Univerza
v Ljubljani

Fakulteta za
gradbeništvo in
geodezijo

Jamova 2
1000 Ljubljana, Slovenija
telefon (01) 47 68 500
faks (01) 42 50 681
fgg@fgg.uni-lj.si

VISOKOŠOLSKI ŠTUDIJSKI PROGRAM GRADBENIŠTVO SMER OPERATIVNO GRADBENIŠTVO

Kandidat:

GREGOR NUČIČ

PROJEKT DVOETAŽNEGA POSLOVNO PROIZVODNEGA OBJEKTA

Diplomska naloga št.: 565/SOG

STRUCTURAL DESIGN OF A TWO STOREY OFFICE BUILDING AND PRODUCTION FACILITY

Graduation thesis No.: 565/SOG

Mentor:
viš. pred. dr. Leon Hladnik

Ljubljana, 12. 09. 2016

STRAN ZA POPRAVKE

Stran z napako	Vrstica z napako	Namesto	Naj bo

Spodaj podpisani študent **Gregor Nučič**, vpisna številka **26012781**, avtor pisnega zaključnega dela študija z naslovom: **Projekt dvoetažnega poslovno proizvodnega objekta**

IZJAVLJAM

1. *Obkrožite eno od variant a) ali b)*
   a) da je pisno zaključno delo študija rezultat mojega samostojnega dela;

   b) da je pisno zaključno delo študija rezultat lastnega dela več kandidatov in izpolnjuje pogoje, ki jih Statut UL določa za skupna zaključna dela študija ter je v zahtevanem deležu rezultat mojega samostojnega dela;

2. da je tiskana oblika pisnega zaključnega dela študija istovetna elektronski obliki pisnega zaključnega dela študija;

3. da sem pridobil vsa potrebna dovoljenja za uporabo podatkov in avtorskih del v pisnem zaključnem delu študija in jih v pisnem zaključnem delu študija jasno označil;

4. da sem pri pripravi pisnega zaključnega dela študija ravnal v skladu z etičnimi načeli in, kjer je to potrebno, za raziskavo pridobil soglasje etične komisije;

5. soglašam, da se elektronska oblika pisnega zaključnega dela študija uporabi za preverjanje podobnosti vsebine z drugimi deli s programsko opremo za preverjanje podobnosti vsebine, ki je povezana s študijskim informacijskim sistemom članice;

6. da na UL neodplačno, neizključno, prostorsko in časovno neomejeno prenašam pravico shranitve avtorskega dela v elektronski obliki, pravico reproduciranja ter pravico dajanja pisnega zaključnega dela študija na voljo javnosti na svetovnem spletu preko Repozitorija UL;

7. da dovoljujem objavo svojih osebnih podatkov, ki so navedeni v pisnem zaključnem delu študija in tej izjavi, skupaj z objavo pisnega zaključnega dela študija.

V/Na: ________________________

Datum: ________________________

Podpis študenta:

____

BIBLIOGRAFSKO - DOKUMENTACIJSKA STRAN IN IZVLEČEK

UDK: **624.04:624.07(497.4)(043.2)**
Avtor: **Gregor Nučič**
Mentor: **viš. pred. dr. Leon Hladnik**
Naslov: **Projekt dvoetažnega poslovno proizvodnega objekta**
Tip dokumenta: **Dipl. nal. - VSŠ**
Obseg in oprema: **135 str., 29 pregl., 113 sl., 6 pril.**
Ključne besede: **jeklena konstrukcija, statični izračun, potresno projektno stanje, Evrokod 3, Evrokod 8**

Izvleček

V diplomski nalogi je prikazan postopek statične analize in dimenzioniranje nosilnih elementov jeklene konstrukcije. Stavba je pravokotne tlorisne oblike, dimenzij 12,10 m in 32,20 m. Višina objekta je 9,50 m. Objekt ima dve etaži. V pritličju je predvidena proizvodnja in skladiščenje, v nadstropju so pisarne. Projekt je narejen v skladu z veljavnimi predpisi in standardi Evrokod ter pripadajočimi slovenskimi nacionalnimi dodatki. Dimenzioniranje na potresno projektno stanje je prikazano na dva načina in sicer, konstrukcije z majhnim sipanjem energije ter konstrukcije s sposobnostjo sipanja energije. Dimenzionirani so tudi štirje značilni spoji obravnavane jeklene konstrukcije. Fasada in streha je iz ognjeodpornih panelov. Dimenzioniranje temeljev ni vključeno v diplomsko nalogo.

BIBLIOGRAPHIC - DOCUMENTALISTIC INFORMATION AND ABSTRACT

UDC: 624.04:624.07(497.4)(043.2)
Author: Gregor Nučič
Supervisor: Sen. Lect. Leon Hladnik, Ph.D.
Title: Structural design of a two storey office building and production facility
Document type: Graduation Thesis – Higher professional studies
Scope and tools: 135 p., 29 tab., 113 fig., 6 ann.
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Abstract

The Graduation Thesis shows the process of structural analysis and design of key elements of the steel structure. The building has a rectangular floor shape, dimensions of 12.10 m and 32.20 m. Building height is 9.50 m. Construction is a two storey facility. On the ground floor is planed production and storage unit, on the first floor there would be offices. The project is designed in accordance with the valid European standards Eurocodes and related Slovenian national annex. Design of structure for earthquake resistance is shown in two ways, constructions with low energy dissipation and constructions with ability of energy dissipation. In Thesis is also shown design of four typical connections. The facade and roof is made of fire resistant panels. Dimensioning of the foundations is not included in the Thesis.

ZAHVALA

Za pomoč, vodenje in razlage pri nastajanju diplomske naloge se zahvaljujem mentorju viš. pred. dr. Leonu Hladnik.

KAZALO VSEBINE

KAZALO PREGLEDNIC

KAZALO SLIK

SEZNAM PRILOG

1 UVOD

V diplomski nalogi bo prikazana izdelava projekta jeklene oziroma sovprežne konstrukcije v obsegu Projekta za gradbeno dovoljenje (PGD).

