002}{738 - 673.33}\right) = 867.04 J/kgK$$

$$h_{net} = h_{net,c} + h_{net,r}$$

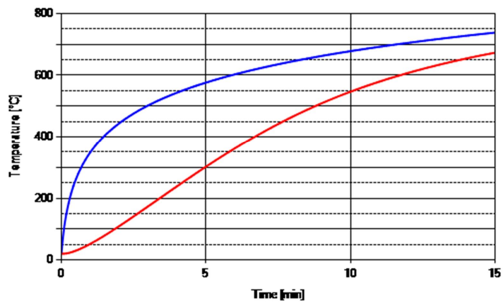
$$h_{net,c} = \alpha_c \cdot (\theta_g - \theta_c) = 25.00 \cdot (738.56 - 673.33) = 1630.88 W/m^2$$

$$h_{net,r} = \varphi \cdot e_m \cdot e_f \cdot \delta \cdot \left((\theta_g + 273)^4 - (\theta_c + 273)^4\right)$$
$$= 1.00 \cdot 0.35 \cdot 1.00 \cdot (5.67E - 08) \cdot ((738.56 + 273)^4 - (673.33 + 273)^4) = 4863.50 W/m^2$$

$$h_{net} = h_{net,c} + h_{net,r} = 1630.88 + 4863.50 = 6494.38 W/m^2$$

Součinitel průřezu nechráněných ocelových prvků A_m/V = 404.938 1/m
Povrch obdélníka opsaného průřezu A_m,b = 0.616 m²/m
Součinitel průřezu obdélníka opsaného průřezu (A_m/V)_b = 304.198 1/m
Uvažovat vliv zastínění = True
Opravný součinitel zastínění k_sh = 0.751 — ČSN EN 1993-1-2 odst. 4.2.5.1 (2), výraz 4.26b

Normová teplotní křivka Θ_g = 738.56 °C — ČSN EN 1991-1-2 odst. 3.2.1 (1), výraz 3.4
Přírůstek tep. oceli Θ_a,t = 1.47 °C — ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Teplota oceli Θ_c = 673.33 °C
Měrné teplo oceli c_a = 867.04 J/kgK — ČSN EN 1993-1-2 odst. 3.4.1.2 (1), výraz 3.2
Čistý tepelný tok h_net = 6494.38 W/m² — ČSN EN 1991-1-2 odst. 3.1 (2), výraz 3.1



REDUKČNÍ SOUČINITELÉ OCELI A SPOJŮ PŘI ZVÝŠENÝCH TEPLOTÁCH DLE ČSN EN 1993-1-2

Redukční součinitel	Redukční součinitel
Součinitelé pro průřezy 1-3. třídy	
k_yQ	0.295
k_PQ	0.103
k_EQ	0.179
Součinitelé pro průřezy 4. třídy	
k_PQ	0.176
k_EQ	0.179
Součinitelé pro spoje	
k_bQ	0.132
k_wQ	0.197

Teplota:

Teplota při požáru T =	673 °C	ČSN EN 1993-1-2

Materiál:

Ocel =	HX500LAD	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez kluzu oceli f_y =	500 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Mez pevnosti oceli f_u =	550 MPa	ČSN EN 1993-1-1 odst. 3.2.1, tab. 3.1
Modul pružnosti oceli E =	210 GPa	
Třída průřezu =	4	
Redukovaná mez kluzu oceli f_y,red =	88.000 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaná mez pevnosti oceli f_u,red =	96.800 MPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25
Redukovaný modul pružnosti oceli E_red =	37.590 GPa	ČSN EN 1993-1-2 odst. 4.2.5.1 (1), výraz 4.25

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 1 **POINT:** 2 **COORDINATE:** x = 0.50 L = 0.88 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX500LAD fy = 500.00 MPa fya = 522.15 MPa fyb = 500.00 MPa

SECTION PARAMETERS: C 100x85x5

h=100 mm	gM0=1.00	gM1=1.00	
b=85 mm	Ay=800 mm2	Az=475 mm2	Ax=1580 mm2
tw=5 mm	Iy=2510000 mm4	Iz=1650000 mm4	Ix=13200 mm4
tf=5 mm	Wply=55303 mm3	Wplz=43011 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=639.60 C	Oay,max=639.60 C	Oaz,max=639.60 C	ky,O=0.37
kE,O=0.24	ky,O,y=0.37	ky,O,z=0.37	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -19.45 kN Mz,fi,Ed = 0.03 kN*m
Nt,fi,t,Rd = 309.34 kN Mz,el,fi,t,Rd = 6.90 kN*m
Mz,c,fi,t,Rd = 6.90 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.06 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.07 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.06 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 2 **POINT:** 2 **COORDINATE:** x = 0.50 L = 0.88 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX500LAD fy = 500.00 MPa fya = 522.15 MPa fyb = 500.00 MPa

SECTION PARAMETERS: C 100x85x5

h=100 mm	gM0=1.00	gM1=1.00	
b=85 mm	Ay=800 mm2	Az=475 mm2	Ax=1580 mm2
tw=5 mm	Iy=2510000 mm4	Iz=1650000 mm4	Ix=13200 mm4
tf=5 mm	Wply=55303 mm3	Wplz=43011 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=639.60 C	Oay,max=639.60 C	Oaz,max=639.60 C	ky,O=0.37
kE,O=0.24	ky,O,y=0.37	ky,O,z=0.37	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -19.46 kN Mz,fi,Ed = 0.03 kN*m
Nt,fi,t,Rd = 309.34 kN Mz,el,fi,t,Rd = 6.90 kN*m
Mz,c,fi,t,Rd = 6.90 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.06 < 1.00 EN313(6.1.8.(1))

N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.07 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.06 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 8 **POINT:** 3 **COORDINATE:** x = 1.00 L = 1.13 m

LOADS:
Governing Load Case: 11 R15 Snow 0.5 right (101+102+104)*1.00+201*0.20+202*0.10

MATERIAL:
S350GD fy = 350.00 MPa fya = 371.00 MPa fyb = 350.00 MPa

SECTION PARAMETERS: C 100x77x3

h=100 mm	gM0=1.00	gM1=1.00	
b=77 mm	Ay=444 mm2	Az=291 mm2	Ax=840 mm2
tw=3 mm	Iy=1440000 mm4	Iz=690000 mm4	Ix=2520 mm4
tf=3 mm	Wely=28800 mm3	Welz=14778 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=699.61 C	Oay,max=699.61 C	Oaz,max=699.61 C	ky,O=0.23
kE,O=0.13	ky,O,y=0.23	ky,O,z=0.23	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = 6.79 kN Vy,fi,Ed = -0.01 kN
Nc,fi,t,Rd = 67.89 kN Vy,c,fi,t,Rd = 20.81 kN
Nb,fi,t,Rd = 51.88 kN
Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.13 m	Lam_y,O = 0.47	Lz = 0.79 m	Lam_z,O = 0.48
Lcr,y = 1.13 m	Xy,fi = 0.77	Lcr,z = 0.79 m	Xz,fi = 0.76

Lamy = 27.37 Lamz = 27.68

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.53	Curve,TF=b	alfa,TF=0.53
Lt=0.79 m	fi,T=1.09	Ncr,y=2323.40 kN	fi,TF=1.19
Ncr,T=710.48 kN	X,T,fi=0.56	Ncr,TF=586.32 kN	X,TF,fi=0.52
Lam_T,O=0.86	Nb,T,Rd,fi=38.26 kN	Lam_TF,O=0.94	Nb,TF,Rd=35.22 kN

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.10 < 1.00 EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00 EN313(6.1.5)
Global stability check of member:
Lambda,y = 27.37 < Lambda,max = 210.00 Lambda,z = 27.68 < Lambda,max = 210.00 STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.19 < 1.00 EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 9 **POINT:** 3 **COORDINATE:** x = 1.00 L = 1.13 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
S350GD fy = 350.00 MPa fya = 371.00 MPa fyb = 350.00 MPa

SECTION PARAMETERS: C 100x77x3

h=100 mm	gM0=1.00	gM1=1.00	
b=77 mm	Ay=444 mm2	Az=291 mm2	Ax=840 mm2
tw=3 mm	Iy=1440000 mm4	Iz=690000 mm4	Ix=2520 mm4
tf=3 mm	Wely=28800 mm3	Welz=14778 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=699.61 C	Oay,max=699.61 C	Oaz,max=699.61 C	ky,O=0.23
kE,O=0.13	ky,O,y=0.23	ky,O,z=0.23	

INTERNAL FORCES AND CAPACITIES:

N,fi,Ed = 5.87 kN
Nc,fi,t,Rd = 67.89 kN
Nb,fi,t,Rd = 51.88 kN
Vy,fi,Ed = -0.01 kN
Vy,c,fi,t,Rd = 20.81 kN
Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.13 m	Lam_y,O = 0.47	Lz = 0.79 m	Lam_z,O = 0.48
Lcr,y = 1.13 m	Xy,fi = 0.77	Lcr,z = 0.79 m	Xz,fi = 0.76
Lamy = 27.38		Lamz = 27.68	

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.53	Curve,TF=b	alfa,TF=0.53
Lt=0.79 m	fi,T=1.09	Ncr,y=2323.02 kN	fi,TF=1.19
Ncr,T=710.37 kN	X,T,fi=0.56	Ncr,TF=586.23 kN	X,TF,fi=0.52
Lam_T,O=0.86	Nb,T,Rd,fi=38.26 kN	Lam_TF,O=0.94	Nb,TF,Rd=35.22 kN

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.09 < 1.00 EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00 EN313(6.1.5)
Global stability check of member:
Lambda,y = 27.38 < Lambda,max = 210.00 Lambda,z = 27.68 < Lambda,max = 210.00 STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.17 < 1.00 EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 14 **POINT:** 2 **COORDINATE:** x = 0.50 L = 0.72 m

LOADS:
Governing Load Case: 40 R15 Wind Cpe min (101+102+104)*1.00+(301+320)*0.20

MATERIAL:
S350GD fy = 350.00 MPa fya = 364.20 MPa fyb = 350.00 MPa

SECTION PARAMETERS: C 100x75x2

h=100 mm	gM0=1.00	gM1=1.00	
b=75 mm	Ay=292 mm2	Az=196 mm2	Ax=552 mm2
tw=2 mm	Iy=967000 mm4	Iz=434000 mm4	Ix=736 mm4
tf=2 mm	Wply=21075 mm3	Wplz=13226 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain

Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=714.75 C	Oay,max=714.75 C	Oaz,max=714.75 C	ky,O=0.21
kE,O=0.12	ky,O,y=0.21	ky,O,z=0.21	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -2.78 kN
Nt,fi,t,Rd = 42.68 kN
Mz,fi,Ed = 0.01 kN*m
Mz,el,fi,t,Rd = 0.73 kN*m
Mz,c,fi,t,Rd = 0.64 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.07 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.07 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.06 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 15 **POINT:** 3 **COORDINATE:** x = 1.00 L = 1.43 m

LOADS:
Governing Load Case: 20 R15 Wind Cpe max (101+102)*1.00+(300+321)*0.20

MATERIAL:
S350GD fy = 350.00 MPa fya = 364.20 MPa fyb = 350.00 MPa

SECTION PARAMETERS: C 100x75x2

h=100 mm	gM0=1.00	gM1=1.00	
b=75 mm	Ay=292 mm2	Az=196 mm2	Ax=552 mm2
tw=2 mm	Iy=967000 mm4	Iz=434000 mm4	Ix=736 mm4
tf=2 mm	Wely=19340 mm3	Welz=9388 mm3	

Weff,z=9388 mm3 Aeff=472 mm2

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=714.75 C	Oay,max=714.75 C	Oaz,max=714.75 C	ky,O=0.12
kE,O=0.12	ky,O,y=0.12	ky,O,z=0.12	

INTERNAL FORCES AND CAPACITIES:

N,fi,Ed = 1.09 kN		Vy,fi,Ed = 0.01 kN
Nc,fi,t,Rd = 20.03 kN	Mz,el,fi,t,Rd = 0.41 kN*m	Vy,c,fi,t,Rd = 7.18 kN
Nb,fi,t,Rd = 15.85 kN	Mz,c,fi,t,Rd = 0.41 kN*m	
	dMz,fi,Ed = 0.00 kN*m	Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.43 m	Lam_y,O = 0.41	Lz = 1.00 m	Lam_z,O = 0.42
Lcr,y = 1.43 m	Xy,fi = 0.80	Lcr,z = 1.00 m	Xz,fi = 0.79
Lamy = 34.17		Lamz = 35.71	kz = 1.04

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.53	Curve,TF=b	alfa,TF=0.53
Lt=1.00 m	fi,T=0.98	Ncr,y=979.77 kN	fi,TF=1.05
Ncr,T=290.09 kN	X,T,fi=0.62	Ncr,TF=241.44 kN	X,TF,fi=0.58
Lam_T,O=0.75	Nb,T,Rd,fi=12.46 kN	Lam_TF,O=0.82	Nb,TF,Rd=11.69 kN

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.05 < 1.00 EN313(6.1.9.(1))
N,fi,Ed/Nc,fi,t,Rd + (Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rdcom = 0.06 < 1.00 EN313(6.1.9.(1))
(Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rdten - N,fi,Ed/Nc,fi,t,Rd = 0.05 < 1.00 EN313(6.1.9.(2))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00 EN313(6.1.5)
Global stability check of member:
Lambda,y = 34.17 < Lambda,max = 210.00 Lambda,z = 35.71 < Lambda,max = 210.00 STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.09 < 1.00 EN312(4.2.3.5.(1))
N,fi,Ed/(Xmin,fi*N,c,fi,t,Rd) + kz*(Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rd = 0.07 < 1.00 EN312(4.2.3.5.(1))
N,fi,Ed/(Xz,fi*N,c,fi,t,Rd) + kz*(Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rd = 0.07 < 1.00 EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 20 **POINT:** 3 **COORDINATE:** x = 1.00 L = 1.04 m

LOADS:
Governing Load Case: 40 R15 Wind Cpe min (101+102+104)*1.00+(301+320)*0.20

MATERIAL:
S350GD fy = 350.00 MPa fya = 364.20 MPa fyb = 350.00 MPa

SECTION PARAMETERS: C 100x75x2

h=100 mm	gM0=1.00	gM1=1.00	
b=75 mm	Ay=292 mm2	Az=196 mm2	Ax=552 mm2
tw=2 mm	Iy=967000 mm4	Iz=434000 mm4	Ix=736 mm4
tf=2 mm	Wely=19340 mm3	Welz=9388 mm3	
		Weff,z=9388 mm3	Aeff=472 mm2

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=714.75 C	Oay,max=714.75 C	Oaz,max=714.75 C	ky,O=0.12
kE,O=0.12	ky,O,y=0.12	ky,O,z=0.12	

INTERNAL FORCES AND CAPACITIES:

N,fi,Ed = 1.57 kN		Vy,fi,Ed = -0.01 kN
Nc,fi,t,Rd = 20.03 kN	Mz,el,fi,t,Rd = 0.41 kN*m	Vy,c,fi,t,Rd = 7.18 kN
Nb,fi,t,Rd = 16.99 kN	Mz,c,fi,t,Rd = 0.41 kN*m	
	dMz,fi,Ed = 0.00 kN*m	Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.04 m	Lam_y,O = 0.30	Lz = 0.73 m	Lam_z,O = 0.31
Lcr,y = 1.04 m	Xy,fi = 0.85	Lcr,z = 0.73 m	Xz,fi = 0.85
Lamy = 24.90		Lamz = 26.01	kz = 1.04

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.53	Curve,TF=b	alfa,TF=0.53
Lt=0.73 m	fi,T=0.80	Ncr,y=1845.79 kN	fi,TF=0.84
Ncr,T=538.76 kN	X,T,fi=0.73	Ncr,TF=449.57 kN	X,TF,fi=0.70
Lam_T,O=0.55	Nb,T,Rd,fi=14.59 kN	Lam_TF,O=0.60	Nb,TF,Rd=14.04 kN

VERIFICATION FORMULAS:

Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.08 < 1.00   EN313(6.1.9.(1))
N,fi,Ed/Nc,fi,t,Rd + (Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rdcom = 0.08 < 1.00   EN313(6.1.9.(1))
(Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rdten - N,fi,Ed/Nc,fi,t,Rd = 0.07 < 1.00   EN313(6.1.9.(2))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00   EN313(6.1.5)
Global stability check of member:
Lambda,y = 24.90 < Lambda,max = 210.00   Lambda,z = 26.01 < Lambda,max = 210.00   STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.11 < 1.00   EN312(4.2.3.5.(1))
N,fi,Ed/(Xmin,fi*N,c,fi,t,Rd) + kz*(Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rd = 0.10 < 1.00   EN312(4.2.3.5.(1))
N,fi,Ed/(Xz,fi*N,c,fi,t,Rd) + kz*(Mz,fi,Ed+dMz,fi,Ed)/Mz,c,fi,t,Rd = 0.10 < 1.00   EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 21   **POINT:** 2   **COORDINATE:** x = 0.50 L = 0.52 m

LOADS:
Governing Load Case: 40 R15 Wind Cpe min (101+102+104)*1.00+(301+320)*0.20

MATERIAL:
S350GD   fy = 350.00 MPa   fya = 364.20 MPa   fyb = 350.00 MPa

SECTION PARAMETERS: C 100x75x2

h=100 mm	gM0=1.00	gM1=1.00	
b=75 mm	Ay=292 mm2	Az=196 mm2	Ax=552 mm2
tw=2 mm	Iy=967000 mm4	Iz=434000 mm4	Ix=736 mm4
tf=2 mm	Wply=21075 mm3	Wplz=13226 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=714.75 C	Oay,max=714.75 C	Oaz,max=714.75 C	ky,O=0.21
kE,O=0.12	ky,O,y=0.21	ky,O,z=0.21	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -1.36 kN   Mz,fi,Ed = -0.00 kN*m
Nt,fi,t,Rd = 42.68 kN   Mz,el,fi,t,Rd = 0.73 kN*m
Mz,c,fi,t,Rd = 0.73 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.03 < 1.00   EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.03 < 1.00   EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.03 < 1.00   EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 26   **POINT:** 3   **COORDINATE:** x = 1.00 L = 1.27 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
S350GD   fy = 350.00 MPa   fya = 371.00 MPa   fyb = 350.00 MPa

SECTION PARAMETERS: C 100x77x3

h=100 mm	gM0=1.00	gM1=1.00	
b=77 mm	Ay=444 mm2	Az=291 mm2	Ax=840 mm2
tw=3 mm	Iy=1440000 mm4	Iz=690000 mm4	Ix=2520 mm4
tf=3 mm	Wely=28800 mm3	Welz=14778 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=699.61 C	Oay,max=699.61 C	Oaz,max=699.61 C	ky,O=0.23
kE,O=0.13	ky,O,y=0.23	ky,O,z=0.23	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = 9.16 kN   Vy,fi,Ed = 0.02 kN
Nc,fi,t,Rd = 67.89 kN   Vy,c,fi,t,Rd = 20.81 kN

Nb,fi,t,Rd = 49.84 kN
Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.27 m	Lam_y,O = 0.53	Lz = 0.89 m	Lam_z,O = 0.54
Lcr,y = 1.27 m	Xy,fi = 0.74	Lcr,z = 0.89 m	Xz,fi = 0.73
Lamy = 30.71		Lamz = 31.06	

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.53	Curve,TF=b	alfa,TF=0.53
Lt=0.89 m	fi,T=1.21	Ncr,y=1845.61 kN	fi,TF=1.33
Ncr,T=570.29 kN	X,T,fi=0.51	Ncr,TF=469.70 kN	X,TF,fi=0.46
Lam_T,O=0.95	Nb,T,Rd,fi=34.77 kN	Lam_TF,O=1.05	Nb,TF,Rd=31.55 kN

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.13 < 1.00   EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00   EN313(6.1.5)
Global stability check of member:
Lambda,y = 30.71 < Lambda,max = 210.00   Lambda,z = 31.06 < Lambda,max = 210.00   STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.29 < 1.00   EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 27   **POINT:** 3   **COORDINATE:** x = 1.00 L = 1.28 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
S350GD   fy = 350.00 MPa   fya = 371.00 MPa   fyb = 350.00 MPa

SECTION PARAMETERS: C 100x77x3

h=100 mm	gM0=1.00	gM1=1.00	
b=77 mm	Ay=444 mm2	Az=291 mm2	Ax=840 mm2
tw=3 mm	Iy=1440000 mm4	Iz=690000 mm4	Ix=2520 mm4
tf=3 mm	Wely=28800 mm3	Welz=14778 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=699.61 C	Oay,max=699.61 C	Oaz,max=699.61 C	ky,O=0.23
kE,O=0.13	ky,O,y=0.23	ky,O,z=0.23	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = 8.99 kN   Vy,fi,Ed = 0.02 kN
Nc,fi,t,Rd = 67.89 kN   Vy,c,fi,t,Rd = 20.81 kN
Nb,fi,t,Rd = 49.77 kN
Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.28 m	Lam_y,O = 0.53	Lz = 0.89 m	Lam_z,O = 0.54
Lcr,y = 1.28 m	Xy,fi = 0.74	Lcr,z = 0.89 m	Xz,fi = 0.73
Lamy = 30.82		Lamz = 31.16	

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.53	Curve,TF=b	alfa,TF=0.53
Lt=0.89 m	fi,T=1.21	Ncr,y=1833.08 kN	fi,TF=1.34
Ncr,T=566.62 kN	X,T,fi=0.51	Ncr,TF=466.64 kN	X,TF,fi=0.46
Lam_T,O=0.96	Nb,T,Rd,fi=34.66 kN	Lam_TF,O=1.06	Nb,TF,Rd=31.44 kN

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.13 < 1.00   EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00   EN313(6.1.5)
Global stability check of member:
Lambda,y = 30.82 < Lambda,max = 210.00   Lambda,z = 31.16 < Lambda,max = 210.00   STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.29 < 1.00   EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 32   **POINT:** 2   **COORDINATE:** x = 0.50 L = 0.45 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
S350GD   fy = 350.00 MPa   fya = 371.00 MPa   fyb = 350.00 MPa

SECTION PARAMETERS: C 100x77x3

h=100 mm	gM0=1.00	gM1=1.00	
b=77 mm	Ay=444 mm2	Az=291 mm2	Ax=840 mm2
tw=3 mm	Iy=1440000 mm4	Iz=690000 mm4	Ix=2520 mm4
tf=3 mm	Wply=31305 mm3	Wplz=20267 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=699.61 C	Oay,max=699.61 C	Oaz,max=699.61 C	ky,O=0.23
kE,O=0.13	ky,O,y=0.23	ky,O,z=0.23	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -9.08 kN — Mz,fi,Ed = -0.00 kN*m
Nt,fi,t,Rd = 71.97 kN — Mz,el,fi,t,Rd = 1.27 kN*m
Mz,c,fi,t,Rd = 1.27 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.13 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.13 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.12 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 33 **POINT:** 2 **COORDINATE:** x = 0.50 L = 0.45 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
S350GD fy = 350.00 MPa fya = 371.00 MPa fyb = 350.00 MPa

SECTION PARAMETERS: C 100x77x3

h=100 mm	gM0=1.00	gM1=1.00	
b=77 mm	Ay=444 mm2	Az=291 mm2	Ax=840 mm2
tw=3 mm	Iy=1440000 mm4	Iz=690000 mm4	Ix=2520 mm4
tf=3 mm	Wply=31305 mm3	Wplz=20267 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=699.61 C	Oay,max=699.61 C	Oaz,max=699.61 C	ky,O=0.23
kE,O=0.13	ky,O,y=0.23	ky,O,z=0.23	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -8.95 kN — Mz,fi,Ed = -0.00 kN*m
Nt,fi,t,Rd = 71.97 kN — Mz,el,fi,t,Rd = 1.27 kN*m
Mz,c,fi,t,Rd = 1.27 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.12 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.13 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.12 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 38 **POINT:** 3 **COORDINATE:** x = 1.00 L = 1.12 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX420LAD fy = 420.00 MPa fya = 443.04 MPa fyb = 420.00 MPa

SECTION PARAMETERS: C 100x82x4

h=100 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=592 mm2	Az=368 mm2	Ax=1167 mm2
tw=4 mm	Iy=1880698 mm4	Iz=1097602 mm4	Ix=5370 mm4
tf=4 mm	Wply=41760 mm3	Wplz=30773 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=676.66 C	Oay,max=676.66 C	Oaz,max=676.66 C	ky,O=0.29
kE,O=0.17	ky,O,y=0.29	ky,O,z=0.29	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = 24.81 kN — Vy,fi,Ed = 0.03 kN
Nc,fi,t,Rd = 140.17 kN — Vy,c,fi,t,Rd = 41.25 kN
Nb,fi,t,Rd = 106.81 kN
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.12 m	Lam_y,O = 0.51	Lz = 0.79 m	Lam_z,O = 0.47
Lcr,y = 1.12 m	Xy,fi = 0.76	Lcr,z = 0.79 m	Xz,fi = 0.78
Lamy = 27.96		Lamz = 25.62	

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.49	Curve,TF=b	alfa,TF=0.49
Lt=0.79 m	fi,T=1.01	fi,TF=1.14	
Ncr,T=1276.58 kN	X,T,fi=0.61	Ncr,y=3092.71 kN	X,TF,fi=0.55
Lam_T,O=0.80	Nb,T,Rd,fi=85.63 kN	Ncr,TF=972.40 kN	Nb,TF,Rd=76.86 kN
		Lam_TF,O=0.92	

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.18 < 1.00 EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00 EN313(6.1.5)
Global stability check of member:
Lambda,y = 27.96 < Lambda,max = 210.00 Lambda,z = 25.62 < Lambda,max = 210.00 STABLE

N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.32 < 1.00 EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 39 **POINT:** 3 **COORDINATE:** x = 1.00 L = 1.12 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX420LAD fy = 420.00 MPa fya = 443.04 MPa fyb = 420.00 MPa

SECTION PARAMETERS: C 100x82x4

h=100 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=592 mm2	Az=368 mm2	Ax=1167 mm2
tw=4 mm	Iy=1880698 mm4	Iz=1097602 mm4	Ix=5370 mm4
tf=4 mm	Wply=41760 mm3	Wplz=30773 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	

Thermal properties of steel:

roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		

Calculated parameters:

Oa,max=676.66 C	Oay,max=676.66 C	Oaz,max=676.66 C	ky,O=0.29
kE,O=0.17	ky,O,y=0.29	ky,O,z=0.29	

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = 24.82 kN — Vy,fi,Ed = 0.03 kN
Nc,fi,t,Rd = 140.17 kN — Vy,c,fi,t,Rd = 41.25 kN
Nb,fi,t,Rd = 106.81 kN
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.12 m	Lam_y,O = 0.51	Lz = 0.79 m	Lam_z,O = 0.47
Lcr,y = 1.12 m	Xy,fi = 0.76	Lcr,z = 0.79 m	Xz,fi = 0.78
Lamy = 27.96		Lamz = 25.62	

