REV.	PŘEDMĚT ZMĚN	DATUM	PODPIS

VŠEOBECNÉ POKYNY NA
A-CZ1138-01

VYPRACOVAL	KONTROLOVAL	ZODP. PROJEKTANT
Lenka Burgerová	Jaroslav Kosinka	Jaroslav Kosinka

MÍSTO STAVBY: K.Ú. FRÝDEK, P.Č. 5319 /73, 5319 /238

O. Ú.: FRÝDEK-MÍSTEK KRAJ: MORAVSKOSLEZKÝ

AKCE:

BASKETBALOVÁ HALA

BASKETPOINT FRÝDEK-MÍSTEK Z.S.

TŘ.T.G.MASARYKA 503, 738 01 FRÝDEK-MÍSTEK

SMLUVNÍ GARANT	Jan Kolašín	
DATUM	FORMÁT	MĚŘÍTKO
24.7.2018	A4	.
ČÍS. ZAKÁZKY	OBJEKT	STUPEŇ P. D.
CZ1138	-	DSP

OBSAH: ČÁST OCELOVÁ KONSTRUKCE

STATICKÝ VÝPOČET

ČÍSLO VÝKRESU: S CZ1138 -

REVIZE: .

TECHNICKÁ ZPRÁVA

Obsah:

PROJEKTANT : ING. LENKA BURGEROVÁ

TELEFON : +420 725 319 232

1 Úvod.

Stavba se bude nacházet na území České republiky, v městě Frýdek-Místek.

Předmětem tohoto statického výpočtu je návrh a posouzení ocelové nosné konstrukce stavby. Jedná se o tělocvičnu přiléhající jednou podélnou stranou ke zděnému objektu. Objekty jsou vzájemně oddilatovány.

2 Popis objektu

Objekt je jednolodní hala se sedlovou střechou o sklonu 6°.

Rozměry haly: délka 43.5m, šířka 33.3m, světlá výška 7m, výška hřebene je 10.05m (nad podlahou haly). Základní modul budovy je 5.7m. U hlavního vchodu je přístřešek šířky 6.3m a délky 4.815m. Světlá výška přístřešku je 3m.

3 Střešní plášť

Střešní plášť tvoří sendvičové panely kladené po spádu na vaznice ze profilů Z150. Vaznice jsou z ocelových za studena válcovaných profilů s roztečí 750mm. Panely jsou k vaznicím uchyceny samořeznými šrouby po max 300mm.

Skladba:

-Střešní krytina:	Sendvičové panely tl. izolace 160 mm, povrchová úprava polyesterový lak
-Nosný profil:	Vaznice profil Z150, kladený á 750mm, povrchová úprava - žárové zinkování.
-Podhled:	Akustický podhled – **Samostatná dodávka stavby.**
-Nosná konstrukce:	Příhradové vazníky. Povrchová úprava – žárové zinkování.

4 Stěnový plášť

Stěnový plášť tvoří opět sendvičové panely. Jsou kladeny horizontálně a kotveny do ocelových sloupů. Maximální rozteč sloupů je 5.7m.

Skladba:

-Opláštění:	Horizontální sendvičové panely tl. izolace 120 mm, povrchová úprava polyesterový lak.
-Nosný profil:	Sloupy složené z ocelových za studena válcovaných C profilů (C360). Povrchová úprava - žárové zinkování.

5 Popis konstrukcí

Sekundární nosnou konstrukci střechy tvoří vaznice kladené v podélném směru s roztečí 750mm. Vaznice jsou ze za studena válcovaných profilů tvaru Z, výšky 150mm. Zatížení se na vaznice přenáší ze sendvičových střešních panelů. Panely jsou s vaznicemi spojeny samořeznými šrouby s roztečí maximálně 300mm, tím je zajištěna stabilita vaznic v klopení, rotaci a vybočení ve směru „měkké osy“. Vaznice jsou také součástí střešního ztužení a jsou posuzovány na normálové síly od ztužení.

Primární nosnou konstrukcí střechy jsou příhradové vazníky. Vazníky jsou uspořádány v příčném směru - v modulech 5.7m. Jsou složené z ocelových za studena válcovaných profilů tvaru H a C. Pásy vazníků tvoří H (omega) profily, do nichž jsou připojovány diagonály z C-profilů (C100). Zatížení na vazníky je přenášeno z vaznic. Vazníky jsou stabilizovány vaznicemi spolu se střešními ztužidly. Stabilitu spodního pásu vazníku zajišťují táhla ke střešním vaznicím.

Primární nosnou konstrukcí střechy v rovině štítů jsou štítové nosníky. Nosníky jsou opět ze zastudena válcovaných ocelových profilů tvaru C (C250). Nosníky jsou osazeny v příčném směru. Jsou kotveny na štítové sloupy v modulech 5.7m. Stabilita horní části štítových nosníků je zajištěna spojením s vaznicemi spolu se střešním ztužidlem a dolní pas stabilizují vzpěry k vaznicím (z L-profilů).

Primární svislé nosné konstrukce jsou sloupy.

Sloupy v podélných stěnách tvoří spolu s příhradovými vazníky rámovou konstrukci. Ve spoji sloupů a vazníku je rámový roh přenášející moment. Sloupy v podélných stěnách jsou složeny ze za studena válcovaných profilů C360. Tvoří je čtveřice profilů spojených rámovými propojkami po 1500mm do členěného prutu. Stabilita sloupů je zajištěna vetknutým kotvením, rámovým rohem ve spoji s vazníkem a stěnovými ztužidly.

Sloupy ve štítových stěnách jsou opět složeny ze za studena válcovaných profilů C360. Tvoří je dvojice profilů spojených rámovými propojkami po 1500mm do členěného prutu. Sloupy ve štítových stěnách jsou kloubově spojeny se štítovými nosníky. Stabilita sloupů je zajištěna vetknutím, střešním ztužidlem a stěnovými ztužidly.

Sekundární nosnou konstrukcí ve stěnách je systém výměn pro otvory , akustickou předstěnu a vybavení tělocvičny. Výměny jsou z ocelových za studena válcovaných profilů tvaru C, o výškách 170 a 250mm.

Ztužidla stěn a střechy jsou provedena z táhel z ocelových pásků nebo ze vzpěr z ocelových za studena válcovaných profilů C- profilů.

6 Základní zatížení

Sněhová oblast :	III.
Charakteristické zatížení sněhem na zemi:	**1.3 kPa** (upřesněno dle mapy www.snehovamapa.cz)
Větrová oblast:	II.
Základní rychlost větru $v_{b,0}$	25m/s
Kategorie terénu:	III.
Maximální dynamický tlak větru $q_{p(z)}$	**0.67kPa**
Vlastní tíha střešního pláště:	**25kg/m²**
Plošné přitížení střechy akustickým podhledem:	**25kg/m²**
Plošné přitížení střechy instalacemi:	**20kg/m²**
4x teplovzdušný agregát v rozích haly:	**25kg/ks**
Plošina pro časomíru:	**do 600 kg (včetně obsluhy)**
Dělící roleta:	**1 kN/bm**
Zatížení od basketbalových košů:	**2x 3.7kN**

7 Požární odolnost

Střešní plášť:	EW15/DP3	Sendvičové panely s požadovanou odolností.
Stěnové opláštění:	EW15/DP3	Sendvičové panely s požadovanou odolností.
Střešní nosná konstrukce:	R15/DP1	Ocelová konstrukce dimenzovaná dle normové křivky na odolnost 15 minut.
Nosná konstrukce stěn:	R15/DP1	Ocelová konstrukce dimenzovaná dle normové křivky na odolnost 15 minut.

8 Použité profily a materiály

Ocelová konstrukce je z typizovaných za studena válcovaných profilů. Konstrukce je navržena se šroubovými spoji. Není–li specifikováno jinak, jsou použity pozinkované šrouby M12 třídy pevnosti 8.8. (dále mohou být použity šrouby M16 8.8). Pro spoje plechů jsou použity pozinkované/nerezové šrouby 4.8, 5.5 a 6.3mm - přesné (pevnosti dle dodavatele spojovacího materiálu).

Používané materiály profilů:
Všechny prvky jsou vyrobeny ohýbáním za studena ze svitků s oboustranným pozinkováním.

Prvky ocelové konstrukce			
síla materiálu	norma	materiál	zinkování (dle ČSN EN 10 147)
1,5 mm	EN 10346	S 350 GD	275 g/m2
2,0 mm	EN 10346	S 350 GD	275 g/m2
3,0 mm	EN 10346	S 350 GD	450 g/m2
4,0 mm	EN 10346	HX 420 LAD	450 g/m2
5,0 mm	EN 10346	HX 500 LAD	450 g/m2
6,0 mm	EN 10346	HX 500 LAD	450 g/m2
7,0 mm	EN 10346	HX 420 LAD	450 g/m2

Ocel tl.- 1,5-2mm	S350GD,	Pozink – Z275MA	f_{yb}=350MPa,f_u=420MPa
Ocel tl.- 3mm	S350GD,	Pozink – Z450MA	f_{yb}=350MPa,f_u=420MPa
Ocel tl.- 4mm	HX420LAD	Pozink – Z450MA	f_{yb}=420MPa,f_u=480MPa
Ocel tl.- 5-6mm	HX500LAD	Pozink – Z450MA	f_{yb}=500MPa,f_u=550MPa
Ocel tl.- 7mm	HX420LAD	Pozink – Z450MA	f_{yb}=420MPa,f_u=480MPa
Ocel tl. >7mm	S355	Nátěr.	f_{yb}=355MPa,f_u=510MPa

9 Použité normy, podklady,software:

-Požadavky stavebníka.
-Stavebně technická a statická část projektu DSP.
-ČSN EN 1990-1-1
- ČSN EN 1991-1-1
- ČSN EN 1991-1-3
- ČSN EN 1991-1-4
- ČSN EN 1993-1-1
- ČSN EN 1993-1-3
- ČSN EN 1993-1-8

Použitý software:
-Autodesk Robot Structural Analysis
-HILTI PROFIS Anchor 1.8.0

Zatížení

Soubory modelu konstrukce:

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

Soubory zatížení:

CZ1138 Loading.xlsm
…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

Poznámky:

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

…………………………………………………………………………………………………..

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 : **Typ 6-PUR panel tl. 160mm , Z150**

Šířka haly:	33.302	m	Horní pás:	5.71	°	Výška u žlabu:	8.35	m
Délka haly:	43.465	m	Dolní pás:	0	°	Výška hřebene:	10.05	m

Stálé zatížení: ČSN EN 1991 ()

Gravitační tíhu zadávat se znaménkem "-"

Tíha střechy R =	0.25	kN/m^2	
Extra přitížení E =	-0.20	kN/m^2	
Tíha podhledu C =	-0.25	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 =	-1.30	kN/m^2	Tvarový součinitel:
Tvarový součinitel μ_i =	0.8		Platí pro sklony 0° ≤ α ≤ 30°
Součinitel expozice c_e =	1.0		μ_1 = 0.80
Tepelný součinitel c_t =	1.0		μ_2 = 0.95
$s = s_k \cdot \mu_i \cdot c_e \cdot c_t$ =	-1.04	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: III. Oblasti rovnoměrně pokryté vegetací, budovami nebo překážkami (vesnice, lesy)

Základní rychlost větru v_b =	25	m/s	Parametr drsnosti terénu z_0 =	0.3	m
Součinitel terénu k_r =	0.215		Minimální výška $z_{min.}$ =	5	m
Součinitel drsnosti terénu $c_r(z)$ =	0.756		Výška nad zemí z =	10.05	m
Střední rychlost větru $v_m(z)$ =	18.9	m/s	Součinitel směru =	1.0	
Základní dynamický tlak větru $q_b(z)$ =	223.5	N/m^2	Součinitel období =	1.0	
			Součinitel orografie $c_o(z)$ =	1.0	
Součinitel expozice $c_e(z)$ =	2.993		Součinitel turbulence k_I =	1.0	

Maximální dynamický tlak $q_p(z)$ = **0.669** kN/m^2

B2. Zatížení větrem (ČSN EN 1991-1-4)

Šířka haly:	33.30	m	Horní pás:	5.71	°	Výška u žlabu:	8.35	m
Délka haly:	43.47	m	Dolní pás:	0.00	°	Výška hřebene:	10.05	m
				6				

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 = 43.47 m d = 33.30 m e = 20.10 m

	Oblast	A	B	C	D	E	
Stěny	Délka [m]	4.0	16.1	13.2	43.5	43.5	délka oblasti působení
	Cpe	-1.20	-0.80	-0.50	0.71	-0.31	tlak/sání na stěnu

	Oblast	F	G	H	I	J	
Střecha	Délka [m]	2.0	2.0	14.6	14.6	2.0	délka oblasti působení
	Cpe sání	-1.62	-1.16	-0.57	-0.58	-0.64	sání na střechu
	Cpe tlak	0.02	0.02	0.02	-0.54	0.18	tlak na střechu
	-	-	-	-	-	-	-

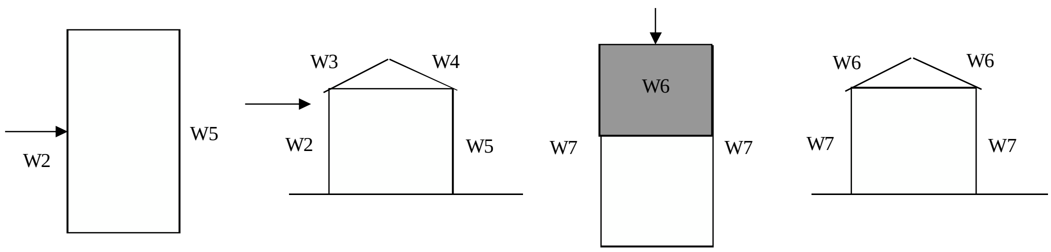
Podélný vítr

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

b = 33.30 m d = 43.47 m e = 20.10 m

	Oblast	A	B	C	D	E	
Stěny	Délka [m]	4.0	16.1	23.4	33.3	33.3	délka oblasti působení
	Cpe	-1.20	-0.80	-0.50	0.70	-0.30	tlak/sání na stěnu

	Oblast	F	-	G	H	I	
Střecha	Délka [m]	2.0	-	2.0	8.0	33.4	délka oblasti působení
	Cpe sání	-1.57	-	-1.30	-0.69	-0.59	sání na střechu
	-	-	-	-	-	-	-



$w_e = q_p(z) \cdot c_{pe}$

W2	c_{pe} =	0.71	w_e =	0.47	kN/m²	Příčný vítr	Stěna
W3	c_{pe} =	0.57	w_e =	0.38	kN/m²	Příčný vítr	Střecha sání
W3a	c_{pe} =	-0.02	w_e =	-0.01	kN/m²	Příčný vítr	Střecha tlak
W4	c_{pe} =	0.58	w_e =	0.39	kN/m²	Příčný vítr	Střecha sání
W4a	c_{pe} =	0.54	w_e =	0.36	kN/m²	Příčný vítr	Střecha tlak
W5	c_{pe} =	0.31	w_e =	0.21	kN/m²	Příčný vítr	Stěna závětrná
W6	c_{pe} =	0.69	w_e =	0.46	kN/m²	Podélný vítr	Střecha
W7	c_{pe} =	0.80	w_e =	0.54	kN/m²	Podélný vítr	Stěna
W8	c_{pe} =	**1.30**	w_e =	0.87	kN/m³	Atika	
W9	c_{pe} =		w_e =	0.00	kN/m⁴	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, C** — ČSN EN 1991-1-4 (7.2.2)

Platí e = min(b,2h), kde b je rozměr kolmý na směr větru, d je rozměr ve směru větru — Obrázek 7.5

b = 43.5 m e/10 = 2.0 m e/4 = 5.0 m Poměr h/b = 0.30 0.30

d = 33.3 m e/5 = 4.0 m e/2 = 10.1 m e = 20.1 m

	Oblast	A	B	C	D	E	
Stěny	Délka [m]	4.0	16.1	13.2	43.5	43.5	délka oblasti působení
	C_{pe}	-1.20	-0.80	-0.50	0.71	-0.31	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]	2.0	2.0	14.6	14.6	2.0	délka oblasti působení
	C_{pe} sání	-1.62	-1.16	-0.57	-0.58	-0.64	sání na střechu
	C_{pe} tlak	0.02	0.02	0.02	-0.54	0.18	tlak na střechu
	-	-	-	-	-	-	-

Podélný vítr

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

Platí e = min(b,2h), kde b je rozměr kolmý na směr větru, d je rozměr ve směru větru — Obrázek 7.5

b = 33.3 m e/10 = 2.0 m e/4 = 5.0 m Poměr h/b = 0.23 0.25

d = 43.5 m e/5 = 4.0 m e/2 = 10.1 m e = 20.1 m

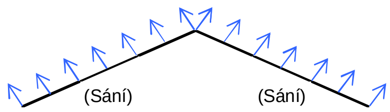
	Oblast	A	B	C	D	E	
Stěny	Délka [m]	4.0	16.1	23.4	33.3	33.3	délka oblasti působení
	C_{pe}	-1.20	-0.80	-0.50	0.70	-0.30	tlak/sání na stěnu

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

	Oblast	F	-	G	H	I	
Střecha	Délka [m]	2.0	-	2.0	8.0	33.4	délka oblasti působení
	C_{pe} sání	-1.57	-	-1.30	-0.69	-0.59	sání na střechu
	-	-	-	-	-	-	-

Příčný vítr

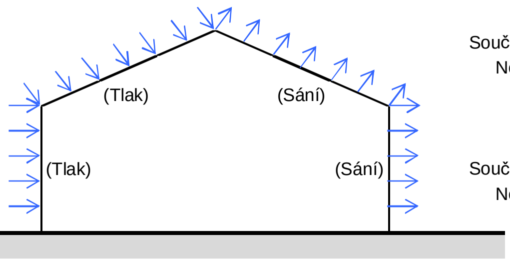
Střecha - Návětrná strana (Oblast H)
Součinitel vněj. tlaku c_{pe} = **-0.57**
Normové zatížení w_e = **-0.38** kN/m²



Střecha - Závětrná strana (Oblast I)
Součinitel vněj. tlaku c_{pe} = **-0.58**
Normové zatížení w_e = **-0.39** kN/m²

Střecha - Návětrná strana (Oblast H)
Součinitel vněj. tlaku c_{pe} = **0.02**
Normové zatížení w_e = **0.01** kN/m²

Stěna - Návětrná strana (Oblast D)
Součinitel vněj. tlaku c_{pe} = **0.71**
Normové zatížení w_e = **0.47** kN/m²



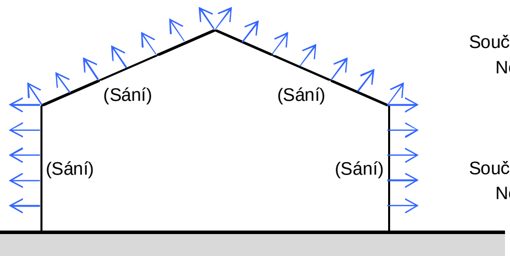
Střecha - Závětrná strana (Oblast I)
Součinitel vněj. tlaku c_{pe} = **-0.54**
Normové zatížení w_e = **-0.36** kN/m²

Stěna - Závětrná strana (Oblast E)
Součinitel vněj. tlaku c_{pe} = **-0.31**
Normové zatížení w_e = **-0.21** kN/m²

Podélný vítr

Střecha (Oblast H)
Součinitel vněj. tlaku c_{pe} = **-0.69**
Normové zatížení w_e = **-0.46** kN/m²

Stěna (Oblast B)
Součinitel vněj. tlaku c_{pe} = **-0.80**
Normové zatížení w_e = **-0.54** kN/m²



Střecha (Oblast H)
Součinitel vněj. tlaku c_{pe} = **-0.69**
Normové zatížení w_e = **-0.46** kN/m²

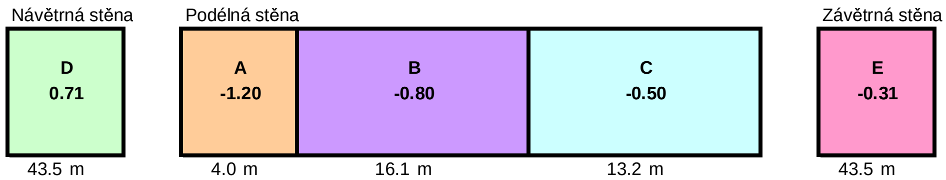
Stěna (Oblast B)
Součinitel vněj. tlaku c_{pe} = **-0.80**
Normové zatížení w_e = **-0.54** kN/m²

B3. Sedlová střecha - Schéma zatížení c_{pe}

Stěny

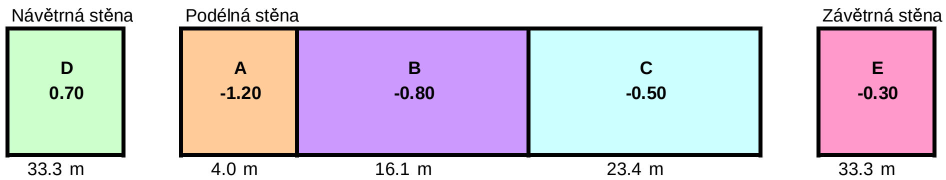
Příčný vítr

ČSN EN 1991-1-4 (7.2.2)
Obrázek 7.5



Podélný vítr

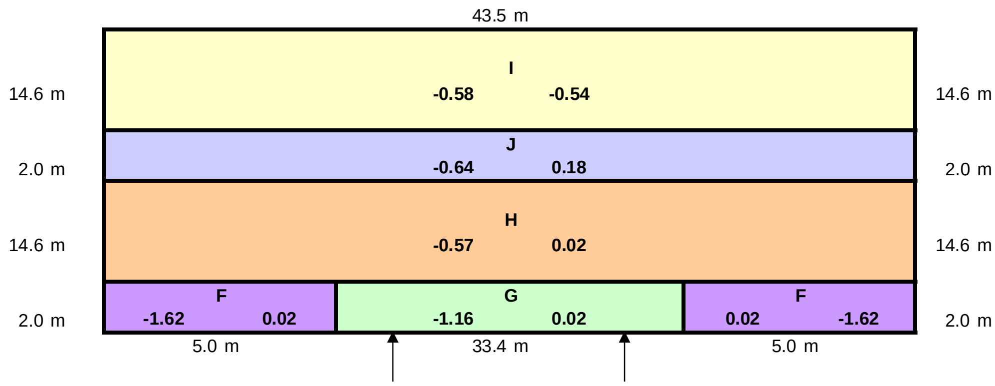
ČSN EN 1991-1-4 (7.2.2)
Obrázek 7.5



Střecha

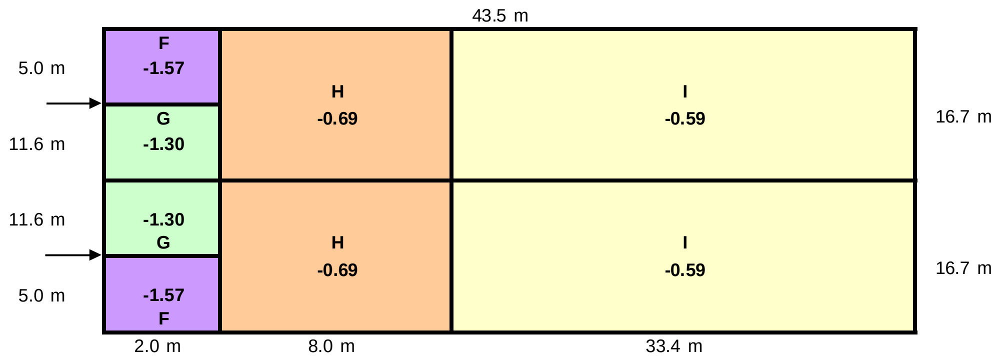
Příčný vítr

ČSN EN 1991-1-4 (7.2.5)
Obrázek 7.8



Podélný vítr

ČSN EN 1991-1-4 (7.2.5)
Obrázek 7.8



Vaznice PU1

Soubory modelu konstrukce:

CZ1138 PU1.Vaznice.standard.rtd

Soubory zatížení:

CZ1138 Loading.xlsm

Poznámky:

Standardní vaznice (není součástí zavětrování) Ned=5kN

Požadovaná požární odolnost R15.

Rozteče vaznic - 0.75m = zatěžovací šířka

Objekt:	**Basketbalová hala**	Místo výstavby:	**Frýdek-Místek**
Číslo projektu:	**CZ1138**	Datum:	**19.7.2018**

C1. Vaznice - zatížení

Zatěžovací šířka standardní L_{w1} =	**0.75**	m		Šířka haly:	33.30	m
Zatěžovací šířka krajní L_{w2} =	**0.75**	m		Délka haly:	43.47	m

Typ střechy: **Sedlová**

střecha+podhled

Stálé zatížení **ČSN EN 1991**

Normová tíha střechy $g_{r,n}$ =	-0.50	kN/m²
Normové extra přitížení $E_{x,n}$ =	-0.20	kN/m²
Celkem stálé zatížení normové g_n =	**-0.70**	kN/m²
Součinitel zatížení $\gamma_{fg,n}$ =	1.35	
Součinitel zatížení γ_f =	1.35	
Výpočtová tíha střechy $g_{r,d}$ =	-0.68	kN/m²
Výpočtové extra přitížení $E_{x,d}$ =	-0.27	kN/m²
Celkem stálé zatížení výpočtové g_d =	**-0.95**	kN/m²

technol. přitížení

Zatížení sněhem **ČSN EN 1991-1-3**

Charakteristická hodnota s_k =	**-1.3**	kN/m²
Tvarový součinitel μ_i =	0.8	
Součinitel expozice c_e =	1.0	
Tepelný součinitel c_t =	1.0	
Normové zatížení sněhem s_n =	**-1.04**	kN/m²
Součinitel zatížení γ_s=	1.5	
Výpočtové zatížení sněhem s_d =	**-1.56**	kN/m²

Zatížení větrem **ČSN EN 1991-1-4**

Základní větrné zatížení w_o =	25	m/s	Tlak zadáván zápornou hodnotou
Maximální dynamický tlak q_p (z) =	**0.669**	kN/m²	

Příčný vítr

Oblast =	F	G	H	I	J	
Součinitel vnějšího tlaku $C_{pe\,(sání)}$ =	1.62	1.16	0.57	0.58	0.64	-
Součinitel vnějšího tlaku $C_{pe\,(tlak)}$ =	-0.02	-0.02	-0.02	0.54	-0.18	-
Součinitel vnějšího tlaku $C_{pe\,(sání)}$ =	0.00	0.00	0.00	0.00	0.00	-

Normová hodnota zatížení $w_{n\,(sání)}$ =	1.08	0.78	0.38	0.39	0.43	kN/m²
Normová hodnota zatížení $w_{n\,(tlak)}$ =	-0.01	-0.01	-0.01	0.36	-0.12	kN/m²
Normová hodnota zatížení $w_{n\,(sání)}$ =	0.00	0.00	0.00	0.00	0.00	kN/m²

Součinitel zatížení γ_w =	1.5	1.5	1.5	1.5	1.5	-
Výpočtová hodnota zatížení $w_{d\,(sání)}$ =	1.63	1.16	0.57	0.58	0.64	kN/m²
Výpočtová hodnota zatížení $w_{d\,(tlak)}$ =	-0.02	-0.02	-0.02	0.54	-0.18	kN/m²
Výpočtová hodnota zatížení $w_{d\,(sání)}$ =	0.00	0.00	0.00	0.00	0.00	kN/m²

Podélný vítr

Oblast =	F	-	G	H	I	
Součinitel vnějšího tlaku $C_{pe\,(sání)}$ =	1.57	0.00	1.30	0.69	0.59	-
Součinitel vnějšího tlaku $C_{pe\,(tlak)}$ =	0.00	0.00	0.00	0.00	0.00	-

Normová hodnota zatížení $w_{n\,(sání)}$ =	1.05	0.00	0.87	0.46	0.39	kN/m²
Normová hodnota zatížení $w_{n\,(tlak)}$ =	0.00	0.00	0.00	0.00	0.00	kN/m²

Součinitel zatížení γ_w =	1.5	1.5	1.5	1.5	1.5	-
Výpočtová hodnota zatížení $w_{d\,(sání)}$ =	1.58	0.00	1.30	0.69	0.59	kN/m²
Výpočtová hodnota zatížení $w_{d\,(tlak)}$ =	0.00	0.00	0.00	0.00	0.00	kN/m²

Vypracopval:	Kontroloval:	Strana:
Lenka Burgerová	**Jaroslav Kosinka**	C1.1
Podpis:	Podpis:	

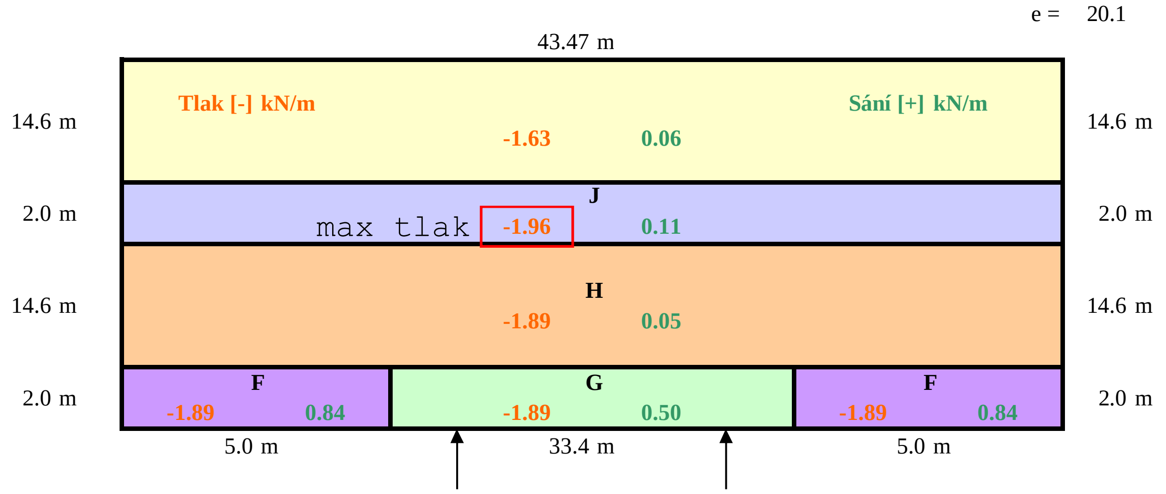
Filename: CZxxxx.Loading.EN.v6.79.110401.xls Výpočet dle: ČSN EN

C2. Vaznice - sedlová střecha

Zatížení vaznice - kombinace

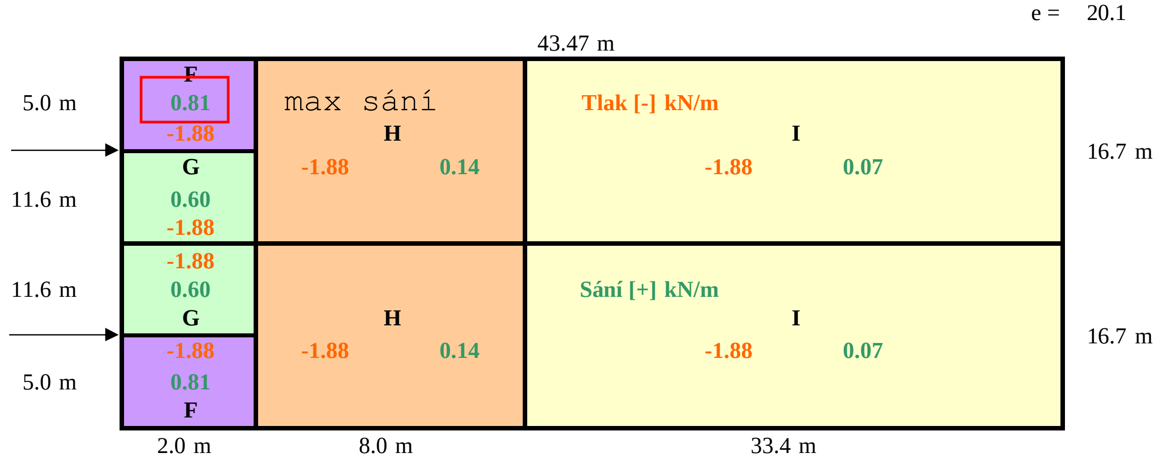
A) Příčný vítr

Oblast =	F	G	H	I	J	
Zatěžovací šířka L_{w1}, L_{w2} =	0.8	0.8	0.8	0.8	0.8	m
Délka oblasti l =	2.0	2.0	14.6	14.6	2.0	m
1. Stálé + sníh q_d =	-1.88	-1.88	-1.88	-1.88	-1.88	kN/m
2. Stálé + vítr (sání) q_d =	0.84	0.50	0.05	0.06	0.11	kN/m
3. Stálé + vítr (tlak) + sníh dom. q_d =	-1.89	-1.89	-1.89	-1.63	-1.96	kN/m
4. Stálé + vítr (tlak) + sníh dom. (norm.) q_k =	-1.31	-1.31	-1.31	-1.14	-1.36	kN/m



B) Podélný vítr

Oblast =	F	-	G	H	I	
Zatěžovací šířka L_{w1}, L_{w2} =	0.75	-	0.75	0.75	0.75	m
Délka oblasti l =	2.0	-	2.0	8.0	33.4	m
1. Stálé + sníh q_d =	-1.88	-	-1.88	-1.88	-1.88	kN/m
2. Stálé + vítr (sání) q_d =	0.81	-	0.60	0.14	0.07	kN/m
3. Stálé + vítr (tlak) + sníh dom. q_d =	-1.88	-	-1.88	-1.88	-1.88	kN/m
4. Stálé + vítr (tlak) + sníh dom. (norm.) q_k =	-1.31	-	-1.31	-1.31	-1.31	kN/m



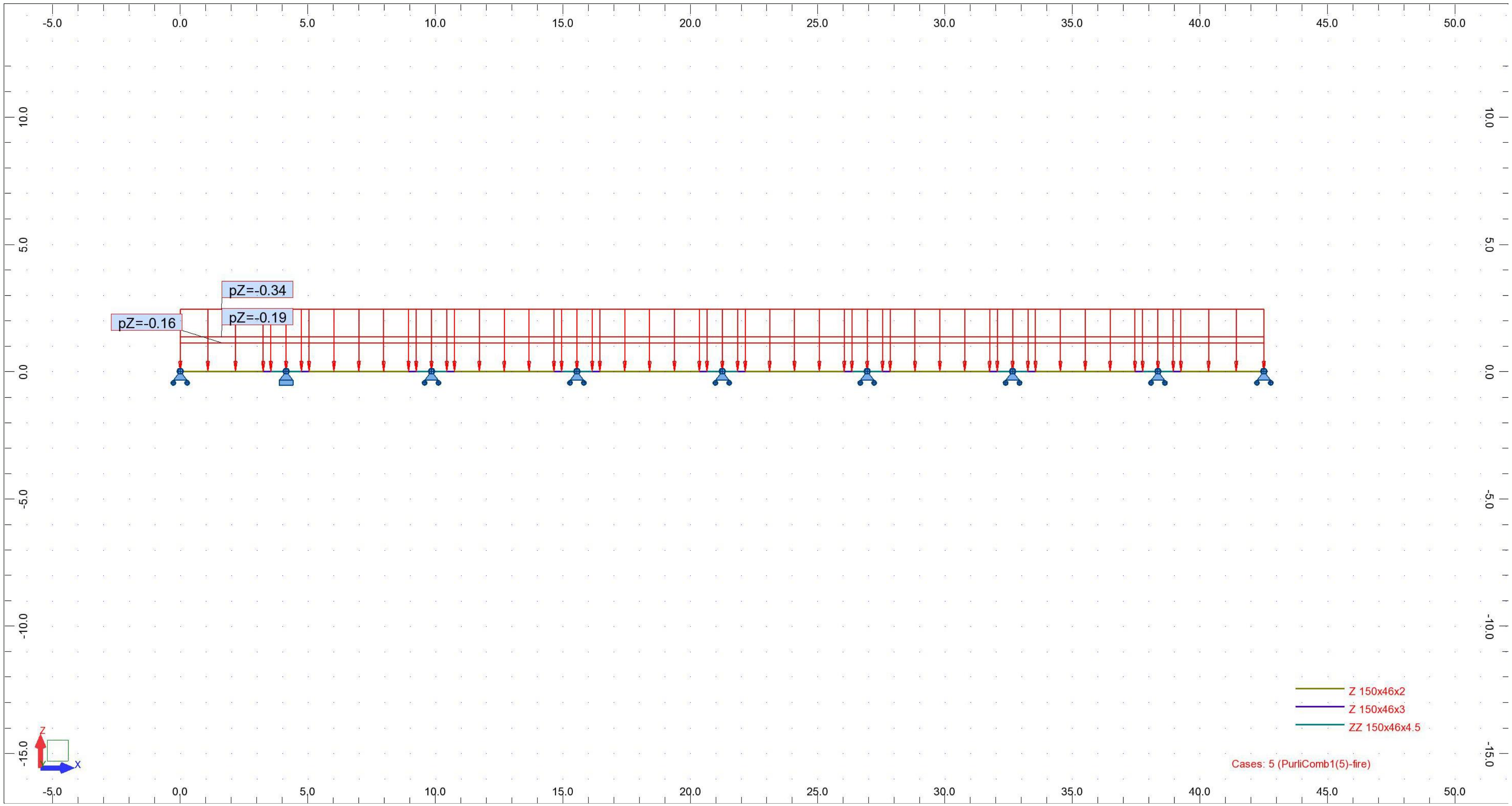
Síla od zavětrování

Nezavětrovaná vaznice Ned=5kN

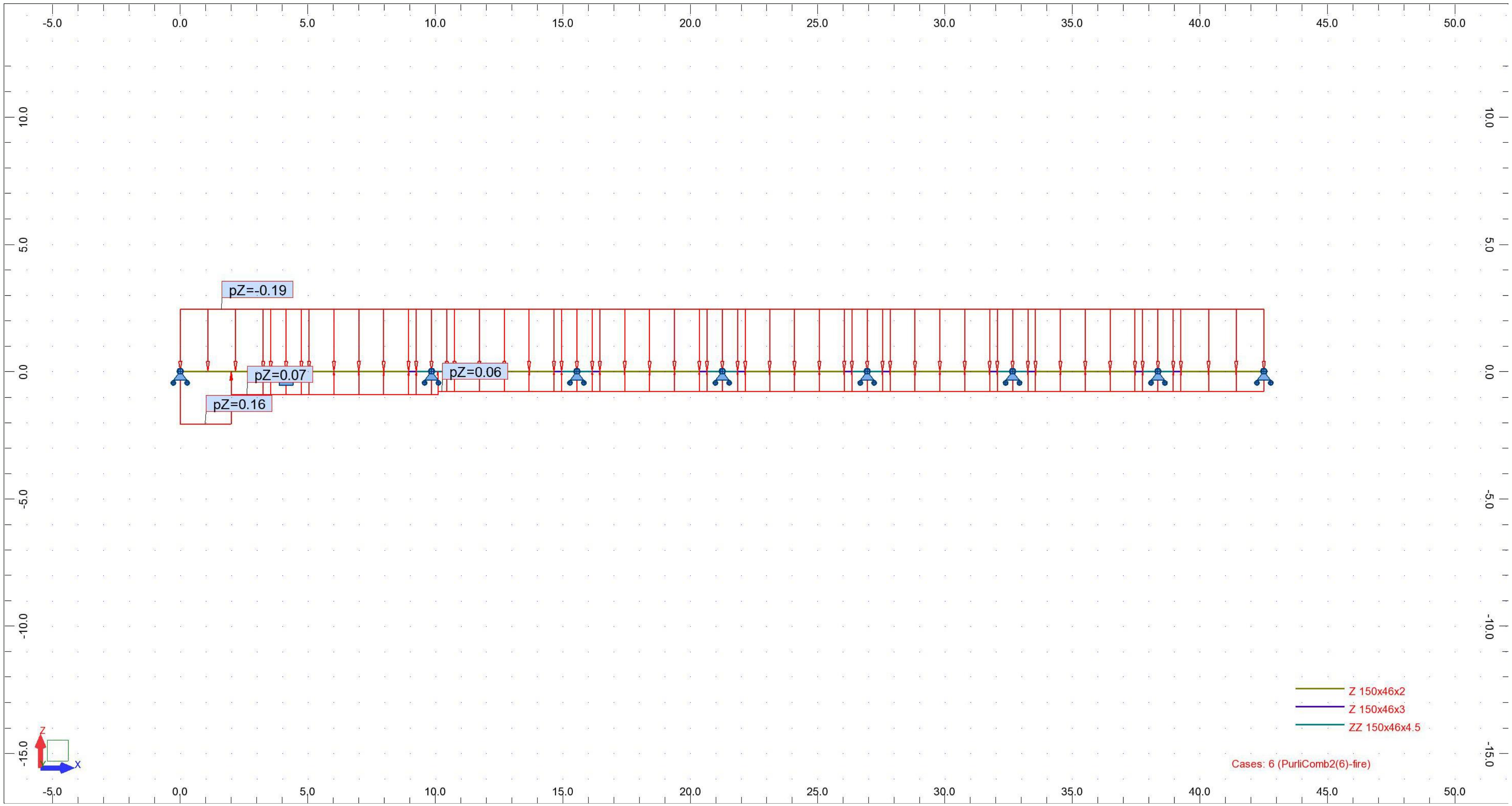
~~Tlaková síla do zavětrovaných vaznice. Uvažována jako 60% síly ze zavětrování do 1 stěny~~

~~Tlaková síla od zavětrování N_{Ed} = **51.47** kN~~

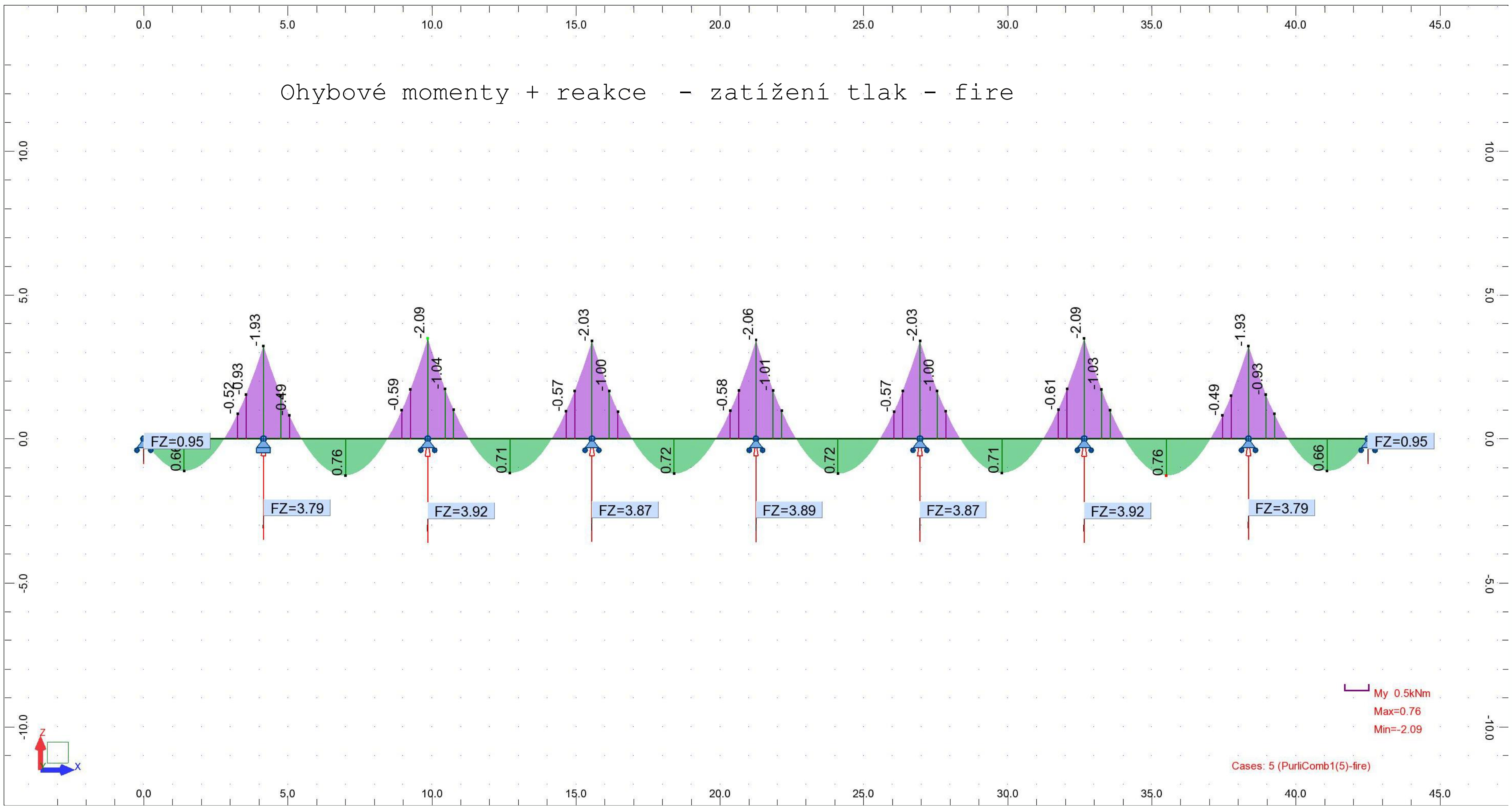
View - Cases: 5 (PurliComb1(5)-fire) Zatížení - tlak - fire



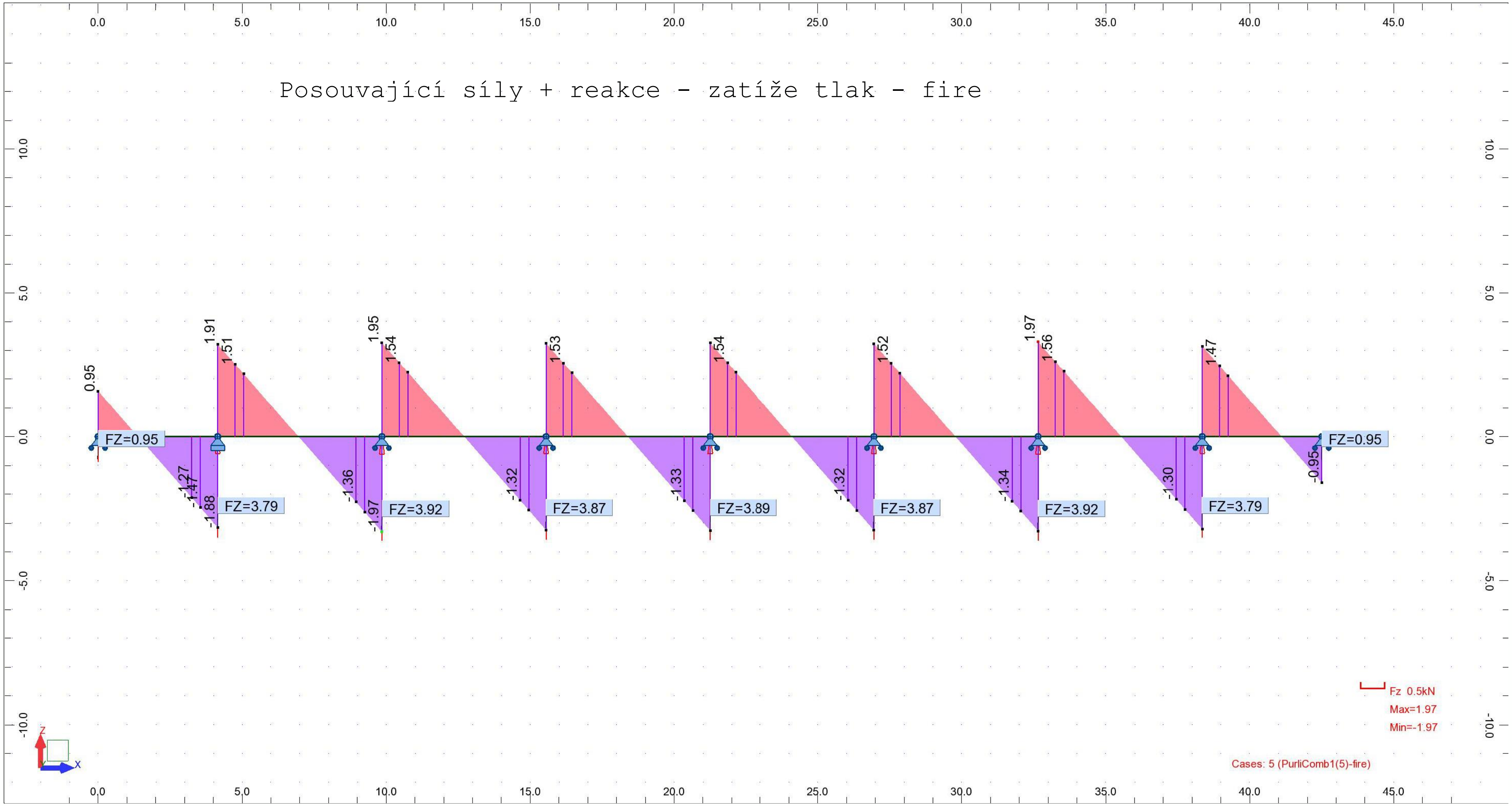
View - Cases: 6 (PurliComb2(6)-fire) Zatížení - sání - fire



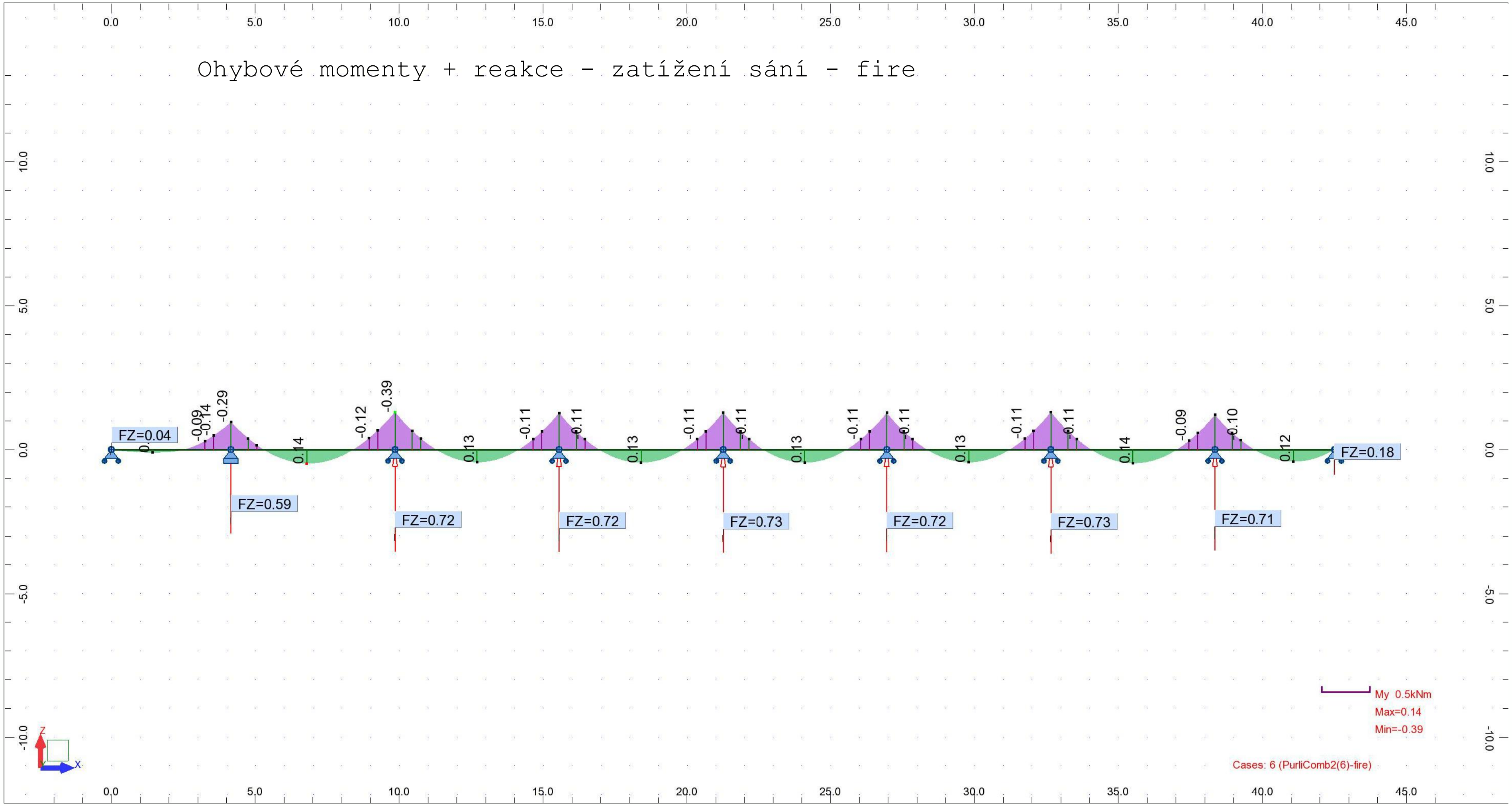
View - MY;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 5 (PurliComb1(5)-fire)



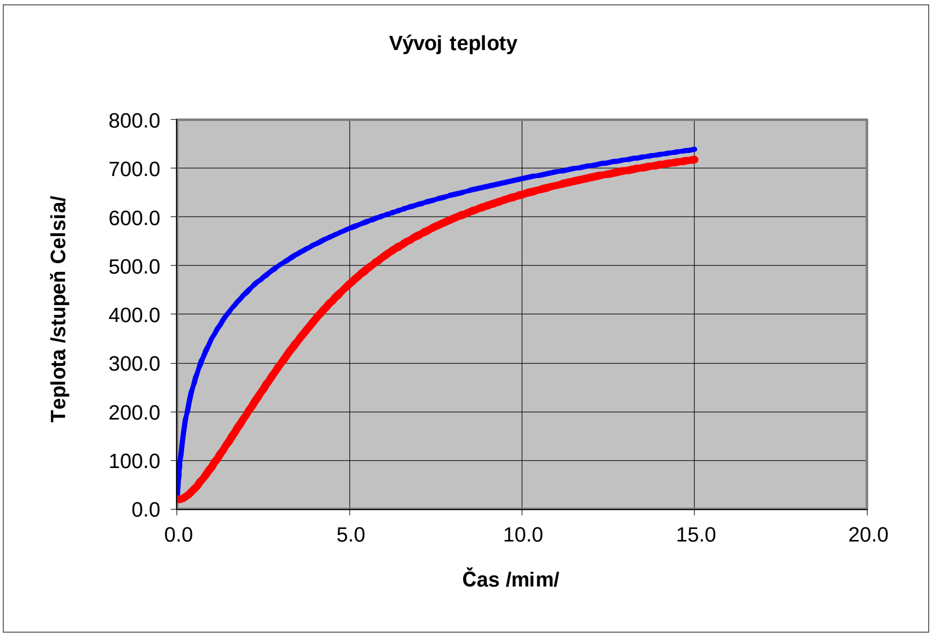
View - FZ;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 5 (PurliComb1(5)-fire)



View - MY;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 6 (PurliComb2(6)-fire)



Redukční součinitelé vaznice pro požární odolnost R 15



Čas t=15 min.

	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
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			
717.59	**0.209**	**0.071**	**0.123**

	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
717.59	**0.119**	**0.123**	

$k_{y\Theta}$ red. součinitel na mez kluzu

$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu

$k_{E\Theta}$ red. součinitel pro linerní pružnou část

	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
717.59	**0.094**	**0.120**	**0.000**

Posudek vaznice na R15 - vnitřní pole

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

$N := newton \quad kN := 1000 \cdot N$

$\begin{pmatrix} f_{yb} \\ f_u \end{pmatrix} := \begin{pmatrix} 42 \\ 50 \end{pmatrix} \cdot \frac{N}{mm^2}$

values for γ_{Mi}: $\gamma_{M0} := 1$ $\gamma_{M1} := 1$ $\gamma_{M2} := 1.25$

Global values: all measures in (mm)

$t_{rp} := 0.62 \cdot mm$ thickness roofplate ($_{rp}$)

$t_{rp.cor} := 0.5842 \cdot mm$ core thickness

$h_{w.rp} := 45 \cdot mm$ height roofplate ($_{rp}$)

$b_{roof} := 33302 \cdot mm$ width of roof ($_{roof}$)

$L_{sp} := 5700 \cdot mm$ span of purlin

$cc_{purlin} := 750 \cdot mm$ distance between purlins

$s_s := 108 \cdot mm$ Support width

Stresses on roof purlin

maximum gravity load $q_{Ed.g} := 0.68 \cdot \frac{kN}{m}$

maximum upplift load $q_{Ed.u} := 0.03 \cdot \frac{kN}{m}$

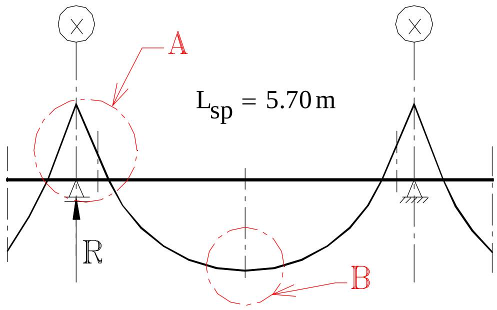
Compression force in purlin: $N_{Ed} := 0 \cdot kN$

Stresses due to upplift load:

Location B:

$M_{y.Ed.B.u} := 0.14 \cdot kN \cdot m$ Moment in span

$M_{y.Ed.AB.u} := 0.12 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)



Stresses due to gravity load:

Location A:

$M_{y.Ed.A.g} := 2.09 \cdot kN \cdot m$ Moment at support

$V_{Ed.A.g} := 1.97 \cdot kN$ Shear force at support

$R_{Ed.A.g} := 3.92 \cdot kN$ Reaktion force

$M_{y.Ed.AB.g} := 0.61 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)

$V_{Ed.A.AB.g} := 1.36 \cdot kN$ Shear force att connection between purlins (NOT support profile)

Location B:

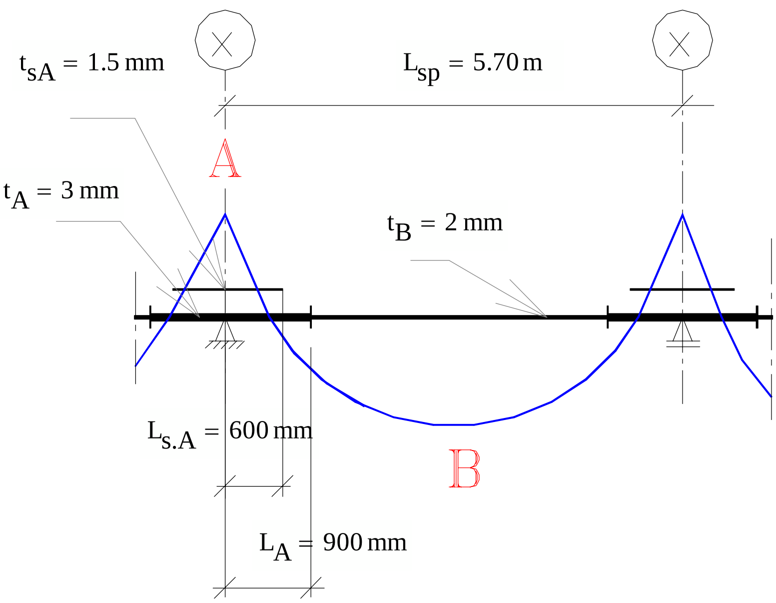
$M_{y.Ed.B.g} := 0.76 \cdot kN \cdot m$ Moment in span

Purlin profiles:

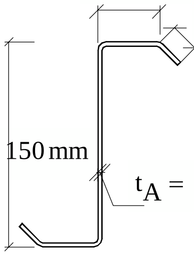
profile over support A Z150x1.5, Z150x2 or Z150x3: support_A := "Z150x3" length of profile over A: $L_A := 900 \cdot mm$

profile in span B Z150x1.5, Z150x2 or Z150x3: span_B := "Z150x2"

supporting profile over support location A ZX-1.5 , ZX-2 , ZX-3 or NO: sup_profile_A := "ZX-1.5" Lenght of extra supporting profile over support A: $L_{s.A} := 600 \cdot mm$



Profile over support A:



$b_A = 46\,mm$

$c_A = 21\,mm$

$h_A = 150\,mm$

$t_A = 3\,mm$

$M_{yRk.A} = 12.79\,kN\cdot m$

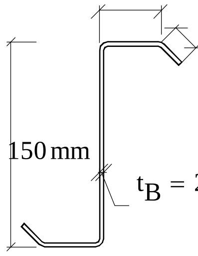
$V_{bh.Rk.A} = 89.11\,kN$

$R_{w.Rk.A} = 6.202\,kN$

$W_{yeff.A} = 3.66 \times 10^4\,mm^3$

$I_{y.A} = 2.69 \times 10^6\,mm^4$

Profile i span B:



$b_B = 46\,mm$

$c_B = 18\,mm$

$h_B = 150\,mm$

$t_B = 2\,mm$

$M_{yRk.B} = 8.07\,kN\cdot m$

$V_{bh.Rk.B} = 47.57\,kN$

$W_{yeff.B} = 2.31 \times 10^4\,mm^3$

$I_{y.B} = 1.8 \times 10^6\,mm^4$

$I_{T.B} = 726.25\,mm^4$

$I_{\omega.B} = 1.125 \times 10^9\,mm^6$

Shear stiffness of trapezoidal sheeting connected to purlin EN 1993-1-3: 10.1.1 (6)+(10):

Shear stiffness roof:

$X := 0.5$ only one half of the roof is accounted for

$$S_{roof} := \sqrt{\left(\frac{t_{rp}}{mm}\right)^3}\cdot\left[50 + 10\cdot\sqrt[3]{\left(\frac{b_{roof}\cdot X}{mm}\right)}\right]\cdot\frac{cc_{purlin}}{h_{w.rp}} \qquad S_{roof} = 2484.52 \text{ in kN}$$

purlin:

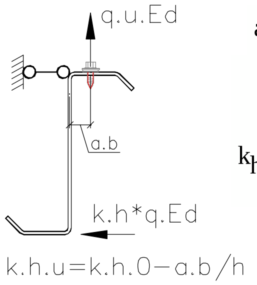
$$Z_{purlin} := \left(E\cdot I_\omega\cdot\frac{\pi^2}{L_{sp}^2} + G\cdot I_T + E\cdot I_z\cdot\frac{\pi^2}{L_{sp}^2}\cdot 0.25\cdot h_B^2\right)\cdot\frac{70}{h_B^2} \qquad Z_{purlin} = 89\,kN$$

Restrained := if($S_{roof}\cdot kN \ge Z_{purlin}$, "Purlins are laterally restrained" , "No laterally restrain")

The purlin at the connection is regarded as beeing laterally restrained in plane of the sheeting.

Restrained = "Purlins are laterally restrained"

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



$a_{b.B} = 27.6\,mm$

$$k_{h.0.B} := \frac{h_{cB}\cdot t_B\cdot\left(b_{cB}^2 + 2\cdot b_{cB}\cdot c_{cB} - 2\cdot c_{cB}^2\cdot\frac{b_{cB}}{h_{cB}}\right)}{4\cdot I_{y.B}}$$

$k_{h.0.B} = 0.139$

Equivalent lateral load acting on free flange, due to torsion and lateral bending (EN 1993-1-3: 10.1.4.1 (3)

$$k_{h.u.B} := k_{h.0.B} - \frac{a_{b.B}}{h_{cB}} \qquad h.Ed.u.B := q_{Ed.u}\cdot k_{h.u.B}$$

$q_{h.Ed.u.B} = -0\,\frac{kN}{m}$

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): C_D

tension in upper flange: $I_{eff.TP46.t} := 125400.93\cdot mm^4\cdot\frac{1}{m}$ $\quad C_{D.C.t} := \frac{6\cdot E\cdot I_{eff.TP46.t}}{cc_{purlin}}$ $\quad C_{D.C.t} = 2.53 \times 10^4\,\frac{N\cdot m}{m}$

compression in upper flange: $I_{eff.TP46.c} := 130983.88\cdot mm^4\cdot\frac{1}{m}$ $\quad C_{D.C.c} := \frac{6\cdot E\cdot I_{eff.TP46.c}}{cc_{purlin}}$ $\quad C_{D.C.c} = 2.641 \times 10^4\,\frac{N\cdot m}{m}$

Rotational stiffness of connection between sheeting and purlin EN 1993-1-3: 10.1.5.2 (5)

$$k_{t.g} := \left(\frac{t_{rp}}{0.75}\right)^{1.5}\cdot\frac{1}{mm^{1.5}} \qquad k_{t.g} = 0.752 \quad \text{for b<125mm}$$

for uplift load pin in every trough $C_{100.cc150.u} := 2.6\cdot\frac{kN\cdot m}{m}$ pin in alternate trough $C_{100.cc300.u} := 1.7\cdot\frac{kN\cdot m}{m}$

$b := 46\cdot mm$

$$C_{D.A.cc150.u} := C_{100.cc150.u}\cdot\left(\frac{\frac{b}{mm}}{100}\right)^2\cdot k_{t.g} \qquad C_{D.A.cc150.u} = 0.4\,\frac{kN\cdot m}{m}$$

$$C_{D.A.cc300.u} := C_{100.cc300.u}\cdot\left(\frac{\frac{b}{mm}}{100}\right)^2\cdot k_{t.g} \qquad C_{D.A.cc300.u} = 0.3\,\frac{kN\cdot m}{m}$$

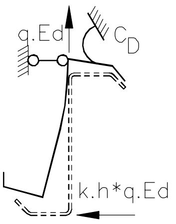
Rotational stiffness:

$$C_{D.cc150.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc150.u}} + \frac{1}{C_{D.C.t}}\right)} \qquad C_{D.cc150.u} = 0.407\,\frac{kN\cdot m}{m} \text{ pin in every trough}$$

$$C_{D.cc300.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc300.u}} + \frac{1}{C_{D.C.t}}\right)} \qquad C_{D.cc300.u} = 0.268\,\frac{kN\cdot m}{m} \text{ pin in alternate trough}$$

For uplift load EN 1993-1-3: 10.1.5.1 (4)

Span B:



$$b_{mod.u} := \begin{cases} a_{b.B} & \text{if } q_{h.Ed.u.B} \geq 0 \\ (2 \cdot a_{b.B} + b) & \text{if } q_{h.Ed.u.B} < 0 \end{cases}$$

Flexibility from web in bending:

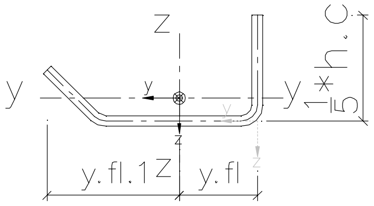
$$1 / K_{B.u.B} := \frac{4 \cdot (1 - \upsilon^2) \cdot h_B^2 \cdot (h_B + b_{mod.u})}{E \cdot t_B^3} \qquad 1 / K_{B.u.B} = 102.05 \frac{mm^2}{N}$$

Lateral spring stiffnes per unit length:

$$K_{u.cc150.B} := \frac{1}{\left(K_{B.u.B} + \frac{h_B^2}{C_{D.cc150.u}}\right)} \qquad K_{u.cc300.B} := \frac{1}{\left(K_{B.u.B} + \frac{h_B^2}{C_{D.cc300.u}}\right)}$$

$K_{u.cc150.B} = 0.0064 \frac{N}{mm^2}$ pin in every trough $\qquad K_{u.cc300.B} = 0.0054 \frac{N}{mm^2}$ pin in alternate trough

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

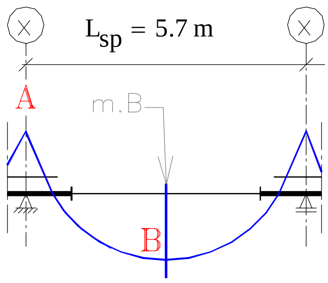


Span B: $t_B = 2.0\,mm$

$I_{z.fl.B} = 70968\,mm^4$

$W_{z.fl.B} = 1965.91\,mm^3$

$i_{z.fl.B} = 19.87\,mm$



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):

Span B $\qquad R_{rp.B} := \frac{K_{u.cc300.B} \cdot L_a^4}{\pi^4 \cdot E \cdot I_{z.fl.B}} \qquad R_{rp.B} = 32.55$

correction factor κ_R and initial moment $M_{0.fz.Ed}$ acc. to table 10.1 EN 1993-1-3: 10.1.4.1

Location: $M_{0.fz.Ed}$: $\qquad \kappa_R$:

$m_B \qquad M_{0.fz.Ed.m.B} := \frac{1}{24} \cdot |q_{h.Ed.u.B}| \cdot L_a^2 \qquad \kappa_{R.m.B} := \frac{1 - 0.0125 \cdot R_{rp.B}}{1 + 0.198 \cdot R_{rp.B}}$

lateral bending moment from EN 1993-1-3: 10.1.4.1(5):

$M_{fz.Ed}$:

$M_{fz.Ed.m.B} := \kappa_{R.m.B} \cdot M_{0.fz.Ed.m.B}$

$M_{fz.Ed.m.B} = 0.2\,N \cdot m$

Stresses due to gravity load as given above:

Location A:

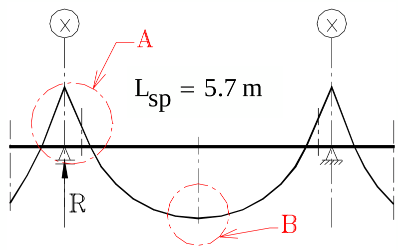
$M_{y.Ed.A.g} = 2.09\,kN \cdot m \qquad M_{y.Ed.AB.g} = 0.61\,kN \cdot m$

$V_{Ed.A.g} = 1.97\,kN \qquad V_{Ed.A.AB.g} = 1.36\,kN$

$R_{Ed.A.g} = 3.92\,kN$

Location B: $\qquad N_{Ed} = 0\,kN$

$M_{y.Ed.B.g} = 0.76\,kN \cdot m$



Stresses due to uplift load as given above: Location B:

$M_{y.Ed.B.u} = 0.14\,kN \cdot m$

Location A:

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

purlin: $t_A = 3.0\,mm$ supporting profile over support: sup_profile_A = "ZX-1.5" length of supp. profile: $L_{s.A} = 600\,mm$

check single profile:

$$\frac{M_{y.Ed.A.g} \cdot \gamma_{M0}}{M_{yRk.A}} + \frac{R_{Ed.A.g} \cdot \gamma_{M1}}{R_{w.Rk.A}} = 0.8 \quad < 1,25$$

check with supporting profile:

$$\frac{M_{y.Ed.A.g} \cdot \gamma_{M0}}{M_{yRk.A} + M_{yRk.sA}} + \frac{R_{Ed.A.g} \cdot \gamma_{M1}}{R_{w.Rk.A} + R_{w.Rk.sA}} = 0.59 \quad < 1,25$$

Continue Location A:

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

single profile at support:

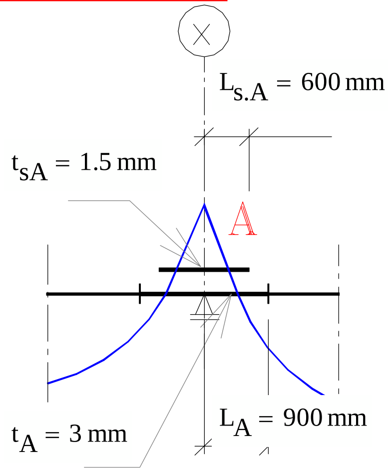
$$\left(\frac{M_{y.Ed.A.g}}{W_{yeff.A}} + \frac{N_{Ed}}{A_{eff.A}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}} = 1.36 \qquad <1,0$$

check with supporting profile:

$$\left(\frac{M_{y.Ed.A.g}}{W_{yeff.A} + W_{yeff.sA}} + \frac{N_{Ed}}{A_{eff.A}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}} = 0.94 \qquad <1,0$$

single profile att end of supporting profile:

$$M_{L.s.A} := M_{y.Ed.A.g} - \frac{L_{s.A}\cdot\left(M_{y.Ed.A.g} - M_{y.Ed.AB.g}\right)}{L_A} \qquad M_{L.s.A} = 1.1\,kN\cdot m$$



single profile at end of supporting profile:

$$\left(\frac{\left|M_{L.s.A}\right|}{W_{yeff.A}} + \frac{N_{Ed}}{A_{eff.A}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}} = 0.72 \qquad < 1,0$$

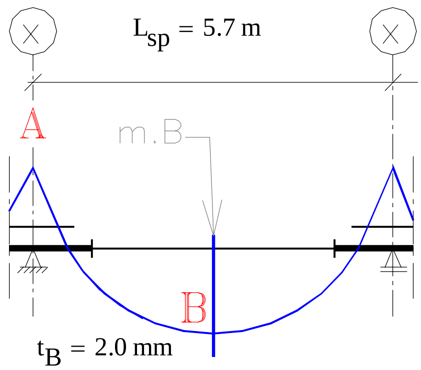
Bending and shear force:
Because $V_{Ed} < 0.5{*}V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

$$combined\_V\_and\_M := if\left(\frac{V_{Ed.A.g}\cdot\gamma_{M0}}{V_{bh.Rk.A}\cdot 0.5} \le 1.0, \text{"Not neccessary"}, \text{"Needs to be checked"}\right)$$

$$combined\_V\_and\_M = \text{"Not neccessary"}$$

Location B:

purlin: $t_B = 2.0\,mm$



Combined bending moment due to gravity load and compression force:
EN 1993-1-3:10.1.4.1 eqv. 10.3b

lateral bending moment: gravity load $\quad M_{fz.Ed.g} := 0\cdot kN\cdot m$
EN 1993-1-3:101.4.1(5)

single profile in span gravity load:

$$\left(\frac{M_{y.Ed.B.g}}{W_{yeff.B}} + \frac{N_{Ed}}{A_{eff.B}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}} = 0.78 \qquad < 1,0$$

single profile at connection between A and B, gravity load:

$$\left(\frac{\left|M_{y.Ed.AB.g}\right|}{W_{yeff.B}} + \frac{N_{Ed}}{A_{eff.B}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}} = 0.63 \qquad < 1,0$$

Bending and shear force:
Because $V_{Ed} < 0.5{*}V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

$$combined\_V\_and\_M := if\left(\frac{V_{Ed.A.AB.g}\cdot\gamma_{M0}}{V_{bh.Rk.B}\cdot 0.5} \le 1.0, \text{"Not neccessary"}, \text{"Needs to be checked"}\right)$$

$$combined\_V\_and\_M = \text{"Not neccessary"}$$

Check joint between purlins:

$$F_{b.Rk} := \left(\begin{array}{l} 18.9 \text{ if } span\_B = \text{"Z150x1.5"} \\ 25.2 \text{ if } span\_B = \text{"Z150x2"} \\ 37.8 \text{ if } span\_B = \text{"Z150x3"} \end{array}\right)\cdot kN \qquad F_{b.Rk} = 25.2\,kN$$

Moment for joint near support A:

$$M_{y.Ed.AB.g} = 0.61\,kN\cdot m$$

joint near support A gravity load:

$$\frac{\left|M_{y.Ed.AB.g}\right|\cdot\gamma_{M2}}{4\cdot F_{b.Rk}\cdot 0.075\cdot m} + \frac{N_{Ed}\cdot\gamma_{M2}}{4\cdot F_{b.Rk}} = 0.1 \qquad < 1,0$$

joint near support A uplift load:

$$\frac{\left|M_{y.Ed.AB.u}\right|\cdot\gamma_{M2}}{4\cdot F_{b.Rk}\cdot 0.075\cdot m} + \frac{N_{Ed}\cdot\gamma_{M2}}{4\cdot F_{b.Rk}} = 0.02 \qquad < 1,0$$

Continue Location B:

Buckling resistance of free flange in compression (uplift load) EN 1993-1-3: 10.1.4.2: The relevant position is "m.B".

lateral bending moment: uplift load $M_{fz.Ed.m.B} = 0.2\,N\cdot m$
EN 1993-1-3:101.4.1(5)

non-dim. slenderness as defined above: $\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 76.95$

Coefficients from table 10.2b: $\eta_1 := 0.306$ $\eta_2 := 0.232$ $\eta_3 := 0.742$ $\eta_4 := -0.279$

Buckling lenght for free flangein compression EN 1993-1-3: 10.1.4.2 (3)+(6): provided that 0< Rrp <200:

$R_{rp.check} = \text{"OK"}$

$$L_{f.z.B} := \eta_1 \cdot L_a \cdot \left(1 + \eta_2 \cdot R_{rp.B}^{\eta_3}\right)^{\eta_4} \qquad L_{f.z.B} = 1.18\,m$$

Relative slenderness for flexural buckling of free flange: $\lambda_{r.f.z.B} := \frac{L_{f.z.B}}{i_{z.fl.B}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.f.z.B} = 0.771$

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

$\alpha_{LT} := 0.34$ $\lambda_{r.LT.0} := 0.4$ $\beta := 0.75$ $\phi_{LT.B} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot \left(\lambda_{r.f.z.B} - \lambda_{r.LT.0}\right) + \beta \cdot \lambda_{r.f.z.B}^2\right]$ $\phi_{LT.B} = 0.79$

$$\chi_{LT.B} := \min\left(\frac{1}{\phi_{LT.B} + \sqrt{\phi_{LT.B}^2 - \beta \cdot \lambda_{r.f.z.B}^2}}, 1, \frac{1}{\lambda_{r.f.z.B}^2}\right) \qquad \chi_{LT.B} = 0.83$$

Reduction factor for flexural buckling: According to EN 1993-1-1: 6.3.1: $\lambda_1 = 76.95$ $i_{y.B} := \sqrt{\frac{I_{y.rc.B}}{A_{g.rc.B}}}$

$\lambda_{r.y.FB.B}(L) := \frac{L}{i_{y.B}} \cdot \sqrt{\frac{A_{eff.B}}{A_{g.rc.B}}} \cdot \frac{1}{\lambda_1}$ $N_{cr.y.B}(L) := \frac{\pi^2 \cdot E \cdot I_{y.rc.B}}{L^2}$ Imperfection factor α relating to bucking curve b $\alpha_{y.FB} := 0.34$

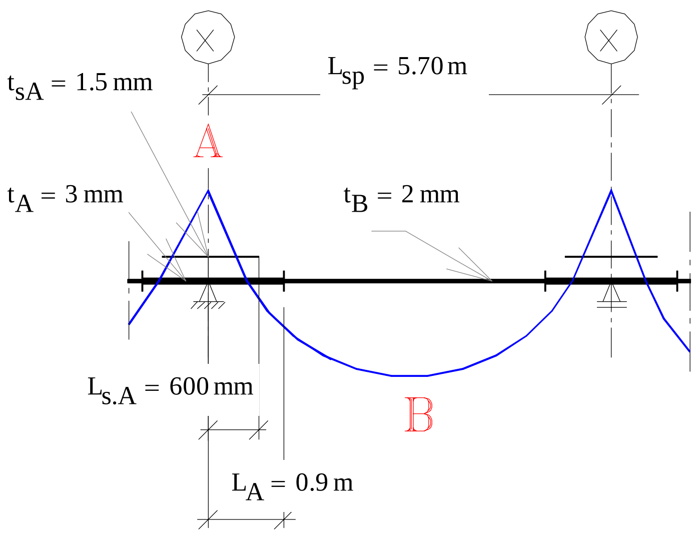
$$\phi_{y.FB.B}(L) := 0.5 \cdot \left[1 + \alpha_{y.FB} \cdot \left(\lambda_{r.y.FB.B}(L) - 0.2\right) + \lambda_{r.y.FB.B}(L)^2\right]$$

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

$\chi_{y.FB.B}(L_{sp}) = 0.55$ reduction factor for span B

single profile in span uplift load:

eqv. 10.7: $\left(\frac{1}{\chi_{LT.B}} \cdot \frac{M_{y.Ed.B.u}}{W_{yeff.B}} + \frac{1}{\min\left(\chi_{LT.B}, \chi_{y.FB.B}(L_{sp})\right)} \cdot \frac{N_{Ed}}{A_{eff.B}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} + \frac{\left|M_{fz.Ed.m.B}\right| \cdot \gamma_{M1}}{W_{z.fl.B} \cdot f_{yb}} = 0.18$ < 1,0



Posudek vaznice na R15 - krajní pole

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

$\begin{pmatrix} f_{yb} \\ f_u \end{pmatrix} := \begin{pmatrix} 42 \\ 50 \end{pmatrix} \cdot \frac{N}{mm^2}$

values for γ_{Mi}: $\gamma_{M0} := 1$ $\gamma_{M1} := 1$ $\gamma_{M2} := 1.25$

Global values: all measures in (mm)

$t_{rp} := 0.63 \cdot mm$ thickness roofplate ($_{rp}$)

$t_{rp.cor} := 0.5842 \cdot mm$ core thickness

$h_{w.rp} := 45 \cdot mm$ height roofplate ($_{rp}$)

$b_{roof} := 33302 \cdot mm$ width of roof ($_{roof}$)

$L_{sp} := 4152 \cdot mm$ span of purlin

$cc_{purlin} := 750 \cdot mm$ distance between purlins

$s_s := 108 \cdot mm$ Support width

Stresses on roof purlin

maximum gravity load $q_{Ed.g} := 0.68 \cdot \frac{kN}{m}$

maximum uplift load $q_{Ed.u} := 0.03 \cdot \frac{kN}{m}$

Compression force in purlin: $N_{Ed} := 0 \cdot kN$

Stresses due to gravity load:

Location C:

$M_{y.Ed.C.g} := 1.93 \cdot kN \cdot m$ Moment at support

$V_{Ed.C.g} := 1.91 \cdot kN$ Shear force at support

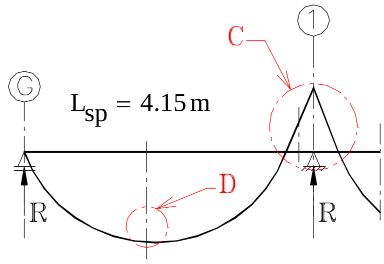
$R_{Ed.C.g} := 3.79 \cdot kN$ Reaktion force

$M_{y.Ed.CD.g} := 0.52 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)

$V_{Ed.CD.g} := 1.27 \cdot kN$ Shear force att connection between purlins (NOT support profile)

Location D:

$M_{y.Ed.D.g} := 0.66 \cdot kN \cdot m$ Moment in span



Stress due to uplift load:

Location D:

$M_{y.Ed.D.u} := 0.03 \cdot kN \cdot m$ Moment in span

$M_{y.Ed.CD.u} := 0.09 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)

Purlin profiles:

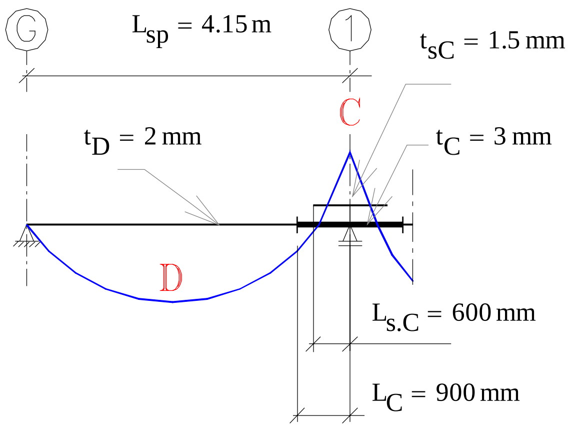
profile over support C Z150x1.5, Z150x2 or Z150x3: support_C := "Z150x3"

length of profile over C: $L_C := 900 \cdot mm$

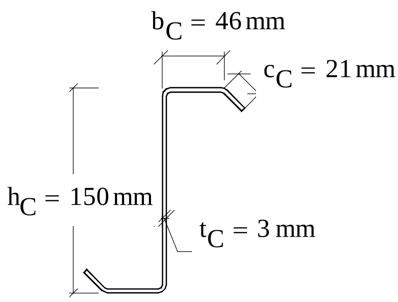
profile in span D Z150x1.5, Z150x2 or Z150x3: span_D := "Z150x2"

supporting profile over support location C ZX-1.5 , ZX-2 , ZX-3 or NO: sup_profile_C := "ZX-1.5"

Length of extra supporting profile over support C: $L_{s.C} := 600 \cdot mm$



Profile over support C:



$M_{yRk.C} = 12.79\,kN \cdot m$

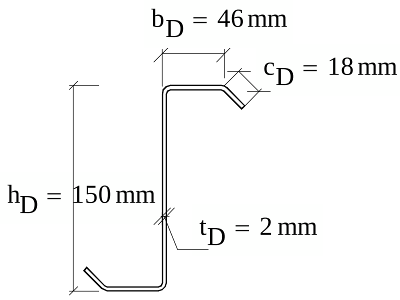
$V_{bh.Rk.C} = 89.11\,kN$

$R_{w.Rk.C} = 6.2\,kN$

$W_{yeff.C} = 3.66 \times 10^4\,mm^3$

$I_{y.C} = 2.69 \times 10^6\,mm^4$

Profile i span D:



$M_{yRk.D} = 8.07\,kN \cdot m$

$V_{bh.Rk.D} = 47.57\,kN$

$W_{yeff.D} = 2.31 \times 10^4\,mm^3$

$I_{y.D} = 1.8 \times 10^6\,mm^4$

$I_{T.D} = 726.25\,mm^4$

$I_{\omega.D} = 1.125 \times 10^9\,mm^6$

Shear stiffness of trapezoidal sheeting connected to purlin EN 1993-1-3: 10.1.1 (6)+(10):

Shear stiffness roof:

$X := 0.5$ only one half of the roof is accounted for

$$S_{roof} := \sqrt{\left(\frac{t_{rp}}{mm}\right)^3} \cdot \left[50 + 10 \cdot \sqrt[3]{\left(\frac{b_{roof} \cdot X}{mm}\right)}\right] \cdot \frac{cc_{purlin}}{h_{w.rp}} \qquad S_{roof} = 2544.88 \quad \text{in kN}$$

purlin:

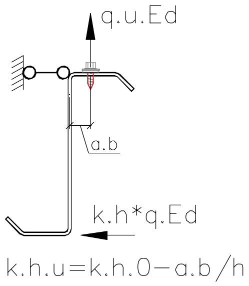
$$Z_{purlin} := \left(E \cdot I_{\omega} \cdot \frac{\pi^2}{L_{sp}^2} + G \cdot I_T + E \cdot I_z \cdot \frac{\pi^2}{L_{sp}^2} \cdot 0.25 \cdot h_D^2\right) \cdot \frac{70}{h_D^2} \qquad Z_{purlin} = 148\,kN$$

$Restrained := if\left(S_{roof} \cdot kN \geq Z_{purlin}, \text{"Purlins are laterally restrained"}, \text{"No laterally restrain"}\right)$

$Restrained =$ "Purlins are laterally restrained"

The purlin at the connection is regarded as beeing laterally restrained in plane of the sheeting.