Uporaba jeklenih konstrukcij se pri nas v zadnjih letih ves čas povečuje. Te konstrukcije se največ uporabljajo pri gradnji industrijskih objektov in nakupovalnih središč, narašča pa tudi uporaba pri gradnji poslovnih, zabaviščnih, športnih in ekoloških objektov. Jeklene konstrukcije nam omogočajo premoščanje velikih razponov in prostore brez ovir, kot so stebri in stene. Ostale prednosti jeklenih konstrukcij so še: hitra montaža (strošek gradbišča), dobro razmerje med nosilnostjo in težo konstrukcije, enostavno vzdrževanje, dolga življenska doba, enostavno kombiniranje z drugimi materiali in enostavna kontrola izdelave in montaže. Jeklo pa ima tudi nekaj pomanjkljivosti, kot so: zahtevni detajli, potrebna je velika natančnost pri projektiranju, izdelavi in montaži, akustika ter toplotna stabilnost.

Namen diplomske naloge je statična analiza konstrukcije in dimenzioniranje nosilnih elementov objekta. Diplomska naloga bo narejena v skladu z veljavnimi evropskimi standardi za gradbene konstrukcije in pripadajočimi slovenskimi nacionalnimi dodatki. Dimenzioniranje na potresno projektno stanje bo prikazano na dva načina in sicer, konstrukcije z majhnim sipanjem energije ter konstrukcije s sposobnostjo sipanja energije. Prikazan bo tudi izračun značilnih stikov konstrukcijskih elementov. Statična analiza in dimenzioniranje bo opravljeno s pomočjo komercialnega računalniškega programa SCIA Engineer 14, pozicijski načrti bodo zrisani v Tekla Structures.

Lokacija obravnavanega objekta se nahaja v Grosuplju. Jeklena stavba je pravokotne tlorisne oblike zunanjih dimenzij 12,10 m v prečni smeri in 32,20 m v vzdolžni smeri. Višina stavbe je 9,50 m. Objekt ima dve etaži. V pritličju je predvidena proizvodnja in skladiščenje, nadstropje pa je namenjeno pisarnam. Temeljenje objekta ni del te diplomske naloge.

2 TEHNIČNO POROČILO

2.1 ZASNOVA

Grafična zasnova objekta je prikazana na sliki 1. Jeklena konstrukcija objekta ima dve etaži zunanjih tlorisnih dimenzij 12,10 m v prečni smeri in 32,20 m v vzdolžni smeri. Višina stavbe je 9,50 m. Naklon strehe je 6°. Svetla višina pritličja (proizvodnja in skladiščenje) je 5,00 m, nadstropja (pisarne) pa 3,00 m. Nosilna konstrukcijaje je sestavljena iz momentnih okvirov v prečni smeri, medosna razdalja med njimi je 5,30 m. Okviri so med seboj povezani s členkasto vpetimi sovprežnimi nosilci. Horizontalno nosilnost v vzdolžni smeri zagotavljajo okviri s centričnimi povezji na zunanjih stenah in strehi.

2.1.1 Streha

Streha je dvokapnica z naklonom 6°. Na strešne lege so položeni strešni paneli Trimo SNV 150. Strešni paneli se pritrdijo na konstrukcijo po navodilih proizvajalca.

2.1.2 Fasada

Fasada se izvede s fasadnimi paneli TrimoRaster FTV R 150. Paneli so pritrjeni direktno na stebre objekta po navodilih proizvajalca.

2.1.3 Medetažna konstrukcija

Medetažna konstrukcija je sestavljena iz armirano betonske monolitne plošče in jeklenih nosilcev IPE. Spodnja in zgornja armatura v plošči je iz armaturnih mrež. Sovprežni nosilci so členkasto pritrjeni v primarne nosilce momentnega okvirja. Na sovprežne nosilce so privarjeni strižni čepi, ki v povezavi s ploščo tvorijo sovprežno konstrukcijo.