Torsional buckling:
Curve,T=b alfa,T=0.49
Lt=0.79 m fi,T=1.01
Ncr,T=1276.97 kN X,T,fi=0.61
Lam_T,O=0.80 Nb,T,Rd,fi=85.64 kN

Flexural-torsional buckling
Curve,TF=b alfa,TF=0.49
Ncr,y=3093.70 kN fi,TF=1.14
Ncr,TF=972.70 kN X,TF,fi=0.55
Lam_TF,O=0.92 Nb,TF,Rd=76.87 kN

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.18 < 1.00 EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00 EN313(6.1.5)
Global stability check of member:
Lambda,y = 27.96 < Lambda,max = 210.00 Lambda,z = 25.62 < Lambda,max = 210.00 STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.32 < 1.00 EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 44 **POINT:** 2 **COORDINATE:** x = 0.50 L = 0.39 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX420LAD fy = 420.00 MPa fya = 443.04 MPa fyb = 420.00 MPa

SECTION PARAMETERS: C 100x82x4

h=100 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=592 mm2	Az=368 mm2	Ax=1167 mm2
tw=4 mm	Iy=1880698 mm4	Iz=1097602 mm4	Ix=5370 mm4
tf=4 mm	Wply=41760 mm3	Wplz=30773 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected
treq=15.00 min expos=4 gnr=1.00 gnc=1.00
gm,fi=1.00 k1=1.00 k2=1.00
Thermal properties of steel:
roa=7850.00 kg/m3 alfc=25.00 W/(m2*K) Fi=1.00 em=0.35
ef=1.00 'Shadow' effect - active
Calculated parameters:
Oa,max=676.66 C Oay,max=676.66 C Oaz,max=676.66 C ky,O=0.29
kE,O=0.17 ky,O,y=0.29 ky,O,z=0.29

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -22.95 kN Mz,fi,Ed = -0.01 kN*m

Nt,fi,t,Rd = 147.85 kN Mz,el,fi,t,Rd = 3.10 kN*m
Mz,c,fi,t,Rd = 3.10 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.16 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.16 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.15 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 45 **POINT:** 2 **COORDINATE:** x = 0.50 L = 0.39 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX420LAD fy = 420.00 MPa fya = 443.04 MPa fyb = 420.00 MPa

SECTION PARAMETERS: C 100x82x4

h=100 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=592 mm2	Az=368 mm2	Ax=1167 mm2
tw=4 mm	Iy=1880698 mm4	Iz=1097602 mm4	Ix=5370 mm4
tf=4 mm	Wply=41760 mm3	Wplz=30773 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected
treq=15.00 min expos=4 gnr=1.00 gnc=1.00
gm,fi=1.00 k1=1.00 k2=1.00
Thermal properties of steel:
roa=7850.00 kg/m3 alfc=25.00 W/(m2*K) Fi=1.00 em=0.35
ef=1.00 'Shadow' effect - active
Calculated parameters:
Oa,max=676.66 C Oay,max=676.66 C Oaz,max=676.66 C ky,O=0.29
kE,O=0.17 ky,O,y=0.29 ky,O,z=0.29

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = -22.96 kN Mz,fi,Ed = -0.01 kN*m
Nt,fi,t,Rd = 147.85 kN Mz,el,fi,t,Rd = 3.10 kN*m
Mz,c,fi,t,Rd = 3.10 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.16 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.16 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.15 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 50 **POINT:** 3 **COORDINATE:** x = 1.00 L = 0.97 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX420LAD fy = 420.00 MPa fya = 443.04 MPa fyb = 420.00 MPa

SECTION PARAMETERS: C 100x82x4

h=100 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=592 mm2	Az=368 mm2	Ax=1167 mm2
tw=4 mm	Iy=1880698 mm4	Iz=1097602 mm4	Ix=5370 mm4
tf=4 mm	Wply=41760 mm3	Wplz=30773 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005
Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected
treq=15.00 min expos=4 gnr=1.00 gnc=1.00
gm,fi=1.00 k1=1.00 k2=1.00
Thermal properties of steel:
roa=7850.00 kg/m3 alfc=25.00 W/(m2*K) Fi=1.00 em=0.35
ef=1.00 'Shadow' effect - active
Calculated parameters:

Oa,max=676.66 C Oay,max=676.66 C Oaz,max=676.66 C ky,O=0.29
kE,O=0.17 ky,O,y=0.29 ky,O,z=0.29

INTERNAL FORCES AND CAPACITIES:
N,fi,Ed = 48.68 kN Vy,fi,Ed = 0.03 kN
Nc,fi,t,Rd = 140.17 kN Vy,c,fi,t,Rd = 41.25 kN
Nb,fi,t,Rd = 111.58 kN
Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:
About y axis:
Ly = 0.97 m Lam_y,O = 0.44
Lcr,y = 0.97 m Xy,fi = 0.80
Lamy = 24.20

About z axis:
Lz = 0.68 m Lam_z,O = 0.41
Lcr,z = 0.68 m Xz,fi = 0.81
Lamz = 22.17

Torsional buckling:
Curve,T=b alfa,T=0.49
Lt=0.68 m fi,T=0.91
Ncr,T=1684.35 kN X,T,fi=0.67
Lam_T,O=0.70 Nb,T,Rd,fi=93.48 kN

Flexural-torsional buckling
Curve,TF=b alfa,TF=0.49
Ncr,y=4130.75 kN fi,TF=1.01
Ncr,TF=1287.04 kN X,TF,fi=0.61
Lam_TF,O=0.80 Nb,TF,Rd=85.88 kN

VERIFICATION FORMULAS:
Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.35 < 1.00 EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00 EN313(6.1.5)
Global stability check of member:
Lambda,y = 24.20 < Lambda,max = 210.00 Lambda,z = 22.17 < Lambda,max = 210.00 STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.57 < 1.00 EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 51 **POINT:** 3 **COORDINATE:** x = 1.00 L = 0.97 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX420LAD fy = 420.00 MPa fya = 443.04 MPa fyb = 420.00 MPa

SECTION PARAMETERS: C 100x82x4

h=100 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=592 mm2	Az=368 mm2	Ax=1167 mm2

tw=4 mm	Iy=1880698 mm4	Iz=1097602 mm4	Ix=5370 mm4
tf=4 mm	Wply=41760 mm3	Wplz=30773 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005

Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	
Thermal properties of steel:			
roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		
Calculated parameters:			
Oa,max=676.66 C	Oay,max=676.66 C	Oaz,max=676.66 C	ky,O=0.29
kE,O=0.17	ky,O,y=0.29	ky,O,z=0.29	

INTERNAL FORCES AND CAPACITIES:

N,fi,Ed = 48.68 kN	Vy,fi,Ed = 0.03 kN
Nc,fi,t,Rd = 140.17 kN	Vy,c,fi,t,Rd = 41.25 kN
Nb,fi,t,Rd = 111.61 kN	
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 0.97 m	Lam_y,O = 0.44	Lz = 0.68 m	Lam_z,O = 0.41
Lcr,y = 0.97 m	Xy,fi = 0.80	Lcr,z = 0.68 m	Xz,fi = 0.81
Lamy = 24.17		Lamz = 22.15	

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.49	Curve,TF=b	alfa,TF=0.49
Lt=0.68 m	fi,T=0.91	Ncr,y=4139.03 kN	fi,TF=1.01
Ncr,T=1687.61 kN	X,T,fi=0.67	Ncr,TF=1289.55 kN	X,TF,fi=0.61
Lam_T,O=0.69	Nb,T,Rd,fi=93.53 kN	Lam_TF,O=0.79	Nb,TF,Rd=85.94 kN

VERIFICATION FORMULAS:

Section strength check:
N,fi,Ed/Nc,fi,t,Rd = 0.35 < 1.00 EN313(6.1.9.(1))
Vy,fi,Ed/Vy,c,fi,t,Rd = 0.00 < 1.00 EN313(6.1.5)
Global stability check of member:
Lambda,y = 24.17 < Lambda,max = 210.00 Lambda,z = 22.15 < Lambda,max = 210.00 STABLE
N,fi,Ed/Min(Nb,fi,t,Rd , Nb,T,fi,t,Rd , Nb,TF,fi,t,Rd) = 0.57 < 1.00 EN312(4.2.3.5.(1))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 56 **POINT:** 2 **COORDINATE:** x = 0.50 L =

0.32 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX500LAD fy = 500.00 MPa fya = 522.15 MPa fyb = 500.00 MPa

SECTION PARAMETERS: C 100x85x5

h=100 mm	gM0=1.00	gM1=1.00	
b=85 mm	Ay=800 mm2	Az=475 mm2	Ax=1580 mm2
tw=5 mm	Iy=2510000 mm4	Iz=1650000 mm4	Ix=13200 mm4
tf=5 mm	Wply=55303 mm3	Wplz=43011 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005

Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	
Thermal properties of steel:			
roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		
Calculated parameters:			
Oa,max=639.60 C	Oay,max=639.60 C	Oaz,max=639.60 C	ky,O=0.37
kE,O=0.24	ky,O,y=0.37	ky,O,z=0.37	

INTERNAL FORCES AND CAPACITIES:

N,fi,Ed = -59.49 kN	Mz,fi,Ed = -0.01 kN*m	
Nt,fi,t,Rd = 309.34 kN	Mz,el,fi,t,Rd = 6.90 kN*m	
	Mz,c,fi,t,Rd = 6.90 kN*m	
		Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.19 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.19 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.19 < 1.00 EN313(6.1.8.(2))

Section OK !!!

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 57 **POINT:** 2 **COORDINATE:** x = 0.50 L =
0.32 m

LOADS:
Governing Load Case: 10 R15 Snow (101+102+104)*1.00+(201+202)*0.20

MATERIAL:
HX500LAD fy = 500.00 MPa fya = 522.15 MPa fyb = 500.00 MPa

SECTION PARAMETERS: C 100x85x5

h=100 mm	gM0=1.00	gM1=1.00	
b=85 mm	Ay=800 mm2	Az=475 mm2	Ax=1580 mm2
tw=5 mm	Iy=2510000 mm4	Iz=1650000 mm4	Ix=13200 mm4
tf=5 mm	Wply=55303 mm3	Wplz=43011 mm3	

PARAMETERS OF FIRE ANALYSIS: EN 1993-1-2:2005

Analysis type - Resistance domain
Temperature curve -Standard ISO 834
Steel member protection -Unprotected

treq=15.00 min	expos=4	gnr=1.00	gnc=1.00
gm,fi=1.00	k1=1.00	k2=1.00	
Thermal properties of steel:			
roa=7850.00 kg/m3	alfc=25.00 W/(m2*K)	Fi=1.00	em=0.35
ef=1.00	'Shadow' effect - active		
Calculated parameters:			
Oa,max=639.60 C	Oay,max=639.60 C	Oaz,max=639.60 C	ky,O=0.37
kE,O=0.24	ky,O,y=0.37	ky,O,z=0.37	

INTERNAL FORCES AND CAPACITIES:

N,fi,Ed = -59.51 kN	Mz,fi,Ed = -0.01 kN*m	
Nt,fi,t,Rd = 309.34 kN	Mz,el,fi,t,Rd = 6.90 kN*m	
	Mz,c,fi,t,Rd = 6.90 kN*m	
		Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

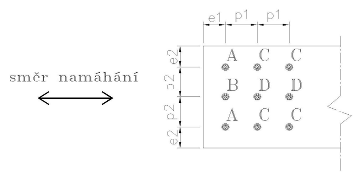
Section strength check:
N,fi,Ed/Nt,fi,t,Rd = 0.19 < 1.00 EN313(6.1.8.(1))
N,fi,Ed/Nt,fi,t,Rd + Mz,fi,Ed/Mz,c,fi,t,Rdten = 0.19 < 1.00 EN313(6.1.8.(1))
Mz,fi,Ed/Mz,c,fi,t,Rdcom - N,fi,Ed/Nt,fi,t,Rd = 0.19 < 1.00 EN313(6.1.8.(2))

Section OK !!!

PŘÍPOJ SPODNÍHO PASU UC56 M16

Únosnot šroubového spoje OTLAČENÍ / STŘICH / OSL. PRŮŘEZ / VYTRŽENÍ SKUPINY ŠROUBŮ

Dle: ČSN EN 1993-1-1, ČSN EN 1993-1-3,ČSN EN 1993-1-8



Geometrie

Ideální geometrie pro k_1=2,5 a α_d=1,0

e_{1A}=	**30**	mm	$e_1=3d_0$=	49.5	mm
e_{1B}=	**30**	mm	$e_1=3d_0$=	49.5	mm
e_2=	**25**	mm	$e_2=1,5d_0$=	24.8	mm
p_1=	**50**	mm	$p_1=3,75d_0$=	61.9	mm
p_2=	**62**	mm	$p_2=3d_0$=	49.5	mm

šroub Střih v dříku — 1-střižný

Pevnostní třída šroubu **8.8**

f_{ub}= **800** MPa

Průřez -Průměr šroubu d= **16** mm = > A= **201.0** mm2 (Plocha šroubu); A_s= **139.0** mm2 (Plocha jádra šroubu)

-Průměr otvoru d_0= **16.5** mm

Počet šroubů n_b= **12**

Profil C100x129x5 — t_1= **5.0** mm

Ocel **HX500LAD**

f_{u1}= **550.0** MPa

Zákl. materiál Stejný nebo únosnější

Koeficient bez. γ_{M0}= **1.00** γ_{M1}= **1.00** γ_{M2}= **1.25**

Únosnost v otlačení jednotlivých šroubů

(Viz. ČSN EN 1993-1-3: 8.3, ČSN EN 1993-1-8: 3.6.1)

$F_{b,Rd,1}=k_1\alpha_b f_u d t/\gamma_{M2}$

VE SMĚRU ZATÍŽENÍ

Koncové šrouby A: min { $\alpha_d=e_{1A}/3d_0$= ; $\alpha_d=f_{ub}/f_{u1}$= ; α_d=1.0 } 0.61

Koncové šrouby B: min { $\alpha_d=e_{1B}/3d_0$= ; $\alpha_d=f_{ub}/f_{u1}$= ; α_d=1.0 } 0.61

Vnitřní šrouby: min { $\alpha_d=(p_1/3d_0)-1/4$= ; $\alpha_d=f_{ub}/f_u$= ; α_d=1.0 } 0.76

KOLMO NA SMĚR ZATÍŽENÍ

Šrouby u okraje: min { $K_1=(2,8e_2/d_0)-1,7$= ; $K_1=(1,4p_2/d_0)-1,7$= ; K_1=2,5 } 2.50

Vnitřní šrouby: min { $K_1=(1.4p_2/d_0)-1,7$= ; K_1=2,5 } 2.50

t1= **5** mm; Ocel **HX500LAD**; fu1= **550** MPa	Poloha šroubu		$F_{b,Rd}$		Počet typu štroubu ve spoji
	Konec/okraj	A	**53.3**	kN	**2**
	Konec/vnitřní	B	**53.3**	kN	**2**
	Vnitřní/okraj	C	**66.9**	kN	**4**
	Vnitřní/vnitřní	D	**66.9**	kN	**4**
				SUMA=	**12** (kontrola)

Únosnost ve střihu jednoho šroubu

Únosnost ve střihu celého spoje

(Viz. ČSN EN 1993-1-3: 8.3)

α_v= **0.6**

n_v= **1** (počet střižných ploch)

$F_{v,R,1}=n_v\alpha_v f_{ub}\{A,A_s\}/\gamma_{M2}$

$F_{v,Rd,1}$= 77.2 kN — **$F_{v,Rd}$= 926.2** kN

Únosnost v otlačení/střihu celého spoje

(Střih) $F_{v,R,1}$= 77.2 kN > (Otlak) $\max(F_{b,Rd,(A,B,C,D)})$= 66.9 kN =>

(Viz. ČSN EN 1993-1-8: 3.7) Celková únosnost spoje bude rovna součtu únosností jednotlivých spojovacích prostředků v otlačení.

$F_{v/b,Rd}$= 748.4 kN

Únosnost oslabeného průřezu

Plocha dle zadaného profilu

(Viz. ČSN EN 1993-1-3: 8.3, ČSN EN 1993-1-1: 6.2.3)

A_g= **2025.6** mm2

t= **5.0** mm

f_u= **550.0** Mpa

f_y= **500.0** Mpa

Celkový počet šroubů ve spoji n_b= **12**

Počet šroubů v řezu n_c= **4**

Poměr $r=n_c/n_b$ r= **0.33**

$u=\min(2e_2;p_2)$ u= **50** mm

Oslabená plocha profilu A_{net}= **1696** mm2 $A_{net}=A_g-(n_c d_0 t)$

$F_{n,Rd,1}=(1 + 3r(d_0/u - 0,3))A_{net}f_u/\gamma_{M2}$

$F_{n,Rd,1}$= **768** kN

$F_{n,Rd,2}=0,9A_{net}f_u/\gamma_{M2}$ — $F_{pl,Rd}=A_g f_y/\gamma_{M0}$

$F_{n,Rd,2}$= **671** kN — $F_{pl,Rd}$= **810** kN

$F_{n,Rd}=\min(F_{n,Rd,1};F_{n,Rd,2}; F_{pl,Rd})$

$F_{n,Rd}$= 671 kN

Vytržení skupiny šroubů

(Viz. ČSN EN 1993-1-8: 3.10.2)

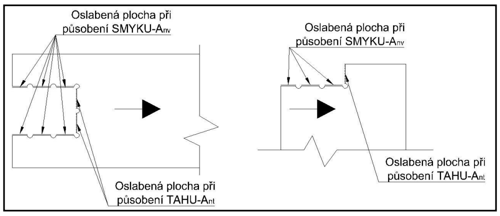
L_{nt}= **91** mm => A_{nt}= **455** mm^2

L_{nv}= **360** mm => A_{nv}= **1800** mm^2

f_u= **550.0** Mpa

f_y= **500.0** Mpa

t= **5.0** mm



Osové zatížení

$V_{eff,1,Rd}=f_u A_{nt}/\gamma_{M2} + (1/3^{0.5})\, f_y A_{nv}/\gamma_{M0}$

$V_{eff,1,Rd}$= **720** kN

Excentrické zatížení

$V_{eff,1,Rd}=0,5f_u A_{nt}/\gamma_{M2} + (1/3^{0.5})\, f_y A_{nv}/\gamma_{M0}$

$V_{eff,1,Rd}$= **620** kN

Check M, V och N acc. to EC FR1 R15

$\boxed{Country := "CZ"}$

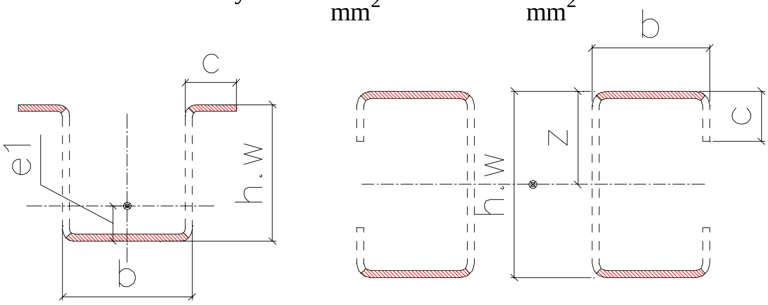
$\boxed{PR \equiv "Hx5"}$

Profile data for a single profile

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

$h_w = 120\,mm \quad b_c = 112\,mm \quad c = 43\,mm \quad t = 5\,mm \quad e_1 = 56.31\,mm \quad A_g = 2025.62\,mm^2$

$N_{cRk} = 981.63\,kN \quad M_{Rk} = 33.76\,kN \cdot m \quad f_{yb} := 90 \frac{N}{mm^2} \quad f_u := 99 \frac{N}{mm^2}$



The beam is H-profile ("H") or double C-profile ("C")

Hole diameter: $\boxed{d_o := 12.5 \cdot mm}$

Number of bolts at the cross-section in ONE web: $\boxed{n_{cross} := 3}$

Number screws and diameter i FLANGES: $\boxed{X_{h.fl} := 0}$ $\boxed{d_{o.fl} := 0 \cdot mm}$

Momentcap.one profile:

Axialforcecapacity one profile:

Axialforce acting as TENTION : "YES" or "NO" $\boxed{N_{drag} := "YES"}$

Netto area for shearforce:

$$A_w := \begin{cases} \left[\left[(h_w - t) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} = 0 \cdot mm \\ \left[\left[(L_{II} - t) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} \neq 0 \cdot mm \end{cases} \qquad A_w = 775.00\,mm^2$$

Nettoarea for profiles in tention:

$$A_{net} := A_g - t \cdot \left(n_{cross} \cdot d_o \cdot \frac{2}{X} + X_{h.fl} \cdot d_{o.fl}\right)$$

Moment capacity profile:

$$M_{Rd} := \frac{X \cdot M_{Rk}}{\gamma_{M0}} \qquad M_{Rd} = 33.76\,kN \cdot m$$

Axialforce capacity compresion member:

$$N_{Rd.1} := \frac{N_{cRk} \cdot X}{\gamma_{M0}} \quad (6.2.4) \qquad N_{Rd.1} = 981.6\,kN$$

Capacity for axial tension:

For material <= 4.0mm :

$$N_{Rd.t.a} := \min\left(\frac{f_{ya} \cdot A_g}{\gamma_{M0}}, \frac{A_{net} \cdot f_u}{\gamma_{M2}}\right)$$

(EN 1993-1-3: 6.1.2 (6.1) and EN 1993-1-3: 8 (table 8.4)) $N_{Rd.t.a} = 130.73\,kN$

For material >= 5.0mm :

$$N_{Rd.t.b} := \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2.t}} \qquad N_{Rd.t.b} = 117.7\,kN \quad \text{(EN1993-1-1: 6.2.3)} \qquad \gamma_{M2.t} = 1.25$$

$$N_{Rd.t} := \left(\begin{cases} N_{Rd.t.a} & \text{if } t \leq 4 \cdot mm \\ N_{Rd.t.b} & \text{if } t \geq 5 \cdot mm \end{cases}\right) \cdot X \qquad N_{Rd.t} = 117.66\,kN$$

Normalkraftscapacity:

$$N_{Rd} := \begin{cases} N_{Rd.1} & \text{if } N_{drag} = "NO" \\ \min(N_{Rd.1}, N_{Rd.t}) & \text{if } N_{drag} = "YES" \end{cases} \qquad N_{Rd} = 117.66\,kN$$

Shearforce capacity:

$$\lambda_{wh} := 0.346 \cdot \frac{h_w - t}{t} \cdot \sqrt{\frac{f_{yb}}{E}} \qquad \lambda_{wh} = 0.16$$

$$f_{vb} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{wh} \leq 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{wh}} & \text{if } 0.83 < \lambda_{wh} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{wh}^2} & \text{if } \lambda_{wh} \geq 1.40 \end{cases} \qquad f_{vb} = 51.96 \frac{N}{mm^2}$$

$$V_{Rd} := \frac{A_w \cdot f_{vb}}{\gamma_{M0}} \qquad V_{Rd} = 40.27\,kN$$

elast. momentcap.: $M_{elRd} := M_{Rd}$

$\theta_{pl} := 1.0$ faktor M_{el} -> M_{pl}

Section value for interaction formula M + N + V:
Momentresistance of a cross-section consisting only flanges: $M_{flRd} = 4.57\,kN \cdot m$

Redukční součinitele pro ocelové prvky

Profil 2C360X5

Čas t = **15** **min.**

Spočtená teplota T_{ep} = **626.994 °C**

Pozinkovaný: PRAVDA

Ohřev ze 3 stran: NEPRAVDA

Uvažovat zastínění: PRAVDA

$k_{y\Theta}$ red. součinitel na mez kluzu

$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu

$k_{E\Theta}$ red. součinitel pro linerní pružnou část

$k_{b\Theta}$ red. součinitel pro šrouobvé spoje

$k_{w\Theta}$ red. součinitel pro svary

Součinitelé pro průřezy 1-3			
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
20	1	1	1
100	1	1	1
200	1	0.807	0.9
300	1	0.613	0.8
400	1	0.42	0.7
500	0.78	0.36	0.6
600	0.47	0.18	0.31
700	0.23	0.075	0.13
800	0.11	0.05	0.09
Teplota nosníku			
627.0	**0.405**	**0.152**	**0.261**

Souč. pro 4. tř. za studena		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$
20	1	1
100	1	1
200	0.89	0.9
300	0.78	0.8
400	0.65	0.7
500	0.53	0.6
600	0.3	0.31
700	0.13	0.13
800	0.07	0.09
Teplota nosníku		
627.0	**0.254**	**0.261**

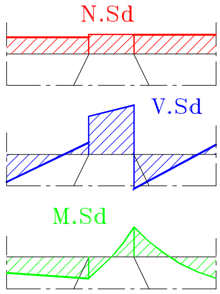
Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$
20	1	1
100	0.968	1
150	0.952	1
200	0.935	1
300	0.903	1
400	0.775	0.876
500	0.55	0.627
600	0.22	0.378
700	0.1	0.13
800	0.067	0.074
Teplota nosníku		
627.0	**0.188**	**0.311**

Element Nr: dolní pás

$N_{Sd} := 30 \cdot kN$ $V_{Sd} := 28 \cdot kN$ $M_{Sd} := -3 \cdot kN \cdot m$

$\frac{|V_{Sd}|}{V_{Rd}} = 0.7$ <1.0 (EN 1993-1-1: 6.2.6) $\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 0.34$ <1.0 (EN 1993-1-3 6.1.8+6.1.9)

Ratio for check
Moment + Axialforce + Shear force $M\_N\_V_{EC} = 0.48$ <1.0 (EN 1993-1-3 6.1.10)

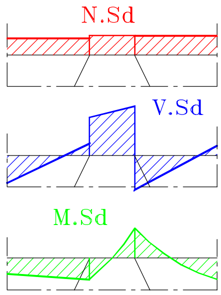


Element Nr: horní pás

$N_{Sd} := 29 \cdot kN$ $V_{Sd} := 23 \cdot kN$ $M_{Sd} := -2 kN \cdot m$

$\frac{|V_{Sd}|}{V_{Rd}} = 0.57$ <1.0 (EN 1993-1-1: 6.2.6) $\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 0.31$ <1.0 (EN 1993-1-3 6.1.8+6.1.9)

Ratio for check
Moment + Axialforce + Shear force $M\_N\_V_{EC} = 0.32$ <1.0 (EN 1993-1-3 6.1.10)



Frame Column FR1 R15

$h \equiv 360 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 39 \cdot mm$ $t \equiv 5 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $cc_{batt} := 1500 \cdot mm$ distance between battens $h_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $kb = 599.63\ mm$ $\gamma_{M1} \equiv 1.0$

$b_{batt} := 200 \cdot mm$

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 6.051 \times 10^3\ mm^2$

$I_{zz} := 2 \cdot \left[I_z + A_g \cdot \left(e_1 + \frac{d}{2} \right)^2 \right]$ $I_{zz} = 4.45 \times 10^7\ mm^4$