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



$$k_{h.0.D} := \frac{h_{cD} \cdot t_D \cdot \left(b_{cD}^2 + 2 \cdot b_{cD} \cdot c_{cD} - 2 \cdot c_{cD}^2 \cdot \frac{b_{cD}}{h_{cD}}\right)}{4 \cdot I_{y.D}} \qquad a_{b.D} = 27.6\,mm \qquad k_{h.0.D} = 0.139$$

Equivalent lateral load acting on free flange, due to torsion and lateral bending (EN 1993-1-3: 10.1.4.1 (3)

$$k_{h.u.D} := k_{h.0.D} - \frac{a_{b.D}}{h_{cD}} \qquad q_{h.Ed.u.D} := q_{Ed.u} \cdot k_{h.u.D} \qquad q_{h.Ed.u.D} = -0\,\frac{kN}{m}$$

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): C_D

tension in upper flange: $I_{eff.TP46.t} := 125400.93 \cdot mm^4 \cdot \frac{1}{m}$ $\quad C_{D.C.t} := \frac{6 \cdot E \cdot I_{eff.TP46.t}}{cc_{purlin}}$ $\quad C_{D.C.t} = 2.53 \times 10^4 \frac{N \cdot m}{m}$

compression in upper flange: $I_{eff.TP46.c} := 130983.88 \cdot mm^4 \cdot \frac{1}{m}$ $\quad C_{D.C.c} := \frac{6 \cdot E \cdot I_{eff.TP46.c}}{cc_{purlin}}$ $\quad C_{D.C.c} = 2.641 \times 10^4 \frac{N \cdot m}{m}$

Rotational stiffness of connection between sheeting and purlin EN 1993-1-3: 10.1.5.2 (5)

$$k_{t.g} := \left(\frac{t_{rp}}{0.75}\right)^{1.5} \cdot \frac{1}{mm^{1.5}} \qquad k_{t.g} = 0.77 \quad \text{for b<125mm}$$

for uplift load pin in every trough $C_{100.cc150.u} := 2.6 \cdot \frac{kN \cdot m}{m}$ pin in alternate trough $C_{100.cc300.u} := 1.7 \cdot \frac{kN \cdot m}{m}$

$b := 46 \cdot mm$

$$C_{D.A.cc150.u} := C_{100.cc150.u} \cdot \left(\frac{\frac{b}{mm}}{100}\right)^2 \cdot k_{t.g} \qquad C_{D.A.cc300.u} := C_{100.cc300.u} \cdot \left(\frac{\frac{b}{mm}}{100}\right)^2 \cdot k_{t.g}$$

Rotational stiffness:

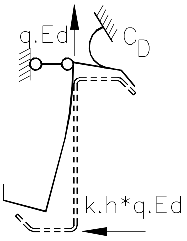
$$C_{D.cc150.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc150.u}} + \frac{1}{C_{D.C.t}}\right)} \qquad C_{D.cc150.u} = 0.417\,\frac{kN \cdot m}{m} \quad \text{pin in every trough}$$

$$C_{D.cc300.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc300.u}} + \frac{1}{C_{D.C.t}}\right)} \qquad C_{D.cc300.u} = 0.274\,\frac{kN \cdot m}{m} \quad \text{pin in alternate trough}$$

For uplift load
EN 1993-1-3: 10.1.5.1 (4)

$$b_{mod.u.D} := \begin{cases} a_{b.D} & \text{if } q_{h.Ed.u.D} \geq 0 \\ \left(2 \cdot a_{b.D} + b\right) & \text{if } q_{h.Ed.u.D} < 0 \end{cases} \qquad b_{mod.u.D} = 101.2\,mm$$

Span D:



Flexibility from web in bending:

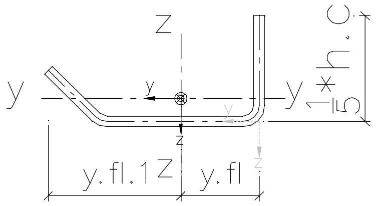
$$1/K_{B.u.D} := \frac{4 \cdot \left(1 - \upsilon^2\right) \cdot h_D^2 \cdot \left(h_D + b_{mod.u.D}\right)}{E \cdot t_D^3} \qquad 1/K_{B.u.D} = 102.05\,\frac{mm^2}{N}$$

$$K_{u.cc150.D} := \frac{1}{\left(K_{B.u.D} + \frac{h_D^2}{C_{D.cc150.u}}\right)} \qquad K_{u.cc300.D} := \frac{1}{\left(K_{B.u.D} + \frac{h_D^2}{C_{D.cc300.u}}\right)}$$

Lateral spring stiffnes per unit length:

$K_{u.cc150.D} = 0.0064\,\frac{N}{mm^2}$ pin in every trough $\qquad K_{u.cc300.D} = 0.0054\,\frac{N}{mm^2}$ n in alternate trough

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

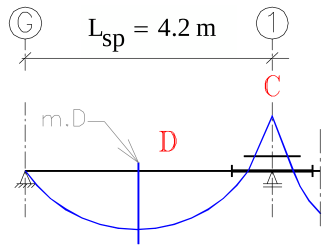


Span D: $t_D = 2.0\,mm$

$I_{z.fl.D} = 70968\,mm^4$

$W_{z.fl.D} = 1965.91\,mm^3$

$i_{z.fl.D} = 19.87\,mm$



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):

Span D $\quad R_{rp.D} := \dfrac{K_{u.cc300.D} \cdot L_a^{\,4}}{\pi^4 \cdot E \cdot I_{z.fl.D}} \qquad R_{rp.D} = 9.26$

correction factor κ_R and initial moment $M_{0.fz.Ed}$ acc. to table 10.1 EN 1993-1-3: 10.1.4.1

Location: $\quad M_{0.fz.Ed}$: $\qquad \kappa_R$:

$m_D \quad M_{0.fz.Ed.m.D} := \dfrac{9}{128} \cdot \left|q_{h.Ed.u.D}\right| \cdot L_a^{\,2} \quad \kappa_{R.m.D.2} := \dfrac{1 - 0.0141 \cdot R_{rp.D}}{1 + 0.416 \cdot R_{rp.D}}$

lateral bending moment from EN 1993-1-3: 10.1.4.1(5):

$M_{fz.Ed}$:

$M_{fz.Ed.m.D} := \kappa_{R.m.D.2} \cdot M_{0.fz.Ed.m.D}$

$M_{fz.Ed.m.D} = 0.3\,N \cdot m$

Stresses due to gravity load as given above:

Location C:

$M_{y.Ed.C.g} = 1.93\,kN \cdot m \quad M_{y.Ed.CD.g} = 0.52\,kN \cdot m$

$V_{Ed.C.g} = 1.91\,kN \quad V_{Ed.CD.g} = 1.27\,kN$

$R_{Ed.C.g} = 3.79\,kN$

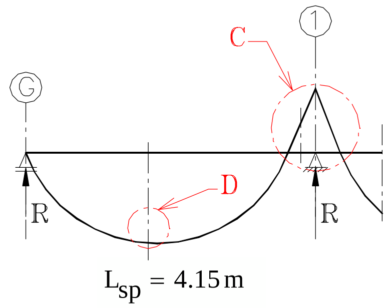
Location D:

$M_{y.Ed.D.g} = 0.66\,kN \cdot m$

Stresses due to uplift load as given above:

Location D:

$M_{y.Ed.D.u} = 0.03\,kN \cdot m$



Location C:

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

purlin: $t_C = 3.0\,mm$ supporting profile over support: sup_profile_C = "ZX-1.5" length of supp. profile: $L_{s.C} = 600\,mm$

check single profile:

$$\frac{M_{y.Ed.C.g} \cdot \gamma_{M0}}{M_{yRk.C}} + \frac{R_{Ed.C.g} \cdot \gamma_{M1}}{R_{w.Rk.C}} = 0.76 \quad < 1{,}25$$

check with supporting profile:

$$\frac{M_{y.Ed.C.g} \cdot \gamma_{M0}}{M_{yRk.C} + M_{yRk.sC}} + \frac{R_{Ed.C.g} \cdot \gamma_{M1}}{R_{w.Rk.C} + R_{w.Rk.sC}} = 0.57 \quad < 1{,}25$$

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

single profile at support:

$$\left(\frac{M_{y.Ed.C.g}}{W_{yeff.C}} + \frac{N_{Ed}}{A_{eff.C}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} = 1.26 \quad <1{,}0$$

check with supporting profile:

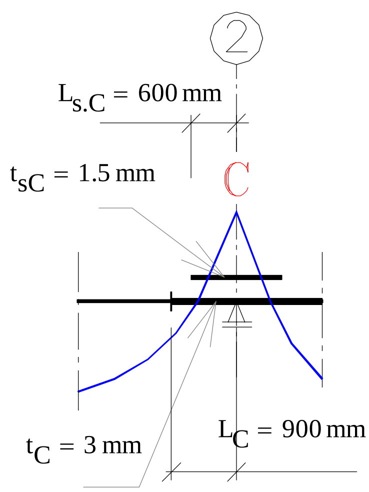
$$\left(\frac{M_{y.Ed.C.g}}{W_{yeff.C} + W_{yeff.sC}} + \frac{N_{Ed}}{A_{eff.C}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} = 0.87 \quad <1{,}0$$

single profile att end of supporting profile:

$$M_{L.s.C} := M_{y.Ed.C.g} - \frac{L_{s.C} \cdot \left(M_{y.Ed.C.g} - M_{y.Ed.CD.g}\right)}{L_C} \qquad M_{L.s.C} = 0.99\,kN \cdot m$$

single profile at end of supporting profile:

$$\left(\frac{\left|M_{L.s.C}\right|}{W_{yeff.C}} + \frac{N_{Ed}}{A_{eff.C}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} = 0.64 \quad < 1{,}0$$

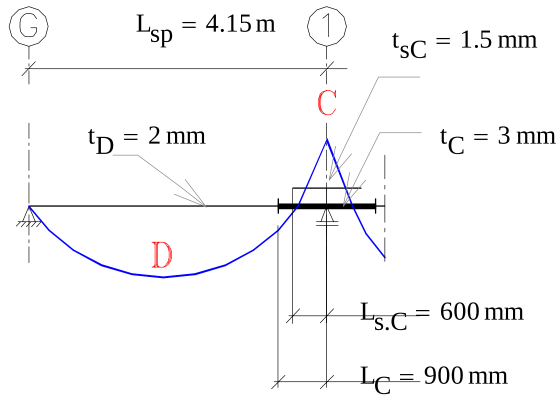


Bending and shear force:
Because $V_{Ed} < 0.5 {*} V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

$$\text{combined\_V\_and\_M} := \text{if}\left(\frac{V_{Ed.C.g} \cdot \gamma_{M0}}{V_{bh.Rk.C} \cdot 0.5} \le 1.0, \text{"Not neccessary"}, \text{"Needs to be checked"}\right)$$

combined_V_and_M = "Not neccessary"

Location D: purlin: $t_D = 2.0\,mm$



Combined bending moment due to gravity load and compression force:
EN 1993-1-3:10.1.4.1 eqv. 10.3b

lateral bending moment: gravity load $M_{fz.Ed.g} := 0 \cdot kN \cdot m$
EN 1993-1-3:101.4.1(5)

single profile in span gravity load:

$$\left(\frac{M_{y.Ed.D.g}}{W_{yeff.D}} + \frac{N_{Ed}}{A_{eff.D}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} = 0.68 \qquad <1,0$$

single profile at connection between A and B, gravity load:

$$\left(\frac{\left|M_{y.Ed.CD.g}\right|}{W_{yeff.D}} + \frac{N_{Ed}}{A_{eff.D}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} = 0.54 \qquad < 1,0$$

Bending and shear force:
Because $V_{Ed} < 0.5^*V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

combined_V_and_M = "Not neccessary"

Check joint between purlins:

$$F_{b.Rk} := \left(\begin{array}{l} 18.9 \text{ if } span\_D = \text{"Z150x1.5"} \\ 25.2 \text{ if } span\_D = \text{"Z150x2"} \\ 37.8 \text{ if } span\_D = \text{"Z150x3"} \end{array}\right) \cdot kN \qquad F_{b.Rk} = 25.2\,kN$$

joint near support C:

$$\frac{\max\left(\left|M_{y.Ed.CD.g}\right|, \left|M_{y.Ed.CD.u}\right|\right) \cdot \gamma_{M2}}{4 \cdot F_{b.Rk} \cdot 0.075 \cdot m} + \frac{N_{Ed} \cdot \gamma_{M2}}{4 \cdot F_{b.Rk}} = 0.09 \qquad < 1,0$$

Buckling resistance of free flange in compression (uplift load) EN 1993-1-3: 10.1.4.2: The relevant position is "m.D".

lateral bending moment: uplift load $M_{fz.Ed.m.D} = 0.3\,N \cdot m$
EN 1993-1-3:10.1.4.1(5)

non-dim. slenderness as defined above: $\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 76.95$

Coefficients from table 10.2b: $\eta_1 := 0.515$ $\eta_2 := 1.26$ $\eta_3 := 0.868$ $\eta_4 := -0.242$

Buckling lenght for free flangein compression
EN 1993-1-3: 10.1.4.2 (3)+(6): provided that 0< Rrp <200: $R_{rp.check}$ = "OK"

$$L_{f.z.D} := \eta_1 \cdot L_a \cdot \left(1 + \eta_2 \cdot R_{rp.D}^{\eta_3}\right)^{\eta_4} \qquad L_{f.z.D} = 1.23\,m$$

Relative slenderness for flexural buckling of free flange:

$$\lambda_{r.f.z.D} := \frac{L_{f.z.D}}{i_{z.fl.D}} \cdot \frac{1}{\lambda_1} \qquad \lambda_{r.f.z.D} = 0.807$$

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

$\alpha_{LT} := 0.34$ $\lambda_{r.LT.0} := 0.4$ $\beta := 0.75$

$$\phi_{LT.D} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot \left(\lambda_{r.f.z.D} - \lambda_{r.LT.0}\right) + \beta \cdot \lambda_{r.f.z.D}^2\right] \qquad \phi_{LT.D} = 0.81$$

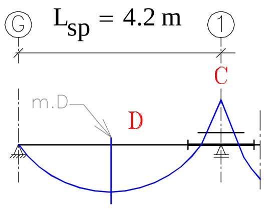
$$\chi_{LT.D} := \min\left(\frac{1}{\phi_{LT.D} + \sqrt{\phi_{LT.D}^2 - \beta \cdot \lambda_{r.f.z.D}^2}}, 1, \frac{1}{\lambda_{r.f.z.D}^2}\right) \quad LT.D = 0.81$$

$$i_{y.D} := \sqrt{\frac{I_{y.rc.D}}{A_{g.rc.D}}} \qquad \lambda_1 = 76.95$$

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

Imperfection factor α relating to bucking curve b $\alpha_{y.FB} := 0.34$

$$\lambda_{r.y.FB.D}(L) := \frac{L}{i_{y.D}} \cdot \sqrt{\frac{A_{eff.D}}{A_{g.rc.D}}} \cdot \frac{1}{\lambda_1} \qquad N_{cr.y.D}(L) := \frac{\pi^2 \cdot E \cdot I_{y.rc.D}}{L^2}$$



$$\phi_{y.FB.D}(L) := 0.5 \cdot \left[1 + \alpha_{y.FB} \cdot \left(\lambda_{r.y.FB.D}(L) - 0.2\right) + \lambda_{r.y.FB.D}(L)^2\right]$$

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

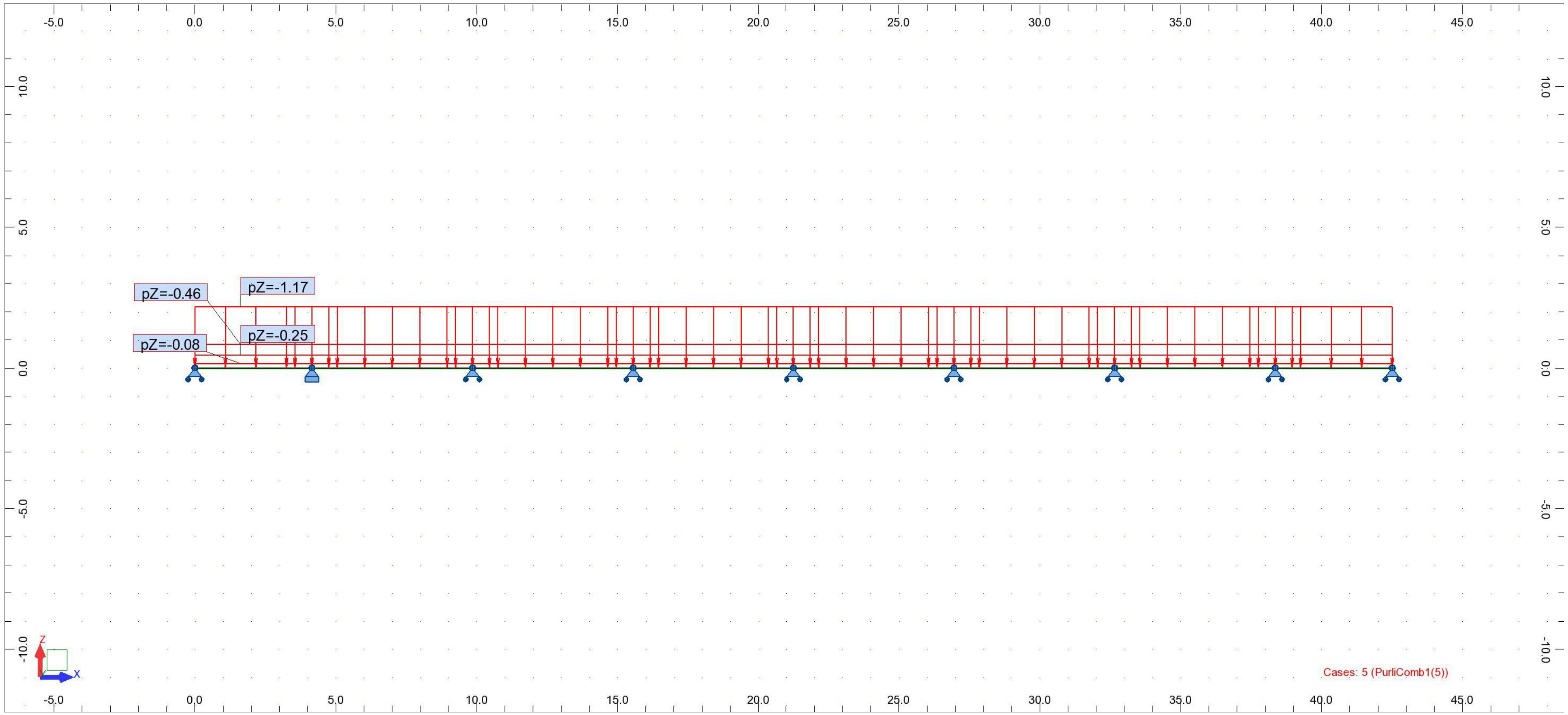
$\chi_{y.FB.D}(L_{sp}) = 0.74$ reduction factor for span D

single profile in span uplift load:

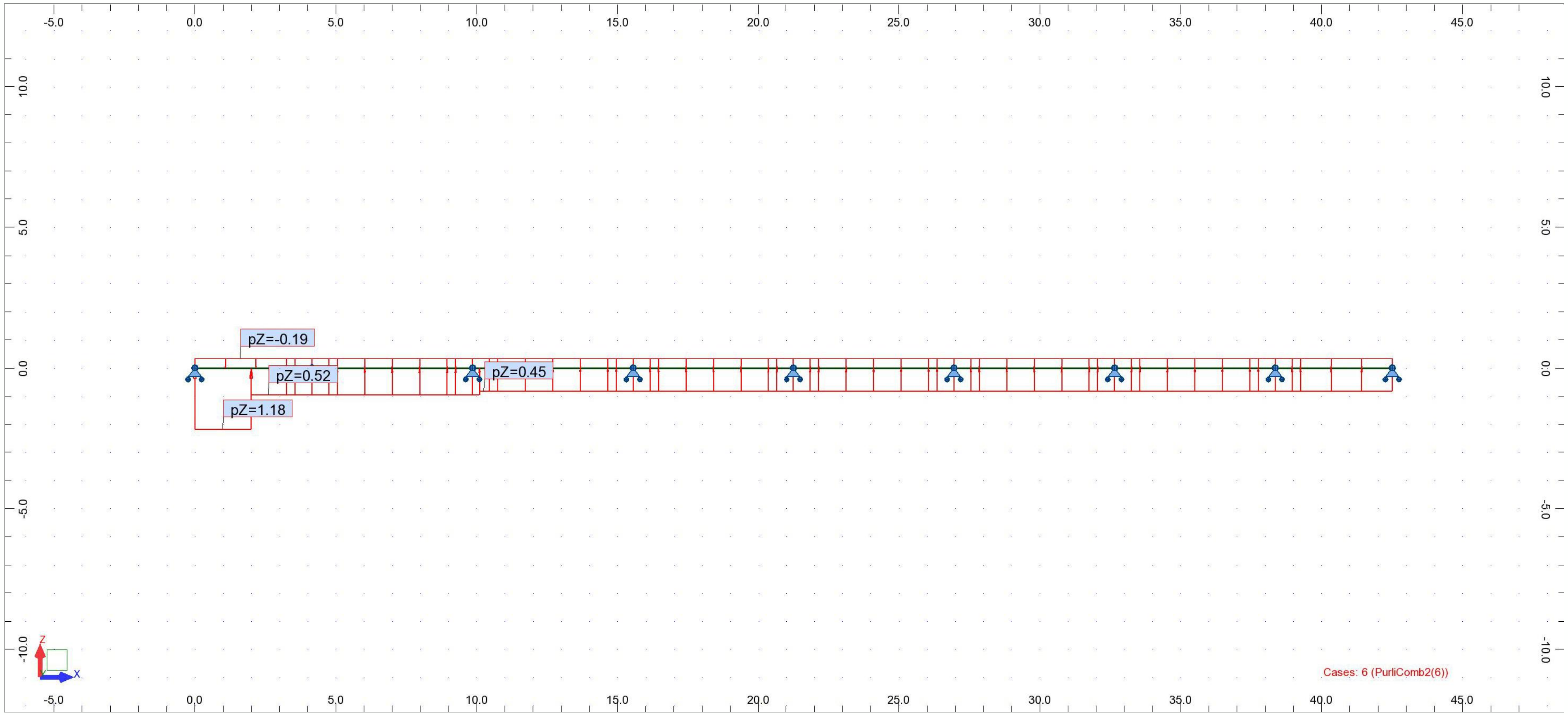
eqv. 10.7

$$\left(\frac{1}{\chi_{LT.D}} \cdot \frac{M_{y.Ed.D.u}}{W_{yeff.D}} + \frac{1}{\min\left(\chi_{LT.D}, \chi_{y.FB.D}(L_{sp})\right)} \cdot \frac{N_{Ed}}{A_{eff.D}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} + \frac{\left|M_{fz.Ed.m.D}\right| \cdot \gamma_{M1}}{W_{z.fl.D} \cdot f_{yb}} = 0.04 \qquad < 1,0$$

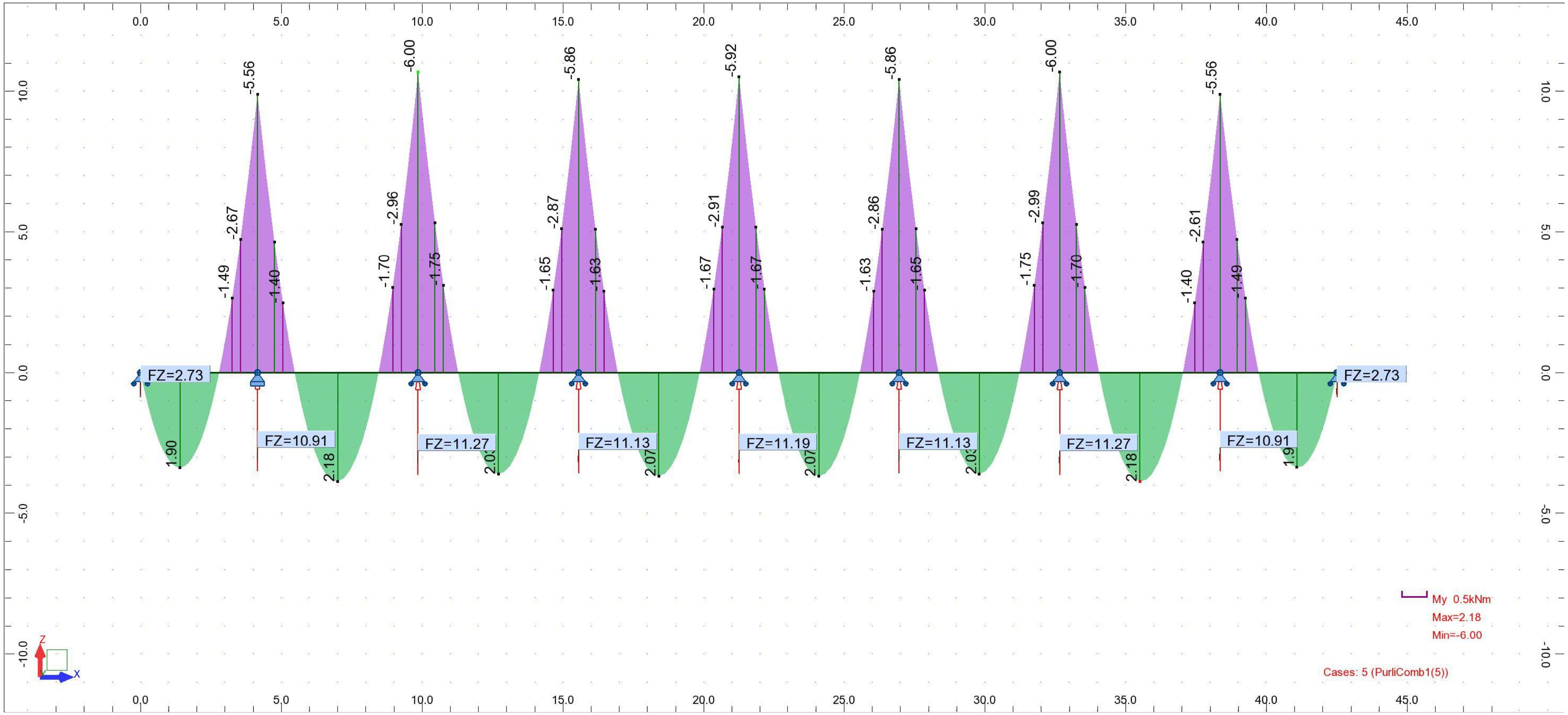
View - Cases: 5 (PurliComb1(5)) Zatížení - tlak



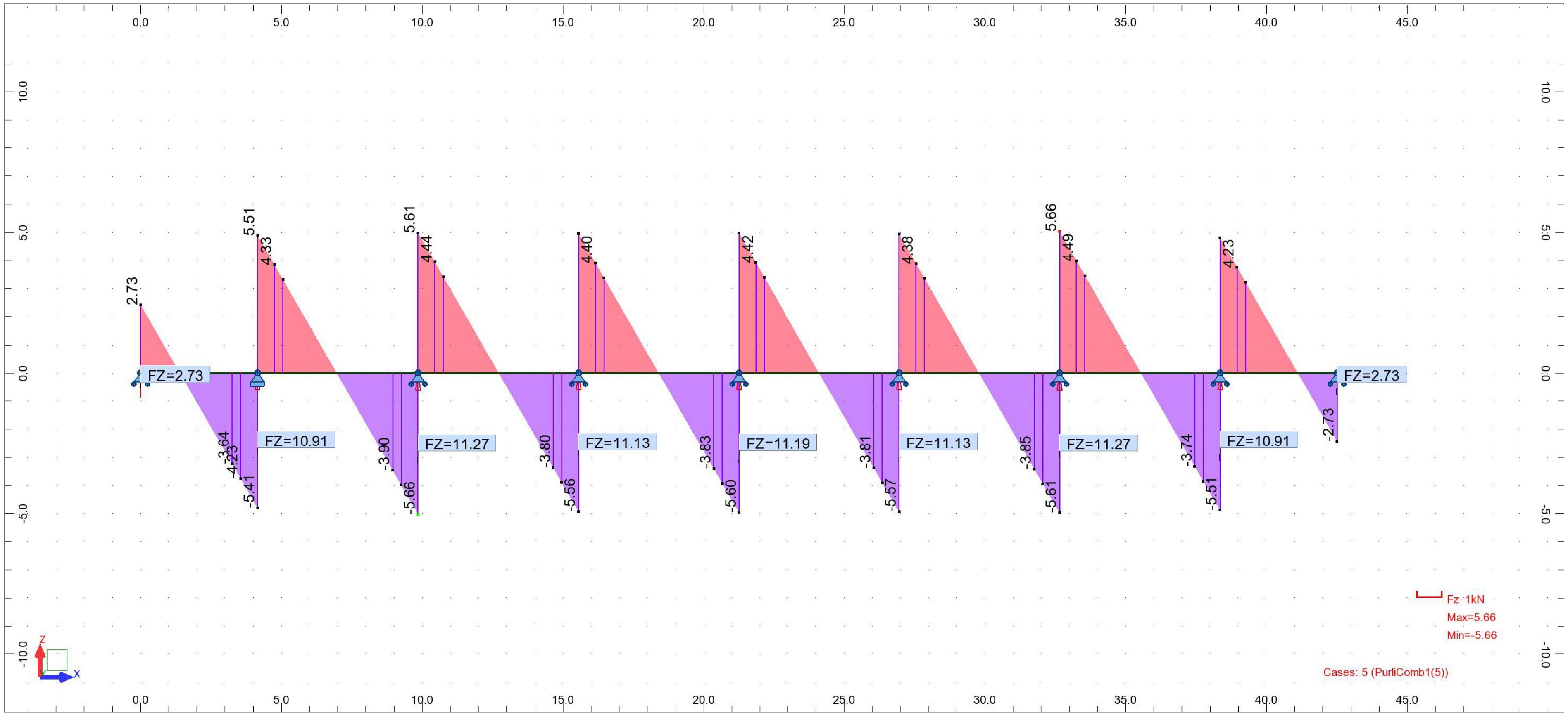
View - Cases: 6 (PurliComb2(6)) Zatížení - sání



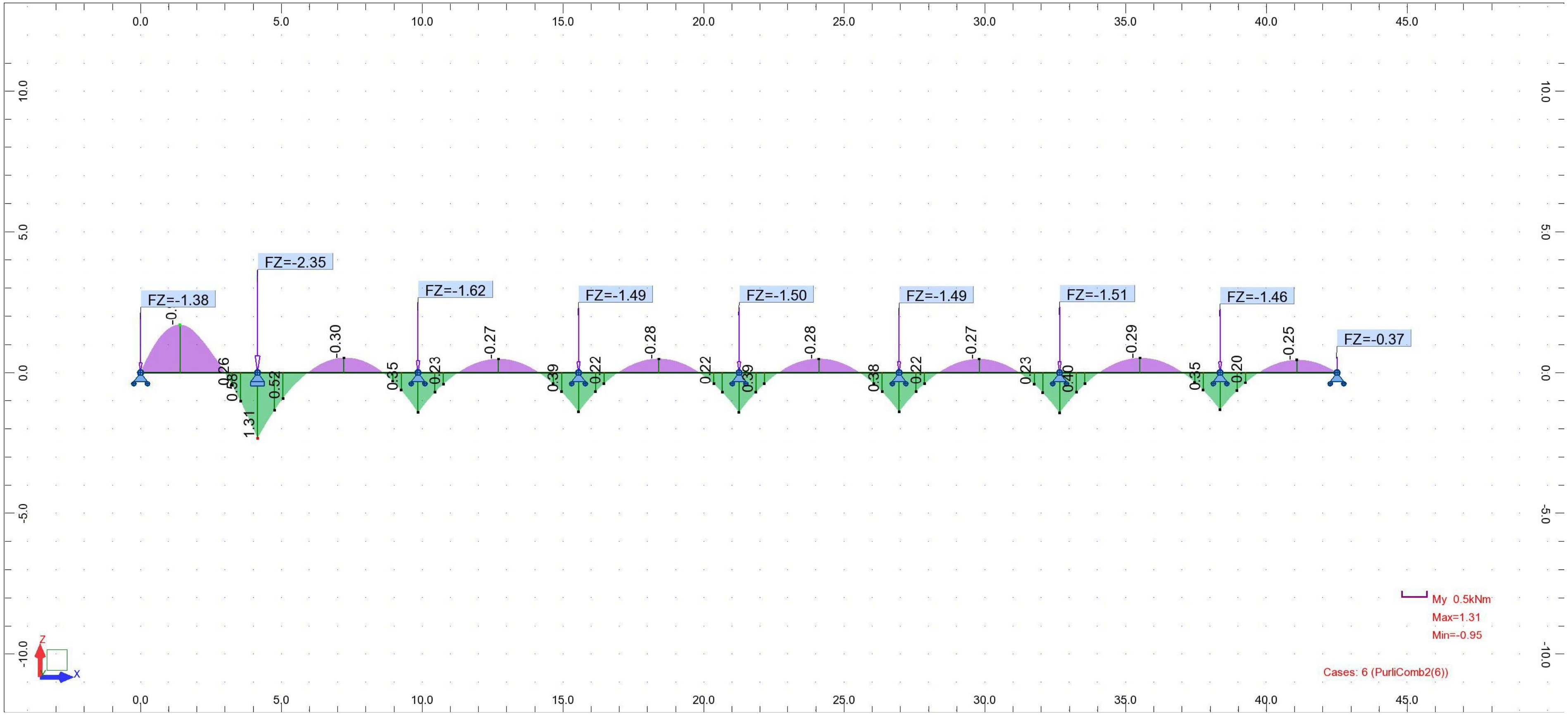
View - MY;Reaction forces(kN,kN/m); Cases: 5 (PurliComb1(5)) Ohybové momenty + reakce - zatížení tlak



View - FZ;Reaction forces(kN,kN/m); Cases: 5 (PurliComb1(5)) Posouvající síly + reakce - zatížení tlak



View - MY;Reaction forces(kN,kN/m); Cases: 6 (PurliComb2(6)) Ohybové momenty + reakce - zatížení sání



Posudek vaznice - vnitřní pole

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

$N := newton \quad kN := 1000 \cdot N$ $\quad \begin{pmatrix} f_{yb} \\ f_u \end{pmatrix} := \begin{pmatrix} 350 \\ 420 \end{pmatrix} \cdot \frac{N}{mm^2}$

values for γ_{Mi}: $\gamma_{M0} := 1.0$ $\quad \gamma_{M1} := 1.0$ $\quad \gamma_{M2} := 1.25$

Global values: all measures in (mm)

$t_{rp} := 0.62 \cdot mm$ thickness roofplate (.rp)

$t_{rp.cor} := 0.5842 \cdot mm$ core thickness

$h_{w.rp} := 45 \cdot mm$ height roofplate (.rp)

$b_{roof} := 33302 \cdot mm$ width of roof (.roof)

$L_{sp} := 5700 \cdot mm$ span of purlin

$cc_{purlin} := 750 \cdot mm$ distance between purlins

$s_s := 108 \cdot mm$ Support width

Stresses on roof purlin

maximum gravity load $q_{Ed.g} := 1.96 \cdot \frac{kN}{m}$

maximum upplift load $q_{Ed.u} := 1.0 \cdot \frac{kN}{m}$

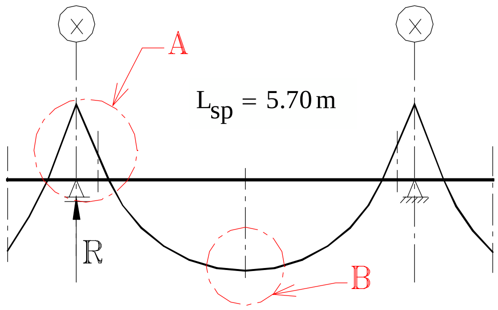
Compression force in purlin: $N_{Ed} := 5 \cdot kN$

Stresses due to upplift load:

Location B:

$M_{y.Ed.B.u} := 0.3 \cdot kN \cdot m$ Moment in span

$M_{y.Ed.AB.u} := 0.23 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)



Stresses due to gravity load:

Location A:

$M_{y.Ed.A.g} := 6 \cdot kN \cdot m$ Moment at support

$V_{Ed.A.g} := 5.66 \cdot kN$ Shear force at support

$R_{Ed.A.g} := 11.27 \cdot kN$ Reaktion force

$M_{y.Ed.AB.g} := 1.75 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)

$V_{Ed.A.AB.g} := 3.9 \cdot kN$ Shear force att connection between purlins (NOT support profile)

Location B:

$M_{y.Ed.B.g} := 2.18 \cdot kN \cdot m$ Moment in span

Purlin profiles:

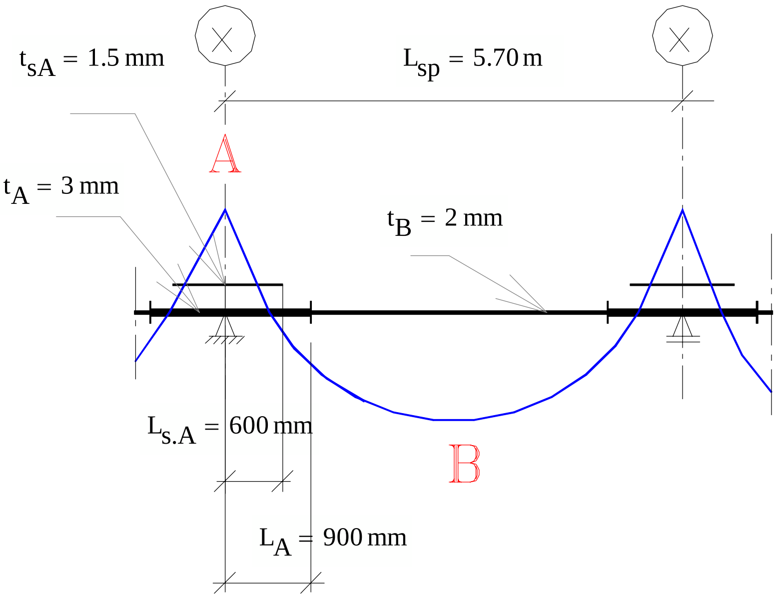
profile over support A Z150x1.5, Z150x2 or Z150x3: support_A := "Z150x3"

length of profile over A: $L_A := 900 \cdot mm$

profile in span B Z150x1.5, Z150x2 or Z150x3: span_B := "Z150x2"

supporting profile over support location A ZX-1.5 , ZX-2 , ZX-3 or NO: sup_profile_A := "ZX-1.5"

Lenght of extra supporting profile over support A: $L_{s.A} := 600 \cdot mm$



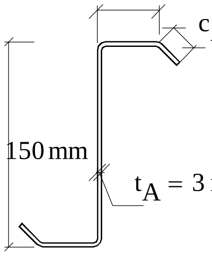
Profile over support A:

$b_A = 46\,mm$

$c_A = 21\,mm$

$h_A = 150\,mm$

$t_A = 3\,mm$



$M_{yRk.A} = 12.79\,kN\cdot m$

$V_{bh.Rk.A} = 89.11\,kN$

$R_{w.Rk.A} = 51.679\,kN$

$W_{yeff.A} = 3.66\times10^{4}\,mm^{3}$

$I_{y.A} = 2.69\times10^{6}\,mm^{4}$

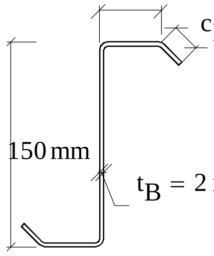
Profile i span B:

$b_B = 46\,mm$

$c_B = 18\,mm$

$h_B = 150\,mm$

$t_B = 2\,mm$



$M_{yRk.B} = 8.07\,kN\cdot m$

$V_{bh.Rk.B} = 47.57\,kN$

$W_{yeff.B} = 2.31\times10^{4}\,mm^{3}$

$I_{y.B} = 1.8\times10^{6}\,mm^{4}$

$I_{T.B} = 726.25\,mm^{4}$

$I_{\omega.B} = 1.125\times10^{9}\,mm^{6}$

Shear stiffness of trapezoidal sheeting connected to purlin EN 1993-1-3: 10.1.1 (6)+(10):

Shear stiffness roof:

$X := 0.5$ only one half of the roof is accounted for

$$S_{roof} := \sqrt{\left(\frac{t_{rp}}{mm}\right)^3}\cdot\left[50 + 10\cdot\sqrt[3]{\left(\frac{b_{roof}\cdot X}{mm}\right)}\right]\cdot\frac{cc_{purlin}}{h_{w.rp}}$$

$S_{roof} = 2484.52$ in kN

purlin:

$$Z_{purlin} := \left(E\cdot I_\omega\cdot\frac{\pi^2}{L_{sp}^2} + G\cdot I_T + E\cdot I_z\cdot\frac{\pi^2}{L_{sp}^2}\cdot 0.25\cdot h_B^2\right)\cdot\frac{70}{h_B^2}$$

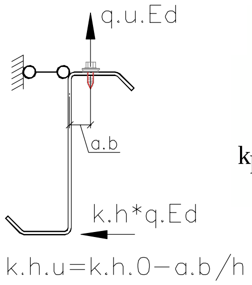
$Z_{purlin} = 740\,kN$

$Restrained := if\left(S_{roof}\cdot kN \geq Z_{purlin}, \text{"Purlins are laterally restrained"}, \text{"No laterally restrain"}\right)$

The purlin at the connection is regarded as beeing laterally restrained in plane of the sheeting.

$Restrained = \text{"Purlins are laterally restrained"}$

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



$a_{b.B} = 27.6\,mm$

$$k_{h.0.B} := \frac{h_{cB}\cdot t_B\cdot\left(b_{cB}^2 + 2\cdot b_{cB}\cdot c_{cB} - 2\cdot c_{cB}^2\cdot\frac{b_{cB}}{h_{cB}}\right)}{4\cdot I_{y.B}}$$

$k_{h.0.B} = 0.139$

Equivalent lateral load acting on free flange, due to torsion and lateral bending (EN 1993-1-3: 10.1.4.1 (3)

$k_{h.u.B} := k_{h.0.B} - \frac{a_{b.B}}{h_{cB}}$ $\quad h.Ed.u.B := q_{Ed.u}\cdot k_{h.u.B}$

$q_{h.Ed.u.B} = -0.05\,\frac{kN}{m}$

<u>**Rotational restraint given by sheeting EN 1993-13 10.1.5**</u>

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): C_D

tension in upper flange: $I_{eff.TP46.t} := 125400.93\cdot mm^4\cdot\frac{1}{m}$ $\qquad C_{D.C.t} := \frac{6\cdot E\cdot I_{eff.TP46.t}}{cc_{purlin}}$ $\qquad C_{D.C.t} = 2.11\times10^{5}\,\frac{N\cdot m}{m}$

compression in upper flange: $I_{eff.TP46.c} := 130983.88\cdot mm^4\cdot\frac{1}{m}$ $\qquad C_{D.C.c} := \frac{6\cdot E\cdot I_{eff.TP46.c}}{cc_{purlin}}$ $\qquad C_{D.C.c} = 2.201\times10^{5}\,\frac{N\cdot m}{m}$

Rotational stiffness of connection between sheeting and purlin EN 1993-1-3: 10.1.5.2 (5)

$k_{t.g} := \left(\frac{t_{rp}}{0.75}\right)^{1.5}\cdot\frac{1}{mm^{1.5}}$ $\qquad k_{t.g} = 0.752$ for b<125mm

for uplift load pin in every trough $C_{100.cc150.u} := 2.6\cdot\frac{kN\cdot m}{m}$ $\qquad$ pin in alternate trough $C_{100.cc300.u} := 1.7\cdot\frac{kN\cdot m}{m}$

$b := 46\cdot mm$

$$C_{D.A.cc150.u} := C_{100.cc150.u}\cdot\left(\frac{\frac{b}{mm}}{100}\right)^2\cdot k_{t.g}$$ $C_{D.A.cc150.u} = 0.4\,\frac{kN\cdot m}{m}$

$$C_{D.A.cc300.u} := C_{100.cc300.u}\cdot\left(\frac{\frac{b}{mm}}{100}\right)^2\cdot k_{t.g}$$ $C_{D.A.cc300.u} = 0.3\,\frac{kN\cdot m}{m}$

Rotational stiffness:

$$C_{D.cc150.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc150.u}} + \frac{1}{C_{D.C.t}}\right)}$$

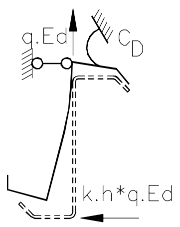
$C_{D.cc150.u} = 0.413\,\frac{kN\cdot m}{m}$ pin in every trough

$$C_{D.cc300.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc300.u}} + \frac{1}{C_{D.C.t}}\right)}$$

$C_{D.cc300.u} = 0.27\,\frac{kN\cdot m}{m}$ pin in alternate trough

For uplift load EN 1993-1-3: 10.1.5.1 (4)

Span B:



$$b_{mod.u} := \begin{cases} a_{b.B} & \text{if } q_{h.Ed.u.B} \geq 0 \\ (2 \cdot a_{b.B} + b) & \text{if } q_{h.Ed.u.B} < 0 \end{cases}$$

Flexibility from web in bending:

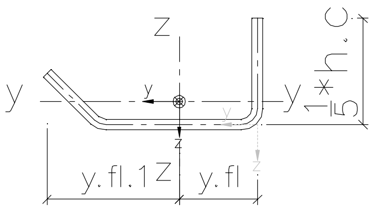
$$1/K_{B.u.B} := \frac{4 \cdot (1 - \upsilon^2) \cdot h_B^2 \cdot (h_B + b_{mod.u})}{E \cdot t_B^3} \qquad 1/K_{B.u.B} = 12.246 \frac{mm^2}{N}$$

Lateral spring stiffnes per unit length:

$$K_{u.cc150.B} := \frac{1}{\left(K_{B.u.B} + \frac{h_B^2}{C_{D.cc150.u}}\right)} \qquad K_{u.cc300.B} := \frac{1}{\left(K_{B.u.B} + \frac{h_B^2}{C_{D.cc300.u}}\right)}$$

$K_{u.cc150.B} = 0.0150 \frac{N}{mm^2}$ pin in every trough $\qquad K_{u.cc300.B} = 0.0105 \frac{N}{mm^2}$ pin in alternate trough

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



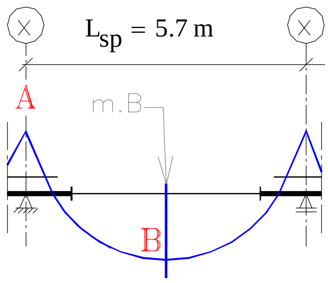
Span B: $t_B = 2.0\,mm$

$I_{z.fl.B} = 70968\,mm^4$

$W_{z.fl.B} = 1965.91\,mm^3$

$i_{z.fl.B} = 19.87\,mm$

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):

Span B $\qquad R_{rp.B} := \frac{K_{u.cc300.B} \cdot L_a^4}{\pi^4 \cdot E \cdot I_{z.fl.B}} \qquad R_{rp.B} = 7.61$

correction factor κ_R and initial moment $M_{0.fz.Ed}$ acc. to table 10.1 EN 1993-1-3: 10.1.4.1

Location: $M_{0.fz.Ed}$: $\qquad \kappa_R$:

lateral bending moment from EN 1993-1-3: 10.1.4.1(5): $M_{fz.Ed}$:

$m_B \qquad M_{0.fz.Ed.m.B} := \frac{1}{24} \cdot |q_{h.Ed.u.B}| \cdot L_a^2 \qquad \kappa_{R.m.B} := \frac{1 - 0.0125 \cdot R_{rp.B}}{1 + 0.198 \cdot R_{rp.B}}$

$M_{fz.Ed.m.B} := \kappa_{R.m.B} \cdot M_{0.fz.Ed.m.B}$

$M_{fz.Ed.m.B} = 23.4\,N \cdot m$

Stresses due to gravity load as given above:

Location A:

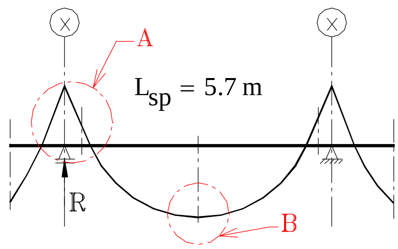
$M_{y.Ed.A.g} = 6\,kN \cdot m \qquad M_{y.Ed.AB.g} = 1.75\,kN \cdot m$

$V_{Ed.A.g} = 5.66\,kN \qquad V_{Ed.A.AB.g} = 3.9\,kN$

$R_{Ed.A.g} = 11.27\,kN$

Location B: $\qquad N_{Ed} = 5\,kN$

$M_{y.Ed.B.g} = 2.18\,kN \cdot m$



Stresses due to uplift load as given above: Location B:

$M_{y.Ed.B.u} = 0.3\,kN \cdot m$

Location A:

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

purlin: $t_A = 3.0\,mm$ supporting profile over support: sup_profile_A = "ZX-1.5" length of supp. profile: $L_{s.A} = 600\,mm$

check single profile:

$$\frac{M_{y.Ed.A.g} \cdot \gamma_{M0}}{M_{yRk.A}} + \frac{R_{Ed.A.g} \cdot \gamma_{M1}}{R_{w.Rk.A}} = 0.69 \quad < 1{,}25$$

check with supporting profile:

$$\frac{M_{y.Ed.A.g} \cdot \gamma_{M0}}{M_{yRk.A} + M_{yRk.sA}} + \frac{R_{Ed.A.g} \cdot \gamma_{M1}}{R_{w.Rk.A} + R_{w.Rk.sA}} = 0.49 \quad < 1{,}25$$

Continue Location A:

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

single profile at support:

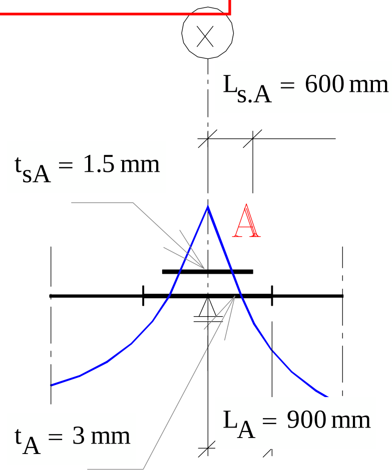
$$\left(\frac{M_{y.Ed.A.g}}{W_{yeff.A}}+\frac{N_{Ed}}{A_{eff.A}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}}=0.49$$ <1,0

check with supporting profile:

$$\left(\frac{M_{y.Ed.A.g}}{W_{yeff.A}+W_{yeff.sA}}+\frac{N_{Ed}}{A_{eff.A}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}}=0.34$$ <1,0

single profile att end of supporting profile:

$$M_{L.s.A}:=M_{y.Ed.A.g}-\frac{L_{s.A}\cdot\left(M_{y.Ed.A.g}-M_{y.Ed.AB.g}\right)}{L_A}\qquad M_{L.s.A}=3.2\,kN\cdot m$$



single profile at end of supporting profile:

$$\left(\frac{\left|M_{L.s.A}\right|}{W_{yeff.A}}+\frac{N_{Ed}}{A_{eff.A}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}}=0.27$$ < 1,0

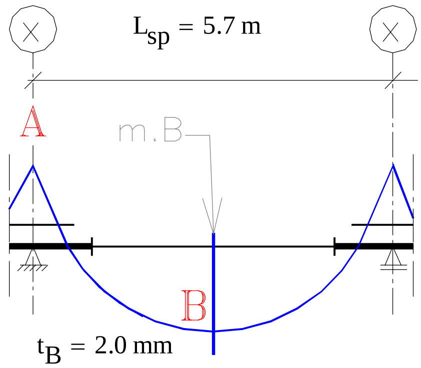
Bending and shear force:
Because $V_{Ed} < 0.5*V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

$$combined\_V\_and\_M:=\text{if}\left(\frac{V_{Ed.A.g}\cdot\gamma_{M0}}{V_{bh.Rk.A}\cdot 0.5}\le 1.0,\text{"Not neccessary"},\text{"Needs to be checked"}\right)$$

combined_V_and_M = "Not neccessary"

Location B:

purlin: $t_B = 2.0\,mm$



Combined bending moment due to gravity load and compression force:
EN 1993-1-3:10.1.4.1 eqv. 10.3b

lateral bending moment: gravity load EN 1993-1-3:101.4.1(5) $M_{fz.Ed.g}:=0\cdot kN\cdot m$

single profile in span gravity load:

$$\left(\frac{M_{y.Ed.B.g}}{W_{yeff.B}}+\frac{N_{Ed}}{A_{eff.B}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}}=0.31$$ < 1,0

single profile at connection between A and B, gravity load:

$$\left(\frac{\left|M_{y.Ed.AB.g}\right|}{W_{yeff.B}}+\frac{N_{Ed}}{A_{eff.B}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}}=0.25$$ < 1,0

Bending and shear force:
Because $V_{Ed} < 0.5*V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

$$combined\_V\_and\_M:=\text{if}\left(\frac{V_{Ed.A.AB.g}\cdot\gamma_{M0}}{V_{bh.Rk.B}\cdot 0.5}\le 1.0,\text{"Not neccessary"},\text{"Needs to be checked"}\right)$$

combined_V_and_M = "Not neccessary"

Check joint between purlins:

$$F_{b.Rk}:=\left(\begin{array}{l}18.9\ \text{if span\_B = "Z150x1.5"}\\25.2\ \text{if span\_B = "Z150x2"}\\37.8\ \text{if span\_B = "Z150x3"}\end{array}\right)\cdot kN\qquad F_{b.Rk}=25.2\,kN$$

Moment for joint near support A:

$$M_{y.Ed.AB.g}=1.75\,kN\cdot m$$

joint near support A gravity load:

$$\frac{\left|M_{y.Ed.AB.g}\right|\cdot\gamma_{M2}}{4\cdot F_{b.Rk}\cdot 0.075\cdot m}+\frac{N_{Ed}\cdot\gamma_{M2}}{4\cdot F_{b.Rk}}=0.35$$ < 1,0

joint near support A uplift load:

$$\frac{\left|M_{y.Ed.AB.u}\right|\cdot\gamma_{M2}}{4\cdot F_{b.Rk}\cdot 0.075\cdot m}+\frac{N_{Ed}\cdot\gamma_{M2}}{4\cdot F_{b.Rk}}=0.1$$ < 1,0

Continue Location B:

Buckling resistance of free flange in compression (uplift load) EN 1993-1-3: 10.1.4.2: The relevant position is "m.B".

lateral bending moment: uplift load EN 1993-1-3:101.4.1(5) $M_{fz.Ed.m.B} = 23.4\,N\cdot m$

non-dim. slenderness as defined above: $\lambda_1 := \pi\cdot\sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 76.95$

Coefficients from table 10.2b: $\eta_1 := 0.306$ $\eta_2 := 0.232$ $\eta_3 := 0.742$ $\eta_4 := -0.279$

Buckling lenght for free flangein compression EN 1993-1-3: 10.1.4.2 (3)+(6): provided that 0< Rrp <200:

$R_{rp.check} = \text{"OK"}$

$$L_{f.z.B} := \eta_1\cdot L_a\cdot\left(1 + \eta_2\cdot R_{rp.B}{}^{\eta_3}\right)^{\eta_4} \qquad L_{f.z.B} = 1.43\,m$$

Relative slenderness for flexural buckling of free flange: $\lambda_{r.f.z.B} := \frac{L_{f.z.B}}{i_{z.fl.B}}\cdot\frac{1}{\lambda_1}$ $\lambda_{r.f.z.B} = 0.934$

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

$\alpha_{LT} := 0.34$ $\lambda_{r.LT.0} := 0.4$ $\beta := 0.75$ $\phi_{LT.B} := 0.5\cdot\left[1 + \alpha_{LT}\cdot\left(\lambda_{r.f.z.B} - \lambda_{r.LT.0}\right) + \beta\cdot\lambda_{r.f.z.B}{}^2\right]$ $\phi_{LT.B} = 0.92$

$$\chi_{LT.B} := \min\left(\frac{1}{\phi_{LT.B} + \sqrt{\phi_{LT.B}{}^2 - \beta\cdot\lambda_{r.f.z.B}{}^2}}, 1, \frac{1}{\lambda_{r.f.z.B}{}^2}\right) \qquad \chi_{LT.B} = 0.74$$

Reduction factor for flexural buckling: According to EN 1993-1-1: 6.3.1: $\lambda_1 = 76.95$ $i_{y.B} := \sqrt{\frac{I_{y.rc.B}}{A_{g.rc.B}}}$

$\lambda_{r.y.FB.B}(L) := \frac{L}{i_{y.B}}\cdot\sqrt{\frac{A_{eff.B}}{A_{g.rc.B}}}\cdot\frac{1}{\lambda_1}$ $N_{cr.y.B}(L) := \frac{\pi^2\cdot E\cdot I_{y.rc.B}}{L^2}$ Imperfection factor α relating to bucking curve b $\alpha_{y.FB} := 0.34$

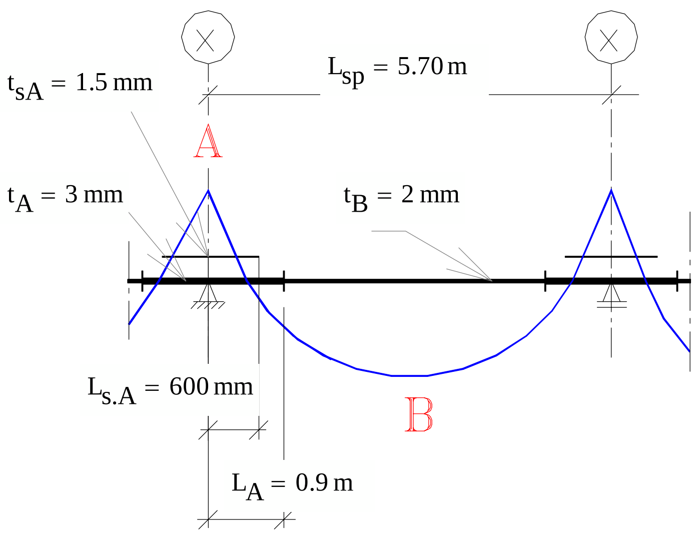
$$\phi_{y.FB.B}(L) := 0.5\cdot\left[1 + \alpha_{y.FB}\cdot\left(\lambda_{r.y.FB.B}(L) - 0.2\right) + \lambda_{r.y.FB.B}(L)^2\right]$$

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

$\chi_{y.FB.B}(L_{sp}) = 0.55$ reduction factor for span B

single profile in span uplift load:

eqv. 10.7 $$\left(\frac{1}{\chi_{LT.B}}\cdot\frac{M_{y.Ed.B.u}}{W_{yeff.B}} + \frac{1}{\min\left(\chi_{LT.B}, \chi_{y.FB.B}(L_{sp})\right)}\cdot\frac{N_{Ed}}{A_{eff.B}}\right)\cdot\frac{\gamma_{M1}}{f_{yb}} + \frac{\left|M_{fz.Ed.m.B}\right|\cdot\gamma_{M1}}{W_{z.fl.B}\cdot f_{yb}} = 0.15$$ < 1,0



Posudek vaznice - krajní pole

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

$\begin{pmatrix} f_{yb} \\ f_u \end{pmatrix} := \begin{pmatrix} 350 \\ 420 \end{pmatrix} \cdot \frac{N}{mm^2}$

values for γ_{Mi}: $\gamma_{M0} := 1.0$ $\gamma_{M1} := 1.0$ $\gamma_{M2} := 1.25$

Global values: all measures in (mm)

$t_{rp} := 0.63 \cdot mm$ thickness roofplate ($_{rp}$)

$t_{rp.cor} := 0.5842 \cdot mm$ core thickness

$h_{w.rp} := 45 \cdot mm$ height roofplate ($_{rp}$)

$b_{roof} := 32302 \cdot mm$ width of roof ($_{roof}$)

$L_{sp} := 4152 \cdot mm$ span of purlin

$cc_{purlin} := 750 \cdot mm$ distance between purlins

$s_s := 108 \cdot mm$ Support width

Stresses on roof purlin

maximum gravity load $q_{Ed.g} := 1.96 \cdot \frac{kN}{m}$

maximum uplift load $q_{Ed.u} := 1.0 \cdot \frac{kN}{m}$

Compression force in purlin: $N_{Ed} := 5 \cdot kN$

Stresses due to gravity load:

Location C:

$M_{y.Ed.C.g} := 5.56 \cdot kN \cdot m$ Moment at support

$V_{Ed.C.g} := 5.51 \cdot kN$ Shear force at support

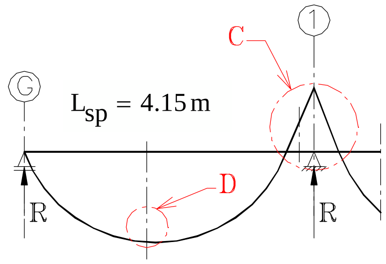
$R_{Ed.C.g} := 10.91 \cdot kN$ Reaktion force

$M_{y.Ed.CD.g} := 1.49 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)

$V_{Ed.CD.g} := 3.64 \cdot kN$ Shear force att connection between purlins (NOT support profile)

Location D:

$M_{y.Ed.D.g} := 1.90 \cdot kN \cdot m$ Moment in span



Stress due to upplift load:

Location D:

$M_{y.Ed.D.u} := 0.95 \cdot kN \cdot m$ Moment in span

$M_{y.Ed.CD.u} := 0.26 \cdot kN \cdot m$ Moment att connection between purlins (NOT support profile)

Purlin profiles:

profile over support C Z150x1.5, Z150x2 or Z150x3: support_C := "Z150x3"

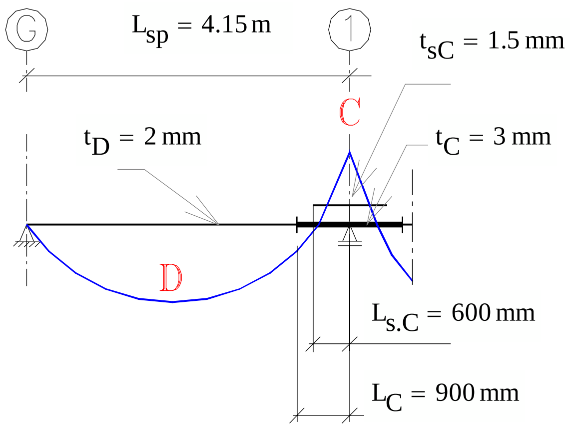
length of profile over C: $L_C := 900 \cdot mm$

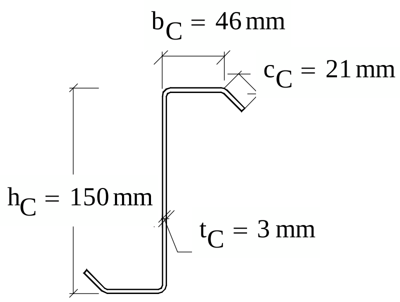
profile in span D Z150x1.5, Z150x2 or Z150x3: span_D := "Z150x2"

supporting profile over support location C ZX-1.5 , ZX-2 , ZX-3 or NO: sup_profile_C := "ZX-1.5"

Length of extra supporting profile over support C: $L_{s.C} := 600 \cdot mm$

Profile over support C:





$M_{yRk.C} = 12.79\,kN \cdot m$

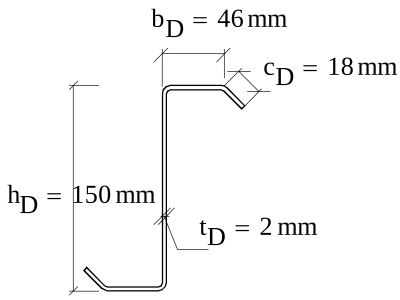
$V_{bh.Rk.C} = 89.11\,kN$

$R_{w.Rk.C} = 51.68\,kN$

$W_{yeff.C} = 3.66 \times 10^4\,mm^3$

$I_{y.C} = 2.69 \times 10^6\,mm^4$

Profile i span D:



$M_{yRk.D} = 8.07\,kN \cdot m$

$V_{bh.Rk.D} = 47.57\,kN$

$W_{yeff.D} = 2.31 \times 10^4\,mm^3$

$I_{y.D} = 1.8 \times 10^6\,mm^4$

$I_{T.D} = 726.25\,mm^4$

$I_{\omega.D} = 1.125 \times 10^9\,mm^6$

Shear stiffness of trapezoidal sheeting connected to purlin EN 1993-1-3: 10.1.1 (6)+(10):

Shear stiffness roof:
$X := 0.5$ only one half of the roof is accounted for

$$S_{roof} := \sqrt{\left(\frac{t_{rp}}{mm}\right)^3} \cdot \left[50 + 10 \cdot \sqrt[3]{\left(\frac{b_{roof} \cdot X}{mm}\right)}\right] \cdot \frac{cc_{purlin}}{h_{w.rp}} \qquad S_{roof} = 2523.36 \text{ in kN}$$

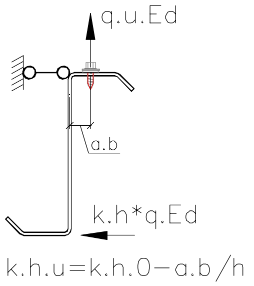
purlin:

$$Z_{purlin} := \left(E \cdot I_{\omega} \cdot \frac{\pi^2}{L_{sp}^2} + G \cdot I_T + E \cdot I_z \cdot \frac{\pi^2}{L_{sp}^2} \cdot 0.25 \cdot h_D^2\right) \cdot \frac{70}{h_D^2} \qquad Z_{purlin} = 1233\,kN$$

$$Restrained := if\left(S_{roof} \cdot kN \geq Z_{purlin}, \text{"Purlins are laterally restrained"}, \text{"No laterally restrain"}\right)$$

Restrained = "Purlins are laterally restrained"

The purlin at the connection is regarded as beeing laterally restrained in plane of the sheeting.
Lateral coefficient EN 1993-1-3: 10.1.4.1 (4): for uplift loading



$$k_{h.0.D} := \frac{h_{cD} \cdot t_D \cdot \left(b_{cD}^2 + 2 \cdot b_{cD} \cdot c_{cD} - 2 \cdot c_{cD}^2 \cdot \frac{b_{cD}}{h_{cD}}\right)}{4 \cdot I_{y.D}} \qquad a_{b.D} = 27.6\,mm \qquad k_{h.0.D} = 0.139$$

Equivalent lateral load acting on free flange, due to torsion and lateral bending (EN 1993-1-3: 10.1.4.1 (3)

$$k_{h.u.D} := k_{h.0.D} - \frac{a_{b.D}}{h_{cD}} \qquad q_{h.Ed.u.D} := q_{Ed.u} \cdot k_{h.u.D} \qquad q_{h.Ed.u.D} = -0.05 \frac{kN}{m}$$

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): C_D

tension in upper flange: $I_{eff.TP46.t} := 125400.93 \cdot mm^4 \cdot \frac{1}{m}$ $\qquad C_{D.C.t} := \frac{6 \cdot E \cdot I_{eff.TP46.t}}{cc_{purlin}}$ $\qquad C_{D.C.t} = 2.11 \times 10^5 \frac{N \cdot m}{m}$

compression in upper flange: $I_{eff.TP46.c} := 130983.88 \cdot mm^4 \cdot \frac{1}{m}$ $\qquad C_{D.C.c} := \frac{6 \cdot E \cdot I_{eff.TP46.c}}{cc_{purlin}}$ $\qquad C_{D.C.c} = 2.201 \times 10^5 \frac{N \cdot m}{m}$

Rotational stiffness of connection between sheeting and purlin EN 1993-1-3: 10.1.5.2 (5)

$$k_{t.g} := \left(\frac{t_{rp}}{0.75}\right)^{1.5} \cdot \frac{1}{mm^{1.5}} \qquad k_{t.g} = 0.77 \qquad \text{for } b<125mm$$

for uplift load pin in every trough $C_{100.cc150.u} := 2.6 \cdot \frac{kN \cdot m}{m}$ pin in alternate trough $C_{100.cc300.u} := 1.7 \cdot \frac{kN \cdot m}{m}$

$b := 46 \cdot mm$

$$C_{D.A.cc150.u} := C_{100.cc150.u} \cdot \left(\frac{\frac{b}{mm}}{100}\right)^2 \cdot k_{t.g} \qquad C_{D.A.cc300.u} := C_{100.cc300.u} \cdot \left(\frac{\frac{b}{mm}}{100}\right)^2 \cdot k_{t.g}$$

Rotational stiffness:

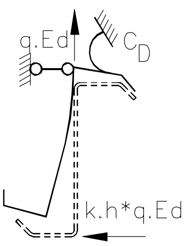
$$C_{D.cc150.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc150.u}} + \frac{1}{C_{D.C.t}}\right)} \qquad C_{D.cc150.u} = 0.423 \frac{kN \cdot m}{m} \text{ pin in every trough}$$

$$C_{D.cc300.u} := \frac{1}{\left(\frac{1}{C_{D.A.cc300.u}} + \frac{1}{C_{D.C.t}}\right)} \qquad C_{D.cc300.u} = 0.277 \frac{kN \cdot m}{m} \text{ pin in alternate trough}$$

For uplift load
EN 1993-1-3: 10.1.5.1 (4)

$$b_{mod.u.D} := \begin{cases} a_{b.D} & \text{if } q_{h.Ed.u.D} \geq 0 \\ \left(2 \cdot a_{b.D} + b\right) & \text{if } q_{h.Ed.u.D} < 0 \end{cases} \qquad b_{mod.u.D} = 101.2\,mm$$

Span D:



Flexibility from web in bending:

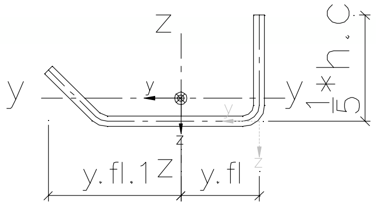
$$1 / K_{B.u.D} := \frac{4 \cdot \left(1 - \upsilon^2\right) \cdot h_D^2 \cdot \left(h_D + b_{mod.u.D}\right)}{E \cdot t_D^3} \qquad 1 / K_{B.u.D} = 12.246 \frac{mm^2}{N}$$

$$K_{u.cc150.D} := \frac{1}{\left(K_{B.u.D} + \frac{h_D^2}{C_{D.cc150.u}}\right)} \qquad K_{u.cc300.D} := \frac{1}{\left(K_{B.u.D} + \frac{h_D^2}{C_{D.cc300.u}}\right)}$$

Lateral spring stiffnes per unit length:

$K_{u.cc150.D} = 0.0153 \frac{N}{mm^2}$ pin in every trough $\qquad K_{u.cc300.D} = 0.0107 \frac{N}{mm^2}$ n in alternate trough

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



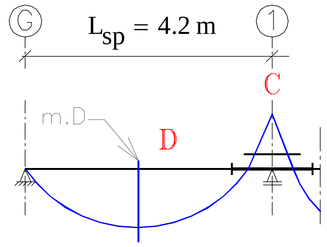
Span D: $t_D = 2.0\,mm$

$I_{z.fl.D} = 70968\,mm^4$

$W_{z.fl.D} = 1965.91\,mm^3$

$i_{z.fl.D} = 19.87\,mm$

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):

Span D $\quad R_{rp.D} := \dfrac{K_{u.cc300.D} \cdot L_a^4}{\pi^4 \cdot E \cdot I_{z.fl.D}} \qquad R_{rp.D} = 2.19$

correction factor κ_R and initial moment $M_{0.fz.Ed}$ acc. to table 10.1 EN 1993-1-3: 10.1.4.1

lateral bending moment from EN 1993-1-3: 10.1.4.1(5):

Location: $\quad M_{0.fz.Ed}$: $\quad \kappa_R$: $\quad M_{fz.Ed}$:

$m_D \quad M_{0.fz.Ed.m.D} := \dfrac{9}{128} \cdot \left|q_{h.Ed.u.D}\right| \cdot L_a^2 \quad \kappa_{R.m.D.2} := \dfrac{1 - 0.0141 \cdot R_{rp.D}}{1 + 0.416 \cdot R_{rp.D}}$

$M_{fz.Ed.m.D} := \kappa_{R.m.D.2} \cdot M_{0.fz.Ed.m.D}$

$M_{fz.Ed.m.D} = 29.4\,N \cdot m$

Stresses due to gravity load as given above:

Location C:

$M_{y.Ed.C.g} = 5.56\,kN \cdot m \quad M_{y.Ed.CD.g} = 1.49\,kN \cdot m$

$V_{Ed.C.g} = 5.51\,kN \quad V_{Ed.CD.g} = 3.64\,kN$

$R_{Ed.C.g} = 10.91\,kN$

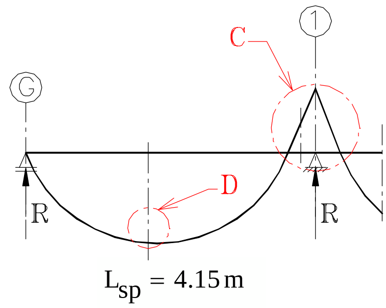
Location D:

$M_{y.Ed.D.g} = 1.9\,kN \cdot m$

Stresses due to uplift load as given above:

Location D:

$M_{y.Ed.D.u} = 0.95\,kN \cdot m$



Location C:

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

purlin: $t_C = 3.0\,mm$ supporting profile over support: sup_profile_C = "ZX-1.5" length of supp. profile: $L_{s.C} = 600\,mm$

check single profile:

$\dfrac{M_{y.Ed.C.g} \cdot \gamma_{M0}}{M_{yRk.C}} + \dfrac{R_{Ed.C.g} \cdot \gamma_{M1}}{R_{w.Rk.C}} = 0.65 \quad < 1,25$

check with supporting profile:

$\dfrac{M_{y.Ed.C.g} \cdot \gamma_{M0}}{M_{yRk.C} + M_{yRk.sC}} + \dfrac{R_{Ed.C.g} \cdot \gamma_{M1}}{R_{w.Rk.C} + R_{w.Rk.sC}} = 0.46 \quad < 1,25$

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

single profile at support:

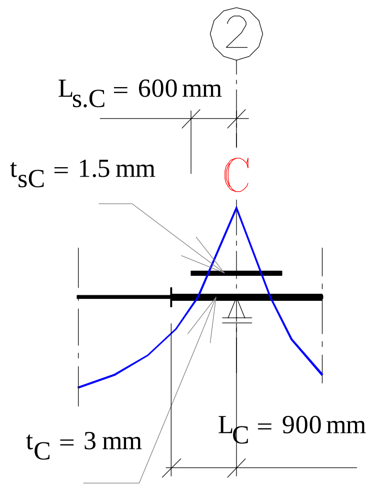
$\left(\dfrac{M_{y.Ed.C.g}}{W_{yeff.C}} + \dfrac{N_{Ed}}{A_{eff.C}}\right) \cdot \dfrac{\gamma_{M1}}{f_{yb}} = 0.45 \quad <1,0$

check with supporting profile:

$\left(\dfrac{M_{y.Ed.C.g}}{W_{yeff.C} + W_{yeff.sC}} + \dfrac{N_{Ed}}{A_{eff.C}}\right) \cdot \dfrac{\gamma_{M1}}{f_{yb}} = 0.32 \quad <1,0$

single profile att end of supporting profile:

$M_{L.s.C} := M_{y.Ed.C.g} - \dfrac{L_{s.C} \cdot \left(M_{y.Ed.C.g} - M_{y.Ed.CD.g}\right)}{L_C} \qquad M_{L.s.C} = 2.85\,kN \cdot m$



single profile at end of supporting profile:

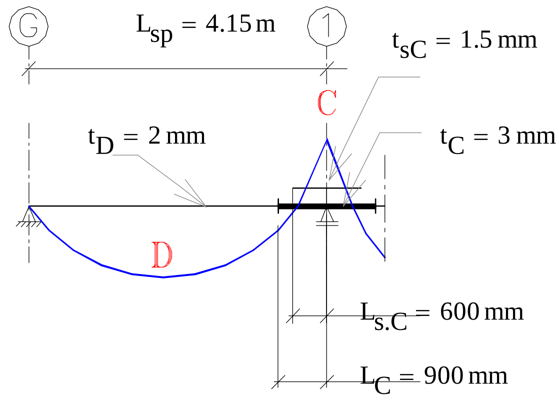
$\left(\dfrac{\left|M_{L.s.C}\right|}{W_{yeff.C}} + \dfrac{N_{Ed}}{A_{eff.C}}\right) \cdot \dfrac{\gamma_{M1}}{f_{yb}} = 0.24 \quad < 1,0$

Bending and shear force:
Because $V_{Ed} < 0.5 * V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

combined_V_and_M := if$\left(\dfrac{V_{Ed.C.g} \cdot \gamma_{M0}}{V_{bh.Rk.C} \cdot 0.5} \le 1.0, \text{"Not neccessary"}, \text{"Needs to be checked"}\right)$

combined_V_and_M = "Not neccessary"

Location D: purlin: $t_D = 2.0\,mm$



Combined bending moment due to gravity load and compression force:
EN 1993-1-3:10.1.4.1 eqv. 10.3b

lateral bending moment: gravity load $M_{fz.Ed.g} := 0 \cdot kN \cdot m$
EN 1993-1-3:101.4.1(5)

single profile in span gravity load:

$$\left(\frac{M_{y.Ed.D.g}}{W_{yeff.D}} + \frac{N_{Ed}}{A_{eff.D}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} = 0.27 \quad <1,0$$

single profile at connection between A and B, gravity load:

$$\left(\frac{\left|M_{y.Ed.CD.g}\right|}{W_{yeff.D}} + \frac{N_{Ed}}{A_{eff.D}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} = 0.22 \quad < 1,0$$

Bending and shear force:
Because $V_{Ed} < 0.5*V_{b.Rd}$ combined action of bending moment and shear force according to EN 1993 -1-3: 6.1.10 does not need to be checked.

combined_V_and_M = "Not neccessary"

Check joint between purlins:

$$F_{b.Rk} := \left(\begin{array}{l} 18.9 \text{ if span\_D} = \text{"Z150x1.5"} \\ 25.2 \text{ if span\_D} = \text{"Z150x2"} \\ 37.8 \text{ if span\_D} = \text{"Z150x3"} \end{array}\right) \cdot kN \qquad F_{b.Rk} = 25.2\,kN$$

joint near support C:

$$\frac{\max\left(\left|M_{y.Ed.CD.g}\right|, \left|M_{y.Ed.CD.u}\right|\right) \cdot \gamma_{M2}}{4 \cdot F_{b.Rk} \cdot 0.075 \cdot m} + \frac{N_{Ed} \cdot \gamma_{M2}}{4 \cdot F_{b.Rk}} = 0.31 \quad < 1,0$$

Buckling resistance of free flange in compression (uplift load) EN 1993-1-3: 10.1.4.2: The relevant position is "m.D".

lateral bending moment: uplift load $M_{fz.Ed.m.D} = 29.4\,N \cdot m$
EN 1993-1-3:10.1.4.1(5)

non-dim. slenderness as defined above: $\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 76.95$

Coefficients from table 10.2b: $\eta_1 := 0.515$ $\eta_2 := 1.26$ $\eta_3 := 0.868$ $\eta_4 := -0.242$

Buckling lenght for free flangein compression
EN 1993-1-3: 10.1.4.2 (3)+(6): provided that 0< Rrp <200: $R_{rp.check}$ = "OK"

$$L_{f.z.D} := \eta_1 \cdot L_a \cdot \left(1 + \eta_2 \cdot R_{rp.D}^{\eta_3}\right)^{\eta_4} \qquad L_{f.z.D} = 1.58\,m$$

Relative slenderness for flexural buckling of free flange:

$$\lambda_{r.f.z.D} := \frac{L_{f.z.D}}{i_{z.fl.D}} \cdot \frac{1}{\lambda_1} \qquad \lambda_{r.f.z.D} = 1.034$$

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

$\alpha_{LT} := 0.34$ $\lambda_{r.LT.0} := 0.4$ $\beta := 0.75$

$$\phi_{LT.D} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot \left(\lambda_{r.f.z.D} - \lambda_{r.LT.0}\right) + \beta \cdot \lambda_{r.f.z.D}^2\right] \qquad \phi_{LT.D} = 1.01$$

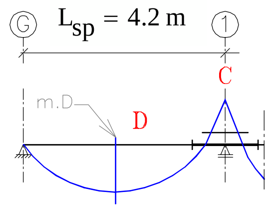
$$\chi_{LT.D} := \min\left(\frac{1}{\phi_{LT.D} + \sqrt{\phi_{LT.D}^2 - \beta \cdot \lambda_{r.f.z.D}^2}}, 1, \frac{1}{\lambda_{r.f.z.D}^2}\right) \qquad \chi_{LT.D} = 0.68$$

$$i_{y.D} := \sqrt{\frac{I_{y.rc.D}}{A_{g.rc.D}}} \qquad \lambda_1 = 76.95$$

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

Imperfection factor α relating to bucking curve b $\alpha_{y.FB} := 0.34$

$$\lambda_{r.y.FB.D}(L) := \frac{L}{i_{y.D}} \cdot \sqrt{\frac{A_{eff.D}}{A_{g.rc.D}}} \cdot \frac{1}{\lambda_1} \qquad N_{cr.y.D}(L) := \frac{\pi^2 \cdot E \cdot I_{y.rc.D}}{L^2}$$



$$\phi_{y.FB.D}(L) := 0.5 \cdot \left[1 + \alpha_{y.FB} \cdot \left(\lambda_{r.y.FB.D}(L) - 0.2\right) + \lambda_{r.y.FB.D}(L)^2\right]$$

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

$\chi_{y.FB.D}(L_{sp}) = 0.74$ reduction factor for span D

single profile in span uplift load:

eqv. 10.7:

$$\left(\frac{1}{\chi_{LT.D}} \cdot \frac{M_{y.Ed.D.u}}{W_{yeff.D}} + \frac{1}{\min\left(\chi_{LT.D}, \chi_{y.FB.D}(L_{sp})\right)} \cdot \frac{N_{Ed}}{A_{eff.D}}\right) \cdot \frac{\gamma_{M1}}{f_{yb}} + \frac{\left|M_{fz.Ed.m.D}\right| \cdot \gamma_{M1}}{W_{z.fl.D} \cdot f_{yb}} = 0.27 \quad < 1,0$$

Posudek stabilizace vaznice

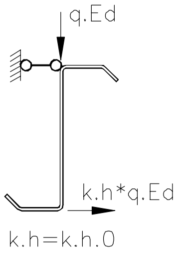
Bracing of Z-roof purlin

$roofslope := "6"$ degrees   Loading width frame: $L_{lw} := 5.8 \cdot m$ rigde flashing is mounted YES or NO: $rigdge\_flashing := "NO"$

$type\_of\_hall := "S"$ S for S-Hall, P for P-hall $b_{roof.1} := \begin{cases} b_{roof} & \text{if } type\_of\_hall = "S" \\ b_{roof} \cdot 2 & \text{if } type\_of\_hall = "P" \end{cases}$ $b_{roof.1} = 32.30\,m$

$q_{Ed.g} = 1.96 \frac{kN}{m}$ gravity load $q_{Ed.g.slope} := \cos(\alpha_{roof}) \cdot q_{Ed.g}$ $q_{Ed.g.slope} = 1.95 \frac{kN}{m}$

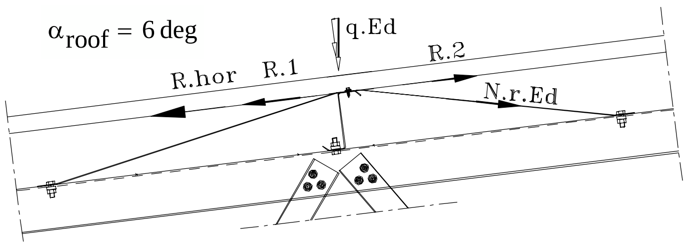
Lateral coefficient
EN 1993-1-3: 10.1.4.1 (4):



from above: $k_{h.0.D} = 0.139$

for gravity load: $k_{h.g.D} := k_{h.0.D}$

If the forces due to lateral and torsional bending is acting positive for sloped roofs, it will be reduced due to imperfection.