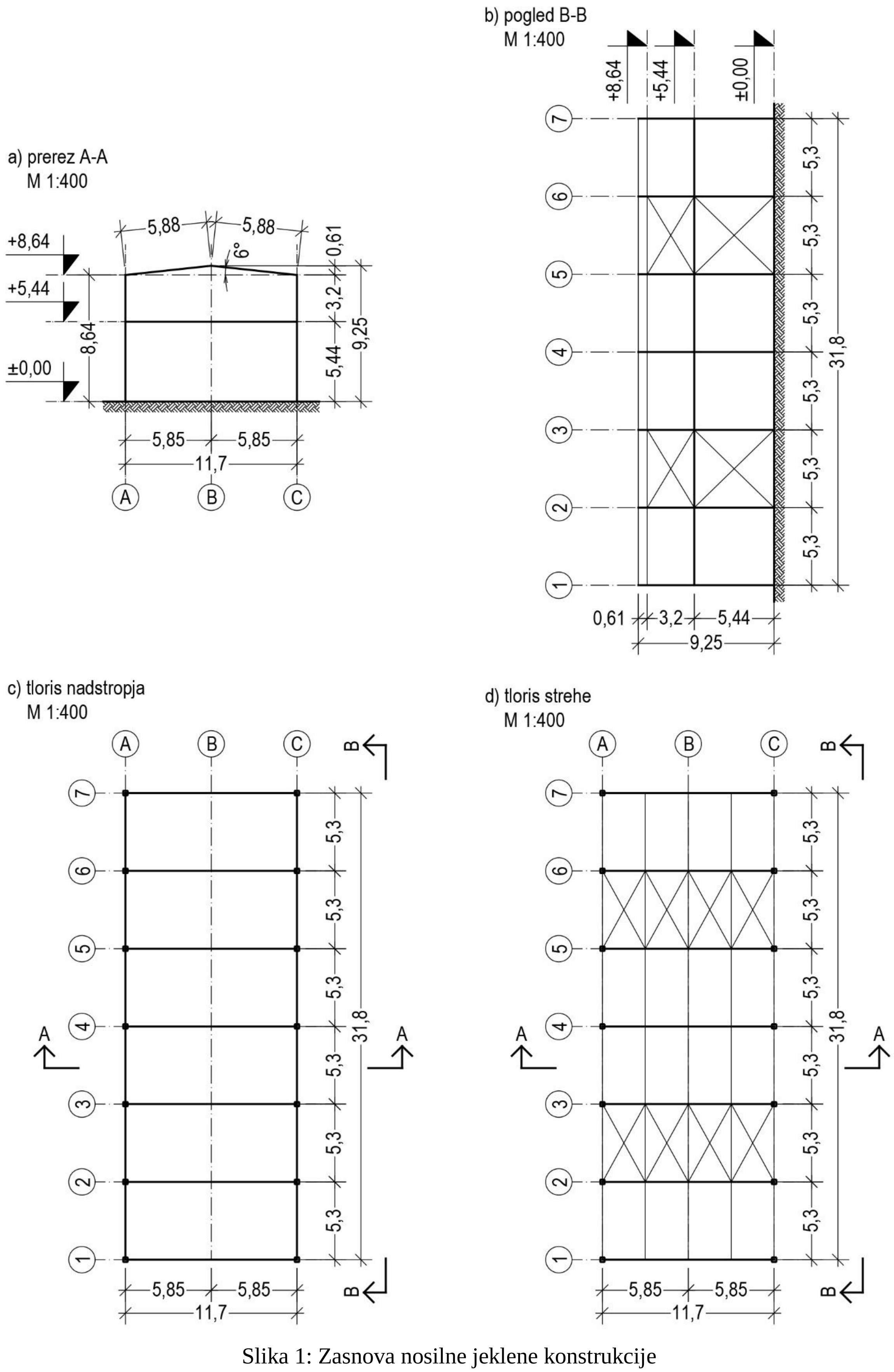
2.1.4 Temeljenje

Temeljna tla so dobro nosilna. Objekt je temeljen na točkovnih temeljih, ki so povezani s temeljnimi gredami. Stebri so členkasto pritrjeni na točkovne temelje. Temelji prevzemajo vertikalno in horizontalno obtežbo preko stebrov ter jo prenašajo v temeljna tla. Za raznos tlačne obtežbe je med temeljem in čelno pločevino stebra vgrajeno betonsko podlitje. Grede zagotavljajo stabilnost objekta in podlago za fasadne panele. Dimenzioniranje temeljev ni vključeno v diplomsko nalogo.

2.1.5 Spoji

V projektu so upoštevani značilni spoji:

- momentni spoj prečke na steber,
- členkasti spoj primarnega in sekundarnega nosilca,
- spoj natezne diagonale in stebra,
- členkasti spoj stebra na temelj.



Slika 1: Zasnova nosilne jeklene konstrukcije

2.2 MATERIAL

Material uporabljen v projektu:

- jeklo kvalitete S235 JR
  Objekt bo ogrevan. Jeklo S235 JR pri največji dovoljeni debelini elementov 40 mm lahko uporabimo pri referenčni temperaturi nad -10°C.
- jeklo kvalitete S235 J0
  Jeklo S235 J0 se uporabi za zagotovitev lokalne duktilnosti, to je za elemente, v katerih je predvideno sipanje energije (SIST EN 10025: za cone sipanja se izbere S235 J0 ali podobno mehko in dovolj žilavo konstrukcijsko jeklo).
- vijaki kvalitete 8.8
- beton kvalitete C25/30
- armaturne mreže kvalitete S500

2.3 OBTEŽBA

V izračunih so zajete obtežbe:

- lastna teža jeklene konstrukcije,
- stalna obtežba strehe,
- stalna obtežba medetažne konstrukcije,
- stalna obtežba fasade,
- spremenljiva obtežba medetažne konstrukcije,
- spremenljiva obtežba snega,
- spremenljiva obtežba vetra,
- spremenljiva potresna obtežba.

2.4 RAČUN NOTRANJIH SIL IN POMIKOV

Notranje sile in pomiki so računani po teoriji drugega reda (nelinearni izračun) z upoštevanjem začetne globalne nepopolnosti. Statična analiza in dimenzioniranje je opravljena s pomočjo komercialnega računalniškega programa SCIA Engineer 14.

2.5 POTRESNOODPORNA ANALIZA

Potresnoodporna analiza konstrukcije je narejena po metodi vodoravnih sil (linearno elastična analiza »Base Shear«). Potres se obravnava za vsako od obeh glavnih smeri posebej (ravninski model). Predstavljena bosta dva načina projektiranja jeklenih konstrukcij, konstrukcije z majhnim sipanjem energije (nizka stopnja duktilnosti) in konstrukcije s sposobnostjo sipanja energije (srednja stopnja duktilnosti).

2.6 PREDPISI IN STANDARDI

Projekt je narejen v skladu z veljavnimi evropskimi standardi za gradbene konstrukcije in pripadajočimi slovenskimi nacionalnimi dodatki.

3 VPLIVI NA KONSTRUKCIJO

3.1 LASTNA IN STALNA OBTEŽBA KONSTRUKCIJE

3.1.1 Lastna teža jeklene konstrukcije

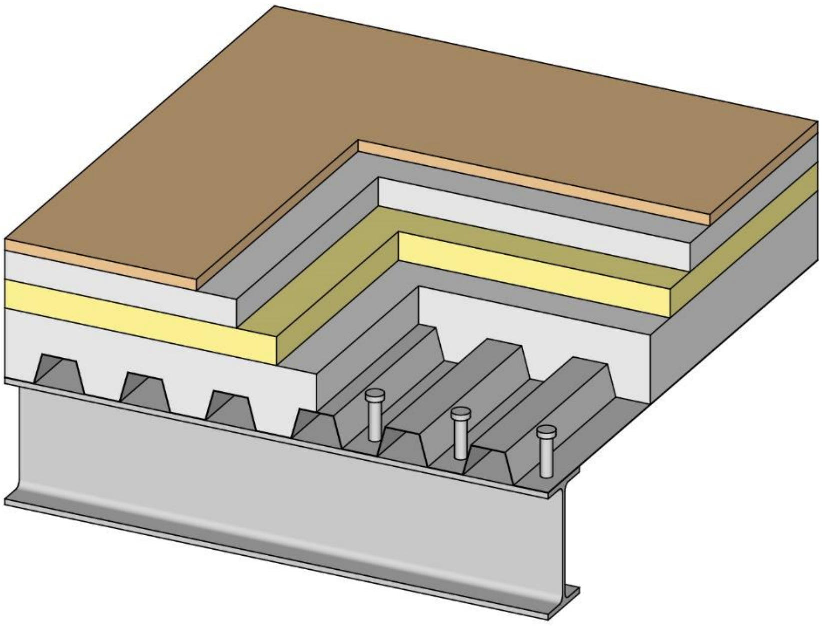
- primarni jekleni nosilci (lastna teža je zajeta v izračunu z računalniškim programom)
- sekundarni jekleni nosilci (lastna teža je zajeta v izračunu z računalniškim programom)
- strešne jeklene lege (lastna teža je zajeta v izračunu z računalniškim programom)