$I_{yy} := 2 \cdot I_y$ $I_{yy} = 1.12 \times 10^8\ mm^4$ ingle C:

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 6.29 \times 10^5\ mm^3$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 6.12 \times 10^5\ mm^3$

$W_{zz} := \frac{I_{zz}}{b + \frac{d}{2}}$ $W_{zz} = 2.97 \times 10^5\ mm^3$

$I_{TT} = 51500\ mm^4$

$M_{yy.cRk} := 2 \cdot M_{ycRk}$

$M_{yy.cRk} = 77.69\ kN \cdot m$

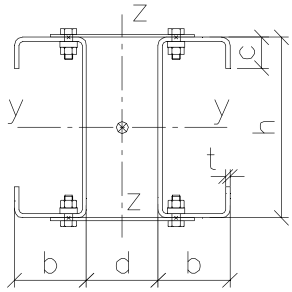
$M_{ycRk} = 38.84\ kN \cdot m$

$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$N_{cc.Rk} = 668.01\ kN$

$f_{yb} = 127 \frac{N}{mm^2}$

$f_u = 139.7 \frac{N}{mm^2}$



$i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 135.84\ mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 85.77\ mm$

torsion_plate_pillar = "NO"

forces from:

$Loadcase := 30$

Stresses and buckling length according to first order frame analys

$M_{Ed.1} := 25 \cdot kN \cdot m$ max moment in pillar

The moment is distributed according to frame modelling $Moment\_dis :=$ "A"

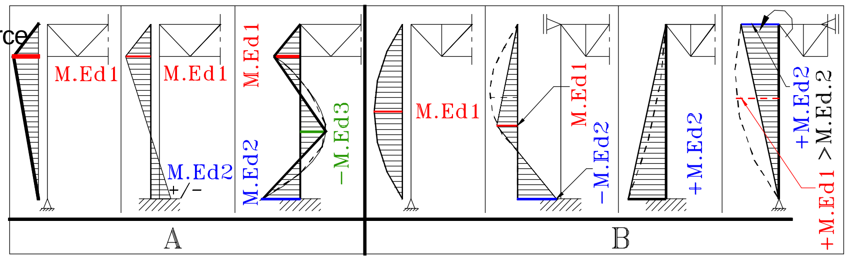
$M_{Ed.1.z} := 0 \cdot kN \cdot m$ moment about z-z

$N_{Ed} := 75 \cdot kN$ axial force in pillar = reaction force

$N_{Ed.1} := 77 \cdot kN$ axial force in element between top and bottom beam

$N_{Ed.2} := 35 \cdot kN$ reaction force in second pillar

$V_{Ed} := 70 \cdot kN$ --> $n_{cross} := 4$ =Numbers of holes in **ONE** web



Global geometry. $d_0 := 12.5 \cdot mm$

$M_{Ed.2} := -20 \cdot kN \cdot m$ $M_{Ed.3} := 0 \cdot kN \cdot m$ $M_{Ed.2.z} := 0 \cdot kN \cdot m$

$L := 5.8 \cdot m$ length pillar

$L_z := 6.2 \cdot m$ length pillar for buckling z-z

$B_{fr} := 14.56 \cdot m$ width frame

$H_{tr} := 975 \cdot mm$ height truss

$A_{u.chord} := 1616 \cdot mm^2$ Area upper chord

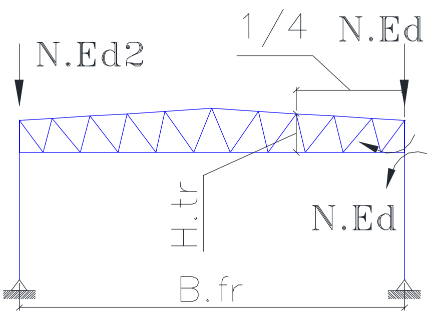
$I_{u.chord} := 3.29 \cdot 10^6 \cdot mm^4$ I upper chord

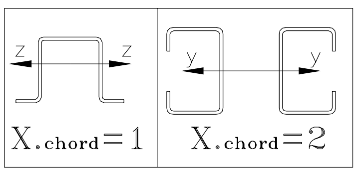
$X_{u.chord} := 1$ number of profiles upper chord

$A_{lo.chord} := 1616 \cdot mm^2$ Area lower chord

$I_{lo.chord} := 3.29 \cdot 10^6 \cdot mm^4$ I lower chord

$X_{lo.chord} := 1$ number of profiles lower chord





Buckling lengths:

The global buckling length in y-y is calculated accounting for the stiffness behavior of the frame. The method from "Stahlbau in Beispielen, Berechnungspraxis nach DIN 18800-1 bis 3", Hünersen, Fritzsche is uesd.

$L_y := 2 \cdot L$ $L_y = 11.6\ m$ exural buckling axis y-y: according to modelled system

$L_z = 6.2\ m$ Flexural buckling axis z-z $L_y = 11.6\ m$

$\boxed{L_{LT} := 1.0 \cdot L}$ $L_{LT} = 5.8\,m$ Distance flange bracings - LTB inner flange in compression

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 127\,\frac{N}{mm^2}$ $f_u = 140\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Flexural buckling resistance - axis y-y and z-z EN 1993-1-3: 6.2.2 and EN 1993-1-1: 6.3.1:

Buckling curve for double C-section: EN 1993-1-3 table 6.3 about:

y-y:	a	EN1993-1-1,table 6.1: α =0,21
z-z:	b	EN1993-1-1,table 6.1: α =0,34

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about y-y

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 65.26$ $\lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.y.FBcc} = 1.22$

Imperfection factor α relating to bucking curve a $\alpha_y := 0.21$

$\phi_{y.FB} := 0.5 \cdot \left[1 + \alpha_y \cdot (\lambda_{r.y.FBcc} - 0.2) + \lambda_{r.y.FBcc}^2\right]$ $\phi_{y.FB} = 1.35$

$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^2 - \lambda_{r.y.FBcc}^2}}, 1\right)$ $\chi_{y.FBcc} = 0.52$

Design buckling resistance for buckling mode -flexural buckling-: y-y

$N_{b.Rd.y.FBcc} := \frac{\chi_{y.FBcc} \cdot 2 \cdot A_{eff} \cdot f_{yb}}{\gamma_{M1}}$ $N_{b.Rd.y.FBcc} = 345.7\,kN$

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about z-z

$\lambda_1 = 65.26$ $\lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.z.FBcc} = 1.03$

Imperfection factor α relating to bucking curve b $\alpha_z := 0.34$

$\phi_{z.FB} := 0.5 \cdot \left[1 + \alpha_z \cdot (\lambda_{r.z.FBcc} - 0.2) + \lambda_{r.z.FBcc}^2\right]$ $\phi_{z.FB} = 1.17$

$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^2 - \lambda_{r.z.FBcc}^2}}, 1\right)$ $\chi_{z.FBcc} = 0.58$

Design buckling resistance for buckling mode -flexural buckling-: z-z

$N_{b.Rd.z.FBcc} := \frac{\chi_{z.FBcc} \cdot 2 \cdot A_{eff} \cdot f_{yb}}{\gamma_{M1}}$ $N_{b.Rd.z.FBcc} = 385.1\,kN$

Lateral-torsional buckling resistance for uniform member in bending EN 1993-1-3: 6.2.4 and EN 1993-1-1: 6.3.2:

Procedure to calculate the elastic critical buckling moment M_{cr}, based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints, is given neither in EN 1993-1-1 nor in EN 1993-1-3. Any appropriate calculation method can be used, here the procedure given in German ENV 1993-1-1 Annex F.

Moment_dis = "A" $L_{LT} = 5.8\,m$ $k_{M.cr} := 1.0$ (hinged at ends) $k_{w.M.cr} := 1.0$ (no special wrap restraints at ends)

For frame pillar with fixed both upper and lower beam (end moments): according to picture "A":

$$\psi_{md.A} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \geq |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}$$

$\psi_{md.A} = -0.8$

$C_{1.A} := \min\left(1.88 - 1.40 \cdot \psi_{md.A} + 0.52 \cdot \psi_{md.A}^2, 2.7\right)$

$C_{1.A} = 2.7$

$$M_{cr.A} := C_{1.A} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{(k_{M.cr} \cdot L_{LT})^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr} \cdot L_{LT})^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}}\right]^{0.5}$$

$M_{cr.A} = 156.78\,kN \cdot m$

For framepillar with moment distribution according to picture "B":

$C_{1.B} := 1.132$

$$M_{cr.B} := C_{1.B} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{(k_{M.cr} \cdot L_{LT})^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr} \cdot L_{LT})^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}}\right]^{0.5}$$

$M_{cr.B} = 65.73\,kN \cdot m$

Elastic critical moment for lateral-torsional buckling

based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints for double C-section:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if Moment\_dis} = \text{"A"} \\ M_{cr.B} & \text{if Moment\_dis} = \text{"B"} \end{cases}$$

$M_{cr} = 157\,kN \cdot m$

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 127\,\frac{N}{mm^2}$ $f_u = 140\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

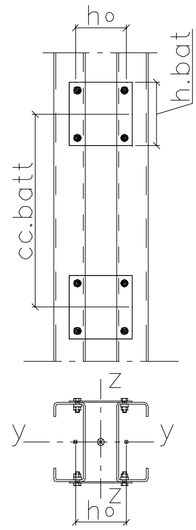
Relative slenderness: $\lambda_{rLT} := \sqrt{\frac{2 \cdot W_{eff.y.1} \cdot f_{yb}}{M_{cr}}}$ $\lambda_{rLT} = 0.7$ $\alpha_{LT} := 0.34$ $\phi_{LT} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot \left(\lambda_{rLT} - 0.2\right) + \lambda_{rLT}^2\right]$ $\phi_{LT} = 0.83$

$\chi_{LT.y} := \min\left(\frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{rLT}^2}}, 1\right)$ $\chi_{LT.y} = 0.78$ $\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{yy} \geq I_{zz} \\ 1 & \text{otherwise} \end{cases}$ Reduction buckling factor: $\chi_{LT} = 0.78$

Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

Desing buckling moment resistance double profile based on effective section modulus W_{eff}. $M_{b.Rd} := \frac{\chi_{LT} \cdot M_{yy.cRk}}{\gamma_{M1}}$ $M_{b.Rd} = 60.71\,kN \cdot m$

Check Uniform built-up member EN 1993-1-1: 6.4



bow imperfection: $e_0 := \frac{L}{500}$ $\text{if}\left(\frac{cc_{batt}}{i_z} \leq 70, \text{"OK"}, \text{"Shorter-cc.bat"}\right) = \text{"OK"}$

non-dimensional slenderness: $\lambda_1 = 65.26$

Effective second moment of area of battened built-up member: $A_{ch} := A_g$: area of one chord

$h_0 := d + 2 \cdot e_1$ $h_0 = 155.7\,mm$ distance centroids of chords $I_{ch} := I_z$: I of one chord

$I_{l.CC} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot I_{ch}$ I built-up member $i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2 \cdot A_{ch}}}$ $\lambda_{CC} := \frac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 72.28$

EN 1993-1-1: table 6.8 Efficiency factor: $\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \geq 150 \\ \left(2 - \frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \leq 75 \end{cases}$ $\mu_{CC} = 1$

$I_{l.CC} = 4.45 \times 10^7\,mm^4$ $r_{CC} := 2$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu_{CC} \cdot I_{ch}$ $I_{eff} = 4.45 \times 10^7\,mm^4$ effective I of built-up member

Shear stiffness EN 1993-1-1:6.4.3.1:(2) $I_b := \frac{t_{batt} \cdot h_{batt}^3}{12}$ I of batten $n_{batt} := 2$ number of planes of lacings

$S_v := \min\left[\left[\frac{24 \cdot E \cdot I_{ch}}{cc_{batt}^2 \cdot \left(1 + \frac{2 \cdot I_{ch}}{n_{batt} \cdot I_b} \cdot \frac{h_0}{cc_{batt}}\right)}\right], \left(\frac{2 \cdot \pi^2 \cdot E \cdot I_{ch}}{cc_{batt}^2}\right)\right]$ $S_v = 1883\,kN$

effective critical force of built-up member: $N_{cr.CC} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_z^2}$ $N_{cr.CC} = 626.5\,kN$

Maximum moment in middle of built-up member: EN 1993-1-1:6.4.1 (6):

$M_{z.Ed.I} := \max\left(\left|M_{Ed.1.z}\right|, \left|M_{Ed.2.z}\right|\right)$ moment with second order effects $M_{z.Ed.C} := \frac{N_{Ed} \cdot e_0 + M_{z.Ed.I}}{1 - \frac{N_{Ed} \cdot \gamma_{M1}}{N_{cr.CC}} - \frac{N_{Ed} \cdot \gamma_{M1}}{S_v}}$ $M_{z.Ed.C} = 1.04\,kN \cdot m$

moment z-z without second order effects

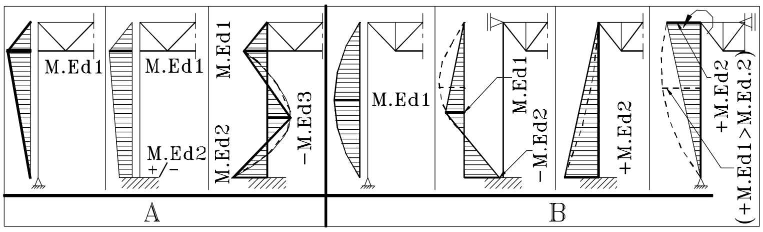
Compression force in one member: EN 1993-1-1:6.4.1 (6) Compression force in built-up member taking accout to second order effects:

$X_d := \begin{cases} 1 & \text{if } d \geq 8 \cdot mm \\ 0 & \text{otherwise} \end{cases}$ $N_{ch.Ed} := \frac{N_{Ed}}{r_{CC}} + \frac{\left|M_{z.Ed.C}\right| \cdot h_0 \cdot A_{ch} \cdot X_d}{2 \cdot I_{eff}}$ $N_{ch.Ed} = 42.98\,kN$

$b = 100\,\mathrm{mm}$ $h = 360\,\mathrm{mm}$ $c = 39\,\mathrm{mm}$ $t = 5\,\mathrm{mm}$ $f_{yb} = 127\,\frac{N}{mm^2}$ $f_u = 140\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):
The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3
Moment distribution like shown on figure: $Moment\_dis = \text{"A"}$



$\psi_{md.A} = -0.8$ $\psi_{m.B} = 0$ $\psi_{md.z} = 0$

$\alpha_{h.2} = -0.$ $\alpha_{h.2.z} = 0$ $\alpha_{s.2} = 1$

$C_{my.2.A} := 0.9$ $C_{my.2.B} = 1$

$C_{my.2} = 0.9$ $C_{mz.2} = 0.95$

$C_{mLT.2} = 0.4$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$$n_{y.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} \qquad n_{y.2} = 0.22 \qquad n_{z.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} \qquad n_{z.2} = 0.19$$

$$k_{yy.2} := \min\left[C_{my.2}\cdot\left(1 + 0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}\right), C_{my.2}\cdot\left(1 + 0.6\cdot n_{y.2}\right)\right] \qquad k_{yy.2} = 1.02$$

$$k_{zy.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FBcc}}{C_{mLT.2} - 0.25}\cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25}\cdot n_{z.2}\right) \qquad k_{zy.2} = 0.94$$

$$k_{zz.2} := \min\left[C_{mz.2}\cdot\left(1 + 0.6\cdot\lambda_{r.z.FBcc}\cdot n_{z.2}\right), C_{mz.2}\cdot\left(1 + 0.6\cdot n_{z.2}\right)\right] \qquad k_{zz.2} = 1.06 \qquad k_{yz.2} := k_{zz.2}$$

Reduction factor for L-T buckling: $\chi_{LT} = 0.78$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.52$ $\chi_{z.FBcc} = 0.58$

$$I_{zz.eff} := 2\cdot I_z + A_g\cdot\left(e_1 + \frac{d}{2}\right)^2 + A_{eff}\cdot\left(e_N + \frac{d}{2}\right)^2 \qquad W_{zz.eff} := \frac{I_{zz.eff}}{b + \frac{d}{2}} \qquad I_{zz.eff} = 4.13\times 10^7\,\mathrm{mm}^4 \qquad M_{zz.cRk} := W_{zz.eff}\cdot f_{yb}$$

$$W_{zz.eff} = 2.76\times 10^5\,\mathrm{mm}^3$$

Double CC-profile: Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{\max\left(N_{Ed}, N_{Ed.1}\right)\cdot\gamma_{M0}}{2\cdot N_{c.Rk}} + \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right)\cdot\gamma_{M0}}{2\cdot M_{ycRk}} + \frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M0}}{M_{zz.cRk}} = 0.47$$ <1.0

Double CC-profile: Combined M + N + V acc to EN 1993-1-3: 6.1.10

$$\frac{V_{Ed}\cdot\gamma_{M0}}{2\cdot V_{bh.Rk.pillar}} = 0.45$$ < 1.0 according to EN 1993-1-3: 6.1.10 $M\_N\_V_{EC} = 0$ <1.0

Double CC-profile: Combined bending an axial compression EN 1993-1-1: 6.3.3 (4):
Evading in y-y:

$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} + k_{yy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{yz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.67$$ <1.0

Evading in z-z: $\psi_{c.N} := \frac{N_{Ed}}{N_{Ed.1}}$ $k_{c.N} := \frac{1}{1.33 - 0.33\cdot\psi_{c.N}}$ $k_{c.N} = 0.99$ EN 1993-1-1 Tab. 6.6

$$\frac{\max\left(N_{Ed}, N_{Ed.1}\right)\cdot k_{c.N}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} + k_{zy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{zz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.61$$ <1.0

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 127\,\frac{N}{mm^2}$ $f_u = 140\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords and battens are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1).

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := \max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.3}|)\cdot 0.5$ $M_{y.max.C} = 12.5\,kN\cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25\cdot cc_{batt}$ $M_{y.AA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = 8.14\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5\cdot cc_{batt}$ $M_{y.BA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = 9.59\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75\cdot cc_{batt}$ $M_{y.CA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = 11.05\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot M_{y.max.C}}{2.5\cdot M_{y.max.C} + 3\cdot|M_{y.AA}| + 4\cdot|M_{y.BA}| + 3\cdot|M_{y.CA}|}$$

$y_{0.cr} := (y_M + e_{1c.rc})\cdot -1$ $z_{0.cr} := 0\cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot cc_{batt}}{i_z}\right)^2} \qquad \sigma_{ez} = 311.17\,\frac{N}{mm^2}$$

$$\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{cc_{batt}^2}\right) \qquad \sigma_T = 341.14\,\frac{N}{mm^2}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_A} := C_{bA}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 189.4\,kN\cdot m$

Relative slenderness:
$$\lambda_{rLT.C\_A} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0\\ 0.2 & \text{if } M_{e.cr.C\_A} = 0\end{cases}$$

$\lambda_{rLT.C\_A} = 0.45$ Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right] \quad \chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right) \quad \chi_{LT.C\_A} = 0.9$$

$\phi_{LT.C\_A} = 0.65$

For moment distribution accord. to modell B: $q_{wind\_pillar} := \frac{|M_{Ed.1}\cdot 0.5|\cdot 8}{L^2}$ $M_{y.i}(x_i) := \frac{q_{wind\_pillar}}{2}\cdot(L\cdot x_i - x_i^2)$

Moment at quater point of unbraced segment: $x_{1\_4} := \frac{L - cc_{batt}}{2} + \frac{1}{4}\cdot cc_{batt}$ $x_{1\_4} = 2.52\,m$ $M_{y.i}(x_{1\_4}) = 12.29\,kN\cdot m$ $M_{y.AB} := M_{y.i}(x_{1\_4})$ $M_{y.AB} = 12.29\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 2.9\,m$ $M_{y.i}(x_{1\_2}) = 12.5\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 12.5\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := \frac{L - cc_{batt}}{2} + \frac{3}{4}\cdot cc_{batt}$ $x_{3\_4} = 3.27\,m$ $M_{y.i}(x_{3\_4}) = 12.29\,kN\cdot m$ $M_{y.CB} := M_{y.i}(x_{3\_4})$ $M_{y.CB} = 12.29\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot(|M_{Ed.1}\cdot 0.5|)}{2.5\cdot(|M_{Ed.1}\cdot 0.5|) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := C_{bB}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 155.4\,kN\cdot m$

Relative slenderness:
$$\lambda_{rLT.C\_B} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0\\ 0.2 & \text{if } M_{e.cr.C\_B} = 0\end{cases}$$

$\phi_{LT.C\_B} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_B} - 0.2) + \lambda_{rLT.C\_B}^2\right]$

$\lambda_{rLT.C\_B} = 0.5$ $\phi_{LT.C\_B} = 0.68$

$$\chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right) \quad \chi_{LT.C\_B} = 0.88$$

Moment distribution: $Moment\_dis = "A"$
$$\chi_{LT.C} := \begin{cases}\chi_{LT.C\_A} & \text{if } Moment\_dis = "A"\\ \chi_{LT.C\_B} & \text{if } Moment\_dis = "B"\end{cases} \qquad \chi_{LT.C} = 0.9$$

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 127\,\frac{N}{mm^2}$ $f_u = 140\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Stresses on one member profile at mid-span from above: chord force from above: $N_{ch.Ed} = 42.98\,kN$ *max moment y-y:* $M_{y.max.C} = 12.5\,kN\cdot m$

Chord at end panel: highest shear force: $V_{Ed.max} := \pi\cdot\frac{M_{z.Ed.C}}{L}$ $V_{Ed.max} = 0.56\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 0.28\,kN$

$V_{bb.Rd} = 69.66\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.004$ << 1,0 The shear force is negligible

Chord at end panel: "corner" moment: $M_{z.ch} := \frac{V_{Ed.max}}{r_{CC}}\cdot\frac{cc_{batt}}{2}$ *maximum moment z-z:* $M_{z.ch} = 0.21\,kN\cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C}\cdot\sin\left(\frac{\pi\cdot cc_{batt}}{L}\right)$ $M_{z.Ed.C.end} = 0.75\,kN\cdot m$ moment due to bow imperfection at end of panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end}\cdot\frac{(h_0\cdot A_{ch})}{(2\cdot I_{eff})}$ $N_{ch.end} = 41.48\,kN$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$n_{y.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{y.C.2} = 0.12$ $n_{z.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{z.C.2} = 0.15$ $\chi_{y.FB}(cc_{batt}) = 1$ $\chi_{z.FB}(cc_{batt}) = 0.84$

Equivalent uniform moment factors: Moment_dis = "A"

Moment at start of unbraced segment: $x_{0\_0} := 0\cdot cc_{batt}$ $M_{y.A0} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{0\_0})$ $M_{y.A0} = 6.68\,kN\cdot m$

Moment at end of unbraced segment: $M_{y.max.C} = 12.5\,kN\cdot m$

$$\psi_{m.C.A} := \frac{M_{y.A0}}{\begin{cases} M_{Ed.1}\cdot 0.5 & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ M_{Ed.2}\cdot 0.5 & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}}$$ $\psi_{m.C.A} = 0.53$ range for model "A"

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 2.9\,m$ $M_{y.i}(x_{1\_2}) = 12.5\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 12.5\,kN\cdot m$

Moment at start of unbraced segment: $x_{0\_0} := \frac{L - cc_{batt}}{2} + 0\cdot cc_{batt}$ $x_{0\_0} = 2.15\,m$ $M_{y.i}(x_{0\_0}) = 11.66\,kN\cdot m$ $M_{y.B0} := M_{y.i}(x_{0\_0})$ $M_{y.B0} = 11.66\,kN\cdot m$

$\alpha_{h.C.2} := \frac{|M_{y.B0}|}{|M_{y.BB}|}$ $\alpha_{h.C.2} = 0.93$ $\psi_{m.C.B} := 1$ range for model "B"

$$C_{mLT.C.2} := \begin{cases} \max(0.6 + 0.4\cdot\psi_{m.C.A}, 0.4) & \text{if Moment\_dis = "A"} \\ (0.95 + 0.05\cdot\alpha_{h.C.2}) & \text{if Moment\_dis = "B"} \end{cases}$$ $C_{mLT.C.2} = 0.814$ $C_{my.C.2} := C_{mLT.C.2}$ $C_{mz.C.2} := 1$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$\lambda_{r.y.FB}(cc_{batt}) = 0.16$ $k_{yy.C.2} := \min[C_{my.C.2}\cdot(1 + 0.6\cdot\lambda_{r.y.FB}(cc_{batt})\cdot n_{y.C.2}), C_{my.C.2}\cdot(1 + 0.6\cdot n_{y.C.2})]$ $k_{yy.C.2} = 0.82$

$\lambda_{r.z.FB}(cc_{batt}) = 0.6$ $k_{zz.C.2} := \min[C_{mz.C.2}\cdot(1 + 0.6\cdot\lambda_{r.z.FB}(cc_{batt})\cdot n_{z.C.2}), C_{mz.C.2}\cdot(1 + 0.6\cdot n_{z.C.2})]$ $k_{zz.C.2} = 1.05$

$$k_{zy.C.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FB}(cc_{batt})}{C_{my.C.2} - 0.25}\cdot n_{z.C.2}, 1 - \frac{0.05}{C_{mLT.C.2} - 0.25}\cdot n_{z.C.2}\right)$$ $k_{zy.C.2} = 0.99$ $k_{yz.C.2} := k_{zz.C.2}$

$b = 100\,mm$ $h = 360\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 127 \frac{N}{mm^2}$ $f_u = 140 \frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Controll single C-profile between battens:

$M_{y.max.C} = 12.5\,kN \cdot m$ $M_{z.ch} = 0.21\,kN \cdot m$ $\Delta M_{z.shift} := \left|\Delta e_N \cdot N_{ch.end}\right|$ $\Delta M_{z.shift} = 0.02\,kN \cdot m$ $N_{ch.end} = 41.48\,kN$

Buckling moment resistance y-y
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C} \cdot M_{ycRk}$ $\chi_{LT.C} = 0.9$ $M_{y.Rk.C} = 35.14\,kN \cdot m$

Buckling resistance moment z-z
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk})$ $M_{z.Rk.C} = 7.09\,kN \cdot m$

Resistance for one profile buckling mode
flexural buckling-: y-y $cc_{batt} = 1.5\,m$ $\chi_{y.FB}(cc_{batt}) = 1$ $N_{b.Rk.y.FB}(cc_{batt}) = 334.01\,kN$

Resistance for one profile buckling mode
flexural buckling-: z-z $cc_{batt} = 1.5\,m$ $\chi_{z.FB}(cc_{batt}) = 0.84$ $N_{b.Rk.z.FB}(cc_{batt}) = 280.31\,kN$