$\alpha_{roof} = 6\,deg$

from roof: $R_1 := \sin(\alpha_{roof}) \cdot q_{Ed.g} \cdot L_{lw}$

$R_1 = 1.13\,kN$

from profil: $R_2 := q_{Ed.g.slope} \cdot k_{h.g.D} \cdot L_{lw}$

$R_2 = 1.57\,kN$

adjusted value from profile: $R_{2.a} := \begin{cases} R_2 & \text{if } \alpha_{roof} \le \operatorname{atan}\left(\frac{1}{10}\right) \\ 0.5 \cdot R_2 & \text{if } \alpha_{roof} > \operatorname{atan}\left(\frac{1}{10}\right) \end{cases}$

$R_{2.a} = 1.57\,kN$

$R_{hor} := R_1 - R_{2.a}$ $R_{hor} = -0.44\,kN$

Force acting in the roof plane for one half of the building: $P_{roof} := \frac{b_{roof.1} \cdot R_{hor}}{2 \cdot cc_{purlin}}$ $P_{roof} = -9.4\,kN$

Shear resistance of screws
[redacted]

$F_{4.8\_sRd} := \frac{3.33}{\gamma_{M2}} \cdot kN$ $F_{4.8\_sRd} = 2.7\,kN$ $F_{6.3\_sRd} := \frac{6.24}{\gamma_{M2}} \cdot kN$ $F_{6.3\_sRd} = 5.0\,kN$

$\gamma_{M2} = 1.25$

Overlap screw for use in ridge connection (Ø 4.8)

$d_{oscr} := 4.8mm$ $t_{ridgeplate} := 0.5 \cdot mm$ ultimate strength rigdge plate: $f_{u.rp} := 330 \cdot \frac{N}{mm^2}$

$\alpha_{oscr} := 3.2 \cdot \sqrt{\left(\frac{t_{ridgeplate}}{d_{oscr}}\right)}$ $\alpha_{oscr} = 1.033$ $F_{oscr\_bRd1} := \frac{(\alpha_{oscr} \cdot f_{u.rp} \cdot d_{oscr} \cdot t_{ridgeplate})}{\gamma_{M2}}$ $F_{oscr\_bRd1} = 0.654\,kN$

$F_{oscr\_bRd} := \min(F_{oscr\_bRd1}, F_{4.8\_sRd})$ $F_{oscr\_bRd} = 0.654\,kN$ Bearing resistance plate to plate or shear capacity of screw

Plate screw for use in fastening of roofbrace to purlin (Ø 6.3)

$d_{dscr} := 6.3mm$ $dw_{dscr} := 14mm$ Tickness of support $t_1 := t_C$ $t_1 = 3.0\,mm$ thickness of brace: $t := 1.5 \cdot mm$

$\alpha := 3.2 \cdot \sqrt{\left(\frac{t}{d_{dscr}}\right)}$ $\alpha = 1.561$ $\alpha_1 := \text{if}(\alpha > 2.1, 2.1, \alpha)$ $\alpha_1 = 1.561$ $\alpha_2 := \frac{(2.1 - \alpha_1)}{2.5 - 1} \cdot \left(\frac{t_1}{t} - 1\right) + \alpha_1$ $\alpha_2 = 1.92$

$\alpha_q := \text{if}(t < 1mm, \alpha_1, 2.1)$ $\alpha_q = 2.1$ $\alpha_{qq} := \left[\frac{(\alpha_q - \alpha_1)}{2.5 - 1} \cdot \left(\frac{t_1}{t} - 1\right) + \alpha_1\right]$ $\alpha_{qq} = 1.92$

$\alpha_{dscrEN} := \left(\begin{cases} \alpha_1 & \text{if } t = t_1 \\ \alpha_1 & \text{if } t_1 \ge 2.5 \cdot t \wedge t < 1 \cdot mm \\ 2.1 & \text{if } t_1 \ge 2.5 \cdot t \wedge t \ge 1mm \\ \alpha_{qq} & \text{if } t < t_1 \wedge t_1 < 2.5 \cdot t \end{cases} \right)$ $\alpha_{dscrEN} = 1.92$

$F_{dscr\_bRdEN} := \frac{(\alpha_{dscrEN} \cdot f_u \cdot d_{dscr} \cdot t)}{\gamma_{M2}}$ $f_u = 420 \frac{N}{mm^2}$

$F_{dscr\_bRdEN} = 6.098\,kN$ $\gamma_{M2} = 1.25$

$F_{dscr\_bRd} := \min(F_{6.3\_sRd}, F_{dscr\_bRdEN})$ $F_{dscr\_bRd} = 5.0\,kN$ Bearing resistance plate to support or max. shear in screw

<u>Platescrew for connection roofplate to purlin (Ø6.3)</u>

$d_{dscr} := 6.3mm$ $dw_{dscr} := 19mm$ Tickness of support $t_1 := t_C$ $t_1 = 3.0\ mm$ thickness of roofplate $t := t_{rp.cor}$

$\alpha := 3.2 \cdot \sqrt{\left(\frac{t}{d_{dscr}}\right)}$ $\alpha = 0.974$ $\alpha_1 := if(\alpha > 2.1, 2.1, \alpha)$ $\alpha_1 = 0.974$ $\alpha_2 := \frac{(2.1 - \alpha_1)}{2.5 - 1} \cdot \left(\frac{t_1}{t} - 1\right) + \alpha_1$ $\alpha_2 = 4.077$

$\alpha_q := if(t < 1mm, \alpha_1, 2.1)$ $\alpha_q = 0.974$ $\alpha_{qq} := \left[\frac{(\alpha_q - \alpha_1)}{2.5 - 1} \cdot \left(\frac{t_1}{t} - 1\right) + \alpha_1\right]$ $\alpha_{qq} = 0.974$

$$\alpha_{dscrEN} := \begin{cases} \alpha_1 & \text{if } t = t_1 \\ \alpha_1 & \text{if } t_1 \geq 2.5 \cdot t \wedge t < 1 \cdot mm \\ 2.1 & \text{if } t_1 \geq 2.5 \cdot t \wedge t \geq 1mm \\ \alpha_{qq} & \text{if } t < t_1 \wedge t_1 < 2.5 \cdot t \end{cases}$$

$\alpha_{dscrEN} = 0.974$

$F_{platesc.bRd} := \frac{(\alpha_{dscrEN} \cdot f_{u.tp} \cdot d_{dscr} \cdot t)}{\gamma_{M2}} \cdot 5$

$F_{platesc.bRd} = 5.59\,kN$

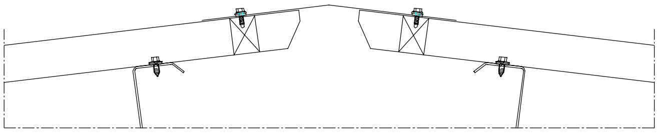
$F_{dscr\_bRd} := min(F_{6.3\_sRd}, F_{dscr\_bRdEN}, F_{platesc.bRd})$ $F_{dscr\_bRd} = 4.99\,kN$

$\alpha_{roof} = 6\,deg$

$$Lw_{ridge} := \begin{cases} L_{lw} & \text{if rigdge\_flashing = "YES"} \wedge P_{roof} > 0 \\ 0 \cdot m & \text{otherwise} \end{cases}$$

rigdge_flashing = "NO" $Lw_{ridge} = 0\,m$

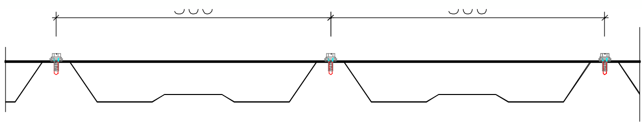
Part of force in plane of roof which is taken by ridge flashing:
(If force is compression no rigdge flashing is in account)



resistance of overlap screw: $F_{oscr\_bRd} = 0.654\,kN$

$P_{Rd.ridge} := \frac{Lw_{ridge} \cdot F_{oscr\_bRd}}{300 \cdot mm}$ $P_{Rd.ridge} = 0\,kN$

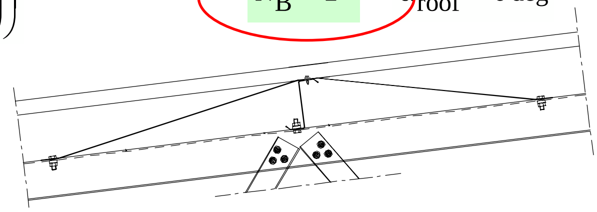
$P_{roof} = -9.4\,kN$



Number of braces needed for each half of the roof:

Design resistance for one brace 38x1.5 mm fastened with platescrew (Ø 6.3): $F_{dscr\_bRd} = 4.99\,kN$

$N_B := if\left(ceil\left(\frac{|P_{roof}| - P_{Rd.ridge}}{F_{dscr\_bRd}}\right) \leq 0, 1, ceil\left(\frac{|P_{roof}| - P_{Rd.ridge}}{F_{dscr\_bRd}}\right)\right)$ Number of braces: $N_B = 2$ $\alpha_{roof} = 6\,deg$



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

<u>Plate screw for use in fastening of roofplate towards C-profile (Ø 4.8)</u>

$d_{dscr} := 4.8mm$ $dw_{dscr} := 14mm$ Tickness of support $t_1 := 2.0 \cdot mm$ thickness of roofplate: $t := t_{rp.cor}$ $t = 0.58\,mm$

$\alpha := 3.2 \cdot \sqrt{\left(\frac{t}{d_{dscr}}\right)}$ $\alpha = 1.116$ $\alpha_1 := if(\alpha > 2.1, 2.1, \alpha)$ $\alpha_1 = 1.116$ $\alpha_2 := \frac{(2.1 - \alpha_1)}{2.5 - 1} \cdot \left(\frac{t_1}{t} - 1\right) + \alpha_1$ $\alpha_2 = 2.706$

$\alpha_q := if(t < 1mm, \alpha_1, 2.1)$ $\alpha_q = 1.116$ $qq := \left[\frac{(\alpha_q - \alpha_1)}{2.5 - 1} \cdot \left(\frac{t_1}{t} - 1\right) + \alpha_1\right]$ $\alpha_{qq} = 1.116$

$$\alpha_{dscrEN} := \begin{cases} \alpha_1 & \text{if } t = t_1 \\ \alpha_1 & \text{if } t_1 \geq 2.5 \cdot t \wedge t < 1 \cdot mm \\ 2.1 & \text{if } t_1 \geq 2.5 \cdot t \wedge t \geq 1mm \\ \alpha_{qq} & \text{if } t < t_1 \wedge t_1 < 2.5 \cdot t \end{cases}$$

$\alpha_{dscrEN} = 1.116$

$F_{dscr\_bRdEN} := \frac{(\alpha_{dscrEN} \cdot f_{u.tp} \cdot d_{dscr} \cdot t)}{\gamma_{M2}}$ $F_{dscr\_bRdEN} = 0.977\,kN$

$F_{dscr\_bRd} := min(F_{4.8\_sRd}, F_{dscr\_bRdEN})$ $F_{dscr\_bRd} = 1.0\,kN$

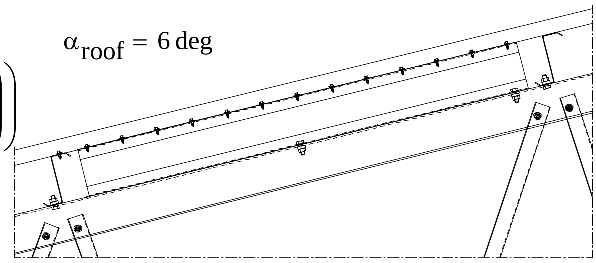
Design resistance for one C-profile $t \geq 2mm$ $P_{Rd.C\_profile} := 13 \cdot F_{dscr\_bRd}$ 13 4.8mm Plate-Screw connected to the frame with 2 M12x35 $P_{Rd.C\_profile} = 12.7\,kN$

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

$\alpha_{roof} = 6\,deg$

$N_B := if\left(ceil\left(\frac{|P_{roof}| - P_{Rd.ridge}}{P_{Rd.C\_profile}}\right) \leq 0, 1, ceil\left(\frac{|P_{roof}| - P_{Rd.ridge}}{P_{Rd.C\_profile}}\right)\right)$

Number of braces: $N_B = 1$

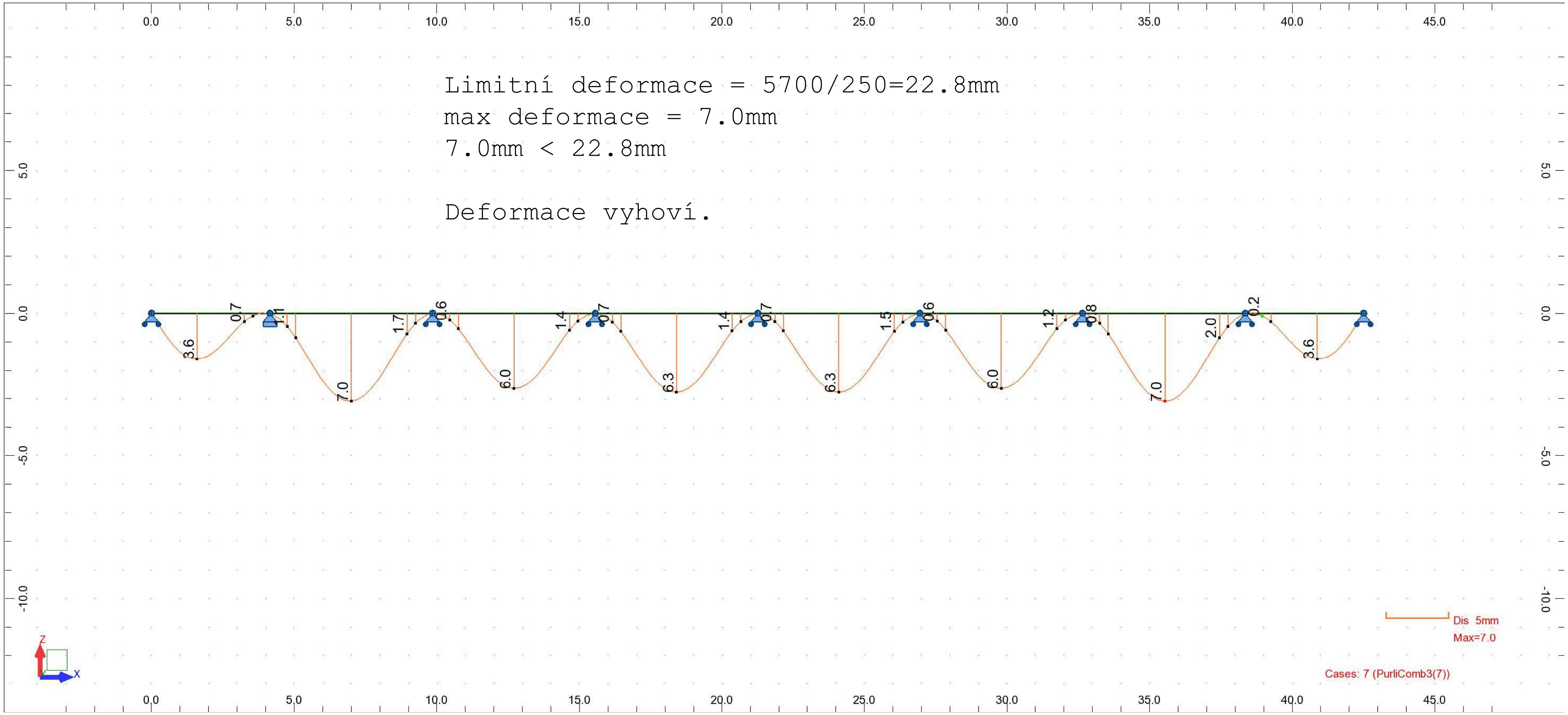


View - Exact deformation(s); Cases: 7 (PurliComb3(7))

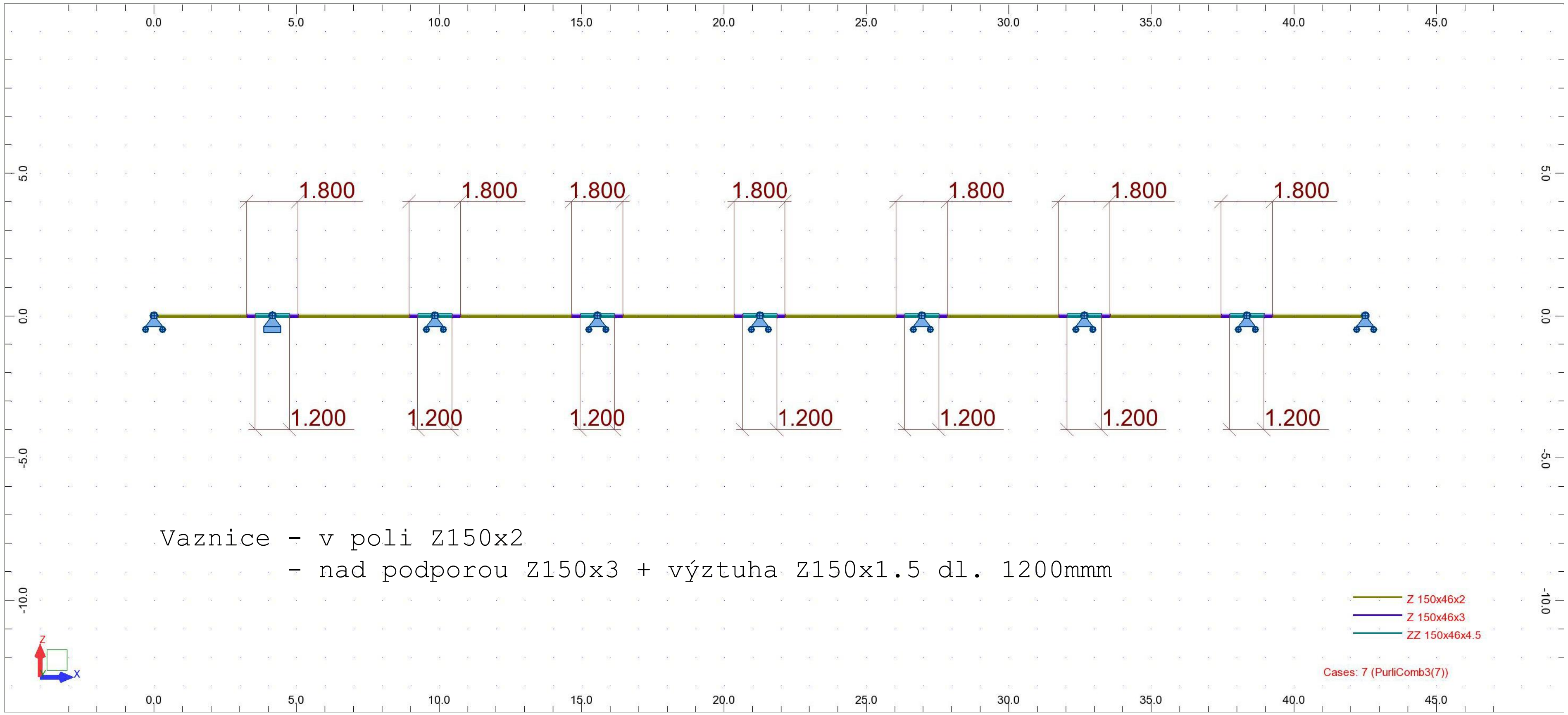
Posudek deformace vaznice

Limitní deformace = 5700/250=22.8mm
max deformace = 7.0mm
7.0mm < 22.8mm

Deformace vyhoví.



View - Cases: 7 (PurliComb3(7)) Profily vaznice



Rám FR1

Soubory modelu konstrukce:

CZ1138_FR1_fire.rtd
CZ1138_FR1.rtd

Soubory zatížení:

CZ1138 Loading.xlsm

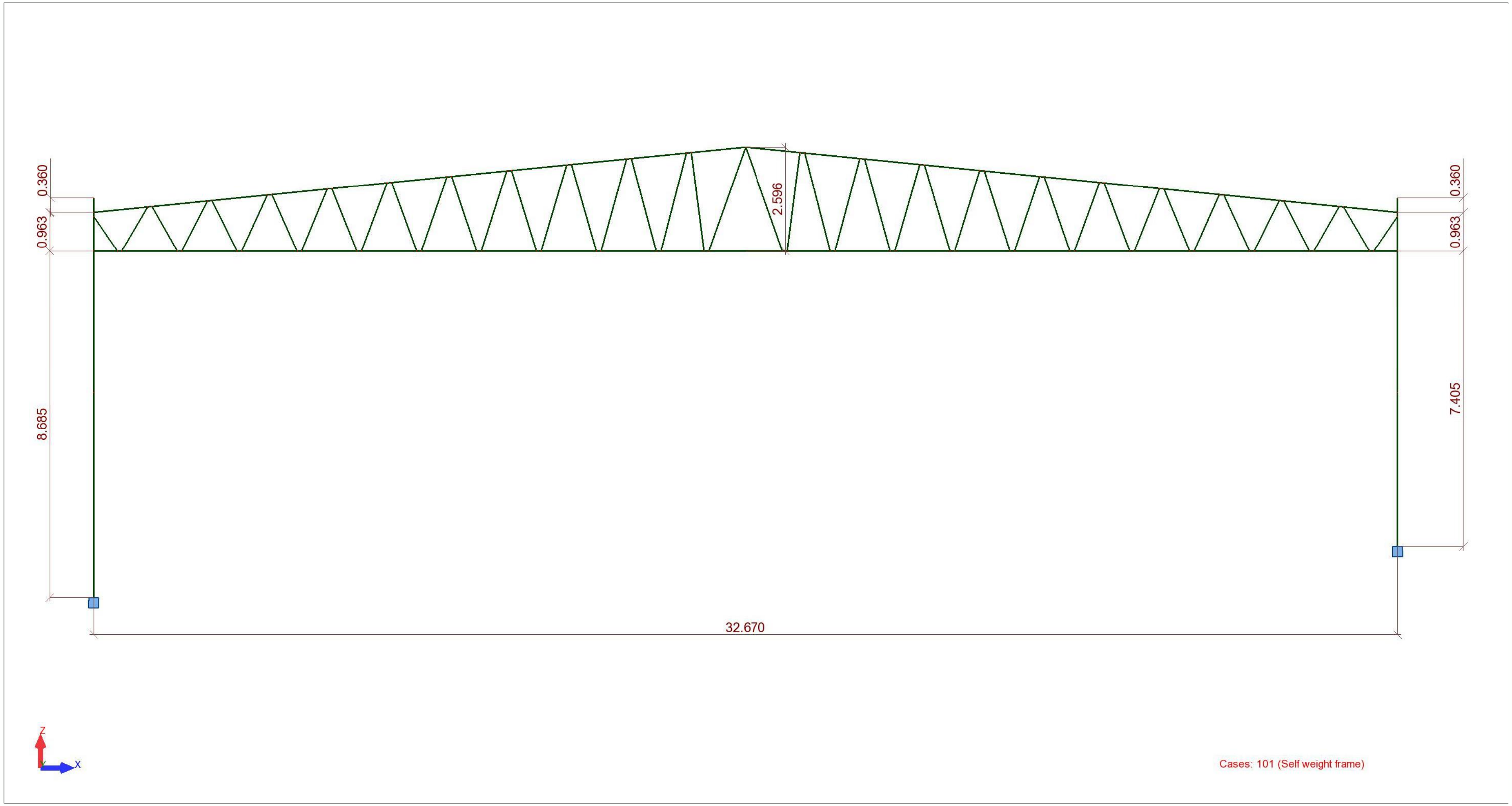
Poznámky:

ZATĚŽOVACÍ ŠÍŘKA - VAZNÍK = 5.8M, SLOUPY=5.7M

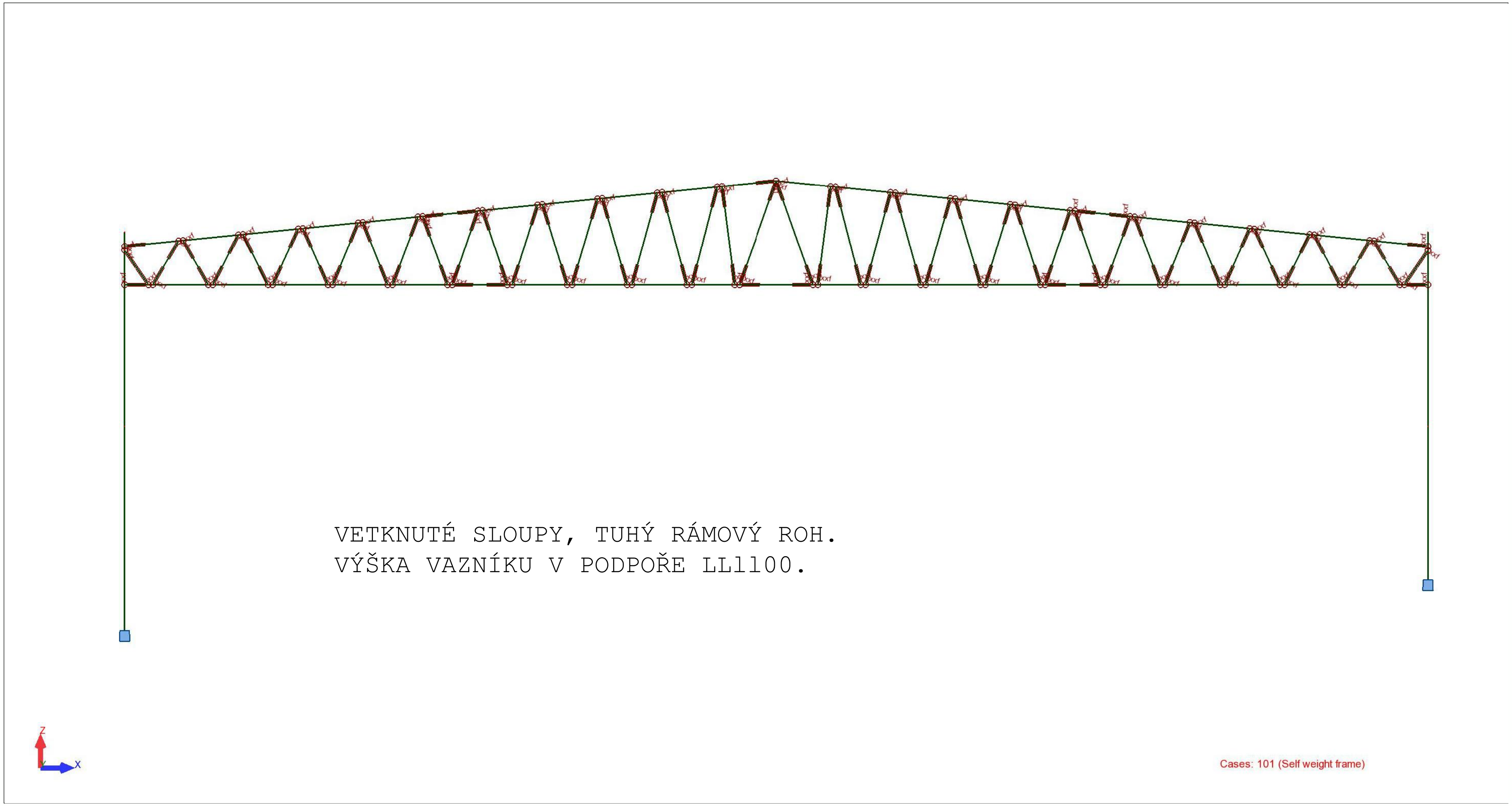
SLOUPY RÁMU VETKNUTY, NEUVOLNĚNÝ RÁMOVÝ ROH.

POŽÁRNÍ ODOLNOST VAZNÍKŮ A SLOUPŮ R15.

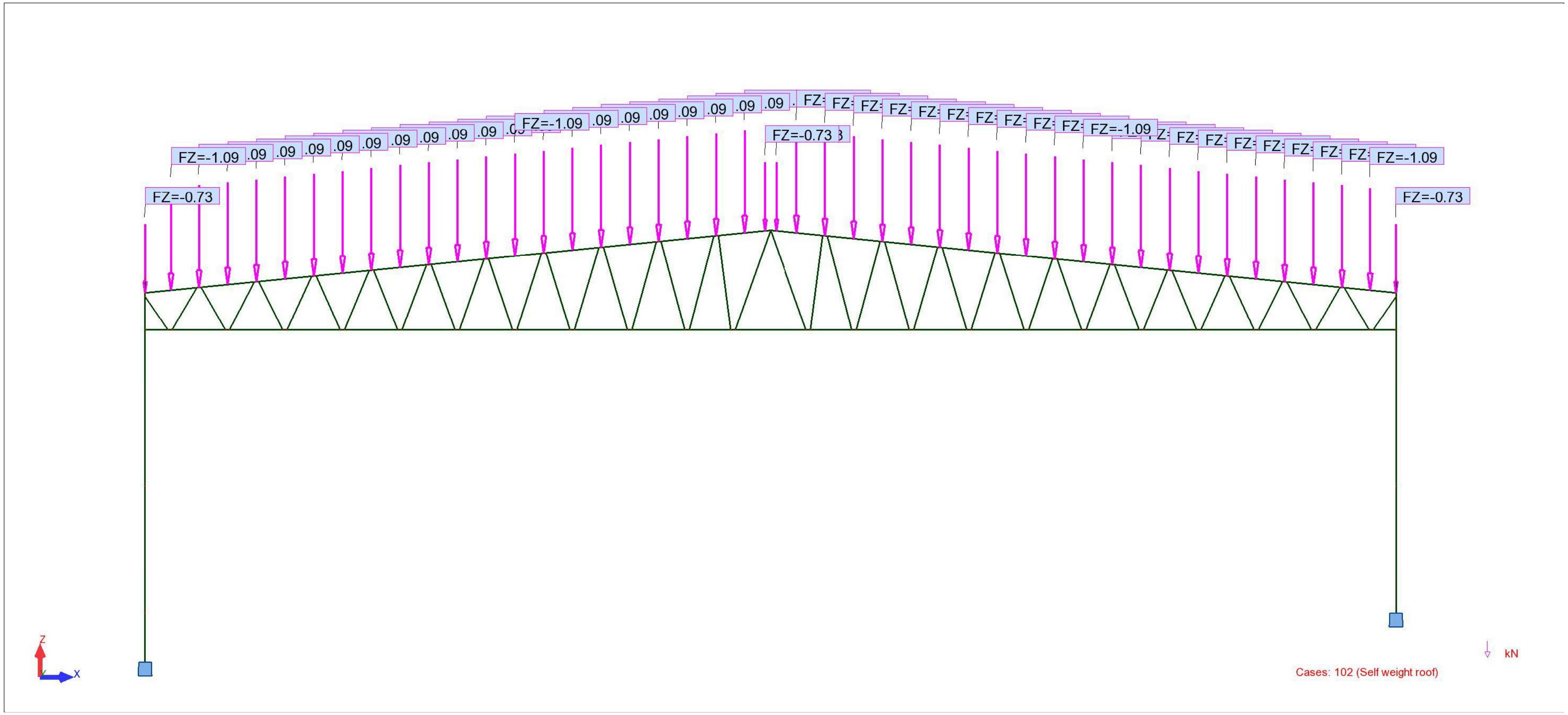
View - Cases: 101 (Self weight frame) GEOMETRIE KONSTRUKCE



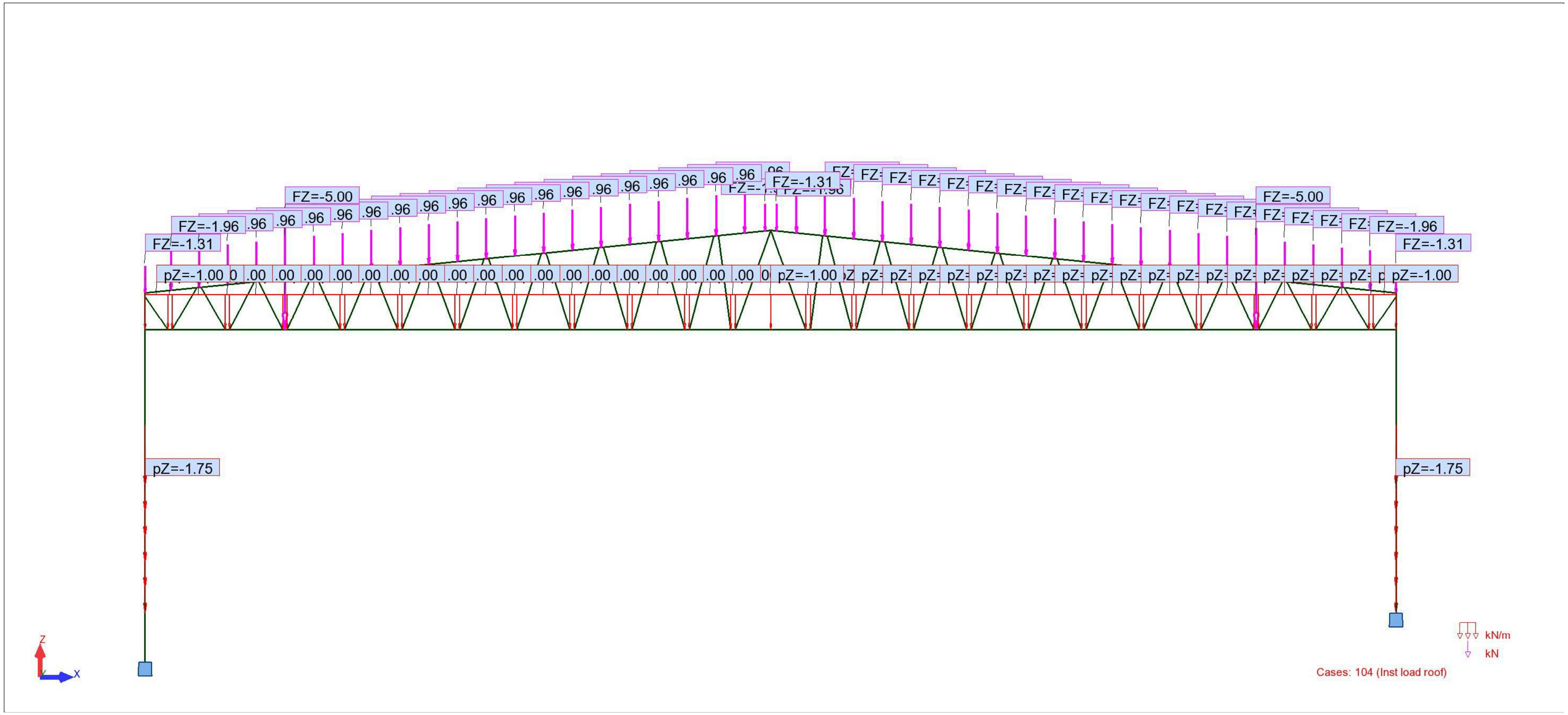
View - Cases: 101 (Self weight frame) MODEL KONSTRUKCE



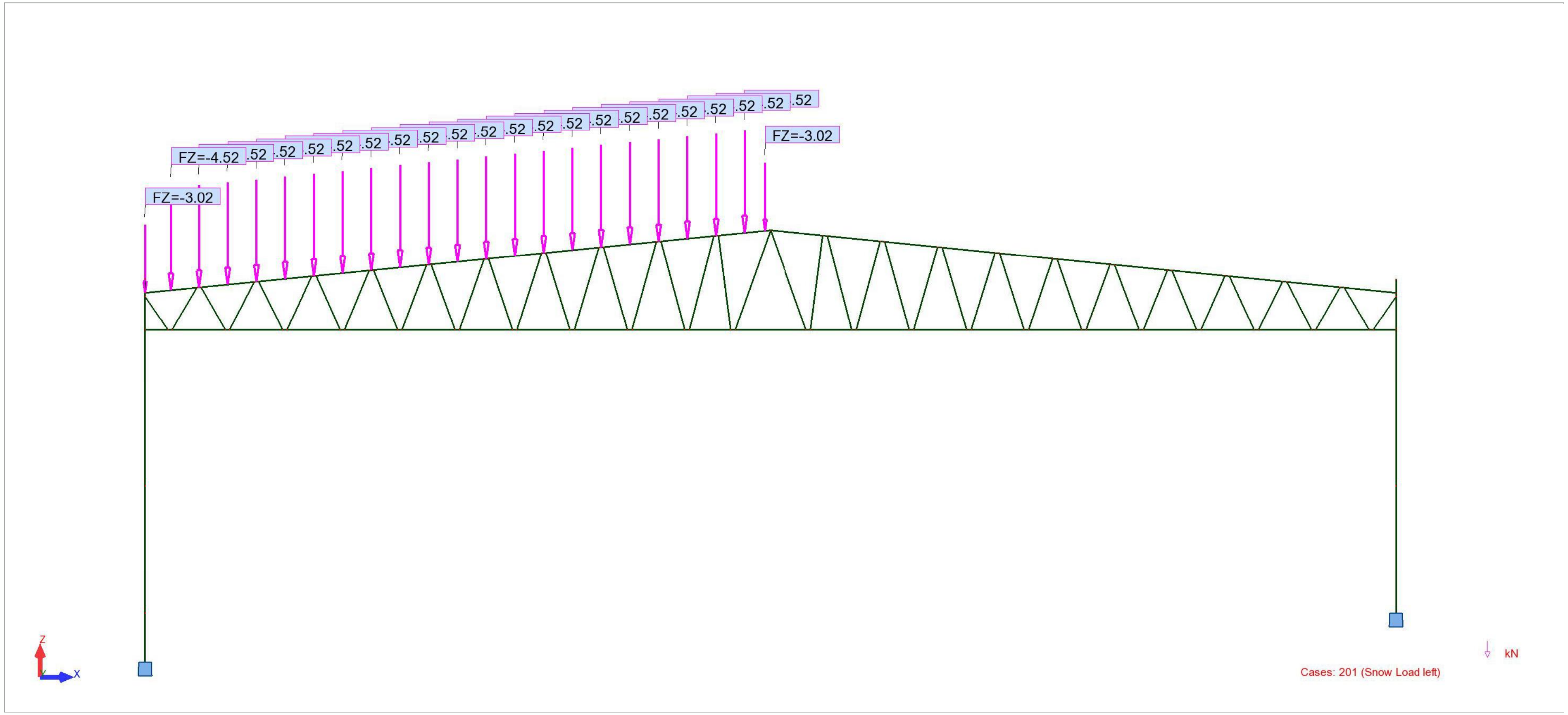
View - Cases: 102 (Self weight roof)



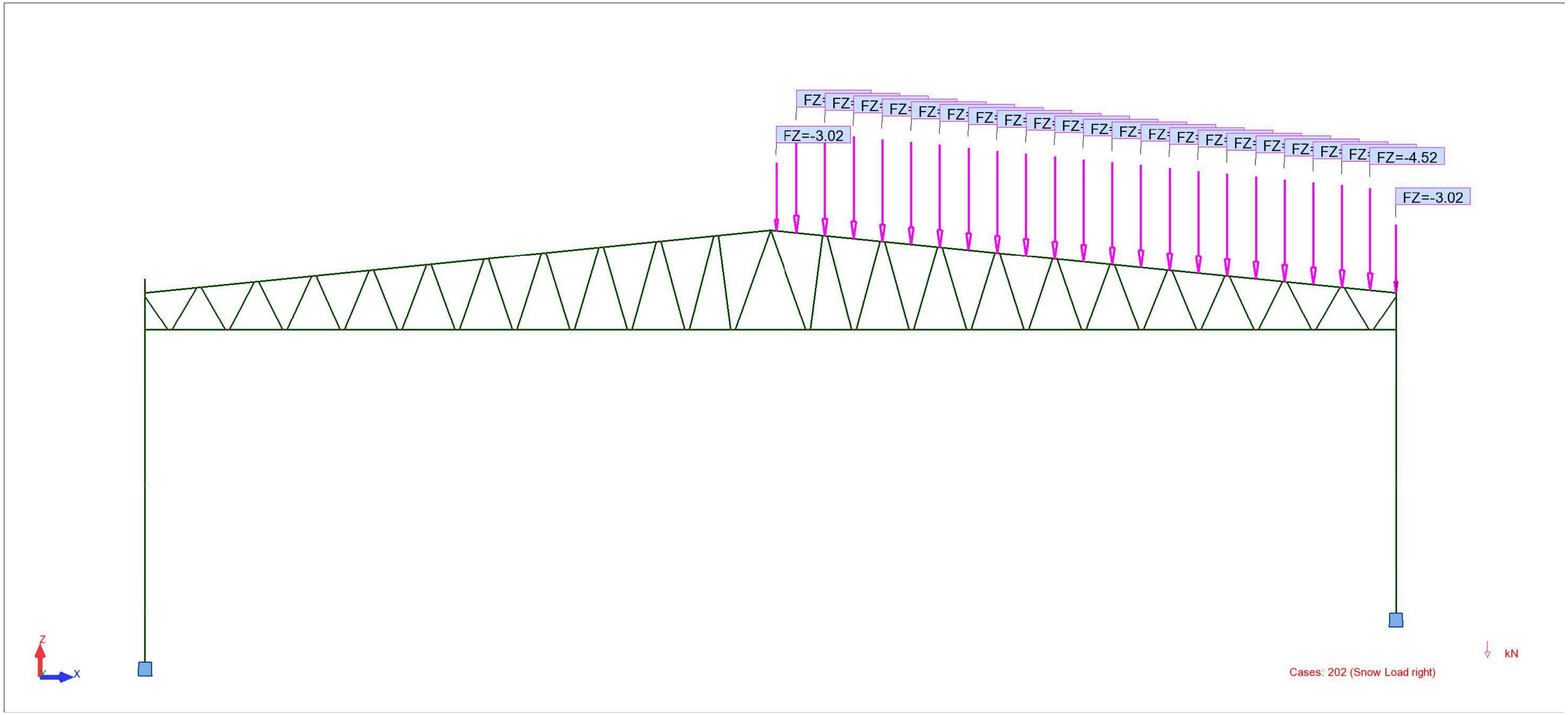
View - Cases: 104 (Inst load roof)



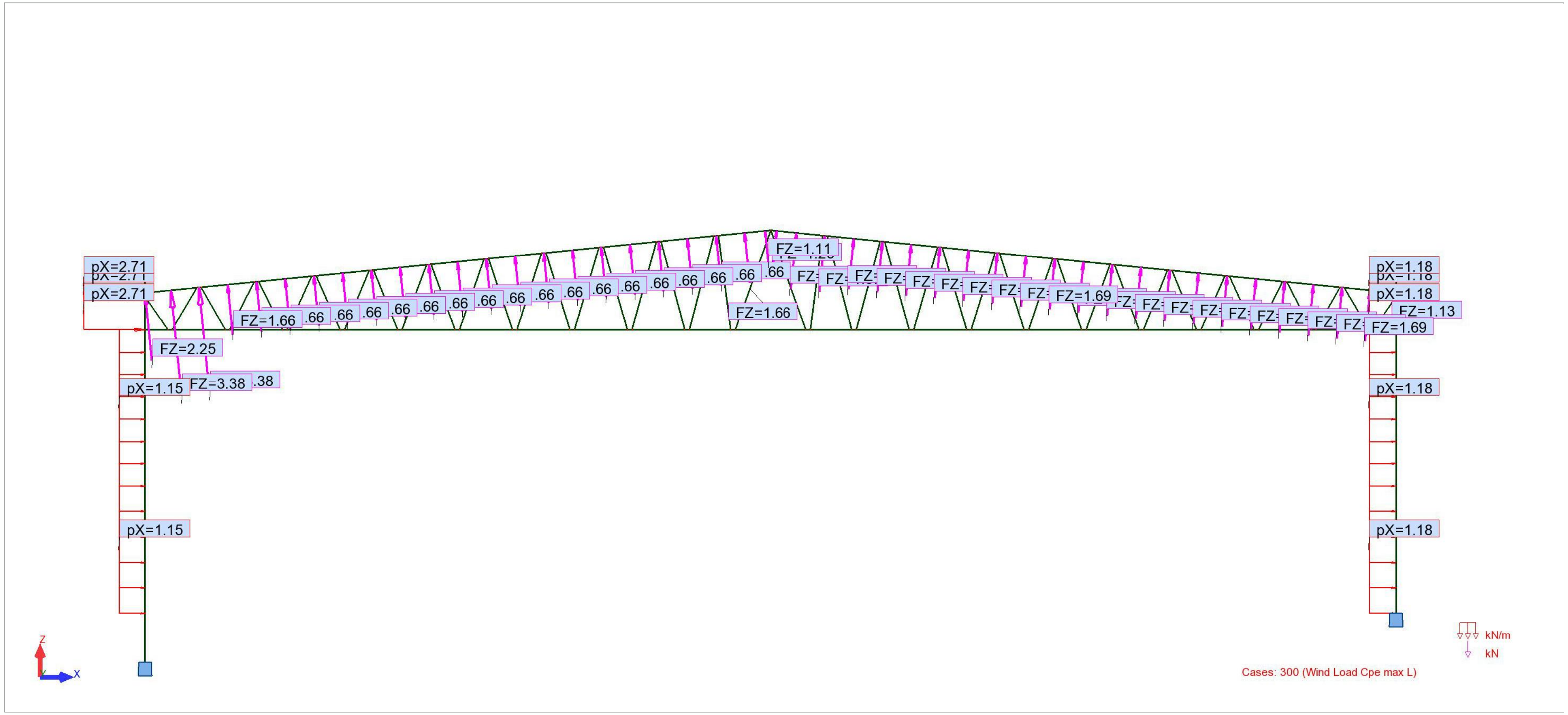
View - Cases: 201 (Snow Load left)



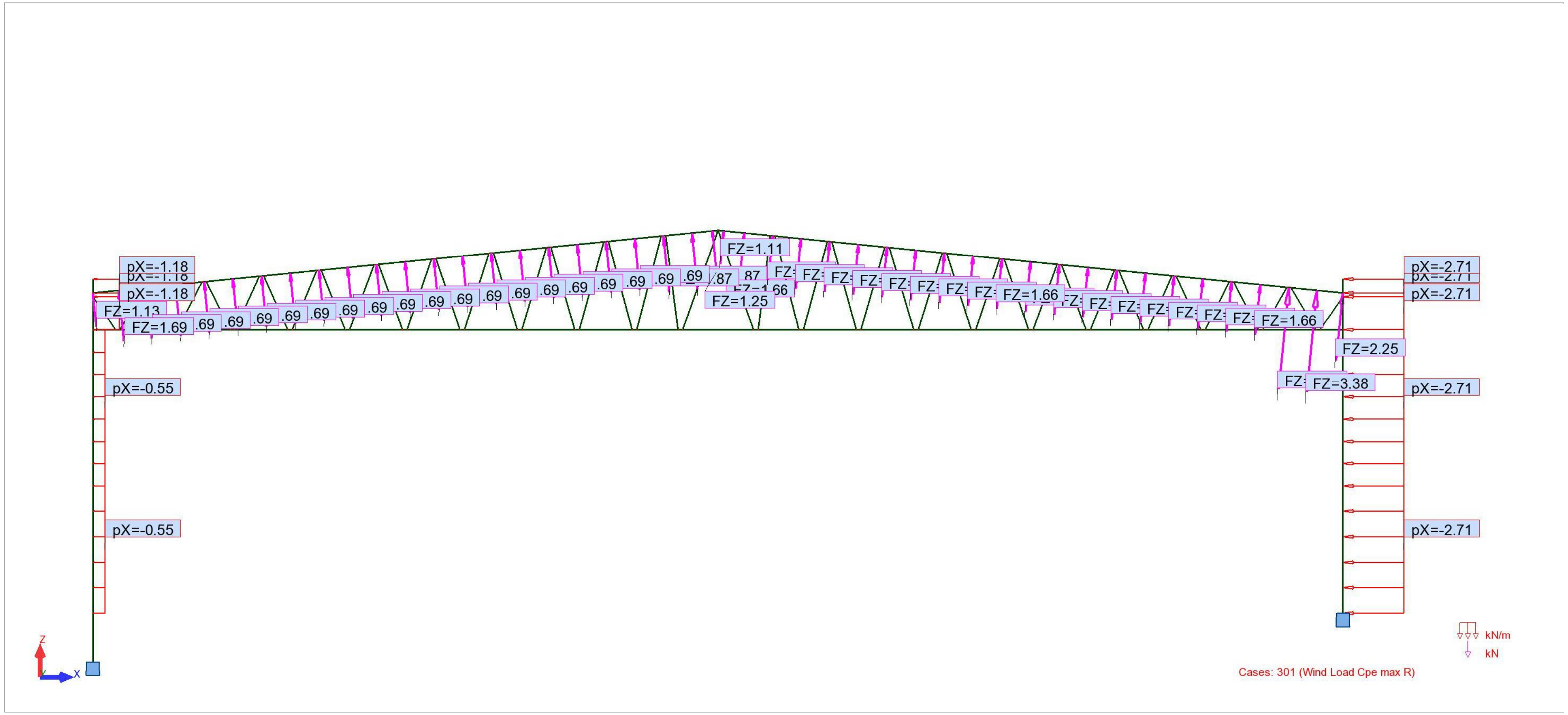
View - Cases: 202 (Snow Load right)



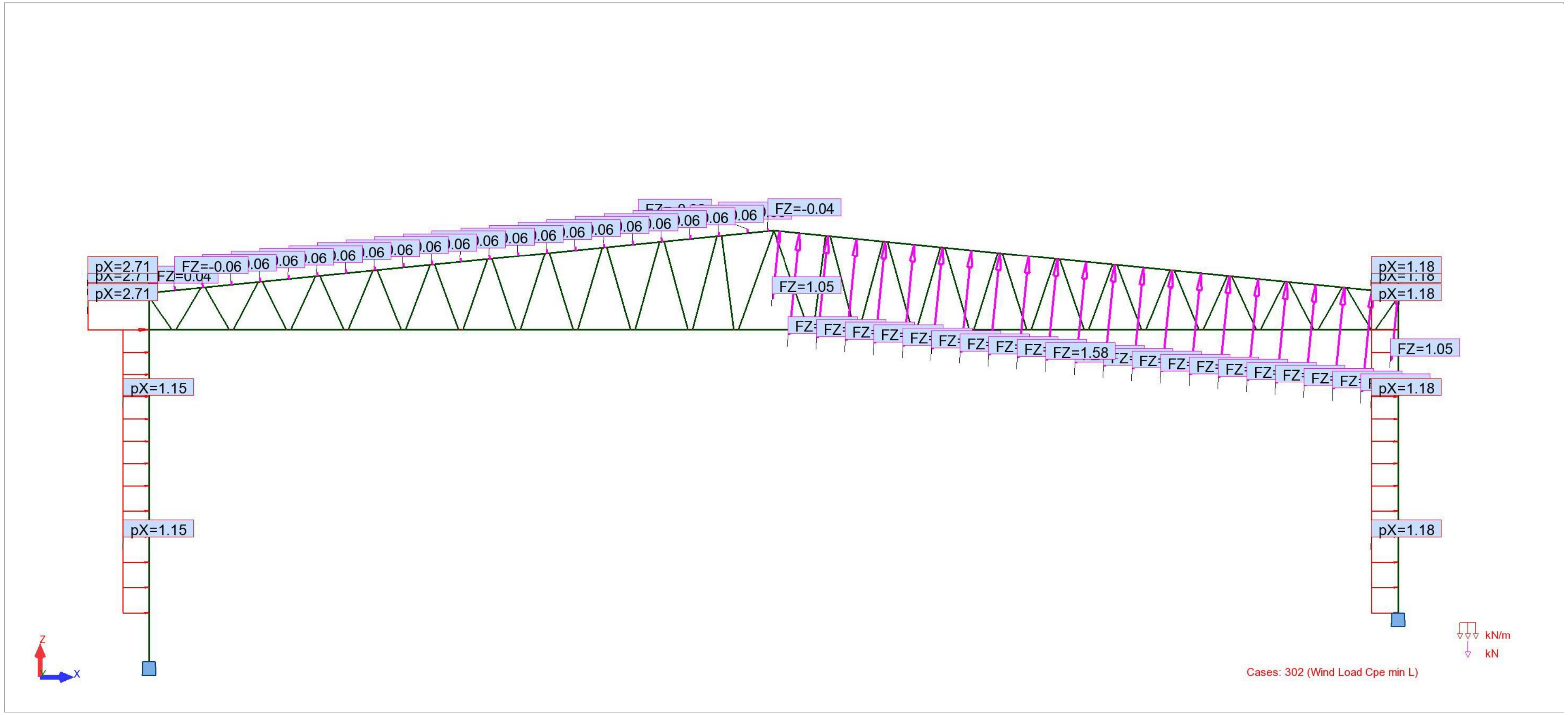
View - Cases: 300 (Wind Load Cpe max L)



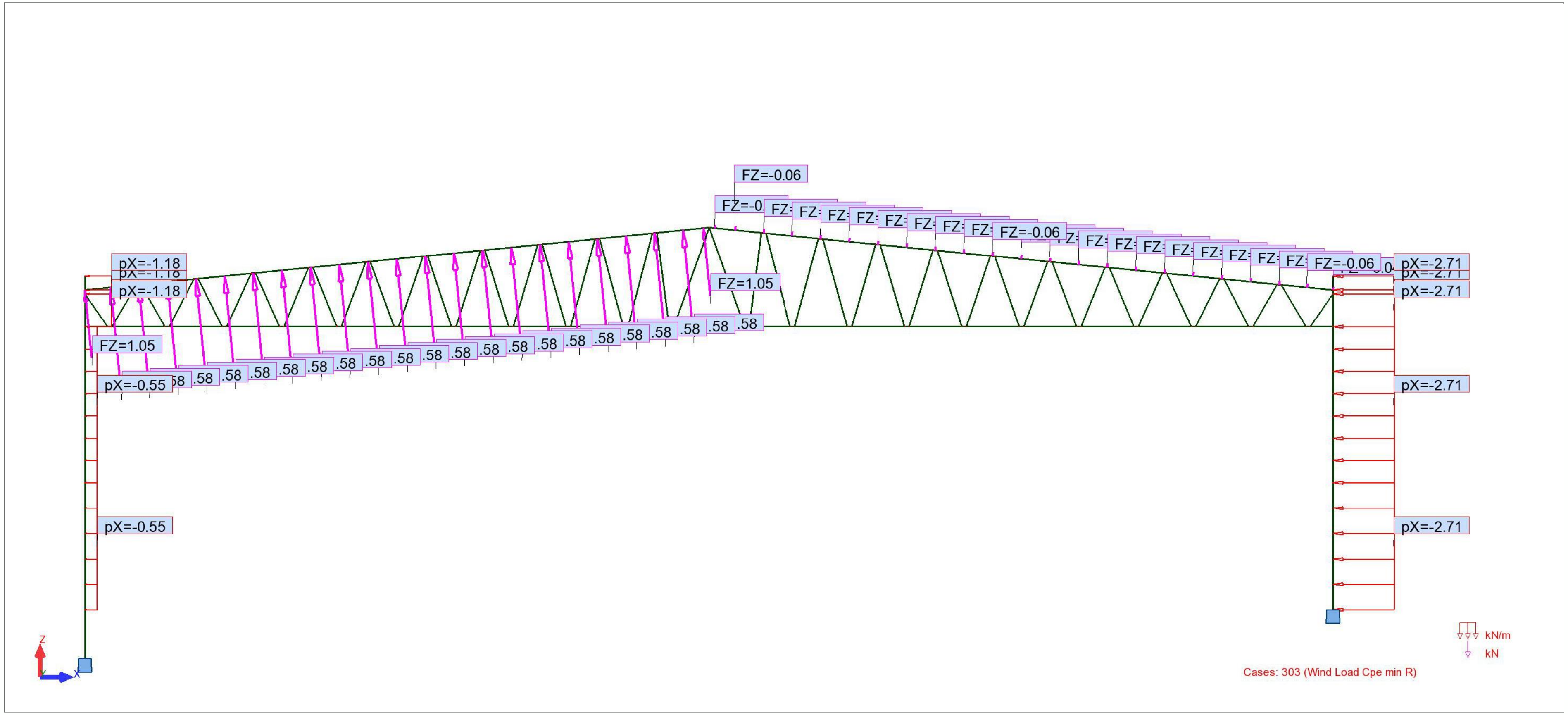
View - Cases: 301 (Wind Load Cpe max R)



View - Cases: 302 (Wind Load Cpe min L)

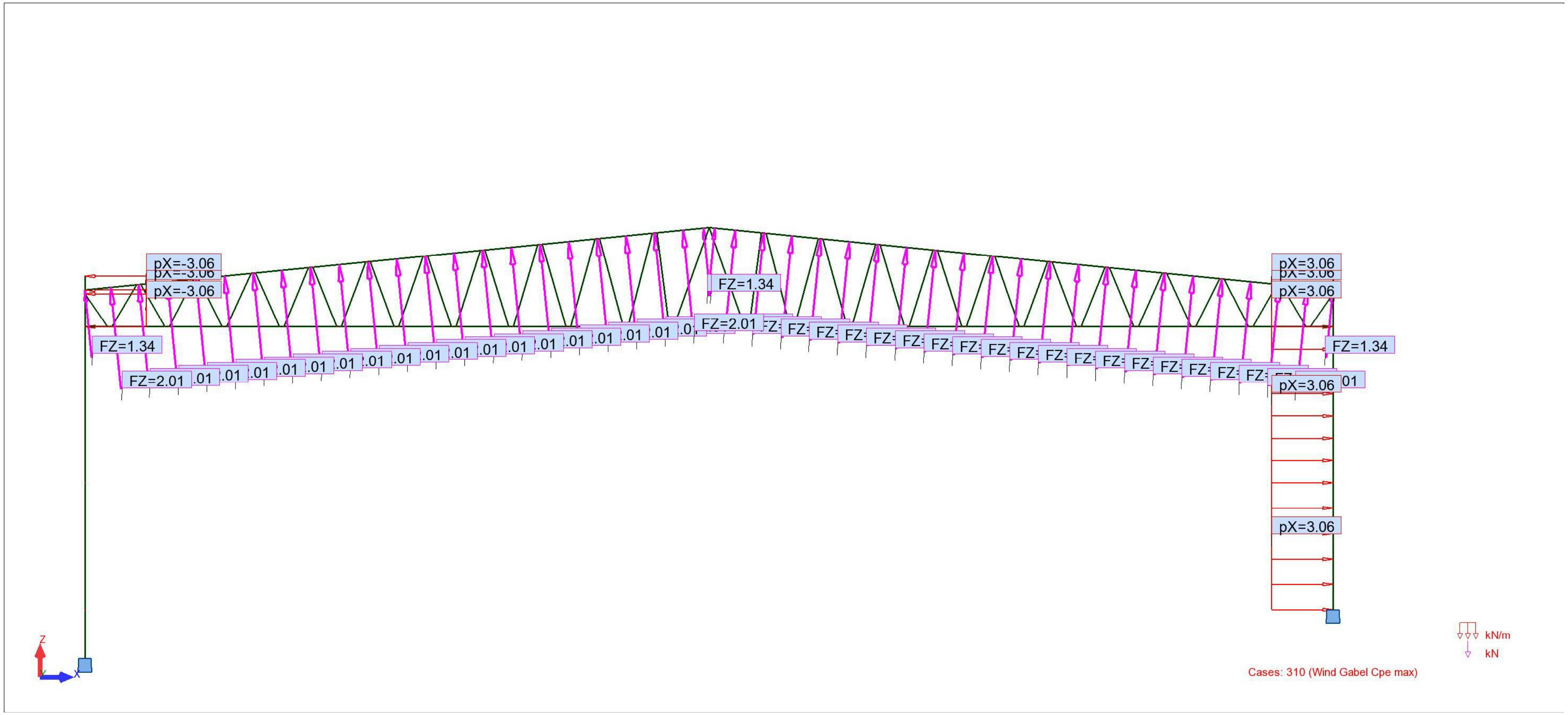


View - Cases: 303 (Wind Load Cpe min R)



Autodesk Robot Structural Analysis Professional 2018	
Author:	File: **CZ1138_FR1_fire.rtd**
Address:	Project: MTH110

View - Cases: 310 (Wind Gabel Cpe max)



Date : 23/07/18	Page : **1**

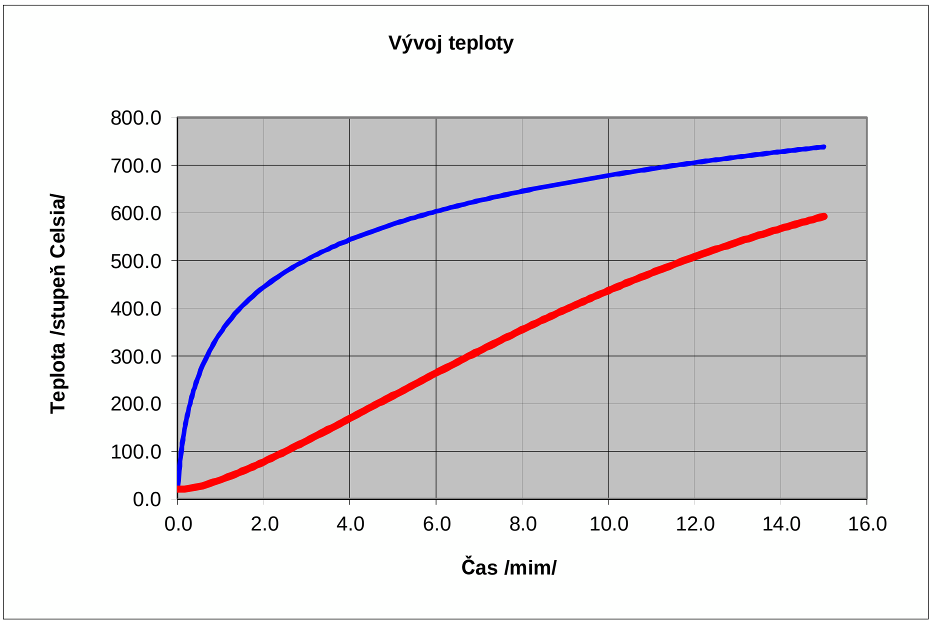
KOMBINACE ZATĚŽOVACÍCH STAVŮ PRO POSOUZENÍ RÁMU NA POŽÁRNÍ ODOLNOST R15

No.	Combination	Name and definition of combination
1	10	Snow
		1x(101+102+104)+0.2x(201+202)
2	11	Snow 0.5 left
		1x(101+102+104)+0.1x(201)+0.2x(202)
3	12	Snow 0.5 right
		1x(101+102+104)+0.2x(201)+0.1x(202)
4	20	Wind Cpe max L
		1x(101+102)+0.2x(300)
5	21	Wind Cpe max R
		1x(101+102)+0.2x(301)
6	22	Wind Cpe min L
		1x(101+102+104)+0.2x(302)
7	23	Wind Cpe min R
		1x(101+102+104)+0.2x(303)
8	50	Wind Cpe max from gabel
		1x(101+102)+0.2x(310)

VLIV TEPLOTY ZAVEDEN POMOCÍ NORMOVÉ KŘIVKY.

VÝPOČET PROVEDEN V PROGRAMU AUTODESK ROBOT STRUCTURAL ANALYSIS PROFESSIONAL 2018

REDUKČNÍ SOUČINITELÉ PRO NÁVRH NA R15
HORNÍ PÁS VAZNÍKU 2X C170X6



Čas t=15 min.