3.1.2 Stalna obtežba strehe

- strešni paneli TRIMO SNV 150 .... 0,30 kN/m^2
- spuščen strop .... 0,20 kN/m^2
- lahke inštalacije .... 0,10 kN/m^2

g_s = 0,60 kN/m^2

3.1.3 Stalna obtežba medetažne konstrukcije



Slika 2: Medetažna konstrukcija po plasteh

- zaključni sloj - parket, d = 2 cm .... 0,14 kN/m^2
- cementni estrih, d = 5 cm .... 1,20 kN/m^2
- toplotna izolacija - ekspandiran polistiren, d = 5 cm .... 0,03 kN/m^2
- AB plošča + pločevina, d = 12 cm .... 2,50 kN/m^2
- inštalacije .... 0,20 kN/m^2

g_m = 4,57 kN/m^2

3.1.4 Stalna obtežba fasade

- fasadni paneli TRIMO FTV 150 .... 0,30 kN/m^2

g_f = 0,30 kN/m^2

3.2 SPREMENLJIVA OBTEŽBA

3.2.1 Koristna obtežba nadstropja

- pisarne .... 3,00 kN/m^2
- predelne stene .... 1,30 kN/m^2

q_m = 4,30 kN/m^2

3.2.2 Obtežba snega na strehi

Za Grosuplje, ki je na nadmorski višini 350 m se izbere cona A2 (slika 1 v nacionalnem dodatku SIST EN 1991-1-3:2004/oA101:2008).

Obtežba snega *s* na strehi

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k = 0{,}8 \cdot 1{,}0 \cdot 1{,}0 \cdot 1{,}59\,\mathrm{kN/m^2} = 1{,}27\,\mathrm{kN/m^2}$$

$\mu_i = 0{,}8$ oblikovni koeficient obtežbe snega (naklon strehe je 6°)
$C_e = 1{,}0$ koeficient izpostavljenosti
$C_t = 1{,}0$ toplotni koeficient
Karakteristična obtežba snega na tleh s_k (cona A2 na nadmorski višini *A* = 350 m)

$$s_k = 1{,}293\left[1 + \left(\frac{A}{728}\right)^2\right] = 1{,}293\left[1 + \left(\frac{350}{728}\right)^2\right] = 1{,}59\ \mathrm{kN/m^2}$$

3.2.3 Vplivi vetra

Za Grosuplje, ki je na nadmorski višini 350 m se izbere cona 1 (slika 1 v nacionalnem dodatku SIST EN 1991-1-4:2005/oA101:2007).

Osnovna hitrost vetra v_b

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} = 1{,}0 \cdot 1{,}0 \cdot 20\,\mathrm{m/s} = 20\,\mathrm{m/s}$$

$v_{b,0} = 20\ \mathrm{m/s}$ temeljna vrednost osnovne hitrosti vetra
$c_{dir} = 1{,}0$ smerni faktor
$c_{season} = 1{,}0$ faktor letnega časa

Tlak pri največji hitrosti ob sunkih vetra q_p(z) na višini *z*

$$q_p(\mathrm{z}) = [1 + 7 \cdot l_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b = 2{,}3 \cdot 0{,}25\,\mathrm{kN/m^2} = 0{,}58\,\mathrm{kN/m^2}$$

$\rho = 1{,}25\ \mathrm{kg/m^3}$ gostota zraka
Osnovni tlak vetra q_b

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 = \frac{1}{2} \cdot 1{,}25\,\mathrm{kg/m^3} \cdot (20\,\mathrm{m/s})^2 = 250\ \mathrm{N/m^2} = 0{,}25\ \mathrm{kN/m^2}$$

Faktor izpostavljenosti c_e(z)

$c_e(\mathrm{z}) = c_e(9{,}25\ \mathrm{m}) \cong 2{,}3$ (slika 4.2 v standardu SIST EN 1991-1-4:2005)

z = 9,25 m višina objekta

kategorija terena je II (slika 4.1 v standardu SIST EN 1991-1-4:2005)

3.2.3.1 Tlak vetra na notranje ploskve objekta

Tlak vetra na notranje ploskve w_i

$$w_i = q_p(z_i) \cdot c_{pi}$$

$q_p(z_i)$ največji tlak pri sunkih vetra.
z_i referenčna višina za notranji tlak (enaka referenčni višini z_e za zunanje tlake).
c_{pi} koeficient notranjega tlaka.

V preglednici 1 sta navedeni vrednosti notranjega zračnega tlaka w_i z upoštevanjem priporočenih vrednosti koeficientov notranjega tlaka c_{pi}. Vrednosti veljata za celoto (vsa področja od A do J).