Resistance for one profile buckling mode
-torsional or torsional-flexural $cc_{batt} = 1.5\,m$ $\chi_{TF}(cc_{batt}, cc_{batt}) = 0.85$ $N_{b.Rk.TF}(cc_{batt}, cc_{batt}) = 284\,kN$

single C-profile: Axial compression EN 1993-1-1: 6.3.1 (3): Buckling about relevant axis in mid-span or en panel of built-up member

$$\frac{\max(N_{ch.Ed}, N_{ch.end}) \cdot \gamma_{M1}}{\min(\chi_{y.FB}(cc_{batt}), \chi_{z.FB}(cc_{batt}), \chi_{TF}(cc_{batt}, cc_{batt})) \cdot N_{c.Rk}} = 0.15 \quad <1.0$$

Controll buil-up member: single C-profile (chords) in mid-span of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.Ed} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{(\left|\Delta M_{z.shift}\right|) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.45 \quad <1.0$$

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{y.FB}(cc_{batt}) \cdot N_{c.Rk}} + k_{yy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C.2} \cdot \frac{(\left|\Delta M_{z.shift}\right|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.42 \quad <1.0$$

Evading in z-z:

$$\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{z.FB}(cc_{batt}) \cdot N_{c.Rk}} + k_{zy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C.2} \cdot \frac{(\left|\Delta M_{z.shift}\right|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.51 \quad <1.0$$

Controll buil-up member: single C-profile (chords) at end panel of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.end} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{(M_{z.ch} + \left|\Delta M_{z.shift}\right|) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.48 \quad <1.0$$

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(cc_{batt}) \cdot N_{c.Rk}} + k_{yy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C.2} \cdot \frac{(M_{z.ch} + \left|\Delta M_{z.shift}\right|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.45 \quad <1.0$$

Evading in z-z:

$$\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(cc_{batt}) \cdot N_{c.Rk}} + k_{zy.C.2} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C.2} \cdot \frac{(M_{z.ch} + \left|\Delta M_{z.shift}\right|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.54 \quad <1.0$$

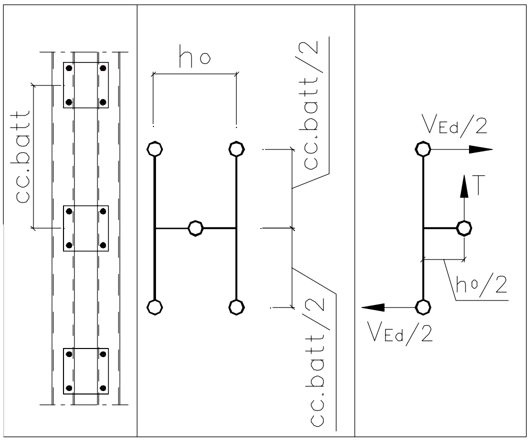
$b = 100\,mm \qquad h = 360\,mm \qquad c = 39\,mm \qquad t = 5\,mm \qquad f_{yb} = 127\,\frac{N}{mm^2} \qquad f_u = 140\,\frac{N}{mm^2} \qquad \gamma_{M0} = 1.00 \qquad \gamma_{M1} = 1.00$

Forces on battens: max shear flow in one batten:

$$T_{batt} := \frac{V_{Ed.max} \cdot cc_{batt}}{h_0 \cdot r_{CC}} \qquad T_{batt} = 2.7\,kN$$

max moment in one batten:

$$M_{batt} := \frac{T_{batt} \cdot h_0}{2} \qquad M_{batt} = 0.21\,kN \cdot m$$

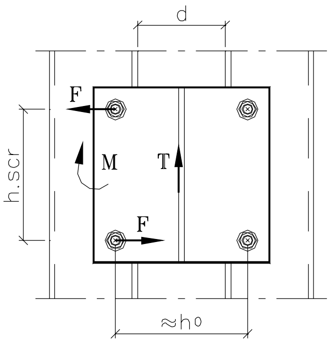


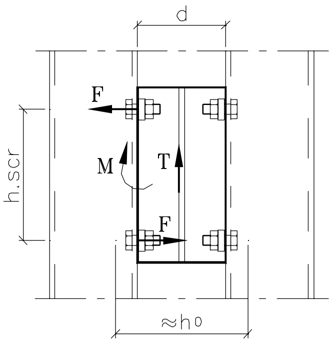
$$\lambda_{w.batt} := 0.346 \cdot \frac{h_{batt}}{t_{batt}} \cdot \sqrt{\frac{f_{yb}}{E}} \qquad f_{vb.batt} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{w.batt} \le 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{w.batt}} & \text{if } 0.83 < \lambda_{w.batt} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{w.batt}^2} & \text{if } \lambda_{w.batt} \ge 1.40 \end{cases}$$

$$V_{bbatt.Rd} := \frac{h_{batt} \cdot t_{batt} \cdot f_{vb.batt}}{\gamma_{M0}} \qquad V_{bbatt.Rd} = 58.56\,kN \qquad \frac{T_{batt}}{V_{bbatt.Rd}} = 0.05 \quad < 1{,}0$$

$$M_{Rd.batt} := \frac{f_{yb}}{\gamma_{M0}} \cdot \frac{t_{batt} \cdot h_{batt}^2}{6} \qquad M_{Rd.batt} = 3.39\,kN \cdot m \qquad \frac{M_{batt}}{M_{Rd.batt}} = 0.06 \quad < 1{,}0$$

Forces on screws:





$h_{scr} := 150 \cdot mm \qquad n_c := 2$ numbers of screws on one side of one PXK

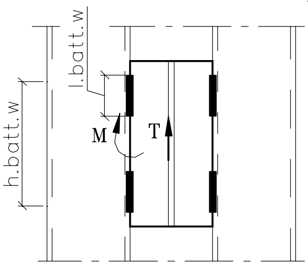
$$T_{batt} = 2.7\,kN \qquad F := \frac{T_{batt} \cdot h_0}{2 \cdot h_{scr}} \qquad F = 1.4\,kN$$

$$F_{scew} := \sqrt{F^2 + \left(\frac{T_{batt}}{n_c}\right)^2}$$

$F_{scew} = 1.95\,kN$ = force on one screw

< 30 kN

Forces on weld: welding high: $h_{batt.w} := 150 \cdot mm$ welding length: $l_{batt.w} := 2 \cdot 40 \cdot mm$ welding t=: $a_{batt.w} := 4 \cdot mm$



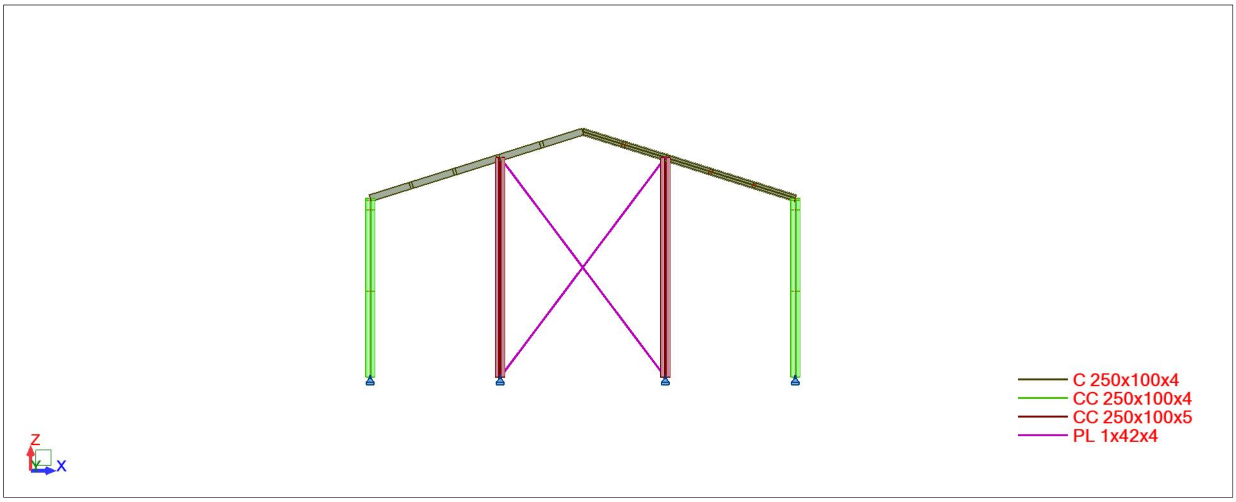
$$\sigma_{batt.w} := \frac{3 \cdot T_{batt} \cdot h_0}{a_{batt.w} \cdot h_{batt.w}^2} \qquad \tau_{batt.w} := \frac{T_{batt}}{l_{batt.w} \cdot a_{batt.w}}$$

$$\sigma_{V.batt.w} := \sqrt{\sigma_{batt.w}^2 + 3 \cdot \tau_{batt.w}^2} \qquad \sigma_{V.batt.w} = 20.25\,\frac{N}{mm^2}$$ = stress in one weld

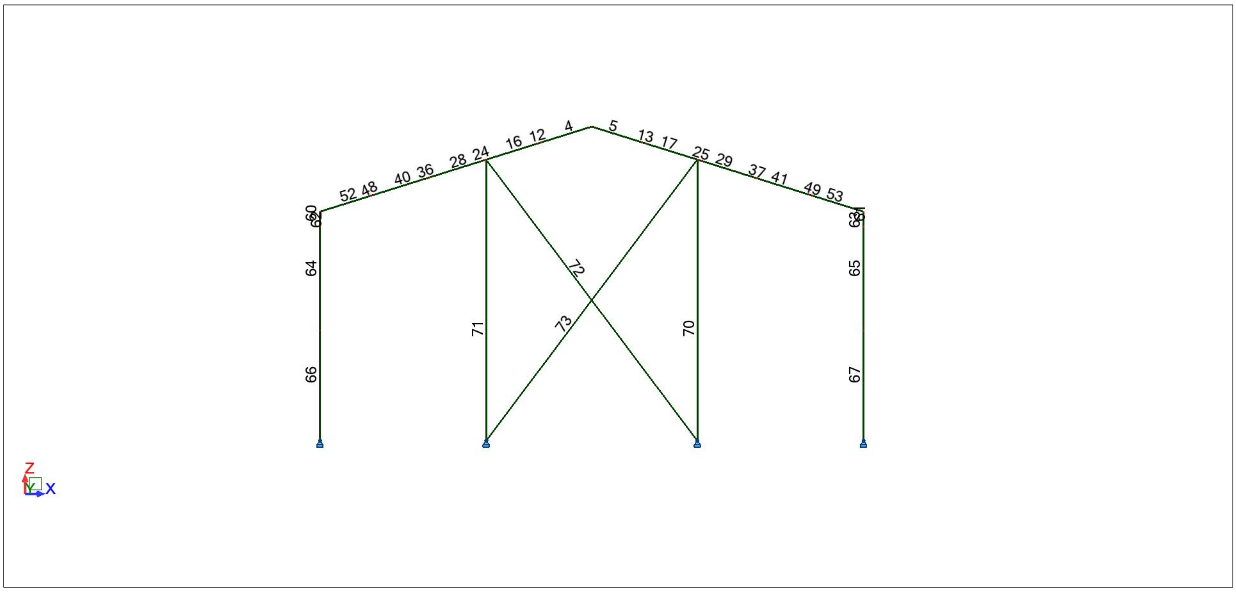
Část konstrukce: GA1 Štít vnější R15

Poznámky:
Zatěžovací šířka štítu Lw=2m.
Štít je posouzen na zatížení při požáru R15.

View - Cases: 101 (Self weight frame)



View - Cases: 101 (Self weight frame) 1



Combinations

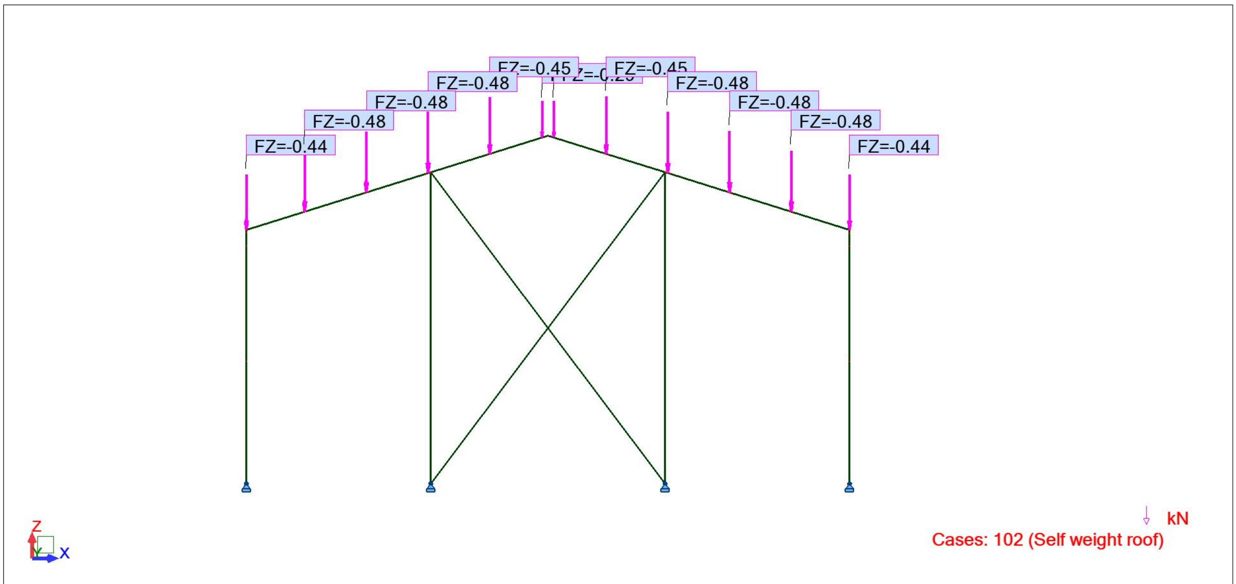
- Cases: 10 11 20 30to32 40to42 50 80to84 90to93

Combinations	Name	Analysis type	Combination	Case nature
10	Snow	Nonlin. Combinat	ULS	snow
11	Snow 0.5 right	Nonlin. Combinat	ULS	snow
20	Wind Cpe max	Nonlin. Combinat	ULS	wind
30	Snow + reduced	Nonlin. Combinat	ULS	snow
31	Snow 0.5 left + r	Nonlin. Combinat	ULS	snow
32	Snow 0.5 right +	Nonlin. Combinat	ULS	snow
40	Wind Cpe min +	Nonlin. Combinat	ULS	snow
41	Wind Cpe min +	Nonlin. Combinat	ULS	snow
42	Wind Cpe min +	Nonlin. Combinat	ULS	snow
50	Wind Cpe max fr	Nonlin. Combinat	ULS	snow
80	R15 Snow	Nonlin. Combinat	ULS	snow

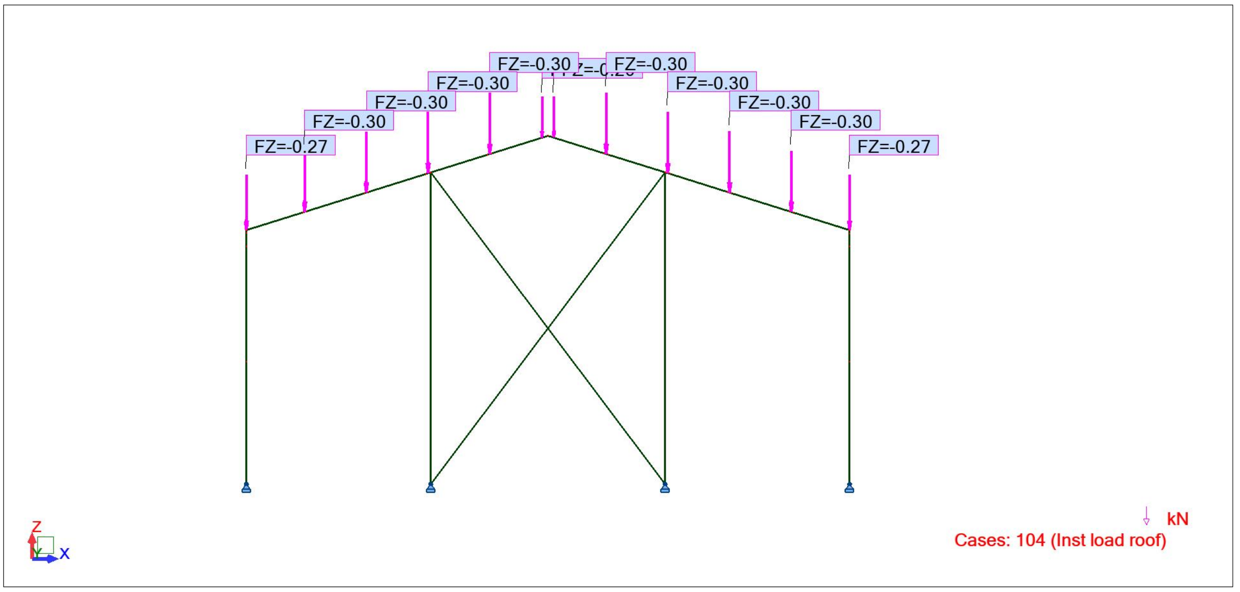
Combinations	Name	Analysis type	Combination	Case nature
81	R15 Snow 0.5 rig	Nonlin. Combinat	ULS	snow
82	R15 Wind Cpe m	Nonlin. Combinat	ULS	wind
83	R15 Wind Cpe m	Nonlin. Combinat	ULS	snow
84	R15 Wind Cpe m	Nonlin. Combinat	ULS	snow
90	SLS Characterist	Nonlin. Combinat	ULS	snow
91	SLS Characterist	Nonlin. Combinat	ULS	snow
92	SLS Characterist	Nonlin. Combinat	ULS	snow
93	SLS Characterist	Nonlin. Combinat	ULS	snow

Combinations	Definition
10	(101+102+104)*1.35+(201+202)*1.50
11	(101+102+104)*1.35+201*1.50+202*0.75
20	(101+102)*1.00+(300+321)*1.50
30	(101+102+104)*1.35+(201+202)*1.50+(301+320)*
31	(101+102+104)*1.35+201*0.75+202*1.50+(301+3
32	(101+102+104)*1.35+201*1.50+202*0.75+(301+3
40	(101+102+104)*1.35+(201+202)*0.75+(301+320)*
41	(101+102+104)*1.35+201*0.38+202*0.75+(301+3
42	(101+102+104)*1.35+201*0.75+202*0.38+(301+3
50	(101+102)*1.00+(310+321)*1.50
80	(101+102+104)*1.00+(201+202)*0.20
81	(101+102+104)*1.00+201*0.20+202*0.10
82	(101+102)*1.00+(300+321)*0.20
83	(101+102+104)*1.00+(301+320)*0.20
84	(101+102)*1.00+(310+321)*0.20
90	(101+102+104+201+202)*1.00
91	(101+102+300+321)*1.00
92	(101+102+201+202)*1.00+(301+320)*0.60
93	(101+102+301+320)*1.00+(201+202)*0.50

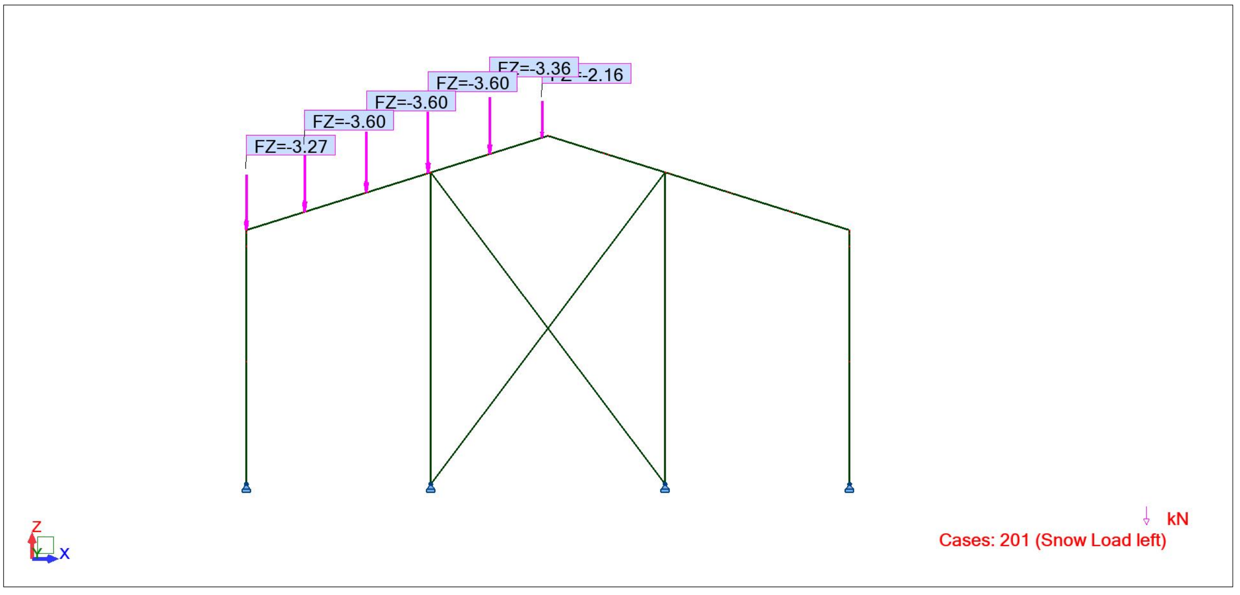
View - Cases: 102 (Self weight roof)



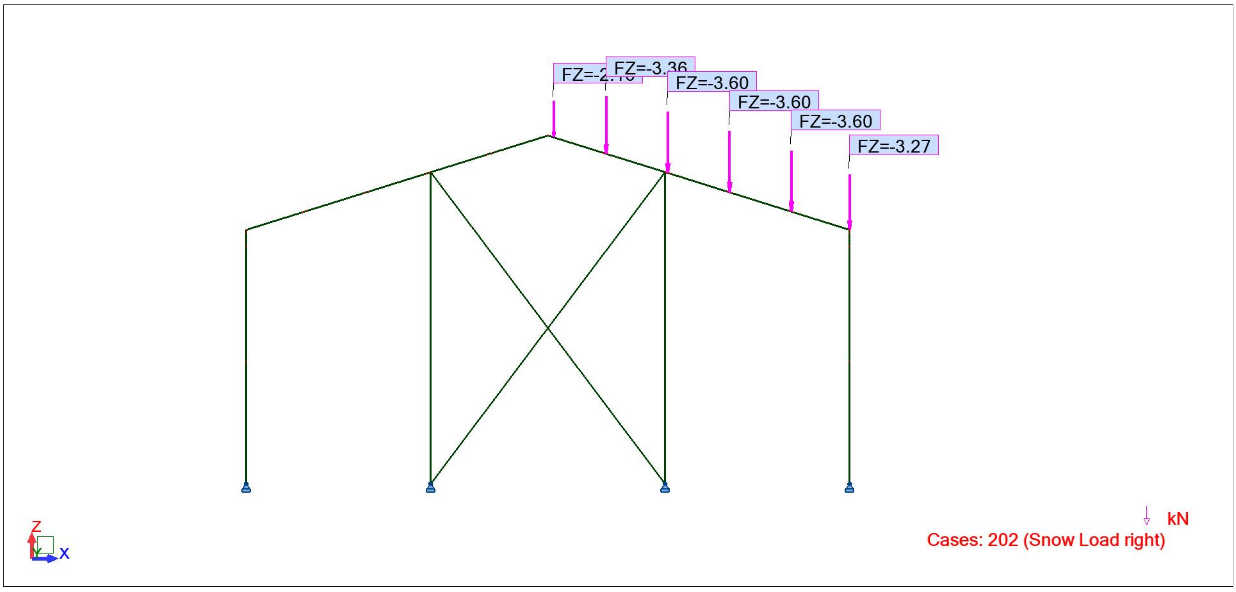
View - Cases: 104 (Inst load roof)



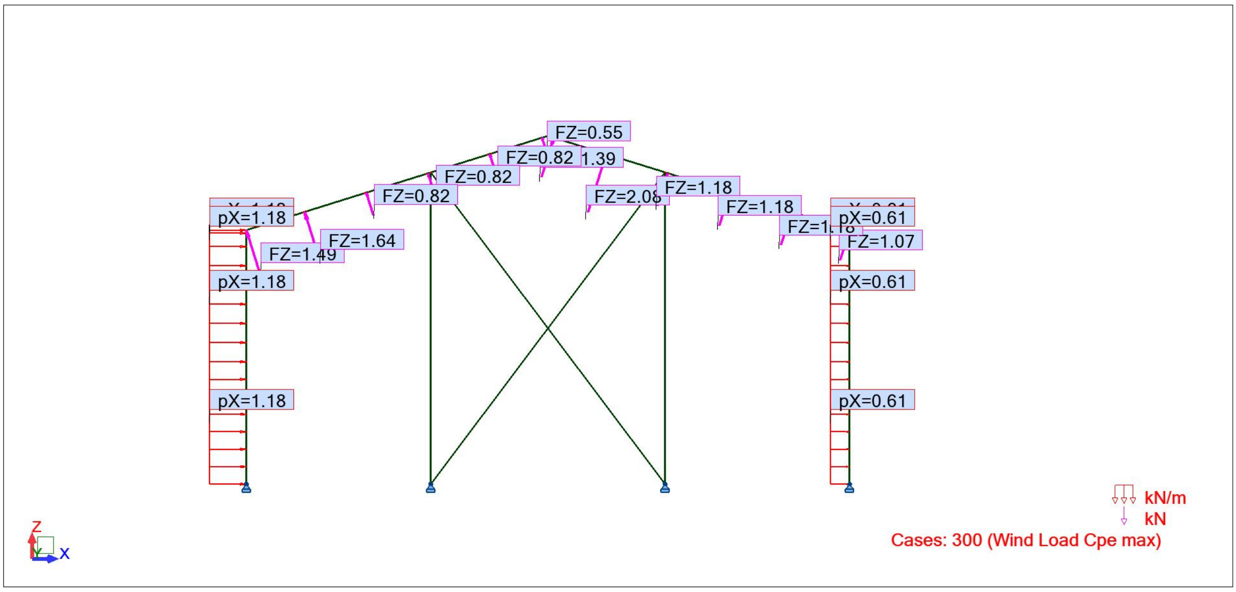
View - Cases: 201 (Snow Load left)