	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
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			
592.74	**0.493**	**0.193**	**0.331**

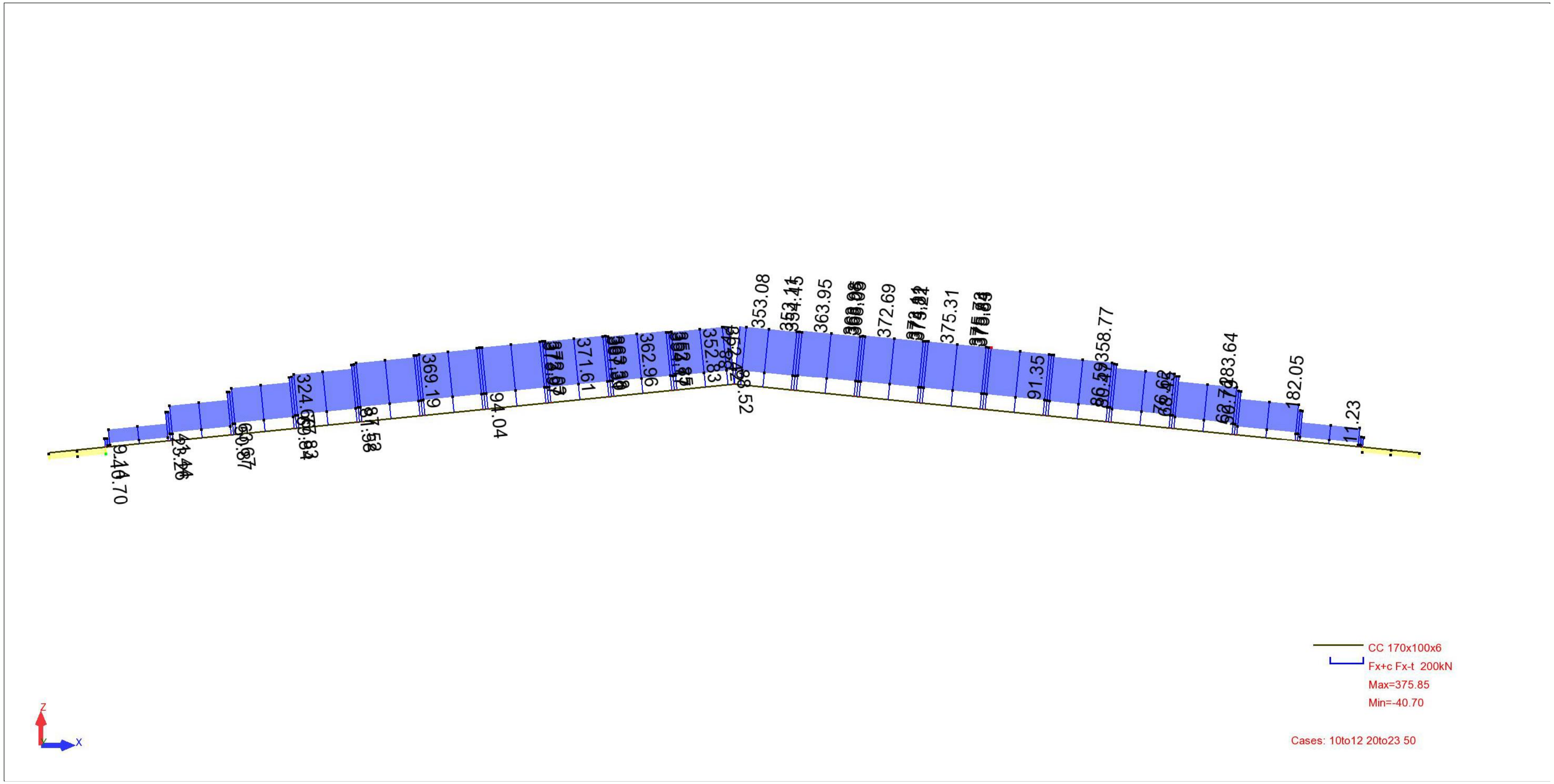
	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
592.74	**0.317**	**0.331**	

$k_{y\Theta}$ red. součinitel na mez kluzu
$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu
$k_{E\Theta}$ red. součinitel pro linerní pružnou část

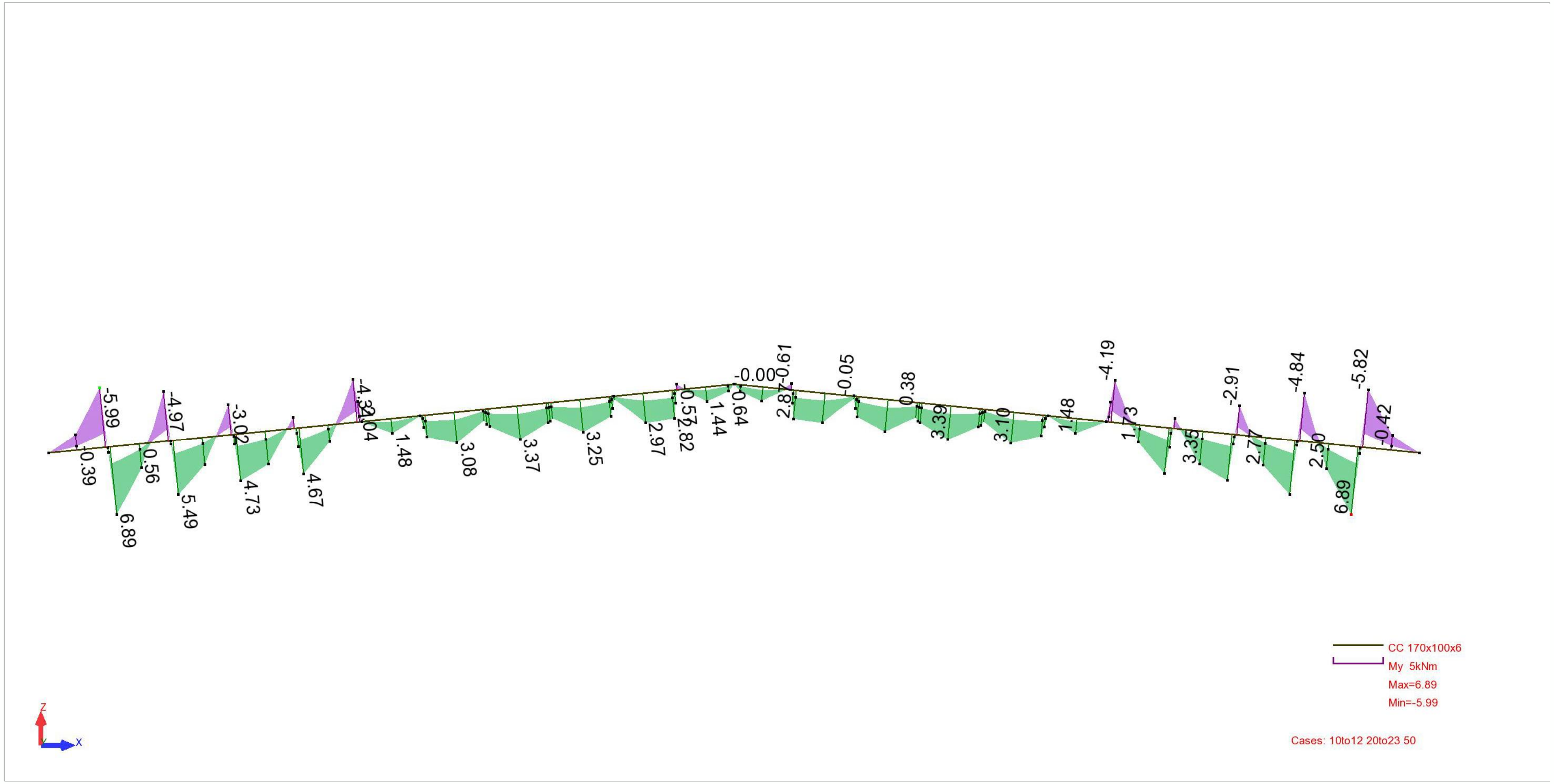
	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
592.74	**0.244**	**0.396**	**0.000**

View:1 - FX; Cases: 10to12 20to23 50

HORNÍ PÁS – NORMÁLOVÉ SÍLY



View:1 - MY; Cases: 10to12 20to23 50 HORNÍ PÁS – OHYBOVÉ MOMENTY



POSUDEK HORNÍHO PÁSU VAZNÍKU NA R15

Double-C Truss member

$h \equiv 170 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 38 \cdot mm$ $t \equiv 6 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $h_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $kb = 400.92\ mm$ $\gamma_{M1} \equiv 1.0$

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

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 4.899 \times 10^3\ mm^2$

$I_{zz} = 4.59 \times 10^7\ mm^4$ $W_{zz.eff} = 2.97 \times 10^5\ mm^3$

$I_{yy} := 2 \cdot I_y$ $I_{yy} = 2.2 \times 10^7\ mm^4$

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 2.69 \times 10^5\ mm^3$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 2.69 \times 10^5\ mm^3$

$W_{zz} := \frac{I_{zz}}{b + \frac{d}{2}}$ $W_{zz} = 3.06 \times 10^5\ mm^3$

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

$M_{yy.cRk} = 34.55\ kN \cdot m$

$M_{ycRk} = 17.27\ kN \cdot m$

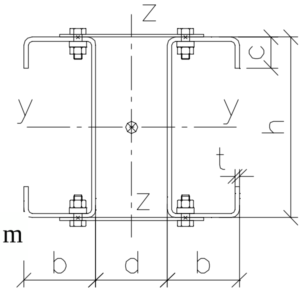
$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$N_{cc.Rk} = 629.17\ kN$

$f_{yb} = 128.5 \frac{N}{mm^2}$

$f_u = 141.35 \frac{N}{mm^2}$

$M_{zz.cRk} = 38.19\ kN \cdot m$



$i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 67.08\ mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 96.78\ mm$

torsion_plate_truss = "NO"

Stresses and global geometry:

Beam is designed as uniform built up member: YES or NO: built_up ≡ "YES"

Number of pair battens per length L: $n_{batt} := 2$

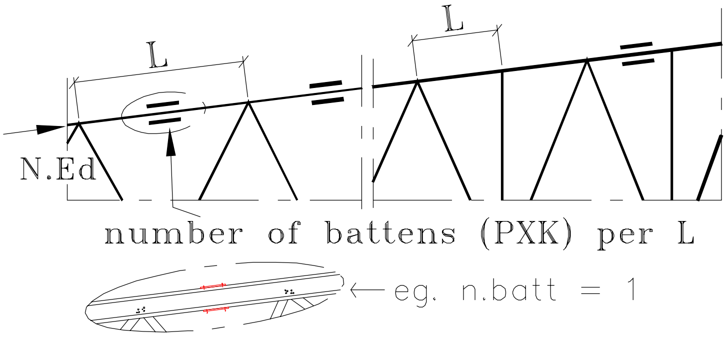
Buckling lengths:

$L := 1.5 \cdot m$	Length between diagonals
$L_y := 0.9 \cdot L$	Buckling length y-y
$L_z := 1.5 \cdot m$	Buckling length z-z
$L_{LT} := 1.0 \cdot L$	Length for LT-buckling

The moment is distributed according to frame modelling like shown in figure: ("A": Z-purlins; "B" longspan decking)

Moment_dis := "A"

$L_y = 1.35\ m$	Flexural buckling axis y-y
$L_z = 1.5\ m$	Flexural buckling axis z-z
$L_{LT} = 1.5\ m$	Lateral-torsional buckling



For single C-profile: $L_{T.C} = 0.35\ m$ Torsional buckling

$L_{z.C} = 0.5\ m$ FB z-z for single profile

stresses:

$M_{Ed.end.1} := 6.9 \cdot kN \cdot m$ sign: (+)

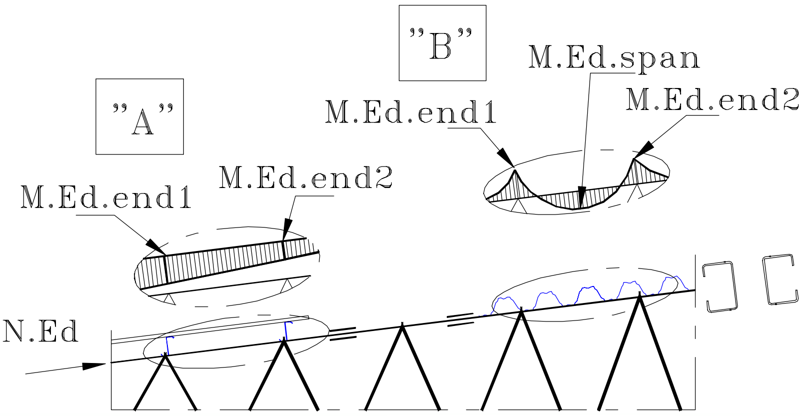
$M_{Ed.end.2} := -6 \cdot kN \cdot m$ sign: (+) or (-)

$M_{Ed.span} := 0 \cdot kN \cdot m$ sign: (+) or (-) if reverse moment (only of model "B")

$N_{Ed} := 376 \cdot kN$

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = 0.01\ kN \cdot m$

$M_{z.Ed} := \left| \Delta M_{z.shift} \right|$ shift of neutral axis for member in compression



$$\begin{pmatrix} f_{yb} \\ f_u \end{pmatrix} := \begin{pmatrix} 159 \\ 174 \end{pmatrix} \cdot \frac{N}{mm^2}$$

$b = 100\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 6\,mm$ $f_{yb} = 159\,\frac{N}{mm^2}$ $f_u = 174\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

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

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

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 58.77$ $\lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.y.FBcc} = 0.34$

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

$\phi_{y.FB.cc} := 0.5 \cdot \left[1 + \alpha_y \cdot (\lambda_{r.y.FBcc} - 0.2) + \lambda_{r.y.FBcc}^2\right]$ $\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB.cc} + \sqrt{\phi_{y.FB.cc}^2 - \lambda_{r.y.FBcc}^2}}, 1\right)$ $\chi_{y.FBcc} = 0.97$

$\phi_{y.FB.cc} = 0.57$

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

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

non-dimensional slenderness:

$\lambda_1 = 58.77$ $\lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.z.FBcc} = 0.26$

$\phi_{z.FB.cc} := 0.5 \cdot \left[1 + \alpha_z \cdot (\lambda_{r.z.FBcc} - 0.2) + \lambda_{r.z.FBcc}^2\right]$ $\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB.cc} + \sqrt{\phi_{z.FB.cc}^2 - \lambda_{r.z.FBcc}^2}}, 1\right)$ $\chi_{z.FBcc} = 0.98$

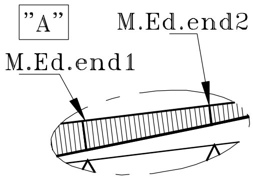
$\phi_{z.FB.cc} = 0.5$

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 critikal buckling momtent M_{cr} : given in German ENV 1993-1-1 Annex F.

$k_{M.cr} := 1.0$ (hinged at ends) $k_{w.M.cr} := 1.0$ (no special wrap restraints at ends) Moment_dis = "A"

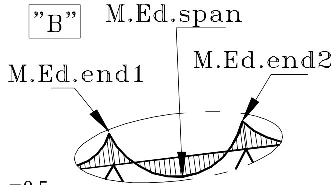
$\psi_m := \begin{cases} \frac{M_{Ed.end.2}}{M_{Ed.end.1}} & \text{if } |M_{Ed.end.1}| \geq |M_{Ed.end.2}| \\ \frac{M_{Ed.end.1}}{M_{Ed.end.2}} & \text{if } |M_{Ed.end.1}| < |M_{Ed.end.2}| \end{cases}$



$C_{1.tr.A} := \min(1.88 - 1.40 \cdot \psi_m + 0.52 \cdot \psi_m^2, 2.7)$

$C_{1.tr.A} = 2.7$

$C_{1.tr.B} := 1.285$



$C_{1.tr} := \begin{cases} C_{1.tr.A} & \text{if Moment\_dis = "A"} \\ C_{1.tr.B} & \text{if Moment\_dis = "B"} \end{cases}$ $C_{1.tr} = 2.7$

Elastic critical moment for lateral-torsional buckling:

$M_{cr.tr} := C_{1.tr} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{(k_{M.cr} \cdot L_{LT})^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr} \cdot L_{LT})^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}}\right]^{0.5}$ $M_{cr.tr} = 1175.1\,kN \cdot m$

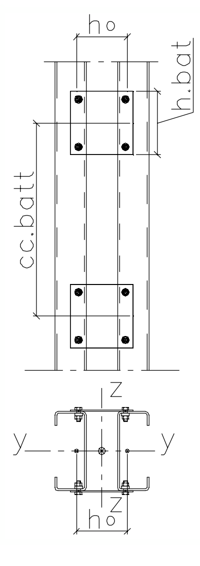
Relative slenderness: $\lambda_{rLT.cc} := \sqrt{\frac{2 \cdot W_{eff.y.1} \cdot f_{yb}}{M_{cr.tr}}}$ $\lambda_{rLT.cc} = 0.19$

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

$\phi_{LT.cc} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{rLT.cc} - 0.2) + \lambda_{rLT.cc}^2\right]$ Reduction buckling factor $\chi_{LT.y} := \min\left(\frac{1}{\phi_{LT.cc} + \sqrt{\phi_{LT.cc}^2 - \lambda_{rLT.cc}^2}}, 1\right)$ $\chi_{LT.y} = 1$

$\phi_{LT.cc} = 0.52$ $\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{yy} \geq I_{zz} \\ 1 & \text{otherwise} \end{cases}$

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



$cc_{batt} = 0.75\,m$ bow imperfection: $e_0 := \frac{L_z}{500}$ $e_0 = 3\,mm$

Effective second moment of area of battended built-up member: $A_{ch} := A_g$: area of one chord

$h_0 := d + 2 \cdot e_1$ $h_0 = 178.4\,mm$ 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 $i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2 \cdot A_{ch}}}$ $\lambda_{CC} := \frac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 15.5$

EN 1993-1-1: table 6.8 Efficiency factor: $\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \geq 150 \\ \left(2 - \frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \leq 75 \end{cases}$ $\mu_{CC} = 1$

$I_{l.CC} = 4.59 \times 10^7\,mm^4$ $r_{CC} := 2$

$b = 100\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 6\,mm$ $f_{yb} = 159 \frac{N}{mm^2}$ $f_u = 174 \frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu_{CC} \cdot I_{ch}$ $I_{eff} = 4.589 \times 10^7\,mm^4$ effective I of built-up member

Shear stiffnes *EN 1993-1-1:6.4.3.1:(2)*

$I_b := \frac{t_{batt} \cdot h_{batt}^3}{12}$ I of batten $n_{batt.0} := 2$ number of planes of lacings

$$S_v := \min\left[\left[\frac{24 \cdot E \cdot I_{ch}}{cc_{batt}^2 \cdot \left(1 + \frac{2 \cdot I_{ch}}{n_{batt.0} \cdot I_b} \cdot \frac{h_0}{cc_{batt}}\right)}\right], \left(\frac{2 \cdot \pi^2 \cdot E \cdot I_{ch}}{cc_{batt}^2}\right)\right] \quad S_v = 6283\,kN$$

effective critical force of built-up member: $N_{cr.CC} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_z^2}$ $N_{cr.CC} = 11202\,kN$

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

moment z-z without second order effects $M_{z.Ed.I} := 0 \cdot kN \cdot m$

moment with second order effects

$$M_{z.Ed.C} := \begin{cases} \left| \dfrac{N_{Ed} \cdot e_0 + M_{z.Ed.I}}{1 - \dfrac{N_{Ed} \cdot \gamma_{M1}}{N_{cr.CC}} - \dfrac{N_{Ed} \cdot \gamma_{M1}}{S_v}} \right| & \text{if built\_up} = \text{"YES"} \\ (0 \cdot kN \cdot m) & \text{if built\_up} = \text{"NO"} \end{cases}$$

$M_{z.Ed.C} = 1.2\,kN \cdot m$

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:

$$X_d := \begin{cases} 1 & \text{if } d \ge 8 \cdot mm \\ 0 & \text{otherwise} \end{cases} \quad N_{ch.Ed} := \frac{N_{Ed}}{r_{CC}} + \frac{|M_{z.Ed.C}| \cdot h_0 \cdot A_{ch} \cdot X_d}{2 \cdot I_{eff}} \quad N_{ch.Ed} = 193.92\,kN$$

Interaktion formulae 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

$C_{my} = 0.4$ $C_{mLT} := C_{my}$ $n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}}$ $n_{y.2} = 0.62$ $n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}}$ $n_{z.2} = 0.61$ $C_{mz.2} := 0.95$

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

$k_{yy} := \min\left[C_{my} \cdot \left(1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}\right), C_{my} \cdot \left(1 + 0.6 \cdot n_{y.2}\right)\right]$ $k_{yy} = 0.45$

$k_{zy} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FBcc}}{C_{mLT} - 0.25} \cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT} - 0.25} \cdot n_{z.2}\right)$ $k_{zy} = 0.95$

$k_{zz.2} := \min\left[C_{mz.2} \cdot \left(1 + 0.6 \cdot \lambda_{r.z.FBcc} \cdot n_{z.2}\right), C_{mz.2} \cdot \left(1 + 0.6 \cdot n_{z.2}\right)\right]$ $k_{zz.2} = 1.04$ $k_{yz.2} := k_{zz.2}$

Reduction factor for L-T buckling: $\chi_{LT} = 1$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.97$ $\chi_{z.FBcc} = 0.98$

Combined bending an axial compression EN 1993-1-3: 6.2.5 (2):

$$\frac{N_{Ed} \cdot \gamma_{M0}}{2 \cdot N_{c.Rk}} + \frac{\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \cdot \gamma_{M0}}{2 \cdot M_{ycRk}} + \frac{|M_{z.Ed.C} \cdot \gamma_{M0}|}{M_{zz.cRk}} = 0.83$$

<1.0

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

Evading in y-y:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}} + k_{yy} \cdot \frac{\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{yz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.74$$

<1.0

Evading in z-z:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}} + k_{zy} \cdot \frac{\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{zz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.83$$

<1.0

If not designed as a built-up member (that means no PXK-plates for connecting), check only single C-profiles according to formulaes further down. built_up = "YES"

$b = 100\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 6\,mm$ $f_{yb} = 159\,\frac{N}{mm^2}$ $f_u = 174\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1) if the truss beam is designed as built-up member. Otherwise: EN 1993-1-1: 6.3.3

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

$$M_{y.i}(x_i) := \frac{M_{Ed.end.2} - M_{Ed.end.1}}{L} \cdot x_i + M_{Ed.end.1}$$

For moment distribution accord. to model "A"

Moment at quarter point of unbraced segment: $x_{1\_4} := L \cdot 0.25$ $M_{y.AA} := M_{y.i}(x_{1\_4})$ $M_{y.AA} = 3.67\,kN \cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := L \cdot 0.5$ $M_{y.BA} := M_{y.i}(x_{1\_2})$ $M_{y.BA} = 0.45\,kN \cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := L \cdot 0.75$ $M_{y.CA} := M_{y.i}(x_{3\_4})$ $M_{y.CA} = -2.77\,kN \cdot m$

$$C_{bA} := \frac{12.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3 \cdot M_{y.AA} + 4 \cdot M_{y.BA} + 3 \cdot M_{y.CA}}$$

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

$z_{0.cr} := 0 \cdot mm$ (coord. shear center)

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

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

$$\sigma_{ez} := \frac{\pi^2 \cdot E}{\left(\frac{1 \cdot L_{z.C}}{i_z}\right)^2}$$ $\sigma_{ez} = 3106.59\,\frac{N}{mm^2}$

$$\sigma_T := \frac{1}{A_g \cdot r_0^2} \cdot \left(G \cdot I_T + \frac{\pi^2 \cdot E \cdot I_\omega}{L_{T.C}^2}\right)$$ $\sigma_T = 3417.09\,\frac{N}{mm^2}$

Elastic ... ymmetric sections, bending about the axis of symmetry:

$M_{e.cr.C\_A} := |C_{bA}| \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T}$ $M_{e.cr.C\_A} = 3710.2\,kN \cdot m$

Relative slenderness:

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

$\lambda_{rLT.C\_A} = 0$

$\phi_{LT.C\_A} := 0.5 \cdot \left[1 + \alpha_{LT.C} \cdot (\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right]$ $\phi_{LT.C\_A} = 0.48$

$$\chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right)$$ $\chi_{LT.C\_A} = 1$

For moment distribution accord. to model "B"

Moment at quarter point of unbraced segment:

$$M_{y.AB} := \frac{(M_{Ed.span} - M_{Ed.end.1}) \cdot L \cdot 0.25}{L \cdot 0.5} + M_{Ed.end.1}$$ $M_{y.AB} = 3.45\,kN \cdot m$

Moment at centerline of unbraced segment: $M_{y.BB} := M_{Ed.span}$ $M_{y.BB} = 0\,kN \cdot m$

Moment at 3/4-point of unbraced segment:

$$M_{y.CB} := \frac{(M_{Ed.span} - M_{Ed.end.2}) \cdot L \cdot 0.25}{L \cdot 0.5} + M_{Ed.end.2}$$ $M_{y.CB} = -3\,kN \cdot m$

$$C_{bB} := \frac{12.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3 \cdot M_{y.AB} + 4 \cdot M_{y.BB} + 3 \cdot M_{y.CB}}$$

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

$M_{e.cr.C\_B} := |C_{bB}| \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T}$ $M_{e.cr.C\_B} = 4338.5\,kN \cdot m$

Relative slenderness:

$$\lambda_{rLT.C\_B} := \begin{cases} \sqrt{\frac{W_{eff.y.1} \cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0 \\ 0.2 & \text{if } M_{e.cr.C\_B} = 0 \end{cases}$$ $\lambda_{rLT.C\_B} = 0.07$

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

$$\chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right)$$ $\chi_{LT.C\_B} = 1$

$\phi_{LT.C\_B} = 0.48$

Moment distribution:

$$\chi_{LT.C} := \begin{cases} \chi_{LT.C\_A} & \text{if Moment\_dis} = \text{"A"} \\ \chi_{LT.C\_B} & \text{if Moment\_dis} = \text{"B"} \end{cases}$$ $\chi_{LT.C} = 1$

Chord at end panel: highest shear force: $V_{Ed.max} := \pi \cdot \frac{M_{z.Ed.C}}{L_z}$ $V_{Ed.max} = 2.61\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 1.3\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.016$ << 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:* $M_{z.ch} = 0.49\,kN \cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C} \cdot \sin\left(\frac{\pi \cdot cc_{batt}}{L_z}\right)$ $M_{z.Ed.C.end} = 1.24\,kN \cdot m$ moment due to bow imperfection att end panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end} \cdot \frac{(h_0 \cdot A_{ch})}{(2 \cdot I_{eff})}$ $N_{ch.end} = 193.92\,kN$

$b = 100\,mm \quad h = 170\,mm \quad c = 38\,mm \quad t = 6\,mm \quad f_{yb} = 159\frac{N}{mm^2} \quad f_u = 174\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

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

$n_{y.C} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}} \qquad n_{y.C} = 0.64$

$n_{z.C} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}} \qquad n_{z.C} = 0.62$

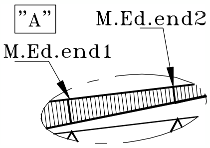
$\chi_{y.FB}(L_y) = 0.96$

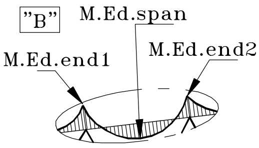
$\chi_{z.FB}(L_{z.C}) = 1$

Equivalent uniform moment factors:

$C_{my} = 0.4 \quad C_{mLT} = 0.4 \quad \psi_{m.z} := 1$

$C_{mz.C} := \max[(0.6 + 0.4 \cdot \psi_{m.z}), 0.4] \quad C_{mz.C} = 1$





$M_{Ed.end.1} = 6.9\,kN \cdot m$

$M_{Ed.end.2} = -6\,kN \cdot m$

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

$\lambda_{r.y.FB}(L_y) = 0.31 \qquad k_{yy.C} := \min[C_{my} \cdot (1 + 0.6 \cdot \lambda_{r.y.FB}(L_y) \cdot n_{y.C}), C_{my} \cdot (1 + 0.6 \cdot n_{y.C})] \qquad k_{yy.C} = 0.45$

$\lambda_{r.z.FB}(L_{z.C}) = 0.2 \qquad k_{zz.C} := \min[C_{mz.C} \cdot (1 + 0.6 \cdot \lambda_{r.z.FB}(L_{z.C}) \cdot n_{z.C}), C_{mz.C} \cdot (1 + 0.6 \cdot n_{z.C})] \qquad k_{zz.C} = 1.08$

$k_{zy.C} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FB}(L_{z.C})}{C_{my} - 0.25} \cdot n_{z.C}, 1 - \frac{0.05}{C_{mLT} - 0.25} \cdot n_{z.C}\right) \qquad k_{zy.C} = 0.96 \qquad k_{yz.C} := k_{zz.C}$

Controll single C-profile between battens:

max moment y-y: $M_{y.max.C} := 0.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \qquad M_{y.max.C} = 3.45\,kN \cdot m$

max moment z-z: $M_{z.C} = 0.49\,kN \cdot m \qquad \Delta M_{z.shift} := |\Delta e_N \cdot N_{ch.end}| \qquad \Delta M_{z.shift} = 0.01\,kN \cdot m$

max chord force from above: $N_{Ed.C} = 193.92\,kN$

Buckling moment resistance y-y
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C} \cdot M_{ycRk} \qquad \chi_{LT.C} = 1 \qquad M_{y.Rk.C} = 17.27\,kN \cdot m$

Buckling resistance moment z-z
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk}) \qquad M_{z.Rk.C} = 7.7\,kN \cdot m$

Resistance for one profile buckling mode flexural buckling-: y-y $L_y = 1.35\,m \qquad \chi_{y.FB}(L_y) = 0.96 \qquad N_{b.Rk.y.FB}(L_y) = 302.4\,kN$

Resistance for one profile buckling mode flexural buckling-: z-z $L_{z.C} = 0.5\,m \qquad \chi_{z.FB}(L_{z.C}) = 1 \qquad N_{b.Rk.z.FB}(L_{z.C}) = 314.21\,kN$

Resistance for one profile buckling mode -torsional or torsional-flexural $L_{T.C} = 0.35\,m \qquad \chi_{TF}(L_{T.C}, L_y) = 0.95 \qquad N_{b.Rk.TF}(L_{T.C}, L_y) = 298\,kN$

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

At mid-span of built-up member: $\frac{N_{ch.Ed} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{|\Delta M_{z.shift}| \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.82$ <1.0

In end panel of built-up member: $\frac{N_{ch.end} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{(M_{z.C} + |\Delta M_{z.shift}|) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.88$ <1.0

Combined bending and axial compression
EN 1993-1-1: 6.3.3 (4): $\frac{\max(N_{ch.Ed}, N_{ch.end}) \cdot \gamma_{M1}}{\min(\chi_{y.FB}(L_y), \chi_{z.FB}(L_{z.C}), \chi_{TF}(L_{T.C}, L_y)) \cdot N_{c.Rk}} = 0.65$ <1.0

Combined bending + axial compression EN 1993-1-1: 6.3.3 (4): Buckling about y-y

At mid-span of built-up member: $\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}} + k_{yy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C} \cdot \frac{|\Delta M_{z.shift}| \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.73$ <1.0

In end panel of built-up member: $\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}} + k_{yy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C} \cdot \frac{(M_{z.C} + |\Delta M_{z.shift}|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.8$ <1.0

Combined bending + axial compression EN 1993-1-1: 6.3.3 (4): Buckling about z-z

At mid-span of built-up member: $\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}} + k_{zy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C} \cdot \frac{|\Delta M_{z.shift}| \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.81$ <1.0

In end panel of built-up member: $\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}} + k_{zy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C} \cdot \frac{(M_{z.C} + |\Delta M_{z.shift}|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.88$ <1.0

Check M, V och N acc. to EC

$Country := "Cz"$

$PR \equiv "CC170x6"$

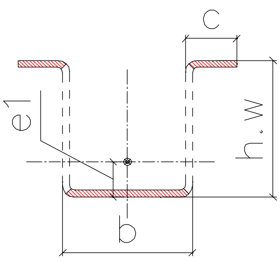
Profile data for a single profile

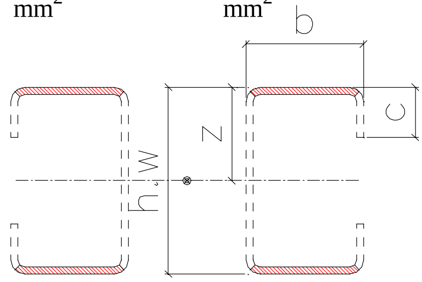
$\gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00 \quad \gamma_{M2} = 1.25$

$h_w = 170\,mm \quad b_c = 100\,mm \quad c = 38\,mm \quad t = 6\,mm \quad e_1 = 39.18\,mm \quad A_g = 2449.59\,mm^2$

$N_{cRk} = 1217.47\,kN \quad M_{Rk} = 67.22\,kN\cdot m \quad f_{yb} = 128.5\,\frac{N}{mm^2} \quad f_u = 141.35\,\frac{N}{mm^2}$

The beam is H-profile ("H") or double C-profile ("C")





Hole diameter: $d_o := 12.5\cdot mm$

Number of bolts at the cross-section in ONE web: $n_{cross} := 4$

Number screws and diameter i FLANGES: $X_{h.fl} := 0 \quad d_{o.fl} := 0\cdot mm$

Momentcap.one profile:

Axialforcecapacity one profile:

Axialforce acting as TENTION : "YES" or "NO" $N_{drag} := "YES"$

Netto area for shearforce:

$$A_w := \begin{cases} \left[\left[(h_w - t)\cdot t - n_{cross}\cdot d_o\cdot t\right]\cdot 2\right] & \text{if } L_{II} = 0\cdot mm \\ \left[\left[(L_{II} - t)\cdot t - n_{cross}\cdot d_o\cdot t\right]\cdot 2\right] & \text{if } L_{II} \neq 0\cdot mm \end{cases} \qquad A_w = 1368.00\,mm^2$$

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: $M_{Rd} := \frac{X\cdot M_{Rk}}{\gamma_{M0}} \qquad M_{Rd} = 134.44\,kN\cdot m$

Axialforce capacity compresion member: $N_{Rd.1} := \frac{N_{cRk}\cdot X}{\gamma_{M0}}$ (6.2.4) $\qquad N_{Rd.1} = 2434.9\,kN$

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)) $\quad N_{Rd.t.a} = 243.08\,kN$

For material >= 5.0mm : $N_{Rd.t.b} := \frac{0.9\cdot A_{net}\cdot f_u}{\gamma_{M2.t}} \qquad N_{Rd.t.b} = 218.8\,kN$ (EN1993-1-1: 6.2.3) $\quad \gamma_{M2.t} = 1.25$

$$N_{Rd.t} := \left(\begin{cases} N_{Rd.t.a} & \text{if } t \le 4\cdot mm \\ N_{Rd.t.b} & \text{if } t \ge 5\cdot mm \end{cases}\right)\cdot X \qquad N_{Rd.t} = 437.54\,kN$$

Normalkraftscapacity:

$$N_{Rd} := \begin{cases} N_{Rd.1} & \text{if } N_{drag} = "NO" \\ \min(N_{Rd.1}, N_{Rd.t}) & \text{if } N_{drag} = "YES" \end{cases} \qquad N_{Rd} = 437.54\,kN$$

Shearforce capacity:

$$\lambda_{wh} := 0.346\cdot\frac{h_w - t}{t}\cdot\sqrt{\frac{f_{yb}}{E}} \qquad \lambda_{wh} = 0.45$$

$$f_{vb} := \begin{cases} \frac{1}{\sqrt{3}}\cdot f_{yb} & \text{if } \lambda_{wh} \le 0.83 \\ 0.48\cdot\frac{f_{yb}}{\lambda_{wh}} & \text{if } 0.83 < \lambda_{wh} < 1.40 \\ 0.67\cdot\frac{f_{yb}}{\lambda_{wh}^2} & \text{if } \lambda_{wh} \ge 1.40 \end{cases} \qquad f_{vb} = 74.19\,\frac{N}{mm^2}$$

$$V_{Rd} := \frac{A_w\cdot f_{vb}}{\gamma_{M0}} \qquad V_{Rd} = 101.49\,kN$$

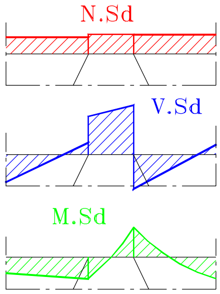
elast. momentcap.: $M_{elRd} := M_{Rd}$

$\theta_{pl} := 1.0$ faktor M_{el} -> M_{pl}

Section value for interaction formula M + N + V:
Momentresistance of a cross-section consisting only flanges: $M_{flRd} = 24.4\,kN\cdot m$

Element Nr: ....

$N_{Sd} := 376 \cdot kN$ $V_{Sd} := 65 \cdot kN$ $M_{Sd} := 6.9 \cdot kN \cdot m$



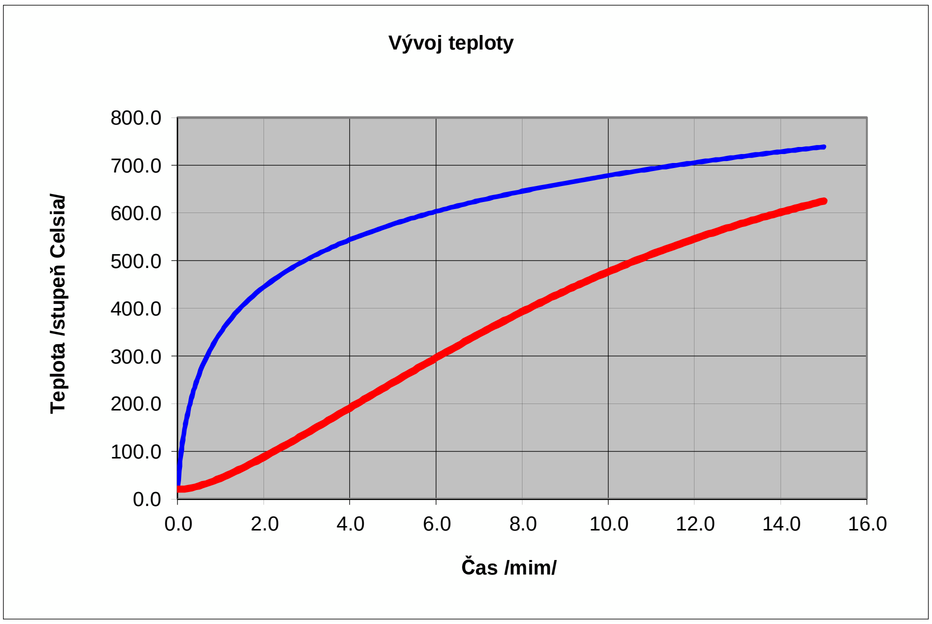
$\frac{|V_{Sd}|}{V_{Rd}} = 0.64$ <1.0 (EN 1993-1-1: 6.2.6)

$\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 0.91$ <1.0 (EN 1993-1-3 6.1.8+6.1.9)

Ratio for check
Moment + Axialforce + Shear force

$M\_N\_V_{EC} = 0.98$ <1.0 (EN 1993-1-3 6.1.10)

REDUKČNÍ SOUČINITELÉ PRO NÁVRH NA R15
DOLNÍ PÁS VAZNÍKU 2X C170X5



Čas t=15 min.

	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
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			
625.04	**0.410**	**0.154**	**0.265**

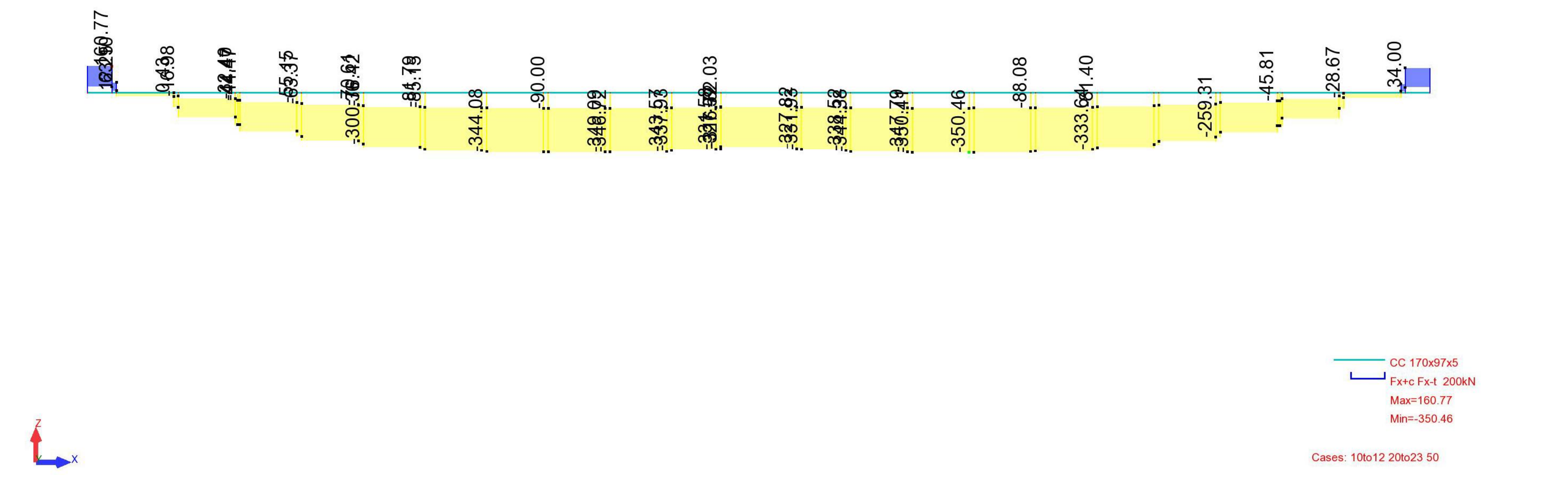
	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
625.04	**0.257**	**0.265**	

$k_{y\Theta}$ red. součinitel na mez kluzu
$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu
$k_{E\Theta}$ red. součinitel pro linerní pružnou část

	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
625.04	**0.190**	**0.316**	**0.000**

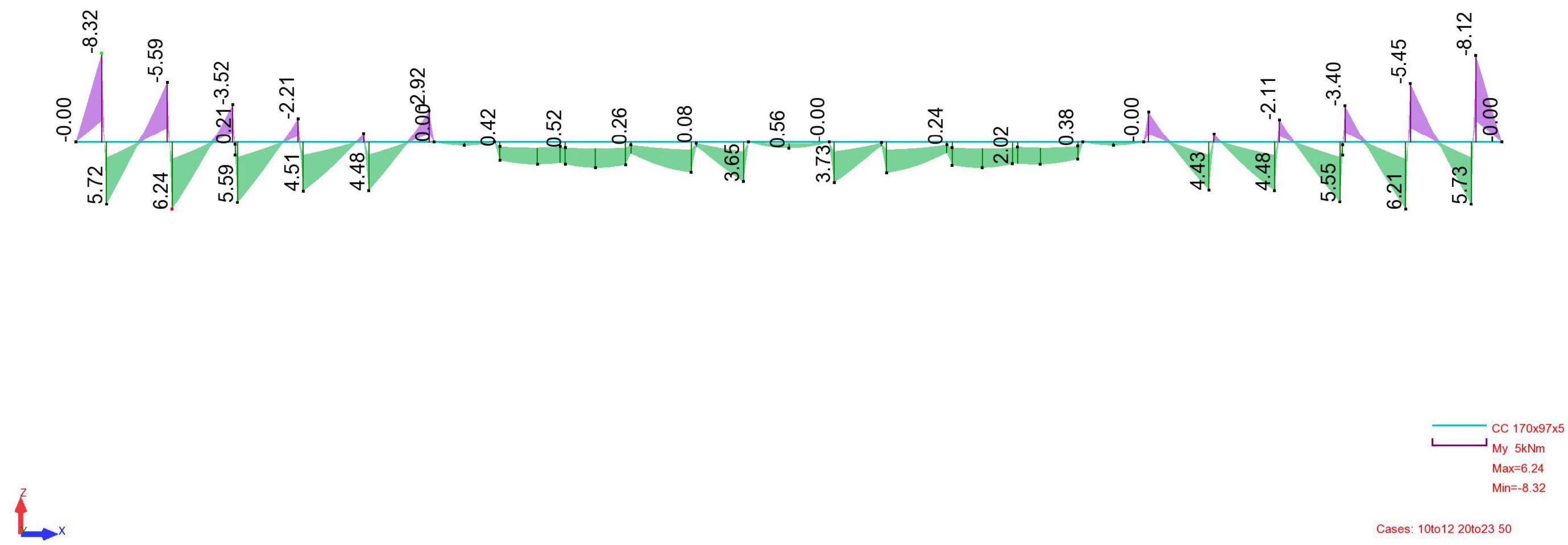
View:1 - FX; Cases: 10to12 20to23 50

DOLNÍ PÁS VAZNÍKU - NORMÁLOVÉ SÍLY



View:1 - MY; Cases: 10to12 20to23 50

DOLNÍ PÁS VAZNÍKU - OHYBOVÉ MOMENTY



Double-C Truss member

$h \equiv 170 \cdot mm$ $b \equiv 97 \cdot mm$ $c \equiv 38 \cdot mm$ $t \equiv 5 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $h_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $kb = 401.63\ mm$ $\gamma_{M1} \equiv 1.0$

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

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 4.071 \times 10^3\ mm^2$

$I_{zz} = 3.7 \times 10^7\ mm^4$ $W_{zz.eff} = 2.43 \times 10^5\ mm^3$

$I_{yy} := 2 \cdot I_y$ $I_{yy} = 1.84 \times 10^7\ mm^4$

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 2.23 \times 10^5\ mm^3$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 2.23 \times 10^5\ mm^3$

$W_{zz} := \frac{I_{zz}}{b + \frac{d}{2}}$ $W_{zz} = 2.52 \times 10^5\ mm^3$

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

$M_{yy.cRk} = 28.69\ kN \cdot m$

$M_{ycRk} = 14.35\ kN \cdot m$

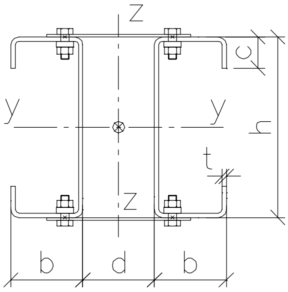
$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$N_{cc.Rk} = 515.69\ kN$

$f_{yb} = 128.5 \frac{N}{mm^2}$

$f_u = 141.35 \frac{N}{mm^2}$

$M_{zz.cRk} = 31.2\ kN \cdot m$



$i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 67.29\ mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 95.33\ mm$

torsion_plate_truss = "NO"

Stresses and global geometry:

Beam is designed as uniform built up member: YES or NO: built_up ≡ "YES"

Number of pair battens per length L: $n_{batt} := 1$

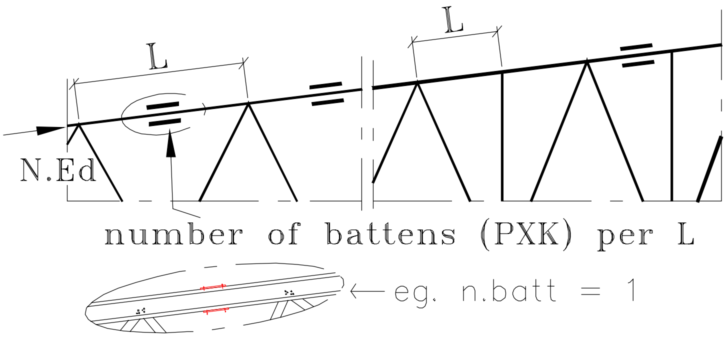
Buckling lengths:

$L := 1.5 \cdot m$	Length between diagonals
$L_y := 0.9 \cdot L$	Buckling length y-y
$L_z := 6 \cdot m$	Buckling length z-z
$L_{LT} := 1.0 \cdot L$	Length for LT-buckling

The moment is distributed according to frame modelling like shown in figure: ("A": Z-purlins; "B" longspan decking)

Moment_dis := "A"

$L_y = 1.35\ m$	Flexural buckling axis y-y
$L_z = 6\ m$	Flexural buckling axis z-z
$L_{LT} = 1.5\ m$	Lateral-torsional buckling



For single C-profile: $L_{T.C} = 0.52\ m$ Torsional buckling

$L_{z.C} = 0.75\ m$ FB z-z for single profile

stresses:

$M_{Ed.end.1} := 6.2 \cdot kN \cdot m$ sign: (+)

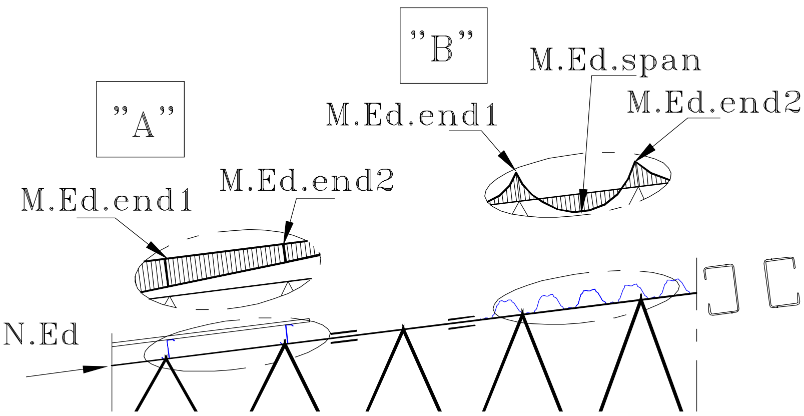
$M_{Ed.end.2} := -8.3 \cdot kN \cdot m$ sign: (+) or (-)

$M_{Ed.span} := 0 \cdot kN \cdot m$ sign: (+) or (-) if reverse moment (only of model "B")

$N_{Ed} := 161 \cdot kN$

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = 0.1\ kN \cdot m$

$M_{z.Ed} := |\Delta M_{z.shift}|$ shift of neutral axis for member in compression



$b = 97\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 5\,mm$

$f_{yb} = 129\,\frac{N}{mm^2}$ $f_u = 141\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

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

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

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 65.38$ $\lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.y.FBcc} = 0.3$

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

$\phi_{y.FB.cc} := 0.5 \cdot \left[1 + \alpha_y \cdot \left(\lambda_{r.y.FBcc} - 0.2\right) + \lambda_{r.y.FBcc}^2\right]$ $\phi_{y.FB.cc} = 0.56$

$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB.cc} + \sqrt{\phi_{y.FB.cc}^2 - \lambda_{r.y.FBcc}^2}}, 1\right)$ $\chi_{y.FBcc} = 0.98$

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

non-dimensional slenderness:

$\lambda_1 = 65.38$ $\lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.z.FBcc} = 0.96$

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

$\phi_{z.FB.cc} := 0.5 \cdot \left[1 + \alpha_z \cdot \left(\lambda_{r.z.FBcc} - 0.2\right) + \lambda_{r.z.FBcc}^2\right]$ $\phi_{z.FB.cc} = 1.0$

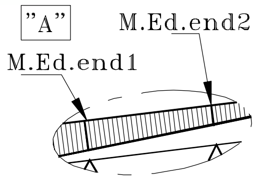
$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB.cc} + \sqrt{\phi_{z.FB.cc}^2 - \lambda_{r.z.FBcc}^2}}, 1\right)$ $\chi_{z.FBcc} = 0.63$

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 critikal buckling momtent M_{cr} : given in German ENV 1993-1-1 Annex F.

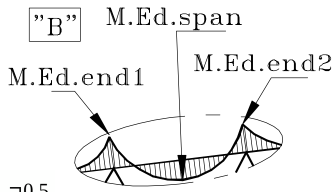
$k_{M.cr} := 1.0$ (hinged at ends) $k_{w.M.cr} := 1.0$ (no special wrap restraints at ends) Moment_dis = "A"

$\psi_m := \begin{cases} \frac{M_{Ed.end.2}}{M_{Ed.end.1}} & \text{if } \left|M_{Ed.end.1}\right| \ge \left|M_{Ed.end.2}\right| \\ \frac{M_{Ed.end.1}}{M_{Ed.end.2}} & \text{if } \left|M_{Ed.end.1}\right| < \left|M_{Ed.end.2}\right| \end{cases}$



$C_{1.tr.A} := \min\left(1.88 - 1.40 \cdot \psi_m + 0.52 \cdot \psi_m^2, 2.7\right)$

$C_{1.tr.A} = 2.7$



$C_{1.tr.B} := 1.285$

$C_{1.tr} := \begin{cases} C_{1.tr.A} & \text{if Moment\_dis} = \text{"A"} \\ C_{1.tr.B} & \text{if Moment\_dis} = \text{"B"} \end{cases}$ $C_{1.tr} = 2.7$

Elastic critical moment for lateral-torsional buckling:

$M_{cr.tr} := C_{1.tr} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{\left(k_{M.cr} \cdot L_{LT}\right)^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr} \cdot L_{LT}\right)^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}}\right]^{0.5}$ $M_{cr.tr} = 892.7\,kN \cdot m$

Relative slenderness: $\lambda_{rLT.cc} := \sqrt{\frac{2 \cdot W_{eff.y.1} \cdot f_{yb}}{M_{cr.tr}}}$ $\lambda_{rLT.cc} = 0.18$

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

$\phi_{LT.cc} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot \left(\lambda_{rLT.cc} - 0.2\right) + \lambda_{rLT.cc}^2\right]$ $\phi_{LT.cc} = 0.51$

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

$\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{yy} \ge I_{zz} \\ 1 & \text{otherwise} \end{cases}$

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

$cc_{batt} = 1.5\,m$ bow imperfection: $e_0 := \frac{L_z}{500}$ $e_0 = 12\,mm$

Effective second moment of area of battended built-up member:

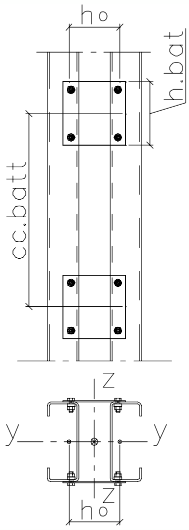
$A_{ch} := A_g$: area of one chord

$h_0 := d + 2 \cdot e_1$ $h_0 = 175.8\,mm$ 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

$i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2 \cdot A_{ch}}}$ $\lambda_{CC} := \frac{L_z}{i_{0.CC}}$ $\lambda_{CC} = 62.94$



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

$\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \ge 150 \\ \left(2 - \frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \le 75 \end{cases}$ $\mu_{CC} = 1$

$I_{l.CC} = 3.7 \times 10^7\,mm^4$

$r_{CC} := 2$

$b = 97\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 5\,mm$ $f_{yb} = 129\,\frac{N}{mm^2}$ $f_u = 141\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu_{CC} \cdot I_{ch}$ $I_{eff} = 3.7 \times 10^7\,mm^4$ effective I of built-up member

Shear stiffnes *EN 1993-1-1:6.4.3.1:(2)*

$I_b := \frac{t_{batt} \cdot h_{batt}^3}{12}$ I of batten $n_{batt.0} := 2$ number of planes of lacings

effective critical force of built-up member: $N_{cr.CC} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_z^2}$ $N_{cr.CC} = 564\,kN$

$$S_v := \min\left[\left[\frac{24 \cdot E \cdot I_{ch}}{cc_{batt}^2 \cdot \left(1 + \frac{2 \cdot I_{ch}}{n_{batt.0} \cdot I_b} \cdot \frac{h_0}{cc_{batt}}\right)}\right], \left(\frac{2 \cdot \pi^2 \cdot E \cdot I_{ch}}{cc_{batt}^2}\right)\right] \qquad S_v = 1357\,kN$$

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

moment z-z without second order effects $M_{z.Ed.I} := 0 \cdot kN \cdot m$

moment with second order effects

$$M_{z.Ed.C} := \begin{cases} \left|\frac{N_{Ed} \cdot e_0 + M_{z.Ed.I}}{1 - \frac{N_{Ed} \cdot \gamma_{M1}}{N_{cr.CC}} - \frac{N_{Ed} \cdot \gamma_{M1}}{S_v}}\right| & \text{if built\_up = "YES"} \\ (0 \cdot kN \cdot m) & \text{if built\_up = "NO"} \end{cases} \qquad M_{z.Ed.C} = 3.2\,kN \cdot m$$

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:

$$X_d := \begin{cases} 1 & \text{if } d \geq 8 \cdot mm \\ 0 & \text{otherwise} \end{cases} \qquad N_{ch.Ed} := \frac{N_{Ed}}{r_{CC}} + \frac{|M_{z.Ed.C}| \cdot h_0 \cdot A_{ch} \cdot X_d}{2 \cdot I_{eff}} \qquad N_{ch.Ed} = 96.17\,kN$$

Interaktion formulae 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

$C_{my} = 0.4$ $C_{mLT} := C_{my}$ $n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}}$ $n_{y.2} = 0.32$ $n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}}$ $n_{z.2} = 0.5$ $C_{mz.2} := 0.95$

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

$k_{yy} := \min\left[C_{my} \cdot \left(1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}\right), C_{my} \cdot \left(1 + 0.6 \cdot n_{y.2}\right)\right]$ $k_{yy} = 0.42$

$k_{zy} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FBcc}}{C_{mLT} - 0.25} \cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT} - 0.25} \cdot n_{z.2}\right)$ $k_{zy} = 0.84$

$k_{zz.2} := \min\left[C_{mz.2} \cdot \left(1 + 0.6 \cdot \lambda_{r.z.FBcc} \cdot n_{z.2}\right), C_{mz.2} \cdot \left(1 + 0.6 \cdot n_{z.2}\right)\right]$ $k_{zz.2} = 1.22$ $k_{yz.2} := k_{zz.2}$

Reduction factor for L-T buckling: $\chi_{LT} = 1$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.98$ $\chi_{z.FBcc} = 0.63$

Combined bending an axial compression EN 1993-1-3: 6.2.5 (2):

$$\frac{N_{Ed} \cdot \gamma_{M0}}{2 \cdot N_{c.Rk}} + \frac{\max\left(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|\right) \cdot \gamma_{M0}}{2 \cdot M_{ycRk}} + \frac{|M_{z.Ed.C} \cdot \gamma_{M0}|}{M_{zz.cRk}} = 0.71 \qquad <1.0$$

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

Evading in y-y:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}} + k_{yy} \cdot \frac{\max\left(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{yz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.57 \qquad <1.0$$

Evading in z-z:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}} + k_{zy} \cdot \frac{\max\left(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{zz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.87 \qquad <1.0$$

If not designed as a built-up member (that means no PXK-plates for connecting), check only single C-profiles according to formulaes further down. built_up = "YES"

$b = 97\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 5\,mm$ $f_{yb} = 129\frac{N}{mm^2}$ $f_u = 141\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1) if the truss beam is designed as built-up member. Otherwise: EN 1993-1-1: 6.3.3

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

$$M_{y.i}(x_i) := \frac{M_{Ed.end.2} - M_{Ed.end.1}}{L}\cdot x_i + M_{Ed.end.1}$$

For moment distribution accord. to model "A"

Moment at quarter point of unbraced segment: $x_{1\_4} := L\cdot 0.25$ $M_{y.AA} := M_{y.i}(x_{1\_4})$ $M_{y.AA} = 2.58\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := L\cdot 0.5$ $M_{y.BA} := M_{y.i}(x_{1\_2})$ $M_{y.BA} = -1.05\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := L\cdot 0.75$ $M_{y.CA} := M_{y.i}(x_{3\_4})$ $M_{y.CA} = -4.67\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3\cdot M_{y.AA} + 4\cdot M_{y.BA} + 3\cdot M_{y.CA}}$$

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

$z_{0.cr} := 0\cdot mm$ (coord. shear center)

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

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

$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot L_{z.C}}{i_z}\right)^2}$ $\sigma_{ez} = 1332.96\frac{N}{mm^2}$ $\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{L_{T.C}^2}\right)$ $\sigma_T = 1530.27\frac{N}{mm^2}$

Elastic critical moment for singly symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_A} := |C_{bA}|\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 3409.5\,kN\cdot m$

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

$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right]$ $\phi_{LT.C\_A} = 0.48$

$\chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right)$ $\chi_{LT.C\_A} = 1$

For moment distribution accord. to model "B"

Moment at quarter point of unbraced segment: $M_{y.AB} := \frac{(M_{Ed.span} - M_{Ed.end.1})\cdot L\cdot 0.25}{L\cdot 0.5} + M_{Ed.end.1}$ $M_{y.AB} = 3.1\,kN\cdot m$

Moment at centerline of unbraced segment: $M_{y.BB} := M_{Ed.span}$ $M_{y.BB} = 0\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $M_{y.CB} := \frac{(M_{Ed.span} - M_{Ed.end.2})\cdot L\cdot 0.25}{L\cdot 0.5} + M_{Ed.end.2}$ $M_{y.CB} = -4.15\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := |C_{bB}|\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 1985.7\,kN\cdot m$

Relative slenderness: $\lambda_{rLT.C\_B} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0\\ 0.2 & \text{if } M_{e.cr.C\_B} = 0\end{cases}$ $\lambda_{rLT.C\_B} = 0.09$

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

$\phi_{LT.C\_B} = 0.48$

Moment distribution: $\chi_{LT.C} := \begin{cases}\chi_{LT.C\_A} & \text{if Moment\_dis = "A"}\\ \chi_{LT.C\_B} & \text{if Moment\_dis = "B"}\end{cases}$ $\chi_{LT.C} = 1$

Chord at end panel: highest shear force: $V_{Ed.max} := \pi\cdot\frac{M_{z.Ed.C}}{L_z}$ $V_{Ed.max} = 1.7\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 0.85\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.012$ << 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:* $M_{z.ch} = 0.64\,kN\cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C}\cdot\sin\left(\frac{\pi\cdot cc_{batt}}{L_z}\right)$ $M_{z.Ed.C.end} = 2.29\,kN\cdot m$ moment due to bow imperfection att end panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end}\cdot\frac{(h_0\cdot A_{ch})}{(2\cdot I_{eff})}$ $N_{ch.end} = 91.58\,kN$

$b = 97\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 5\,mm$ $f_{yb} = 129\dfrac{N}{mm^2}$ $f_u = 141\dfrac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

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

$n_{y.C} := \dfrac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}}$ $n_{y.C} = 0.37$

$n_{z.C} := \dfrac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}}$ $n_{z.C} = 0.37$

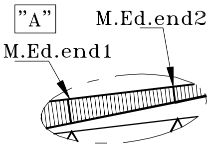
$\chi_{y.FB}(L_y) = 0.96$

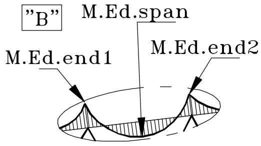
$\chi_{z.FB}(L_{z.C}) = 0.96$

Equivalent uniform moment factors:

$C_{my} = 0.4$ $C_{mLT} = 0.4$ $\psi_{m.z} := 1$

$C_{mz.C} := \max[(0.6 + 0.4 \cdot \psi_{m.z}), 0.4]$ $C_{mz.C} = 1$





$M_{Ed.end.1} = 6.2\,kN \cdot m$

$M_{Ed.end.2} = -8.3\,kN \cdot m$

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

$\lambda_{r.y.FB}(L_y) = 0.3$ $k_{yy.C} := \min[C_{my} \cdot (1 + 0.6 \cdot \lambda_{r.y.FB}(L_y) \cdot n_{y.C}), C_{my} \cdot (1 + 0.6 \cdot n_{y.C})]$ $k_{yy.C} = 0.43$

$\lambda_{r.z.FB}(L_{z.C}) = 0.31$ $k_{zz.C} := \min[C_{mz.C} \cdot (1 + 0.6 \cdot \lambda_{r.z.FB}(L_{z.C}) \cdot n_{z.C}), C_{mz.C} \cdot (1 + 0.6 \cdot n_{z.C})]$ $k_{zz.C} = 1.07$

$k_{zy.C} := \max\left(1 - \dfrac{0.05 \cdot \lambda_{r.z.FB}(L_{z.C})}{C_{my} - 0.25} \cdot n_{z.C}, 1 - \dfrac{0.05}{C_{mLT} - 0.25} \cdot n_{z.C}\right)$ $k_{zy.C} = 0.96$ $k_{yz.C} := k_{zz.C}$

Controll single C-profile between battens:

max moment y-y: $M_{y.max.C} := 0.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)$ $M_{y.max.C} = 4.15\,kN \cdot m$

max moment z-z: $M_{z.C} = 0.64\,kN \cdot m$ $\Delta M_{z.shift} := |\Delta e_N \cdot N_{ch.end}|$ $\Delta M_{z.shift} = 0.06\,kN \cdot m$

max chord force from above: $N_{Ed.C} = 91.58\,kN$

Buckling moment resistance y-y
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C} \cdot M_{ycRk}$ $\chi_{LT.C} = 1$ $M_{y.Rk.C} = 14.35\,kN \cdot m$

Buckling resistance moment z-z
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk})$ $M_{z.Rk.C} = 6.31\,kN \cdot m$

Resistance for one profile buckling mode flexural buckling-: y-y $L_y = 1.35\,m$ $\chi_{y.FB}(L_y) = 0.96$ $N_{b.Rk.y.FB}(L_y) = 248.15\,kN$

Resistance for one profile buckling mode flexural buckling-: z-z $L_{z.C} = 0.75\,m$ $\chi_{z.FB}(L_{z.C}) = 0.96$ $N_{b.Rk.z.FB}(L_{z.C}) = 247.81\,kN$

Resistance for one profile buckling mode -torsional or torsional-flexural $L_{T.C} = 0.52\,m$ $\chi_{TF}(L_{T.C}, L_y) = 0.93$ $N_{b.Rk.TF}(L_{T.C}, L_y) = 239.6\,kN$

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

At mid-span of built-up member: $\dfrac{N_{ch.Ed} \cdot \gamma_{M0}}{N_{c.Rk}} + \dfrac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \dfrac{|\Delta M_{z.shift}| \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.67$ <1.0

In end panel of built-up member: $\dfrac{N_{ch.end} \cdot \gamma_{M0}}{N_{c.Rk}} + \dfrac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \dfrac{(M_{z.C} + |\Delta M_{z.shift}|) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.75$ <1.0

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

$\dfrac{\max(N_{ch.Ed}, N_{ch.end}) \cdot \gamma_{M1}}{\min(\chi_{y.FB}(L_y), \chi_{z.FB}(L_{z.C}), \chi_{TF}(L_{T.C}, L_y)) \cdot N_{c.Rk}} = 0.4$ <1.0

Combined bending + axial compression EN 1993-1-1: 6.3.3 (4): Buckling about y-y

At mid-span of built-up member: $\dfrac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}} + k_{yy.C} \cdot \dfrac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C} \cdot \dfrac{|\Delta M_{z.shift}| \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.52$ <1.0

In end panel of built-up member: $\dfrac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}} + k_{yy.C} \cdot \dfrac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C} \cdot \dfrac{(M_{z.C} + |\Delta M_{z.shift}|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.61$ <1.0

Combined bending + axial compression EN 1993-1-1: 6.3.3 (4): Buckling about z-z

At mid-span of built-up member: $\dfrac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}} + k_{zy.C} \cdot \dfrac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C} \cdot \dfrac{|\Delta M_{z.shift}| \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.68$ <1.0

In end panel of built-up member: $\dfrac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}} + k_{zy.C} \cdot \dfrac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C} \cdot \dfrac{(M_{z.C} + |\Delta M_{z.shift}|) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.77$ <1.0

Check M, V och N acc. to EC

Country := "Cz"

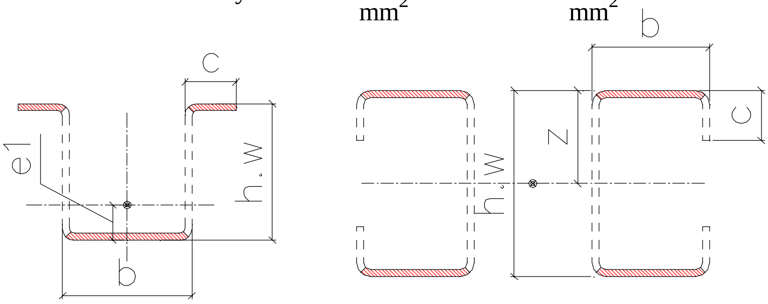
PR ≡ "CC170x5"

Profile data for a single profile

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

$h_w = 170\,mm$ $b_c = 97\,mm$ $c = 38\,mm$ $t = 5\,mm$ $e_1 = 37.88\,mm$ $A_g = 2035.62\,mm^2$

$N_{cRk} = 955.64\,kN$ $M_{Rk} = 55.66\,kN \cdot m$ $f_{yb} = 128.5\,\frac{N}{mm^2}$ $f_u = 141.35\,\frac{N}{mm^2}$



The beam is H-profile ("H") or double C-profile ("C")

Hole diameter: $d_o := 12.5 \cdot mm$

Number of bolts at the cross-section in ONE web: $n_{cross} := 4$

Number screws and diameter i FLANGES: $X_{h.fl} := 0$ $d_{o.fl} := 0 \cdot mm$

Momentcap.one profile:

Axialforcecapacity one profile:

Axialforce acting as TENTION : "YES" or "NO" $N_{drag} :=$ "YES"

Netto area for shearforce:

$$A_w := \begin{cases} \left[\left[\left(h_w - t\right) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} = 0 \cdot mm \\ \left[\left[\left(L_{II} - t\right) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} \neq 0 \cdot mm \end{cases}$$

$A_w = 1150.00\,mm^2$

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: $M_{Rd} := \frac{X \cdot M_{Rk}}{\gamma_{M0}}$ $M_{Rd} = 111.32\,kN \cdot m$

Axialforce capacity comprension member: $N_{Rd.1} := \frac{N_{cRk} \cdot X}{\gamma_{M0}}$ (6.2.4) $N_{Rd.1} = 1911.3\,kN$

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)) $N_{Rd.t.a} = 201.92\,kN$

For material >= 5.0mm : $N_{Rd.t.b} := \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2.t}}$ $N_{Rd.t.b} = 181.7\,kN$ (EN1993-1-1: 6.2.3) $\gamma_{M2.t} = 1.25$

$$N_{Rd.t} := \left(\begin{cases} N_{Rd.t.a} & \text{if } t \le 4 \cdot mm \\ N_{Rd.t.b} & \text{if } t \ge 5 \cdot mm \end{cases}\right) \cdot X$$

$N_{Rd.t} = 363.45\,kN$

Normalkraftscapacity:

$$N_{Rd} := \begin{cases} N_{Rd.1} & \text{if } N_{drag} = \text{"NO"} \\ \min\left(N_{Rd.1}, N_{Rd.t}\right) & \text{if } N_{drag} = \text{"YES"} \end{cases}$$

$N_{Rd} = 363.45\,kN$

Shearforce capacity:

$\lambda_{wh} := 0.346 \cdot \frac{h_w - t}{t} \cdot \sqrt{\frac{f_{yb}}{E}}$

$\lambda_{wh} = 0.55$

$$f_{vb} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{wh} \le 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{wh}} & \text{if } 0.83 < \lambda_{wh} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{wh}^2} & \text{if } \lambda_{wh} \ge 1.40 \end{cases}$$

$f_{vb} = 74.19\,\frac{N}{mm^2}$

$V_{Rd} := \frac{A_w \cdot f_{vb}}{\gamma_{M0}}$

$V_{Rd} = 85.32\,kN$

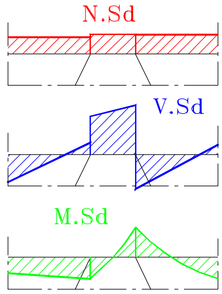
elast. momentcap.: $M_{elRd} := M_{Rd}$

$\theta_{pl} := 1.0$ faktor M_{el} -> M_{pl}

Section value for interaction formula M + N + V:
Momentresistance of a cross-section consisting only flanges: $M_{flRd} = 19.96\,kN \cdot m$

Element Nr: ....

$N_{Sd} := 350 \cdot kN$ $V_{Sd} := 40 \cdot kN$ $M_{Sd} := 4.5 \cdot kN \cdot m$



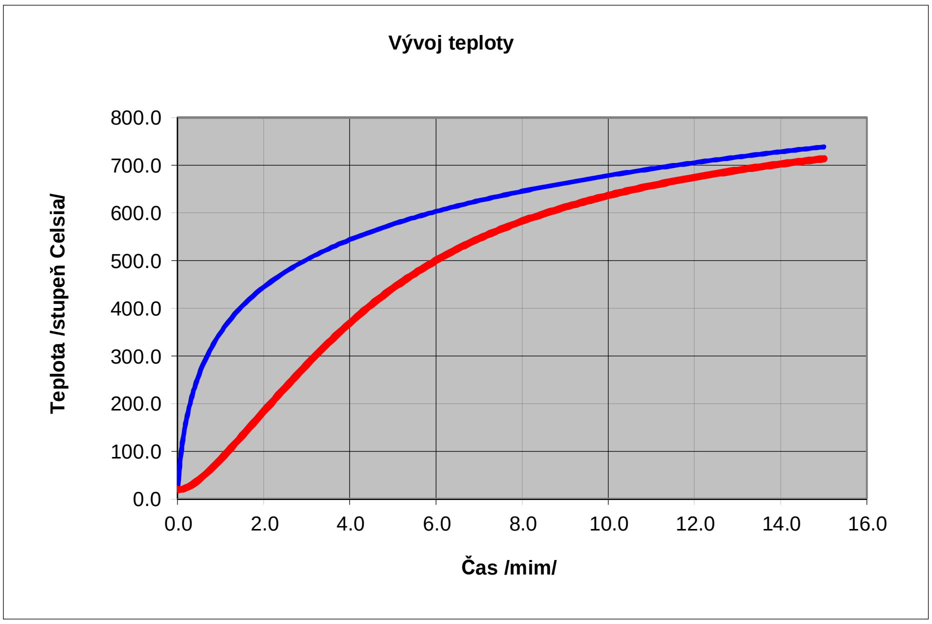
$\frac{|V_{Sd}|}{V_{Rd}} = 0.47$ <1.0 (EN 1993-1-1: 6.2.6)

$\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 1$ <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)

REDUKČNÍ SOUČINITELÉ PRO NÁVRH NA R15
DIAGONÁLA C100X2



Čas t=15 min.

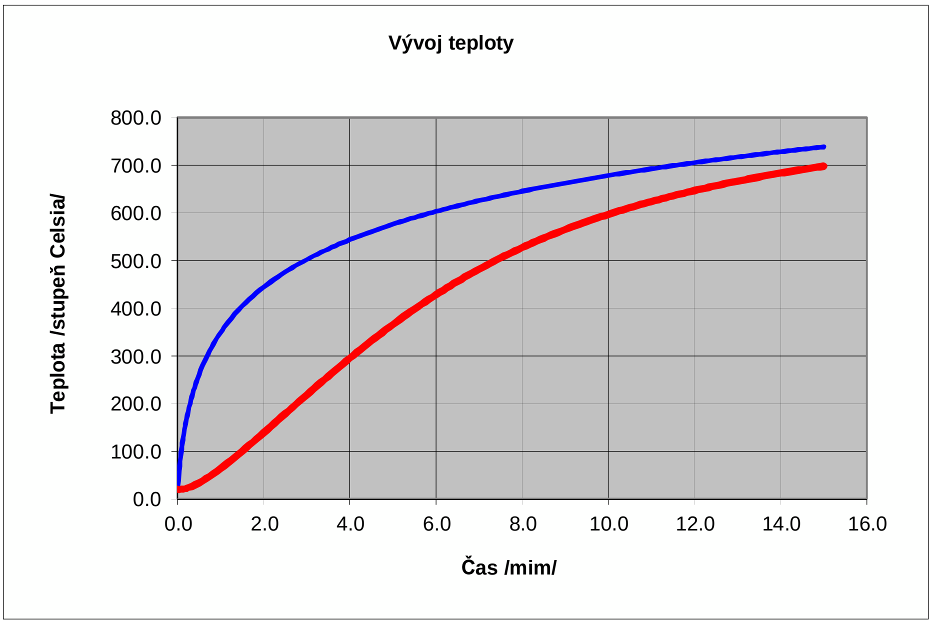
	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
500	0.78	0.36	0.6
600	0.47	0.18	0.31
700	0.23	0.075	0.13
800	0.11	0.05	0.09
Teplota nosníku			
713.68	**0.214**	**0.072**	**0.125**

	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
713.68	**0.122**	**0.125**	

$k_{y\Theta}$ red. součinitel na mez kluzu
$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu
$k_{E\Theta}$ red. součinitel pro linerní pružnou část

	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
713.68	**0.095**	**0.122**	**0.000**

REDUKČNÍ SOUČINITELÉ PRO NÁVRH NA R15
DIAGONÁLA C100X3



Čas t=15 min.

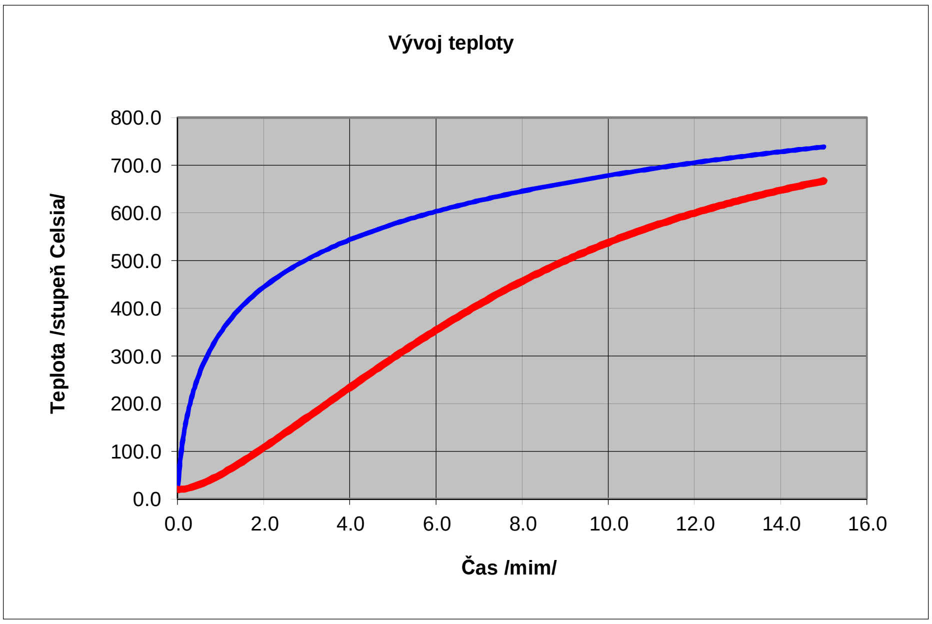
	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
500	0.78	0.36	0.6
600	0.47	0.18	0.31
700	0.23	0.075	0.13
800	0.11	0.05	0.09
Teplota nosníku			
697.93	**0.235**	**0.077**	**0.134**

	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
697.93	**0.134**	**0.134**	

$k_{y\Theta}$ red. součinitel na mez kluzu
$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu
$k_{E\Theta}$ red. součinitel pro linerní pružnou část

	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
697.93	**0.102**	**0.135**	**0.000**

REDUKČNÍ SOUČINITELÉ PRO NÁVRH NA R15
DIAGONÁLA C100X4



Čas t=15 min.

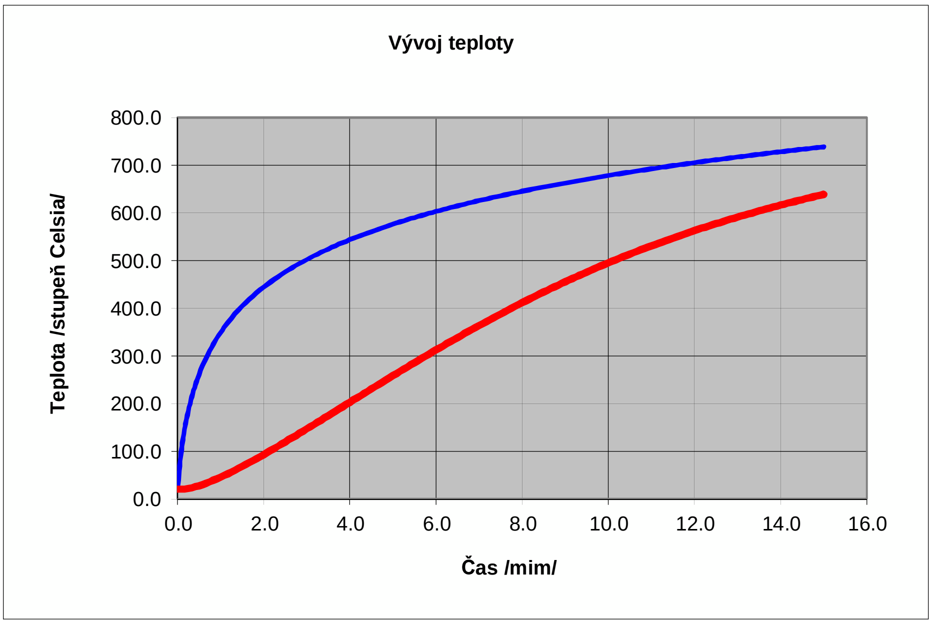
	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
500	0.78	0.36	0.6
600	0.47	0.18	0.31
700	0.23	0.075	0.13
800	0.11	0.05	0.09
Teplota nosníku			
667.16	**0.309**	**0.109**	**0.189**

	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
667.16	**0.186**	**0.189**	

$k_{y\Theta}$ red. součinitel na mez kluzu
$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu
$k_{E\Theta}$ red. součinitel pro linerní pružnou část

	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
667.16	**0.139**	**0.211**	**0.000**

REDUKČNÍ SOUČINITELÉ PRO NÁVRH NA R15
DIAGONÁLA C100X5



Čas t=15 min.