Preglednica 1: Koeficienti in obtežbe notranjega tlaka

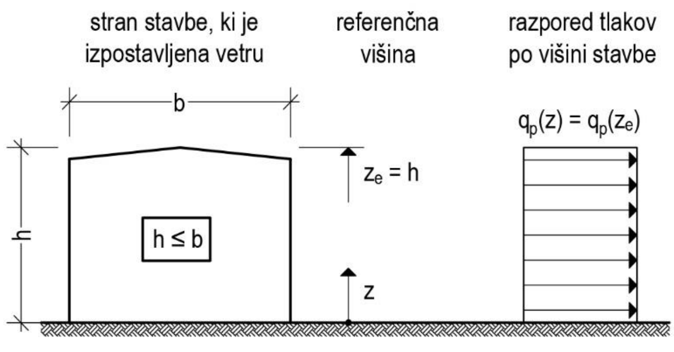
Področje	**$q_p(z)$ [kN/m^2]**	**c_{pi}**	**w_i [kN/m^2]**	**Opomba**
A, ..., J	0,58	+0,20	+0,12	notranji tlak
A, ..., J	0,58	–0,30	–0,17	notranji srk

3.2.3.2 Tlak vetra na zunanje ploskve objekta

Tlak vetra na zunanje ploskve w_e

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

$q_p(z_e)$ največji tlak pri sunkih vetra
z_e referenčna višina za zunanji tlak je prikazana na sliki 3 ($z_e = h$ če je $h \leq b$, q_p je konstanten po višini)
c_{pe} koeficient zunanjega tlaka



Slika 3: Referenčna višina in razporeditev tlakov po višini stavbe

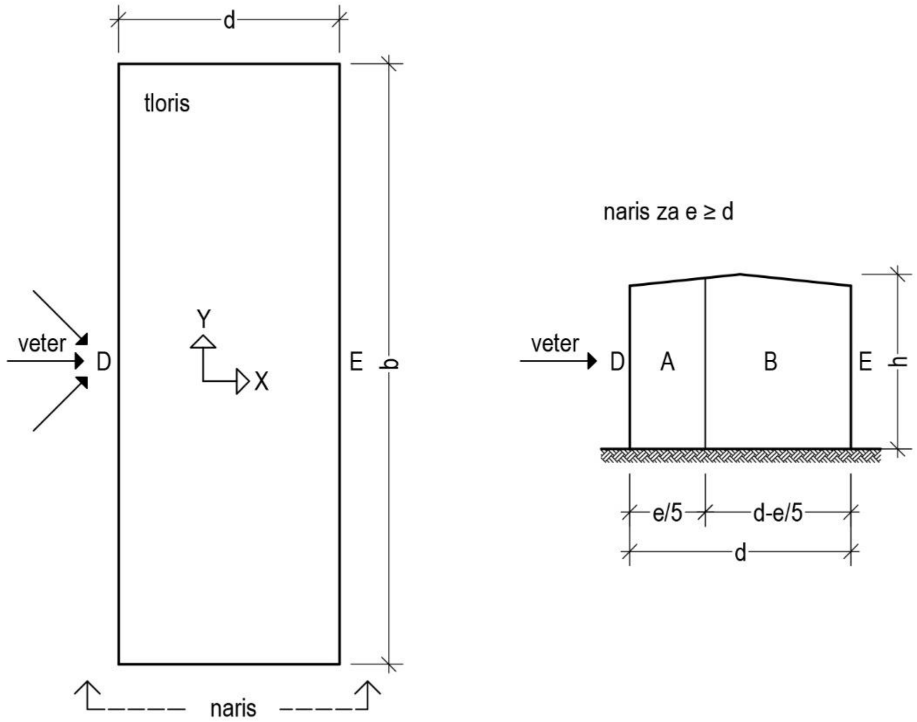
Koeficienti zunanjega tlaka na navpične stene

Na sliki 4 je prikazana razdelitev sten na področja za obravnavani objekt pri delovanju vetra v smeri X in razmerju $e \geq d$.

Dolžina e

$$e = \min\{b\ ali\ 2h\} = \min\{31{,}80\ \text{m}\ ali\ 18{,}50\ \text{m}\} = 18{,}50\ \text{m}$$

$b = 31{,}80$ m dolžina stranice objekta pravokotno smeri delovanja vetra
$h = 9{,}25$ m višina objekta
$d = 11{,}70$ m dolžina stranice objekta vzporebno smeri delovanja vetra



Slika 4: Razdelitev sten na področja pri delovanju vetra v smeri X in razmerju $e \geq d$

Priporočene vrednosti koeficientov so v preglednici 2 (h/d = 0,79 in preglednica 7.1 v standardu SIST EN 1991-1-4:2005).

Preglednica 2: Koeficienti in obtežbe zunanjega tlaka na navpične stene pri delovanju vetra v smeri X

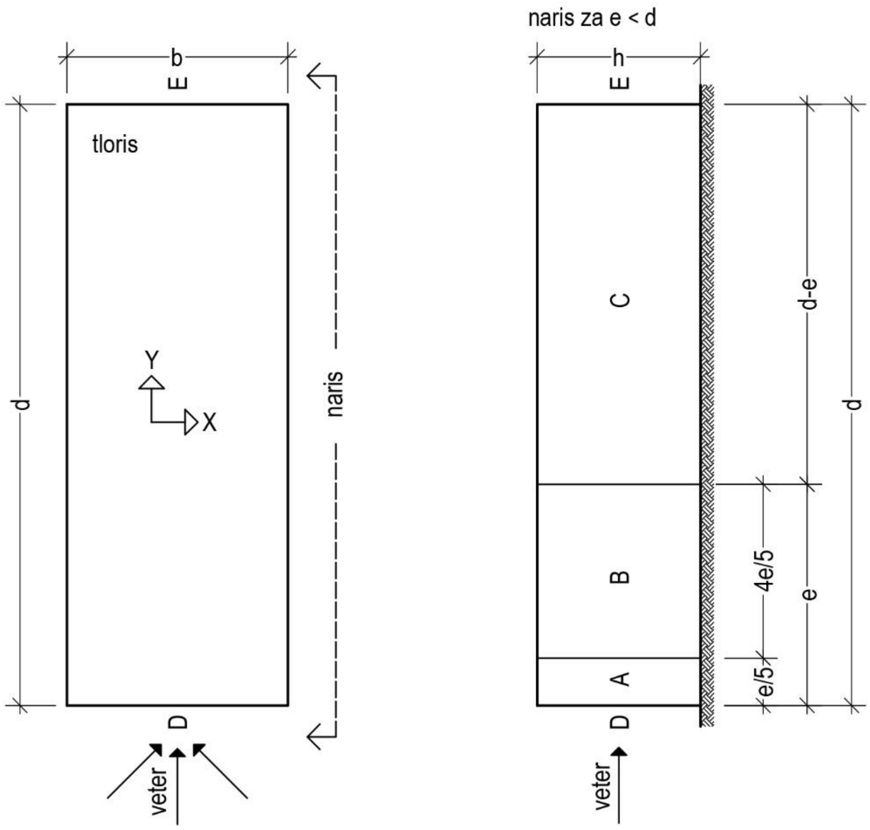
Področje	$q_p(z)$ **[kN/m²]**	c_{pe}	w_e **[kN/m²]**
A	0,58	–1,20	–0,70
B	0,58	–0,80	–0,46
D	0,58	+0,77	+0,45
E	0,58	–0,44	–0,26

Na sliki 5 je prikazana razdelitev sten na področja za obravnavani objekt pri delovanju vetra v smeri Y in razmerju $e < d$.