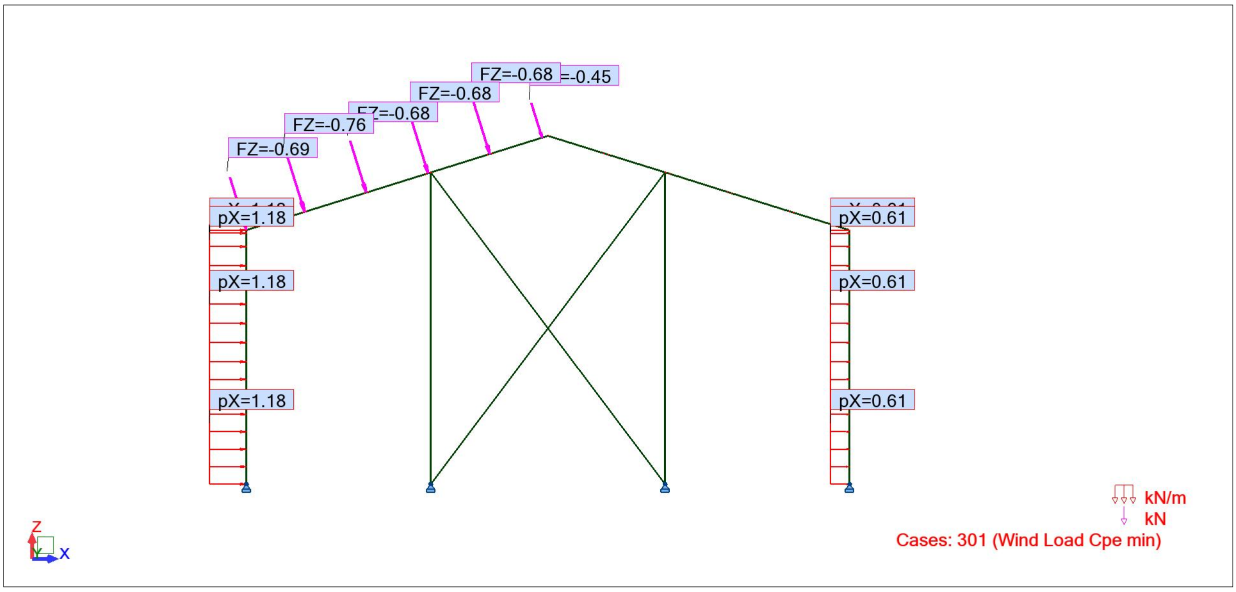
View - Cases: 202 (Snow Load right)



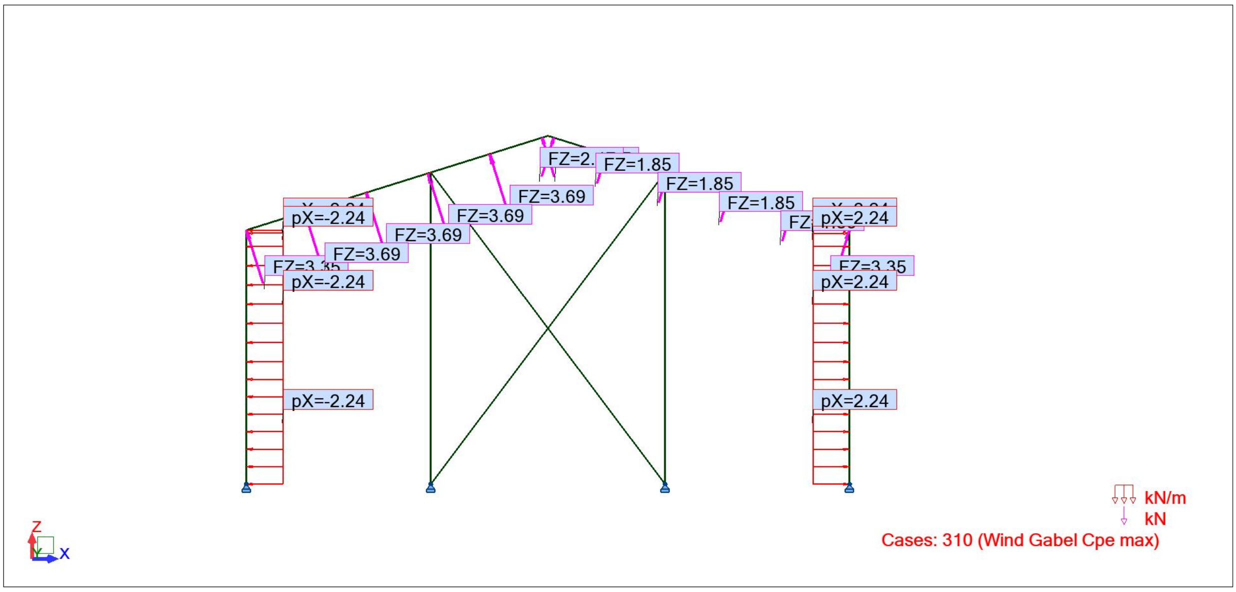
View - Cases: 300 (Wind Load Cpe max)



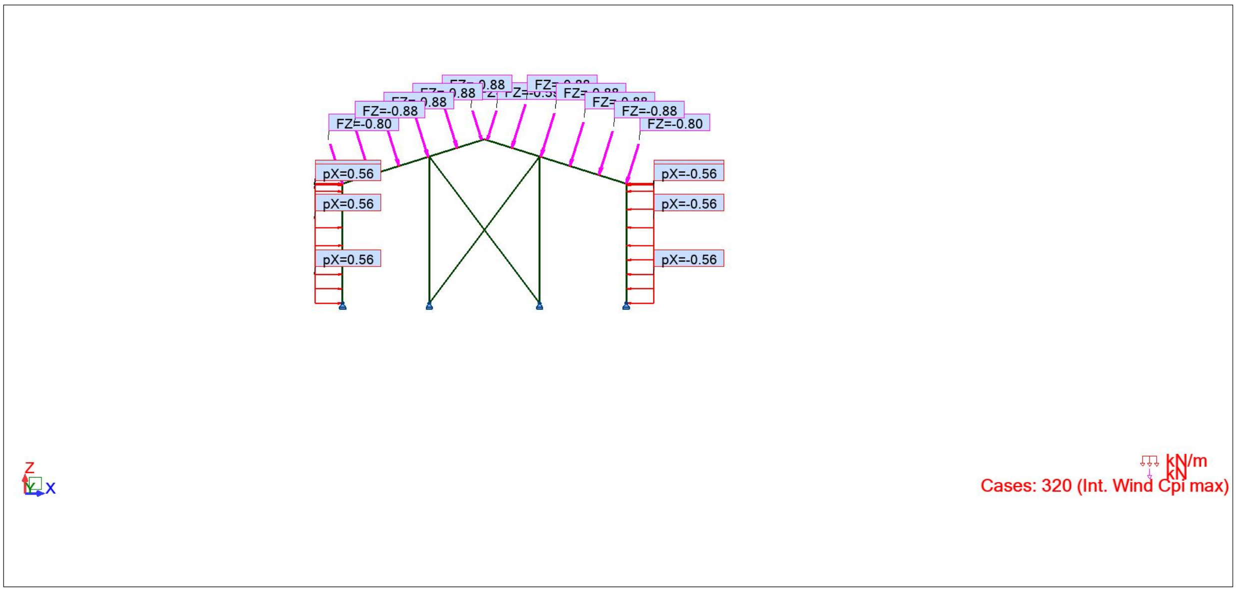
View - Cases: 301 (Wind Load Cpe min)



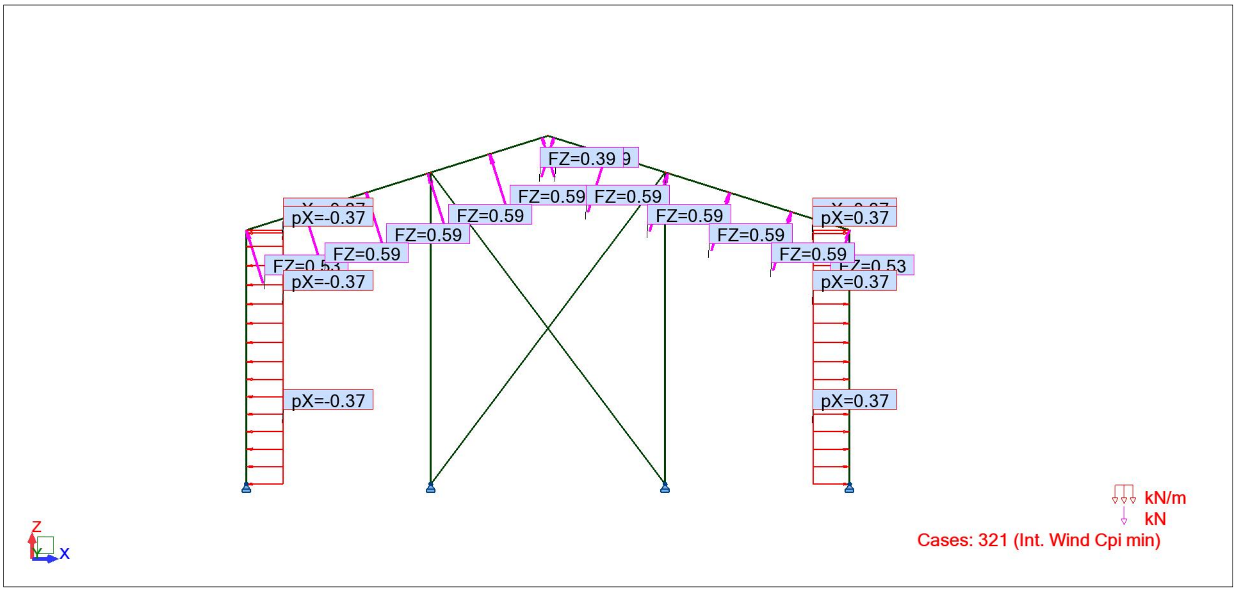
View - Cases: 310 (Wind Gabel Cpe max)



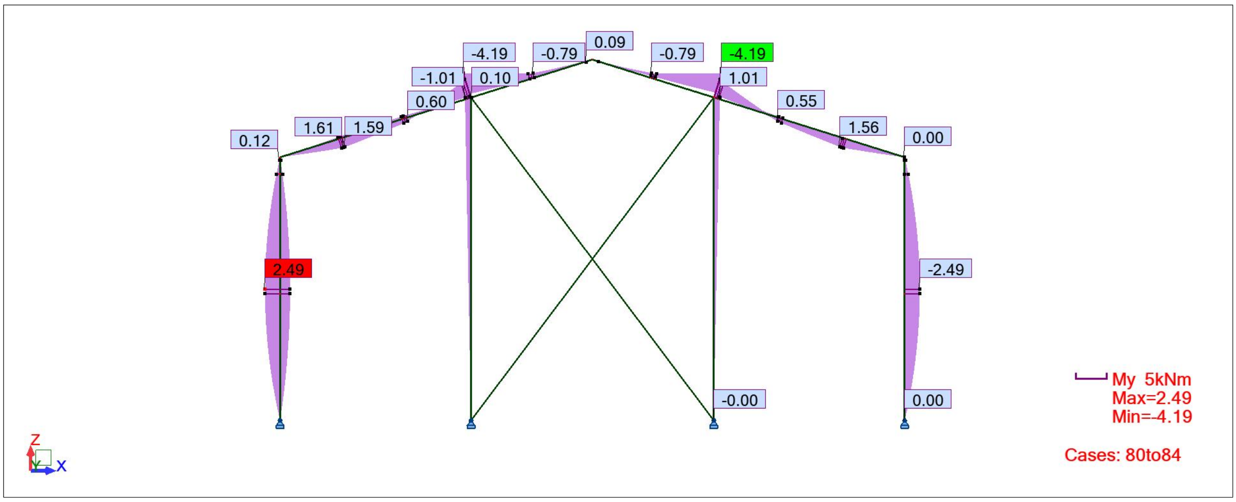
View - Cases: 320 (Int. Wind Cpi max)



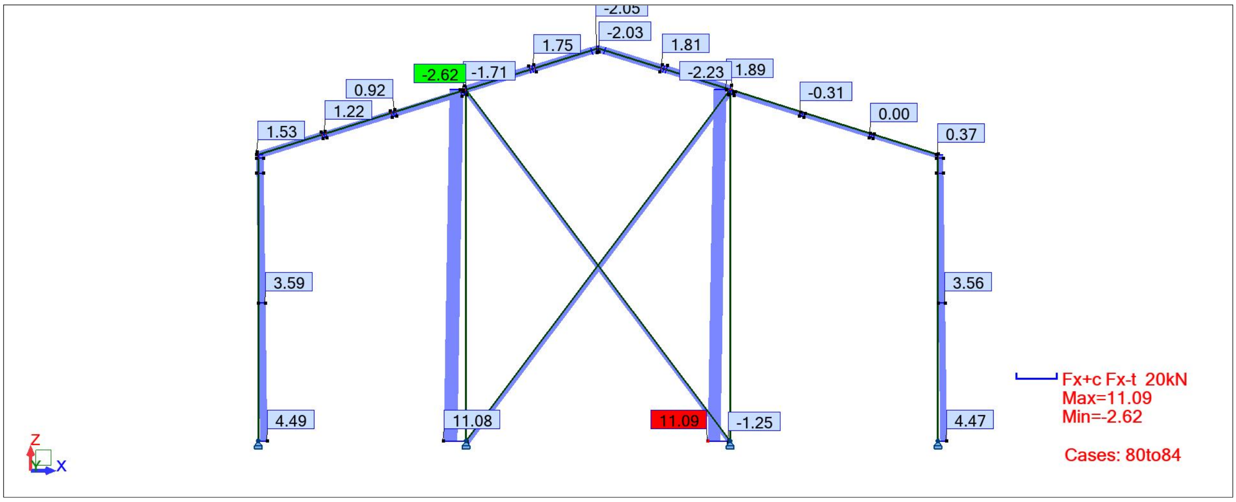
View - Cases: 321 (Int. Wind Cpi min)



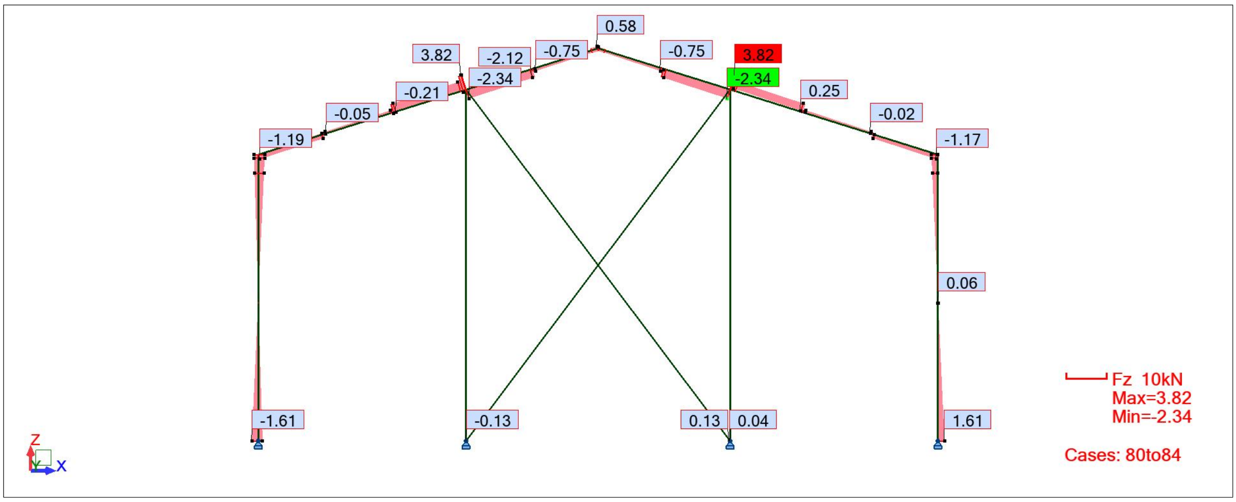
View - MY; Cases: 80to84



View - FX; Cases: 80to84



View - FZ; Cases: 80to84



Gablebeam Single C-profile R15

$h \equiv 250 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 36 \cdot mm$ $t \equiv 4 \cdot mm$ $\gamma_{M0} \equiv 1.0$ $\gamma_{M1} \equiv 1.0$

Stresses and global geometry:

$f_{yb} = 63.42 \frac{N}{mm^2}$ $_1 = 72.48 \frac{N}{mm^2}$ $N_{c.Rk} = 120.2\,kN$ $M_{ycRk} = 9.5\,kN \cdot m$

Buckling lengths:

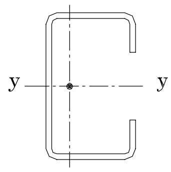
$L := 4.7 \cdot m$ Length

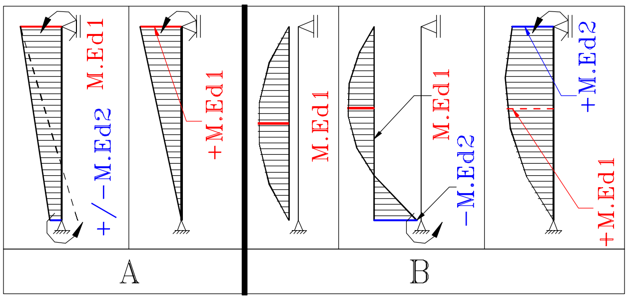
$L_y := 1L$ Flexural buckling axis y-y

$L_z := 1L$ Flexural buckling axis z-z

$L_{LT} := 0.5L$ Distance flange bracings - LTB innerflange in compression

$L_T := 1L$ Torsional buckling





$Moment\_dis := "B"$ The moment is distributed according to frame modelling like shown in figure:

stresses:

$M_{Ed.1} := 1.6 \cdot kN \cdot m$

$M_{Ed.2} := -5 \cdot kN \cdot m$

$M_{Ed.z} := 0 \cdot kN \cdot m$

$N_{Ed} := 3 \cdot kN$

$V_{Ed} := 4 \cdot kN$

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = 0.01\,kN \cdot m$

$M_{z.Ed} := \left|\Delta M_{z.shift}\right| + M_{Ed.z}$

Combined bending an axial compression EN 1993-1-3: 6.2.2 and 6.2.3

Design buckling resistance for buckling mode -flexural buckling-: y-y

$\lambda_1 = 70.71$ $\lambda_{r.y.FB}(L_y) = 0.67$ $N_{cr.y.C}(L_y) = 271.7\,kN$ $\alpha_{y.FB} = 0.34$ $\phi_{y.FB}(L_y) = 0.8$ $\chi_{y.FB}(L_y) = 0.8$

$N_{b.Rk.y.FB}(L_y) = 96.5\,kN$ $N_{b.Rd.y.FB}(L_y) = 96.5\,kN$

Design buckling resistance for buckling mode -flexural buckling-: z-z

$\lambda_1 = 70.71$ $\lambda_{r.z.FB}(L_z) = 1.74$ $N_{cr.z.C}(L_z) = 39.9\,kN$ $\alpha_{z.FB} = 0.34$ $\phi_{z.FB}(L_z) = 2.27$ $\chi_{z.FB}(L_z) = 0.27$

$N_{b.Rk.z.FB}(L_z) = 32.27\,kN$ $N_{b.Rd.z.FB}(L_z) = 32.27\,kN$

Design buckling resistance for buckling mode -torsional or torsional-flexural buckling-:

$I_T = 1.08 \times 10^4\,mm^4$ $.72 \times 10^{10}\,mm^6$ $y_0 = 78.64\,mm$ $z_0 = 0\,mm$ $i_0 = 131.04\,mm$ $_{TF} = 0.64$ $N_{cr.TF}(L_T, L_y) = 36.77\,kN$

$\lambda_{r.TF}(L_T, L_y) = 1.81$ $\iota_{TF} = 0.34$ $\phi_{TF}(L_T, L_y) = 2.41$ $\chi_{TF}(L_T, L_y) = 0.25$ $N_{b.Rd.TF}(L_T, L_y) = 30.06\,kN$

$N_{b.Rk.TF}(L_T, L_y) = 30.06\,kN$

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := \max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|\right)$ $M_{y.max.C} = 5\,kN \cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25 \cdot L_{LT}$ $M_{y.AA} := M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = -2.52\,kN \cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5 \cdot L_{LT}$ $M_{y.BA} := M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = -3.35\,kN \cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75 \cdot L_{LT}$ $M_{y.CA} := M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = -4.18\,kN \cdot m$

$$C_{bA} := \frac{12.5 \cdot M_{y.max.C}}{2.5 \cdot M_{y.max.C} + 3 \cdot \left|M_{y.AA}\right| + 4 \cdot \left|M_{y.BA}\right| + 3 \cdot \left|M_{y.CA}\right|}$$

$y_{0.cr} := (y_M + e_{1c.rc}) \cdot -1$ $z_{0.cr} := 0 \cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2 \cdot E}{\left(\frac{1 \cdot L_{LT}}{i_z}\right)^2} \qquad \sigma_{ez} = 80.79 \frac{N}{mm^2}$$

$$\sigma_T := \frac{1}{A_g \cdot r_0^2} \cdot \left(G \cdot I_T + \frac{\pi^2 \cdot E \cdot I_\omega}{L_T^2}\right) \qquad \sigma_T = 19.66 \frac{N}{mm^2}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry:

$M_{e.cr.C\_A} := C_{bA} \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T}$ $M_{e.cr.C\_A} = 14\,kN \cdot m$

about the axis of symmetry:

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 63\dfrac{N}{mm^2}$ $f_u = 72\dfrac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Relative slenderness: $\lambda_{rLT.C\_A} := \begin{cases} \sqrt{\dfrac{W_{eff.y.1} \cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0 \\ 0.2 & \text{if } M_{e.cr.C\_A} = 0 \end{cases}$ $\lambda_{rLT.C\_A} = 0.82$

Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$\phi_{LT.C\_A} := 0.5 \cdot \left[1 + \alpha_{LT.C} \cdot \left(\lambda_{rLT.C\_A} - 0.2\right) + \lambda_{rLT.C\_A}^2\right]$ $\chi_{LT.C\_A} := \min\left(\dfrac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right)$ $\chi_{LT.C\_A} = 0.71$

For moment distribution accord. to modell B: $q_{Ed.M} := \dfrac{\left|M_{Ed.1}\right| \cdot 8}{L^2}$ $M_{y.i}\left(x_i\right) := \dfrac{q_{Ed.M}}{2} \cdot \left(L \cdot x_i - x_i^2\right)$

Moment at quater point of unbraced segment: $x_{1\_4} := \begin{cases} \dfrac{L_{LT}}{4} & \text{if } L_{LT} = 0.5 \cdot L \\ \dfrac{L - L_{LT}}{2} + \dfrac{1}{4} \cdot L_{LT} & \text{otherwise} \end{cases}$ $x_{1\_4} = 0.59\,m$ $M_{y.i}\left(x_{1\_4}\right) = 0.7\,kN \cdot m$ $M_{y.AB} := M_{y.i}\left(x_{1\_4}\right)$ $M_{y.AB} = 0.7\,kN \cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := \begin{cases} \dfrac{2 \cdot L_{LT}}{4} & \text{if } L_{LT} = 0.5 \cdot L \\ \dfrac{L - L_{LT}}{2} + \dfrac{1}{2} \cdot L_{LT} & \text{otherwise} \end{cases}$ $x_{1\_2} = 1.18\,m$ $M_{y.i}\left(x_{1\_2}\right) = 1.2\,kN \cdot m$ $M_{y.BB} := M_{y.i}\left(x_{1\_2}\right)$ $M_{y.BB} = 1.2\,kN \cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := \begin{cases} \dfrac{3 \cdot L_{LT}}{4} & \text{if } L_{LT} = 0.5 \cdot L \\ \dfrac{L - L_{LT}}{2} + \dfrac{3}{4} \cdot L_{LT} & \text{otherwise} \end{cases}$ $x_{3\_4} = 1.76\,m$ $M_{y.i}\left(x_{3\_4}\right) = 1.5\,kN \cdot m$ $M_{y.CB} := M_{y.i}\left(x_{3\_4}\right)$ $M_{y.CB} = 1.5\,kN \cdot m$

$$C_{bB} := \frac{12.5 \cdot \left(\left|M_{Ed.1}\right|\right)}{2.5 \cdot \left(\left|M_{Ed.1}\right|\right) + 3 \cdot M_{y.AB} + 4 \cdot M_{y.BB} + 3 \cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := C_{bB} \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T}$ $M_{e.cr.C\_B} = 13.4\,kN \cdot m$

Relative slenderness: $\lambda_{rLT.C\_B} := \begin{cases} \sqrt{\dfrac{W_{eff.y.1} \cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0 \\ 0.2 & \text{if } M_{e.cr.C\_B} = 0 \end{cases}$ $\lambda_{rLT.C\_B} = 0.84$

$\phi_{LT.C\_B} := 0.5 \cdot \left[1 + \alpha_{LT.C} \cdot \left(\lambda_{rLT.C\_B} - 0.2\right) + \lambda_{rLT.C\_B}^2\right]$ $\chi_{LT.C\_B} := \min\left(\dfrac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right)$ $\chi_{LT.C\_B} = 0.7$

$\phi_{LT.C\_B} = 0.96$

Moment distribution: $Moment\_dis = \text{"B"}$ $\chi_{LT} := \begin{cases} \chi_{LT.C\_A} & \text{if } Moment\_dis = \text{"A"} \\ \chi_{LT.C\_B} & \text{if } Moment\_dis = \text{"B"} \end{cases}$ $\chi_{LT} = 0.7$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):

The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **<u>method 2</u>** of EN 1993-1-1: 6.3.3(4)

<u>Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3</u>

Moment distribution like shown on figure: $Moment\_dis = \text{"B"}$

$\psi_{md.A} := \begin{cases} \dfrac{M_{Ed.2}}{M_{Ed.1}} & \text{if } \left|M_{Ed.1}\right| \ge \left|M_{Ed.2}\right| \\ \dfrac{M_{Ed.1}}{M_{Ed.2}} & \text{if } \left|M_{Ed.1}\right| < \left|M_{Ed.2}\right| \end{cases}$ $\psi_{md.A} = -0.32$ $\psi_{m.B} := 0$

${}_{h.2} := \begin{cases} \dfrac{M_{Ed.2}}{M_{Ed.1}} & \text{if } \left|M_{Ed.1}\right| \ge \left|M_{Ed.2}\right| \\ 0 & \text{otherwise} \end{cases}$ $\alpha_{h.2} = 0$

$\psi_{m.B} := 0$

$\alpha_{s.2} := \begin{cases} \dfrac{M_{Ed.1}}{M_{Ed.2}} & \text{if } \left|M_{Ed.1}\right| < \left|M_{Ed.2}\right| \\ 0 & \text{otherwise} \end{cases}$ $\alpha_{s.2} = -0.32$

$\psi_{md.z} := 0$ $\alpha_{h.2.z} := 0$

$b = 100\,mm \quad h = 250\,mm \quad c = 36\,mm \quad t = 4\,mm$ $\quad f_{yb} = 63\,\frac{N}{mm^2} \quad f_u = 72\,\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

$C_{my.2.A} := \max(0.6 + 0.4\cdot\psi_{md.A}, 0.4) \quad C_{my.2.A} = 0.47$

$C_{my.2.B.s} := \begin{cases} \max[(0.2 + 0.8\cdot\alpha_{s.2}), 0.4] & \text{if } 0 \le \alpha_{s.2} \le 1 \\ \max[(0.1 - 0.8\cdot\alpha_{s.2}), 0.4] & \text{if } -1 \le \alpha_{s.2} < 0 \end{cases} \quad C_{my.2.B.s} = 0.4$