	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
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			
639.02	**0.376**	**0.139**	**0.240**

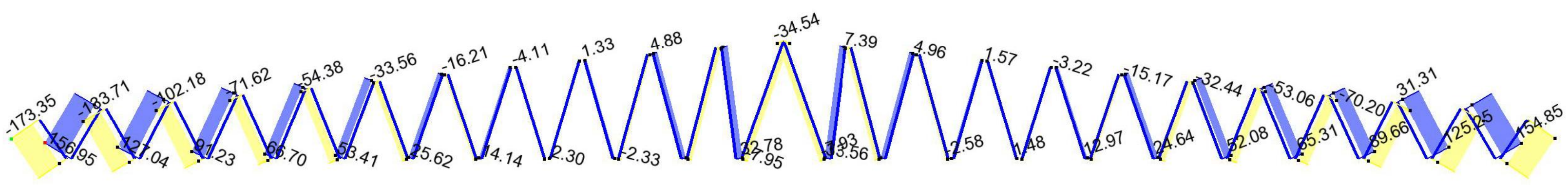
	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
639.02	**0.234**	**0.240**	

$k_{y\Theta}$ red. součinitel na mez kluzu
$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu
$k_{E\Theta}$ red. součinitel pro linerní pružnou část

	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
639.02	**0.173**	**0.281**	**0.000**

View:1 - FX; Cases: 10to12 20to23 50

DIAGONÁLY VAZNÍKU - NORMÁLOVÉ SÍLY



Fx+c Fx-t 200kN
Max=156.95
Min=-173.35

Cases: 10to12 20to23 50

Vazník - diagonály

Materiál:

Prvek	Typ	Member type	Profil	Délka [m]	Materiál	Hmotnost [kg]	A [m²]	A_eff [m²]	Komb. N_Ed
1	Strut	Strut 2 screw fire	C 100x85x5	2.756	HX500LAD	11.45	1580	1580	11
2	Strut	Strut 2 screw fire	C 100x85x5	2.756	HX500LAD	11.45	1580	1580	12
8	Strut	Strut 2 screw fire	C 100x85x5	2.48	HX500LAD	11.45	1580	1580	12
9	Strut	Strut 2 screw fire	C 100x85x5	2.48	HX500LAD	11.45	1580	1580	11
14	Strut	Strut 2 screw fire	C 100x82x4	2.529	HX420LAD	8.99	1166.8	1166.8	12
15	Strut	Strut 2 screw fire	C 100x82x4	2.529	HX420LAD	8.99	1166.8	1166.8	11
20	Strut	Strut 2 screw fire	C 100x82x4	2.392	HX420LAD	8.99	1166.8	1166.8	12
21	Strut	Strut 2 screw fire	C 100x82x4	2.392	HX420LAD	8.99	1166.8	1166.8	11
26	Strut	Strut 2 screw fire	C 100x82x4	2.384	HX420LAD	8.99	1166.8	1166.8	12
27	Strut	Strut 2 screw fire	C 100x82x4	2.384	HX420LAD	8.99	1166.8	1166.8	11
32	Strut	Strut 2 screw fire	C 100x77x3	2.247	S350GD	6.21	840	840	12
33	Strut	Strut 2 screw fire	C 100x77x3	2.247	S350GD	6.21	840	840	11
38	Strut	Strut 2 screw fire	C 100x75x2	2.24	S350GD	4.14	552	472.35	11
39	Strut	Strut 2 screw fire	C 100x75x2	2.24	S350GD	4.14	552	472.35	12
44	Strut	Strut 2 screw fire	C 100x75x2	2.104	S350GD	4.14	552	552	11
45	Strut	Strut 2 screw fire	C 100x75x2	2.104	S350GD	4.14	552	552	22
50	Strut	Strut 2 screw fire	C 100x77x3	2.097	S350GD	6.21	840	840	11
51	Strut	Strut 2 screw fire	C 100x77x3	2.097	S350GD	6.21	840	840	12
56	Strut	Strut 2 screw fire	C 100x82x4	1.961	HX420LAD	8.99	1166.8	1166.8	11
57	Strut	Strut 2 screw fire	C 100x82x4	1.961	HX420LAD	8.99	1166.8	1166.8	12
62	Strut	Strut 2 screw fire	C 100x82x4	1.955	HX420LAD	8.99	1166.8	1166.8	10
63	Strut	Strut 2 screw fire	C 100x82x4	1.955	HX420LAD	8.99	1166.8	1166.8	10
68	Strut	Strut 2 screw fire	C 100x85x5	1.82	HX500LAD	11.45	1580	1580	10
69	Strut	Strut 2 screw fire	C 100x85x5	1.82	HX500LAD	11.45	1580	1580	10
74	Strut	Strut 2 screw fire	C 100x85x5	1.814	HX500LAD	11.45	1580	1580	10
75	Strut	Strut 2 screw fire	C 100x85x5	1.814	HX500LAD	11.45	1580	1580	10
80	Strut	Strut 2 screw fire	C 100x85x5	1.681	HX500LAD	11.45	1580	1580	10
81	Strut	Strut 2 screw fire	C 100x85x5	1.681	HX500LAD	11.45	1580	1580	10
86	Strut	Strut 2 screw fire	C 100x85x5	1.675	HX500LAD	11.45	1580	1580	10
87	Strut	Strut 2 screw fire	C 100x85x5	1.675	HX500LAD	11.45	1580	1580	10
92	Strut	Strut 2 screw fire	C 100x85x5	1.543	HX500LAD	11.45	1580	1580	10
93	Strut	Strut 2 screw fire	C 100x85x5	1.543	HX500LAD	11.45	1580	1580	10
98	Strut	Strut 2 screw fire	C 100x85x5	1.539	HX500LAD	11.45	1580	1580	10
99	Strut	Strut 2 screw fire	C 100x85x5	1.539	HX500LAD	11.45	1580	1580	10
104	Strut	Strut 2 screw fire	C 100x85x5	1.408	HX500LAD	11.45	1580	1580	10
105	Strut	Strut 2 screw fire	C 100x85x5	1.408	HX500LAD	11.45	1580	1580	10
110	Strut	Strut 2 screw fire	C 100x85x5	1.405	HX500LAD	11.45	1580	1580	10
111	Strut	Strut 2 screw fire	C 100x85x5	1.405	HX500LAD	11.45	1580	1580	10
116	Strut	Strut 2 screw fire	C 100x85x5	1.277	HX500LAD	11.45	1580	1580	10
117	Strut	Strut 2 screw fire	C 100x85x5	1.277	HX500LAD	11.45	1580	1580	10
122	Strut	Strut 2 screw fire	C 100x85x5	1.275	HX500LAD	11.45	1580	1580	10
123	Strut	Strut 2 screw fire	C 100x85x5	1.275	HX500LAD	11.45	1580	1580	10
128	Strut	Strut 2 screw fire	C 100x85x5	1.048	HX500LAD	11.45	1580	1580	10
129	Strut	Strut 2 screw fire	C 100x85x5	1.048	HX500LAD	11.45	1580	1580	10

Vnitřní síly:

Prvek	Typ	Profil	N_Ed [kN]	N_t,Ed [kN]	N_c,Ed [kN]	M_z,Ed [kNm]	V_y,Ed [kN]	dM_z,Ed [kNm]	Komb. N_Ed
1	Strut	C 100x85x5	-34.38	-34.54	-7.93	0.04	0	0	11
2	Strut	C 100x85x5	-33.67	-33.83	-7.95	0.04	0	0	12
8	Strut	C 100x85x5	32.78	7.4	32.78	0	-0.03	0	12
9	Strut	C 100x85x5	33.56	7.39	33.56	0	-0.03	0	11
14	Strut	C 100x82x4	-26.58	-26.69	-5.87	0.02	0.01	0	12
15	Strut	C 100x82x4	-27.45	-27.56	-5.94	0.02	0.01	0	11
20	Strut	C 100x82x4	23.47	4.88	23.47	0	-0.05	0	12
21	Strut	C 100x82x4	24.32	4.96	24.32	0	-0.05	0	11
26	Strut	C 100x82x4	-13.96	-14.06	-2.33	0.02	0.01	0	12
27	Strut	C 100x82x4	-14.91	-15.01	-2.58	0.02	0.01	0	11
32	Strut	C 100x77x3	10.32	1.33	10.32	0	-0.03	0	12
33	Strut	C 100x77x3	11.2	1.57	11.2	0	-0.03	0	11
38	Strut	C 100x75x2	2.3	-1.1	2.3	0	0.01	0.01	11
39	Strut	C 100x75x2	1.48	-1.91	1.48	0	0.01	0	12
44	Strut	C 100x75x2	-4.06	-4.11	-0.49	-0.01	0.01	0	11
45	Strut	C 100x75x2	-3.17	-3.22	0.44	-0.01	0	0	22
50	Strut	C 100x77x3	14.14	3.35	14.14	0	-0.01	0	11
51	Strut	C 100x77x3	12.97	2.92	12.97	0	-0.01	0	12
56	Strut	C 100x82x4	-16.13	-16.21	-4.02	-0.01	0.04	0	11
57	Strut	C 100x82x4	-15.09	-15.17	-3.66	-0.01	0.04	0	12
62	Strut	C 100x82x4	25.62	6.4	25.62	0	-0.04	0	10
63	Strut	C 100x82x4	24.64	6.25	24.64	0	-0.04	0	10
68	Strut	C 100x85x5	-33.45	-33.56	-8.58	-0.02	0.1	0	10
69	Strut	C 100x85x5	-32.34	-32.44	-8.43	-0.02	0.1	0	10
74	Strut	C 100x85x5	53.41	13.58	53.41	0	-0.14	0	10
75	Strut	C 100x85x5	52.08	13.59	52.08	0	-0.14	0	10
80	Strut	C 100x85x5	-54.29	-54.38	-14.02	-0.02	0.2	0	10
81	Strut	C 100x85x5	-52.97	-53.06	-14.03	-0.02	0.2	0	10
86	Strut	C 100x85x5	66.7	17.04	66.7	0	-0.23	0	10
87	Strut	C 100x85x5	65.31	17.2	65.31	0	-0.22	0	10
92	Strut	C 100x85x5	-71.54	-71.62	-18.39	-0.01	0.3	0	10
93	Strut	C 100x85x5	-70.12	-70.2	-18.58	-0.01	0.3	0	10
98	Strut	C 100x85x5	91.23	23.25	91.23	0	-0.38	0	10
99	Strut	C 100x85x5	89.66	23.63	89.66	0	-0.37	0	10
104	Strut	C 100x85x5	-102.1	-102.18	-24.76	-0.01	0.48	0	10
105	Strut	C 100x85x5	-100.48	-100.56	-25.15	-0.01	0.48	0	10
110	Strut	C 100x85x5	127.04	30.79	127.04	0	-0.59	0	10
111	Strut	C 100x85x5	125.25	31.31	125.25	0	-0.59	0	10
116	Strut	C 100x85x5	-133.64	-133.71	-32.47	-0.01	0.67	0	10
117	Strut	C 100x85x5	-131.82	-131.89	-33	-0.01	0.67	0	10
122	Strut	C 100x85x5	156.95	38.09	156.95	0	-0.76	0	10
123	Strut	C 100x85x5	154.85	38.28	154.85	0	-0.76	0	10
128	Strut	C 100x85x5	-173.3	-173.35	-42.16	-0.01	0.85	0	10
129	Strut	C 100x85x5	-170.81	-170.86	-42.27	-0.01	0.86	0	10

Vzpěr:

Prvek	Typ	Profil	X_y [-]	N_y,b,Rd [kN]		N_z,b,Rd [kN]	X_t [-]	N_b,t,Rd [kN]	X_tf [-]	N_b,tf,Rd [kN]
1	Strut	C 100x85x5	0.37	114.59	0.31	98.15	1	0	1	0
2	Strut	C 100x85x5	0.37	114.59	0.31	98.15	1	0	1	0
8	Strut	C 100x85x5	0.42	131.74	0.36	114.25	0.29	90.39	0.21	66.37
9	Strut	C 100x85x5	0.42	131.74	0.36	114.25	0.29	90.39	0.21	66.37
14	Strut	C 100x82x4	0.43	63.92	0.35	51.55	1	0	1	0
15	Strut	C 100x82x4	0.43	63.92	0.35	51.55	1	0	1	0
20	Strut	C 100x82x4	0.46	68.14	0.38	55.58	0.29	42.04	0.22	32.79
21	Strut	C 100x82x4	0.46	68.14	0.38	55.58	0.29	42.04	0.22	32.79
26	Strut	C 100x82x4	0.46	68.39	0.38	55.81	1	0	1	0
27	Strut	C 100x82x4	0.46	68.39	0.38	55.81	1	0	1	0
32	Strut	C 100x77x3	0.52	36.86	0.39	27.67	0.27	19.02	0.23	16.11
33	Strut	C 100x77x3	0.52	36.86	0.39	27.67	0.27	19.02	0.23	16.11
38	Strut	C 100x75x2	0.68	13.75	0.56	11.35	0.42	8.53	0.38	7.58
39	Strut	C 100x75x2	0.68	13.75	0.56	11.35	0.42	8.53	0.38	7.58
44	Strut	C 100x75x2	0.56	23.31	0.42	17.33	1	0	1	0
45	Strut	C 100x75x2	0.56	23.31	0.42	17.33	1	0	1	0
50	Strut	C 100x77x3	0.56	39.14	0.43	30.07	0.3	20.79	0.25	17.75
51	Strut	C 100x77x3	0.56	39.14	0.43	30.07	0.3	20.79	0.25	17.75
56	Strut	C 100x82x4	0.56	82.74	0.48	70.5	1	0	1	0
57	Strut	C 100x82x4	0.56	82.74	0.48	70.5	1	0	1	0
62	Strut	C 100x82x4	0.56	82.97	0.48	70.74	0.36	53.63	0.29	43.38
63	Strut	C 100x82x4	0.56	82.97	0.48	70.74	0.36	53.63	0.29	43.38
68	Strut	C 100x85x5	0.58	181.97	0.53	165.11	1	0	1	0
69	Strut	C 100x85x5	0.58	181.97	0.53	165.11	1	0	1	0
74	Strut	C 100x85x5	0.58	182.45	0.53	165.62	0.41	129.96	0.33	102.44
75	Strut	C 100x85x5	0.58	182.45	0.53	165.62	0.41	129.96	0.33	102.44
80	Strut	C 100x85x5	0.62	193.6	0.57	177.72	1	0	1	0
81	Strut	C 100x85x5	0.62	193.6	0.57	177.72	1	0	1	0
86	Strut	C 100x85x5	0.62	194.02	0.57	178.18	0.45	141.52	0.36	113.64
87	Strut	C 100x85x5	0.62	194.02	0.57	178.18	0.45	141.52	0.36	113.64
92	Strut	C 100x85x5	0.65	205.05	0.61	190.44	1	0	1	0
93	Strut	C 100x85x5	0.65	205.05	0.61	190.44	1	0	1	0
98	Strut	C 100x85x5	0.66	205.39	0.61	190.82	0.49	154.14	0.4	126.32
99	Strut	C 100x85x5	0.66	205.39	0.61	190.82	0.49	154.14	0.4	126.32
104	Strut	C 100x85x5	0.69	216.11	0.65	202.96	1	0	1	0
105	Strut	C 100x85x5	0.69	216.11	0.65	202.96	1	0	1	0
110	Strut	C 100x85x5	0.69	216.36	0.65	203.23	0.53	167.6	0.45	140.41
111	Strut	C 100x85x5	0.69	216.36	0.65	203.23	0.53	167.6	0.45	140.41
116	Strut	C 100x85x5	0.72	226.61	0.69	214.98	1	0	1	0
117	Strut	C 100x85x5	0.72	226.61	0.69	214.98	1	0	1	0
122	Strut	C 100x85x5	0.72	226.74	0.69	215.13	0.58	181.54	0.5	155.66
123	Strut	C 100x85x5	0.72	226.74	0.69	215.13	0.58	181.54	0.5	155.66
128	Strut	C 100x85x5	0.78	244.04	0.75	235.11	1	0	1	0
129	Strut	C 100x85x5	0.78	244.04	0.75	235.11	1	0	1	0

Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek	
Datum:	23.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka	
					Norma projektu:	ČSN EN

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	11	11	10	0	0	0	0	0	11	Prostý tah	11
2	C 100x85x5	0	10	11	10	0	0	0	0	0	11	Tah a ohyb	12
8	C 100x85x5	10	0	0	10	0	33	49	0	0	49	Vzpěr	12
9	C 100x85x5	11	0	0	11	0	33	51	0	0	51	Vzpěr	11
14	C 100x82x4	0	17	18	17	0	0	0	0	0	18	Tah a ohyb	12
15	C 100x82x4	0	18	18	17	0	0	0	0	0	18	Prostý tah	11
20	C 100x82x4	16	0	0	16	0	34	72	0	0	72	Vzpěr	12
21	C 100x82x4	16	0	0	16	0	34	74	0	0	74	Vzpěr	11
26	C 100x82x4	0	9	10	9	0	0	0	0	0	10	Tah a ohyb	12
27	C 100x82x4	0	10	10	9	0	0	0	0	0	10	Prostý tah	11
32	C 100x77x3	15	0	0	15	0	34	64	0	0	64	Vzpěr	12
33	C 100x77x3	16	0	0	16	0	34	70	0	0	70	Vzpěr	11
38	C 100x75x2	11	0	12	25	0	34	30	22	22	30	Štíhlost	11
39	C 100x75x2	7	0	8	7	0	34	20	14	14	20	Štíhlost	12
44	C 100x75x2	0	9	10	8	0	0	0	0	0	10	Tah a ohyb	11
45	C 100x75x2	0	7	8	6	0	0	0	0	0	8	Tah a ohyb	22
50	C 100x77x3	20	0	0	20	0	31	80	0	0	80	Vzpěr	11
51	C 100x77x3	18	0	0	18	0	31	73	0	0	73	Vzpěr	12
56	C 100x82x4	0	10	11	10	0	0	0	0	0	11	Tah a ohyb	11
57	C 100x82x4	0	10	10	9	0	0	0	0	0	10	Prostý tah	12
62	C 100x82x4	17	0	0	17	0	27	59	0	0	59	Vzpěr	10
63	C 100x82x4	17	0	0	17	0	27	57	0	0	57	Vzpěr	10
68	C 100x85x5	0	10	10	10	0	0	0	0	0	10	Prostý tah	10
69	C 100x85x5	0	10	10	10	0	0	0	0	0	10	Prostý tah	10
74	C 100x85x5	17	0	0	17	0	24	52	0	0	52	Vzpěr	10
75	C 100x85x5	16	0	0	16	0	24	51	0	0	51	Vzpěr	10
80	C 100x85x5	0	17	17	16	0	0	0	0	0	17	Prostý tah	10
81	C 100x85x5	0	16	16	16	0	0	0	0	0	16	Prostý tah	10
86	C 100x85x5	21	0	0	21	0	22	59	0	0	59	Vzpěr	10
87	C 100x85x5	21	0	0	21	0	22	57	0	0	57	Vzpěr	10
92	C 100x85x5	0	22	22	22	0	0	0	0	0	22	Prostý tah	10
93	C 100x85x5	0	21	22	21	0	0	0	0	0	22	Tah a ohyb	10
98	C 100x85x5	29	0	0	29	0	20	72	0	0	72	Vzpěr	10
99	C 100x85x5	28	0	0	28	0	20	71	0	0	71	Vzpěr	10
104	C 100x85x5	0	31	31	31	1	0	0	0	0	31	Prostý tah	10
105	C 100x85x5	0	31	31	31	1	0	0	0	0	31	Prostý tah	10
110	C 100x85x5	40	0	0	40	1	19	90	0	0	90	Vzpěr	10
111	C 100x85x5	39	0	0	39	1	19	89	0	0	89	Vzpěr	10
116	C 100x85x5	0	41	41	41	1	0	0	0	0	41	Prostý tah	10
117	C 100x85x5	0	40	40	40	1	0	0	0	0	40	Prostý tah	10
122	C 100x85x5	49	0	0	49	1	17	101	0	0	101	Vzpěr	10
123	C 100x85x5	49	0	0	49	1	17	99	0	0	99	Vzpěr	10
128	C 100x85x5	0	53	53	53	1	0	0	0	0	53	Prostý tah	10
129	C 100x85x5	0	52	52	52	1	0	0	0	0	52	Prostý tah	10

2 krajní diagonály vazníku budou z profilu C40/132/100/132/40x6

Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek	
Datum:	23.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka	
					Norma projektu:	ČSN EN

Spoje:

Prvek	Komb_N_t,Ed	Komb_N_c,Ed	Profil	N_t,Ed [kN]	N_c,Ed [kN]	Spoj	F_v [%]	F_b [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
1	11	50	C 100x85x5	-34.54	-7.93	C56	13	11	7	10	13	Shear
2	12	21	C 100x85x5	-33.83	-7.95	C56	13	11	7	10	13	Shear
8	21	12	C 100x85x5	7.4	32.78	C56	13	10	1	2	13	Shear
9	50	11	C 100x85x5	7.39	33.56	C56	13	11	1	2	13	Shear
14	12	21	C 100x82x4	-26.69	-5.87	C46	10	12	8	12	12	Bearing
15	11	50	C 100x82x4	-27.56	-5.94	C46	11	13	8	12	13	Bearing
20	21	12	C 100x82x4	4.88	23.47	C46	9	11	1	2	11	Bearing
21	50	11	C 100x82x4	4.96	24.32	C46	9	11	1	2	11	Bearing
26	12	21	C 100x82x4	-14.06	-2.33	C46	5	6	4	6	6	Bearing
27	11	50	C 100x82x4	-15.01	-2.58	C46	6	7	4	7	7	Bearing
32	21	12	C 100x77x3	1.33	10.32	C36	4	7	1	1	7	Bearing
33	50	11	C 100x77x3	1.57	11.2	C36	4	8	1	1	8	Bearing
38	22	11	C 100x75x2	-1.1	2.3	C26	1	2	1	1	2	Bearing
39	23	12	C 100x75x2	-1.91	1.48	C26	1	2	1	2	2	Bearing
44	11	22	C 100x75x2	-4.11	-0.49	C26	2	4	3	5	5	Block tear.
45	22	23	C 100x75x2	-3.22	0.44	C26	1	3	2	4	4	Block tear.
50	50	11	C 100x77x3	3.35	14.14	C36	5	10	2	2	10	Bearing
51	21	12	C 100x77x3	2.92	12.97	C36	5	9	1	2	9	Bearing
56	11	50	C 100x82x4	-16.21	-4.02	C46	6	7	5	7	7	Bearing
57	12	21	C 100x82x4	-15.17	-3.66	C46	6	7	5	7	7	Bearing
62	50	10	C 100x82x4	6.4	25.62	C46	10	12	2	3	12	Bearing
63	21	10	C 100x82x4	6.25	24.64	C46	9	11	2	3	11	Bearing
68	10	50	C 100x85x5	-33.56	-8.58	C56	13	11	7	10	13	Shear
69	10	21	C 100x85x5	-32.44	-8.43	C56	12	10	6	10	12	Shear
74	50	10	C 100x85x5	13.58	53.41	C58	15	13	3	4	15	Shear
75	21	10	C 100x85x5	13.59	52.08	C58	15	12	3	4	15	Shear
80	10	50	C 100x85x5	-54.38	-14.02	C58	16	13	11	17	17	Block tear.
81	10	21	C 100x85x5	-53.06	-14.03	C58	15	13	11	17	17	Block tear.
86	50	10	C 100x85x5	17.04	66.7	C58	19	16	3	5	19	Shear
87	21	10	C 100x85x5	17.2	65.31	C58	19	16	3	5	19	Shear
92	10	50	C 100x85x5	-71.62	-18.39	C58	21	17	14	23	23	Block tear.
93	10	21	C 100x85x5	-70.2	-18.58	C58	20	17	14	22	22	Block tear.
98	50	10	C 100x85x5	23.25	91.23	C58	26	22	5	7	26	Shear
99	21	10	C 100x85x5	23.63	89.66	C58	26	21	5	8	26	Shear
104	10	50	C 100x85x5	-102.18	-24.76	C58	29	24	20	33	33	Block tear.
105	10	21	C 100x85x5	-100.56	-25.15	C58	29	24	20	32	32	Block tear.
110	50	10	C 100x85x5	30.79	127.04	C58	37	30	6	10	37	Shear
111	50	10	C 100x85x5	31.31	125.25	C58	36	30	6	10	36	Shear
116	10	50	C 100x85x5	-133.71	-32.47	C58	39	32	27	43	43	Block tear.
117	10	50	C 100x85x5	-131.89	-33	C58	38	31	26	42	42	Block tear.
122	50	10	C 100x85x5	38.09	156.95	C58	45	37	8	12	45	Shear
123	21	10	C 100x85x5	38.28	154.85	C58	45	37	8	12	45	Shear
128	10	50	C 100x85x5	-173.35	-42.16	C58	50	41	35	55	55	Block tear.
129	10	21	C 100x85x5	-170.86	-42.27	C58	49	41	34	55	55	Block tear.

pro C2 Fv a Fb < 10
pro C3 Fv a Fb < 10
pro C4 Fv a Fb < 14
pro C5 Fv a Fb < 17

POUŽÍT ŠROUBY M16

Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek
Datum:	23.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka
				Norma projektu:	ČSN EN

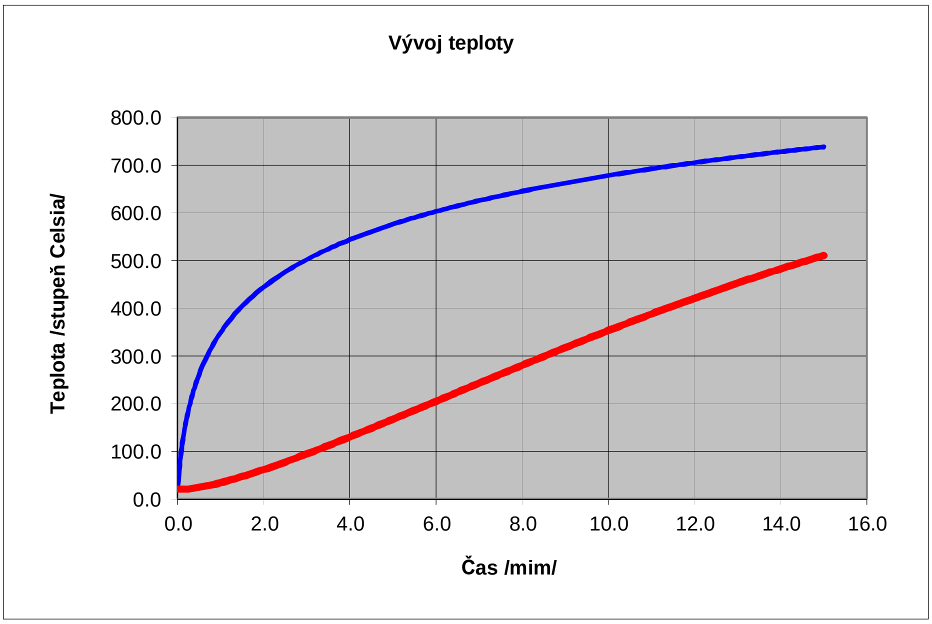
Souhrn:

Prvek	Member type	Profil	Materiál	Komb. N_Ed	Profil [%]	Spoj [%]	Vyhodnoceni	Posouzení
1	Strut 2 screw fire	C 100x85x5	HX500LAD	11	11	13	Shear	Vyhovuje
2	Strut 2 screw fire	C 100x85x5	HX500LAD	12	11	13	Shear	Vyhovuje
8	Strut 2 screw fire	C 100x85x5	HX500LAD	12	49	13	Vzpěr	Vyhovuje
9	Strut 2 screw fire	C 100x85x5	HX500LAD	11	51	13	Vzpěr	Vyhovuje
14	Strut 2 screw fire	C 100x82x4	HX420LAD	12	18	12	Tah a ohyb	Vyhovuje
15	Strut 2 screw fire	C 100x82x4	HX420LAD	11	18	13	Prostý tah	Vyhovuje
20	Strut 2 screw fire	C 100x82x4	HX420LAD	12	72	11	Vzpěr	Vyhovuje
21	Strut 2 screw fire	C 100x82x4	HX420LAD	11	74	11	Vzpěr	Vyhovuje
26	Strut 2 screw fire	C 100x82x4	HX420LAD	12	10	6	Tah a ohyb	Vyhovuje
27	Strut 2 screw fire	C 100x82x4	HX420LAD	11	10	7	Prostý tah	Vyhovuje
32	Strut 2 screw fire	C 100x77x3	S350GD	12	64	7	Vzpěr	Vyhovuje
33	Strut 2 screw fire	C 100x77x3	S350GD	11	70	8	Vzpěr	Vyhovuje
38	Strut 2 screw fire	C 100x75x2	S350GD	11	30	2	Štíhlost	Vyhovuje
39	Strut 2 screw fire	C 100x75x2	S350GD	12	20	2	Štíhlost	Vyhovuje
44	Strut 2 screw fire	C 100x75x2	S350GD	11	10	5	Tah a ohyb	Vyhovuje
45	Strut 2 screw fire	C 100x75x2	S350GD	22	8	4	Tah a ohyb	Vyhovuje
50	Strut 2 screw fire	C 100x77x3	S350GD	11	80	10	Vzpěr	Vyhovuje
51	Strut 2 screw fire	C 100x77x3	S350GD	12	73	9	Vzpěr	Vyhovuje
56	Strut 2 screw fire	C 100x82x4	HX420LAD	11	11	7	Tah a ohyb	Vyhovuje
57	Strut 2 screw fire	C 100x82x4	HX420LAD	12	10	7	Prostý tah	Vyhovuje
62	Strut 2 screw fire	C 100x82x4	HX420LAD	10	59	12	Vzpěr	Vyhovuje
63	Strut 2 screw fire	C 100x82x4	HX420LAD	10	57	11	Vzpěr	Vyhovuje
68	Strut 2 screw fire	C 100x85x5	HX500LAD	10	10	13	Shear	Vyhovuje
69	Strut 2 screw fire	C 100x85x5	HX500LAD	10	10	12	Shear	Vyhovuje
74	Strut 2 screw fire	C 100x85x5	HX500LAD	10	52	15	Vzpěr	Vyhovuje
75	Strut 2 screw fire	C 100x85x5	HX500LAD	10	51	15	Vzpěr	Vyhovuje
80	Strut 2 screw fire	C 100x85x5	HX500LAD	10	17	17	Block tear.	Vyhovuje
81	Strut 2 screw fire	C 100x85x5	HX500LAD	10	16	17	Block tear.	Vyhovuje
86	Strut 2 screw fire	C 100x85x5	HX500LAD	10	59	19	Vzpěr	Vyhovuje
87	Strut 2 screw fire	C 100x85x5	HX500LAD	10	57	19	Vzpěr	Vyhovuje
92	Strut 2 screw fire	C 100x85x5	HX500LAD	10	22	23	Block tear.	Vyhovuje
93	Strut 2 screw fire	C 100x85x5	HX500LAD	10	22	22	Block tear.	Vyhovuje
98	Strut 2 screw fire	C 100x85x5	HX500LAD	10	72	26	Vzpěr	Vyhovuje
99	Strut 2 screw fire	C 100x85x5	HX500LAD	10	71	26	Vzpěr	Vyhovuje
104	Strut 2 screw fire	C 100x85x5	HX500LAD	10	31	33	Block tear.	Vyhovuje
105	Strut 2 screw fire	C 100x85x5	HX500LAD	10	31	32	Block tear.	Vyhovuje
110	Strut 2 screw fire	C 100x85x5	HX500LAD	10	90	37	Vzpěr	Vyhovuje
111	Strut 2 screw fire	C 100x85x5	HX500LAD	10	89	36	Vzpěr	Vyhovuje
116	Strut 2 screw fire	C 100x85x5	HX500LAD	10	41	43	Block tear.	Vyhovuje
117	Strut 2 screw fire	C 100x85x5	HX500LAD	10	40	42	Block tear.	Vyhovuje
122	Strut 2 screw fire	C 100x85x5	HX500LAD	10	101	45	Vzpěr	Nevyhovuje
123	Strut 2 screw fire	C 100x85x5	HX500LAD	10	99	45	Vzpěr	Vyhovuje
128	Strut 2 screw fire	C 100x85x5	HX500LAD	10	53	55	Block tear.	Vyhovuje
129	Strut 2 screw fire	C 100x85x5	HX500LAD	10	52	55	Block tear.	Vyhovuje

Profil C40/132/100/132/40x6

POUŽÍT ŠROUBY M16

REDUKČNÍ SOUČINITELÉ PRO NÁVRH NA R15
SLOUPY RÁMU QC 360X5X4



Čas t=15 min.

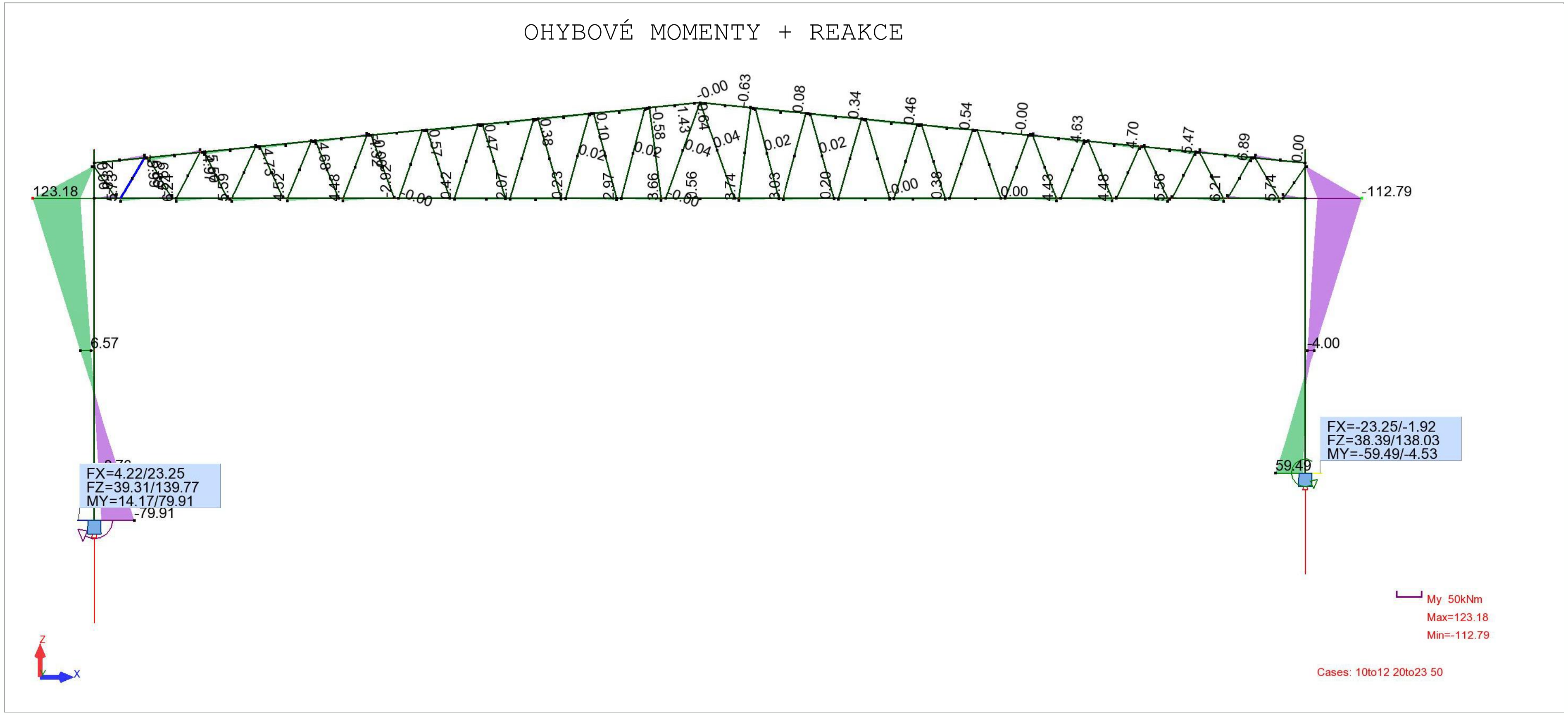
	Součinitelé pro průřezy 1-3		
Teploty	$k_{y\Theta}$	$k_{P\Theta}$	$k_{E\Theta}$
500	0.78	0.36	0.6
600	0.47	0.18	0.31
700	0.23	0.075	0.13
800	0.11	0.05	0.09
Teplota nosníku			
510.82	**0.746**	**0.341**	**0.569**

	Součinitelé pro průřezy 4		
Teploty	$k_{p\Theta}$	$k_{E\Theta}$	
500	0.53	0.6	
600	0.3	0.31	
700	0.13	0.13	
800	0.07	0.09	
Teplota nosníku			
510.82	**0.505**	**0.569**	

$k_{y\Theta}$ red. součinitel na mez kluzu
$k_{P\Theta}$ red. součinitel pro návrhovou mez kluzu
$k_{E\Theta}$ red. součinitel pro linerní pružnou část

	Součinitelé pro spoje		
Teploty	$k_{b\Theta}$	$k_{w\Theta}$	
500	0.55	0.627	
600	0.22	0.378	
700	0.1	0.13	
800	0.067	0.074	
Teplota nosníku			
510.82	**0.514**	**0.600**	**0.000**

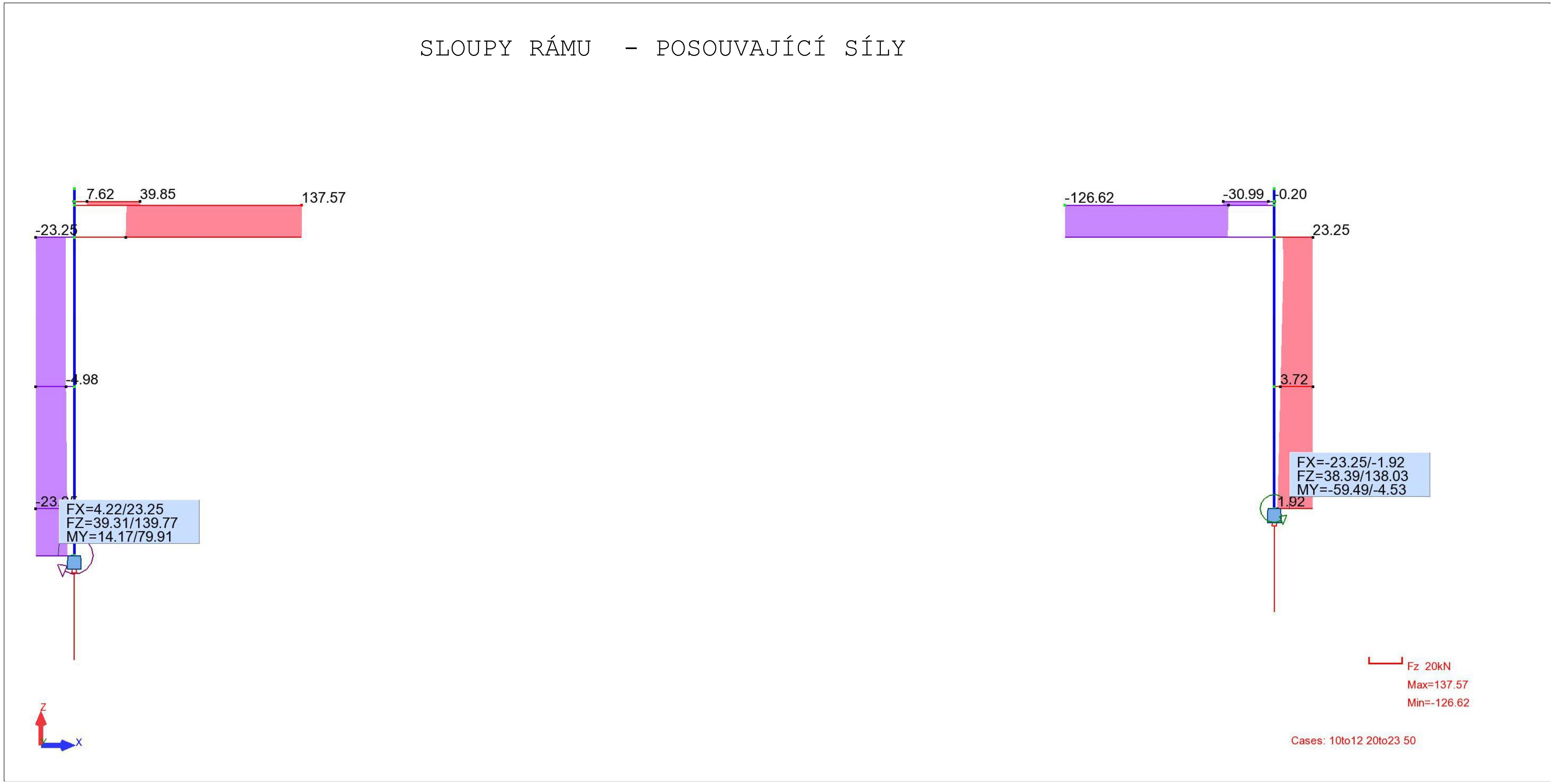
View - MY;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 50



View:1 - FX;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 50

SLOUPY RÁMU - NORMÁLOVÉ SÍLY

View:1 - FZ;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 50



POSUDEK SLOUPŮ RÁMU NA R15

Frame Column 4-C-profiles, d=100mm

$\gamma_{M0} := 1.0$ $\gamma_{M1} := 1.0$ $\gamma_{M2} := 1.25$

Sectionproperties for frame colum: four-part C-profiles

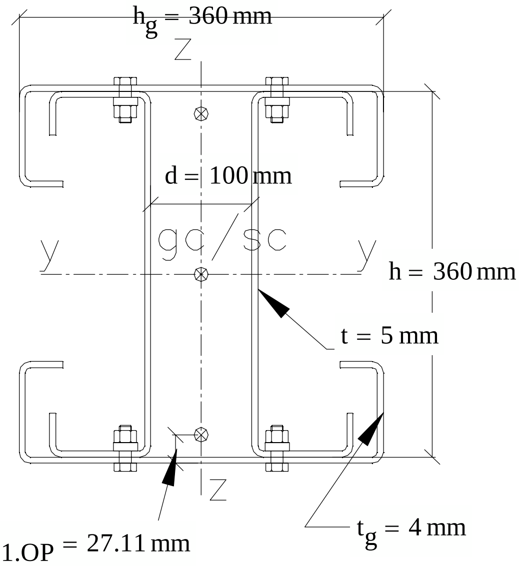
Four-part frame column: $f_{yb} := 212 \cdot \frac{N}{mm^2}$ $f_u := 242 \cdot \frac{N}{mm^2}$

profile for innerprofile is : IP := "360x5D"

profile for outerprofile is : OP := "360x4D"

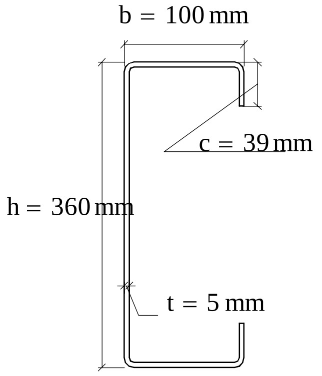
distance be tween C-profiles: $d := 100 \cdot mm$
(do not change)

capacity for bolts: $F_{b.Rk} := 54.3 \cdot kN$

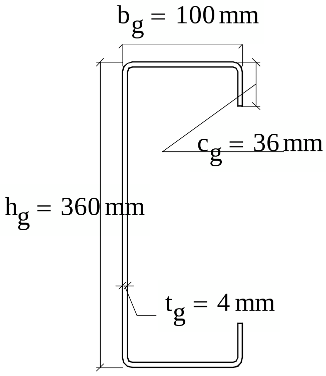


Section properties single column C-profile:

inner profile:



outer profile:



$A_{gg} = 1.09 \times 10^4 \, mm^2$ $A_{eff.cc} = 7.25 \times 10^3 \, mm^2$ $i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 147.52 \, mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 112.09 \, mm$

$I_{yy} = 2.37 \times 10^8 \, mm^4$ $I_{zz} = 1.37 \times 10^8 \, mm^4$ $W_{eff.yy} = 1.03 \times 10^6 \, mm^3$ $I_{TT} = 2.95 \times 10^3 \, cm^4$ $I_{\omega\omega} = 3.79 \times 10^6 \, cm^6$

Stresses and buckling length according to first order frame analys

forces from: Loadcase := "ALL"

$M_{Ed.1} := 124 \cdot kN \cdot m$

$M_{Ed.2} := -80 \cdot kN \cdot m$

$M_{Ed.3} := 0 \cdot kN \cdot m$

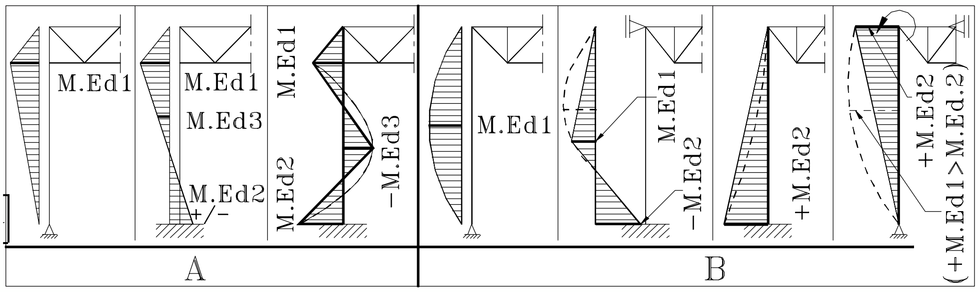
$M_{Ed.1.z} := 0.01 \cdot kN \cdot m$

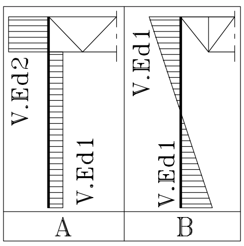
$M_{Ed.2.z} := 0 \cdot kN \cdot m$

$V_{Ed.1} := 24 \cdot kN$

$V_{Ed.2} := 138 \cdot kN$

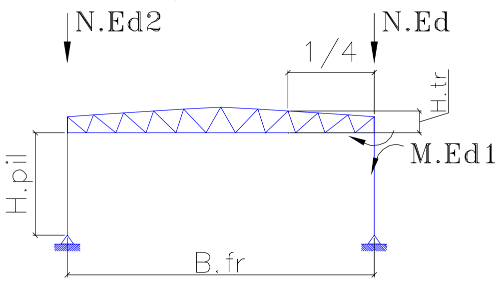
$N_{Ed} := 140 \cdot kN$





The moment is distributed according to frame modelling like shown in figure: Moment_dis := "A"

Global buckling length in y-y accounting for the stiffness behavior of frame: (from "Stahlbau in Beispielen, Berechnungspraxis nach DIN 18800-1 bis 3", Hünersen, Fritzsche)

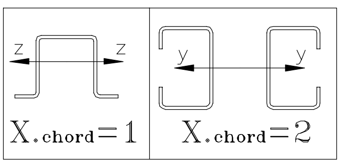


length pillar: $H_{pil} := 8.7 \cdot m$

width frame: $B_{fr} := 32.7 \cdot m$

height truss: $H_{tr} := 1705 \cdot mm$

compr. force on pillar 2: $N_{Ed.2} := 140 \cdot kN$



Area upper chord: $A_{u.chord} := 4899 \cdot mm^2$

I upper chord: $I_{u.chord} := 2.2 \cdot 10^7 \cdot mm^4$

number of profiles upper chord: $X_{u.chord} := 2$

Area lower chord: $A_{lo.chord} := 4071 \cdot mm^2$

I lower chord: $I_{lo.chord} := 18.44 \cdot 10^6 \cdot mm^4$

number of profiles lower chord: $X_{lo.chord} := 2$

$I_{chord} := X_{u.chord} \cdot \left[I_{u.chord} + A_{u.chord} \cdot (H_{tr} \cdot 0.5)^2 \right] + X_{lo.chord} \cdot \left[I_{lo.chord} + A_{lo.chord} \cdot (H_{tr} \cdot 0.5)^2 \right]$ $I_{chord} = 1.31 \times 10^{10} mm^4$

$\alpha_\beta := \frac{4 \cdot I_{yy}}{B_{fr}^2 \cdot A_{gg}}$ $\alpha_\beta = 0$ $c_\beta := \min\left(\frac{I_{yy} \cdot B_{fr}}{I_{chord} \cdot H_{pil}}, 10\right)$ $c_\beta = 0.07$ $m_\beta := \min\left(\frac{N_{Ed.2}}{N_{Ed}}, 1\right)$ $m_\beta = 1$ $scope_\beta = "Ok"$

$\beta_{y.1} := \max\left[\sqrt{0.5 \cdot (1 + m_\beta)} \cdot \sqrt{4 + 1.4 \cdot (c_\beta + 6 \cdot \alpha_\beta) + 0.02 \cdot (c_\beta + 6 \cdot \alpha_\beta)^2}, 2\right]$ $\beta_{y.1} = 2.02$ factor for buckling length in plane (y-y)

$\beta_{y.2} := \sqrt{\frac{1}{2} \cdot (1 + m_\beta)} \cdot \sqrt{1 + 0.35 \cdot (c_\beta + 6 \cdot \alpha_\beta) - 0.017 \cdot (c_\beta + 6 \cdot \alpha_\beta)^2}$ $\beta_{y.2} = 1.01$ factor for buckling length in plane (y-y) pillar fixed att bottom

Buckling lengths: $L := H_{pil}$ Length of pillar= $L = 8.7 m$ like chosen above

$\beta_y = 1.01$

$L_y := 2 \cdot L$ Flexural buckling axis y-y:

$L_z := 1.0 \cdot L$ Flexural buckling axis z-z

$L_{LT} := 1.0 \cdot L$ Distance flange bracings - LTB inner flange in compression

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 pillar: 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

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}}$ $\lambda_1 = 98.88$ $\lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}} \cdot \sqrt{\frac{A_{eff.cc}}{A_{gg}}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.y.FBcc} = 0.97$ Imperfection factor α relating to bucking curve a $\alpha_y := 0.21$

$\phi_{y.FB} := 0.5 \cdot \left[1 + \alpha_y \cdot (\lambda_{r.y.FBcc} - 0.2) + \lambda_{r.y.FBcc}^2\right]$ $\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^2 - \lambda_{r.y.FBcc}^2}}, 1\right)$ $\chi_{y.FBcc} = 0.68$

$\phi_{y.FB} = 1.06$

Design buckling resistance for buckling mode -flexural buckling-:
y-y: INNER PROFILES: $IP = "360x5D"$

$N_{Rd.y.FBcc.IP} := \frac{\chi_{y.FBcc} \cdot A_{eff.cc.IP} \cdot f_{yb}}{\gamma_{M1}}$ $N_{Rd.y.FBcc.IP} = 615.4 kN$

Design buckling resistance for buckling mode -flexural buckling-:
y-y: OUTER PROFILES: $OP = "360x4D"$

$N_{Rd.y.FBcc.OP} := \frac{\chi_{y.FBcc} \cdot A_{eff.cc.OP} \cdot f_{yb}}{\gamma_{M1}}$ $N_{Rd.y.FBcc.OP} = 435.7 kN$

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

$\lambda_1 = 98.88$ $\lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}} \cdot \sqrt{\frac{A_{eff.cc}}{A_{gg}}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.z.FBcc} = 0.64$ Imperfection factor α relating to bucking curve b $\alpha_z := 0.34$

$\phi_{z.FB} := 0.5 \cdot \left[1 + \alpha_z \cdot (\lambda_{r.z.FBcc} - 0.2) + \lambda_{r.z.FBcc}^2\right]$ $\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^2 - \lambda_{r.z.FBcc}^2}}, 1\right)$ $\chi_{z.FBcc} = 0.82$

$\phi_{z.FB} = 0.78$

Design buckling resistance for buckling mode -flexural buckling-:
z-z: INNER PROFILES: $IP = "360x5D"$

$N_{Rd.z.FBcc.IP} := \frac{\chi_{z.FBcc} \cdot A_{eff.cc.IP} \cdot f_{yb}}{\gamma_{M1}}$ $N_{Rd.z.FBcc.IP} = 734.2 kN$

Design buckling resistance for buckling mode -flexural buckling-:
z-z: OUTER PROFILES: $OP = "360x4D"$

$N_{Rd.z.FBcc.OP} := \frac{\chi_{z.FBcc} \cdot A_{eff.cc.OP} \cdot f_{yb}}{\gamma_{M1}}$ $N_{Rd.z.FBcc.OP} = 519.9 kN$

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. $L_{LT} = 8.7 m$ $Moment\_dis = "A"$

$k_{M.cr} := 1.0$ (hinged at ends) $k_{w.M.cr} := 1.0$ (no special wrap restraints at ends)

For pillar with fixed both upper and lower beam (end moments): according to picture "A":

$$\psi_{md.A} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \geq |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}$$

$\psi_{md.A} = -0.65$

$$C_{1.A} := \begin{cases} \min\left(1.88 - 1.40\cdot\psi_{md.A} + 0.52\cdot\psi_{md.A}^2, 2.7\right) & \text{if } |M_{Ed.3}| = 0\cdot kN\cdot m \\ 1.285 & \text{if } |M_{Ed.3}| \neq 0\cdot kN\cdot m \end{cases}$$ $C_{1.A} = 2.7$

$$M_{cr.A} := C_{1.A}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5}$$ $M_{cr.A} = 8237.73\,kN\cdot m$

For framepillar with moment distribution according to picture "B":

$$C_{1.B} := \begin{cases} 1.132 & \text{if } |M_{Ed.3}| = 0\cdot kN\cdot m \\ 1.285 & \text{if } |M_{Ed.3}| \neq 0\cdot kN\cdot m \end{cases}$$ $C_{1.B} = 1.13$

$$M_{cr.B} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5}$$ $M_{cr.B} = 3453.74\,kN\cdot m$

__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:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if Moment\_dis} = \text{"A"} \\ M_{cr.B} & \text{if Moment\_dis} = \text{"B"} \end{cases}$$ $M_{cr} = 8238\,kN\cdot m$

$$\psi_{md.z} := \begin{cases} \frac{M_{Ed.2.z}}{M_{Ed.1.z}} & \text{if } |M_{Ed.1.z}| \geq |M_{Ed.2.z}| \\ \frac{M_{Ed.1.z}}{M_{Ed.2.z}} & \text{if } |M_{Ed.1.z}| < |M_{Ed.2.z}| \end{cases}$$ $\psi_{md.z} = 0$

$$C_{1.A.z.T} := \begin{cases} \min\left(1.88 - 1.40\cdot\psi_{md.z} + 0.52\cdot\psi_{md.z}^2, 2.7\right) & \text{if Moment\_dis} = \text{"A"} \\ 1.132 & \text{if Moment\_dis} = \text{"B"} \end{cases}$$ $C_{1.A.z.T} = 1.88$

$$M_{cr.z} := C_{1.A.z.T}\cdot\frac{\pi^2\cdot E\cdot I_{yy}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{yy}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{yy}}\right]^{0.5}$$ $M_{cr.z} = 7548.64\,kN\cdot m$

Relative slenderness: $\lambda_{rLT} := \sqrt{\frac{W_{eff.yy}\cdot f_{yb}}{M_{cr}}}$ $\lambda_{rLT} = 0.16$

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

$\phi_{LT} := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{rLT} - 0.2) + \lambda_{rLT}^2\right]$ $\phi_{LT} = 0.51$

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

Relative slenderness: $\lambda_{rLT.z} := \sqrt{\frac{W_{eff.zz}\cdot f_{yb}}{M_{cr.z}}}$ $\lambda_{rLT.z} = 0.16$

$\phi_{LT.z} := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{rLT.z} - 0.2) + \lambda_{rLT.z}^2\right]$ $\phi_{LT.z} = 0.51$

$\chi_{LT.z1} := \min\left(\frac{1}{\phi_{LT.z} + \sqrt{\phi_{LT.z}^2 - \lambda_{rLT.z}^2}}, 1\right)$ $\chi_{LT.z1} = 1$ Reduction buckling factor

$$\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{zz} \leq I_{yy} \\ 1 & \text{if } I_{zz} > I_{yy} \end{cases}$$ $\chi_{LT} = 1$

$$\chi_{LT.z} := \begin{cases} \chi_{LT.z1} & \text{if } I_{zz} > I_{yy} \\ 1 & \text{if } I_{zz} \leq I_{yy} \end{cases}$$ $\chi_{LT.z} = 1$

__Buckling resistance EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:__

for outer profile: $M_{yy.Rk.OP} := W_{eff.yy.OP}\cdot f_{yb}$

for inner profile: $M_{zz.Rk.IP} := W_{zz.IP}\cdot f_{yb}$

for outer profile: $M_{zz.Rk.OP} := W_{eff.zz.OP}\cdot f_{yb}$

$M_{yy.R.IP} := \chi_{LT}\cdot M_{yy.Rk.IP}$ $M_{yy.Rk.IP} = 260.4\,kN\cdot m$

$M_{yy.Rk.OP} := \chi_{LT}\cdot M_{yy.Rk.OP}$ $M_{yy.Rk.OP} = 87.06\,kN\cdot m$

$M_{zz.Rk.IP} := \chi_{LT.z}\cdot M_{zz.Rk.IP}$ $M_{zz.Rk.IP} = 66.3\,kN\cdot m$

$M_{zz.Rk.OP} := \chi_{LT.z}\cdot M_{zz.Rk.OP}$ $M_{zz.Rk.OP} = 131.44\,kN\cdot m$

$b = 100\,mm \qquad h = 360\,mm \qquad c = 39\,mm \qquad t = 5\,mm \qquad f_{yb} = 212\,\frac{N}{mm^2} \qquad f_u = 242\,\frac{N}{mm^2} \qquad \gamma_{M0} = 1.00 \qquad \gamma_{M1} = 1.00$

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

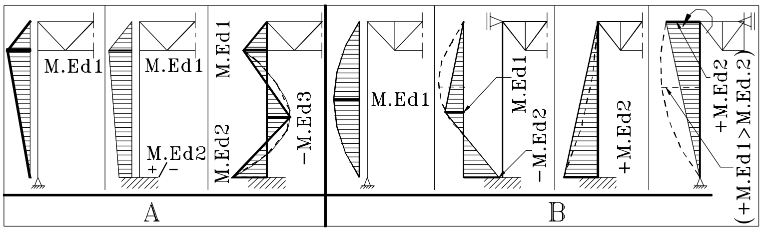
The interaction factors k_{yy} and k_{zy} are obtained from Annex B with **method 2** of EN 1993-1-1: 6.3.3(4)

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

Moment distribution like shown on figure: Moment_dis = "A"

Load is distributed like uniform (UL) or concentrated (CL): Load_dis = "UL"

$\psi_{md.A} = -0.65$



$$\psi_{m.B} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}$$

$\psi_{m.B} = -0.65$

$$\psi_{md.z} := \begin{cases} \frac{M_{Ed.2.z}}{M_{Ed.1.z}} & \text{if } |M_{Ed.1.z}| \ge |M_{Ed.2.z}| \\ \frac{M_{Ed.1.z}}{M_{Ed.2.z}} & \text{if } |M_{Ed.1.z}| < |M_{Ed.2.z}| \end{cases} \qquad \psi_{md.z} = 0$$

$$\alpha_{h.2} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } M_{Ed.1} \ne 0 \cdot kN \cdot m \\ 1 & \text{if } M_{Ed.1} = 0 \cdot kN \cdot m \end{cases} \qquad \alpha_{h.2} = -0.65$$

$$\alpha_{h.2.z} := \begin{cases} \frac{M_{Ed.2.z}}{M_{Ed.1.z}} & \text{if } |M_{Ed.1.z}| > |M_{Ed.2.z}| \\ \frac{M_{Ed.1.z}}{M_{Ed.2.z}} & \text{if } |M_{Ed.1.z}| < |M_{Ed.2.z}| \end{cases} \qquad \alpha_{h.2.z} = 0$$

$$\alpha_{s.2} := \begin{cases} \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } M_{Ed.2} \ne 0 \cdot kN \cdot m \\ 1 & \text{if } (M_{Ed.2} = 0 \cdot kN \cdot m) \vee (M_{Ed.2} < M_{Ed.1}) \end{cases} \qquad \alpha_{s.2} = 1$$

For members with sway mode: C_{my}=0,9 and C_{mz} =0.9

$C_{my.2.A} := 0.9$

$$C_{my.2.B} := \begin{cases} (0.95 + 0.05 \cdot \alpha_{h.2}) & \text{if } [(0 \le \alpha_{h.2} \le 1) \vee (-1 \le \alpha_{h.2} < 0)] \wedge (-1 \le \psi_{m.B} \le 1) \wedge (\text{Moment\_dis} = \text{"B"}) \wedge (\text{Load\_d} \\ (0.90 + 0.10 \cdot \alpha_{h.2}) & \text{if } [(0 \le \alpha_{h.2} \le 1) \vee (-1 \le \alpha_{h.2} < 0)] \wedge (-1 \le \psi_{m.B} \le 1) \wedge (\text{Moment\_dis} = \text{"B"}) \wedge (\text{Load\_d} \\ [[\max[(0.2 + 0.8 \cdot \alpha_{s.2}), 0.4]]] & \text{if } (0 \le \alpha_{s.2} \le 1) \wedge (\text{Moment\_dis} = \text{"B"}) \wedge [(\text{Load\_dis} = \text{"UL"}) \vee (\text{Load\_dis} = \text{"} \\ [[\max[(0.1 - 0.8 \cdot \alpha_{s.2}), 0.4]]] & \text{if } (-1 \le \alpha_{s.2} < 0) \wedge (\text{Moment\_dis} = \text{"B"}) \wedge (\text{Load\_dis} = \text{"UL"}) \wedge |M_{Ed.1}| \le |M \\ [[\max[(-0.8 \cdot \alpha_{s.2}), 0.4]]] & \text{if } (-1 \le \alpha_{s.2} < 0) \wedge (\text{Moment\_dis} = \text{"B"}) \wedge (\text{Load\_dis} = \text{"CL"}) \wedge |M_{Ed.1}| \le |M_{Ed.2} \\ (\max(0.6 + 0.4 \cdot \psi_{m.B}, 0.4)) & \text{if } (M_{Ed.1} = 0 \cdot kN \cdot m) \wedge (\text{Moment\_dis} = \text{"B"}) \\ ((\max(0.6 + 0.4 \cdot \psi_{m.B}, 0.4))) & \text{if } \psi_{m.B} \ne 0 \\ 1 & \text{if } (\text{Moment\_dis} = \text{"A"}) \end{cases}$$

$$C_{my.2} := \begin{cases} C_{my.2.A} & \text{if Moment\_dis} = \text{"A"} \\ C_{my.2.B} & \text{if Moment\_dis} = \text{"B"} \end{cases} \qquad C_{my.2} = 0.9 \qquad C_{my.2.B} = 1$$

$$C_{mz.2} := \begin{cases} 0.9 & \text{if Moment\_dis} = \text{"A"} \\ (0.95 + 0.05 \cdot \alpha_{h.2.z}) & \text{if } (0 \le \alpha_{h.2.z} \le 1) \wedge (-1 \le \psi_{md.z} \le 1) \wedge (\text{Moment\_dis} = \text{"B"}) \wedge (\text{Load\_dis} = \text{"UL"}) \wedge M_{Ed} \\ (0.90 + 0.10 \cdot \alpha_{h.2.z}) & \text{if } (0 \le \alpha_{h.2.z} \le 1) \wedge (-1 \le \psi_{md.z} \le 1) \wedge (\text{Moment\_dis} = \text{"B"}) \wedge (\text{Load\_dis} = \text{"CL"}) \wedge M_{Ed} \\ \max(0.6 + 0.4 \cdot \psi_{md.z}, 0.4) & \text{if } M_{Ed.1.z} = 0 \cdot kN \cdot m \end{cases}$$

$$C_{mLT.2} := \begin{cases} \max(0.6 + 0.4 \cdot \psi_{md.A}, 0.4) & \text{if Moment\_dis} = \text{"A"} \\ C_{my.2} & \text{if Moment\_dis} = \text{"B"} \end{cases} \qquad C_{mLT.2} = 0.4 \qquad C_{mz.2} = 0.9$$

$$n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot (N_{cc.Rk.IP} + N_{cc.Rk.OP})} \qquad n_{y.2} = 0.07 \qquad n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot (N_{cc.Rk.IP} + N_{cc.Rk.OP})} \qquad n_{z.2} = 0.06$$

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

$k_{yy.2} := \min\left[C_{my.2}\cdot\left(1+0.6\cdot\lambda_{r.y.FBcc}\cdot n_{y.2}\right), C_{my.2}\cdot\left(1+0.6\cdot n_{y.2}\right)\right]$ $k_{yy.2} = 0.94$

$k_{zy.2} := \max\left(1-\frac{0.05\cdot\lambda_{r.z.FBcc}}{C_{mLT.2}-0.25}\cdot n_{z.2}, 1-\frac{0.05}{C_{mLT.2}-0.25}\cdot n_{z.2}\right)$ $k_{zy.2} = 0.99$

$k_{zz.2} := \min\left[C_{mz.2}\cdot\left(1+0.6\cdot\lambda_{r.z.FBcc}\cdot n_{z.2}\right), C_{mz.2}\cdot\left(1+0.6\cdot n_{z.2}\right)\right]$ $k_{zz.2} = 0.92$ $k_{yz.2} := k_{zz.2}$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.68$ $\chi_{z.FBcc} = 0.82$

Reduction factor for L-T buckling: $\chi_{LT} = 1$ $\chi_{LT.z} = 1$

Shear force EN 1993-1-3: 6.1.10:

$\frac{\max\left(V_{Ed.1}, V_{Ed.2}\right)\cdot\gamma_{M0}}{2\cdot V_{bh.Rk.IP}} = 0.21$ **< 0.5** Otherwise: combine M + N + V

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

Check outer+inner profile member IP = "360x5D" OP = "360x4D"

$$\frac{N_{Ed}\cdot\gamma_{M0}}{N_{cc.Rk.IP}+N_{cc.Rk.OP}} + \frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right)\cdot\gamma_{M0}}{M_{yy.Rk.IP}+M_{yy.Rk.OP}} + \frac{\max\left(\left|M_{Ed.1.z}\right|, \left|M_{Ed.2.z}\right|\right)\cdot\gamma_{M0}}{M_{zz.Rk.IP}+M_{zz.Rk.OP}} = 0.40$$ <1.0

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

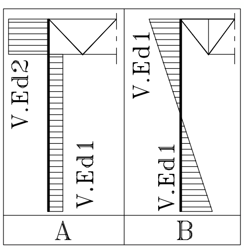
Evading in y-y:

$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot\left(N_{cc.Rk.IP}+N_{cc.Rk.OP}\right)} + k_{yy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot\left(M_{yy.Rk.IP}+M_{yy.Rk.OP}\right)} \ldots + k_{yz.2}\cdot\frac{\max\left(\left|M_{Ed.1.z}\right|, \left|M_{Ed.2.z}\right|\right)\cdot\gamma_{M1}}{\chi_{LT.z}\cdot\left(M_{zz.Rk.IP}+M_{zz.Rk.OP}\right)} = 0.40$$ <1.0

Evading in z-z:

$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot\left(N_{cc.Rk.IP}+N_{cc.Rk.OP}\right)} + k_{zy.2}\cdot\frac{\max\left(\left|M_{Ed.1}\right|, \left|M_{Ed.2}\right|, \left|M_{Ed.3}\right|\right)\cdot\gamma_{M1}}{\chi_{LT}\cdot\left(M_{yy.Rk.IP}+M_{yy.Rk.OP}\right)} \ldots + k_{zz.2}\cdot\frac{\max\left(\left|M_{Ed.1.z}\right|, \left|M_{Ed.2.z}\right|\right)\cdot\gamma_{M1}}{\chi_{LT.z}\cdot\left(M_{zz.Rk.IP}+M_{zz.Rk.OP}\right)} = 0.41$$ <1.0

Screw between C-profiles



$V_{Ed.1} = 24\,kN$

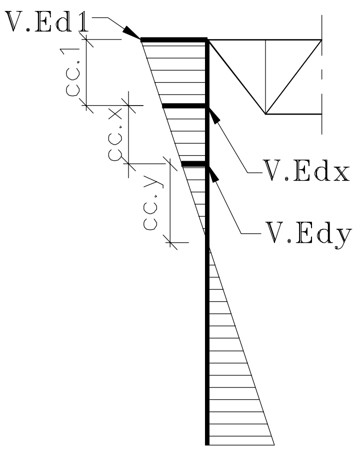
$V_{Ed.2} = 138\,kN$

$n_{bolt} := 2$ $cc_{min} := 500\cdot mm$

$S_{y.OP} := A_{g.OP}\cdot\left(0.5\cdot h + t_g\right)$

$cc_{skruv} := \min\left[\left[\frac{F_{b.Rk}\cdot n_{bolt}}{\gamma_{M2}}\cdot\frac{I_{yy}}{\left(V_{Ed.1}\cdot S_{y.OP}\right)}\right], cc_{min}\right]$

$cc_{skruv} = 500\,mm$ distance between screw per sida <u>over hole pillar length for constant shearforce</u>



If <u>shear force is linear</u> you may reduce the distance:

$V_{Ed.x} := 120\cdot kN$ $cc_x := \min\left[\left[\frac{F_{b.Rk}\cdot n_{bolt}}{\gamma_{M2}}\cdot\frac{I_{yy}}{\left(V_{Ed.x}\cdot S_{y.OP}\right)}\right], cc_{min}\right]$ $cc_x = 386\,mm$

$V_{Ed.y} := 50\cdot kN$ $cc_y := \min\left[\left[\frac{F_{b.Rk}\cdot n_{bolt}}{\gamma_{M2}}\cdot\frac{I_{yy}}{\left(V_{Ed.y}\cdot S_{y.OP}\right)}\right], cc_{min}\right]$ $cc_y = 500\,mm$

and so on....

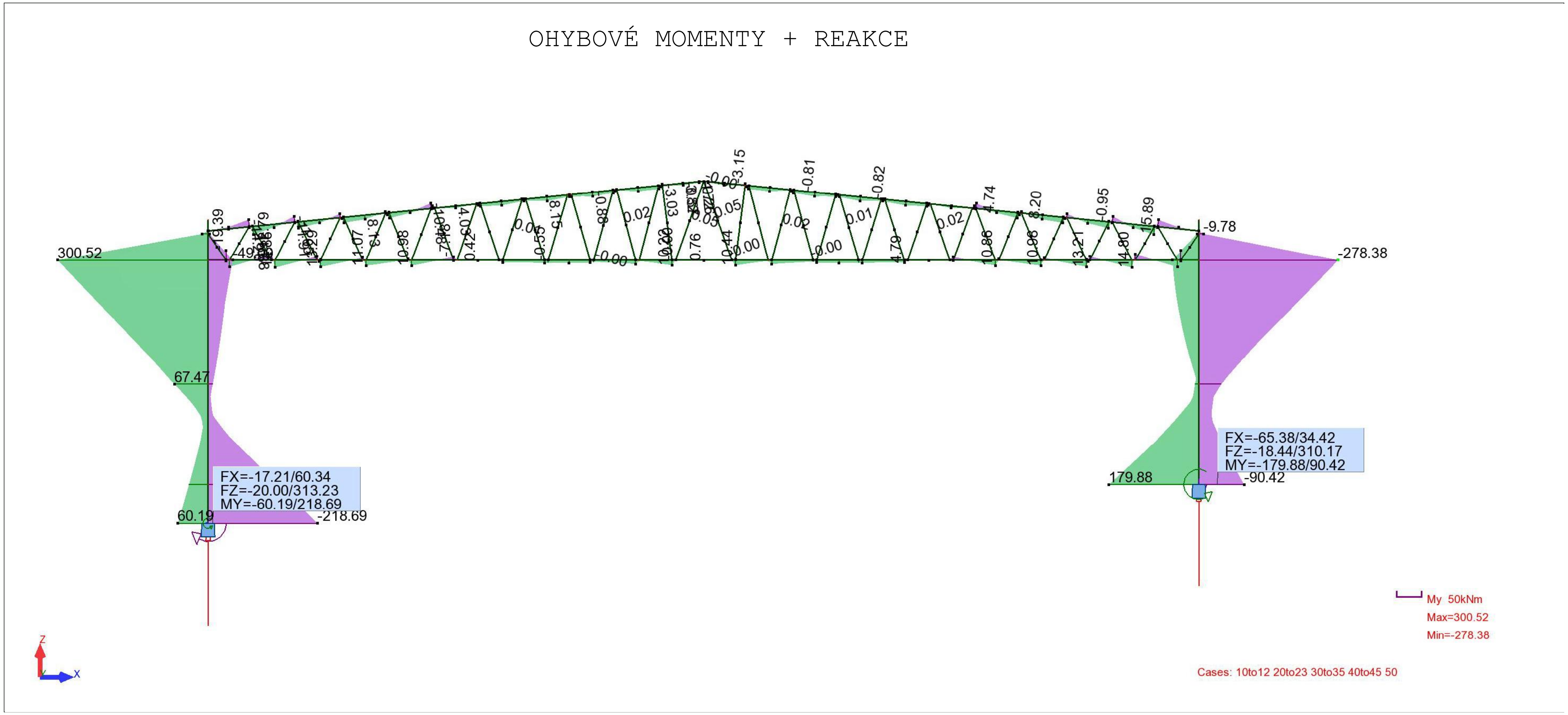
6. Loads combinations.

No.	Combination	Name and definition of combination
1	10	Snow 1.35x(101+102+104)+1.5x(201+202)
2	11	Snow 0.5 left 1.35x(101+102+104)+0.75x(201)+1.5x(202)
3	12	Snow 0.5 right 1.35x(101+102+104)+1.5x(201)+0.75x(202)
4	20	Wind Cpe max L 1x(101+102)+1.5x(300)
5	21	Wind Cpe max R 1x(101+102)+1.5x(301)
6	22	Wind Cpe min L 1.35x(101+102+104)+1.5x(302)
7	23	Wind Cpe min R 1.35x(101+102+104)+1.5x(303)
8	30	Snow + reduced wind Cpe min L 1.35x(101+102+104)+1.5x(201+202)+0.9x(302)
9	31	Snow 0.5 left + reduced wind Cpe min L 1.35x(101+102+104)+0.75x(201)+1.5x(202)+0.9x(302)
10	32	Snow 0.5 right + reduced wind Cpe min L 1.35x(101+102+104)+1.5x(201)+0.75x(202)+0.9x(302)
11	33	Snow + reduced wind Cpe min R 1.35x(101+102+104)+1.5x(201+202)+0.9x(303)
12	34	Snow 0.5 left + reduced wind Cpe min R 1.35x(101+102+104)+0.75x(201)+1.5x(202)+0.9x(303)
13	35	Snow 0.5 right + reduced wind Cpe min R 1.35x(101+102+104)+1.5x(201)+0.75x(202)+0.9x(303)
14	40	Wind Cpe min L + reduced snow 1.35x(101+102+104)+0.75x(201+202)+1.5x(302)
15	41	Wind Cpe min L + reduced snow 0.5 left 1.35x(101+102+104)+0.375x(201)+0.75x(202)+1.5x(302)
16	42	Wind Cpe min L + reduced snow 0.5 right 1.35x(101+102+104)+0.75x(201)+0.375x(202)+1.5x(302)
17	43	Wind Cpe min R + reduced snow 1.35x(101+102+104)+0.75x(201+202)+1.5x(303)
18	44	Wind Cpe min R + reduced snow 0.5 left 1.35x(101+102+104)+0.375x(201)+0.75x(202)+1.5x(303)
19	45	Wind Cpe min R + reduced snow 0.5 right 1.35x(101+102+104)+0.75x(201)+0.375x(202)+1.5x(303)
20	50	Wind Cpe max from gabel 1x(101+102)+1.5x(310)

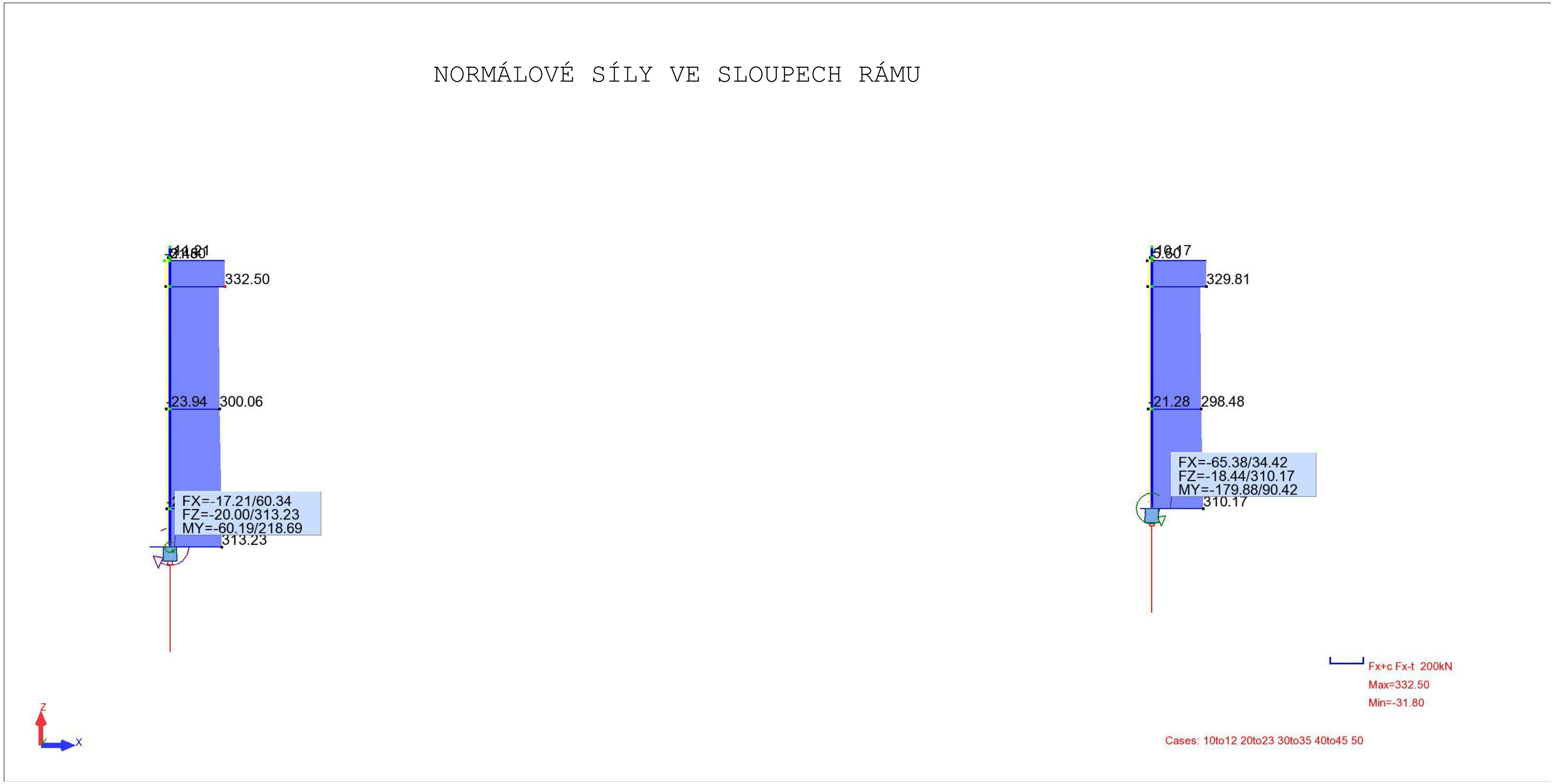
VÝPOČET PROVEDEN V PROGRAMU AUTODESK ROBOT STRUCTURAL ANALYSIS PROFESSIONAL 2018

VÝPOČET PROVEDEN NELINEÁRNÍ ANALÝZOU.