Dolžina e

$$e = \min\{b\ ali\ 2h\} = \min\{11{,}70\ \text{m}\ ali\ 18{,}50\ \text{m}\} = 11{,}70\ \text{m}$$

$b = 11{,}70$ m dolžina stranice objekta pravokotno smeri delovanja vetra
$h = 9{,}25$ m višina objekta
$d = 31{,}80$ m dolžina stranice objekta vzporebno smeri delovanja vetra



Slika 5: Razdelitev sten na področja pri delovanju vetra v smeri Y in razmerju $e < d$

Priporočene vrednosti koeficientov so v preglednici 3 (h/d = 0,29 in preglednica 7.1 v standardu SIST EN 1991-1-4:2005).

Preglednica 3: Koeficienti in obtežbe zunanjega tlaka na navpične stene pri delovanju vetra v smeri Y

Področje	**$q_p(z)$ [kN/m^2]**	**c_{pe}**	**w_e [kN/m^2]**
A	0,58	–1,20	–0,70
B	0,58	–0,80	–0,46
C	0,58	–0,50	–0,29
D	0,58	+0,71	+0,41
E	0,58	–0,31	–0,18

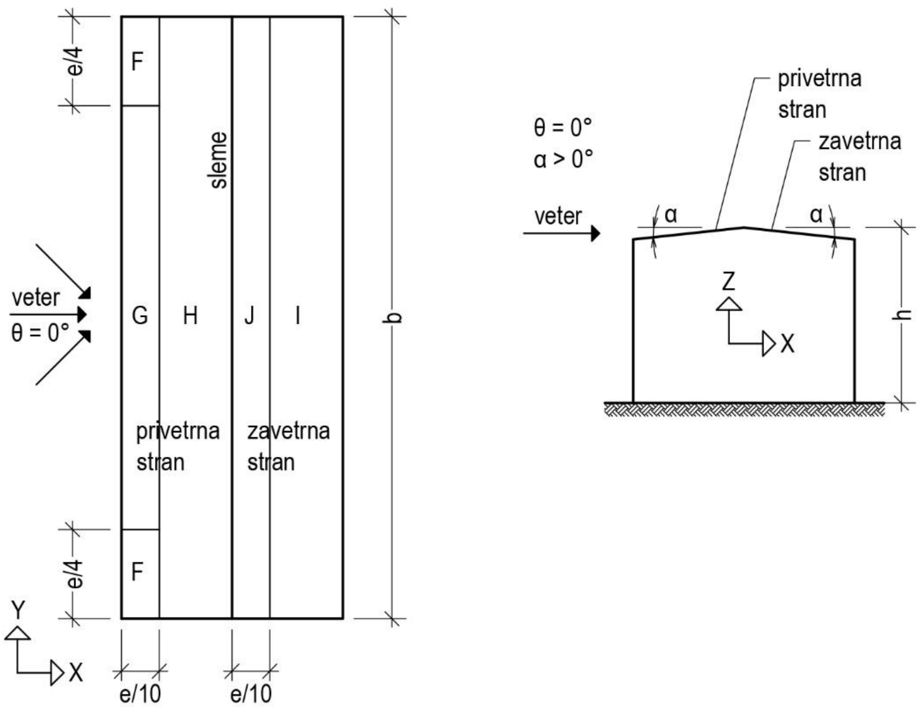
Koeficienti zunanjega tlaka na dvokapno poševno streho

Na sliki 6 je prikazana razdelitev dvokapnice na področja za obravnavani objekt pri delovanju vetra v smeri X (θ = 0°).

Dolžina e

$$e = \min\{b\ ali\ 2h\} = \min\{31{,}80\text{ m } ali\ 18{,}50\text{ m}\} = 18{,}50\text{ m}$$

$b = 31{,}80$ m dolžina stranice objekta pravokotno smeri delovanja vetra
$h = 9{,}25$ m višina objekta



Slika 6: Razdelitev dvokapnice na področja pri delovanju vetra v smeri X (θ = 0°)

Priporočene vrednosti koeficientov so v preglednici 4 (naklon strehe α = 6° in preglednica 7.4a v standardu SIST EN 1991-1-4:2005).

Preglednica 4: Koeficienti in obtežbe zunanjega tlaka za dvokapnice pri delovanju vetra v smeri X

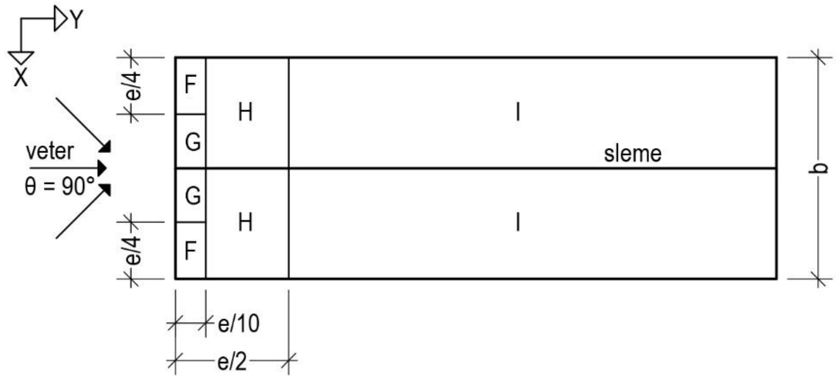
Področje	$q_p(z)$ **[kN/m²]**	c_{pe}	w_e **[kN/m²]**
F	0,58	–1,62 +0,02	–0,94 +0,01
G	0,58	–1,16 +0,02	–0,67 +0,01
H	0,58	–0,57 +0,02	–0,33 +0,01
I	0,58	–0,58	–0,34
J	0,58	–0,64 +0,18	–0,37 +0,10

Na sliki 7 je prikazana razdelitev dvokapnice na področja za obravnavani objekt pri delovanju vetra v smeri Y (θ = 90°).