$C_{my.2.B.h} := 0.95 + 0.05\cdot\alpha_{h.2} \quad C_{my.2.B.h} = 0.95$

$C_{my.2.B} := \begin{cases} C_{my.2.B.h} & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ C_{my.2.B.s} & \text{otherwise} \end{cases} \quad C_{my.2.B} = 0.4$

$C_{mz.2} := 0.95 + 0.05\cdot\alpha_{h.2.z} \quad C_{mz.2} = 0.95$

$C_{my.2} := \begin{cases} C_{my.2.A} & \text{if Moment\_dis} = \text{"A"} \\ C_{my.2.B} & \text{if Moment\_dis} = \text{"B"} \end{cases} \quad C_{my.2} = 0.4 \quad C_{mLT.2} := C_{my.2} \quad C_{mLT.2} = 0.4$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$n_{y.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(L_y)\cdot N_{c.Rk}} \quad n_{y.2} = 0.03 \quad n_{z.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(L_z)\cdot N_{c.Rk}} \quad n_{z.2} = 0.09$

$k_{yy} := \min[C_{my.2}\cdot(1 + 0.6\cdot\lambda_{r.y.FB}(L_y)\cdot n_{y.2}), C_{my.2}\cdot(1 + 0.6\cdot n_{y.2})] \quad k_{yy} = 0.4$

$k_{zy} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FB}(L_z)}{C_{mLT.2} - 0.25}\cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25}\cdot n_{z.2}\right) \quad k_{zy} = 0.97$

$k_{zz} := \min[C_{mz.2}\cdot(1 + 0.6\cdot\lambda_{r.z.FB}(L_z)\cdot n_{z.2}), C_{mz.2}\cdot(1 + 0.6\cdot n_{z.2})] \quad k_{zz} = 1 \quad k_{yz} := k_{zz}$

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$\frac{N_{Ed}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M0}}{M_{ycRk}} + \frac{|M_{z.Ed}|\cdot\gamma_{M0}}{M_{z.Rk}} = 0.56$ <1.0

Axial compression EN 1993-1-1: 6.3.1:

with relevant buckling mode:

$\frac{N_{Ed}\cdot\gamma_{M1}}{\min(\chi_{y.FB}(L_y), \chi_{z.FB}(L_z), \chi_{TF}(L_T, L_y))\cdot N_{c.Rk}} = 0.1$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y: $\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(L_y)\cdot N_{c.Rk}} + k_{yy}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{ycRk}} + k_{yz}\cdot\frac{|M_{z.Ed}|\cdot\gamma_{M1}}{M_{z.Rk}} = 0.34$ <1.0

Evading in z-z $\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(L_z)\cdot N_{c.Rk}} + k_{zy}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{ycRk}} + k_{zz}\cdot\frac{|M_{z.Ed}|\cdot\gamma_{M1}}{M_{z.Rk}} = 0.83$ <1.0

Shear force EN 1993-1-3: 6.1.10:

$\frac{V_{Ed}\cdot\gamma_{M1}\cdot 2}{V_{bh.Rk}} = 0.25$ < 1.0
if not met, check formula below:

$M\_N\_V_{EC} = 0$ <1.0

G. Štítové sloupy A1/1, 8 D1/1, 8 R15

Geometrie:

Zatěžovací šířka sloupku L_w= **3.51** m
Rozpon L= **6.30** m

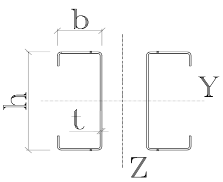
Zatížení větrem: ČSN EN 1991-1-4 ()

Kategorie terénu: II. Oblasti s nízkou vegetací jako je tráva nebo izolované překážky (stromy, budovy)
Charakteristická hodnota rychlosti větru: v_{bo}= 25.0 m/s
Maximální dynamický tlak q_p (z)= 0.936 kN/m²

Součinitel vnějšího tlaku (tlak) C_{pe}=	**0.96**		Součinitel vnějšího tlaku (sání) C_{pe}=	**1.50**	
Zatížení větrem (tlak) w_e=	0.90	kN/m²	Zatížení větrem (sání) w_e=	1.40	kN/m²
Normové zatížení větrem (tlak) q_n=	**3.15**	kN/m	Normové zatížení větrem (sání) q_n=	**4.93**	kN/m
Součinitel zatížení g_w=	0.2		Součinitel zatížení g_w=	0.2	
Výpočtové zatížení větrem (tlak) q_d=	**0.63**	kN/m	Výpočtové zatížení větrem (sání) q_d=	**0.99**	kN/m

Charakteristika profilu:

Průřez profilem = **Dvojitý** Profil = **C250X5**



Výška profilu h=	250	mm	Momentová únosnost M_y=	76.20	kNm
Šířka profilu b=	100	mm	Smyková únosnost V_{wy}=	297.05	kN
Tloušťka t=	5	mm	Smyk v podpoře R_{wy}=	135.51	kN
Mez kluzu oceli f_u=	480	MPa	Efekt. moment setrvačnosti I_{yeff}=	23020000	mm4
Mez pevnosti oceli f_b=	420	MPa	Moment setrvačnosti I_y=	23600000	mm4

Součinitelé materiálu:

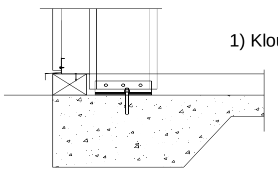
Country = **Czech**
Únosnost průřezu γ_{M0}= 1.00
Únosnost při vzpěru γ_{M1}= 1.00
Únosnost oslabeného průřezu γ_{M2}= 1.25

Deformace:

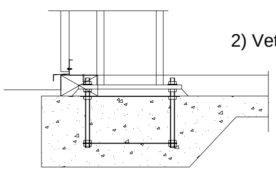
Limit deformace 1/ **250**
Skutečný poměr 603

Kotvení do spodní stavby:

Vetknuté kotvení



1) Kloubové kotvení



2) Vetknuté kotvení

Předběžné posouzení:

Vetknutý sloup

Sání pro C_{pe}= 1.50 Tlak pro C_{pe}= 0.96

Ohybový moment ve vetknutí

M_{Sd}=1/2.q.L²	$M_{Sd,s}$=	**19.55**	kNm	$M_{Sd,s}$=	**12.51**	kNm	My =	**152.40** kNm
	PODMÍNKA VYHOVUJE			**PODMÍNKA VYHOVUJE**				

Podporová reakce

R=q.L	$R_{Sd,s}$=	**6.21**	kN	$R_{Sd,s}$=	**3.97**	kN	Ry =	**271.02** kN
	PODMÍNKA VYHOVUJE			**PODMÍNKA VYHOVUJE**				

Deformace

w = q.L4/185.E.Iy	w_{max} =	**4.34**	mm	w_{max} =	**2.78**	mm	w_{lim} =	**25.20** mm
	PODMÍNKA VYHOVUJE			**PODMÍNKA VYHOVUJE**				

Double gable pillar
A1/1, 8 D1/1, 8

$h \equiv 250 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 36 \cdot mm$ $t \equiv 4 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $cc_{batt} := 1000 \cdot mm$ distance between battens

$b_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $\gamma_{M1} \equiv 1.0$

$h_{batt} := 200 \cdot mm$

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 3.952 \times 10^3 \, mm^2$

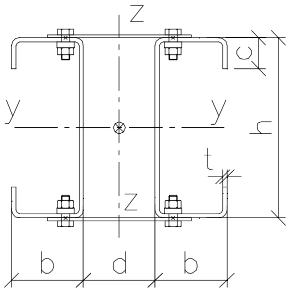
$I_{zz} := 2 \cdot \left[I_z + A_g \cdot \left(e_1 + \frac{d}{2} \right)^2 \right]$ $I_{zz} = 3.26 \times 10^7 \, mm^4$

$M_{yy.cRk} := 2 \cdot M_{ycRk}$

$M_{yy.cRk} = 125.14 \, kN \cdot m$

$f_{yb} = 420 \frac{N}{mm^2}$

$f_u = 480 \frac{N}{mm^2}$



$I_{yy} := 2 \cdot I_y$ $I_{yy} = 3.79 \times 10^7 \, mm^4$ single C: $M_{ycRk} = 62.57 \, kN \cdot m$

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 3.08 \times 10^5 \, mm^3$ $N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 2.98 \times 10^5 \, mm^3$ $N_{cc.Rk} = 1260.25 \, kN$

$W_{zz} := \frac{I_{zz}}{b + \frac{d}{2}}$ $W_{zz} = 2.17 \times 10^5 \, mm^3$ $i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 97.88 \, mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 90.81 \, mm$

$I_{TT} = 21589.33 \, mm^4$ torsion_plate_pillar = "NO"

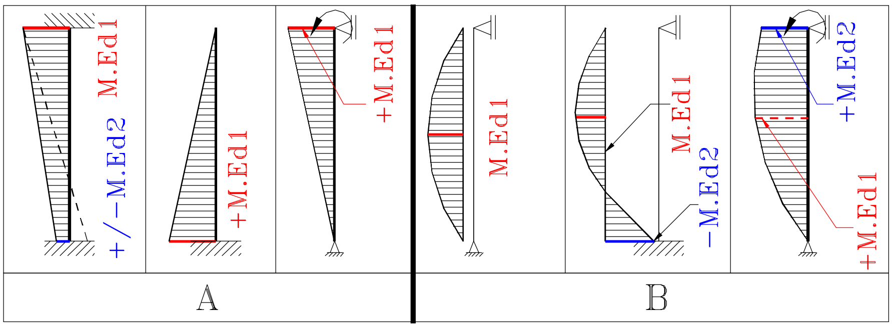
Stresses, global geometry and buckling lengths:

$M_{Ed.1} := 14 \cdot kN \cdot m$ Moment y-y

$M_{Ed.suc.1} := 21 \cdot kN \cdot m$ Moment y-y acting as suction

$M_{Ed.1.z} := 19 \cdot kN \cdot m$ Moment z-z

$N_{Ed} := 15 \cdot kN$



$V_{Ed} := 30 \cdot kN$ > $n_{cross} := 4$ =Numbers of holes in **ONE** web

$d_0 := 12.5 \cdot mm$

Moment_dis := "B" $M_{Ed.2} := 37 \cdot kN \cdot m$

shift of neutral axis for member in compression: $\Delta e_N = -5.24 \, mm$

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = -0.08 \, kN \cdot m$ $_{z.Ed} := \Delta M_{z.shift}$

Buckling lengths:

$L := 6.3 \cdot m$ Length of pillar $cc_{wallZ} := L$ = Distance between wall-Z-profile

$L_y := 2.0 \cdot L$ Flexural buckling axis y-y

$L_z := cc_{wallZ}$ Flexural buckling axis z-z (= Distance between wall-Z-profile)

$L_{LT} := cc_{wallZ}$ Distance flange bracings - LTB outer flange in compression for wind acting as pressure

$L_{LT.suc} := 1.0 \cdot L$ Distance flange bracings - LTB inner flange in compression for wind acting as suction

$L_T := cc_{batt}$ Torsional buckling single C-profile between battens.

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Flexural buckling resistance - axis y-y and z-z EN 1993-1-3: 6.2.2 and EN 1993-1-1: 6.3.1:

Buckling curve for double C-section: EN 1993-1-3 table 6.3 about:

y-y: a EN1993-1-1,table 6.1: α =0,21 z-z: b EN1993-1-1,table 6.1: α =0,34

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about y-y

$$\lambda_1 := \pi\cdot\sqrt{\frac{E}{f_{yb}}} \qquad \lambda_1 = 70.25 \qquad \lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \qquad \lambda_{r.y.FBcc} = 1.6$$

Imperfection factor α relating to bucking curve a $\alpha_y := 0.21$

$$\phi_{y.FB} := 0.5\cdot\left[1 + \alpha_y\cdot\left(\lambda_{r.y.FBcc} - 0.2\right) + \lambda_{r.y.FBcc}^{\,2}\right] \qquad \phi_{y.FB} = 1.92$$

$$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^{\,2} - \lambda_{r.y.FBcc}^{\,2}}}, 1\right) \qquad \chi_{y.FBcc} = 0.33$$

Design buckling resistance for buckling mode -flexural buckling-: y-y

$$N_{b.Rd.y.FBcc} := \frac{\chi_{y.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \qquad N_{b.Rd.y.FBcc} = 421.4\,kN$$

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about z-z

$$\lambda_1 = 70.25 \qquad \lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \qquad \lambda_{r.z.FBcc} = 0.86$$

Imperfection factor α relating to bucking curve b $\alpha_z := 0.34$

$$\phi_{z.FB} := 0.5\cdot\left[1 + \alpha_z\cdot\left(\lambda_{r.z.FBcc} - 0.2\right) + \lambda_{r.z.FBcc}^{\,2}\right] \qquad \phi_{z.FB} = 0.98$$

$$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^{\,2} - \lambda_{r.z.FBcc}^{\,2}}}, 1\right) \qquad \chi_{z.FBcc} = 0.69$$

Design buckling resistance for buckling mode -flexural buckling-: z-z

$$N_{b.Rd.z.FBcc} := \frac{\chi_{z.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \qquad N_{b.Rd.z.FBcc} = 865.1\,kN$$

Lateral-torsional buckling resistance for uniform member in bending EN 1993-1-3: 6.2.4 and EN 1993-1-1: 6.3.2:

Procedure to calculate the elastic critical buckling moment M_{cr}, based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints, is given neither in EN 1993-1-1 nor in EN 1993-1-3. Any appropriate calculation method can be used, here the procedure given in German ENV 1993-1-1 Anex F.

$Moment\_dis = "B"$ $L_{LT} = 6.3\,m$ $k_{M.cr} := 1.0$ (hinged at ends) $k_{w.M.cr} := 1.0$ (no special wrap restraints at ends)

For pillar with end moments: according to picture "A":

$$\psi_{md.A} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \geq |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases} \qquad \psi_{md.A} = 0.38$$

$$C_{1.A} := \min\left(1.88 - 1.40\cdot\psi_{md.A} + 0.52\cdot\psi_{md.A}^{\,2}, 2.7\right) \qquad C_{1.A} = 1.42$$

$$M_{cr.A} := C_{1.A}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{\left(k_{M.cr}\cdot L_{LT}\right)^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr}\cdot L_{LT}\right)^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.A} = 149.23\,kN\cdot m$$

For pillar with moment distribution according to picture "B": $C_{1.B} := 1.132$

$$M_{cr.B} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{\left(k_{M.cr}\cdot L_{LT}\right)^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr}\cdot L_{LT}\right)^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.B} = 118.57\,kN\cdot m$$

Elastic critical moment for lateral-torsional buckling

based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints for double C-section:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if } Moment\_dis = "A" \\ M_{cr.B} & \text{if } Moment\_dis = "B" \end{cases} \qquad M_{cr} = 119\,kN\cdot m$$

Relative slenderness: $\lambda_{rLT} := \sqrt{\frac{2\cdot W_{eff.y.1}\cdot f_{yb}}{M_{cr}}}$ $\lambda_{rLT} = 1.03$ $\alpha_{LT} := 0.34$ $\phi_{LT} := 0.5\cdot\left[1 + \alpha_{LT}\cdot\left(\lambda_{rLT} - 0.2\right) + \lambda_{rLT}^{\,2}\right]$ $\phi_{LT} = 1.17$

Reduction buckling factor: $\chi_{LT} := \min\left(\frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{\,2} - \lambda_{rLT}^{\,2}}}, 1\right)$ $\chi_{LT} = 0.58$

Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

Desing buckling moment resistance double profile based on effective section modulus W_{eff}.

$$M_{b.Rd} := \frac{\chi_{LT}\cdot M_{yy.cRk}}{\gamma_{M1}} \qquad M_{b.Rd} = 72.55\,kN\cdot m$$

$b = 100\,\mathrm{mm}$ $h = 250\,\mathrm{mm}$ $c = 36\,\mathrm{mm}$ $t = 4\,\mathrm{mm}$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

For pillar when wind acting as suction (moment from line load):

$$M_{cr.suc} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT.suc})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT.suc})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.suc} = 118.57\,\mathrm{kN\cdot m}$$

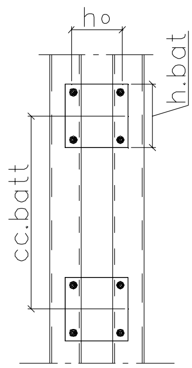
Relative slenderness: $\lambda_{rLT.suc} := \sqrt{\frac{2\cdot W_{eff.y.1}\cdot f_{yb}}{M_{cr.suc}}}$ $\lambda_{rLT.suc} = 1.03$ $\phi_{LT.suc} := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{rLT.suc} - 0.2) + \lambda_{rLT.suc}^2\right]$ $\phi_{LT.suc} = 1.17$

Reduction buckling factor: $\chi_{LT.suc} := \min\left(\frac{1}{\phi_{LT.suc} + \sqrt{\phi_{LT.suc}^2 - \lambda_{rLT.suc}^2}}, 1\right)$ $\chi_{LT.suc} = 0.58$

<u>Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:</u>

Desing buckling moment resistance double profile for wind acting as suction $M_{b.Rd.suc} := \frac{\chi_{LT.suc}\cdot M_{yy.cRk}}{\gamma_{M1}}$ $M_{b.Rd.suc} = 72.55\,\mathrm{kN\cdot m}$

<u>Check Uniform built-up member EN 1993-1-1: 6.4</u>



bow imperfection: $e_0 := \frac{L}{500}$

$\mathrm{if}\left(\frac{cc_{batt}}{i_z} \le 70, \text{"OK"}, \text{"Shorter-cc.bat"}\right) = \text{"OK"}$

non-dimensional slenderness: $\lambda_1 = 70.25$

<u>Effective second moment of area of battened built-up member:</u>

$A_{ch} := A_g$: area of one chord

$h_0 := d + 2\cdot e_1$ $h_0 = 165.4\,\mathrm{mm}$ distance centroids of chords

$I_{ch} := I_z$: I of one chord

$I_{l.CC} := 0.5\cdot h_0^2\cdot A_{ch} + 2\cdot I_{ch}$ I built-up member $i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2\cdot A_{ch}}}$ $\lambda_{CC} := \frac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 69.37$

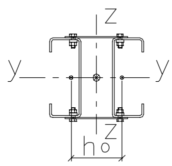
EN 1993-1-1: table 6.8
Efficiency factor:

$$\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \ge 150 \\ \left(2 - \frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \le 75 \end{cases}$$

$I_{l.CC} = 3.26\times 10^7\,\mathrm{mm}^4$

$\mu_{CC} = 1$

$r_{CC} := 2$



$I_{eff} := 0.5\cdot h_0^2\cdot A_{ch} + 2\cdot\mu_{CC}\cdot I_{ch}$ $I_{eff} = 3.26\times 10^7\,\mathrm{mm}^4$ effective I of built-up member

<u>Shear stiffness EN 1993-1-1:6.4.3.1:(2)</u> $I_b := \frac{t_{batt}\cdot h_{batt}^3}{12}$ I of batten $n_{batt} := \frac{L}{cc_{batt}}$ $n_{batt} = 6.3$ ıumber of planes of lacings

$$S_v := \min\left[\left[\frac{24\cdot E\cdot I_{ch}}{cc_{batt}^2\cdot\left(1 + \frac{2\cdot I_{ch}}{n_{batt}\cdot I_b}\cdot\frac{h_0}{cc_{batt}}\right)}\right], \left(\frac{2\cdot\pi^2\cdot E\cdot I_{ch}}{cc_{batt}^2}\right)\right] \qquad S_v = 11525\,\mathrm{kN}$$

effective critical force of built-up member:

$$N_{cr.CC} := \frac{\pi^2\cdot E\cdot I_{eff}}{L_z^2} \qquad N_{cr.CC} = 1701.9\,\mathrm{kN}$$

Maximum moment in middle of built-up member: EN 1993-1-1:6.4.1 (6):

$M_{z.Ed.I} := |M_{Ed.1.z}|$

moment z-z without second order effects

moment with second order effects $M_{z.Ed.C} := \frac{N_{Ed}\cdot e_0 + M_{z.Ed.I}}{1 - \frac{N_{Ed}\cdot\gamma_{M1}}{N_{cr.CC}} - \frac{N_{Ed}\cdot\gamma_{M1}}{S_v}}$ $M_{z.Ed.C} = 19.39\,\mathrm{kN\cdot m}$

Compression force in one member: EN 1993-1-1:6.4.1 (6) Compression force in built-up member taking accout to second order effects:

$$X_d := \begin{cases} 1 & \text{if } d \ge 8\cdot mm \\ 0 & \text{otherwise} \end{cases} \qquad N_{ch.Ed} := \frac{N_{Ed}}{r_{CC}} + \frac{|M_{z.Ed.C}|\cdot h_0\cdot A_{ch}\cdot X_d}{2\cdot I_{eff}} \qquad N_{ch.Ed} = 104.7\,\mathrm{kN}$$

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):
The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3
Moment distribution like shown on figure: Moment_dis = "B" $\psi_{md.A} = 0.38$ $\psi_{m.B} := 0$

$\alpha_{h.2} = 0$ $\alpha_{s.2} = 0.38$ $\psi_{md.z} := 0$ $\alpha_{h.2.z} = 0$ $\psi_{md.suc} := 0$ $\alpha_{h.2.suc} = 0$

$C_{my.2} = 0.5$ $C_{my.2.suc} = 0.95$ $C_{mz.2} = 0.95$ $C_{mLT.2} = 0.5$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$n_{y.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}}$ $n_{y.2} = 0.04$ $n_{z.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}}$ $n_{z.2} = 0.02$

$k_{yy.2} := \min\left[C_{my.2}\cdot\left(1 + 0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}\right), C_{my.2}\cdot\left(1 + 0.6\cdot n_{y.2}\right)\right]$ $k_{yy.2} = 0.51$

$k_{zy.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FBcc}}{C_{mLT.2} - 0.25}\cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25}\cdot n_{z.2}\right)$ $k_{zy.2} = 1$

$k_{zz.2} := \min\left[C_{mz.2}\cdot\left(1 + 0.6\cdot\lambda_{r.z.FBcc}\cdot n_{z.2}\right), C_{mz.2}\cdot\left(1 + 0.6\cdot n_{z.2}\right)\right]$ $k_{zz.2} = 0.96$ $k_{yz.2} := k_{zz.2}$

$k_{yy.2.suc} := \min\left[C_{my.2.suc}\cdot\left(1 + 0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}\right), C_{my.2.suc}\cdot\left(1 + 0.6\cdot n_{y.2}\right)\right]$ $k_{yy.2.suc} = 0.97$

$I_{zz.eff} := 2\cdot I_z + A_g\cdot\left(e_1 + \frac{d}{2}\right)^2 + A_{eff}\cdot\left(e_N + \frac{d}{2}\right)^2$ $W_{zz.eff} := \frac{I_{zz.eff}}{b + \frac{d}{2}}$ $I_{zz.eff} = 3.02\times 10^7\,mm^4$ $W_{zz.eff} = 2.01\times 10^5\,mm^3$ $M_{zz.cRk} := W_{zz.eff}\cdot f_{yb}$

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{Ed}\cdot\gamma_{M0}}{2\cdot N_{c.Rk}} + \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.suc.1}\right|\right)\cdot\gamma_{M0}}{2\cdot M_{ycRk}} + \frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M0}}{M_{zz.cRk}} = 0.54$$ <1.0

Double CC-profile: Combined M + N + V acc to EN 1993-1-3: 6.1.10

$$\frac{V_{Ed}\cdot\gamma_{M1}}{2\cdot V_{bh.Rk.pillar}} = 0.09$$ < 1.0 according to EN 1993-1-3: 6.1.10 $M\_N\_V_{EC} = 0$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as pressure

Evading in y-y: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} + k_{yy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{yz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.52$$ <1.0

Evading in z-z: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} + k_{zy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{zz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.75$$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as suction

Evading in y-y: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} + k_{yy.2.suc}\cdot\frac{\left|M_{Ed.suc.1}\right|\cdot\gamma_{M1}}{\chi_{LT.suc}\cdot M_{yy.cRk}} + k_{yz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.54$$ <1.0

Evading in z-z: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} + k_{zy.2}\cdot\frac{\left|M_{Ed.suc.1}\right|\cdot\gamma_{M1}}{\chi_{LT.suc}\cdot M_{yy.cRk}} + k_{zz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.53$$ <1.0

$b = 100\,\text{mm}$ $h = 250\,\text{mm}$ $c = 36\,\text{mm}$ $t = 4\,\text{mm}$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords and battens are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1).