View - MY;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50



View:1 - FX;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50



View:1 - FZ;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50

POSOUVAJÍCÍ SÍLY VE SLOUPECH RÁMU

POSUDEK SLOUPŮ RÁMU

Frame Column 4-C-profiles, d=100mm

$\gamma_{M0} := 1.0$ $\gamma_{M1} := 1.0$ $\gamma_{M2} := 1.25$

Sectionproperties for frame colum: four-part C-profiles

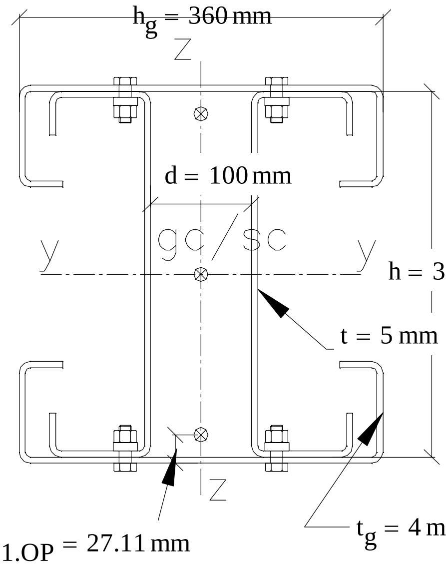
Four-part frame column:

f.yb
420N/mm2 for t=4mm
500N/mm2 for t> 4mm

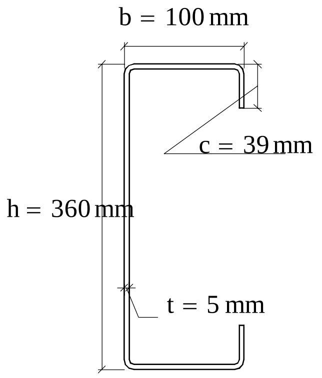
profile for innerprofile is : IP := "360x5D"

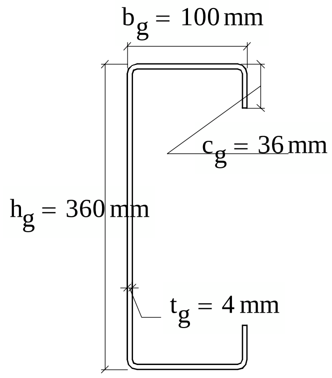
profile for outerprofile is : OP := "360x4D"

distance be tween C-profiles: $d := 100 \cdot mm$
(do not change)



Section properties single column C-profile:
inner profile: outer profile:





$A_{gg} = 1.09 \times 10^4 \, mm^2$ $A_{eff.cc} = 7.03 \times 10^3 \, mm^2$ $i_{yy} := \sqrt{\frac{I_{yy}}{A_{gg}}}$ $i_{yy} = 146.58 \, mm$ $i_{zz} := \sqrt{\frac{I_{zz}}{A_{gg}}}$ $i_{zz} = 111.11 \, mm$

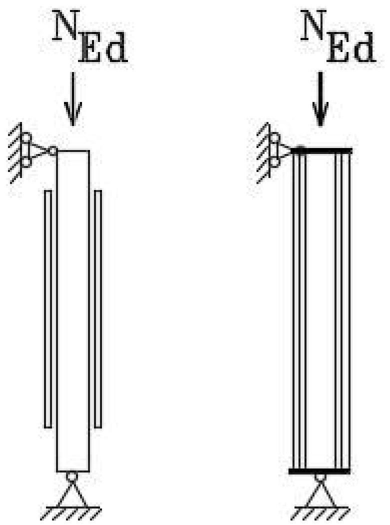
$I_{yy} = 2.34 \times 10^8 \, mm^4$ $I_{zz} = 1.34 \times 10^8 \, mm^4$ $W_{eff.yy} = 1.02 \times 10^6 \, mm^3$ $I_{TT} = 2.95 \times 10^3 \, cm^4$ $I_{\omega\omega} = 3.01 \times 10^4 \, cm^6$

Global geometry and constructive details

Type of connection in the ends, where the axial force is distributed:

Conn_type := "simple"

simple = picture to the left
full = picture to the right

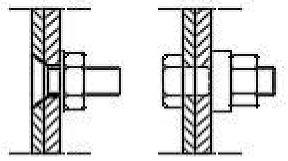


Type of screw connecting inner- and outer profile:

Conn_screw := "LL_M12" OBS = "ok"

Countersunk bolt "CS_M12" LLentab standard "LL_M12"

"CS_M12"



"LL_M12"

Buckling length according to first order frame analys

$M_{Ed.1} := 301 \cdot kN \cdot m$

$M_{Ed.2} := -219 \cdot kN \cdot m$

$M_{Ed.3} := 0 \cdot kN \cdot m$

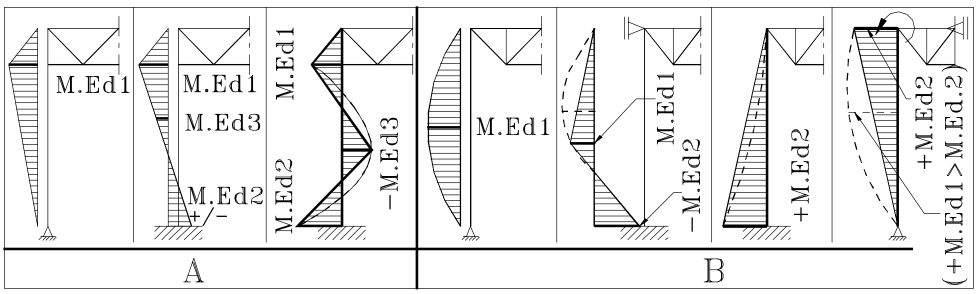
$M_{Ed.1.z} := 10 \cdot kN \cdot m$

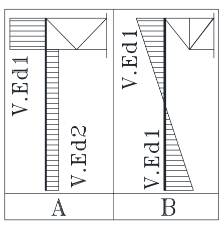
$M_{Ed.2.z} := 0 \cdot kN \cdot m$

$V_{Ed.1} := 333 \cdot kN$

$V_{Ed.2} := 61 \cdot kN$

$N_{Ed} := 314 \cdot kN$





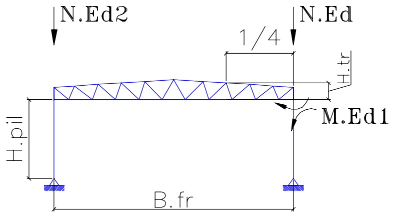
The moment is distributed according to frame modelling like shown in figure:

Moment_dis := "A"

kombinace 33

Buckling lengths:

$L := 8.7 \cdot m$ length pillar



$B_{fr} := 32.7 \cdot m$ width frame

compr. force on pillar 2: $N_{Ed.2} := 311 \cdot kN$

$H_{tr} := 1705 \cdot mm$ height truss

$A_{u.chord} := 2450 \cdot mm^2$ Area upper chord

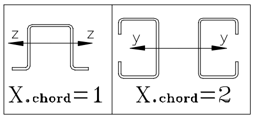
$I_{u.chord} := 1.1 \cdot 10^7 \cdot mm^4$ I upper chord

$X_{u.chord} := 2$ number of profiles upper chord

$A_{lo.chord} := 2036 \cdot mm^2$ Area lower chord

$I_{lo.chord} := 9.22 \cdot 10^6 \cdot mm^4$ I lower chord

$X_{lo.chord} := 2$ number of profiles lower chord



Global buckling length in y-y accounting for the stiffness behavior of frame: (from "Stahlbau in Beispielen, Berechnungspraxis nach DIN 18800-1 bis 3", Hünersen, Fritzsche)

$\beta_y = 1.02$

$L_y := 2 \cdot L$ Flexural buckling axis y-y:

$L_z := 9.6 \cdot m$ Flexural buckling axis z-z

$L_{LT} := 1.0 \cdot L$ Lateral torsional buckling length

<u>Flexural buckling resistance - axis y-y and z-z EN 1993-1-3: 6.2.2 and EN 1993-1-1: 6.3.1:</u>

Buckling curve for pillar: 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

<u>Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about y-y</u>

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb.IP}}}$ $\lambda_1 = 64.38$ $\lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}} \cdot \sqrt{\frac{A_{eff.cc}}{A_{gg}}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.y.FBcc} = 1.48$ Imperfection factor α relating to bucking curve a $\alpha_y := 0.21$

$\phi_{y.FB} := 0.5 \cdot \left[1 + \alpha_y \cdot \left(\lambda_{r.y.FBcc} - 0.2\right) + \lambda_{r.y.FBcc}^{\;2}\right]$ $\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB} + \sqrt{\phi_{y.FB}^{\;2} - \lambda_{r.y.FBcc}^{\;2}}}, 1\right)$ $\chi_{y.FBcc} = 0.38$

$\phi_{y.FB} = 1.73$

Design buckling resistance for buckling mode -<u>flexural buckling</u>-:
y-y: INNER PROFILES: IP = "360x5D"

$N_{Rd.y.FBcc.IP} := \frac{\chi_{y.FBcc} \cdot A_{eff.cc.IP} \cdot f_{yb.IP}}{\gamma_{M1}}$ $N_{Rd.y.FBcc.IP} = 765.5\,kN$

Design buckling resistance for buckling mode -<u>flexural buckling</u>-:
y-y: OUTER PROFILES: OP = "360x4D"

$N_{Rd.y.FBcc.OP} := \frac{\chi_{y.FBcc} \cdot A_{eff.cc.OP} \cdot f_{yb.OP}}{\gamma_{M1}}$ $N_{Rd.y.FBcc.OP} = 479.6\,kN$

<u>Slenderness for flexural buckling EN 1993-1-1: 6.3.1.3: about z-z</u>

$\lambda_1 = 64.38$ $\lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}} \cdot \sqrt{\frac{A_{eff.cc}}{A_{gg}}} \cdot \frac{1}{\lambda_1}$ $\lambda_{r.z.FBcc} = 1.08$ Imperfection factor α relating to bucking curve b $\alpha_z := 0.34$

$\phi_{z.FB} := 0.5 \cdot \left[1 + \alpha_z \cdot \left(\lambda_{r.z.FBcc} - 0.2\right) + \lambda_{r.z.FBcc}^{\;2}\right]$ $\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB} + \sqrt{\phi_{z.FB}^{\;2} - \lambda_{r.z.FBcc}^{\;2}}}, 1\right)$ $\chi_{z.FBcc} = 0.55$

$\phi_{z.FB} = 1.23$

Design buckling resistance for buckling mode -<u>flexural buckling</u>-:
z-z: INNER PROFILES: IP = "360x5D"

$N_{Rd.z.FBcc.IP} := \frac{\chi_{z.FBcc} \cdot A_{eff.cc.IP} \cdot f_{yb.IP}}{\gamma_{M1}}$ $N_{Rd.z.FBcc.IP} = 1103.9\,kN$

Design buckling resistance for buckling mode -<u>flexural buckling</u>-:
z-z: OUTER PROFILES: OP = "360x4D"

$N_{Rd.z.FBcc.OP} := \frac{\chi_{z.FBcc} \cdot A_{eff.cc.OP} \cdot f_{yb.OP}}{\gamma_{M1}}$ $N_{Rd.z.FBcc.OP} = 691.5\,kN$

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.

$k_{M.cr} := 1$ (hinged at ends) $k_{w.M.cr} := 1$ (no special wrap restraints at ends)

For pillar with fixed both upper and lower beam (end moments): according to picture "A":

$$\psi_{md.A} := \begin{cases} \dfrac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \ge |M_{Ed.2}| \\ \dfrac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases} \qquad \psi_{md.A} = -0.73$$

$$C_{1.A} := \begin{cases} \min\left(1.88 - 1.40\cdot\psi_{md.A} + 0.52\cdot\psi_{md.A}^2, 2.7\right) & \text{if } |M_{Ed.3}| = 0\cdot kN\cdot m \\ 1.285 & \text{if } |M_{Ed.3}| \ne 0\cdot kN\cdot m \end{cases} \qquad C_{1.A} = 2.7$$

$$M_{cr.A} := C_{1.A}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.A} = 7994.64\ kN\cdot m$$

For framepillar with moment distribution according to picture "B":

$$C_{1.B} := \begin{cases} 1.132 & \text{if } |M_{Ed.3}| = 0\cdot kN\cdot m \\ 1.285 & \text{if } |M_{Ed.3}| \ne 0\cdot kN\cdot m \end{cases} \qquad C_{1.B} = 1.13$$

$$M_{cr.B} := C_{1.B}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \qquad M_{cr.B} = 3351.83\ kN\cdot m$$

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:

$$M_{cr} := \begin{cases} M_{cr.A} & \text{if Moment\_dis} = \text{"A"} \\ M_{cr.B} & \text{if Moment\_dis} = \text{"B"} \end{cases} \qquad M_{cr} = 7995\ kN\cdot m$$

$$\psi_{md.z} := \begin{cases} \dfrac{M_{Ed.2.z}}{M_{Ed.1.z}} & \text{if } |M_{Ed.1.z}| \ge |M_{Ed.2.z}| \\ \dfrac{M_{Ed.1.z}}{M_{Ed.2.z}} & \text{if } |M_{Ed.1.z}| < |M_{Ed.2.z}| \end{cases} \qquad \psi_{md.z} = 0$$

$$C_{1.A.z.T} := \begin{cases} \min\left(1.88 - 1.40\cdot\psi_{md.z} + 0.52\cdot\psi_{md.z}^2, 2.7\right) & \text{if Moment\_dis} = \text{"A} \\ 1.132 & \text{if Moment\_dis} = \text{"B"} \end{cases} \qquad C_{1.A.z.T} = 1.88$$

$$M_{cr.z} := C_{1.A.z.T}\cdot\frac{\pi^2\cdot E\cdot I_{yy}}{(k_{M.cr}\cdot L_{LT})^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{yy}} + \frac{(k_{M.cr}\cdot L_{LT})^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{yy}}\right]^{0.5} \qquad M_{cr.z} = 7343.66\ kN\cdot m$$

Relative slenderness: $\lambda_{rLT} := \sqrt{\dfrac{W_{eff.yy}\cdot f_{yb.IP}}{M_{cr}}}$ $\qquad \lambda_{rLT} = 0.25$

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

$\phi_{LT} := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{rLT} - 0.2) + \lambda_{rLT}^2\right]$ $\quad \phi_{LT} = 0.54$ $\quad \chi_{LT.y} := \min\left(\dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{rLT}^2}}, 1\right)$ $\quad \chi_{LT.y} = 0.98$ Reduction buckling factor

Relative slenderness: $\lambda_{rLT.z} := \sqrt{\dfrac{W_{eff.zz}\cdot f_{yb.IP}}{M_{cr.z}}}$ $\quad \lambda_{rLT.z} = 0.25$ $\quad \phi_{LT.z} := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{rLT.z} - 0.2) + \lambda_{rLT.z}^2\right]$ $\quad \phi_{LT.z} = 0.54$

$\chi_{LT.z1} := \min\left(\dfrac{1}{\phi_{LT.z} + \sqrt{\phi_{LT.z}^2 - \lambda_{rLT.z}^2}}, 1\right)$ $\quad \chi_{LT.z1} = 0.98$ Reduction buckling factor

$$\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{zz} \le I_{yy} \\ 1 & \text{if } I_{zz} > I_{yy} \end{cases} \qquad \chi_{LT} = 0.98 \qquad \chi_{LT.z} := \begin{cases} \chi_{LT.z1} & \text{if } I_{zz} > I_{yy} \\ 1 & \text{if } I_{zz} \le I_{yy} \end{cases} \qquad \chi_{LT.z} = 1$$

Buckling resistance EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4:

for outer profile: $M_{yy.Rk.OP} := W_{eff.yy.OP}\cdot f_{yb.OP}$

for inner profile: $M_{zz.Rk.IP} := W_{zz.IP}\cdot f_{yb.IP}$

for outer profile: $M_{zz.Rk.OP} := W_{eff.zz.OP}\cdot f_{yb.OP}$

$M_{yy.R.IP} := \chi_{LT}\cdot M_{yy.Rk.IP}$ $\qquad M_{yy.Rk.IP} = 305.06\ kN\cdot m$

$M_{yy.Rk.OP} := \chi_{LT}\cdot M_{yy.Rk.OP}$ $\qquad M_{yy.Rk.OP} = 169.24\ kN\cdot m$

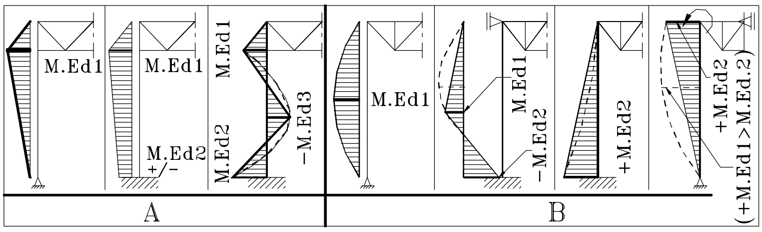
$M_{zz.Rk.IP} := \chi_{LT.z}\cdot M_{zz.Rk.IP}$ $\qquad M_{zz.Rk.IP} = 148.4\ kN\cdot m$

$M_{zz.Rk.OP} := \chi_{LT.z}\cdot M_{zz.Rk.OP}$ $\qquad M_{zz.Rk.OP} = 256.25\ kN\cdot m$

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"
Load is distributed like uniform (UL) or concentrated (CL): Load_dis = "UL"

$\psi_{md.A} = -0.73$



$$\psi_{m.B} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } |M_{Ed.1}| \geq |M_{Ed.2}| \\ \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } |M_{Ed.1}| < |M_{Ed.2}| \end{cases}$$

$\psi_{m.B} = -0.73$

$$\psi_{md.z} := \begin{cases} \frac{M_{Ed.2.z}}{M_{Ed.1.z}} & \text{if } |M_{Ed.1.z}| \geq |M_{Ed.2.z}| \\ \frac{M_{Ed.1.z}}{M_{Ed.2.z}} & \text{if } |M_{Ed.1.z}| < |M_{Ed.2.z}| \end{cases} \qquad \psi_{md.z} = 0$$

$$\alpha_{h.2} := \begin{cases} \frac{M_{Ed.2}}{M_{Ed.1}} & \text{if } M_{Ed.1} \neq 0 \cdot kN \cdot m \\ 1 & \text{if } M_{Ed.1} = 0 \cdot kN \cdot m \end{cases} \qquad \alpha_{h.2} = -0.73$$

$$\alpha_{h.2.z} := \begin{cases} \frac{M_{Ed.2.z}}{M_{Ed.1.z}} & \text{if } |M_{Ed.1.z}| > |M_{Ed.2.z}| \\ \frac{M_{Ed.1.z}}{M_{Ed.2.z}} & \text{if } |M_{Ed.1.z}| < |M_{Ed.2.z}| \end{cases} \qquad \alpha_{h.2.z} = 0$$

$$\alpha_{s.2} := \begin{cases} \frac{M_{Ed.1}}{M_{Ed.2}} & \text{if } M_{Ed.2} \neq 0 \cdot kN \cdot m \\ 1 & \text{if } (M_{Ed.2} = 0 \cdot kN \cdot m) \vee (M_{Ed.2} < M_{Ed.1}) \end{cases} \qquad \alpha_{s.2} = 1$$

Equivalent uniform moment factors acc. to EN1993-1-1 table B3 $C_{my.2} = 0.9$ $C_{mz.2} = 0.9$ $C_{mLT.2} = 0.4$

$$n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot (N_{cc.Rk.IP} + N_{cc.Rk.OP})} \qquad n_{y.2} = 0.25 \qquad n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot (N_{cc.Rk.IP} + N_{cc.Rk.OP})} \qquad n_{z.2} = 0.17$$

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

$$k_{yy.2} := \min\left[C_{my.2} \cdot (1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}), C_{my.2} \cdot (1 + 0.6 \cdot n_{y.2})\right] \qquad k_{yy.2} = 1.04$$

$$k_{zy.2} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FBcc}}{C_{mLT.2} - 0.25} \cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT.2} - 0.25} \cdot n_{z.2}\right) \qquad k_{zy.2} = 0.94$$

$$k_{zz.2} := \min\left[C_{mz.2} \cdot (1 + 0.6 \cdot \lambda_{r.z.FBcc} \cdot n_{z.2}), C_{mz.2} \cdot (1 + 0.6 \cdot n_{z.2})\right] \qquad k_{zz.2} = 0.99 \qquad k_{yz.2} := k_{zz.2}$$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.38$ $\chi_{z.FBcc} = 0.55$

Reduction factor for L-T buckling: $\chi_{LT} = 0.98$ $\chi_{LT.z} = 1$

The pillar member is even checked for moment and axial compression: distribution acc. to stiffness/area:

$M_{Ed.IP} = 164.26\,kN \cdot m$ Moment taken of inner profile
$M_{Ed.OP} = 136.74\,kN \cdot m$ Moment taken of outer profile
$M_{Ed.IP.z} = 3.31\,kN \cdot m$ Moment taken of inner profile
$M_{Ed.OP.z} = 6.69\,kN \cdot m$ Moment taken of outer profile

$N_{Ed.IP} = 314\,kN$ axial force taken of inner profile
$N_{Ed.OP} = 0\,kN$ axial force taken of outer profile

The outer profile needs to be checked for compression due to bending moment:

$$N_{Ed.comp.OP\_M} := \frac{M_{Ed.OP}}{h - 2 \cdot e_{1.OP}} \qquad N_{Ed.comp.OP\_M} = 447.19\,kN$$

Shear force EN 1993-1-3: 6.1.10:

$$\frac{\max(V_{Ed.1}, V_{Ed.2})\cdot\gamma_{M0}}{2\cdot V_{bh.Rk.IP}} = 0.47$$ **< 0.5** Otherwise: combine M + N + V -> $M\_N\_V_{EC} = 0$ <1.0

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

Check outer+inner profile member IP = "360x5D" OP = "360x4D"

$$\frac{N_{Ed}\cdot\gamma_{M0}}{N_{cc.Rk.IP} + N_{cc.Rk.OP}} + \frac{\max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.3}|)\cdot\gamma_{M0}}{M_{yy.Rk.IP} + M_{yy.Rk.OP}} + \frac{\max(|M_{Ed.1.z}|, |M_{Ed.2.z}|)\cdot\gamma_{M0}}{M_{zz.Rk.IP} + M_{zz.Rk.OP}} = 0.76$$ <1.0

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

Evading in y-y:

$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot(N_{cc.Rk.IP} + N_{cc.Rk.OP})} + k_{yy.2}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.3}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot(M_{yy.Rk.IP} + M_{yy.Rk.OP})} \ldots = 0.95$$
$$+ k_{yz.2}\cdot\frac{\max(|M_{Ed.1.z}|, |M_{Ed.2.z}|)\cdot\gamma_{M1}}{\chi_{LT.z}\cdot(M_{zz.Rk.IP} + M_{zz.Rk.OP})}$$
<1.0

Evading in z-z:

$$\frac{N_{Ed}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot(N_{cc.Rk.IP} + N_{cc.Rk.OP})} + k_{zy.2}\cdot\frac{\max(|M_{Ed.1}|, |M_{Ed.2}|, |M_{Ed.3}|)\cdot\gamma_{M1}}{\chi_{LT}\cdot(M_{yy.Rk.IP} + M_{yy.Rk.OP})} \ldots = 0.81$$
$$+ k_{zz.2}\cdot\frac{\max(|M_{Ed.1.z}|, |M_{Ed.2.z}|)\cdot\gamma_{M1}}{\chi_{LT.z}\cdot(M_{zz.Rk.IP} + M_{zz.Rk.OP})}$$
<1.0

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

$$\frac{N_{Ed.IP}\cdot\gamma_{M0}}{N_{cc.Rk.IP}} + \frac{M_{Ed.IP}\cdot\gamma_{M0}}{M_{yy.Rk.IP}} = 0.69$$ <1.0 Check inner profile member IP = "360x5D"

$$\frac{N_{Ed.OP}\cdot\gamma_{M0}}{N_{cc.Rk.OP}} + \frac{M_{Ed.OP}\cdot\gamma_{M0}}{M_{yy.Rk.OP}} = 0.81$$ <1.0 Check outer profile member OP = "360x4D"

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

Evading in y-y:

$$\frac{N_{Ed.IP}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk.IP}} + k_{yy.2}\cdot\frac{M_{Ed.IP}\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.Rk.IP}} = 0.98$$ Check inner profile member IP = "360x5D"

$$\frac{N_{Ed.OP}\cdot\gamma_{M1}}{\chi_{y.FBcc}\cdot N_{cc.Rk.OP}} + k_{yy.2}\cdot\frac{M_{Ed.OP}\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.Rk.OP}} = 0.85$$ Check outer profile member OP = "360x4D"

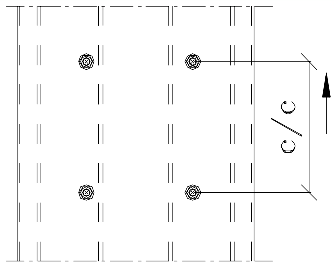
Evading in z-z:

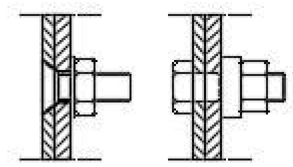
$$\frac{N_{Ed.IP}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk.IP}} + k_{zy.2}\cdot\frac{M_{Ed.IP}\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.Rk.IP}} = 0.80$$ Check inner profile member IP = "360x5D"

$$\frac{N_{Ed.OP}\cdot\gamma_{M1}}{\chi_{z.FBcc}\cdot N_{cc.Rk.OP}} + k_{zy.2}\cdot\frac{M_{Ed.OP}\cdot\gamma_{M1}}{\chi_{LT}\cdot M_{yy.Rk.OP}} = 0.78$$ Check outer profile member OP = "360x4D"

Check outerprofile for buckling between screws for the following axial force: $N_{Ed.comp.OP\_M} = 447.19\ kN$

Screw between C-profiles





<u>Design Resistances for Bolts according to EN 1993-1-8:2005</u>

For LLentab M12 8.8:

$d := 12mm$ = Nominal diameter of bolt $\quad f_{ub} := 800 \cdot \frac{N}{mm^2} \quad \alpha_v := 0.6$

$d_0 := 12.5 \cdot mm$ = Nominal diameter of hole

<u>Shear of bolt (3.6.1):</u> $\quad A := \pi \cdot \frac{d^2}{4} \quad A = 113.1\,mm^2$ gross cross section area

$F_{v.Rk} := \alpha_v \cdot f_{ub} \cdot A \quad F_{v.Rk} = 54.3\,kN$ shear resistance of bolt

$p = 1.75 \quad A_s := \pi \cdot \frac{(d - 0.938194mm \cdot p)^2}{4} \quad A_s = 84.27\,mm^2$ tensile cross section area

$F_{vt.Rk} := \alpha_v \cdot f_{ub} \cdot A_s \quad F_{vt.Rk} = 40.4\,kN$ shear resitance for threaded part of bolt

<u>Tension of bolt (3.6.1):</u>

$k_2 := 0.9 \quad F_{t.Rk} := k_2 \cdot f_{ub} \cdot A_s \quad F_{t.Rk} = 60.7\,kN$ tension resistance

<u>Bearing resistance (3.5 and 3.6.1):</u>

$t_{min} := \min(t_g, t) \quad t_{min} = 4\,mm$

$e_1 := 3.0 \cdot d_0$ (end dist.) $\quad p_1 := \mathrm{if}(t < 3mm, 3 \cdot d_0, \min(3.75 \cdot d_0, 14. \cdot t, 200mm))$ center length)

$e_2 := 1.5 \cdot d_0$ (edge dist.) $\quad p_2 := \mathrm{if}(t < 3mm, 3d_0, \min(3.0 \cdot d_0, 14. \cdot t, 200mm))$ (center width)

$\alpha_{d.end} = 1 \quad \alpha_{d.length} = 1$

$k_{1.edge} = 2.5 \quad k_{1.width} = 2.5 \quad F_{b.Rk} := \alpha_{d.end} \cdot k_{1.edge} \cdot \min(f_{u.IP}, f_{u.OP}) \cdot d \cdot t_{min} \quad F_{b.Rk} = 57.6\,kN$ Bearing resistance

$F_{b.Rk\_LL\_M12} := \min(F_{v.Rk}, F_{b.Rk}) \quad F_{b.Rk\_LL\_M12} = 54.29\,kN$ **LL_M12 Screw capacity**

For countersunk M12 10.9:

$d := 12mm$ = Nominal diameter of bolt $\quad f_{ub.CS} := 1000 \cdot \frac{N}{mm^2} \quad \alpha_v := 0.5$

$d_0 := 12.5 \cdot mm$ = Nominal diameter of hole

<u>Shear of bolt (3.6.1):</u>

$p = 1.75 \quad A_s := \pi \cdot \frac{(d - 0.938194mm \cdot p)^2}{4} \quad A_s = 84.27\,mm^2$ tensile cross section area

$F_{vt.Rk.CS} := \alpha_v \cdot f_{ub.CS} \cdot A_s \quad F_{vt.Rk.CS} = 42.1\,kN$ shear resitance for threaded part of bolt

<u>Tension of bolt (3.6.1):</u>

$k_2 := 0.9 \quad F_{t.Rk.CS} := k_2 \cdot f_{ub.CS} \cdot A_s \quad F_{t.Rk.CS} = 75.8\,kN$ tension resistance

<u>Bearing resistance (3.5 and 3.6.1):</u>

$t_1 := t_g \cdot 0.5 \quad t_1 = 2\,mm \quad \alpha_{d.end} = 1 \quad \alpha_{d.length} = 1 \quad k_{1.edge} = 2.5 \quad k_{1.width} = 2.5$

$F_{b.Rk\_.t.1} := \alpha_{d.end} \cdot k_{1.edge} \cdot f_{u.OP} \cdot d \cdot t_1 \quad F_{b.Rk\_.t.1} = 28.8\,kN$ Bearing resistance counersunk bolt in outer pillar

$t_2 := t \quad t_2 = 5\,mm$

$F_{b.Rk\_t.2} := \alpha_{d.end} \cdot k_{1.edge} \cdot f_{u.IP} \cdot d \cdot t_2 \quad F_{b.Rk\_t.2} = 82.5\,kN$ Bearing resistance counersunk bolt in outer pillar

$F_{b.Rk\_CS\_M12} := \min(F_{vt.Rk.CS}, F_{b.Rk\_.t.1}, F_{b.Rk\_t.2}) \quad F_{b.Rk\_CS\_M12} = 28.8\,kN$ **CS_M12 Screw capacity**

Final capacity for screw

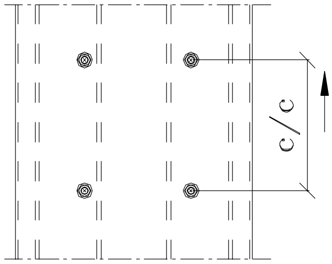
$$F_{b.Rk} := \begin{cases} F_{b.Rk\_LL\_M12} & \text{if Conn\_screw = "LL\_M12"} \\ F_{b.Rk\_CS\_M12} & \text{if Conn\_screw = "CS\_M12"} \end{cases}$$

$F_{b.Rk} = 54.29\,kN \quad$ Conn_screw = "LL_M12"

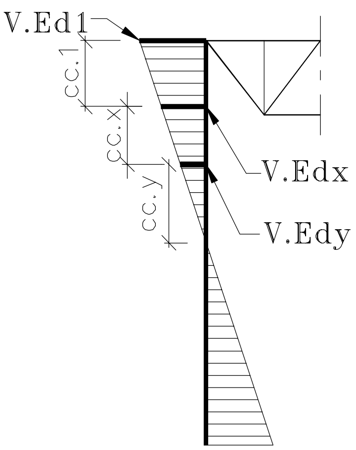
Distance between screws:

$S_{y.OP} := A_{g.OP} \cdot (0.5 \cdot h + t_g - e_{1.OP})$ $\qquad F_{b.Rk} = 54.29\,kN$

$n_{bolt} := 2 \qquad cc_{min} := 500 \cdot mm$ $\qquad \gamma_{M2} = 1.25$



For LINEAR shearforce: <u>reduce</u> the distance



$V_{Ed.1} = 333\,kN \quad cc_x := \min\left[\left[\frac{F_{b.Rk} \cdot n_{bolt}}{\gamma_{M2}} \cdot \frac{I_{yy}}{(V_{Ed.1} \cdot S_{y.OP})}\right], cc_{min}\right]$ $\qquad cc_x = 161\,mm$

$V_{Ed.x} := 120 \cdot kN \quad cc_x := \min\left[\left[\frac{F_{b.Rk} \cdot n_{bolt}}{\gamma_{M2}} \cdot \frac{I_{yy}}{(V_{Ed.x} \cdot S_{y.OP})}\right], cc_{min}\right]$ $\qquad cc_x = 447\,mm$

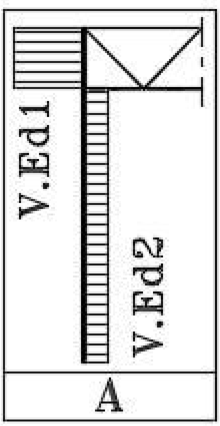
$V_{Ed.y} := 50 \cdot kN \quad cc_y := \min\left[\left[\frac{F_{b.Rk} \cdot n_{bolt}}{\gamma_{M2}} \cdot \frac{I_{yy}}{(V_{Ed.y} \cdot S_{y.OP})}\right], cc_{min}\right]$ $\qquad cc_y = 500\,mm$

$V_{Ed.y.i} := 30 \cdot kN \quad cc_y := \min\left[\left[\frac{F_{b.Rk} \cdot n_{bolt}}{\gamma_{M2}} \cdot \frac{I_{yy}}{(V_{Ed.y.i} \cdot S_{y.OP})}\right], cc_{min}\right]$ $\qquad cc_y = 500\,mm$

$V_{Ed.y.j} := 10 \cdot kN \quad cc_y := \min\left[\left[\frac{F_{b.Rk} \cdot n_{bolt}}{\gamma_{M2}} \cdot \frac{I_{yy}}{(V_{Ed.y.j} \cdot S_{y.OP})}\right], cc_{min}\right]$ $\qquad cc_y = 500\,mm$

and so on....

For KONSTANT shear force :



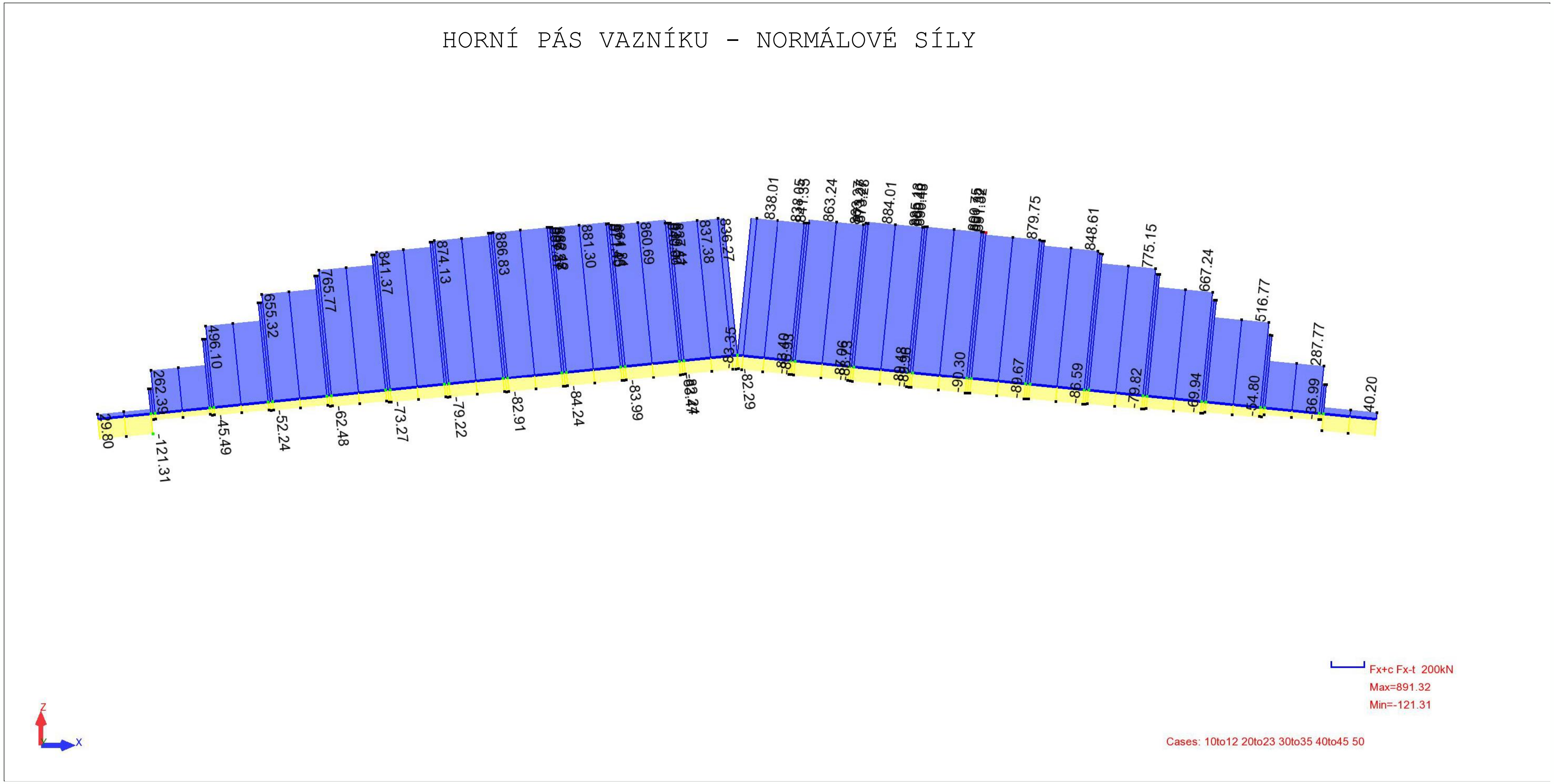
$V_{Ed.1} = 333\,kN \quad cc_{skruv} := \min\left[\left[\frac{F_{b.Rk} \cdot n_{bolt}}{\gamma_{M2}} \cdot \frac{I_{yy}}{(V_{Ed.1} \cdot S_{y.OP})}\right], cc_{min}\right]$

$cc_{skruv} = 161\,mm$ distance between screw per sida over hole pillar length for **constant V_{Ed1}** shearforce

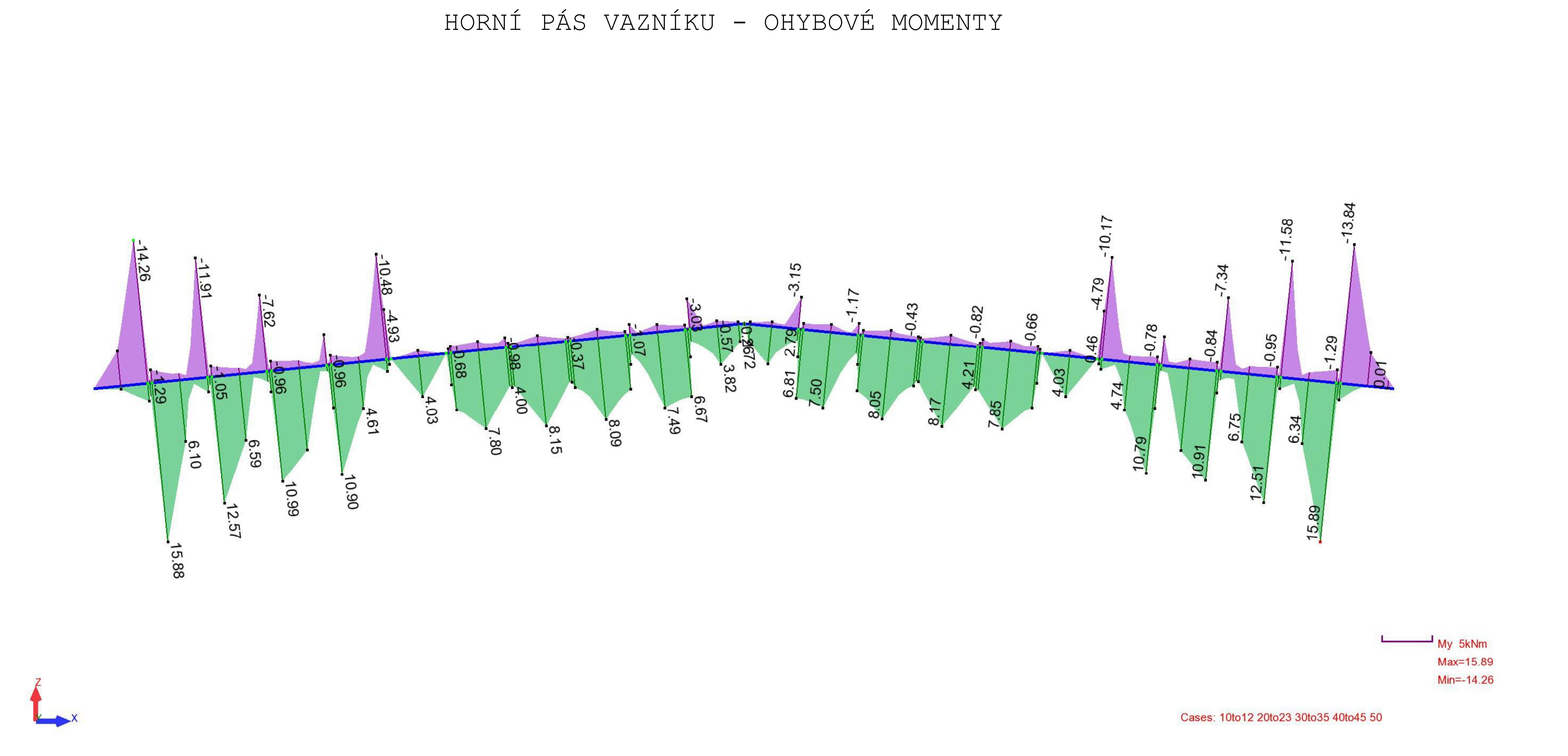
$V_{Ed.2} = 61\,kN \quad cc_{skruv} := \min\left[\left[\frac{F_{b.Rk} \cdot n_{bolt}}{\gamma_{M2}} \cdot \frac{I_{yy}}{[(V_{Ed.2} + 0.0001 \cdot kN) \cdot S_{y.OP}]}\right], cc_{min}\right]$

$cc_{skruv} = 500\,mm$ distance between screw per sida over hole pillar length for **constant V_{Ed2}** shearforce

View:1 - FX;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50



View:1 - MY;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50



POSUDEK HORNÍHU PÁSU VAZNÍKU

Double-C Truss member

$h \equiv 170 \cdot mm$ $b \equiv 100 \cdot mm$ $c \equiv 38 \cdot mm$ $t \equiv 6 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$d := 100 \cdot mm$ $h_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $kb = 400.92\,mm$ $\gamma_{M1} \equiv 1.0$

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

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 4.899 \times 10^3\,mm^2$

$I_{zz} = 4.59 \times 10^7\,mm^4$ $W_{zz.eff} = 2.96 \times 10^5\,mm^3$

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

$M_{yy.cRk} = 134.43\,kN \cdot m$

$M_{ycRk} = 67.22\,kN \cdot m$

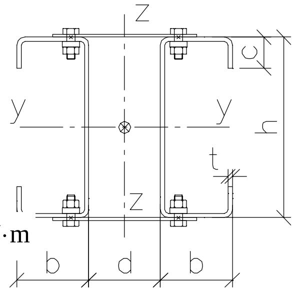
$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$N_{cc.Rk} = 2434.93\,kN$

$f_{yb} = 500 \dfrac{N}{mm^2}$

$f_u = 550 \dfrac{N}{mm^2}$

$M_{zz.cRk} = 148.23\,kN \cdot m$



$I_{yy} := 2 \cdot I_y$ $I_{yy} = 2.2 \times 10^7\,mm^4$

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 2.69 \times 10^5\,mm^3$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 2.69 \times 10^5\,mm^3$

$W_{zz} := \dfrac{I_{zz}}{b + \dfrac{d}{2}}$ $W_{zz} = 3.06 \times 10^5\,mm^3$

$i_{yy} := \sqrt{\dfrac{I_{yy}}{A_{gg}}}$ $i_{yy} = 67.08\,mm$ $i_{zz} := \sqrt{\dfrac{I_{zz}}{A_{gg}}}$ $i_{zz} = 96.78\,mm$

torsion_plate_truss = "NO"

Stresses and global geometry:

Beam is designed as uniform built up member: YES or NO: built_up ≡ "YES"

Number of pair battens per length L: $n_{batt} := 2$

Buckling lengths:

$L := 1.5 \cdot m$ Length between diagonals

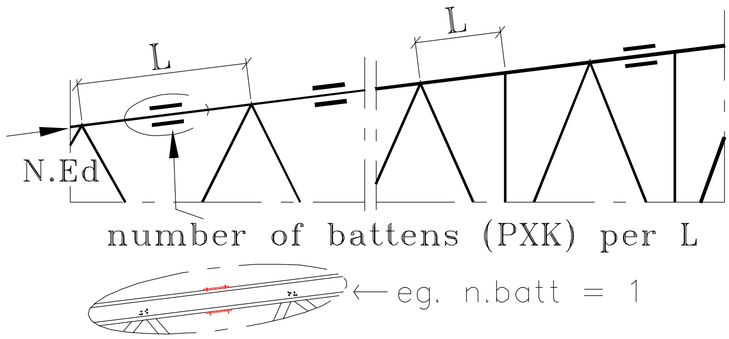
$L_y := 0.9 \cdot L$ Buckling length y-y

$L_z := 1.5 \cdot m$ Buckling length z-z

$L_{LT} := 1.0 \cdot L$ Length for LT-buckling

The moment is distributed according to frame modelling like shown in figure: ("A": Z-purlins; "B" longspan decking)

Moment_dis := "B"



$L_y = 1.35\,m$ Flexural buckling axis y-y

$L_z = 1.5\,m$ Flexural buckling axis z-z

$L_{LT} = 1.5\,m$ Lateral-torsional buckling

For single C-profile: $L_{T.C} = 0.35\,m$ Torsional buckling

$L_{z.C} = 0.5\,m$ FB z-z for single profile

stresses:

$M_{Ed.end.1} := 15.9 \cdot kN \cdot m$ sign: (+)

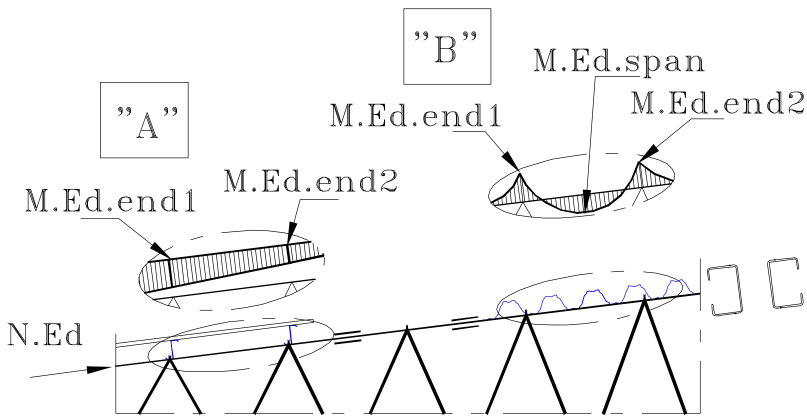
$N_{Ed} := 892 \cdot kN$

$M_{Ed.end.2} := -14.3 \cdot kN \cdot m$ sign: (+) or (-)

$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = 0.05\,kN \cdot m$

$M_{z.Ed} := \left|\Delta M_{z.shift}\right|$ shift of neutral axis for member in compression

$M_{Ed.span} := 10 \cdot kN \cdot m$ sign: (+) or (-) if reverse moment (only of model "B")



$b = 100\,mm \quad h = 170\,mm \quad c = 38\,mm \quad t = 6\,mm \qquad f_{yb} = 500\,\frac{N}{mm^2} \quad f_u = 550\,\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

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

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

$\lambda_1 := \pi\cdot\sqrt{\frac{E}{f_{yb}}} \quad \lambda_1 = 64.38 \quad \lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \quad \lambda_{r.y.FBcc} = 0.31$

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

$\phi_{y.FB.cc} := 0.5\cdot\left[1 + \alpha_y\cdot\left(\lambda_{r.y.FBcc} - 0.2\right) + \lambda_{r.y.FBcc}^2\right] \quad \phi_{y.FB.cc} = 0.56$

$\chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB.cc} + \sqrt{\phi_{y.FB.cc}^2 - \lambda_{r.y.FBcc}^2}}, 1\right) \quad \chi_{y.FBcc} = 0.97$

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

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

non-dimensional slenderness:

$\lambda_1 = 64.38 \quad \lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}}\cdot\sqrt{\frac{2\cdot A_{eff}}{2\cdot A_g}}\cdot\frac{1}{\lambda_1} \quad \lambda_{r.z.FBcc} = 0.24$

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

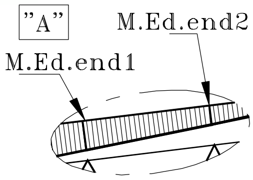
$\chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB.cc} + \sqrt{\phi_{z.FB.cc}^2 - \lambda_{r.z.FBcc}^2}}, 1\right) \quad \chi_{z.FBcc} = 0.99$

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 critikal buckling momtent M_{cr} : given in German ENV 1993-1-1 Annex F.

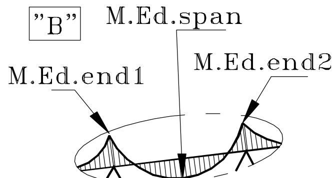
$k_{M.cr} := 1.0$ (hinged at ends) $\quad k_{w.M.cr} := 1.0$ (no special wrap restraints at ends) $\quad$ Moment_dis = "B"

$\psi_m := \begin{cases} \frac{M_{Ed.end.2}}{M_{Ed.end.1}} & \text{if } |M_{Ed.end.1}| \ge |M_{Ed.end.2}| \\ \frac{M_{Ed.end.1}}{M_{Ed.end.2}} & \text{if } |M_{Ed.end.1}| < |M_{Ed.end.2}| \end{cases}$



$C_{1.tr.A} := \min\left(1.88 - 1.40\cdot\psi_m + 0.52\cdot\psi_m^2, 2.7\right)$

$C_{1.tr.A} = 2.7$



$C_{1.tr.B} := 1.285$

$C_{1.tr} := \begin{cases} C_{1.tr.A} & \text{if Moment\_dis = "A"} \\ C_{1.tr.B} & \text{if Moment\_dis = "B"} \end{cases} \quad C_{1.tr} = 1.28$

Elastic critical moment for lateral-torsional buckling:

$M_{cr.tr} := C_{1.tr}\cdot\frac{\pi^2\cdot E\cdot I_{zz}}{\left(k_{M.cr}\cdot L_{LT}\right)^2}\cdot\left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2\cdot\frac{I_{\omega\omega}}{I_{zz}} + \frac{\left(k_{M.cr}\cdot L_{LT}\right)^2\cdot G\cdot I_{TT}}{\pi^2\cdot E\cdot I_{zz}}\right]^{0.5} \quad M_{cr.tr} = 2110.4\,kN\cdot m$

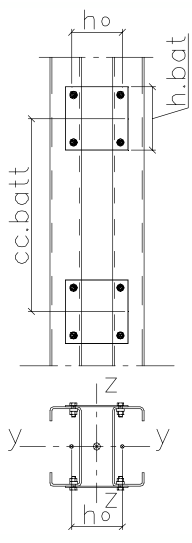
Relative slenderness: $\lambda_{rLT.cc} := \sqrt{\frac{2\cdot W_{eff.y.1}\cdot f_{yb}}{M_{cr.tr}}} \quad \lambda_{rLT.cc} = 0.25$

Imperfection factor α relating to bucking curve b: $\quad \alpha_{LT} := 0.34$

$\phi_{LT.cc} := 0.5\cdot\left[1 + \alpha_{LT}\cdot\left(\lambda_{rLT.cc} - 0.2\right) + \lambda_{rLT.cc}^2\right] \quad \phi_{LT.cc} = 0.54$

Reduction buckling factor $\quad \chi_{LT.y} := \min\left(\frac{1}{\phi_{LT.cc} + \sqrt{\phi_{LT.cc}^2 - \lambda_{rLT.cc}^2}}, 1\right) \quad \chi_{LT.y} = 0.98$

$\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{yy} \ge I_{zz} \\ 1 & \text{otherwise} \end{cases}$



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

$cc_{batt} = 0.75\,m \quad$ bow imperfection: $e_0 := \frac{L_z}{500} \quad e_0 = 3\,mm$

Effective second moment of area of battended built-up member: $\quad A_{ch} := A_g$: area of one chord

$h_0 := d + 2\cdot e_1 \quad h_0 = 178.4\,mm$ distance centroids of chords $\quad I_{ch} := I_z$: I of one chord

$I_{l.CC} := 0.5\cdot h_0^2\cdot A_{ch} + 2\cdot I_{ch}$ I built-up member $\quad i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2\cdot A_{ch}}} \quad \lambda_{CC} := \frac{L_z}{i_{0.CC}} \quad \lambda_{CC} = 15.5$

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

$\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \ge 150 \\ \left(2 - \frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \le 75 \end{cases} \quad \mu_{CC} = 1$

$I_{l.CC} = 4.59 \times 10^7\,mm^4 \quad r_{CC} := 2$

$b = 100\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 6\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu_{CC} \cdot I_{ch}$ $I_{eff} = 4.589 \times 10^7\,mm^4$ effective I of built-up member

Shear stiffnes *EN 1993-1-1:6.4.3.1:(2)*

$I_b := \frac{t_{batt} \cdot h_{batt}^3}{12}$ I of batten $n_{batt.0} := 2$ number of planes of lacings

$$S_v := \min\left[\left[\frac{24 \cdot E \cdot I_{ch}}{cc_{batt}^2 \cdot \left(1 + \frac{2 \cdot I_{ch}}{n_{batt.0} \cdot I_b} \cdot \frac{h_0}{cc_{batt}}\right)}\right], \left(\frac{2 \cdot \pi^2 \cdot E \cdot I_{ch}}{cc_{batt}^2}\right)\right]$$

$S_v = 23711\,kN$

effective critical force of built-up member: $N_{cr.CC} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_z^2}$ $N_{cr.CC} = 42272\,kN$

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

moment z-z without second order effects $M_{z.Ed.I} := 0 \cdot kN \cdot m$

moment with second order effects

$$M_{z.Ed.C} := \begin{cases} \dfrac{N_{Ed} \cdot e_0 + M_{z.Ed.I}}{1 - \dfrac{N_{Ed} \cdot \gamma_{M1}}{N_{cr.CC}} - \dfrac{N_{Ed} \cdot \gamma_{M1}}{S_v}} & \text{if built\_up = "YES"} \\ (0 \cdot kN \cdot m) & \text{if built\_up = "NO"} \end{cases}$$

$M_{z.Ed.C} = 2.8\,kN \cdot m$

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:

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

$N_{ch.Ed} = 459.53\,kN$

Interaktion formulae 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

$C_{my} = 0.7$ $C_{mLT} := C_{my}$ $n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}}$ $n_{y.2} = 0.38$ $n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}}$ $n_{z.2} = 0.37$ $C_{mz.2} := 0.95$

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

$k_{yy} := \min\left[C_{my} \cdot (1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}), C_{my} \cdot (1 + 0.6 \cdot n_{y.2})\right]$ $k_{yy} = 0.75$

$k_{zy} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FBcc}}{C_{mLT} - 0.25} \cdot n_{z.2}, 1 - \frac{0.05}{C_{mLT} - 0.25} \cdot n_{z.2}\right)$ $k_{zy} = 0.99$

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

Reduction factor for L-T buckling: $\chi_{LT} = 1$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.97$ $\chi_{z.FBcc} = 0.99$

Combined bending an axial compression EN 1993-1-3: 6.2.5 (2):

$$\frac{N_{Ed} \cdot \gamma_{M0}}{2 \cdot N_{c.Rk}} + \frac{\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \cdot \gamma_{M0}}{2 \cdot M_{ycRk}} + \frac{|M_{z.Ed.C} \cdot \gamma_{M0}|}{M_{zz.cRk}} = 0.50$$

<1.0

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

Evading in y-y:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}} + k_{yy} \cdot \frac{\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{yz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.48$$

<1.0

Evading in z-z:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}} + k_{zy} \cdot \frac{\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{zz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.51$$

<1.0

If not designed as a built-up member (that means no PXK-plates for connecting), check only single C-profiles according to formulaes further down. built_up = "YES"

$b = 100\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 6\,mm$ $f_{yb} = 500\frac{N}{mm^2}$ $f_u = 550\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1) if the truss beam is designed as built-up member. Otherwise: EN 1993-1-1: 6.3.3

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

$$M_{y.i}(x_i) := \frac{M_{Ed.end.2} - M_{Ed.end.1}}{L}\cdot x_i + M_{Ed.end.1}$$

For moment distribution accord. to model "A"

Moment at quarter point of unbraced segment: $x_{1\_4} := L\cdot 0.25$ $M_{y.AA} := M_{y.i}(x_{1\_4})$ $M_{y.AA} = 8.35\,kN\cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := L\cdot 0.5$ $M_{y.BA} := M_{y.i}(x_{1\_2})$ $M_{y.BA} = 0.8\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := L\cdot 0.75$ $M_{y.CA} := M_{y.i}(x_{3\_4})$ $M_{y.CA} = -6.75\,kN\cdot m$

$$C_{bA} := \frac{12.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3\cdot M_{y.AA} + 4\cdot M_{y.BA} + 3\cdot M_{y.CA}}$$

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

$z_{0.cr} := 0\cdot mm$ (coord. shear center)

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

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

$$\sigma_{ez} := \frac{\pi^2\cdot E}{\left(\frac{1\cdot L_{z.C}}{i_z}\right)^2}$$ $\sigma_{ez} = 1.17\times 10^4\frac{N}{mm^2}$

$$\sigma_T := \frac{1}{A_g\cdot r_0^2}\cdot\left(G\cdot I_T + \frac{\pi^2\cdot E\cdot I_\omega}{L_{T.C}^2}\right)$$ $\sigma_T = 1.29\times 10^4\frac{N}{mm^2}$

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

$M_{e.cr.C\_A} := |C_{bA}|\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_A} = 1.5\times 10^4\,kN\cdot m$

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

$\lambda_{rLT.C\_A} = 0$

$\phi_{LT.C\_A} := 0.5\cdot\left[1 + \alpha_{LT.C}\cdot(\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right]$ $\phi_{LT.C\_A} = 0.48$

$$\chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right)$$ $\chi_{LT.C\_A} = 1$

For moment distribution accord. to model "B"

Moment at quarter point of unbraced segment: $M_{y.AB} := \frac{(M_{Ed.span} - M_{Ed.end.1})\cdot L\cdot 0.25}{L\cdot 0.5} + M_{Ed.end.1}$ $M_{y.AB} = 12.95\,kN\cdot m$

Moment at centerline of unbraced segment: $M_{y.BB} := M_{Ed.span}$ $M_{y.BB} = 10\,kN\cdot m$

Moment at 3/4-point of unbraced segment: $M_{y.CB} := \frac{(M_{Ed.span} - M_{Ed.end.2})\cdot L\cdot 0.25}{L\cdot 0.5} + M_{Ed.end.2}$ $M_{y.CB} = -2.15\,kN\cdot m$

$$C_{bB} := \frac{12.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3\cdot M_{y.AB} + 4\cdot M_{y.BB} + 3\cdot M_{y.CB}}$$

Elastic critical moment for singly-symmetric sections, bending about the axis of symmetry: $M_{e.cr.C\_B} := |C_{bB}|\cdot r_0\cdot A_g\cdot\sqrt{\sigma_{ez}\cdot\sigma_T}$ $M_{e.cr.C\_B} = 6256.9\,kN\cdot m$

Relative slenderness: $\lambda_{rLT.C\_B} := \begin{cases}\sqrt{\frac{W_{eff.y.1}\cdot f_{yb}}{M_{e.cr.C\_B}}} & \text{if } M_{e.cr.C\_B} > 0\\ 0.2 & \text{if } M_{e.cr.C\_B} = 0\end{cases}$ $\lambda_{rLT.C\_B} = 0.1$

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

$\phi_{LT.C\_B} = 0.49$

Moment distribution: $\chi_{LT.C} := \begin{cases}\chi_{LT.C\_A} & \text{if Moment\_dis = "A"}\\ \chi_{LT.C\_B} & \text{if Moment\_dis = "B"}\end{cases}$ $\chi_{LT.C} = 1$

Chord at end panel: highest shear force: $V_{Ed.max} := \pi\cdot\frac{M_{z.Ed.C}}{L_z}$ $V_{Ed.max} = 5.95\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 2.98\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.009$ << 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:* $M_{z.ch} = 1.12\,kN\cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C}\cdot\sin\left(\frac{\pi\cdot cc_{batt}}{L_z}\right)$ $M_{z.Ed.C.end} = 2.84\,kN\cdot m$ moment due to bow imperfection att end panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end}\cdot\frac{(h_0\cdot A_{ch})}{(2\cdot I_{eff})}$ $N_{ch.end} = 459.53\,kN$

$b = 100\,mm \quad h = 170\,mm \quad c = 38\,mm \quad t = 6\,mm \quad f_{yb} = 500\frac{N}{mm^2} \quad f_u = 550\frac{N}{mm^2} \quad \gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

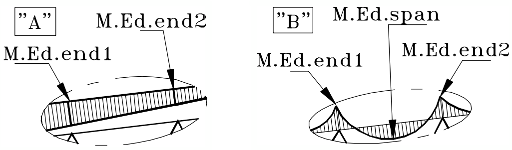
Interaction factors kij for members susceptible to torsional deforations: EN 1993-1-1, Annex B, Table B.2

$n_{y.C} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(L_y)\cdot N_{c.Rk}} \quad n_{y.C} = 0.39 \qquad n_{z.C} := \frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(L_{z.C})\cdot N_{c.Rk}} \quad n_{z.C} = 0.38 \qquad \chi_{y.FB}(L_y) = 0.96 \quad \chi_{z.FB}(L_{z.C}) = 1$

Equivalent uniform moment factors:

$C_{my} = 0.7 \quad C_{mLT} = 0.7 \quad \psi_{m.z} := 1$

$C_{mz.C} := \max[(0.6 + 0.4\cdot\psi_{m.z}), 0.4] \quad C_{mz.C} = 1$



$M_{Ed.end.1} = 15.9\,kN\cdot m$

$M_{Ed.end.2} = -14.3\,kN\cdot m$

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

$\lambda_{r.y.FB}(L_y) = 0.31 \qquad k_{yy.C} := \min[C_{my}\cdot(1 + 0.6\cdot\lambda_{r.y.FB}(L_y)\cdot n_{y.C}), C_{my}\cdot(1 + 0.6\cdot n_{y.C})] \qquad k_{yy.C} = 0.75$

$\lambda_{r.z.FB}(L_{z.C}) = 0.21 \qquad k_{zz.C} := \min[C_{mz.C}\cdot(1 + 0.6\cdot\lambda_{r.z.FB}(L_{z.C})\cdot n_{z.C}), C_{mz.C}\cdot(1 + 0.6\cdot n_{z.C})] \qquad k_{zz.C} = 1.05$

$k_{zy.C} := \max\left(1 - \frac{0.05\cdot\lambda_{r.z.FB}(L_{z.C})}{C_{my} - 0.25}\cdot n_{z.C}, 1 - \frac{0.05}{C_{mLT} - 0.25}\cdot n_{z.C}\right) \qquad k_{zy.C} = 0.99 \qquad k_{yz.C} := k_{zz.C}$

Controll single C-profile between battens:

max moment y-y: $M_{y.max.C} := 0.5\cdot\max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) \qquad M_{y.max.C} = 7.95\,kN\cdot m$

max moment z-z: $M_{z.C} = 1.12\,kN\cdot m \qquad \Delta M_{z.shift} := |\Delta e_N\cdot N_{ch.end}| \qquad \Delta M_{z.shift} = 0.03\,kN\cdot m$

max chord force from above: $N_{Ed.C} = 459.53\,kN$

Buckling moment resistance y-y
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C}\cdot M_{ycRk} \quad \chi_{LT.C} = 1 \quad M_{y.Rk.C} = 67.22\,kN\cdot m$

Buckling resistance moment z-z
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min(M_{z1cRk}, M_{z2cRk}) \quad M_{z.Rk.C} = 29.95\,kN\cdot m$

Resistance for one profile buckling mode flexural buckling-: y-y $L_y = 1.35\,m \quad \chi_{y.FB}(L_y) = 0.96 \quad N_{b.Rk.y.FB}(L_y) = 1168.55\,kN$

Resistance for one profile buckling mode flexural buckling-: z-z $L_{z.C} = 0.5\,m \quad \chi_{z.FB}(L_{z.C}) = 1 \quad N_{b.Rk.z.FB}(L_{z.C}) = 1214.92\,kN$

Resistance for one profile buckling mode -torsional or torsional-flexural $L_{T.C} = 0.35\,m \quad \chi_{TF}(L_{T.C}, L_y) = 0.95 \quad N_{b.Rk.TF}(L_{T.C}, L_y) = 1151.1\,kN$

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

At mid-span of built-up member: $\frac{N_{ch.Ed}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{|\Delta M_{z.shift}|\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.50$ <1.0

In end panel of built-up member: $\frac{N_{ch.end}\cdot\gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C}\cdot\gamma_{M0}}{M_{ycRk}} + \frac{(M_{z.C} + |\Delta M_{z.shift}|)\cdot\gamma_{M0}}{M_{z.Rk.C}} = 0.53$ <1.0

Combined bending and axial compression EN 1993-1-1: 6.3.3 (4): $\frac{\max(N_{ch.Ed}, N_{ch.end})\cdot\gamma_{M1}}{\min(\chi_{y.FB}(L_y), \chi_{z.FB}(L_{z.C}), \chi_{TF}(L_{T.C}, L_y))\cdot N_{c.Rk}} = 0.4$ <1.0

Combined bending + axial compression EN 1993-1-1: 6.3.3 (4): Buckling about y-y

At mid-span of built-up member: $\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{y.FB}(L_y)\cdot N_{c.Rk}} + k_{yy.C}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C}\cdot\frac{|\Delta M_{z.shift}|\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.48$ <1.0

In end panel of built-up member: $\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{y.FB}(L_y)\cdot N_{c.Rk}} + k_{yy.C}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{yz.C}\cdot\frac{(M_{z.C} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.52$ <1.0

Combined bending + axial compression EN 1993-1-1: 6.3.3 (4): Buckling about z-z

At mid-span of built-up member: $\frac{N_{ch.Ed}\cdot\gamma_{M1}}{\chi_{z.FB}(L_{z.C})\cdot N_{c.Rk}} + k_{zy.C}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C}\cdot\frac{|\Delta M_{z.shift}|\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.5$ <1.0

In end panel of built-up member: $\frac{N_{ch.end}\cdot\gamma_{M1}}{\chi_{z.FB}(L_{z.C})\cdot N_{c.Rk}} + k_{zy.C}\cdot\frac{M_{y.max.C}\cdot\gamma_{M1}}{\chi_{LT.C}\cdot M_{ycRk}} + k_{zz.C}\cdot\frac{(M_{z.C} + |\Delta M_{z.shift}|)\cdot\gamma_{M1}}{M_{z.Rk.C}} = 0.54$ <1.0

Check M, V och N acc. to EC

$Country := "Cz"$

$PR \equiv "CC170x6"$

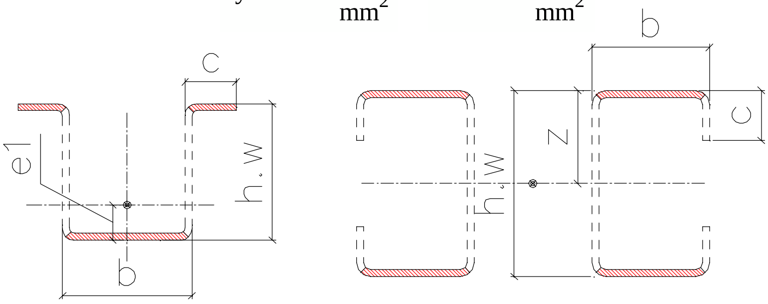
Profile data for a single profile

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

$h_w = 170\,mm$ $b_c = 100\,mm$ $c = 38\,mm$ $t = 6\,mm$ $e_1 = 39.18\,mm$ $A_g = 2449.59\,mm^2$

$N_{cRk} = 1217.47\,kN$ $M_{Rk} = 67.22\,kN \cdot m$ $f_{yb} = 500 \frac{N}{mm^2}$ $f_u = 550 \frac{N}{mm^2}$

The beam is H-profile ("H") or double C-profile ("C")



Hole diameter: $d_o := 12.5 \cdot mm$

Number of bolts at the cross-section in ONE web: $n_{cross} := 4$

Number screws and diameter i FLANGES: $X_{h.fl} := 0$ $d_{o.fl} := 0 \cdot mm$

Momentcap.one profile:

Axialforcecapacity one profile:

Axialforce acting as TENTION : "YES" or "NO" $N_{drag} := "YES"$

Netto area for shearforce:

$$A_w := \begin{cases} \left[\left[(h_w - t) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} = 0 \cdot mm \\ \left[\left[(L_{II} - t) \cdot t - n_{cross} \cdot d_o \cdot t\right] \cdot 2\right] & \text{if } L_{II} \neq 0 \cdot mm \end{cases}$$

$A_w = 1368.00\,mm^2$

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: $M_{Rd} := \frac{X \cdot M_{Rk}}{\gamma_{M0}}$ $M_{Rd} = 134.44\,kN \cdot m$

Axialforce capacity compresion member: $N_{Rd.1} := \frac{N_{cRk} \cdot X}{\gamma_{M0}}$ (6.2.4) $N_{Rd.1} = 2434.9\,kN$

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)) $N_{Rd.t.a} = 945.82\,kN$

For material >= 5.0mm : $N_{Rd.t.b} := \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2.t}}$ $N_{Rd.t.b} = 851.2\,kN$ (EN1993-1-1: 6.2.3) $\gamma_{M2.t} = 1.25$

$$N_{Rd.t} := \left(\begin{cases} N_{Rd.t.a} & \text{if } t \leq 4 \cdot mm \\ N_{Rd.t.b} & \text{if } t \geq 5 \cdot mm \end{cases}\right) \cdot X$$

$N_{Rd.t} = 1702.48\,kN$

Normalkraftscapacity:

$$N_{Rd} := \begin{cases} N_{Rd.1} & \text{if } N_{drag} = "NO" \\ \min(N_{Rd.1}, N_{Rd.t}) & \text{if } N_{drag} = "YES" \end{cases}$$

$N_{Rd} = 1702.48\,kN$

Shearforce capacity:

$\lambda_{wh} := 0.346 \cdot \frac{h_w - t}{t} \cdot \sqrt{\frac{f_{yb}}{E}}$

$\lambda_{wh} = 0.46$

$$f_{vb} := \begin{cases} \frac{1}{\sqrt{3}} \cdot f_{yb} & \text{if } \lambda_{wh} \leq 0.83 \\ 0.48 \cdot \frac{f_{yb}}{\lambda_{wh}} & \text{if } 0.83 < \lambda_{wh} < 1.40 \\ 0.67 \cdot \frac{f_{yb}}{\lambda_{wh}^2} & \text{if } \lambda_{wh} \geq 1.40 \end{cases}$$

$f_{vb} = 288.68 \frac{N}{mm^2}$

$V_{Rd} := \frac{A_w \cdot f_{vb}}{\gamma_{M0}}$

$V_{Rd} = 394.91\,kN$

elast. momentcap.: $M_{elRd} := M_{Rd}$

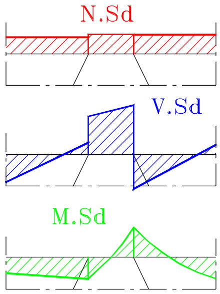
$\theta_{pl} := 1.0$ faktor M_{el} -> M_{pl}

Section value for interaction formula M + N + V:
Momentresistance of a cross-section consisting only flanges: $M_{flRd} = 94.93\,kN \cdot m$

MAX

Element Nr:

$N_{Sd} := 892 \cdot kN$ $V_{Sd} := 280 \cdot kN$ $M_{Sd} := 16 \cdot kN \cdot m$



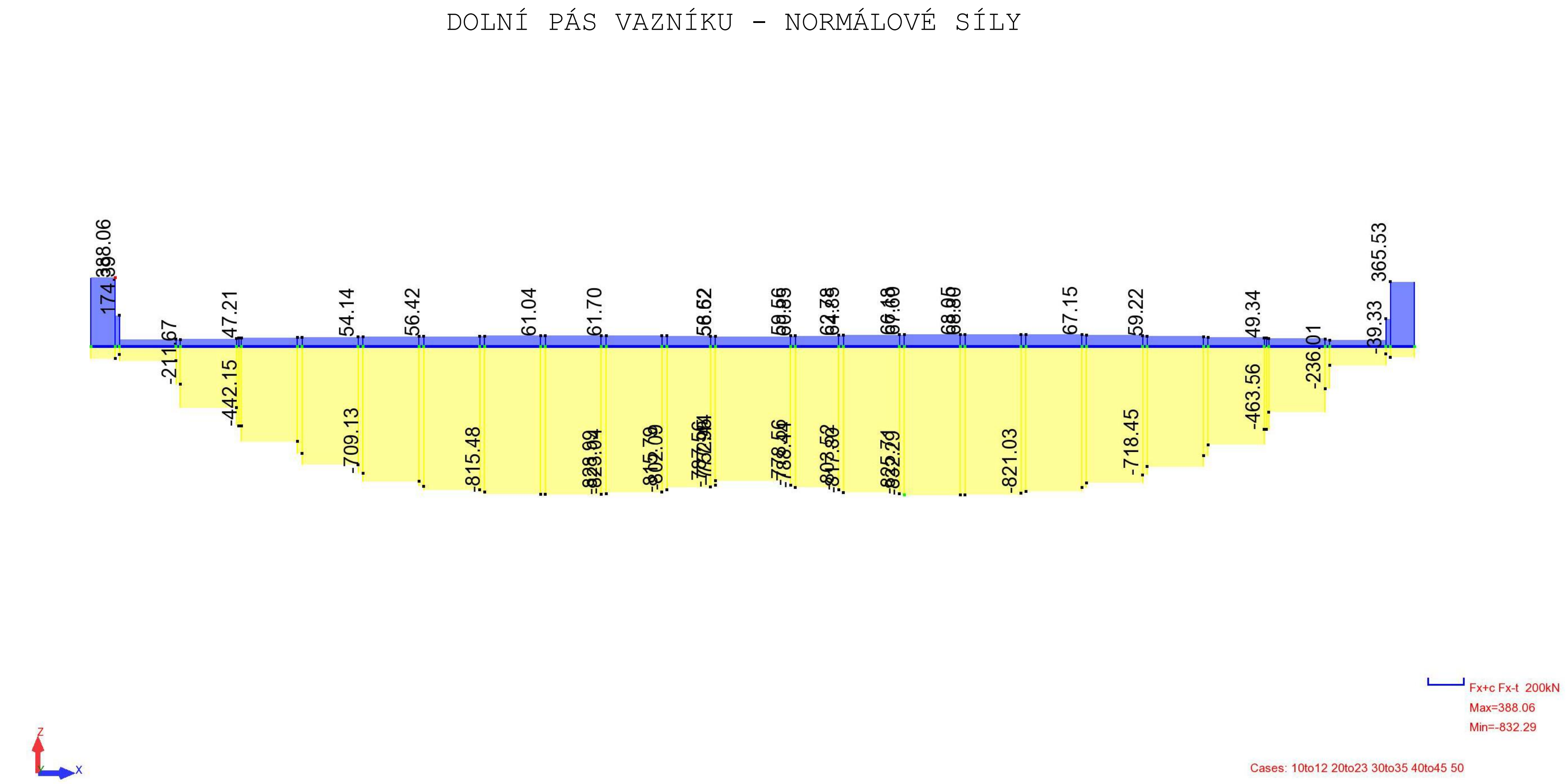
$\frac{|V_{Sd}|}{V_{Rd}} = 0.71$ <1.0 (EN 1993-1-1: 6.2.6)

$\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 0.64$ <1.0 (EN 1993-1-3 6.1.8+6.1.9)

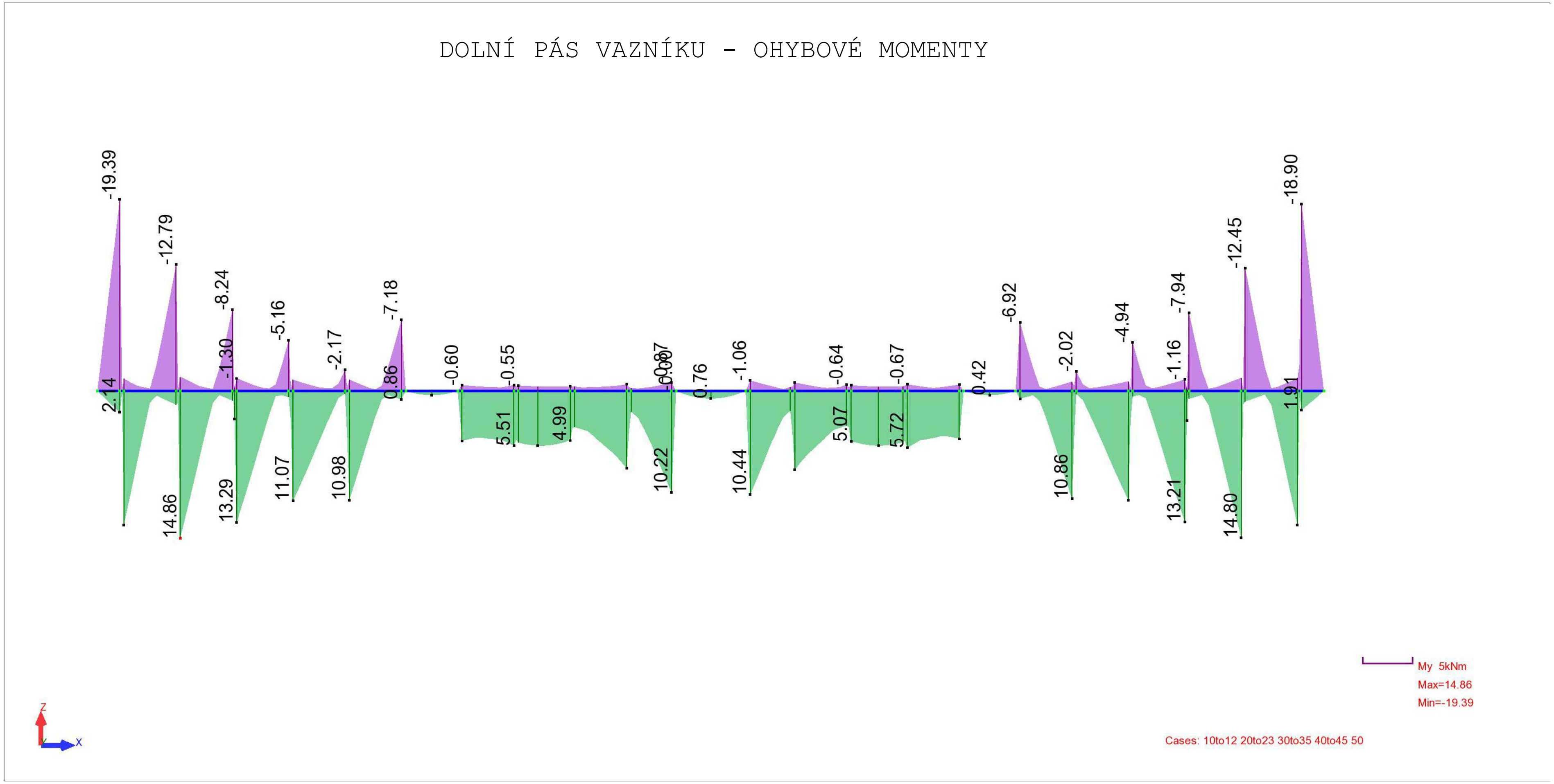
Ratio for check
Moment + Axialforce + Shear force

$M\_N\_V_{EC} = 0.69$ <1.0 (EN 1993-1-3 6.1.10)

View:1 - FX;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50



View:1 - MY;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50



POSUDEK DOLNÍHO PÁSU VAZNÍKU

Double-C Truss bottom member

$d := 100 \cdot mm$

$h \equiv 170 \cdot mm$ $b \equiv 97 \cdot mm$ $c \equiv 38 \cdot mm$ $t \equiv 5 \cdot mm$ $\gamma_{M0} \equiv 1.0$

$h_{batt} := 200 \cdot mm$ $t_{batt} := 4 \cdot mm$ $kb = 401.63\ mm$ $\gamma_{M1} \equiv 1.0$

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

$A_{gg} := 2 \cdot A_g$ $A_{gg} = 4.071 \times 10^3\ mm^2$

$I_{zz} = 3.7 \times 10^7\ mm^4$ $W_{zz.eff} = 2.41 \times 10^5\ mm^3$

$I_{yy} := 2 \cdot I_y$ $I_{yy} = 1.84 \times 10^7\ mm^4$

$W_{yy} := 2 \cdot W_y$ $W_{yy} = 2.23 \times 10^5\ mm^3$

$W_{yy.eff} := 2 \cdot W_{eff.y.1}$ $W_{yy.eff} = 2.23 \times 10^5\ mm^3$

$W_{zz} := \dfrac{I_{zz}}{b + \dfrac{d}{2}}$ $W_{zz} = 2.52 \times 10^5\ mm^3$

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

$M_{yy.cRk} = 111.31\ kN \cdot m$

$M_{ycRk} = 55.66\ kN \cdot m$

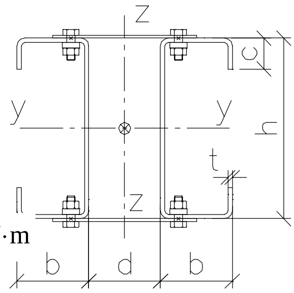
$N_{cc.Rk} := 2 \cdot N_{c.Rk}$

$N_{cc.Rk} = 1911.28\ kN$

$f_{yb} = 500 \dfrac{N}{mm^2}$

$f_u = 550 \dfrac{N}{mm^2}$

$M_{zz.cRk} = 120.72\ kN \cdot m$



$i_{yy} := \sqrt{\dfrac{I_{yy}}{A_{gg}}}$ $i_{yy} = 67.29\ mm$ $i_{zz} := \sqrt{\dfrac{I_{zz}}{A_{gg}}}$ $i_{zz} = 95.33\ mm$

torsion_plate_truss = "NO"

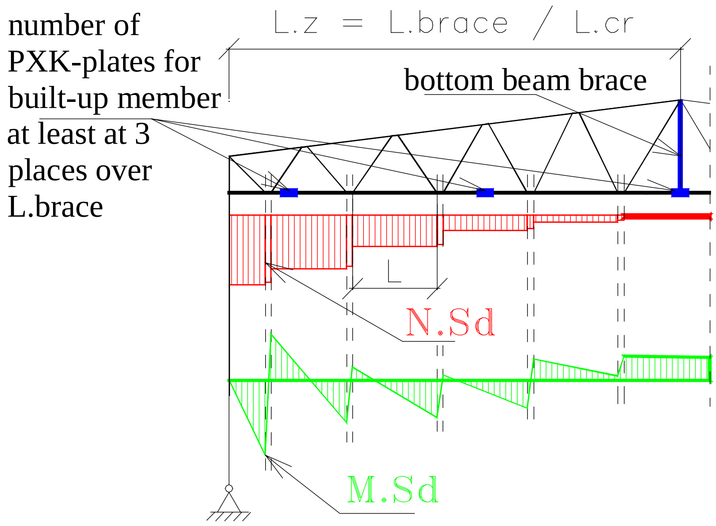
Stresses and global geometry:

Beam is designed as uniform built up member: YES or NO: built_up ≡ "YES"

Number of pair battens per **braced length L_z**: $n_{batt} := 4$
placed att ends and spaced out evenly **3 at least**

Buckling lengths:

$L := 1.5 \cdot m$	Length between diagonals
$L_y := 0.9 \cdot L$	Buckling length y-y
$L_z := 6 \cdot m$	Buckling length z-z
$L_{LT} := 1.0 \cdot L$	Length for LT-buckling



$L_y = 1.35\ m$	Flexural buckling axis y-y
$L_z = 6\ m$	Flexural buckling axis z-z
$L_{LT} = 1.5\ m$	Lateral-torsional buckling

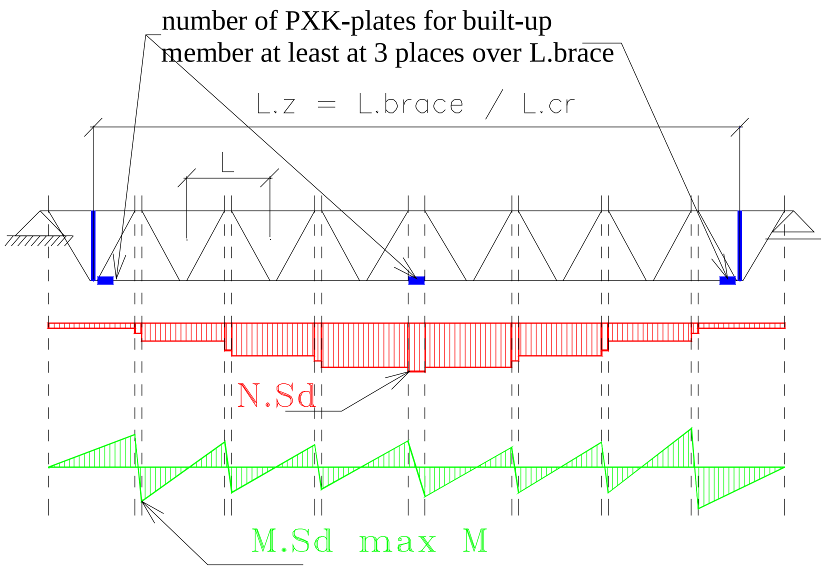
For single C-profile:

$L_{z.C} = 1.5\ m$	FB z-z for single profile
$L_{T.C} = 1.5\ m$	Torsional buckling

stresses:

$M_{Ed.end.1} := 20 \cdot kN \cdot m$ sign: (+) or (-)

$N_{Ed} := 388.1 \cdot kN$



$\Delta M_{z.shift} := \Delta e_N \cdot N_{Ed}$ $\Delta M_{z.shift} = -0.33\ kN \cdot m$

$M_{z.Ed} := \left| \Delta M_{z.shift} \right|$ shift of neutral axis for member in compression

$b = 97\,mm \qquad h = 170\,mm \qquad c = 38\,mm \qquad t = 5\,mm \qquad f_{yb} = 500\,\frac{N}{mm^2} \qquad f_u = 550\,\frac{N}{mm^2} \qquad \gamma_{M0} = 1.00 \qquad \gamma_{M1} = 1.00$

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

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

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yb}}} \qquad \lambda_1 = 64.38 \qquad \lambda_{r.y.FBcc} := \frac{L_y}{i_{yy}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1} \qquad \lambda_{r.y.FBcc} = 0.3$

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

$\phi_{y.FB.cc} := 0.5 \cdot \left[1 + \alpha_y \cdot (\lambda_{r.y.FBcc} - 0.2) + \lambda_{r.y.FBcc}^2\right] \qquad \chi_{y.FBcc} := \min\left(\frac{1}{\phi_{y.FB.cc} + \sqrt{\phi_{y.FB.cc}^2 - \lambda_{r.y.FBcc}^2}}, 1\right) \qquad \chi_{y.FBcc} = 0.98$

$\phi_{y.FB.cc} = 0.56$

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

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

non-dimensional slenderness:

$\lambda_1 = 64.38 \qquad \lambda_{r.z.FBcc} := \frac{L_z}{i_{zz}} \cdot \sqrt{\frac{2 \cdot A_{eff}}{2 \cdot A_g}} \cdot \frac{1}{\lambda_1} \qquad \lambda_{r.z.FBcc} = 0.95$

$\phi_{z.FB.cc} := 0.5 \cdot \left[1 + \alpha_z \cdot (\lambda_{r.z.FBcc} - 0.2) + \lambda_{r.z.FBcc}^2\right] \qquad \chi_{z.FBcc} := \min\left(\frac{1}{\phi_{z.FB.cc} + \sqrt{\phi_{z.FB.cc}^2 - \lambda_{r.z.FBcc}^2}}, 1\right) \qquad \chi_{z.FBcc} = 0.63$

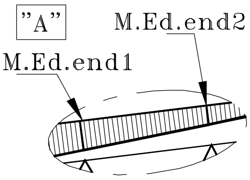
$\phi_{z.FB.cc} = 1.C$

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 critikal buckling momtent M_{cr} : given in German ENV 1993-1-1 Annex F.