Dolžina e

$$e = \min\{b\ ali\ 2h\} = \min\{11{,}70\text{ m}\ ali\ 18{,}50\text{ m}\} = 11{,}70\text{ m}$$

$b = 11{,}70$ m dolžina stranice objekta pravokotno smeri delovanja vetra
$h = 9{,}25$ m višina objekta



Slika 7: Razdelitev dvokapnice na področja pri delovanju vetra v smeri Y (θ = 90°)

Priporočene vrednosti koeficientov so v preglednici 5 (naklon streh α = 6° in preglednica 7.4b v standardu SIST EN 1991-1-4:2005).

Preglednica 5: Koeficienti in obtežbe zunanjega tlaka za dvokapnice pri delovanju vetra v smeri Y

Področje	$q_p(z)$ [kN/m^2]	c_{pe}	w_e [kN/m^2]
F	0,58	–1,57	–0,91
G	0,58	–1,30	–0,75
H	0,58	–0,69	–0,40
I	0,58	–0,59	–0,34

3.2.3.3 Neto tlak na steno ali streho

Neto tlak (razlika med tlakoma na nasprotnih ploskvah) w

$$w = q_p(z_e) \cdot c_{pe} - q_p(z_e) \cdot c_{pi} = w_e - w_i$$

V preglednicah 6, 7, 8 in 9 so prikazani neto tlaki na navpično steno ali dvokapno streho pri delovanju vetra v smeri X ali Y za vsa področja.

Preglednica 6: Neto tlaki w (razlika med zunanjim in notranjim tlakom) na steno in streho pri delovanju vetra v smeri X

Področje	$q_p(z)$ [kN/m^2]	c_{pe}	c_{pi}	w_e [kN/m^2]	w_i [kN/m^2]	w [kN/m^2]
A	0,58	–1,20	+0,20	–0,70	+0,12	–0,82
B	0,58	–0,80	+0,20	–0,46	+0,12	–0,58
D	0,58	+0,77	+0,20	+0,45	+0,12	+0,33
E	0,58	–0,44	+0,20	–0,26	+0,12	–0,38
F	0,58	–1,62 +0,02	+0,20 +0,20	–0,94 +0,01	+0,12 +0,12	–1,06 –0,11
G	0,58	–1,16 +0,02	+0,20 +0,20	–0,67 +0,01	+0,12 +0,12	–0,79 –0,11
H	0,58	–0,57 +0,02	+0,20 +0,20	–0,33 +0,01	+0,12 +0,12	–0,45 –0,11
I	0,58	–0,58	+0,20	–0,34	+0,12	–0,46
J	0,58	–0,64 +0,18	+0,20 +0,20	–0,37 +0,10	+0,12 +0,12	–0,49 –0,02

Preglednica 7: Neto tlaki *w* (razlika med zunanjim tlakom in notranjim srkom) na steno in streho pri delovanju vetra v smeri X

Področje	$q_p(z)$ [kN/m²]	c_{pe}	c_{pi}	w_e [kN/m²]	w_i [kN/m²]	w [kN/m²]
A	0,58	–1,20	–0,30	–0,70	–0,17	–0,53
B	0,58	–0,80	–0,30	–0,46	–0,17	–0,29
D	0,58	+0,77	–0,30	+0,45	–0,17	+0,62
E	0,58	–0,44	–0,30	–0,26	–0,17	–0,09
F	0,58	–1,62	–0,30	–0,94	–0,17	–0,77
		+0,02	–0,30	+0,01	–0,17	+0,18
G	0,58	–1,16	–0,30	–0,67	–0,17	–0,50
		+0,02	–0,30	+0,01	–0,17	+0,18
H	0,58	–0,57	–0,30	–0,33	–0,17	–0,16
		+0,02	–0,30	+0,01	–0,17	+0,18
I	0,58	–0,58	–0,30	–0,34	–0,17	–0,17
J	0,58	–0,64	–0,30	–0,37	–0,17	–0,20
		+0,18	–0,30	+0,10	–0,17	+0,27

Preglednica 8: Neto tlaki *w* (razlika med zunanjim in notranjim tlakom) na steno in streho pri delovanju vetra v smeri Y

Področje	$q_p(z)$ [kN/m²]	c_{pe}	c_{pi}	w_e [kN/m²]	w_i [kN/m²]	w [kN/m²]
A	0,58	–1,20	+0,20	–0,70	+0,12	–0,81
B	0,58	–0,80	+0,20	–0,46	+0,12	–0,58
C	0,58	–0,50	+0,20	–0,29	+0,12	–0,41
D	0,58	+0,71	+0,20	+0,41	+0,12	+0,30
E	0,58	–0,31	+0,20	–0,18	+0,12	–0,30
F	0,58	–1,57	+0,20	–0,91	+0,12	–1,03
G	0,58	–1,30	+0,20	–0,75	+0,12	–0,87
H	0,58	–0,69	+0,20	–0,40	+0,12	–0,52
I	0,58	–0,59	+0,20	–0,34	+0,12	–0,46

Preglednica 9: Neto tlaki *w* (razlika med zunanjim tlakom in notranjim srkom) na steno in streho pri delovanju vetra v smeri Y