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := \max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.suc.1}|)\cdot 0.5$ $M_{y.max.C} = 18.5\,kN\cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25\cdot cc_{batt}$ $M_{y.AA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = 17.13\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5\cdot cc_{batt}$ $M_{y.BA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = 17.59\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75\cdot cc_{batt}$ $M_{y.CA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = 18.04\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot M_{y.max.C}}{2.5\cdot M_{y.max.C} + 3\cdot|M_{y.AA}| + 4\cdot|M_{y.BA}| + 3\cdot|M_{y.CA}|}$$

$y_{0.cr} := (y_M + e_{1c.rc})\cdot -1$ $z_{0.cr} := 0\cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot cc_{batt}}{i_z}\right)^2} \qquad \sigma_{ez} = 2916.25\,\frac{N}{mm^2}$$

$$\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{cc_{batt}^2}\right) \qquad \sigma_T = 2295.94\,\frac{N}{mm^2}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_A} := C_{bA}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 697.5\,kN\cdot m$

Relative slenderness:
$$\lambda_{rLT.C\_A} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0\\ 0.2 & \text{if } M_{e.cr.C\_A} = 0\end{cases}$$
$\lambda_{rLT.C\_A} = 0.3$ Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right] \qquad \chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right) \qquad \chi_{LT.C\_A} = 0.96$$

$\phi_{LT.C\_A} = 0.56$

For moment distribution accord. to modell B: $q_{wind\_pillar} := \frac{|M_{Ed.1}\cdot 0.5|\cdot 8}{L^2}$ $M_{y.i}(x_i) := \frac{q_{wind\_pillar}}{2}\cdot(L\cdot x_i - x_i^2)$

Moment at quater point of unbraced segment: $x_{1\_4} := \frac{L - cc_{batt}}{2} + \frac{1}{4}\cdot cc_{batt}$ $x_{1\_4} = 2.9\,m$ $M_{y.i}(x_{1\_4}) = 6.96\,kN\cdot m$ $M_{y.AB} := M_{y.i}(x_{1\_4})$ $M_{y.AB} = 6.96\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 3.15\,m$ $M_{y.i}(x_{1\_2}) = 7\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 7\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := \frac{L - cc_{batt}}{2} + \frac{3}{4}\cdot cc_{batt}$ $x_{3\_4} = 3.4\,m$ $M_{y.i}(x_{3\_4}) = 6.96\,kN\cdot m$ $M_{y.CB} := M_{y.i}(x_{3\_4})$ $M_{y.CB} = 6.96\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot(|M_{Ed.1}\cdot 0.5|)}{2.5\cdot(|M_{Ed.1}\cdot 0.5|) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := C_{bB}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 672\,kN\cdot m$

Relative slenderness:
$$\lambda_{rLT.C\_B} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0\\ 0.2 & \text{if } M_{e.cr.C\_B} = 0\end{cases}$$
$\lambda_{rLT.C\_B} = 0.31$

$$\phi_{LT.C\_B} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_B} - 0.2) + \lambda_{rLT.C\_B}^2\right] \qquad \chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right) \qquad \chi_{LT.C\_B} = 0.96$$

$\phi_{LT.C\_B} = 0.56$

Moment distribution: $Moment\_dis = \text{"B"}$
$$\chi_{LT.C} := \begin{cases}\chi_{LT.C\_A} & \text{if } Moment\_dis = \text{"A"}\\ \chi_{LT.C\_B} & \text{if } Moment\_dis = \text{"B"}\end{cases}$$
$\chi_{LT.C} = 0.96$

Stresses on one member profile at mid-span from above: chord force from above: $N_{ch.Ed} = 104.7\,kN$ *max moment y-y:* $M_{y.max.C} = 18.5\,kN\cdot m$

$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Chord at end panel: highest shear force: $r_{CC} = 2$ $V_{Ed.max} := \pi\cdot\frac{M_{z.Ed.C}}{L}$ $V_{Ed.max} = 9.67\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 4.83\,kN$

$V_{bb.Rd} = 186.23\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.026$ << 1,0 The shear force is negligible

Chord at end panel: "corner" moment: $M_{z.ch} := \frac{V_{Ed.max}}{r_{CC}}\cdot\frac{cc_{batt}}{2}$ *maximum moment z-z:* $M_{z.ch} = 2.42\,kN\cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C}\cdot\sin\left(\frac{\pi\cdot cc_{batt}}{L}\right)$ $M_{z.Ed.C.end} = 9.27\,kN\cdot m$ moment due to bow imperfection at end of panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end}\cdot\frac{(h_0\cdot A_{ch})}{(2\cdot I_{eff})}$ $N_{ch.end} = 53.99\,kN$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$n_{y.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{y.C.2} = 0.09$ $n_{z.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{z.C.2} = 0.09$

Equivalent uniform moment factors: Moment_dis = "B"

Moment at start of unbraced segment: $x_{0\_0} := 0\cdot cc_{batt}$ $M_{y.A0} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{0\_0})$ $M_{y.A0} = 16.67\,kN\cdot m$

Moment at end of unbraced segment: $M_{y.max.C} = 18.5\,kN\cdot m$

$$\psi_{m.C.A} := \frac{M_{y.A0}}{\begin{cases} M_{Ed.1}\cdot 0.5 & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ M_{Ed.2}\cdot 0.5 & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}}$$ $\psi_{m.C.A} = 0.9$ range for model "A"

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 3.15\,m$ $M_{y.i}(x_{1\_2}) = 7\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 7\,kN\cdot m$

Moment at start of unbraced segment: $x_{0\_0} := \frac{L - cc_{batt}}{2} + 0\cdot cc_{batt}$ $x_{0\_0} = 2.65\,m$ $M_{y.i}(x_{0\_0}) = 6.82\,kN\cdot m$ $M_{y.B0} := M_{y.i}(x_{0\_0})$ $M_{y.B0} = 6.82\,kN\cdot m$

$\alpha_{h.C.2} := \frac{|M_{y.B0}|}{|M_{y.BB}|}$ $\alpha_{h.C.2} = 0.97$ $\psi_{m.C.B} := 1$ range for model "B"

$$C_{mLT.C.2} := \begin{cases} \max(0.6 + 0.4\cdot\psi_{m.C.A}, 0.4) & \text{if Moment\_dis = "A"} \\ (0.95 + 0.05\cdot\alpha_{h.C.2}) & \text{if Moment\_dis = "B"} \end{cases}$$ $C_{mLT.C.2} = 0.999$ $C_{my.C.2} := C_{mLT.C.2}$ $C_{mz.C.2} := 1$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$\lambda_{r.y.FB}(cc_{batt}) = 0.13$ $k_{yy.C.2} := \min\left[C_{my.C.2}\cdot(1 + 0.6\cdot\lambda_{r.y.FB}(cc_{batt})\cdot n_{y.C.2}), C_{my.C.2}\cdot(1 + 0.6\cdot n_{y.C.2})\right]$ $k_{yy.C.2} = 1.01$

$\lambda_{r.z.FB}(cc_{batt}) = 0.33$ $k_{zz.C.2} := \min\left[C_{mz.C.2}\cdot(1 + 0.6\cdot\lambda_{r.z.FB}(cc_{batt})\cdot n_{z.C.2}), C_{mz.C.2}\cdot(1 + 0.6\cdot n_{z.C.2})\right]$ $k_{zz.C.2} = 1.02$

$$k_{zy.C.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FB}(cc_{batt})}{C_{my.C.2} - 0.25}\cdot n_{z.C.2}, 1 - \frac{0.05}{C_{mLT.C.2} - 0.25}\cdot n_{z.C.2}\right)$$ $k_{zy.C.2} = 1$ $k_{yz.C.2} := k_{zz.C.2}$

Controll single C-profile between battens:

$M_{y.max.C} = 18.5\,kN\cdot m$ $M_{z.ch} = 2.42\,kN\cdot m$ $\Delta M_{z.shift} := |\Delta e_N\cdot N_{ch.end}|$ $\Delta M_{z.shift} = 0.28\,kN\cdot m$ $N_{ch.end} = 53.99\,kN$

Buckling moment resistance y-y EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C}\cdot M_{ycRk}$ $\chi_{LT.C} = 0.96$ $M_{y.Rk.C} = 60.21\,kN\cdot m$

Buckling resistance moment z-z EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk})$ $M_{z.Rk.C} = 17.43\,kN\cdot m$

Resistance buckling mode flexural buckling-: y-y $cc_{batt} = 1\,m$ $\chi_{y.FB}(cc_{batt}) = 1$ $N_{b.Rk.y.FB}(cc_{batt}) = 630.13\,kN$

Resistance buckling mode flexural buckling-: z-z $cc_{batt} = 1\,m$ $\chi_{z.FB}(cc_{batt}) = 0.95$ $N_{b.Rk.z.FB}(cc_{batt}) = 600.35\,kN$

Resistance buckling mode -torsional or torsional-flexural $cc_{batt} = 1\,m$ $\chi_{TF}(cc_{batt}, cc_{batt}) = 0.93$ $N_{b.Rk.TF}(cc_{batt}, cc_{batt}) = 588.3\,kN$

$b = 100\,\mathrm{mm}$ $h = 250\,\mathrm{mm}$ $c = 36\,\mathrm{mm}$ $t = 4\,\mathrm{mm}$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

single C-profile: Axial compression EN 1993-1-1: 6.3.1 (3): Buckling about relevant axis in mid-span or en panel of built-up member

$$\frac{\max(N_{ch.Ed}, N_{ch.end})\cdot\gamma_{M1}}{\min(\chi_{y.FB}(cc_{batt}), \chi_{z.FB}(cc_{batt}), \chi_{TF}(cc_{batt}, cc_{batt}))\cdot N_{c.Rk}} = 0.18$$ <1.0

Controll buil-up member: single C-profile (chords) in mid-span of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.Ed}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.48$$ <1.0

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.49$$ <1.0

Evading in z-z:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.5$$ <1.0

Controll buil-up member: single C-profile (chords) at end panel of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.end}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.54$$ <1.0

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.55$$ <1.0

Evading in z-z:

$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.55$$ <1.0

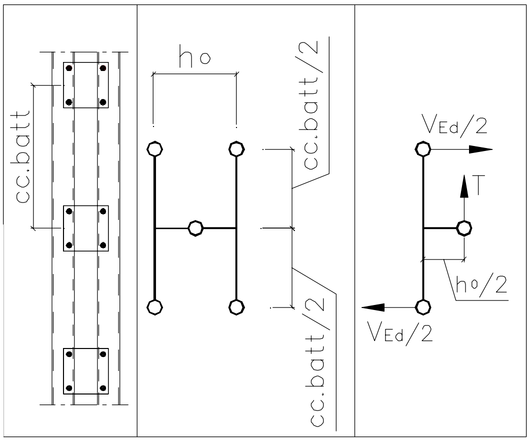
$b = 100\,mm$ $h = 250\,mm$ $c = 36\,mm$ $t = 4\,mm$ $f_{yb} = 420\,\frac{N}{mm^2}$ $f_u = 480\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Forces on battens: max shear flow in one batten:

$$T_{batt} := \frac{V_{Ed.max} \cdot cc_{batt}}{h_0 \cdot r_{CC}} \qquad T_{batt} = 29.22\,kN$$

max moment in one batten:

$$M_{batt} := \frac{T_{batt} \cdot h_0}{2} \qquad M_{batt} = 2.42\,kN \cdot m$$

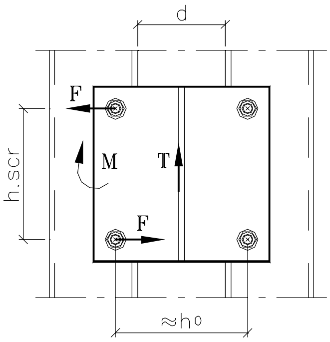


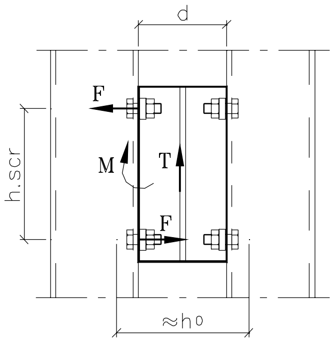
$$\lambda_{w.batt} := 0.346 \cdot \frac{h_{batt}}{t_{batt}} \cdot \sqrt{\frac{f_{yb}}{E}} \qquad f_{vb.batt} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{w.batt} \le 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{w.batt}} & \text{if } 0.83 < \lambda_{w.batt} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{w.batt}^2} & \text{if } \lambda_{w.batt} \ge 1.40 \end{cases}$$

$$V_{bbatt.Rd} := \frac{h_{batt} \cdot t_{batt} \cdot f_{vb.batt}}{\gamma_{M0}} \qquad V_{bbatt.Rd} = 193.99\,kN \qquad \frac{T_{batt}}{V_{bbatt.Rd}} = 0.15 \quad < 1{,}0$$

$$M_{Rd.batt} := \frac{f_{yb}}{\gamma_{M0}} \cdot \frac{t_{batt} \cdot h_{batt}^2}{6} \qquad M_{Rd.batt} = 11.2\,kN \cdot m \qquad \frac{M_{batt}}{M_{Rd.batt}} = 0.22 \quad < 1{,}0$$

Forces on screws:





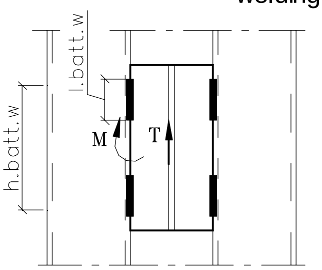
$h_{scr} := 150 \cdot mm$ $n_c := 2$ numbers of screws on one side of one PXK

$$T_{batt} = 29.22\,kN \qquad F := \frac{T_{batt} \cdot h_0}{2 \cdot h_{scr}} \qquad F = 16.11\,kN$$

$$F_{scew} := \sqrt{F^2 + \left(\frac{T_{batt}}{n_c}\right)^2}$$

$F_{scew} = 21.75\,kN$ = force on one screw
< 30 kN

Forces on weld:



welding high: $h_{batt.w} := 150 \cdot mm$ welding length: $l_{batt.w} := 2 \cdot 40 \cdot mm$ welding t=: $a_{batt.w} := 4 \cdot mm$

$$\sigma_{batt.w} := \frac{3 \cdot T_{batt} \cdot h_0}{a_{batt.w} \cdot h_{batt.w}^2} \qquad \tau_{batt.w} := \frac{T_{batt}}{l_{batt.w} \cdot a_{batt.w}}$$

$$\sigma_{V.batt.w} := \sqrt{\sigma_{batt.w}^2 + 3 \cdot \tau_{batt.w}^2} \qquad \sigma_{V.batt.w} = 225.77\,\frac{N}{mm^2}$$ = stress in one weld

G. Štítové sloupy B1/1, 8 C1/1, 8 R15

Geometrie:

Zatěžovací šířka sloupku L_w= **5.31** m
Rozpon L= **7.50** m

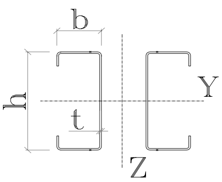
Zatížení větrem: ČSN EN 1991-1-4 ()

Kategorie terénu: II. Oblasti s nízkou vegetací jako je tráva nebo izolované překážky (stromy, budovy)
Charakteristická hodnota rychlosti větru: v_{bo}= 25.0 m/s
Maximální dynamický tlak q_p (z)= 0.936 kN/m^2

Součinitel vnějšího tlaku (tlak) C_{pe}=	**0.96**		Součinitel vnějšího tlaku (sání) C_{pe}=	**1.1**	
Zatížení větrem (tlak) w_e=	0.90	kN/m^2	Zatížení větrem (sání) w_e=	0.98	kN/m^2
Normové zatížení větrem (tlak) q_n=	**4.77**	kN/m	Normové zatížení větrem (sání) q_n=	**5.22**	kN/m
Součinitel zatížení g_w=	0.2		Součinitel zatížení g_w=	0.2	
Výpočtové zatížení větrem (tlak) q_d=	**0.95**	kN/m	Výpočtové zatížení větrem (sání) q_d=	**1.04**	kN/m

Charakteristika profilu:

Průřez profilem = **Dvojitý** Profil = **C250X5**



Výška profilu h=	250	mm	Momentová únosnost M_y=	76.20	kNm
Šířka profilu b=	100	mm	Smyková únosnost V_{wy}=	297.05	kN
Tloučťka t=	5	mm	Smyk v podpoře R_{wy}=	135.51	kN
Mez kluzu oceli f_u=	480	MPa	Efekt. moment setrvačnosti I_{yeff}=	23020000	mm4
Mez pevnosti oceli f_b=	420	MPa	Moment setrvačnosti I_y=	23600000	mm4

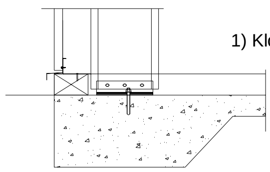
Součinitelé materiálu:

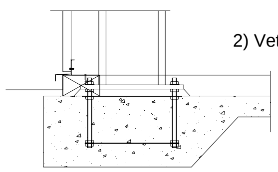
Country = **Czech**
Únosnost průřezu γ_{M0}= 1.00
Únosnost při vzpěru γ_{M1}= 1.00
Únosnost oslabeného průřezu γ_{M2}= 1.25

Deformace:

Limit deformace 1/ **250**
Skutečný poměr 337

Kotvení do spodní stavby:


1) Kloubové kotvení


2) Vetknuté kotvení

Vetknuté kotvení

Předběžné posouzení:

Vetknutý sloup

Sání pro C_{pe}= 1.05 Tlak pro C_{pe}= 0.96

Ohybový moment ve vetknutí

$M_{Sd}=1/2.q.L^2$ $M_{Sd,s}$= **29.34** kNm $M_{Sd,s}$= **26.83** kNm My = **152.40** kNm

PODMÍNKA VYHOVUJE **PODMÍNKA VYHOVUJE**

Podporová reakce

R=q.L $R_{Sd,s}$= **7.82** kN $R_{Sd,s}$= **7.15** kN Ry = **271.02** kN

PODMÍNKA VYHOVUJE **PODMÍNKA VYHOVUJE**

Double gable pillar B1/1, 8 C1/1, 8 R15

$h \equiv 250 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 39 \cdot mm$ $t \equiv 5 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $cc_{batt} := 1500 \cdot mm$ distance between battens

$b_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $\gamma_{M1} \equiv 1.0$

$h_{batt} := 200 \cdot mm$

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 4.951 \times 10^3 \, mm^2$

$I_{zz} := 2 \cdot \left[I_z + A_g \cdot \left(e_1 + \frac{d}{2} \right)^2 \right]$ $I_{zz} = 4.15 \times 10^7 \, mm^4$

$M_{yy.cRk} := 2 \cdot M_{ycRk}$ $f_{yb} = 124.5 \frac{N}{mm^2}$

$M_{yy.cRk} = 46.82 \, kN \cdot m$

single C: $M_{ycRk} = 23.41 \, kN \cdot m$ $f_u = 136.95 \frac{N}{mm^2}$

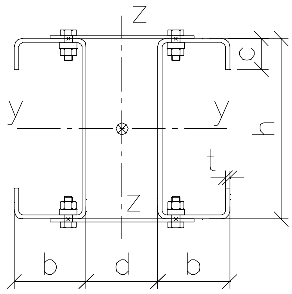
$I_{yy} := 2 \cdot I_y$ $I_{yy} = 4.68 \times 10^7 \, mm^4$

$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 3.82 \times 10^5 \, mm^3$

$N_{cc.Rk} = 599.18 \, kN$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 3.76 \times 10^5 \, mm^3$



$W_{zz} := \frac{I_{zz}}{b + \frac{d}{2}}$ $W_{zz} = 2.77 \times 10^5 \, mm^3$ $i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 97.26 \, mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 91.54 \, mm$

$I_{TT} = 42333.33 \, mm^4$ torsion_plate_pillar = "NO"

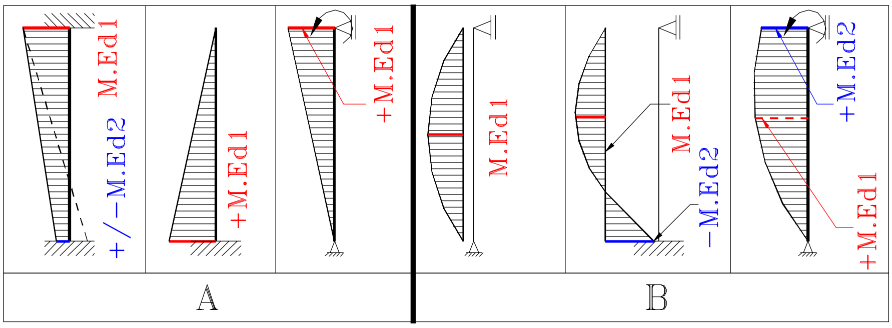
Stresses, global geometry and buckling lengths:

$M_{Ed.1} := 27 \cdot kN \cdot m$ Moment y-y

$M_{Ed.suc.1} := 29 \cdot kN \cdot m$ Moment y-y acting as suction

$M_{Ed.1.z} := 0 \cdot kN \cdot m$ Moment z-z

$N_{Ed} := 11 \cdot kN$



$V_{Ed} := 9 \cdot kN$ --> $n_{cross} := 4$ =Numbers of holes in **ONE** web

$d_0 := 12.5 \cdot mm$

Moment_dis := "A" $M_{Ed.2} := 0 \cdot kN \cdot m$

shift of neutral axis for member in compression: $\Delta e_N = 1.44 \, mm$

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = 0.02 \, kN \cdot m$ $M_{z.Ed} := \Delta M_{z.shift}$

Buckling lengths:

$L := 7.5 \cdot m$ Length of pillar

$cc_{wallZ} := 1L$ = Distance between wall-Z-profile

$L_y := 2 \cdot L$ Flexural buckling axis y-y

$L_z := cc_{wallZ}$ Flexural buckling axis z-z (= Distance between wall-Z-profile)

$L_{LT} := cc_{wallZ}$ Distance flange bracings - LTB outer flange in compression for wind acting as pressure

$L_{LT.suc} := 0.5 \cdot L$ Distance flange bracings - LTB inner flange in compression for wind acting as suction

$L_T := cc_{batt}$ Torsional buckling single C-profile between battens.

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 125\,\frac{N}{mm^2}$ $f_u = 137\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Flexural buckling resistance - axis y-y and z-z EN 1993-1-3: 6.2.2 and EN 1993-1-1: 6.3.1:

Buckling curve for double C-section: EN 1993-1-3 table 6.3 about:

y-y: a EN1993-1-1,table 6.1: α =0,21 z-z: b EN1993-1-1,table 6.1: α =0,34

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about y-y

$$\lambda_1 := \pi\cdot\sqrt{\frac{E}{f_{yb}}} \quad \lambda_1 = 65.28 \quad \lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \quad \lambda_{r.y.FBcc} = 2.33$$

Imperfection factor α relating to bucking curve a $\alpha_y := 0.21$

$$\phi_{y.FB} := 0.5\cdot\left[1 + \alpha_y\cdot(\lambda_{r.y.FBcc} - 0.2) + \lambda_{r.y.FBcc}^2\right] \quad \phi_{y.FB} = 3.44$$

$$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^2 - \lambda_{r.y.FBcc}^2}}, 1\right) \quad \chi_{y.FBcc} = 0.17$$

Design buckling resistance for buckling mode -flexural buckling-: y-y

$$N_{b.Rd.y.FBcc} := \frac{\chi_{y.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \quad N_{b.Rd.y.FBcc} = 100.5\,kN$$

Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about z-z

$$\lambda_1 = 65.28 \quad \lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \quad \lambda_{r.z.FBcc} = 1.24$$

Imperfection factor α relating to bucking curve b $\alpha_z := 0.34$

$$\phi_{z.FB} := 0.5\cdot\left[1 + \alpha_z\cdot(\lambda_{r.z.FBcc} - 0.2) + \lambda_{r.z.FBcc}^2\right] \quad \phi_{z.FB} = 1.44$$

$$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^2 - \lambda_{r.z.FBcc}^2}}, 1\right) \quad \chi_{z.FBcc} = 0.46$$

Design buckling resistance for buckling mode -flexural buckling-: z-z

$$N_{b.Rd.z.FBcc} := \frac{\chi_{z.FBcc}\cdot 2\cdot A_{eff}\cdot f_{yb}}{\gamma_{M1}} \quad N_{b.Rd.z.FBcc} = 274.6\,kN$$

Lateral-torsional buckling resistance for uniform member in bending EN 1993-1-3: 6.2.4 and EN 1993-1-1: 6.3.2:

Procedure to calculate the elastic critical buckling moment M_{cr}, based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints, is given neither in EN 1993-1-1 nor in EN 1993-1-3. Any appropriate calculation method can be used, here the procedure given in German ENV 1993-1-1 Annex F.

Moment_dis = "A" $L_{LT} = 7.5\,m$ $k_{M.cr} := 1.0$ (hinged at ends) $k_{w.M.cr} := 1.0$ (no special wrap restraints at ends)

For pillar with end moments: according to picture "A":

$$\psi_{md.A} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases} \quad \psi_{md.A} = 0$$

$$C_{1.A} := \min\left(1.88 - 1.40\cdot\psi_{md.A} + 0.52\cdot\psi_{md.A}^2, 2.7\right) \quad C_{1.A} = 1.88$$

$$M_{cr.A} := C_{1.A}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \quad M_{cr.A} = 50.91\,kN\cdot m$$

For pillar with moment distribution according to picture "B": $C_{1.B} := 1.132$

$$M_{cr.B} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \quad M_{cr.B} = 30.66\,kN\cdot m$$

Elastic critical moment for lateral-torsional buckling

based on gross cross sectional properties, taking into account the loading conditions, real moment distribution and lateral restraints for double C-section:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if Moment\_dis = "A"} \\ M_{cr.B} & \text{if Moment\_dis = "B"} \end{cases} \quad M_{cr} = 51\,kN\cdot m$$

Relative slenderness: $\lambda_{rLT} := \sqrt{\frac{2\cdot W_{eff.y.1}\cdot f_{yb}}{M_{cr}}}$ $\lambda_{rLT} = 0.96$ $\alpha_{LT} := 0.34$ $\phi_{LT} := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{rLT} - 0.2) + \lambda_{rLT}^2\right]$ $\phi_{LT} = 1.09$

Reduction buckling factor: $\chi_{LT} := \min\left(\frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{rLT}^2}}, 1\right)$ $\chi_{LT} = 0.62$

Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

Desing buckling moment resistance double profile based on effective section modulus W_{eff}.

$$M_{b.Rd} := \frac{\chi_{LT}\cdot M_{yy.cRk}}{\gamma_{M1}} \quad M_{b.Rd} = 29.18\,kN\cdot m$$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 125\dfrac{N}{mm^2}$ $f_u = 137\dfrac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

For pillar when wind acting as suction (moment from line load):

$$M_{cr.suc} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT.suc})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT.suc})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.suc} = 98.85\,kN\cdot m$$

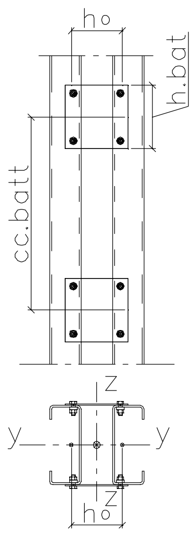
Relative slenderness: $\lambda_{rLT.suc} := \sqrt{\dfrac{2\cdot W_{eff.y.1}\cdot f_{yb}}{M_{cr.suc}}}$ $\lambda_{rLT.suc} = 0.69$ $\phi_{LT.suc} := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{rLT.suc} - 0.2) + \lambda_{rLT.suc}^2\right]$ $\phi_{LT.suc} = 0.82$

Reduction buckling factor: $\chi_{LT.suc} := \min\left(\dfrac{1}{\phi_{LT.suc} + \sqrt{\phi_{LT.suc}^2 - \lambda_{rLT.suc}^2}}, 1\right)$ $\chi_{LT.suc} = 0.79$

Design buckling resistance moment EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

Desing buckling moment resistance double profile for wind acting as suction $M_{b.Rd.suc} := \dfrac{\chi_{LT.suc}\cdot M_{yy.cRk}}{\gamma_{M1}}$ $M_{b.Rd.suc} = 37\,kN\cdot m$

Check Uniform built-up member EN 1993-1-1: 6.4



bow imperfection: $e_0 := \dfrac{L}{500}$ $\text{if}\left(\dfrac{cc_{batt}}{i_z} \le 70, \text{"OK"}, \text{"Shorter-cc.bat"}\right) = \text{"OK"}$

non-dimensional slenderness: $\lambda_1 = 65.28$

Effective second moment of area of battened built-up member:

$A_{ch} := A_g$: area of one chord

$h_0 := d + 2\cdot e_1$ $h_0 = 167\,mm$ distance centroids of chords

$I_{ch} := I_z$: I of one chord

$I_{l.CC} := 0.5\cdot h_0^2\cdot A_{ch} + 2\cdot I_{ch}$ I built-up member $i_{0.CC} := \sqrt{\dfrac{I_{l.CC}}{2\cdot A_{ch}}}$ $\lambda_{CC} := \dfrac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 81.94$

EN 1993-1-1: table 6.8
Efficiency factor:

$$\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \ge 150 \\ \left(2 - \dfrac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \le 75 \end{cases}$$