$k_{M.cr} := 1.0$ (hinged at ends) $\qquad k_{w.M.cr} := 1.0$ (no special wrap restraints at ends) $\qquad$ Moment_dis = "A"

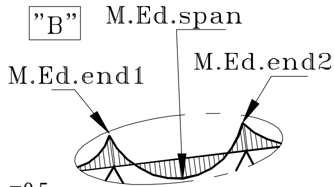
$\psi_m := \begin{cases} \frac{M_{Ed.end.2}}{M_{Ed.end.1}} & \text{if } |M_{Ed.end.1}| \ge |M_{Ed.end.2}| \\ \frac{M_{Ed.end.1}}{M_{Ed.end.2}} & \text{if } |M_{Ed.end.1}| < |M_{Ed.end.2}| \end{cases}$



$C_{1.tr.A} := \min\left(1.88 - 1.40 \cdot \psi_m + 0.52 \cdot \psi_m^2, 2.7\right)$

$C_{1.tr.A} = 1$

$C_{1.tr.B} := 1.285$



$C_{1.tr} := \begin{cases} C_{1.tr.A} & \text{if Moment\_dis = "A"} \\ C_{1.tr.B} & \text{if Moment\_dis = "B"} \end{cases} \qquad C_{1.tr} = 1$

Elastic critical moment for lateral-torsional buckling:

$M_{cr.tr} := C_{1.tr} \cdot \frac{\pi^2 \cdot E \cdot I_{zz}}{(k_{M.cr} \cdot L_{LT})^2} \cdot \left[\left(\frac{k_{M.cr}}{k_{w.M.cr}}\right)^2 \cdot \frac{I_{\omega\omega}}{I_{zz}} + \frac{(k_{M.cr} \cdot L_{LT})^2 \cdot G \cdot I_{TT}}{\pi^2 \cdot E \cdot I_{zz}}\right]^{0.5} \qquad M_{cr.tr} = 1247.6\,kN \cdot m$

Relative slenderness: $\lambda_{rLT.cc} := \sqrt{\frac{2 \cdot W_{eff.y.1} \cdot f_{yb}}{M_{cr.tr}}} \qquad \lambda_{rLT.cc} = 0.3$

Imperfection factor α relating to bucking curve b: $\qquad \alpha_{LT} := 0.34$

$\phi_{LT.cc} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{rLT.cc} - 0.2) + \lambda_{rLT.cc}^2\right]$

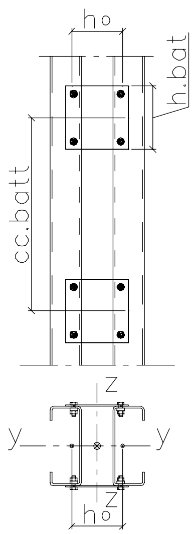
Reduction buckling factor $\qquad \chi_{LT.y} := \min\left(\frac{1}{\phi_{LT.cc} + \sqrt{\phi_{LT.cc}^2 - \lambda_{rLT.cc}^2}}, 1\right) \qquad \chi_{LT.y} = 0.96$

$\phi_{LT.cc} = 0.56$

$\chi_{LT} := \begin{cases} \chi_{LT.y} & \text{if } I_{yy} \ge I_{zz} \\ 1 & \text{otherwise} \end{cases}$

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

bow imperfection: $e_0 := \frac{L_z}{500} \qquad e_0 = 12\,mm$



Effective second moment of area of battended built-up member:

$A_{ch} := A_g$: area of one chord

$h_0 := d + 2 \cdot e_1 \quad h_0 = 175.8\,mm$ 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

$i_{0.CC} := \sqrt{\frac{I_{l.CC}}{2 \cdot A_{ch}}} \qquad \lambda_{CC} := \frac{L_z}{i_{0.CC}} \qquad \lambda_{CC} = 62.94$

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

$\mu_{CC} := \begin{cases} 0 & \text{if } \lambda_{CC} \ge 150 \\ \left(2 - \frac{\lambda_{CC}}{75}\right) & \text{if } 75 < \lambda_{CC} < 150 \\ 1.0 & \text{if } \lambda_{CC} \le 75 \end{cases} \qquad \mu_{CC} = 1$

$I_{l.CC} = 3.7 \times 10^7\,mm^4$

$r_{CC} := 2$

$b = 97\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu_{CC} \cdot I_{ch}$ $I_{eff} = 3.7 \times 10^7\,mm^4$ effective I of built-up member

Shear stiffnes *EN 1993-1-1:6.4.3.1:(2)*

$I_b := \frac{t_{batt} \cdot h_{batt}^3}{12}$ I of batten $n_{batt.1} := 2$ number of planes of lacings

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

$S_v = 5120\,kN$

effective critical force of built-up member: $N_{cr.CC} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_z^2}$ $N_{cr.CC} = 2130\,kN$

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

moment z-z without second order effects $M_{z.Ed.I} := 0 \cdot kN \cdot m$

moment with second order effects

$$M_{z.Ed.C} := \begin{cases} \left|\frac{N_{Ed} \cdot e_0 + M_{z.Ed.I}}{1 - \frac{N_{Ed} \cdot \gamma_{M1}}{N_{cr.CC}} - \frac{N_{Ed} \cdot \gamma_{M1}}{S_v}}\right| & \text{if built\_up = "YES"} \\ (0 \cdot kN \cdot m) & \text{if built\_up = "NO"} \end{cases}$$

$M_{z.Ed.C} = 6.3\,kN \cdot m$

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:

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

$N_{ch.Ed} = 224.4\,kN$

Interaktion formulae 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

$C_{my} = 1$ $C_{mLT} := C_{my}$ $n_{y.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}}$ $n_{y.2} = 0.21$ $n_{z.2} := \frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}}$ $n_{z.2} = 0.32$ $C_{mz.2} := 1$

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

$k_{yy} := \min\left[C_{my} \cdot \left(1 + 0.6 \cdot \lambda_{r.y.FBcc} \cdot n_{y.2}\right), C_{my} \cdot \left(1 + 0.6 \cdot n_{y.2}\right)\right]$ $k_{yy} = 1.04$

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

$k_{zz.2} := \min\left[C_{mz.2} \cdot \left(1 + 0.6 \cdot \lambda_{r.z.FBcc} \cdot n_{z.2}\right), C_{mz.2} \cdot \left(1 + 0.6 \cdot n_{z.2}\right)\right]$ $k_{zz.2} = 1.18$ $k_{yz.2} := k_{zz.2}$

Reduction factor for L-T buckling: $\chi_{LT} = 1$

Reduction factor for F- buckling: $\chi_{y.FBcc} = 0.98$ $\chi_{z.FBcc} = 0.63$

Combined bending an axial compression EN 1993-1-3: 6.2.5 (2):

$$\frac{N_{Ed} \cdot \gamma_{M0}}{2 \cdot N_{c.Rk}} + \frac{\max\left(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|\right) \cdot \gamma_{M0}}{2 \cdot M_{ycRk}} + \frac{|M_{z.Ed.C}| \cdot \gamma_{M0}}{M_{zz.cRk}} = 0.43$$

<1.0

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

Evading in y-y:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{y.FBcc} \cdot N_{cc.Rk}} + k_{yy} \cdot \frac{\max\left(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{yz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.46$$

<1.0

Evading in z-z:

$$\frac{N_{Ed} \cdot \gamma_{M1}}{\chi_{z.FBcc} \cdot N_{cc.Rk}} + k_{zy} \cdot \frac{\max\left(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|\right) \cdot \gamma_{M1}}{\chi_{LT} \cdot M_{yy.cRk}} + k_{zz.2} \cdot \frac{|M_{z.Ed.C}| \cdot \gamma_{M1}}{M_{zz.cRk}} = 0.56$$

<1.0

$b = 97\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 5\,mm$ $f_{yb} = 500\frac{N}{mm^2}$ $f_u = 550\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

The chords are checked for the actual moments and forces according to EN 1993-1-1:6.4.3.1 (1) if the truss beam is designed as built-up member. Otherwise: EN 1993-1-1: 6.3.3

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

$$M_{y.i}(x_i) := \frac{M_{Ed.end.2} - M_{Ed.end.1}}{L} \cdot x_i + M_{Ed.end.1}$$

For moment distribution accord. to model "A"

Moment at quarter point of unbraced segment: $x_{1\_4} := L \cdot 0.25$ $M_{y.AA} := M_{y.i}(x_{1\_4})$ $M_{y.AA} = 20\,kN \cdot m$

Moment at centerline of unbraced segment: $x_{1\_2} := L \cdot 0.5$ $M_{y.BA} := M_{y.i}(x_{1\_2})$ $M_{y.BA} = 20\,kN \cdot m$

Moment at 3/4-point of unbraced segment: $x_{3\_4} := L \cdot 0.75$ $M_{y.CA} := M_{y.i}(x_{3\_4})$ $M_{y.CA} = 20\,kN \cdot m$

$$C_{bA} := \frac{12.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3 \cdot M_{y.AA} + 4 \cdot M_{y.BA} + 3 \cdot M_{y.CA}}$$

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

$z_{0.cr} := 0 \cdot mm$ (coord. shear center)

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

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

$\sigma_{ez} := \frac{\pi^2 \cdot E}{\left(\frac{1 \cdot L_{z.C}}{i_z}\right)^2}$ $\sigma_{ez} = 1257.51\frac{N}{mm^2}$

$\sigma_T := \frac{1}{A_g \cdot r_0^2} \cdot \left(G \cdot I_T + \frac{\pi^2 \cdot E \cdot I_\omega}{L_{T.C}^2}\right)$ $\sigma_T = 752.78\frac{N}{mm^2}$

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

$M_{e.cr.C\_A} := |C_{bA}| \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T}$ $M_{e.cr.C\_A} = 229.5\,kN \cdot m$

Relative slenderness:

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

$\lambda_{rLT.C\_A} = 0$

$\phi_{LT.C\_A} := 0.5 \cdot \left[1 + \alpha_{LT.C} \cdot (\lambda_{rLT.C\_A} - 0.2) + \lambda_{rLT.C\_A}^2\right]$ $\phi_{LT.C\_A} = 0.67$

$\chi_{LT.C\_A} := \min\left(\frac{1}{\phi_{LT.C\_A} + \sqrt{\phi_{LT.C\_A}^2 - \lambda_{rLT.C\_A}^2}}, 1\right)$ $\chi_{LT.C\_A} = 0.89$

For moment distribution accord. to model "B"

Moment at quarter point of unbraced segment: $M_{y.AB} := \frac{(M_{Ed.span} - M_{Ed.end.1}) \cdot L \cdot 0.25}{L \cdot 0.5} + M_{Ed.end.1}$ $M_{y.AB} = 20\,kN \cdot m$

Moment at centerline of unbraced segment: $M_{y.BB} := M_{Ed.span}$ $M_{y.BB} = 20\,kN \cdot m$

Moment at 3/4-point of unbraced segment: $M_{y.CB} := \frac{(M_{Ed.span} - M_{Ed.end.2}) \cdot L \cdot 0.25}{L \cdot 0.5} + M_{Ed.end.2}$ $M_{y.CB} = 20\,kN \cdot m$

$$C_{bB} := \frac{12.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|)}{2.5 \cdot \max(|M_{Ed.end.1}|, |M_{Ed.end.2}|, |M_{Ed.span}|) + 3 \cdot M_{y.AB} + 4 \cdot M_{y.BB} + 3 \cdot M_{y.CB}}$$

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

$M_{e.cr.C\_B} := |C_{bB}| \cdot r_0 \cdot A_g \cdot \sqrt{\sigma_{ez} \cdot \sigma_T}$ $M_{e.cr.C\_B} = 229.5\,kN \cdot m$

Relative slenderness:

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

$\lambda_{rLT.C\_B} = 0.49$

$\phi_{LT.C\_B} := 0.5 \cdot \left[1 + \alpha_{LT.C} \cdot (\lambda_{rLT.C\_B} - 0.2) + \lambda_{rLT.C\_B}^2\right]$ $\chi_{LT.C\_B} := \min\left(\frac{1}{\phi_{LT.C\_B} + \sqrt{\phi_{LT.C\_B}^2 - \lambda_{rLT.C\_B}^2}}, 1\right)$ $\chi_{LT.C\_B} = 0.89$

$\phi_{LT.C\_B} = 0.67$

Moment distribution:

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

$\chi_{LT.C} = 0.89$

Chord at end panel: highest shear force: $V_{Ed.max} := \pi \cdot \frac{M_{z.Ed.C}}{L_z}$ $V_{Ed.max} = 3.29\,kN$ $V_{ch} := \frac{V_{Ed.max}}{r_{CC}}$ $V_{ch} = 1.64\,kN$ $\frac{V_{ch}}{V_{bb.Rd}} = 0.006$ << 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:* $M_{z.ch} = 1.23\,kN \cdot m$

maximum chord force at batten (end): $M_{z.Ed.C.end} := M_{z.Ed.C} \cdot \sin\left(\frac{\pi \cdot cc_{batt}}{L_z}\right)$ $M_{z.Ed.C.end} = 4.44\,kN \cdot m$ moment due to bow imperfection att end panel

maximum compression force: $N_{ch.end} := \frac{N_{Ed}}{r_{CC}} + M_{z.Ed.C.end} \cdot \frac{(h_0 \cdot A_{ch})}{(2 \cdot I_{eff})}$ $N_{ch.end} = 215.51\,kN$

$b = 97\,mm$ $h = 170\,mm$ $c = 38\,mm$ $t = 5\,mm$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$ $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

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

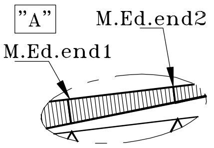
$n_{y.C} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}}$ $n_{y.C} = 0.23$

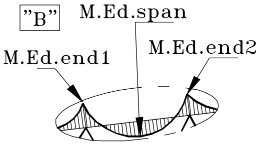
$n_{z.C} := \frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}}$ $n_{z.C} = 0.27$

$\chi_{y.FB}(L_y) = 0.96$

$\chi_{z.FB}(L_{z.C}) = 0.83$

Equivalent uniform moment factors:





$C_{my} = 1$ $C_{mLT} = 1$ $\psi_{m.z} := 1$

$C_{mz.C} := \max\left[\left(0.6 + 0.4 \cdot \psi_{m.z}\right), 0.4\right]$ $C_{mz.C} = 1$

$M_{Ed.end.1} = 20\,kN \cdot m$

$M_{Ed.end.2} = 20\,kN \cdot m$

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

$\lambda_{r.y.FB}(L_y) = 0.3$ $k_{yy.C} := \min\left[C_{my} \cdot \left(1 + 0.6 \cdot \lambda_{r.y.FB}(L_y) \cdot n_{y.C}\right), C_{my} \cdot \left(1 + 0.6 \cdot n_{y.C}\right)\right]$ $k_{yy.C} = 1.04$

$\lambda_{r.z.FB}(L_{z.C}) = 0.61$ $k_{zz.C} := \min\left[C_{mz.C} \cdot \left(1 + 0.6 \cdot \lambda_{r.z.FB}(L_{z.C}) \cdot n_{z.C}\right), C_{mz.C} \cdot \left(1 + 0.6 \cdot n_{z.C}\right)\right]$ $k_{zz.C} = 1.1$

$k_{zy.C} := \max\left(1 - \frac{0.05 \cdot \lambda_{r.z.FB}(L_{z.C})}{C_{my} - 0.25} \cdot n_{z.C}, 1 - \frac{0.05}{C_{mLT} - 0.25} \cdot n_{z.C}\right)$ $k_{zy.C} = 0.99$ $k_{yz.C} := k_{zz.C}$

Controll single C-profile between battens:

max moment y-y: $M_{y.max.C} := 0.5 \cdot \max\left(\left|M_{Ed.end.1}\right|, \left|M_{Ed.end.2}\right|, \left|M_{Ed.span}\right|\right)$ $M_{y.max.C} = 10\,kN \cdot m$

max moment z-z: $M_{z.C} = 1.23\,kN \cdot m$ $\Delta M_{z.shift} := \left|\Delta e_N \cdot N_{ch.end}\right|$ $\Delta M_{z.shift} = 0.18\,kN \cdot m$

Buckling moment resistance y-y
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{y.Rk.C} := \chi_{LT.C} \cdot M_{ycRk}$ $\chi_{LT.C} = 0.89$ $_{.Rk.C} = 49.4\,kN \cdot m$

Buckling resistance moment z-z
EN 1993-1-1:6.3 and EN 1993-1-3: 6.2.4: $M_{z.Rk.C} := \min\left(M_{z1cRk}, M_{z2cRk}\right)$ $M_{z.Rk.C} = 24.54\,kN \cdot m$

Resistance for one profile buckling mode flexural buckling-: y-y $L_y = 1.35\,m$ $\chi_{y.FB}(L_y) = 0.96$ $N_{b.Rk.y.FB}(L_y) = 920.65\,kN$

Resistance for one profile buckling mode flexural buckling-: z-z $L_{z.C} = 1.5\,m$ $\chi_{z.FB}(L_{z.C}) = 0.83$ $N_{b.Rk.z.FB}(L_{z.C}) = 794.62\,kN$

Resistance for one profile buckling mode -torsional or torsional-flexural $L_{T.C} = 1.5\,m$ $\chi_{TF}(L_{T.C}, L_y) = 0.71$ $N_{b.Rk.TF}(L_{T.C}, L_y) = 678.4\,kN$

Axial compression EN 1993-1-1: 6.3.1 (3): Buckling about relevant axis in mid-span or end panel of built-up member

$$\frac{\max\left(N_{ch.Ed}, N_{ch.end}\right) \cdot \gamma_{M1}}{\min\left(\chi_{y.FB}(L_y), \chi_{z.FB}(L_{z.C}), \chi_{TF}(L_{T.C}, L_y)\right) \cdot N_{c.Rk}} = 0.33$$ <1.0

Controll single C-profile (chords) in mid-span of member:

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

$$\frac{N_{ch.Ed} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{\left(\left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.42$$ <1.0

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

Evading in y-y:
$$\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}} + k_{yy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C} \cdot \frac{\left(\left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.46$$ <1.0

Evading in z-z:
$$\frac{N_{ch.Ed} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}} + k_{zy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C} \cdot \frac{\left(\left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.49$$ <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):

$$\frac{N_{ch.end} \cdot \gamma_{M0}}{N_{c.Rk}} + \frac{M_{y.max.C} \cdot \gamma_{M0}}{M_{ycRk}} + \frac{\left(M_{z.C} + \left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M0}}{M_{z.Rk.C}} = 0.46$$ <1.0

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

Evading in y-y:
$$\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{y.FB}(L_y) \cdot N_{c.Rk}} + k_{yy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{yz.C} \cdot \frac{\left(M_{z.C} + \left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.51$$ <1.0

Evading in z-z:
$$\frac{N_{ch.end} \cdot \gamma_{M1}}{\chi_{z.FB}(L_{z.C}) \cdot N_{c.Rk}} + k_{zy.C} \cdot \frac{M_{y.max.C} \cdot \gamma_{M1}}{\chi_{LT.C} \cdot M_{ycRk}} + k_{zz.C} \cdot \frac{\left(M_{z.C} + \left|\Delta M_{z.shift}\right|\right) \cdot \gamma_{M1}}{M_{z.Rk.C}} = 0.53$$ <1.0

Check M, V och N acc. to EC

Country := "Cz"

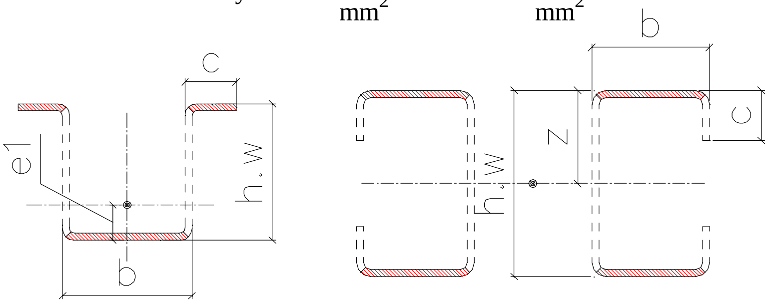
PR ≡ "CC170x5"

Profile data for a single profile

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

$h_w = 170\,mm$ $b_c = 97\,mm$ $c = 38\,mm$ $t = 5\,mm$ $e_1 = 37.88\,mm$ $A_g = 2035.62\,mm^2$

$N_{cRk} = 955.64\,kN$ $M_{Rk} = 55.66\,kN\cdot m$ $f_{yb} = 500\,\frac{N}{mm^2}$ $f_u = 550\,\frac{N}{mm^2}$



The beam is H-profile ("H") or double C-profile ("C")

Hole diameter: $d_o := 12.5\cdot mm$

Number of bolts at the cross-section in ONE web: $n_{cross} := 4$

Number screws and diameter i FLANGES: $X_{h.fl} := 0$ $d_{o.fl} := 0\cdot mm$

Momentcap.one profile:

Axialforcecapacity one profile:

Axialforce acting as TENTION : "YES" or "NO" $N_{drag} :=$ "YES"

Netto area for shearforce:

$$A_w := \begin{cases} \left[\left[(h_w - t)\cdot t - n_{cross}\cdot d_o\cdot t\right]\cdot 2\right] & \text{if } L_{II} = 0\cdot mm \\ \left[\left[(L_{II} - t)\cdot t - n_{cross}\cdot d_o\cdot t\right]\cdot 2\right] & \text{if } L_{II} \neq 0\cdot mm \end{cases}$$

$A_w = 1150.00\,mm^2$

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: $M_{Rd} := \frac{X\cdot M_{Rk}}{\gamma_{M0}}$ $M_{Rd} = 111.32\,kN\cdot m$

Axialforce capacity compresion member: $N_{Rd.1} := \frac{N_{cRk}\cdot X}{\gamma_{M0}}$ (6.2.4) $N_{Rd.1} = 1911.3\,kN$

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)) $N_{Rd.t.a} = 785.67\,kN$

For material >= 5.0mm : $N_{Rd.t.b} := \frac{0.9\cdot A_{net}\cdot f_u}{\gamma_{M2.t}}$ $N_{Rd.t.b} = 707.1\,kN$ (EN1993-1-1: 6.2.3) $\gamma_{M2.t} = 1.25$

$$N_{Rd.t} := \left(\begin{cases} N_{Rd.t.a} & \text{if } t \le 4\cdot mm \\ N_{Rd.t.b} & \text{if } t \ge 5\cdot mm \end{cases}\right)\cdot X$$

$N_{Rd.t} = 1414.21\,kN$

Normalkraftscapacity:

$$N_{Rd} := \begin{cases} N_{Rd.1} & \text{if } N_{drag} = \text{"NO"} \\ \min(N_{Rd.1}, N_{Rd.t}) & \text{if } N_{drag} = \text{"YES"} \end{cases}$$

$N_{Rd} = 1414.21\,kN$

Shearforce capacity:

$\lambda_{wh} := 0.346\cdot\frac{h_w - t}{t}\cdot\sqrt{\frac{f_{yb}}{E}}$

$\lambda_{wh} = 0.56$

$$f_{vb} := \begin{cases} \frac{1}{\sqrt{3}}\cdot f_{yb} & \text{if } \lambda_{wh} \le 0.83 \\ 0.48\cdot\frac{f_{yb}}{\lambda_{wh}} & \text{if } 0.83 < \lambda_{wh} < 1.40 \\ 0.67\cdot\frac{f_{yb}}{\lambda_{wh}^2} & \text{if } \lambda_{wh} \ge 1.40 \end{cases}$$

$f_{vb} = 288.68\,\frac{N}{mm^2}$

$V_{Rd} := \frac{A_w\cdot f_{vb}}{\gamma_{M0}}$

$V_{Rd} = 331.98\,kN$

elast. momentcap.: $M_{elRd} := M_{Rd}$

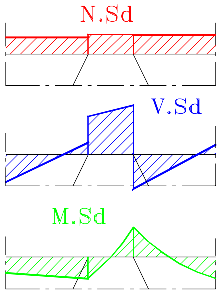
$\theta_{pl} := 1.0$ faktor M_{el} -> M_{pl}

Section value for interaction formula M + N + V:
Momentresistance of a cross-section consisting only flanges: $M_{flRd} = 77.67\,kN\cdot m$

MAX

Element Nr:

$N_{Sd} := 833 \cdot kN$ $V_{Sd} := 299 \cdot kN$ $M_{Sd} := 20 \cdot kN \cdot m$



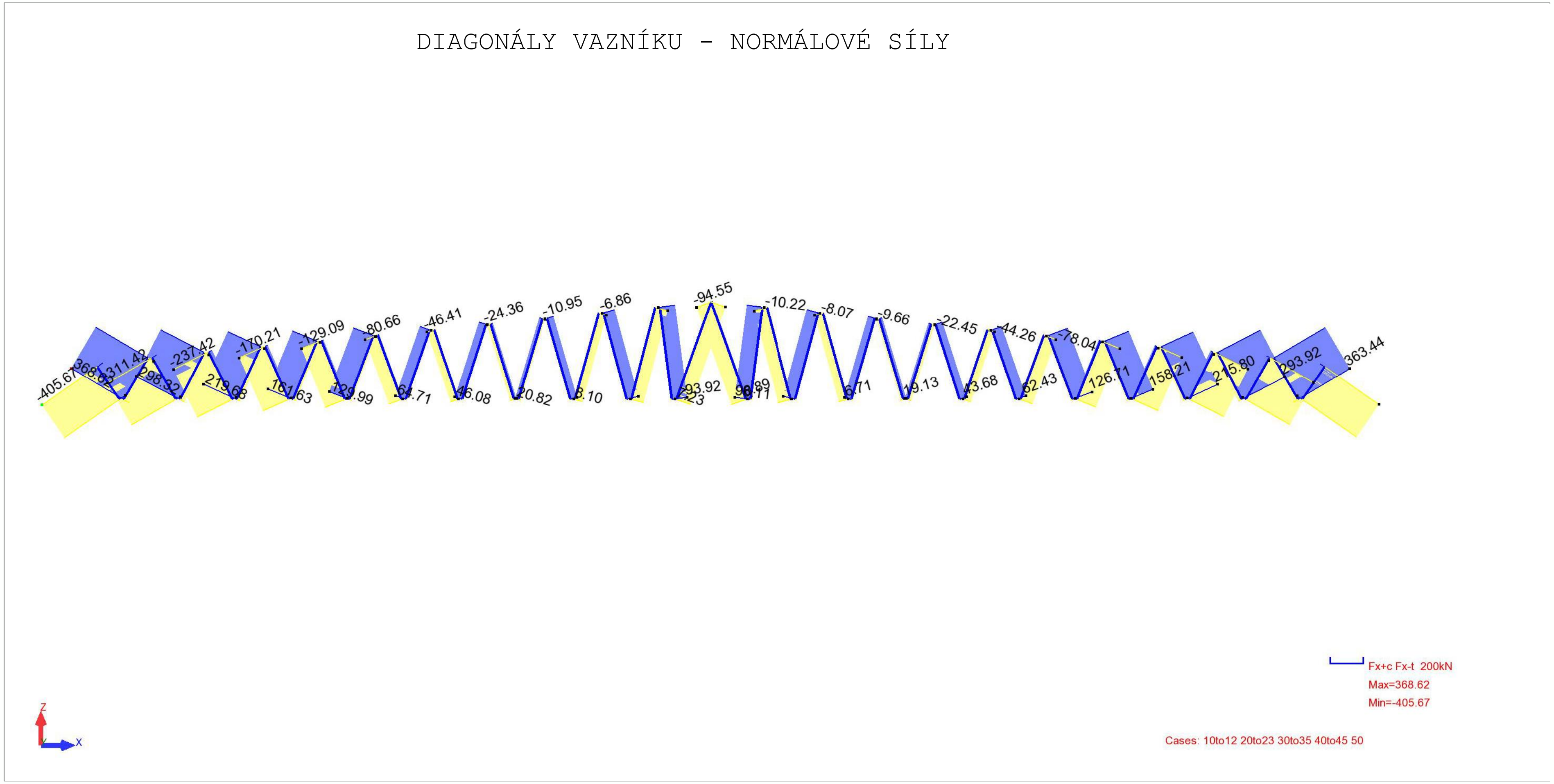
$\frac{|V_{Sd}|}{V_{Rd}} = 0.9$ <1.0 (EN 1993-1-1: 6.2.6)

$\frac{|N_{Sd}|}{N_{Rd}} + \frac{|M_{Sd}|}{M_{Rd}} = 0.77$ <1.0 (EN 1993-1-3 6.1.8+6.1.9)

Ratio for check
Moment + Axialforce + Shear force

$M\_N\_V_{EC} = 0.96$ <1.0 (EN 1993-1-3 6.1.10)

View:1 - FX;Reaction forces(kN,kN/m);Reaction moments(kN*m,kN*m/m); Cases: 10to12 20to23 30to35 40to45 50



Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek
Datum:	24.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka
				Norma projektu:	ČSN EN

Vazník - diagonály

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	2.756	HX500LAD	11.45	1580	1580	34
2	Strut	Strut 2 screw	C 100x85x5	2.756	HX500LAD	11.45	1580	1580	32
8	Strut	Strut 2 screw	C 100x85x5	2.48	HX500LAD	11.45	1580	1580	32
9	Strut	Strut 2 screw	C 100x85x5	2.48	HX500LAD	11.45	1580	1580	34
14	Strut	Strut 2 screw	C 100x82x4	2.529	HX420LAD	8.99	1166.8	1166.8	32
15	Strut	Strut 2 screw	C 100x82x4	2.529	HX420LAD	8.99	1166.8	1166.8	34
20	Strut	Strut 2 screw	C 100x82x4	2.392	HX420LAD	8.99	1166.8	1166.8	32
21	Strut	Strut 2 screw	C 100x82x4	2.392	HX420LAD	8.99	1166.8	1166.8	34
26	Strut	Strut 2 screw	C 100x82x4	2.384	HX420LAD	8.99	1166.8	1166.8	32
27	Strut	Strut 2 screw	C 100x82x4	2.384	HX420LAD	8.99	1166.8	1166.8	34
32	Strut	Strut 2 screw	C 100x77x3	2.247	S350GD	6.21	840	840	32
33	Strut	Strut 2 screw	C 100x77x3	2.247	S350GD	6.21	840	840	34
38	Strut	Strut 2 screw	C 100x75x2	2.24	S350GD	4.14	552	472.35	34
39	Strut	Strut 2 screw	C 100x75x2	2.24	S350GD	4.14	552	472.35	32
44	Strut	Strut 2 screw	C 100x75x2	2.104	S350GD	4.14	552	472.35	32
45	Strut	Strut 2 screw	C 100x75x2	2.104	S350GD	4.14	552	472.35	34
50	Strut	Strut 2 screw	C 100x77x3	2.097	S350GD	6.21	840	840	34
51	Strut	Strut 2 screw	C 100x77x3	2.097	S350GD	6.21	840	840	32
56	Strut	Strut 2 screw	C 100x82x4	1.961	HX420LAD	8.99	1166.8	1166.8	34
57	Strut	Strut 2 screw	C 100x82x4	1.961	HX420LAD	8.99	1166.8	1166.8	32
62	Strut	Strut 2 screw	C 100x82x4	1.955	HX420LAD	8.99	1166.8	1166.8	33
63	Strut	Strut 2 screw	C 100x82x4	1.955	HX420LAD	8.99	1166.8	1166.8	30
68	Strut	Strut 2 screw	C 100x85x5	1.82	HX500LAD	11.45	1580	1580	33
69	Strut	Strut 2 screw	C 100x85x5	1.82	HX500LAD	11.45	1580	1580	30
74	Strut	Strut 2 screw	C 100x85x5	1.814	HX500LAD	11.45	1580	1580	10
75	Strut	Strut 2 screw	C 100x85x5	1.814	HX500LAD	11.45	1580	1580	10
80	Strut	Strut 2 screw	C 100x85x5	1.681	HX500LAD	11.45	1580	1580	10
81	Strut	Strut 2 screw	C 100x85x5	1.681	HX500LAD	11.45	1580	1580	10
86	Strut	Strut 2 screw	C 100x85x5	1.675	HX500LAD	11.45	1580	1580	10
87	Strut	Strut 2 screw	C 100x85x5	1.675	HX500LAD	11.45	1580	1580	10
92	Strut	Strut 2 screw	C 100x85x5	1.543	HX500LAD	11.45	1580	1580	10
93	Strut	Strut 2 screw	C 100x85x5	1.543	HX500LAD	11.45	1580	1580	10
98	Strut	Strut 2 screw	C 100x85x5	1.539	HX500LAD	11.45	1580	1580	10
99	Strut	Strut 2 screw	C 100x85x5	1.539	HX500LAD	11.45	1580	1580	10
104	Strut	Strut 2 screw	C 100x85x5	1.408	HX500LAD	11.45	1580	1580	10
105	Strut	Strut 2 screw	C 100x85x5	1.408	HX500LAD	11.45	1580	1580	10
110	Strut	Strut 2 screw	C 100x85x5	1.405	HX500LAD	11.45	1580	1580	10
111	Strut	Strut 2 screw	C 100x85x5	1.405	HX500LAD	11.45	1580	1580	10
116	Strut	Strut 2 screw	C 100x85x5	1.277	HX500LAD	11.45	1580	1580	10
117	Strut	Strut 2 screw	C 100x85x5	1.277	HX500LAD	11.45	1580	1580	10
122	Strut	Strut 2 screw	C 100x85x5	1.275	HX500LAD	11.45	1580	1580	10
123	Strut	Strut 2 screw	C 100x85x5	1.275	HX500LAD	11.45	1580	1580	10
128	Strut	Strut 2 screw	C 100x85x5	1.048	HX500LAD	11.45	1580	1580	10
129	Strut	Strut 2 screw	C 100x85x5	1.048	HX500LAD	11.45	1580	1580	10

Vnitřní síly:

Prvek	Typ	Profil	N_Ed [kN]	N_t,Ed [kN]	N_c,Ed [kN]	M_z,Ed [kNm]	V_y,Ed [kN]	dM_z,Ed [kNm]	Komb. N_Ed
1	Strut	C 100x85x5	-94.33	-94.55	8.89	0.05	-0.02	0	34
2	Strut	C 100x85x5	-92.31	-92.53	7.23	0.05	-0.02	0	32
8	Strut	C 100x85x5	93.92	-8.51	93.92	0	-0.06	0	32
9	Strut	C 100x85x5	96.11	-10.22	96.11	0	-0.06	0	34
14	Strut	C 100x82x4	-77.46	-77.61	6.45	0.02	0.05	0	32
15	Strut	C 100x82x4	-79.9	-80.05	7.76	0.02	0.05	0	34
20	Strut	C 100x82x4	71.97	-6.86	71.97	0	-0.13	0	32
21	Strut	C 100x82x4	74.4	-8.07	74.4	0	-0.13	0	34
26	Strut	C 100x82x4	-48.16	-48.3	8.1	0.02	0.09	0	32
27	Strut	C 100x82x4	-50.85	-50.99	6.71	0.02	0.09	0	34
32	Strut	C 100x77x3	41.25	-10.95	41.25	0	-0.12	0	32
33	Strut	C 100x77x3	43.75	-9.66	43.75	0	-0.13	0	34
38	Strut	C 100x75x2	20.82	-14.93	20.82	0	-0.05	0.05	34
39	Strut	C 100x75x2	19.13	-17.12	19.13	0	-0.04	0.04	32
44	Strut	C 100x75x2	14.94	-24.36	14.94	0	-0.07	0.03	32
45	Strut	C 100x75x2	17.62	-22.45	17.62	0	-0.08	0.04	34
50	Strut	C 100x77x3	46.08	-5.84	46.08	0	-0.17	0	34
51	Strut	C 100x77x3	43.68	-6.57	43.68	0	-0.16	0	32
56	Strut	C 100x82x4	-46.3	-46.41	5.05	-0.02	0.22	0	34
57	Strut	C 100x82x4	-44.15	-44.26	5.53	-0.02	0.21	0	32
62	Strut	C 100x82x4	64.71	-8.21	64.71	0	-0.37	0	33
63	Strut	C 100x82x4	62.43	-7.39	62.43	0	-0.35	0	30
68	Strut	C 100x85x5	-80.52	-80.66	9.13	-0.02	0.55	0	33
69	Strut	C 100x85x5	-77.9	-78.04	8	-0.02	0.53	0	30
74	Strut	C 100x85x5	129.99	-15.02	129.99	0	-1	0	10
75	Strut	C 100x85x5	126.71	-12.26	126.71	0	-0.98	0	10
80	Strut	C 100x85x5	-128.96	-129.09	13.91	-0.02	1.12	0	10
81	Strut	C 100x85x5	-125.68	-125.81	11.2	-0.02	1.1	0	10
86	Strut	C 100x85x5	161.63	-18.15	161.63	0	-1.48	0	10
87	Strut	C 100x85x5	158.21	-14.09	158.21	0	-1.46	0	10
92	Strut	C 100x85x5	-170.1	-170.21	18.04	-0.02	1.71	0	10
93	Strut	C 100x85x5	-166.57	-166.68	13.79	-0.02	1.69	0	10
98	Strut	C 100x85x5	219.68	-23.93	219.68	0	-2.31	0	10
99	Strut	C 100x85x5	215.8	-17.9	215.8	0	-2.29	0	10
104	Strut	C 100x85x5	-237.31	-237.42	23.95	-0.02	2.65	0	10
105	Strut	C 100x85x5	-233.29	-233.39	17.71	-0.02	2.63	0	10
110	Strut	C 100x85x5	298.32	-30.98	298.32	0	-3.45	0	10
111	Strut	C 100x85x5	293.92	-23.47	293.92	0	-3.43	0	10
116	Strut	C 100x85x5	-311.33	-311.42	31.1	-0.02	3.69	0	10
117	Strut	C 100x85x5	-306.82	-306.91	23.5	-0.02	3.68	0	10
122	Strut	C 100x85x5	368.62	-38.34	368.62	0	-4.37	0	10
123	Strut	C 100x85x5	363.44	-32.64	363.44	0	-4.38	0	10
128	Strut	C 100x85x5	-405.6	-405.67	41.57	-0.01	4.72	0	10
129	Strut	C 100x85x5	-399.45	-399.52	35.68	-0.01	4.74	0	10

Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek
Datum:	24.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka
				Norma projektu:	ČSN EN

Vzpěr:

Prvek	Typ	Profil	X_y [-]	N_y,b,Rd [kN]		N_z,b,Rd [kN]	X_t [-]	N_b,t,Rd [kN]	X_tf [-]	N_b,tf,Rd [kN]
1	Strut	C 100x85x5	0.55	435.21	0.48	379.36	1	0	1	0
2	Strut	C 100x85x5	0.55	435.21	0.48	379.36	1	0	1	0
8	Strut	C 100x85x5	0.62	488.58	0.55	434.1	0.45	351.65	0.33	261.71
9	Strut	C 100x85x5	0.62	488.58	0.55	434.1	0.45	351.65	0.33	261.71
14	Strut	C 100x82x4	0.66	325.09	0.56	274.24	1	0	1	0
15	Strut	C 100x82x4	0.66	325.09	0.56	274.24	1	0	1	0
20	Strut	C 100x82x4	0.69	340.29	0.6	291.83	0.47	229.09	0.37	181.28
21	Strut	C 100x82x4	0.69	340.29	0.6	291.83	0.47	229.09	0.37	181.28
26	Strut	C 100x82x4	0.7	341.14	0.6	292.83	1	0	1	0
27	Strut	C 100x82x4	0.7	341.14	0.6	292.83	1	0	1	0
32	Strut	C 100x77x3	0.78	229.52	0.65	190.58	0.47	139.62	0.41	119.65
33	Strut	C 100x77x3	0.78	229.52	0.65	190.58	0.47	139.62	0.41	119.65
38	Strut	C 100x75x2	0.81	134.69	0.68	112.78	0.5	82.97	0.44	72.72
39	Strut	C 100x75x2	0.81	134.69	0.68	112.78	0.5	82.97	0.44	72.72
44	Strut	C 100x75x2	0.83	138.03	0.72	118.29	0.54	89.33	0.48	79.03
45	Strut	C 100x75x2	0.83	138.03	0.72	118.29	0.54	89.33	0.48	79.03
50	Strut	C 100x77x3	0.81	237.2	0.69	202.18	0.51	151.17	0.45	131.03
51	Strut	C 100x77x3	0.81	237.2	0.69	202.18	0.51	151.17	0.45	131.03
56	Strut	C 100x82x4	0.79	385.32	0.71	348.31	1	0	1	0
57	Strut	C 100x82x4	0.79	385.32	0.71	348.31	1	0	1	0
62	Strut	C 100x82x4	0.79	385.94	0.71	349.13	0.58	283.43	0.48	235.74
63	Strut	C 100x82x4	0.79	385.94	0.71	349.13	0.58	283.43	0.48	235.74
68	Strut	C 100x85x5	0.78	614.96	0.73	577.19	1	0	1	0
69	Strut	C 100x85x5	0.78	614.96	0.73	577.19	1	0	1	0
74	Strut	C 100x85x5	0.78	615.97	0.73	578.4	0.61	483.29	0.5	394.32
75	Strut	C 100x85x5	0.78	615.97	0.73	578.4	0.61	483.29	0.5	394.32
80	Strut	C 100x85x5	0.81	638.77	0.77	605.83	1	0	1	0
81	Strut	C 100x85x5	0.81	638.77	0.77	605.83	1	0	1	0
86	Strut	C 100x85x5	0.81	639.59	0.77	606.83	0.65	516.64	0.55	432.1
87	Strut	C 100x85x5	0.81	639.59	0.77	606.83	0.65	516.64	0.55	432.1
92	Strut	C 100x85x5	0.84	660.78	0.8	632.45	1	0	1	0
93	Strut	C 100x85x5	0.84	660.78	0.8	632.45	1	0	1	0
98	Strut	C 100x85x5	0.84	661.41	0.8	633.22	0.7	550.24	0.6	472.28
99	Strut	C 100x85x5	0.84	661.41	0.8	633.22	0.7	550.24	0.6	472.28
104	Strut	C 100x85x5	0.86	680.98	0.83	656.85	1	0	1	0
105	Strut	C 100x85x5	0.86	680.98	0.83	656.85	1	0	1	0
110	Strut	C 100x85x5	0.86	681.41	0.83	657.37	0.74	583.04	0.65	513.54
111	Strut	C 100x85x5	0.86	681.41	0.83	657.37	0.74	583.04	0.65	513.54
116	Strut	C 100x85x5	0.89	699.4	0.86	678.95	1	0	1	0
117	Strut	C 100x85x5	0.89	699.4	0.86	678.95	1	0	1	0
122	Strut	C 100x85x5	0.89	699.61	0.86	679.21	0.78	614.05	0.7	554.11
123	Strut	C 100x85x5	0.89	699.61	0.86	679.21	0.78	614.05	0.7	554.11
128	Strut	C 100x85x5	0.92	728.97	0.9	713.93	1	0	1	0
129	Strut	C 100x85x5	0.92	728.97	0.9	713.93	1	0	1	0

Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek
Datum:	24.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka
				Norma projektu:	ČSN EN

Pevnosti:

Prvek	Typ	Profil	N_c,Rd [kN]	N_t,Rd [kN]	N_b,Rd [kN]	Vy_c,Rd [kN]	M_z,c,Rd,ten [kNm]	M_z,c,Rd,com [kNm]	Komb. N_Ed
1	Strut	C 100x85x5	825	825	379.36	232	18.41	22.55	34
2	Strut	C 100x85x5	825	825	379.36	232	18.41	22.55	32
8	Strut	C 100x85x5	801.03	825	434.1	232	18.41	22.55	32
9	Strut	C 100x85x5	801.03	825	434.1	232	18.41	22.55	34
14	Strut	C 100x82x4	512.19	516.94	274.24	144.21	10.83	14.68	32
15	Strut	C 100x82x4	512.19	516.94	274.24	144.21	10.83	14.68	34
20	Strut	C 100x82x4	490.1	516.94	291.83	144.21	10.83	14.68	32
21	Strut	C 100x82x4	490.1	516.94	291.83	144.21	10.83	14.68	34
26	Strut	C 100x82x4	512.19	516.94	292.83	144.21	10.83	14.68	32
27	Strut	C 100x82x4	512.19	516.94	292.83	144.21	10.83	14.68	34
32	Strut	C 100x77x3	294	311.64	190.58	90.13	5.48	8.45	32
33	Strut	C 100x77x3	294	311.64	190.58	90.13	5.48	8.45	34
38	Strut	C 100x75x2	165.32	201.04	112.78	59.28	3.42	5.49	34
39	Strut	C 100x75x2	165.32	201.04	112.78	59.28	3.42	5.49	32
44	Strut	C 100x75x2	165.32	201.04	118.29	59.28	3	4.23	32
45	Strut	C 100x75x2	165.32	201.04	118.29	59.28	3	4.23	34
50	Strut	C 100x77x3	294	311.64	202.18	90.13	8.45	5.48	34
51	Strut	C 100x77x3	294	311.64	202.18	90.13	8.45	5.48	32
56	Strut	C 100x82x4	490.1	516.94	348.31	144.21	14.68	10.83	34
57	Strut	C 100x82x4	490.1	516.94	348.31	144.21	14.68	10.83	32
62	Strut	C 100x82x4	490.1	516.94	349.13	144.21	10.83	14.68	33
63	Strut	C 100x82x4	490.1	516.94	349.13	144.21	14.68	10.83	30
68	Strut	C 100x85x5	801.03	825	577.19	232	22.55	18.41	33
69	Strut	C 100x85x5	801.03	825	577.19	232	22.55	18.41	30
74	Strut	C 100x85x5	801.03	825	578.4	232	18.41	22.55	10
75	Strut	C 100x85x5	801.03	825	578.4	232	18.41	22.55	10
80	Strut	C 100x85x5	801.03	825	605.83	232	22.55	18.41	10
81	Strut	C 100x85x5	801.03	825	605.83	232	22.55	18.41	10
86	Strut	C 100x85x5	801.03	825	606.83	232	18.41	22.55	10
87	Strut	C 100x85x5	801.03	825	606.83	232	18.41	22.55	10
92	Strut	C 100x85x5	801.03	825	632.45	232	22.55	18.41	10
93	Strut	C 100x85x5	801.03	825	632.45	232	22.55	18.41	10
98	Strut	C 100x85x5	801.03	825	633.22	232	18.41	22.55	10
99	Strut	C 100x85x5	801.03	825	633.22	232	18.41	22.55	10
104	Strut	C 100x85x5	801.03	825	656.85	232	22.55	18.41	10
105	Strut	C 100x85x5	801.03	825	656.85	232	22.55	18.41	10
110	Strut	C 100x85x5	801.03	825	657.37	232	18.41	22.55	10
111	Strut	C 100x85x5	801.03	825	657.37	232	18.41	22.55	10
116	Strut	C 100x85x5	801.03	825	678.95	232	22.55	18.41	10
117	Strut	C 100x85x5	801.03	825	678.95	232	22.55	18.41	10
122	Strut	C 100x85x5	801.03	825	679.21	232	18.41	22.55	10
123	Strut	C 100x85x5	801.03	825	679.21	232	18.41	22.55	10
128	Strut	C 100x85x5	801.03	825	713.93	232	22.55	18.41	10
129	Strut	C 100x85x5	801.03	825	713.93	232	22.55	18.41	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	11	12	11	0	0	0	0	0	12	Tah a ohyb	34
2	C 100x85x5	0	11	11	11	0	0	0	0	0	11	Prostý tah	32
8	C 100x85x5	12	0	12	12	0	33	36	19	21	36	Vzpěr	32
9	C 100x85x5	12	0	12	12	0	33	37	19	22	37	Vzpěr	34
14	C 100x82x4	0	15	15	15	0	0	0	0	0	15	Prostý tah	32
15	C 100x82x4	0	15	16	15	0	0	0	0	0	16	Tah a ohyb	34
20	C 100x82x4	15	0	15	15	0	34	40	21	25	40	Vzpěr	32
21	C 100x82x4	15	0	15	15	0	34	41	22	26	41	Vzpěr	34
26	C 100x82x4	0	9	10	9	0	0	0	0	0	10	Tah a ohyb	32
27	C 100x82x4	0	10	10	10	0	0	0	0	0	10	Prostý tah	34
32	C 100x77x3	14	0	14	14	0	34	34	18	22	34	Štíhlost	32
33	C 100x77x3	15	0	15	15	0	34	37	19	23	37	Vzpěr	34
38	C 100x75x2	13	0	13	2	0	34	29	17	20	29	Štíhlost	34
39	C 100x75x2	12	0	12	2	0	34	26	15	18	26	Štíhlost	32
44	C 100x75x2	9	0	10	2	0	32	19	11	13	19	Štíhlost	32
45	C 100x75x2	11	0	12	2	0	32	22	13	16	22	Štíhlost	34
50	C 100x77x3	16	0	16	16	0	31	35	20	23	35	Vzpěr	34
51	C 100x77x3	15	0	15	15	0	31	33	19	22	33	Vzpěr	32
56	C 100x82x4	0	9	9	9	0	0	0	0	0	9	Prostý tah	34
57	C 100x82x4	0	9	9	8	0	0	0	0	0	9	Prostý tah	32
62	C 100x82x4	13	0	0	13	0	27	27	0	0	27	Štíhlost	33
63	C 100x82x4	13	0	13	13	0	27	26	16	18	26	Štíhlost	30
68	C 100x85x5	0	10	10	10	0	0	0	0	0	10	Prostý tah	33
69	C 100x85x5	0	9	10	9	0	0	0	0	0	10	Tah a ohyb	30
74	C 100x85x5	16	0	0	16	0	24	33	0	0	33	Vzpěr	10
75	C 100x85x5	16	0	0	16	0	24	32	0	0	32	Vzpěr	10
80	C 100x85x5	0	16	16	16	0	0	0	0	0	16	Prostý tah	10
81	C 100x85x5	0	15	15	15	0	0	0	0	0	15	Prostý tah	10
86	C 100x85x5	20	0	0	20	1	22	37	0	0	37	Vzpěr	10
87	C 100x85x5	20	0	0	20	1	22	37	0	0	37	Vzpěr	10
92	C 100x85x5	0	21	21	21	1	0	0	0	0	21	Prostý tah	10
93	C 100x85x5	0	20	20	20	1	0	0	0	0	20	Prostý tah	10
98	C 100x85x5	27	0	0	27	1	20	47	0	0	47	Vzpěr	10
99	C 100x85x5	27	0	0	27	1	20	46	0	0	46	Vzpěr	10
104	C 100x85x5	0	29	29	29	1	0	0	0	0	29	Prostý tah	10
105	C 100x85x5	0	28	28	28	1	0	0	0	0	28	Prostý tah	10
110	C 100x85x5	37	0	0	37	1	19	58	0	0	58	Vzpěr	10
111	C 100x85x5	37	0	0	37	1	19	57	0	0	57	Vzpěr	10
116	C 100x85x5	0	38	38	38	2	0	0	0	0	38	Prostý tah	10
117	C 100x85x5	0	37	37	37	2	0	0	0	0	37	Prostý tah	10
122	C 100x85x5	46	0	0	46	2	17	67	0	0	67	Vzpěr	10
123	C 100x85x5	45	0	0	45	2	17	66	0	0	66	Vzpěr	10
128	C 100x85x5	0	49	49	49	2	0	0	0	0	49	Prostý tah	10
129	C 100x85x5	0	48	48	48	2	0	0	0	0	48	Prostý tah	10

2 krajní diagonály vazníku budou z profilu C40/132/100/132/40x6 - VIZ POSUDEK NA R15

Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek
Datum:	24.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka
				Norma projektu:	ČSN EN

Spoje:

Prvek	Komb _N_t, Ed	Komb _N_c, Ed	Profil	N_t,Ed [kN]	N_c,Ed [kN]	Spoj	F_v [%]	F_b [%]	F_n [%]	V_eff [%]	Spoj [%]	Rozh. spoj
1	34	50	C 100x85x5	-94.55	8.89	C56	36	30	19	28	36	Shear
2	32	21	C 100x85x5	-92.53	7.23	C56	36	29	18	27	36	Shear
8	21	32	C 100x85x5	-8.51	93.92	C56	36	30	2	3	36	Shear
9	50	34	C 100x85x5	-10.22	96.11	C56	37	30	2	3	37	Shear
14	32	21	C 100x82x4	-77.61	6.45	C46	30	35	23	35	35	Bearing
15	34	50	C 100x82x4	-80.05	7.76	C46	31	36	24	36	36	Bearing
20	21	32	C 100x82x4	-6.86	71.97	C46	28	33	2	3	33	Bearing
21	50	34	C 100x82x4	-8.07	74.4	C46	29	34	2	4	34	Bearing
26	32	44	C 100x82x4	-48.3	8.1	C46	19	22	14	22	22	Bearing
27	34	42	C 100x82x4	-50.99	6.71	C46	20	23	15	23	23	Bearing
32	44	32	C 100x77x3	-10.95	41.25	C36	16	29	6	8	29	Bearing
33	42	34	C 100x77x3	-9.66	43.75	C36	17	30	5	7	30	Bearing
38	42	34	C 100x75x2	-14.93	20.82	C26	8	22	11	17	22	Bearing
39	44	32	C 100x75x2	-17.12	19.13	C26	7	20	13	19	20	Bearing
44	34	32	C 100x75x2	-24.36	14.94	C26	9	25	19	28	28	Block tear.
45	32	34	C 100x75x2	-22.45	17.62	C26	9	23	17	26	26	Block tear.
50	50	34	C 100x77x3	-5.84	46.08	C36	18	32	3	4	32	Bearing
51	21	32	C 100x77x3	-6.57	43.68	C36	17	30	3	5	30	Bearing
56	34	50	C 100x82x4	-46.41	5.05	C46	18	21	14	21	21	Bearing
57	32	21	C 100x82x4	-44.26	5.53	C46	17	20	13	20	20	Bearing
62	50	33	C 100x82x4	-8.21	64.71	C46	25	29	2	4	29	Bearing
63	21	30	C 100x82x4	-7.39	62.43	C46	24	28	2	3	28	Bearing
68	33	50	C 100x85x5	-80.66	9.13	C56	31	26	16	24	31	Shear
69	30	21	C 100x85x5	-78.04	8	C56	30	25	16	23	30	Shear
74	50	10	C 100x85x5	-15.02	129.99	C58	37	31	3	5	37	Shear
75	21	10	C 100x85x5	-12.26	126.71	C58	37	30	2	4	37	Shear
80	10	50	C 100x85x5	-129.09	13.91	C58	37	31	26	41	41	Block tear.
81	10	21	C 100x85x5	-125.81	11.2	C58	36	30	25	40	40	Block tear.
86	50	10	C 100x85x5	-18.15	161.63	C58	47	39	4	6	47	Shear
87	21	10	C 100x85x5	-14.09	158.21	C58	46	38	3	5	46	Shear
92	10	50	C 100x85x5	-170.21	18.04	C58	49	41	34	54	54	Block tear.
93	10	21	C 100x85x5	-166.68	13.79	C58	48	40	33	53	53	Block tear.
98	50	10	C 100x85x5	-23.93	219.68	C58	63	52	5	8	63	Shear
99	21	10	C 100x85x5	-17.9	215.8	C58	62	51	4	6	62	Shear
104	10	50	C 100x85x5	-237.42	23.95	C58	68	57	47	76	76	Block tear.
105	10	21	C 100x85x5	-233.39	17.71	C58	67	56	47	75	75	Block tear.
110	50	10	C 100x85x5	-30.98	298.32	C58	86	71	6	10	86	Shear
111	50	10	C 100x85x5	-23.47	293.92	C58	85	70	5	7	85	Shear
116	10	50	C 100x85x5	-311.42	31.1	C58	90	74	62	99	99	Block tear.
117	10	50	C 100x85x5	-306.91	23.5	C58	88	73	61	98	98	Block tear.
122	50	10	C 100x85x5	-38.34	368.62	C58	106	88	8	12	106	Shear
123	21	10	C 100x85x5	-32.64	363.44	C58	105	87	7	10	105	Shear
128	10	50	C 100x85x5	-405.67	41.57	C58	117	97	81	130	130	Block tear.
129	10	21	C 100x85x5	-399.52	35.68	C58	115	95	80	128	128	Block tear.

BUDOU POUŽITY ŠROUBY M16
VIZ POSUDEK NA R15

Č. projektu:	CZ1138	Název projektu:	Basketbalová hala	Místo výstavby:	Frýdek-Místek
Datum:	24.7.2018	Vypracoval:	Lenka Burgerová	Kontroloval:	Jaroslav Kosinka
				Norma projektu:	ČSN EN

Souhrn:

Prvek	Member type	Profil	Materiál	Komb. N_Ed	Profil [%]	Spoj [%]	Vyhodnoceni	Posouzení
1	Strut 2 screw	C 100x85x5	HX500LAD	34	12	36	Shear	Vyhovuje
2	Strut 2 screw	C 100x85x5	HX500LAD	32	11	36	Shear	Vyhovuje
8	Strut 2 screw	C 100x85x5	HX500LAD	32	36	36	Shear	Vyhovuje
9	Strut 2 screw	C 100x85x5	HX500LAD	34	37	37	Shear	Vyhovuje
14	Strut 2 screw	C 100x82x4	HX420LAD	32	15	35	Bearing	Vyhovuje
15	Strut 2 screw	C 100x82x4	HX420LAD	34	16	36	Bearing	Vyhovuje
20	Strut 2 screw	C 100x82x4	HX420LAD	32	40	33	Vzpěr	Vyhovuje
21	Strut 2 screw	C 100x82x4	HX420LAD	34	41	34	Vzpěr	Vyhovuje
26	Strut 2 screw	C 100x82x4	HX420LAD	32	10	22	Bearing	Vyhovuje
27	Strut 2 screw	C 100x82x4	HX420LAD	34	10	23	Bearing	Vyhovuje
32	Strut 2 screw	C 100x77x3	S350GD	32	34	29	Štíhlost	Vyhovuje
33	Strut 2 screw	C 100x77x3	S350GD	34	37	30	Vzpěr	Vyhovuje
38	Strut 2 screw	C 100x75x2	S350GD	34	29	22	Štíhlost	Vyhovuje
39	Strut 2 screw	C 100x75x2	S350GD	32	26	20	Štíhlost	Vyhovuje
44	Strut 2 screw	C 100x75x2	S350GD	32	19	28	Block tear.	Vyhovuje
45	Strut 2 screw	C 100x75x2	S350GD	34	22	26	Block tear.	Vyhovuje
50	Strut 2 screw	C 100x77x3	S350GD	34	35	32	Vzpěr	Vyhovuje
51	Strut 2 screw	C 100x77x3	S350GD	32	33	30	Vzpěr	Vyhovuje
56	Strut 2 screw	C 100x82x4	HX420LAD	34	9	21	Bearing	Vyhovuje
57	Strut 2 screw	C 100x82x4	HX420LAD	32	9	20	Bearing	Vyhovuje
62	Strut 2 screw	C 100x82x4	HX420LAD	33	27	29	Bearing	Vyhovuje
63	Strut 2 screw	C 100x82x4	HX420LAD	30	26	28	Bearing	Vyhovuje
68	Strut 2 screw	C 100x85x5	HX500LAD	33	10	31	Shear	Vyhovuje
69	Strut 2 screw	C 100x85x5	HX500LAD	30	10	30	Shear	Vyhovuje
74	Strut 2 screw	C 100x85x5	HX500LAD	10	33	37	Shear	Vyhovuje
75	Strut 2 screw	C 100x85x5	HX500LAD	10	32	37	Shear	Vyhovuje
80	Strut 2 screw	C 100x85x5	HX500LAD	10	16	41	Block tear.	Vyhovuje
81	Strut 2 screw	C 100x85x5	HX500LAD	10	15	40	Block tear.	Vyhovuje
86	Strut 2 screw	C 100x85x5	HX500LAD	10	37	47	Shear	Vyhovuje
87	Strut 2 screw	C 100x85x5	HX500LAD	10	37	46	Shear	Vyhovuje
92	Strut 2 screw	C 100x85x5	HX500LAD	10	21	54	Block tear.	Vyhovuje
93	Strut 2 screw	C 100x85x5	HX500LAD	10	20	53	Block tear.	Vyhovuje
98	Strut 2 screw	C 100x85x5	HX500LAD	10	47	63	Shear	Vyhovuje
99	Strut 2 screw	C 100x85x5	HX500LAD	10	46	62	Shear	Vyhovuje
104	Strut 2 screw	C 100x85x5	HX500LAD	10	29	76	Block tear.	Vyhovuje
105	Strut 2 screw	C 100x85x5	HX500LAD	10	28	75	Block tear.	Vyhovuje
110	Strut 2 screw	C 100x85x5	HX500LAD	10	58	86	Shear	Vyhovuje
111	Strut 2 screw	C 100x85x5	HX500LAD	10	57	85	Shear	Vyhovuje
116	Strut 2 screw	C 100x85x5	HX500LAD	10	38	99	Block tear.	Vyhovuje
117	Strut 2 screw	C 100x85x5	HX500LAD	10	37	98	Block tear.	Vyhovuje
122	Strut 2 screw	C 100x85x5	HX500LAD	10	67	106	Shear	Nevyhovuje
123	Strut 2 screw	C 100x85x5	HX500LAD	10	66	105	Shear	Nevyhovuje
128	Strut 2 screw	C 100x85x5	HX500LAD	10	49	130	Block tear.	Nevyhovuje
129	Strut 2 screw	C 100x85x5	HX500LAD	10	48	128	Block tear.	Nevyhovuje

PROFIL C40/132/100/132/40x6

ŠROUBY M16

POSOUZENÍ DEFORMACÍ RÁMU

Displacements - Cases: 10to12 20to23 30to35 40to45 50
Global extremes
1

- Cases: 10to12 20to23 30to35 40to45 50

Filtering	Node	Case
Full list	1to117 272to295	10to12 20to23 3
Selection	1to95 367 368 4	10to12 20to23 3
Total number	247	30
Selected number	98	20

- Cases: 10to12 20to23 30to35 40to45 50

	UX (mm)	UZ (mm)	RY (Rad)
MAX	17.8	10.7	0.015
Node	79	15	82
Case	42	50	10
MIN	-31.7	-122.6	-0.015
Node	78	11	83
Case	43	10	10

Posouzení vodorovné deformace
Limitní deformace = 8300/150 = 55.3mm
Max deformace = 31.7/1.4 = 22.6mm
22.6 mm < 55.3 mm

Vodorovná deformace vyhoví.

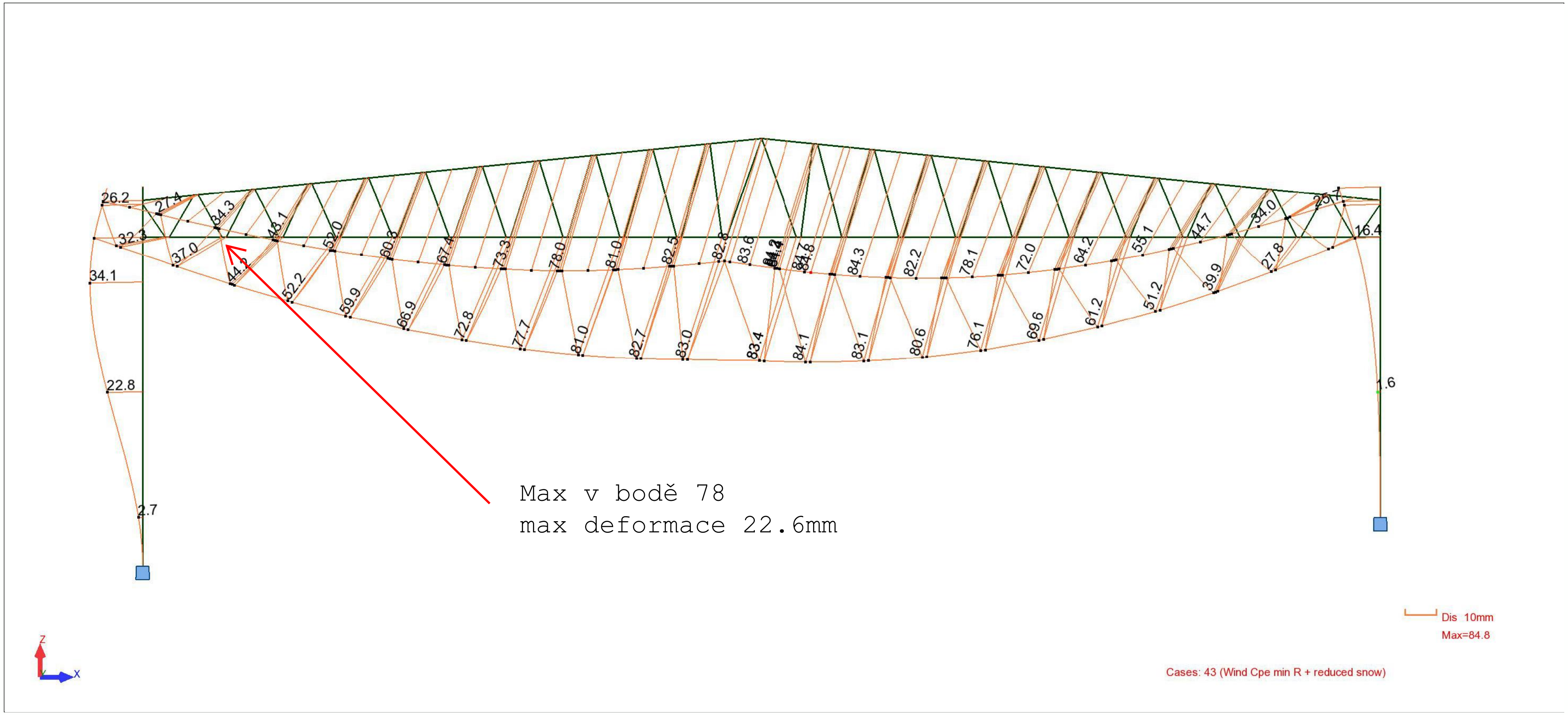
Posouzení svislé deformace
Limitní deformace = 32670/250 = 130.7mm
Max deformace = 122.6/1.4 = 87.6mm
87.6 mm < 130.7 mm

Svislá deformace vyhoví.

View - Exact deformation(s); Cases: 43 (Wind Cpe min R + reduced snow)

Max vodorovná deformace

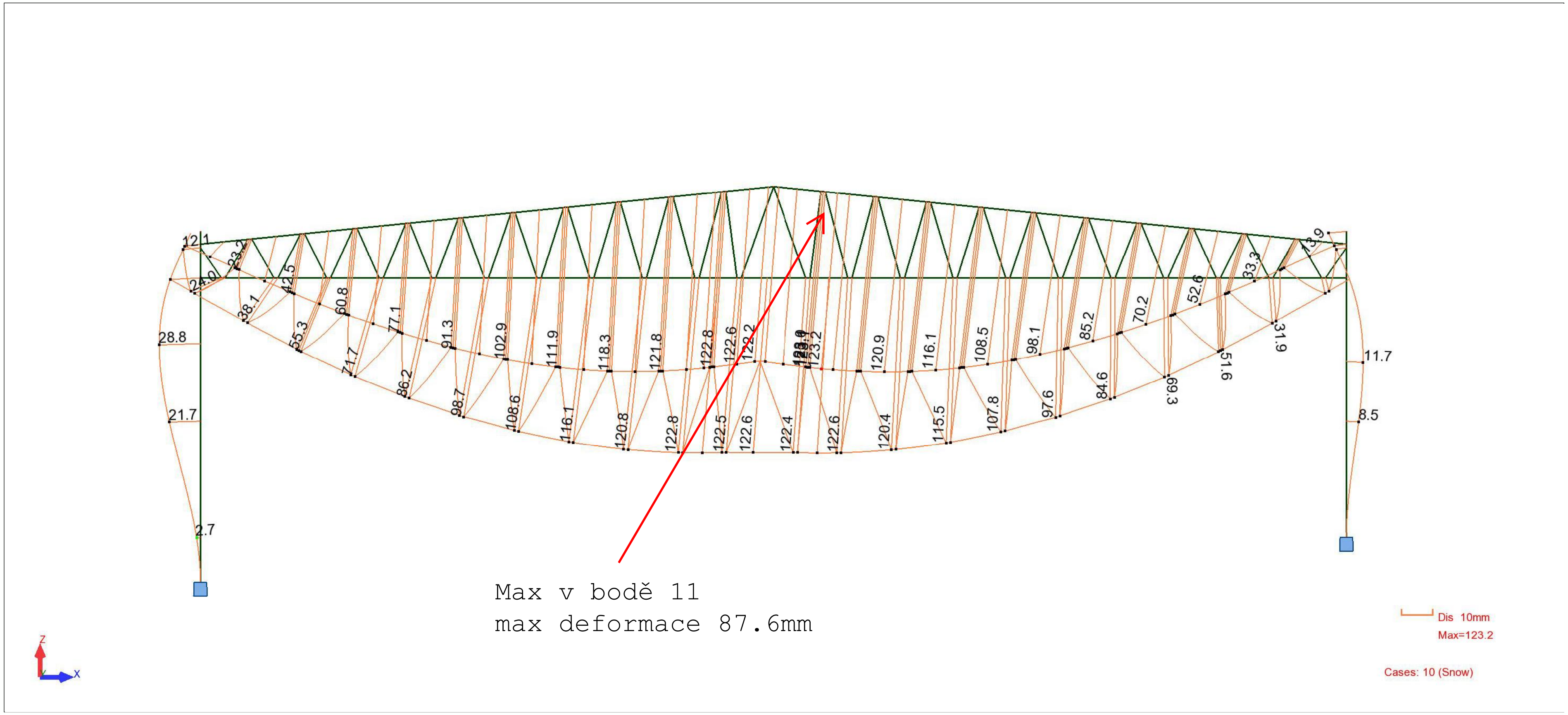
návrhové hodnoty



View - Exact deformation(s); Cases: 10 (Snow)

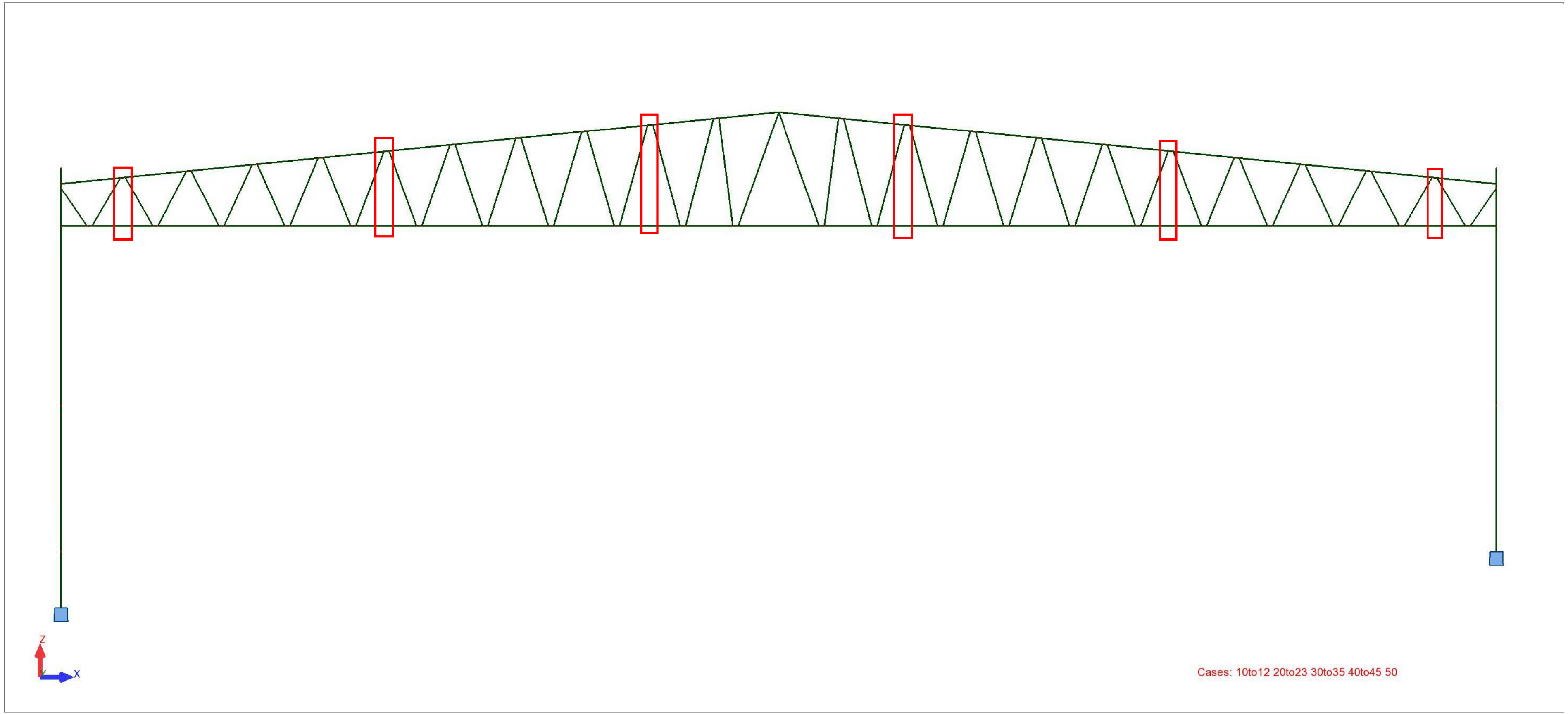
Max svislá deformace

návrhové hodnoty

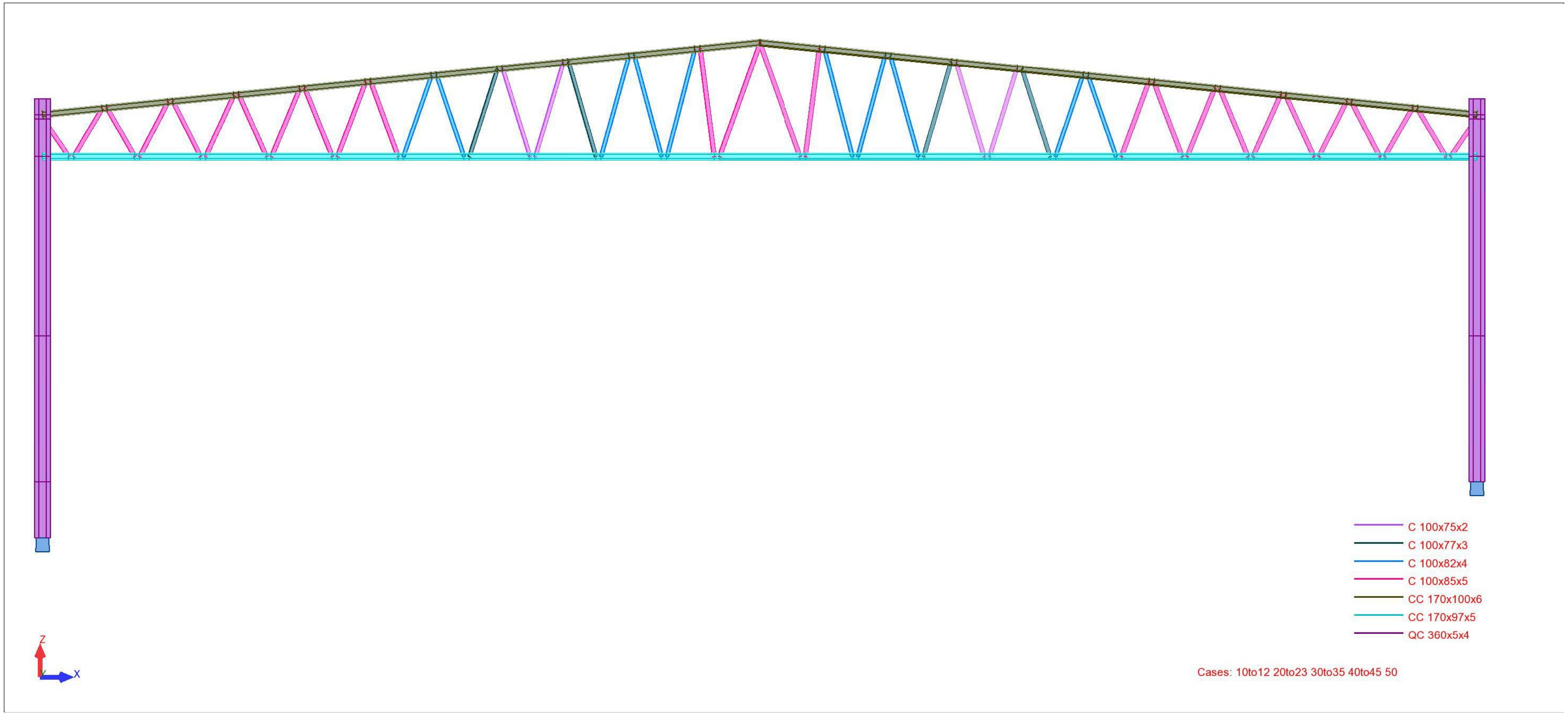


View - Cases: 10to12 20to23 30to35 40to45 50

STABILIZACE DOLNÍHO PÁSU VAZNÍKU



View - Cases: 10to12 20to23 30to35 40to45 50 PROFILY RÁMU



Kotvení rámu FR1

Soubory modelu konstrukce:

……..

……..

Soubory zatížení:

CZ1138 Loading. Xlsm

……..

……..

Poznámky:

VETKNUTÉ KOTVENÍ

6x kotva M30 8.8 dl.750mm, hl. kotvení 620mm

……..

……..

……..

……..

……..

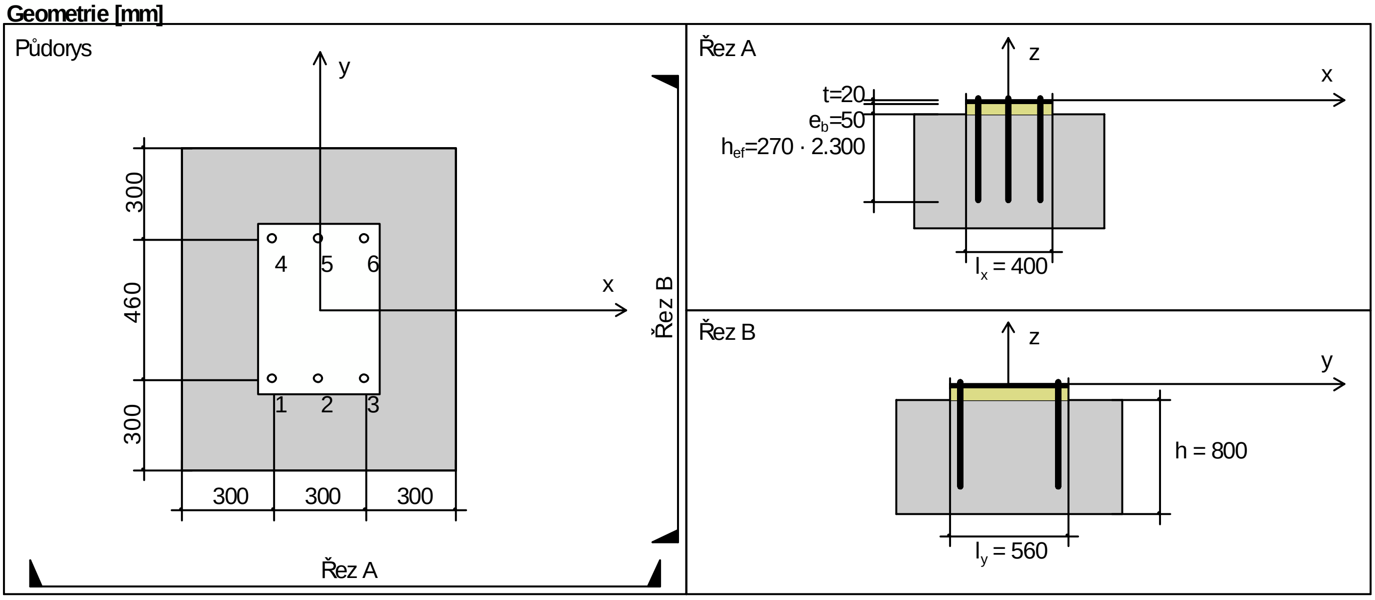
……..

Poznámky:

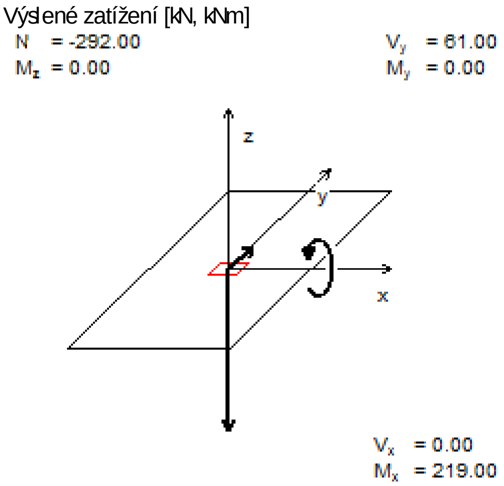
Typ a rozměr kotvy:	**HIT-RE 500 + HAS-E (8.8)-M30**
Efektivní hloubka kotvení:	h_{ef}= 270 mm; součinitel hloubky osazení = 2.300
Materiál:	8.8
Certifikát:	
Platnost:	- / -
Zkouška:	Návrh podle SOFA - po ETAG zkoušce
Distanční montáž:	s upevnění na povrchu; plně podlití (kotevní deska); e_b = 50 mm; t = 20 mm
Kotevní deska:	S355 (ST52); ; l_x x l_y x t = 400 x 560 x 20 mm
Základní materiál:	netrhlinový Beton C20/25, f_{cc} = 25.00 N/mm²; h = 800 mm
Výztuž:	vzdálenost výztuže >= 150 mm
	bez okrajové výztuže

Kotva

Geometrie [mm]



Zatížení



Výpočtová zatížení [kN, kNm]

N	-292.00
V_x	0.00
V_y	61.00
M_x	219.00
M_y	0.00
M_z	0.00

Exentricita [mm]

e_x = 0; e_y = 0

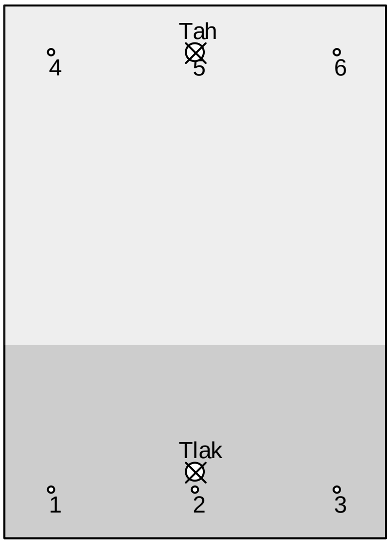
Zatěžovací stav (Výpočtová zatížení):

Kotva - reakce [kN]

Normálová síla: (+ Tah, - Tlak)

Kotva	Normálová síla	Smyková síla
1	0.00	10.17
2	0.00	10.17
3	0.00	10.17
4	118.40	10.17
5	118.40	10.17
6	118.40	10.17

Max.pevnost betonu v tlaku [‰]: 0.61
Max.pevnost betonu v tlaku [N/mm^2]: 15.89
výsledná tahová síla [kN]: 355.20
výsledná tlaková síla [kN]: 647.20



Zatížení tahem (ETAG, příloha C, bod 5.2.2.)

	Výpočtová hodnota [kN]			
Posouzení	Zatížení	Kapacita	Využití β_N [%]	Status
Únosnost oceli	118.40	276.80	43	OK
Vytažení	118.40	334.79	35	OK
Betonový kužel	355.19	760.92	47	OK
Rozlomení	355.19	818.81	43	OK

Únosnost oceli

$N_{Rk,s}$ [kN]	$\gamma_{M,s}$	$N^h_{Rd,s}$ [kN]	N^h_{Sd} [kN]
415.20	1.500	276.80	118.40

Vytažení

$N_{Rk,p}$ [kN]	ψ_c	$\gamma_{M,p}$	$N^h_{Rd,p}$ [kN]	N^h_{Sd} [kN]	zvětšená hloubka osazení
262.01	1.000	1.800	334.79	118.40	2.300

Betonový kužel

$A_{c,N}$ [mm^2]	$A^0_{c,N}$ [mm^2]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	zvětšená hloubka osazení
453600.0	291600.0	270	540	2.300

$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{re,N}$	$\psi_{s,N}$	$\psi_{ucr,N}$
1.000	1.000	1.000	1.000	1.400

$N^0_{Rk,c}$ [kN]	$\gamma_{M,c}$	$N_{Rd,c}$ [kN]	N_{Sd} [kN]
159.72	1.800	760.92	355.19

Rozlomení

$A_{c,N}$ [mm^2]	$A^0_{c,N}$ [mm^2]	$c_{cr,sp}$ [mm]	$s_{cr,sp}$ [mm]	zvětšená hloubka osazení
453600.0	291600.0	270	540	2.300

$\psi_{s,N}$	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{re,N}$	$\psi_{ucr,N}$	$\psi_{h,sp}$
1.000	1.000	1.000	1.000	1.400	1.300

$N^0_{Rk,c}$ [kN]	$\gamma_{M,sp}$	$N_{Rd,sp}$ [kN]	N_{Sd} [kN]
159.72	1.800	818.81	355.19

Zatížení smykem (ETAG, příloha C, bod 5.2.2.)

	Výpočtová hodnota [kN]			
Posouzení	Zatížení	Kapacita	Využití β_V [%]	Status
Únosnost oceli (bez distanční montáže)	10.17	199.30	5	OK
Únosnost oceli (distanční montáž)	10.17	24.42	42	OK
Vylomení betonu	10.17	329.22	3	OK
Selhání okraje betonu ve směru y+	61.00	100.48	61	OK

Únosnost oceli (bez distanční montáže)

$V_{Rk,s}$ [kN]	$\gamma_{M,s}$	$V^h_{Rd,s}$ [kN]	V^h_{Sd} [kN]
249.13	1.250	199.30	10.17

Únosnost oceli (distanční montáž)

l [mm]	α_M
60	2.00

$N_{Sd} / N_{Rd,s}$	$1 - N_{Sd} / N_{Rd,s}$	$M^0_{Rk,s}$ [kNm]	$M_{Rk,s} = M^0_{Rk,s} (1 - N_{Sd} / N_{Rd,s})$ [kNm]
0.428	0.572	1.60	0.92

$V^M_{Rk,s} = \alpha_M * M_{Rk,s} / l$ [kN]	$\gamma_{M,s,b}$	$V^M_{Rd,s}$ [kN]	V^h_{Sd} [kN]
30.53	1.250	24.42	10.17

Vylomení betonu

$A_{c,N}$ [mm^2]	$A^0_{c,N}$ [mm^2]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	k-factor	zvětšená hloubka osazení
840000.0	291600.0	270	540	2.000	2.300

$\psi_{s,N}$	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{re,N}$	$\psi_{ucr,N}$
1.000	1.000	1.000	1.000	1.400

$N^0_{Rk,c}$ [kN]	$\gamma_{M,c,p}$	$V^h_{Rd,c1}$ [kN]	V^h_{Sd} [kN]
159.72	1.500	329.22	10.17

Selhání okraje betonu ve směru y+

l_f [mm]	d_{nom} [mm]	c_1 [mm]	$A_{c,V}$ [mm^2]	$A^0_{c,V}$ [mm^2]
270	30	533	720000.0	1280000.0

$\psi_{s,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	$\psi_{ec,V}$	$\psi_{ucr,V}$
0.812	1.000	1.000	1.000	1.400

$V^0_{Rk,c}$ [kN]	$\gamma_{M,c}$	$V_{Rd,c}$ [kN]	V_{Sd} [kN]
235.55	1.500	100.48	61.00

Kombinované zatížení (ETAG, příloha C, bod 5.2.4)

β_N	β_V	α	Využití $\beta_{N,V}$ [%]	Status
0.467	0.607	1.5	79	OK

$\beta_N^\alpha + \beta_V^\alpha <= 1$

$(\beta_N + \beta_V) / 1.2 <= 1$

Okrajová výztuž

Okrajová výztuž není potřebná pro zabránění rozlomení betonového prvku!
Okrajová výztuž není potřebná z hlediska selhání okraje betonu

Posuny

Posun nejvíce zatížené kotvy by měl být počítán dle příslušného certifikátu. Posuny vlivem tolerance otvoru mohou být zanedbány, protože tato metoda předpokládá vyplnění otvoru (Hilti Dynamická Sada). Charakteristické zatížení nejvíce namáhané kotvy je

N^h_{Sk} = 78.75 [kN]

V^h_{Sk} = 15.06 [kN]

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

Posouzení únosnosti základního materiálu

Přenos zatížení na základní materiál
Kontrola přenosu zatížení na základní materiál musí být v souladu s podmínkami ETAG, bod 7.1!

Pevnost ve smyku základního materiálu
Kontrola pevnosti ve smyku základního materiálu musí být v souladu s příslušným certfikátem nebo Eurokódem 2!

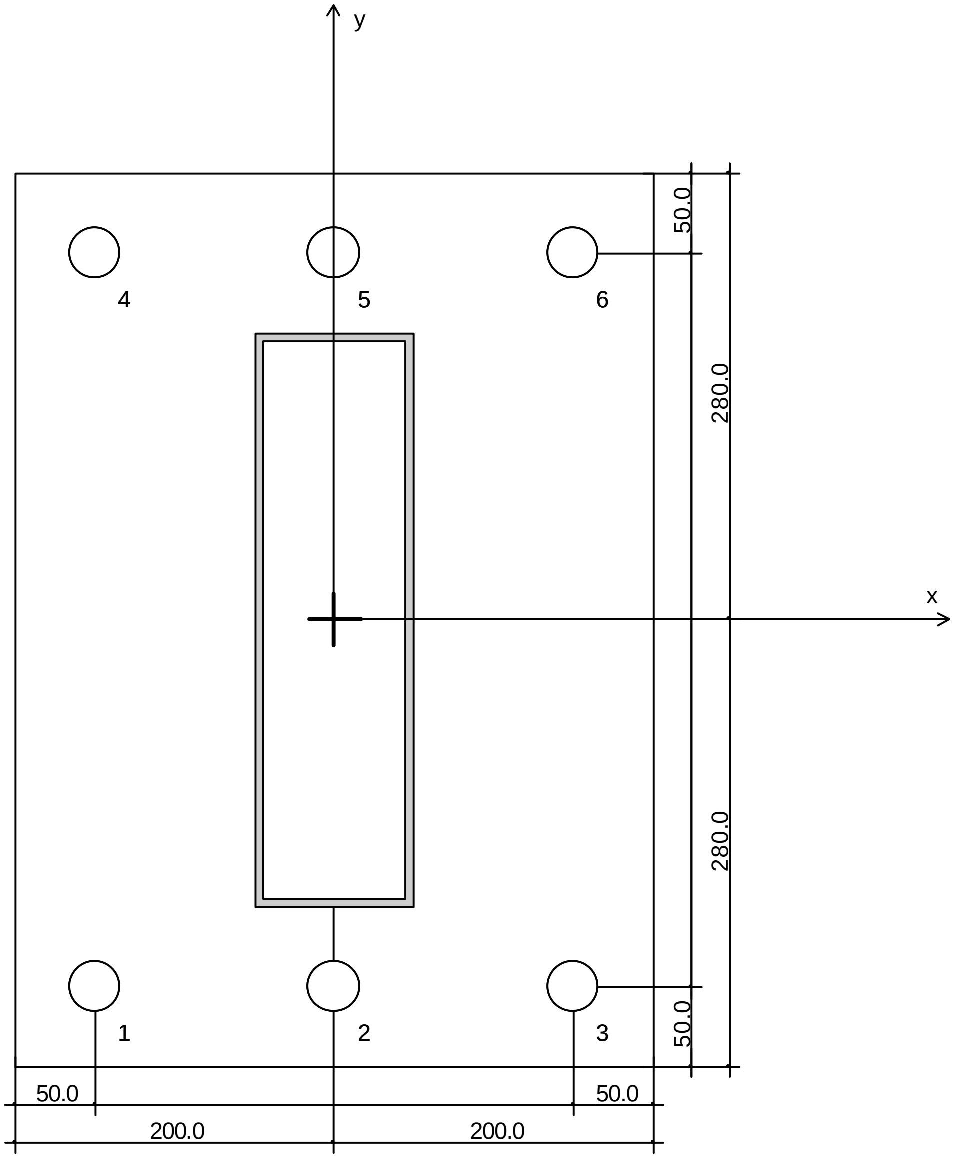
Upozornění

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!

ocelová kotevní deska: S355 (ST52)

Typ profilu: Obdélníkový dutý profil - uživatelsky definováno (360 x 100 x 5)
Průměr otvoru d_f = 33 mm
Doporučená tloušťka desky: nepočítaná



Souřadnice kotvy [mm]

Kotva	x	y	Kotva	x	y
1	-150	-230	4	-150	230
2	0	-230	5	0	230
3	150	-230	6	150	230

Souřadnice kotevní desky [mm]

x	y	x	y
-200	280	200	-280
200	280	-200	-280

Ztužidla

Soubory modelu konstrukce:

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

Soubory zatížení:

CZ1138 Loading. xlsm

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

Poznámky:

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

……………………………………………………………………………………………………..

	Č. projektu:	CZ1138	Projekt:	Basketbalová hala	Místo výst.:	Frýdek-Místek
	Datum:	**19.7.2018**	Vypracoval:	**Lenka Burgerová**	Kontroloval:	**Jaroslav Kosinka**
	Filename:	CZ1138_Loading.v1.16.140630.xlsm			Výpočet dle:	**ČSN EN**

H. Ztužidla v podélné stěně

Základní informace projektu:

Šířka haly:	33.302	m	Horní pás:	5.71	°	Výška u žlabu:	8.35	m
Délka haly:	43.465	m	Dolní pás:	0	°	Výška hřebene:	10.05	m

Modulová vzdálenost rámů: **5.70** m

Síly v podélném stěnovém ztužení

Zatížení větrem ČSN EN 1991-1-4 ()

Kategorie terénu: III. Oblasti rovnoměrně pokryté vegetací, budovami nebo překážkami (vesnice, lesy)

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

Maximální dynamický tlak q_p (z) = 0.669 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)

Součinitel vnějšího tlaku c_{pe} =	0.70	
Výpočtová hodnota $w_{e\,(tlak)}$ =	0.47	kN/m^2
Součinitel zatížení g_w =	1.5	
Normová hodnota $w_{d(tlak)}$ =	0.70	kN/m^2
Síly na horní část stěny w_{dn} =	**117.54**	kN

Závětrná stěna (sání)

Součinitel vnějšího tlaku c_{pe} =	0.30	
Výpočtová hodnota $w_{e\,(sání)}$ =	0.20	kN/m^2
Součinitel zatížení g_w =	1.5	
Normová hodnota $w_{d\,(sání)}$ =	0.30	kN/m^2
Síly na horní část stěny w_{dz} =	**50.37**	kN

Součet sání a tlaku do štítu = **167.91** kN

Tření větru o povrch haly ČSN EN 1991-1-4 ()

Délka na které dochází ke tření (vlna do 60mm) x =	3.27	m
Součinitel tření pro hrubé povrchy c_{fr} =	0.04	
Třecí šířka ve střeše a =	33.48	m
Síla od tření ve střeše w_{fd} =	**4.39**	kN
Třecí šířka ve stěně a =	8.35	m
Síla od tření ve stěně w_{fd} =	**1.09**	kN

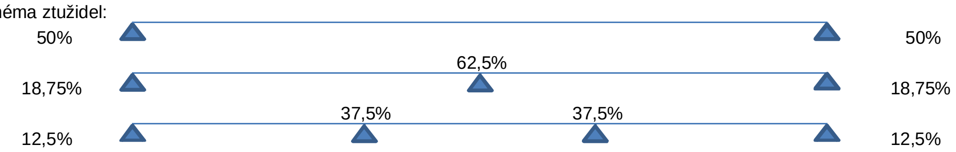
Zatížení jeřábovou dráhou

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** %

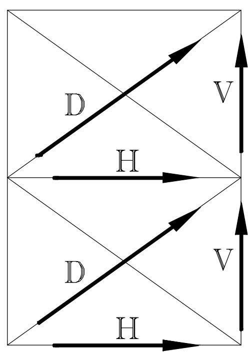
Statické schéma ztužidel:



Typ ztužidel: **Dvojitá ztužidla**

Stěna s dvojitým ztužidlem

Typ zavětrovacích pásků: **Tuhé zavětrování**



Výška ztužidla h =	**8.15**	m
Počet ztužidel v jedné řadě =	2	
Síla přenášená do 1 řady ztužidel w_1 =	**86.69**	kN
Sklon ztužidla a =	35.56	°
Smyková síla do kotvení H =	**43.35**	kN
Tah/tlak do základů V =	**61.98**	kN
Síla v zavětrování D =	**53.29**	kN

Posouzení zavětrování

D = **53.29** kN < $R_{d,p}$ = **NEPRAVDA** kN

ZAVĚTROVÁNÍ VYHOVUJE

Únosnosti profilů

Soubory modelu konstrukce:

………………………………………………………………………………………………………..
………………………………………………………………………………………………………..

Soubory zatížení:

………………………………………………………………………………………………………..
………………………………………………………………………………………………………..

Poznámky:

………………………………………………………………………………………………………..
………………………………………………………………………………………………………..
………………………………………………………………………………………………………..
………………………………………………………………………………………………………..
………………………………………………………………………………………………………..
………………………………………………………………………………………………………..
………………………………………………………………………………………………………..
………………………………………………………………………………………………………..

PROPRTIES OF MATERIALS

Structural steel

Continuous hot dip zinc coated carbon steel for cold forming, material acc. EN10346

Members

Thickness [mm]	Grade	Zink coat	Yeild Strength f_{yb} [N/mm^2]	Ultimate f_u [N/mm^2]
1.5	S 350 GD	Z275MA	350	420
2.0	S 350 GD	Z275MA	350	420
3.0	S 350 GD	Z450MA	350	420
4.0	HX420LAD	Z450MA	420	480
5.0	HX500LAD	Z450MA	500	550
6.0	HX500LAD	Z450MA	500	550
7.0	HX420LAD	Z450MA	420	480

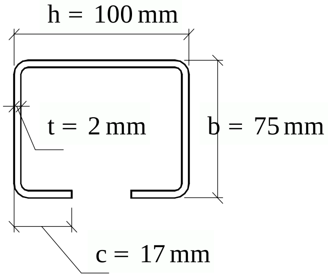
Trapezodial profiled sheets, material acc. EN10346

Thickness [mm]	Grade	Zink coat	Yeild Strength f_{yb} [N/mm^2]	Ultimate f_u [N/mm^2]
0.5	S 250 GD	Z275MA	250	330
0.63	S 320 GD	Z275MA	320	390
0.7 - 1.5	S 350 GD	Z275MA	350	420

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):

$h = 100\,mm$

$b = 75\,mm$

$c = 17\,mm$

$t = 2\,mm$

$A_g = 531.40\,mm^2$

$f_{yb} = 350\,\frac{N}{mm^2}$

$f_u = 420\,\frac{N}{mm^2}$

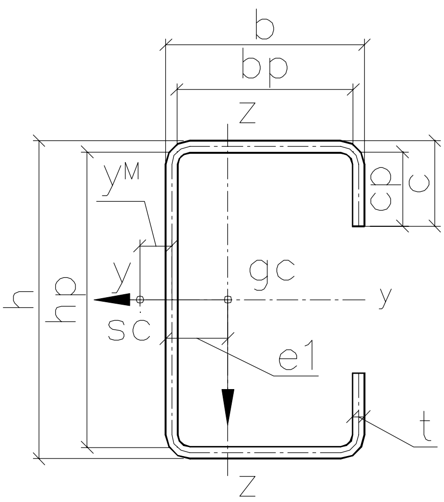
Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 1.52\,mm$

$L_z := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$ $L_y := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$

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



$e_1 = 28.43\,mm$

$y_M = 37.65\,mm$

$W_y = 1.87\times 10^4\,mm^3$

$W_{z1} = 1.47\times 10^4\,mm^3$

$W_{z2} = 8860.64\,mm^3$

$I_y = 9.16\times 10^5\,mm^4$

$I_z = 4.04\times 10^5\,mm^4$

$E\cdot I_y = 192325\,N\cdot m^2$

$E\cdot I_z = 84787\,N\cdot m^2$

$i_y = 41.51\,mm$

$i_z = 27.56\,mm$

$I_T = 736\,mm^4$

$I_\omega = 8.67\times 10^8\,mm^6$

$g_p = 4.16\,\frac{kg}{m}$

corners = "Rounded corners"

date: 2011-05-07

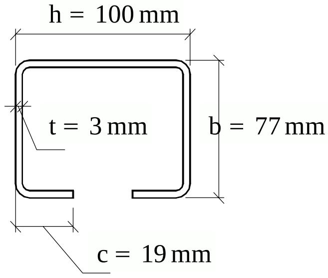
13

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):

$h = 100\,mm$

$b = 77\,mm$

$c = 19\,mm$

$t = 3\,mm$

$A_g = 806.52\,mm^2$

$f_{yb} = 350\,\frac{N}{mm^2}$

$f_u = 420\,\frac{N}{mm^2}$

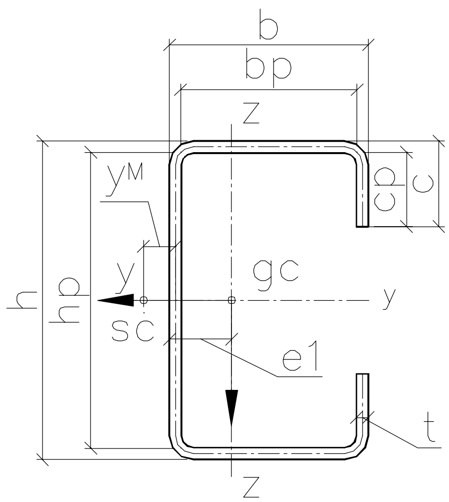
Change of centre of gravity in y-direction, due to effective plates under compression:
(**-**) = to the right

$\Delta e_N = 1.5\,mm$

$L_z := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m \quad L_y := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)
0.5	269.74	267.27
1	259.13	244.14
1.5	242.72	214.83
2	223.23	177.84
2.5	199.81	139.81
3	173.49	108.32
3.5	147.19	84.79
4	123.62	67.61
4.5	103.88	54.94
5	87.83	45.44
5.5	74.91	38.15
6	64.46	32.47
6.5	55.97	27.96
7	49.00	24.32
7.5	43.22	21.34
8	38.39	18.87
8.5	34.32	16.81
9	30.85	15.07
9.5	27.88	13.58
10	25.32	12.31

	$N_{TF.Rd}$ (kN)
0.5	247.91
1	192.65
1.5	131.49
2	91.39
2.5	68.34
3	54.45
3.5	45.47
4	39.27
4.5	34.75
5	31.29
5.5	28.53
6	26.26
6.5	24.32
7	22.64
7.5	21.16
8	19.82
8.5	18.61
9	17.51
9.5	16.49
10	15.55



$e_1 = 29.97\,mm$

$y_M = 38.94\,mm$

$W_y = 2.8\times 10^4\,mm^3$

$W_{z1} = 2.24\times 10^4\,mm^3$

$W_{z2} = 1.4\times 10^4\,mm^3$

$I_y = 1.36\times 10^6\,mm^4$

$I_z = 6.39\times 10^5\,mm^4$

$E\cdot I_y = 285635\,N\cdot m^2$

$E\cdot I_z = 134090\,N\cdot m^2$

$i_y = 41.07\,mm$

$i_z = 28.14\,mm$

$I_T = 2520\,mm^4$

$I_\omega = 1.42\times 10^9\,mm^6$

$g_p = 6.28\,\frac{kg}{m}$

corners = "Rounded corners"

date: 2011-05-07

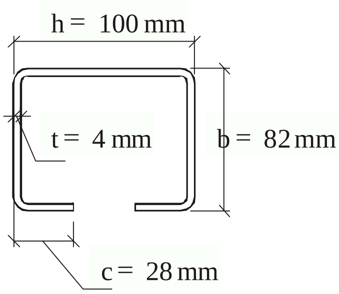
15

Section properties according to EN 1993-1-3:

$\gamma_{M0} = 1.00 \; \gamma_{M1} = 1.00$

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):

$h = 100\ mm$

$b = 82\ mm$

$c = 28\ mm$

$t = 4\ mm$

$A_g = 1167.93\ mm^2$

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

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

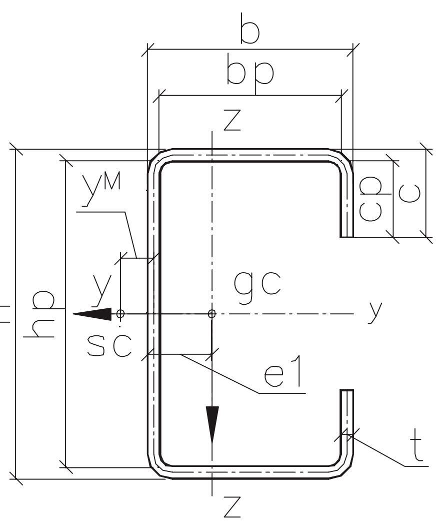
$L_z := 0.5\ m, 1.0\ m .. 10.0\ m \quad L_y := 0.5\ m, 1.0\ m .. 10.0\ m$

$L_z =$ (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

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 0.16\ mm$



$e_1 = 35.12\ mm$

$y_M = 44.9\ mm$

$W_y = 3.93 \times 10^4\ mm^3$

$W_{z1} = 3.33 \times 10^4\ mm^3$

$W_{z2} = 2.45 \times 10^4\ mm^3$

$I_y = 1.89 \times 10^6\ mm^4$

$I_z = 1.1 \times 10^6\ mm^4$

$E \cdot I_y = 396050\ N \cdot m^2$

$E \cdot I_z = 231314\ N \cdot m^2$

$i_y = 40.18\ mm$

$i_z = 30.71\ mm$

$I_T = 6485.33\ mm^4$

$I_\omega = 3.25 \times 10^9\ mm^6$

$g_p = 9.05 \frac{kg}{m}$

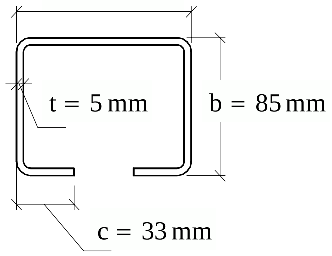
corners = "Rounded corners"

date: 2013-03-06

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):

$h = 100\ mm$

$b = 85\ mm$

$c = 33\ mm$

$t = 5\ mm$

$A_g = 1515.62\ mm^2$

$f_{yb} = 500 \frac{N}{mm^2}$

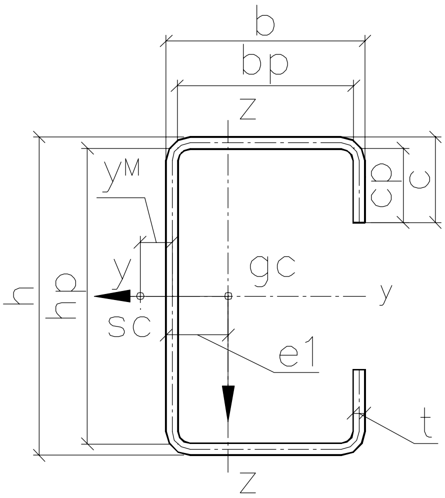
$f_u = 550 \frac{N}{mm^2}$

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 0\ mm$

$L_z := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$ $L_y := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)		$N_{TF.Rd}$ (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



$e_1 = 38.01\ mm$

$y_M = 47.78\ mm$

$W_y = 4.96 \times 10^4\ mm^3$

$W_{z1} = 4.34 \times 10^4\ mm^3$

$W_{z2} = 3.46 \times 10^4\ mm^3$

$I_y = 2.36 \times 10^6\ mm^4$

$I_z = 1.54 \times 10^6\ mm^4$

$E \cdot I_y = 494738\ N \cdot m^2$

$E \cdot I_z = 323343\ N \cdot m^2$

$i_y = 39.43\ mm$

$i_z = 31.87\ mm$

$I_T = 1.32 \times 10^4\ mm^4$

$I_\omega = 5.32 \times 10^9\ mm^6$

$g_p = 11.68 \frac{kg}{m}$

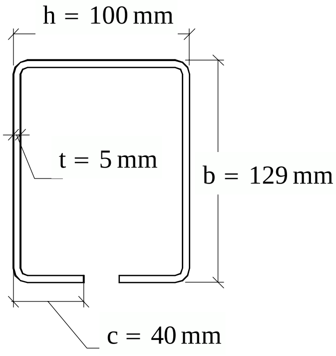
corners = "Rounded corners"

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section AC

kb = 400mm

$M_{ycRd} = 34.15\ kN \cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 35.25\ kN \cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 35.25\ kN \cdot m$	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)
$V_{b.Rd} = 245.37\ kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 357.96\ kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 150.46\ kN$	Reaction force (RF) resistance web (h), 100mm support (6.1.7)
$R_{w.Rd1.1} = 79.34\ kN$	RF resistance web (h), 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 168.94\ kN$	RF resistance web (h), 200mm support (6.1.7)
$R_{w.Rd1.2} = 92.56\ kN$	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)
$R_{w.Rd3.1} = 175.84\ kN$	RF resistance restrained web (h), 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 1047.81\ kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 960.87\ kN$	Axial compression resistance (6.1.3):



$h = 100\ mm$

$b = 129\ mm$

$c = 40\ mm$

$t = 5\ mm$

$A_g = 2025.62\ mm^2$

$f_{yb} = 500 \dfrac{N}{mm^2}$

$f_u = 550 \dfrac{N}{mm^2}$

Change of centre of gravity in y-direction, due to effective plates under compression:
(**-**) = to the right

$\Delta e_N = 2.44\ mm$

$e_1 = 61.44\ mm$

$y_M = 73.07\ mm$

$W_y = 7.08 \times 10^4\ mm^3$

$W_{z1} = 7.78 \times 10^4\ mm^3$

$W_{z2} = 7.05 \times 10^4\ mm^3$

$I_y = 3.36 \times 10^6\ mm^4$

$I_z = 4.59 \times 10^6\ mm^4$

$E \cdot I_y = 706147\ N \cdot m^2$

$E \cdot I_z = 963239\ N \cdot m^2$

$i_y = 40.74\ mm$

$i_z = 47.59\ mm$

$I_T = 1.74 \times 10^4\ mm^4$

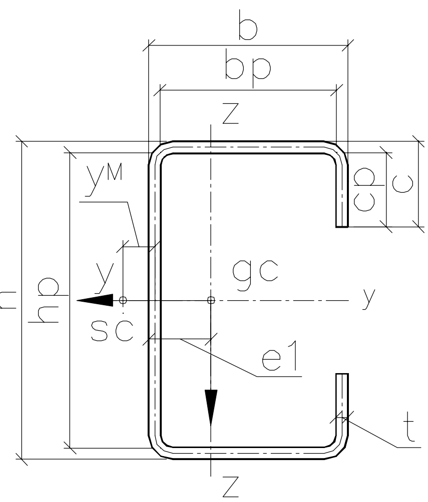
$I_\omega = 1.98 \times 10^{10}\ mm^6$

$g_p = 15.69 \dfrac{kg}{m}$

$L_z := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$ $L_y := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)
0.5	960.87	960.87
1	900.63	920.03
1.5	824.50	859.32
2	729.40	786.63
2.5	617.82	699.19
3	506.33	602.07
3.5	410.37	507.02
4	334.12	423.53
4.5	275.12	354.59
5	229.50	299.08
5.5	193.91	254.62
6	165.77	218.86
6.5	143.21	189.85
7	124.90	166.09
7.5	109.85	146.44
8	97.33	130.02
8.5	86.83	116.18
9	77.93	104.42
9.5	70.32	94.34
10	63.78	85.64

	$N_{TF.Rd}$ (kN)
0.5	893.07
1	710.72
1.5	489.42
2	331.2
2.5	237.63
3	180.93
3.5	144.47
4	119.65
4.5	101.91
5	88.7
5.5	78.51
6	70.4
6.5	63.8
7	58.29
7.5	53.61
8	49.58
8.5	46.04
9	42.92
9.5	40.13
10	37.63



corners = "Rounded corners"

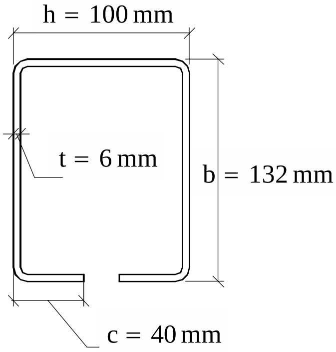
date: 2011-05-07

21

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section YC

kb = 400mm



$M_{ycRd} = 41.66\ kN \cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 42.73\ kN \cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 42.73\ kN \cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 162.81\ kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 290.98\ kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 436.48\ kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 213.23\ kN$	Reaction force (RF) resistance web (h), 100mm support (6.1.7)
$R_{w.Rd1.1} = 55.89\ kN$	RF resistance web (h), 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 235.51\ kN$	RF resistance web (h), 200mm support (6.1.7)
$R_{w.Rd1.2} = 63.87\ kN$	RF resistance web (h), 200mm support close to end (6.1.7)
$R_{w.Rd4.1} = 134.83\ kN$	RF resistance restrained web (h), 100mm supp. (6.1.7)
$R_{w.Rd3.1} = 244.23\ kN$	RF resistance restrained web (h), 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 1269.20\ kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 1180.68\ kN$	Axial compression resistance (6.1.3):

$h = 100\ mm$

$b = 132\ mm$

$c = 40\ mm$

$t = 6\ mm$

$A_g = 2437.59\ mm^2$

$f_{yb} = 500\ \frac{N}{mm^2}$

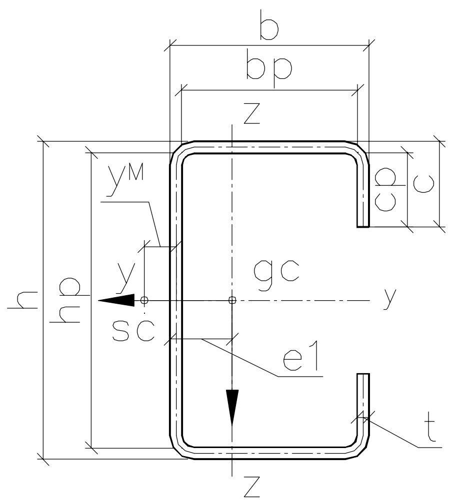
$f_u = 550\ \frac{N}{mm^2}$

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 1.47\ mm$

$L_z := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$ $L_y := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$

$L_z =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rd.z.FB}(L_z) =$ (kN)	L	$N_{TF.Rd}$ (kN)
0.5	1180.68	1180.68	0.5	1093.49
1	1103.56	1130.65	1	863.94
1.5	1007.39	1056.17	1.5	593.56
2	886.80	967.03	2	405.6
2.5	746.32	859.78	2.5	295.29
3	608.16	740.61	3	228.36
3.5	490.99	623.88	3.5	185.04
4	398.80	521.27	4	155.26
4.5	327.87	436.48	4.5	133.69
5	273.24	368.18	5	117.36
5.5	230.70	313.47	5.5	104.55
6	197.12	269.46	6	94.18
6.5	170.23	233.76	6.5	85.57
7	148.42	204.51	7	78.27
7.5	130.50	180.31	7.5	71.98
8	115.62	160.10	8	66.49
8.5	103.12	143.06	8.5	61.63
9	92.54	128.58	9	57.3
9.5	83.50	116.17	9.5	53.42
10	75.71	105.46	10	49.91



$e_1 = 62.9\ mm$

$y_M = 73.95\ mm$

$W_y = 8.48 \times 10^4\ mm^3$

$W_{z1} = 9.43 \times 10^4\ mm^3$

$W_{z2} = 8.55 \times 10^4\ mm^3$

$I_y = 3.99 \times 10^6\ mm^4$

$I_z = 5.65 \times 10^6\ mm^4$

$E \cdot I_y = 836965\ N \cdot m^2$

$E \cdot I_z = 1186272\ N \cdot m^2$

$i_y = 40.44\ mm$

$i_z = 48.14\ mm$

$I_T = 3.02 \times 10^4\ mm^4$

$I_\omega = 2.38 \times 10^{10}\ mm^6$

$g_p = 18.79\ \frac{kg}{m}$

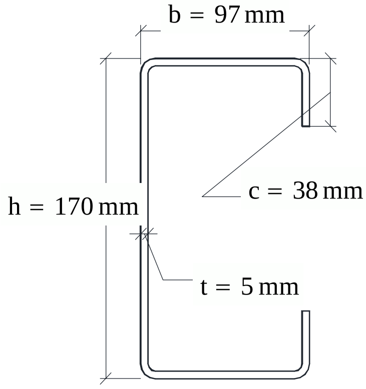
corners = "Rounded corners"

date: 2011-05-07

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section AH

kb = 400mm



$M_{ycRd} = 55.66\ kN \cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 24.54\ kN \cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 24.54\ kN \cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 238.16\ kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 340.64\ kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 265.58\ kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 147.49\ kN$	Reaction force (RF) resistance web (h), 100mm support (6.1.7)
$R_{w.Rd1.1} = 77.21\ kN$	RF resistance web (h), 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 165.6\ kN$	RF resistance web (h), 200mm support (6.1.7)
$R_{w.Rd1.2} = 90.08\ kN$	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)
$R_{w.Rd3.1} = 179.04\ kN$	RF resistance restrained web (h), 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 1052.81\ kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 955.64\ kN$	Axial compression resistance (6.1.3):

$h = 170\ mm$

$b = 97\ mm$

$c = 38\ mm$

$t = 5\ mm$

$A_g = 2035.62\ mm^2$

$f_{yb} = 500 \frac{N}{mm^2}$

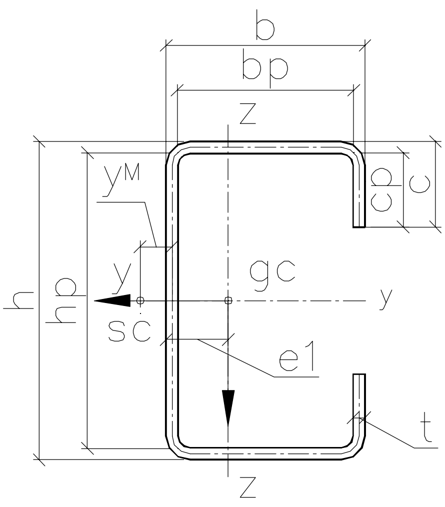
$f_u = 550 \frac{N}{mm^2}$

$L_z := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$ $L_y := 0.5 \cdot m, 1.0 \cdot m .. 10.0 \cdot m$

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = -0.86\ mm$

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	955.64	954.40	0.5	926.51
1	947.62	882.20	1	816.72
1.5	908.76	794.62	1.5	673.47
2	866.61	683.63	2	521.01
2.5	819.01	559.46	2.5	400.24
3	764.35	445.64	3	316.38
3.5	702.38	354.58	3.5	258.94
4	635.13	285.46	4	218.62
4.5	566.60	233.39	4.5	189.37
5	501.09	193.79	5	167.44
5.5	441.53	163.20	5.5	150.49
6	389.11	139.18	6	137.49
6.5	343.82	120.03	6.5	126.06
7	304.99	104.53	7	116.92
7.5	271.79	91.83	7.5	109.15
8	243.37	81.29	8	102.42
8.5	218.94	72.46	8.5	96.51
9	197.87	64.99	9	91.23
9.5	179.60	58.62	9.5	86.47
10	163.68	53.13	10	82.14



$e_1 = 37.88\ mm$

$y_M = 51.4\ mm$

$W_y = 1.12 \times 10^5\ mm^3$

$W_{z1} = 7.85 \times 10^4\ mm^3$

$W_{z2} = 4.91 \times 10^4\ mm^3$

$I_y = 9.22 \times 10^6\ mm^4$

$I_z = 2.78 \times 10^6\ mm^4$

$E \cdot I_y = 1935478\ N \cdot m^2$

$E \cdot I_z = 583567\ N \cdot m^2$

$i_y = 67.29\ mm$

$i_z = 36.95\ mm$

$I_T = 1.75 \times 10^4\ mm^4$

$I_\omega = 2.08 \times 10^{10}\ mm^6$

$g_p = 15.76 \frac{kg}{m}$

corners = "Rounded corners"

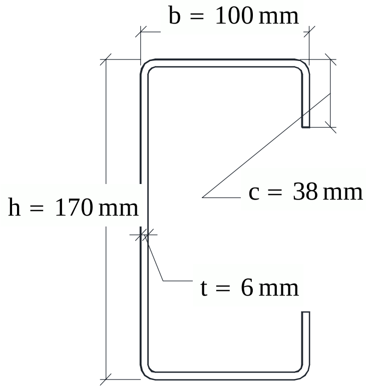
date: 2011-05-07

31

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section YH

kb = 400mm



$M_{ycRd} = 67.22\ kN\cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 29.95\ kN\cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 29.95\ kN\cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 284.06\ kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 405.3\ kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 325.63\ kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 209.74\ kN$	Reaction force (RF) resistance web (h), 100mm support (6.1.7)
$R_{w.Rd1.1} = 54.65\ kN$	RF resistance web (h), 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 231.65\ kN$	RF resistance web (h), 200mm support (6.1.7)
$R_{w.Rd1.2} = 62.46\ kN$	RF resistance web (h), 200mm support close to end (6.1.7)
$R_{w.Rd4.1} = 134.83\ kN$	RF resistance restrained web (h), 100mm supp. (6.1.7)
$R_{w.Rd3.1} = 247.95\ kN$	RF resistance restrained web (h), 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 1275.20\ kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 1217.47\ kN$	Axial compression resistance (6.1.3):

$h = 170\ mm$

$b = 100\ mm$

$c = 38\ mm$

$t = 6\ mm$

$A_g = 2449.59\ mm^2$

$f_{yb} = 500\ \frac{N}{mm^2}$

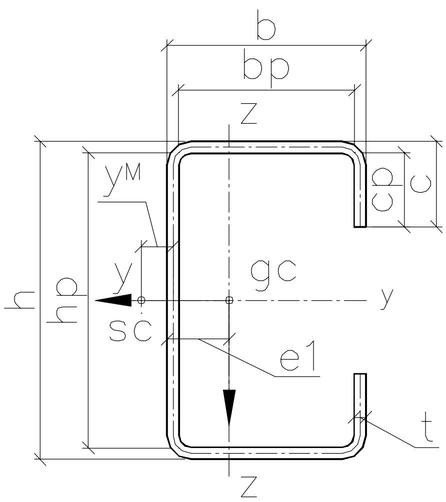
$f_u = 550\ \frac{N}{mm^2}$

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = 0.06\ mm$

$L_z := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$ $L_y := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$

L_z = (m)	$N_{b.Rd.y.FB}(L_y)$ = (kN)	$N_{b.Rd.z.FB}(L_z)$ = (kN)		$N_{TF.Rd}$ (kN)
0.5	1217.47	1214.92	0.5	1176.52
1	1204.14	1121.73	1	1031.89
1.5	1152.80	1008.15	1.5	845.82
2	1096.71	864.13	2	655.27
2.5	1032.88	704.19	2.5	508.7
3	959.27	559.16	3	408.06
3.5	876.02	444.04	3.5	339.2
4	786.67	357.07	4	290.64
4.5	697.15	291.73	4.5	255.09
5	613.11	242.11	5	228.1
5.5	537.88	203.82	5.5	206.89
6	472.48	173.78	6	189.72
6.5	416.46	149.84	6.5	175.44
7	368.75	130.47	7	163.27
7.5	328.15	114.60	7.5	152.69
8	293.50	101.45	8	143.33
8.5	263.82	90.42	8.5	134.94
9	238.26	81.09	9	127.33
9.5	216.13	73.13	9.5	120.37
10	196.88	66.28	10	113.95



$e_1 = 39.18\ mm$

$y_M = 52.3\ mm$

$W_y = 1.34\times 10^5\ mm^3$

$W_{z1} = 9.57\times 10^4\ mm^3$

$W_{z2} = 5.99\times 10^4\ mm^3$

$I_y = 1.1\times 10^7\ mm^4$

$I_z = 3.46\times 10^6\ mm^4$

$E\cdot I_y = 2314932\ N\cdot m^2$

$E\cdot I_z = 727397\ N\cdot m^2$

$i_y = 67.08\ mm$

$i_z = 37.6\ mm$

$I_T = 3.04\times 10^4\ mm^4$

$I_\omega = 2.55\times 10^{10}\ mm^6$

$g_p = 18.88\ \frac{kg}{m}$

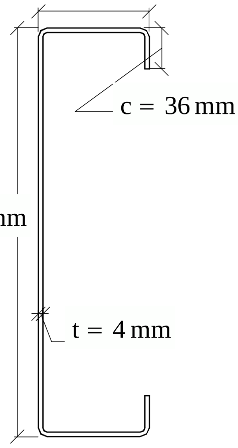
corners = "Rounded corners"

date: 2011-05-07

Section properties according to EN 1993-1-3: $\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section RA

kb = 600mm



$M_{ycRd} = 101.53\,kN\cdot m$	Bending moment resistance y-y (6.1.4)
$M_{z1cRd} = 17.68\,kN\cdot m$	Bending moment resistance z-z tension in web (6.1.4)
$M_{z2cRd} = 17.79\,kN\cdot m$	Bending moment resistance z-z compression in web (6.1.4)
$V_{bh.Rd} = 208.46\,kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{b.Rd} = 274.41\,kN$	Shear force resistance perpendicular to y-axis web+lips (6.1.5)
$V_{bb.Rd} = 186.23\,kN$	Shear force resistance perpendicular to z-axis (6.1.5)
$R_{w.Rd2.1} = 81.76\,kN$	Reaction force (RF) resistance web (h), 100mm support (6.1.7)
$R_{w.Rd1.1} = 44.12\,kN$	RF resistance web (h), 100mm support, close to end (6.1.7)
$R_{w.Rd2.2} = 93.93\,kN$	RF resistance web (h), 200mm support (6.1.7)
$R_{w.Rd1.2} = 52.94\,kN$	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)
$R_{w.Rd3.1} = 107.5\,kN$	RF resistance restrained web (h), 100mm supp., close to end (6.1.7)
$N_{t.Rd} = 1041.57\,kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 630.98\,kN$	Axial compression resistance (6.1.3):

$h = 360\,mm$
$b = 100\,mm$
$c = 36\,mm$
$t = 4\,mm$

$A_g = 2415.93\,mm^2$

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

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

Change of centre of gravity in y-direction, due to effective plates under compression:
(-) = to the right

$\Delta e_N = -9.12\,mm$

$L_z := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$ $L_y := 0.5\cdot m, 1.0\cdot m .. 10.0\cdot m$

L_z = (m)	$N_{b.Rd.y.FB}(L_y)$ = (kN)	$N_{b.Rd.z.FB}(L_z)$ = (kN)		$N_{TF.Rd}$ (kN)
0.5	630.98	630.98	0.5	630.98
1	630.98	605.46	1	607.12
1.5	630.98	566.55	1.5	569.89
2	630.98	520.21	2	526.63
2.5	629.67	464.49	2.5	475.89
3	620.43	402.12	3	419.73
3.5	611.06	340.25	3.5	363.65
4	601.48	285.21	4	312.92
4.5	591.62	239.34	4.5	269.89
5	581.40	202.19	5	234.57
5.5	570.75	172.32	5.5	205.89
6	559.60	148.23	6	182.6
6.5	547.89	128.66	6.5	163.59
7	535.56	112.61	7	147.94
7.5	522.57	99.32	7.5	134.95
8	508.91	88.21	8	124.06
8.5	494.58	78.84	8.5	114.87
9	479.60	70.87	9	107.05
9.5	464.03	64.04	9.5	100.33
10	447.98	58.14	10	94.53

$e_1 = 27.11\,mm$
$y_M = 42.74\,mm$

$W_y = 2.52\times10^5\,mm^3$
$W_{z1} = 1.24\times10^5\,mm^3$
$W_{z2} = 4.4\times10^4\,mm^3$

$I_y = 4.49\times10^7\,mm^4$
$I_z = 3.12\times10^6\,mm^4$

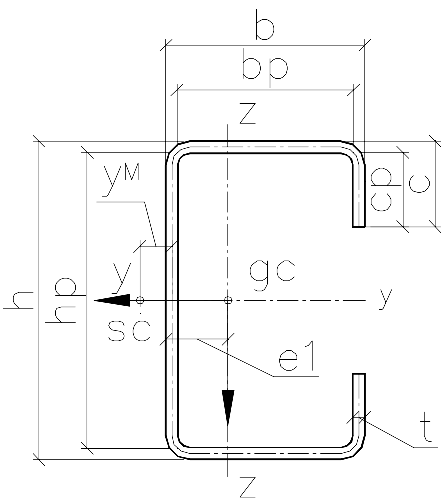
$E\cdot I_y = 9432187\,N\cdot m^2$
$E\cdot I_z = 655209\,N\cdot m^2$

$i_y = 136.35\,mm$
$i_z = 35.94\,mm$

$I_T = 1.31\times10^4\,mm^4$

$I_\omega = 8.07\times10^{10}\,mm^6$

$g_p = 18.85\,\frac{kg}{m}$



corners = "Rounded corners"

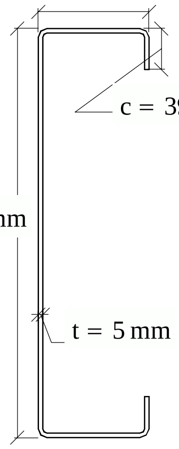
date: 2011-05-07

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):

$h = 360\,mm$

$b = 100\,mm$

$c = 39\,mm$

$t = 5\,mm$

$A_g = 3025.62\,mm^2$

$f_{yb} = 500\,\frac{N}{mm^2}$

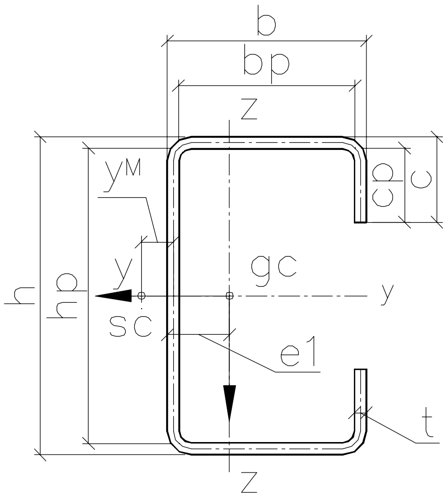
$f_u = 550\,\frac{N}{mm^2}$

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



$e_1 = 27.86\,mm$

$y_M = 43.18\,mm$

$W_y = 3.15\times10^5\,mm^3$

$W_{z1} = 1.54\times10^5\,mm^3$

$W_{z2} = 5.62\times10^4\,mm^3$

$I_y = 5.58\times10^7\,mm^4$

$I_z = 3.92\times10^6\,mm^4$

$E\cdot I_y = 11724166\,N\cdot m^2$

$E\cdot I_z = 822335\,N\cdot m^2$

$i_y = 135.84\,mm$

$i_z = 35.98\,mm$

$I_T = 2.57\times10^4\,mm^4$

$I_\omega = 1.03\times10^{11}\,mm^6$

$g_p = 23.54\,\frac{kg}{m}$

corners = "Rounded corners"

date: 2011-05-07

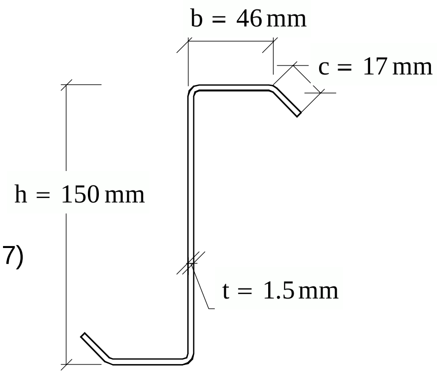
51

Section properties according to EN 1993-1-3:

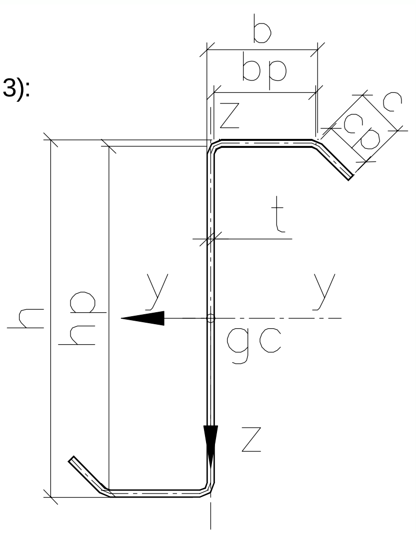
$\gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Section XA

kb = 268mm



$M_{yRd} = 5.76\,kN \cdot m$	Bending moment resistance y-y (6.1.4)
$M_{zRd} = 1.35\,kN \cdot m$	Bending moment resistance z-z (6.1.4)
$V_{bh.Rd} = 26.76\,kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{bb.Rd} = 13.62\,kN$	Shear force resistance perpendicular to z-axis (both flanges) (6.1.5)
$R_{w.Rd2.1} = 12.79\,kN$	Reaction force (RF) resistance web, 108mm support (6.1.7)
$R_{w.Rd1.1} = 7.01\,kN$	RF resistance web, 108mm support, close to end (6.1.7)
$R_{w.Rd4.1} = 15.71\,kN$	RF resistance restrained web, 108mm supp. (6.1.7)
$R_{w.Rd3.1} = 16.17\,kN$	RF resistance restrained web, 108mm supp., close to end (6.1.7)
$N_{t.Rd} = 144.12\,kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 83.9\,kN$	Axial compression resistance (6.1.3):



$h = 150\,mm$

$b = 46\,mm$

$c = 17\,mm$

$t = 1.5\,mm$

$A_g = 402.32\,mm^2$

$f_{yb} = 350 \frac{N}{mm^2}$

$f_u = 420 \frac{N}{mm^2}$

$W_y = 1.83 \times 10^4\,mm^3$

$W_z = 3873.81\,mm^3$

$I_y = 1.36 \times 10^6\,mm^4$

$I_z = 2.2 \times 10^5\,mm^4$

$E \cdot I_y = 285056.7\,N \cdot m^2$

$E \cdot I_z = 46158.4\,N \cdot m^2$

$i_y = 58.09\,mm$

$i_z = 23.37\,mm$

$I_T = 305.73\,mm^4$

$I_\omega = 8.28 \times 10^8\,mm^6$

$g_p = 3.15 \frac{kg}{m}$

Flexural buckling resistance (6.3.1.3)

$L_y =$ (m)	$N_{b.Rd.y.FB}(L_y) =$ (kN)	$N_{b.Rk.z.FB}(L_z) =$ (kN)
0.5	83.90	83.47
1	83.90	76.71
1.5	82.13	68.34
2	79.47	57.71
2.5	76.62	46.28
3	73.51	36.33
3.5	70.04	28.65
4	66.19	22.95
4.5	61.96	18.70
5	57.44	15.49
5.5	52.78	13.03
6	48.18	11.10
6.5	43.79	9.56
7	39.71	8.32
7.5	36.01	7.31
8	32.69	6.46
8.5	29.73	5.76
9	27.10	5.16
9.5	24.78	4.66
10	22.71	4.22
10.5	20.88	3.84
11	19.25	3.51
11.5	17.80	3.22
12	16.49	2.97

torsional buckling resistance (6.3.1.4)

$L_T =$ (m)	$N_{b.Rd.T}(L_T) =$ (kN)
0.5	83.35
1	76.52
1.5	68.22
2	58.08
2.5	47.55
3	38.52
3.5	31.58
4	26.41
4.5	22.56
5	19.66
5.5	17.42
6	15.67
6.5	14.28
7	13.16
7.5	12.25
8	11.49
8.5	10.86
9	10.32

corners = "Rounded corners"

date: 2011-05-16

Section properties according to EN 1993-1-3:

$\gamma_{M0} = 1.00 \quad \gamma_{M1} = 1.00$

Section HA

kb = 268mm

$M_{yRd} = 8.07\,kN \cdot m$ Bending moment resistance y-y (6.1.4)

$M_{zRd} = 1.84\,kN \cdot m$ Bending moment resistance z-z (6.1.4)

$V_{bh.Rd} = 47.57\,kN$ Shear force resistance perpendicular to y-axis web only (6.1.5)

$V_{bb.Rd} = 18.02\,kN$ Shear force resistance perpendicular to z-axis (both flanges) (6.1.5)

$R_{w.Rd2.1} = 21.8\,kN$ Reaction force (RF) resistance web, 108mm support (6.1.7)

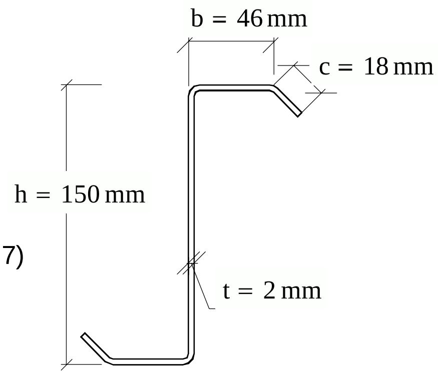
$R_{w.Rd1.1} = 12.82\,kN$ RF resistance web, 108mm support, close to end (6.1.7)

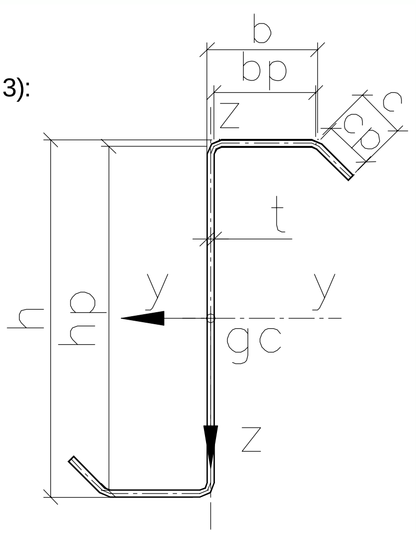
$R_{w.Rd4.1} = 24.47\,kN$ RF resistance restrained web, 108mm supp. (6.1.7)

$R_{w.Rd3.1} = 25.97\,kN$ RF resistance restrained web, 108mm supp., close to end (6.1.7)

$N_{t.Rd} = 193.88\,kN$ Axial tension resistance (6.1.2)

$N_{c.Rd} = 131.31\,kN$ Axial compression resistance (6.1.3):





$h = 150\,mm$

$b = 46\,mm$

$c = 18\,mm$

$t = 2\,mm$

$A_g = 537.13\,mm^2$

$f_{yb} = 350\,\frac{N}{mm^2}$

$f_u = 420\,\frac{N}{mm^2}$

$W_y = 2.43 \times 10^4\,mm^3$

$W_z = 5250.04\,mm^3$

$I_y = 1.8 \times 10^6\,mm^4$

$I_z = 2.99 \times 10^5\,mm^4$

$E \cdot I_y = 377828.8\,N \cdot m^2$

$E \cdot I_z = 62865.9\,N \cdot m^2$

$i_y = 57.88\,mm$

$i_z = 23.61\,mm$

$I_T = 726.25\,mm^4$

$I_\omega = 1.12 \times 10^9\,mm^6$

$g_p = 4.2\,\frac{kg}{m}$

Flexural buckling resistance (6.3.1.3)

L_y = (m)	$N_{b.Rd.y.FB}(L_y)$ = (kN)	$N_{b.Rk.z.FB}(L_z)$ = (kN)
0.5	131.31	129.91
1	131.31	118.37
1.5	127.48	103.64
2	122.87	85.12
2.5	117.87	66.43
3	112.30	51.24
3.5	106.04	40.01
4	99.06	31.85
4.5	91.48	25.86
5	83.58	21.37
5.5	75.74	17.94
6	68.27	15.26
6.5	61.39	13.14
7	55.20	11.42
7.5	49.71	10.02
8	44.88	8.86
8.5	40.64	7.89
9	36.92	7.08
9.5	33.66	6.38
10	30.78	5.78
10.5	28.24	5.26
11	26.00	4.81
11.5	24.00	4.41
12	22.22	4.06

torsional buckling resistance (6.3.1.4)

L_T = (m)	$N_{b.Rd.T}(L_T)$ = (kN)
0.5	129.73
1	118.19
1.5	104.04
2	87.30
2.5	71.05
3	57.93
3.5	48.20
4	41.09
4.5	35.85
5	31.92
5.5	28.92
6	26.58
6.5	24.72
7	23.22
7.5	22.00
8	20.99
8.5	20.15
9	19.44

corners = "Rounded corners"

date: 2011-05-16

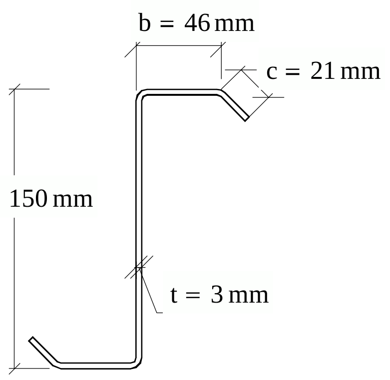
Section properties according to EN 1993-1-3:

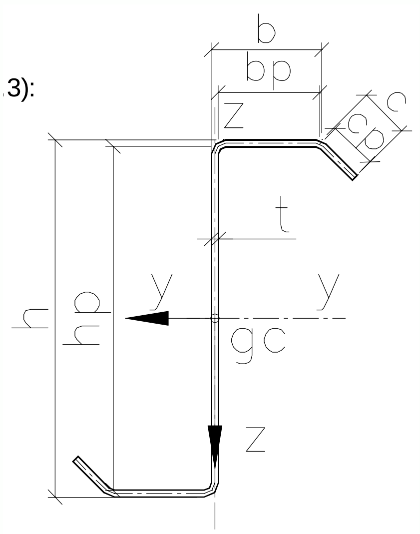
$\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$

Section OA

kb = 268mm

$M_{yRd} = 12.79\ kN \cdot m$	Bending moment resistance y-y (6.1.4)
$M_{zRd} = 2.93\ kN \cdot m$	Bending moment resistance z-z (6.1.4)
$V_{bh.Rd} = 89.11\ kN$	Shear force resistance perpendicular to y-axis web only (6.1.5)
$V_{bb.Rd} = 26.6\ kN$	Shear force resistance perpendicular to z-axis (both flanges) (6.1.5)
$R_{w.Rd2.1} = 47.7\ kN$	Reaction force (RF) resistance web, 108mm support (6.1.7)
$R_{w.Rd1.1} = 29.01\ kN$	RF resistance web, 108mm support, close to end (6.1.7)
$R_{w.Rd4.1} = 44.76\ kN$	RF resistance restrained web, 108mm supp. (6.1.7)
$R_{w.Rd3.1} = 51.68\ kN$	RF resistance restrained web, 108mm supp., close to end (6.1.7)
$N_{t.Rd} = 298.06\ kN$	Axial tension resistance (6.1.2)
$N_{c.Rd} = 245.22\ kN$	Axial compression resistance (6.1.3):





$h = 150\ mm$

$b = 46\ mm$

$c = 21\ mm$

$t = 3\ mm$

$A_g = 813.79\ mm^2$

$f_{yb} = 350 \frac{N}{mm^2}$

$f_u = 420 \frac{N}{mm^2}$

$W_y = 3.66 \times 10^4\ mm^3$

$W_z = 8376.98\ mm^3$

$I_y = 2.69 \times 10^6\ mm^4$

$I_z = 4.88 \times 10^5\ mm^4$

$E \cdot I_y = 564218.5\ N \cdot m^2$

$E \cdot I_z = 102539.3\ N \cdot m^2$

$i_y = 57.46\ mm$

$i_z = 24.5\ mm$

$I_T = 2479.63\ mm^4$

$I_\omega = 1.83 \times 10^9\ mm^6$

$g_p = 6.33 \frac{kg}{m}$

Flexural buckling resistance (6.3.1.3)

L_y = (m)	$N_{b.Rd.y.FB}(L_y)$ = (kN)	$N_{b.Rk.z.FB}(L_z)$ = (kN)
0.5	245.22	241.20
1	244.37	217.67
1.5	235.08	186.76
2	225.16	148.82
2.5	214.11	113.24
3	201.56	86.06
3.5	187.32	66.65
4	171.62	52.81
4.5	155.14	42.75
5	138.83	35.26
5.5	123.50	29.55
6	109.64	25.11
6.5	97.42	21.59
7	86.79	18.76
7.5	77.59	16.46
8	69.65	14.55
8.5	62.79	12.95
9	56.84	11.60
9.5	51.66	10.45
10	47.13	9.47
10.5	43.16	8.61
11	39.66	7.87
11.5	36.56	7.22
12	33.80	6.65

torsional buckling resistance (6.3.1.4)

L_T = (m)	$N_{b.Rd.T}(L_T)$ = (kN)
0.5	240.91
1	217.96
1.5	190.48
2	160.13
2.5	132.91
3	111.98
3.5	96.80
4	85.84
4.5	77.82
5	71.82
5.5	67.24
6	63.68
6.5	60.86
7	58.59
7.5	56.74
8	55.22
8.5	53.95
9	52.87

corners = "Rounded corners"

date: 2011-05-16

Příloha: Překladový slovník výrazů statického výpočtu

Část 1: posudky vaznic (strany 21-47)

Anglické znění:	*překlad:*
Control of purlin Z-150 according to EN 1993-1-3	Posudek vaznice Z-150 dle EN 1993-1-3
Values for	Hodnoty pro
Global values: all measures in (mm)	Obecné parametry: všechny hodnoty v (mm)
Thickness roofplate	Tloušťka střešního plechu
Width of roof	Šířka střechy
Distance between purlins	Vzdálenost mezi vaznicemi
Core thickness	Tloušťka jádra
Span of purlin	Délka vaznice
Support width	Šířka podpory
Stresses on roof purlin	Zatížení na střešní vaznici
Maximum gravity load	Maximální tlakové zatížení
Maximum upplift load	Maximální vztlakové zatížení
Compression force in purlin	Tlaková síla ve vaznici
Stresses due to upplift load	Zatížení vyvozená vztlakovým zatížením
Moment in span	Ohybový moment v poli
Moment at connection between purlins	Ohybový moment v přípoji vaznic
Shear force at support	Smyková síla v podpoře
Reaction force	Reakce
Shear force at connection between purlins	Smyková síla v přípoji vaznic
Purlin profiles	Profily vaznice
Profile over support	Nadpodporový profil
Profile in span	Profil v poli
Supporting profile	Výztužný profil
Length of extra supporting profile over support	Délka výztužného profilu nad podporou
Shear stiffnes of trapezoidal sheeting connected to purlin	Smyková tuhost trapézového plechu připojeného na vaznici
Shear stiffnes roof	Smyková tuhost střešní roviny
Purlin	Vaznice
Purlin at the connection is regarded as Beeing laterally restrained in plane of sheeting	Vaznice v přípoji je uvažovaná jako držená proti vybočení v rovině opláštění
Lateral coefficient	Koeficienty vybočení
Free flange	Volná příruba
Rotational restraint given by sheeting	Rotační tuhost vlivem opláštění
Stiffness	Tuhost
Flexural stiffness	Ohybová tuhost
Tension in upper flange	Tah v horní přírubě
Compression if upper flange	Tlak v horní přírubě
Rotational stiffness of connection between sheeting and purlin	Rotační tuhost přípoje mezi opláštěním a vaznicí
pin in every trough	spojovací prvek v každém otvoru
Lateral spring stiffness per unit lenght	Pružná příčná tuhost na jednotku délky
Gross properties of the free flange	Průřezové vlastnosti volné příruby
Lateral bending moment for free flanges	Příčný ohybový moment pro volné příruby

Anglické znění:	*překlad:*
in compression	v tlaku
coefficient R of the spring support	koeficient R pružné podpory
lateral bending moment	příčný ohybový moment
Stresses due to gravity load as given above	Zatížení od tlakového zatížení uvedených výše
Stresses due to uplift load as given above	Zatížení od vztlakového zatížení uvedených výše
Combined bending moment and support reaction	Kombinace ohybového momentu a reakce v podpoře
The web rotation is prevented	Zamezeno pootočení stojiny
Web	Stojina
Check single profile	Ověření samotného profilu
Check with supporting profile	Ověření i s výztužným profilem
Combined bending moment and compression force	Kombinace ohybového momentu a tlakové síly
Single profile at end of supporting profile	Samotný profil na konci výztužného profilu
Check joint between purlins	Ověření přípojů vaznice
Joint near support A gravity load	Přípoj u podpory A na tlakové zatížení
Joint near support A uplift load	Přípoj u podpory A na vztlakové zatížení
Buckling resistance	Vzpěrná tuhost
Buckling resistence of free flange in compression	Vzpěrná tuhost volné příruby v tlaku
Non-dim. Slenderness	Štíhlost
Buckling length	Vzpěrná délka
Relative slenderness for flexural buckling of free flange	Relativní štíhlost volné příruby v rovinném vzpěru
Reduction factor	Redukční součinitel
Flexural buckling	Rovinný vzpěr
Lateral torsional buckling	Prostorový vzpěr
Imperfection factor α relating to buckling curve b	Faktor imperfekce α vztažený na vzpěrnou křivku b
Bracing of Z-roof purlin	Stabilizace vaznice ze Z-profilů
Degree	Stupeň
Roof slope	Sklon střechy
Load width frame	Zatěžovací šířka rámu
Ridge flashing	Hřebenové lemování
S-hall	Sedlová hala
P-hall	Pultová hala
If the forces due to lateral and torsional bending is acting positive for sloped roofs, it will be reduced due to imperfection	Jestli jsou síly vyvozené příčným a torzním ohybem kladné pro daný sklon střechy, budou redukovány kvůli imperfekcím
Force acting in the roof plane for one half of the building	Síla působící ve střešní rovině na jednu polovinu střechy
Shear resistance	Smyková únosnost
Screw	Šroub
Shear resistance of screws	Smyková únosnost šroubů
Overlap screw	Overlap (plášťový samořezný šroub)
Overlap screw for use in ridge connection	Overlapy v hřebenovém přípoji
Ultimate strength	Pevnost (únosnost)
Plate	Plech

Anglické znění:	*překlad:*
Fastening	Připojení (spojení)
Roofbrace	Střešní ztužení
Bearing resistance	Nosnost
Platescrew	Šroub do plechu
Part of force in plane of roof which is taken By ridge flashing	Část síly ve střešní rovině která je přenášena hřebenovým lemováním
Compression	tlak
Tension	tah
Resistance of overlap screw	únosnost overlapu
Number of braces needed for each half of the roof	Počet ztužidel potřebných na jednu polovinu střechy
Design resistance	Návrhová únosnost
If using C-profile	Za použití C-profilu

Část 2: posudky prvků rámu (příhrady a sloupy) (strany 64-129)

Anglické znění:	*překlad:*
Truss	Příhradový nosník
Member	Prvek
Double	Dvojitý
Stresses and global geometry	Zatížení a geometrické vlastnosti
Design	Návrh (navržen)
Uniform built up member	Celistvý sestavený prvek
Buckling lengths	Vzpěrné délky
Length	Délka
Diagonal	Diagonála
Length for LT-buckling	Délka pro prostorový vzpěr
Number of battens per L	počet spojovacích destiček na délku
Flexural buckling	Ohybový vzpěr
Resistance	Únosnost
Slenderness for flexural buckling	Štíhlost pro ohybový vzpěr
Elastic critical buckling moment	Elastický kritický vzpěrný moment
Check uniform built-up member	Posudek složeného prvku
Effective second moment of area of battened built-up member	Efektivní moment setrvačnosti spojeného prvku
Chord	Pas
Distance centroids of chords	Vzdálenost středů pasů
Efficiency factor	faktor účinnosti
Shear stiffness	Smyková tuhost
Second order effect	Účinky druhého řádu
Interaction formula	Vzorec pro spolupůsobení
Member susceptible to torsion deformation	Prvek náchylný na deformaci krutem
Combined bending and axial compression	Kombinace ohybu a osového tlak
Evading in y-y	Vybočení ve směru y-y
Evading in z-z	Vybočení ve směru z-z
Simply symmetric section	Jednou symetrický průřez

Anglické znění:	*překlad:*
Moment at quater point of unbraced segment	Moment ve čtvrtině nevyztuženého prvku
Negligible	Zanedbatelný
Interaction factors kij	Interakční součinitele kij
Suspectible	Náchylné
Deformation	Deformace
At mid-span of built-up member	Uprostřed složeného prvku
In end panel of built-up member	Na konci složeného prvku
Profile data for single profile	Charakterstiky jednoho profilu
Beam	Prut (nosník)
Hole diameter	Průměr díry (otvoru)
Momentcap. One profile	Momentová únosnost jednoho profilu
Axial force capacity one profile	Únosnost jednoho profilu při namáhání osovou silou
Nett area of profile in tension	aktivní plocha profilu v tahu
Capacity	Únosnost
Shearforce capacity	Smyková únosnost
Cross-section	Průřez
Moment resistance of a cross-section consisting only flanges	Momentová únosnost průřezu uvažující pouze příruby
ratio	poměr (poměr využití)
Frame	Rám
Column	Sloup
Section properties of frame column	Průřezové charakteristiky rámového sloupu
Four-part C-profiles	Složen ze čtyř C-profilů
Innerprofile	Vnitřní profil
Outerprofile	Vnější profil
First order frame analysis	Výpočet teorií prvního řádu
Figure	Obrazec
Length pillar	Délka sloupu
Width frame	Šířka rámu
Height truss	Výška příhradového nosníku
Area upper chord	Plocha horního pasu
I upper chord	Moment setrvačnosti horního pasu
Area lower chord	Plocha spodního pasu
I lower chord	Moment setrvačnosti spodního pasu
Number of profiles upper chord	Počet prvků tvořících horní pas
Number of profiles lower chord	Počet prvků tvořících spodní pas
Lateral torsional buckling	Prostorový vzpěr
Distance flange bracings	Vzdálenost stabilizací příruby
Inner flange in compression	Vnitřní pásnice v tlaku
Wrap restraints	Vyztužení proti vlnění (krabatění)
Buckling resistance	Vzpěrná únosnost
Reduction buckling factor	Redukční vzpěrný součinitel
Moment distribution	Momentový průběh
Screws between C-profiles	Šroubové spoje mezi C-profily
Distance between screws per side over Whole pillar length for constant shear force	Vzdálenost mezi šrouby na jedné straně po celé délce sloupu na konstantní smykovou sílu

Anglické znění:	*překlad:*
Linear	Lineární
Constant	Konstantní

Část 3: Průřezové charakteristiky použitých profilů (strany 153-166)

Anglické znění:	*překlad:*
Properties of materials	Vlastnosti materiálů
Continuous hot dip zinc coated carbon for cold forming	Kontinuálně za horka pozinkovaná ocel pro tvarování za studena
Thickness	Tloušťka
Grade	Třída
Zink coat	Pozinkování
Yeild strength	Mez kluzu
Ultimate strength	Pevnost
Trapezoidal profiled sheets	Trapézové plechy
Section properties according to	Průřezové charakteristiky dle
Bending moment resistance y-y	Momentová únosnost ve směru y-y
Bending moment resistance z-z	Momentová únosnost ve směru z-z
Shear force resistance perpendicular to y-axis web only	Smyková únosnost kolmo na osu y pouze stojina
Shear force resistance perpendicular to z-axis (both flanges)	Smyková únosnost kolmo na osu z (obě pásnice)
Reaction force resistance web	Únosnost stojiny na reakci
Support	Podpora
Reaction force resistance restrained web	Únosnost držené stojiny na reakci
Axial tension resistance	Tahová únosnost
Axial compression resistance	Tlaková únosnost
Flexural buckling resistance	Únosnost v rovinném vzpěru

Poznámky:

Jednotlivé výrazy mohou (jsou) použity napříč všemi částmi statického posudku – nejsou vázány pouze pro tu část, ve které jsou uvedeny v překladu.