$\mu_{CC} = 0.91$

$r_{CC} := 2$

$I_{l.CC} = 4.15 \times 10^7\,mm^4$

$I_{eff} := 0.5\cdot h_0^2\cdot A_{ch} + 2\cdot\mu_{CC}\cdot I_{ch}$ $I_{eff} = 4.08 \times 10^7\,mm^4$ effective I of built-up member

Shear stiffness EN 1993-1-1:6.4.3.1:(2) $I_b := \dfrac{t_{batt}\cdot h_{batt}^3}{12}$ I of batten $n_{batt} := \dfrac{L}{cc_{batt}}$ $n_{batt} = 5$ number of planes of lacings

$$S_v := \min\left[\left[\frac{24\cdot E\cdot I_{ch}}{cc_{batt}^2\cdot\left(1 + \dfrac{2\cdot I_{ch}}{n_{batt}\cdot I_b}\cdot\dfrac{h_0}{cc_{batt}}\right)}\right], \left(\frac{2\cdot\pi^2\cdot E\cdot I_{ch}}{cc_{batt}^2}\right)\right] \qquad S_v = 1642\,kN$$

effective critical force of built-up member:

$N_{cr.CC} := \dfrac{\pi^2\cdot E\cdot I_{eff}}{L_z^2}$ $N_{cr.CC} = 385.24\,kN$

Maximum moment in middle of built-up member: EN 1993-1-1:6.4.1 (6):

$M_{z.Ed.I} := |M_{Ed.1.z}|$

moment z-z without second order effects

moment with second order effects $M_{z.Ed.C} := \dfrac{N_{Ed}\cdot e_0 + M_{z.Ed.I}}{1 - \dfrac{N_{Ed}\cdot\gamma_{M1}}{N_{cr.CC}} - \dfrac{N_{Ed}\cdot\gamma_{M1}}{S_v}}$ $M_{z.Ed.C} = 0.17\,kN\cdot m$

Compression force in one member: EN 1993-1-1:6.4.1 (6) Compression force in built-up member taking accout to second order effects:

$X_d := \begin{cases} 1 & \text{if } d \ge 8\cdot mm \\ 0 & \text{otherwise} \end{cases}$ $N_{ch.Ed} := \dfrac{N_{Ed}}{r_{CC}} + \dfrac{|M_{z.Ed.C}|\cdot h_0\cdot A_{ch}\cdot X_d}{2\cdot I_{eff}}$ $N_{ch.Ed} = 6.37\,kN$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 125\frac{N}{mm^2}$ $f_u = 137\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Interaction formula according to EN 1993-1-1: 6.3.3 (4) eqv.(6.61+6.62):
The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

Equivalent uniform moment factors: EN 1993-1-1, Annex B, Table B.3
Moment distribution like shown on figure: $Moment\_dis = "A"$ $\psi_{md.A} = 0$ $\psi_{m.B} := 0$

$\alpha_{h.2} = 0$ $\alpha_{s.2} = 0$ $\psi_{md.z} := 0$ $\alpha_{h.2.z} = 0$ $\psi_{md.suc} := 0$ $\alpha_{h.2.suc} = 0$

$C_{my.2} = 0.6$ $C_{my.2.suc} = 0.95$ $C_{mz.2} = 0.95$ $C_{mLT.2} = 0.6$

Interaction factors kij for members susceptible to torsional deformations: EN 1993-1-1, Annex B, Table B.2

$n_{y.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}}$ $n_{y.2} = 0.11$ $n_{z.2} := \frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}}$ $n_{z.2} = 0.04$

$k_{yy.2} := \min\left[C_{my.2}\cdot\left(1 + 0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}\right), C_{my.2}\cdot\left(1 + 0.6\cdot n_{y.2}\right)\right]$ $k_{yy.2} = 0.64$

$k_{zy.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FBcc}}{C_{mLT.2} - 0.25}\cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25}\cdot n_{z.2}\right)$ $k_{zy.2} = 0.99$

$k_{zz.2} := \min\left[C_{mz.2}\cdot\left(1 + 0.6\cdot\lambda_{r.z.FBcc}\cdot n_{z.2}\right), C_{mz.2}\cdot\left(1 + 0.6\cdot n_{z.2}\right)\right]$ $k_{zz.2} = 0.97$ $k_{yz.2} := k_{zz.2}$

$k_{yy.2.suc} := \min\left[C_{my.2.suc}\cdot\left(1 + 0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}\right), C_{my.2.suc}\cdot\left(1 + 0.6\cdot n_{y.2}\right)\right]$ $k_{yy.2.suc} = 1.01$

$I_{zz.eff} := 2\cdot I_z + A_g\cdot\left(e_1 + \frac{d}{2}\right)^2 + A_{eff}\cdot\left(e_N + \frac{d}{2}\right)^2$ $W_{zz.eff} := \frac{I_{zz.eff}}{b + \frac{d}{2}}$ $I_{zz.eff} = 3.95\times 10^7\,mm^4$ $M_{zz.cRk} := W_{zz.eff}\cdot f_{yb}$

$W_{zz.eff} = 2.63\times 10^5\,mm^3$

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{Ed}\cdot\gamma_{M0}}{2\cdot N_{c.Rk}} + \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.suc.1}\right|\right)\cdot\gamma_{M0}}{2\cdot M_{ycRk}} + \frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M0}}{M_{zz.cRk}} = 0.64$$ <1.0

Double CC-profile: Combined M + N + V acc to EN 1993-1-3: 6.1.10

$\frac{V_{Ed}\cdot\gamma_{M1}}{2\cdot V_{bh.Rk.pillar}} = 0.06$ < 1.0 according to EN 1993-1-3: 6.1.10 $M\_N\_V_{EC} = 0$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as pressure

Evading in y-y: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} + k_{yy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{yz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.71$$ <1.0

Evading in z-z: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} + k_{zy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.cRk}} + k_{zz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.97$$ <1.0

Combined bending an axial compression EN 1993-1-1: 6.3.3 (4): for wind acting as suction

Evading in y-y: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk}} + k_{yy.2.suc}\cdot\frac{\left|M_{Ed.suc.1}\right|\cdot\gamma_{M1}}{\chi_{LT.suc}\cdot M_{yy.cRk}} + k_{yz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.91$$ <1.0

Evading in z-z: $$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk}} + k_{zy.2}\cdot\frac{\left|M_{Ed.suc.1}\right|\cdot\gamma_{M1}}{\chi_{LT.suc}\cdot M_{yy.cRk}} + k_{zz.2}\cdot\frac{\left|M_{z.Ed.C}\right|\cdot\gamma_{M1}}{M_{zz.cRk}} = 0.82$$ <1.0

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 125\,\frac{N}{mm^2}$ $f_u = 137\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords and battens are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1).

Procedure to calculate the elastic critical buckling moment M_{cr} for singly symmetric sections is taken from:"The North American Specification for the Design of Cold-Formed Steel Structural Members" 2001.

$M_{y.max.C} := \max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.suc.1}\right|\right)\cdot 0.5$ $M_{y.max.C} = 14.5\,kN\cdot m$

Moment at quarter point of unbraced segment: $x_{1\_4} := 0.25\cdot cc_{batt}$ $M_{y.AA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_4})$ $M_{y.AA} = 11.47\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := 0.5\cdot cc_{batt}$ $M_{y.BA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{1\_2})$ $M_{y.BA} = 12.15\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := 0.75\cdot cc_{batt}$ $M_{y.CA} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{3\_4})$ $M_{y.CA} = 12.82\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot M_{y.max.C}}{2.5\cdot M_{y.max.C} + 3\cdot\left|M_{y.AA}\right| + 4\cdot\left|M_{y.BA}\right| + 3\cdot\left|M_{y.CA}\right|}$$

$y_{0.cr} := (y_M + e_{1c.rc})\cdot -1$ $z_{0.cr} := 0\cdot mm$ (coord. shear centre)

$r_0 := \sqrt{i_y^2 + i_z^2 + y_{0.cr}^2 + z_{0.cr}^2}$ Polar radius of gyration about shear center

$$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot cc_{batt}}{i_z}\right)^2} \qquad \sigma_{ez} = 331.71\,\frac{N}{mm^2}$$

$$\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{cc_{batt}^2}\right) \qquad \sigma_T = 275.08\,\frac{N}{mm^2}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_A} := C_{bA}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 112.6\,kN\cdot m$

Relative slenderness:

$$\lambda_{rLT.C\_A} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_A}}} & \text{if } M_{e.cr.C\_A} > 0\\ 0.2 & \text{if } M_{e.cr.C\_A} = 0\end{cases}$$

$\lambda_{rLT.C\_A} = 0.46$ Imperfection factor α rel. to bucking curve b: $\alpha_{LT.C} := 0.34$

$$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right] \quad \chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right) \quad \chi_{LT.C\_A} = 0.9$$

$\phi_{LT.C\_A} = 0.65$

For moment distribution accord. to modell B: $q_{wind\_pillar} := \frac{\left|M_{Ed.1}\cdot 0.5\right|\cdot 8}{L^2}$ $M_{y.i}(x_i) := \frac{q_{wind\_pillar}}{2}\cdot(L\cdot x_i - x_i^2)$

Moment at quater point of unbraced segment: $x_{1\_4} := \frac{L - cc_{batt}}{2} + \frac{1}{4}\cdot cc_{batt}$ $x_{1\_4} = 3.38\,m$ $M_{y.i}(x_{1\_4}) = 13.37\,kN\cdot m$ $M_{y.AB} := M_{y.i}(x_{1\_4})$ $M_{y.AB} = 13.37\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 3.75\,m$ $M_{y.i}(x_{1\_2}) = 13.5\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 13.5\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := \frac{L - cc_{batt}}{2} + \frac{3}{4}\cdot cc_{batt}$ $x_{3\_4} = 4.13\,m$ $M_{y.i}(x_{3\_4}) = 13.37\,kN\cdot m$ $M_{y.CB} := M_{y.i}(x_{3\_4})$ $M_{y.CB} = 13.37\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot\left(\left|M_{Ed.1}\cdot 0.5\right|\right)}{2.5\cdot\left(\left|M_{Ed.1}\cdot 0.5\right|\right) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := C_{bB}\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 98.5\,kN\cdot m$

Relative slenderness:

$$\lambda_{rLT.C\_B} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0\\ 0.2 & \text{if } M_{e.cr.C\_B} = 0\end{cases}$$

$\lambda_{rLT.C\_B} = 0.49$

$$\phi_{LT.C\_B} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_B} - 0.2) + \lambda_{rLT.C\_B}^2\right] \quad \chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right) \quad \chi_{LT.C\_B} = 0.89$$

$\phi_{LT.C\_B} = 0.67$

Moment distribution: Moment_dis = "A"

$$\chi_{LT.C} := \begin{cases}\chi_{LT.C\_A} & \text{if Moment\_dis = "A"}\\ \chi_{LT.C\_B} & \text{if Moment\_dis = "B"}\end{cases} \qquad \chi_{LT.C} = 0.9$$

Stresses on one member profile at mid-span from above: chord force from above: $N_{ch.Ed} = 6.37\,kN$ *max moment y-y:* $M_{y.max.C} = 14.5\,kN\cdot m$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 125\frac{N}{mm^2}$ $f_u = 137\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Chord at end panel: highest shear force: $r_{CC} = 2$ $V_{Ed.max} := \pi\cdot\frac{M_{z.Ed.C}}{L}$ $V_{Ed.max} = 0.07\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 0.04\,kN$

$V_{bb.Rd} = 68.29\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 5.246\times10^{-4}$ The shear force is negligible

Chord at end panel: "corner" moment: $M_{z.ch} := \frac{V_{Ed.max}}{r_{CC}}\cdot\frac{cc_{batt}}{2}$ *maximum moment z-z:* $M_{z.ch} = 0.03\,kN\cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C}\cdot\sin\left(\frac{\pi\cdot cc_{batt}}{L}\right)$ $M_{z.Ed.C.end} = 0.1\,kN\cdot m$ moment due to bow imperfection at end of panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end}\cdot\frac{(h_0\cdot A_{ch})}{(2\cdot I_{eff})}$ $N_{ch.end} = 6.01\,kN$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$n_{y.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{y.C.2} = 0.02$ $n_{z.C.2} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}}$ $n_{z.C.2} = 0.02$

Equivalent uniform moment factors: Moment_dis = "A"

Moment at start of unbraced segment: $x_{0\_0} := 0\cdot cc_{batt}$ $M_{y.A0} := 0.5\cdot M_{y.Ed.ccbatt\_A}(x_{0\_0})$ $M_{y.A0} = 10.8\,kN\cdot m$

Moment at end of unbraced segment: $M_{y.max.C} = 14.5\,kN\cdot m$

$$\psi_{m.C.A} := \frac{M_{y.A0}}{\begin{cases} M_{Ed.1}\cdot0.5 & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ M_{Ed.2}\cdot0.5 & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}}$$ $\psi_{m.C.A} = 0.8$ range for model "A"

Moment at centerline of unbraced segment: $x_{1\_2} := \frac{L - cc_{batt}}{2} + \frac{1}{2}\cdot cc_{batt}$ $x_{1\_2} = 3.75\,m$ $M_{y.i}(x_{1\_2}) = 13.5\,kN\cdot m$ $M_{y.BB} := M_{y.i}(x_{1\_2})$ $M_{y.BB} = 13.5\,kN\cdot m$

Moment at start of unbraced segment: $x_{0\_0} := \frac{L - cc_{batt}}{2} + 0\cdot cc_{batt}$ $x_{0\_0} = 3\,m$ $M_{y.i}(x_{0\_0}) = 12.96\,kN\cdot m$ $M_{y.B0} := M_{y.i}(x_{0\_0})$ $M_{y.B0} = 12.96\,kN\cdot m$

$\alpha_{h.C.2} := \frac{|M_{y.B0}|}{|M_{y.BB}|}$ $\alpha_{h.C.2} = 0.96$ $\psi_{m.C.B} := 1$ range for model "B"

$$C_{mLT.C.2} := \begin{cases} \max(0.6 + 0.4\cdot\psi_{m.C.A}, 0.4) & \text{if Moment\_dis = "A"} \\ (0.95 + 0.05\cdot\alpha_{h.C.2}) & \text{if Moment\_dis = "B"} \end{cases}$$ $C_{mLT.C.2} = 0.920$ $C_{my.C.2} := C_{mLT.C.2}$ $C_{mz.C.2} := 1$

Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$\lambda_{r.y.FB}(cc_{batt}) = 0.23$ $k_{yy.C.2} := \min\left[C_{my.C.2}\cdot(1 + 0.6\cdot\lambda_{r.y.FB}(cc_{batt})\cdot n_{y.C.2}), C_{my.C.2}\cdot(1 + 0.6\cdot n_{y.C.2})\right]$ $k_{yy.C.2} = 0.92$

$\lambda_{r.z.FB}(cc_{batt}) = 0.6$ $k_{zz.C.2} := \min\left[C_{mz.C.2}\cdot(1 + 0.6\cdot\lambda_{r.z.FB}(cc_{batt})\cdot n_{z.C.2}), C_{mz.C.2}\cdot(1 + 0.6\cdot n_{z.C.2})\right]$ $k_{zz.C.2} = 1.01$

$$k_{zy.C.2} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FB}(cc_{batt})}{C_{my.C.2} - 0.25}\cdot n_{z.C.2}, 1 - \frac{0.05}{C_{mLT.C.2} - 0.25}\cdot n_{z.C.2}\right)$$ $k_{zy.C.2} = 1$ $k_{yz.C.2} := k_{zz.C.2}$

Controll single C-profile between battens:

$M_{y.max.C} = 14.5\,kN\cdot m$ $M_{z.ch} = 0.03\,kN\cdot m$ $\Delta M_{z.shift} := |\Delta e_N\cdot N_{ch.end}|$ $\Delta M_{z.shift} = 0.01\,kN\cdot m$ $N_{ch.end} = 6.01\,kN$

Buckling moment resistance y-y EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C}\cdot M_{ycRk}$ $\chi_{LT.C} = 0.9$ $M_{y.Rk.C} = 21.14\,kN\cdot m$

Buckling resistance moment z-z EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk})$ $M_{z.Rk.C} = 6.77\,kN\cdot m$

Resistance buckling mode flexural buckling-: y-y $cc_{batt} = 1.5\,m$ $\chi_{y.FB}(cc_{batt}) = 0.99$ $N_{b.Rk.y.FB}(cc_{batt}) = 296.09\,kN$

Resistance buckling mode flexural buckling-: z-z $cc_{batt} = 1.5\,m$ $\chi_{z.FB}(cc_{batt}) = 0.84$ $N_{b.Rk.z.FB}(cc_{batt}) = 250.17\,kN$

Resistance buckling mode -torsional or torsional-flexural $cc_{batt} = 1.5\,m$ $\chi_{TF}(cc_{batt}, cc_{batt}) = 0.8$ $N_{b.Rk.TF}(cc_{batt}, cc_{batt}) = 238.3\,kN$

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 125\,\frac{N}{mm^2}$ $f_u = 137\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

single C-profile: Axial compression EN 1993-1-1: 6.3.1 (3): Buckling about relevant axis in mid-span or en panel of built-up member

$$\frac{\max(N_{ch.Ed}, N_{ch.end})\cdot\gamma_{M1}}{\min(\chi_{y.FB}(cc_{batt}), \chi_{z.FB}(cc_{batt}), \chi_{TF}(cc_{batt}, cc_{batt}))\cdot N_{c.Rk}} = 0.03$$ <1.0

Controll buil-up member: single C-profile (chords) in mid-span of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.Ed}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.64$$ <1.0

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.66$$ <1.0

Evading in z-z:

$$\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(|\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.71$$ <1.0

Controll buil-up member: single C-profile (chords) at end panel of member:

Combined bending an axial compression EN 1993-1-3: 6.1.9 (1):

$$\frac{N_{ch.end}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.64$$ <1.0

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4):

Evading in y-y:

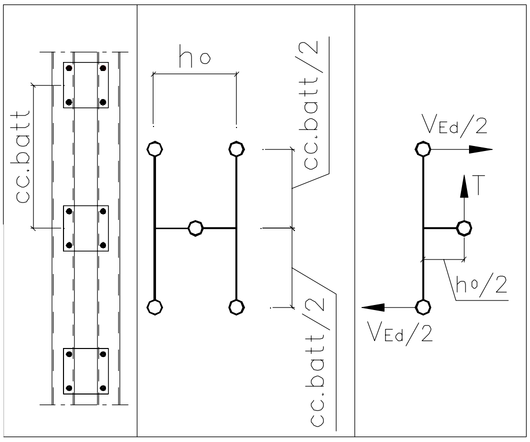
$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{yy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.66$$ <1.0

Evading in z-z:

$$\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(cc_{batt})\cdot N_{c.Rk}} + k_{zy.C.2}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C.2}\cdot\frac{(M_{z.ch} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.71$$ <1.0

$b = 100\,mm$ $h = 250\,mm$ $c = 39\,mm$ $t = 5\,mm$ $f_{yb} = 125\dfrac{N}{mm^2}$ $f_u = 137\dfrac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Forces on battens: max shear flow in one batten:



$$T_{batt} := \frac{V_{Ed.max} \cdot cc_{batt}}{h_0 \cdot r_{CC}} \qquad T_{batt} = 0.32\,kN$$

max moment in one batten:

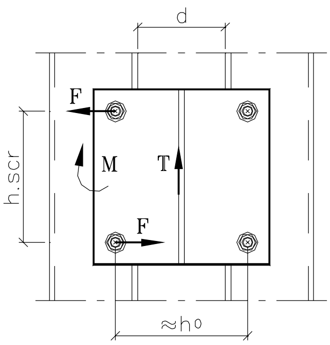
$$M_{batt} := \frac{T_{batt} \cdot h_0}{2} \qquad M_{batt} = 0.03\,kN \cdot m$$

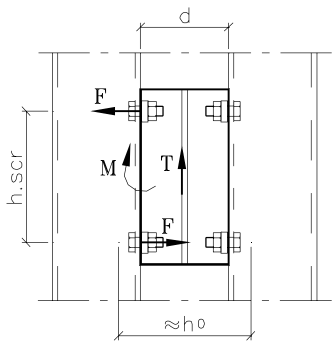
$$\lambda_{w.batt} := 0.346 \cdot \frac{h_{batt}}{t_{batt}} \cdot \sqrt{\frac{f_{yb}}{E}} \qquad f_{vb.batt} := \begin{cases} \dfrac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{w.batt} \le 0.83 \\ 0.48 \cdot \dfrac{f_{yb}}{\lambda_{w.batt}} & \text{if } 0.83 < \lambda_{w.batt} < 1.40 \\ 0.67 \cdot \dfrac{f_{yb}}{\lambda_{w.batt}^{2}} & \text{if } \lambda_{w.batt} \ge 1.40 \end{cases}$$

$$V_{bbatt.Rd} := \frac{h_{batt} \cdot t_{batt} \cdot f_{vb.batt}}{\gamma_{M0}} \qquad V_{bbatt.Rd} = 57.42\,kN \qquad \frac{T_{batt}}{V_{bbatt.Rd}} = 0.01 \quad < 1,0$$

$$M_{Rd.batt} := \frac{f_{yb}}{\gamma_{M0}} \cdot \frac{t_{batt} \cdot h_{batt}^{2}}{6} \qquad M_{Rd.batt} = 3.32\,kN \cdot m \qquad \frac{M_{batt}}{M_{Rd.batt}} = 0.01 \quad < 1,0$$

Forces on screws:





$h_{scr} := 150 \cdot mm$ $n_c := 2$ numbers of screws on one side of one PXK

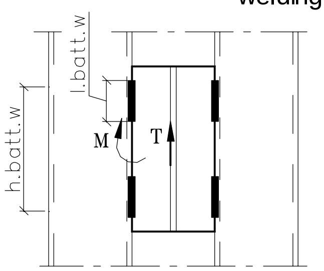
$$T_{batt} = 0.32\,kN \qquad F := \frac{T_{batt} \cdot h_0}{2 \cdot h_{scr}} \qquad F = 0.18\,kN$$

$$F_{scew} := \sqrt{F^2 + \left(\frac{T_{batt}}{n_c}\right)^2}$$

$F_{scew} = 0.24\,kN$ = force on one screw

< 30 kN

Forces on weld:



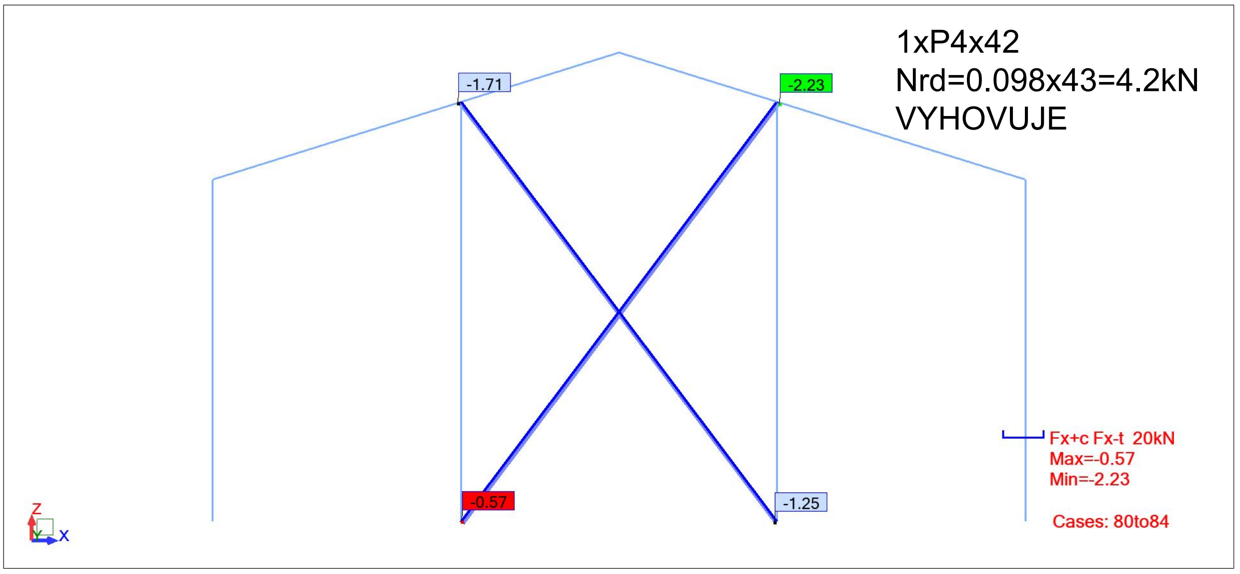
welding high: $h_{batt.w} := 150 \cdot mm$ welding length: $l_{batt.w} := 2 \cdot 40 \cdot mm$ welding t=: $a_{batt.w} := 4 \cdot mm$

$$\sigma_{batt.w} := \frac{3 \cdot T_{batt} \cdot h_0}{a_{batt.w} \cdot h_{batt.w}^{2}} \qquad \tau_{batt.w} := \frac{T_{batt}}{l_{batt.w} \cdot a_{batt.w}}$$

$$\sigma_{V.batt.w} := \sqrt{\sigma_{batt.w}^{2} + 3 \cdot \tau_{batt.w}^{2}} \qquad \sigma_{V.batt.w} = 2.5\,\frac{N}{mm^2}$$

= stress in one weld

View - FX; Cases: 80to84
1



Redukční součinitele pro ocelové prvky

Profil Plech tl. 4mm

Čas t = **15** **min.**

Spočtená teplota T_{ep} = **707.017 °C**

Pozinkovaný: PRAVDA

Ohřev ze 3 stran: NEPRAVDA

Uvažovat zastínění: NEPRAVDA

$k_{y\Theta}$ red. součinitel na mez kluzu

$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu

$k_{E\Theta}$ red. součinitel pro linerní pružnou část

$k_{b\Theta}$ red. součinitel pro šrouobvé spoje

$k_{w\Theta}$ red. součinitel pro svary

Součinitelé pro průřezy 1-3			
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
20	1	1	1
100	1	1	1
200	1	0.807	0.9
300	1	0.613	0.8
400	1	0.42	0.7
500	0.78	0.36	0.6
600	0.47	0.18	0.31
700	0.23	0.075	0.13
800	0.11	0.05	0.09
Teplota nosníku			
707.0	**0.222**	**0.073**	**0.127**

Souč. pro 4. tř. za studena		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$
20	1	1
100	1	1
200	0.89	0.9
300	0.78	0.8
400	0.65	0.7
500	0.53	0.6
600	0.3	0.31
700	0.13	0.13
800	0.07	0.09
Teplota nosníku		
707.0	**0.126**	**0.127**

Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$
20	1	1
100	0.968	1
150	0.952	1
200	0.935	1
300	0.903	1
400	0.775	0.876
500	0.55	0.627
600	0.22	0.378
700	0.1	0.13
800	0.067	0.074
Teplota nosníku		
707.0	**0.098**	**0.126**