Področje	$q_p(z)$ [kN/m²]	c_{pe}	c_{pi}	w_e [kN/m²]	w_i [kN/m²]	w [kN/m²]
A	0,58	–1,20	–0,30	–0,70	–0,17	–0,52
B	0,58	–0,80	–0,30	–0,46	–0,17	–0,29
C	0,58	–0,50	–0,30	–0,29	–0,17	–0,12
D	0,58	+0,71	–0,30	+0,41	–0,17	+0,59
E	0,58	–0,31	–0,30	–0,18	–0,17	–0,01
F	0,58	–1,57	–0,30	–0,91	–0,17	–0,74
G	0,58	–1,30	–0,30	–0,75	–0,17	–0,58
H	0,58	–0,69	–0,30	–0,40	–0,17	–0,23
I	0,58	–0,59	–0,30	–0,34	–0,17	–0,17

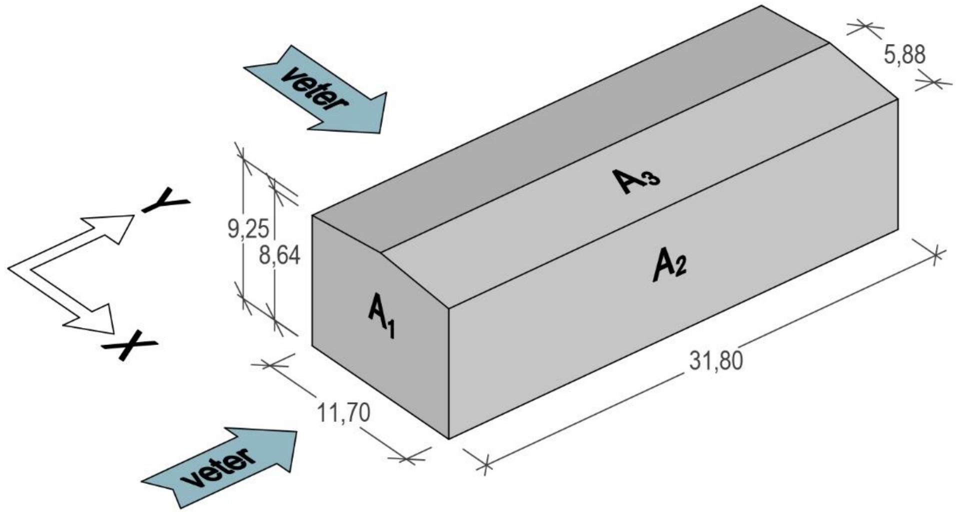
3.2.3.4 Sila trenja vetra

Učinek trenja vetra na ploskve se lahko zanemari, če je izpolnjena neenačba

$$A_{VZP} \leq 4A_{PRAV}$$

A_{VZP} površina vseh ploskev vzporednih (ali pod majhnim kotom) z vetrom.
A_{PRAV} površina vseh ploskev pravokotnih na veter (privetrnih in zavetrnih).

Kontrolo pogoja izvedemo ločeno za delovanje vetra v smeri X in Y. Na sliki 8 so označene smeri vetra in posamezne ploskve.



Slika 8: Smeri delovanja vetra in posamezne ploskve

Površine ploskev:

$$A_1 = 11{,}70\ \text{m} \cdot \frac{8{,}64\ \text{m} + 9{,}25\ \text{m}}{2} = 104{,}66\ \text{m}^2$$

$$A_2 = 8{,}64\ \text{m} \cdot 31{,}80\ \text{m} = 274{,}75\ \text{m}^2$$

$$A_3 = 5{,}88\ \text{m} \cdot 31{,}80\ \text{m} = 186{,}98\ \text{m}^2$$

Kontrola pogoja pri delovanju vetra v smeri X:

$$A_{VZP} = 2A_1 + 2A_3 = 2 \cdot 104{,}66\ \text{m}^2 + 2 \cdot 186{,}98\ \text{m}^2 = 583{,}28\ \text{m}^2$$

$$A_{PRAV} = 2A_2 = 2 \cdot 274{,}78\ \text{m}^2 = 549{,}50\ \text{m}^2$$

$$A_{VZP}(= 583{,}28\ \text{m}^2) \leq 4A_{PRAV}(= 2198{,}00\ \text{m}^2)\ \checkmark$$

Učinek trenja vetra na ploskve pri delovanju vetra v smeri X se zanemari.

Kontrola pogoja pri delovanju vetra v smeri Y:

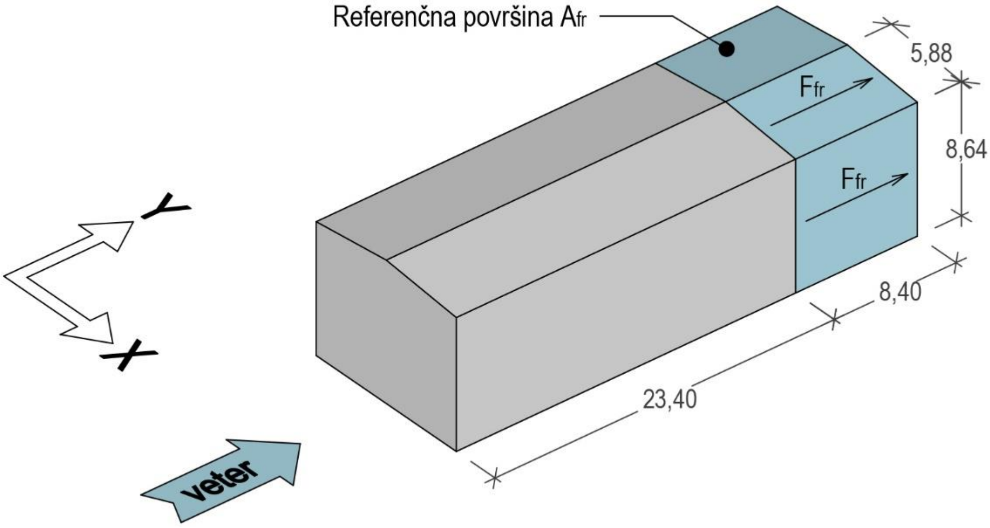
$$A_{VZP} = 2A_2 + 2A_3 = 2 \cdot 274{,}75\ \text{m}^2 + 2 \cdot 186{,}98\ \text{m}^2 = 923{,}46\ \text{m}^2$$

$$A_{PRAV} = 2A_1 = 2 \cdot 104{,}66\ \text{m}^2 = 209{,}32\ \text{m}^2$$

$$A_{VZP}(= 923{,}46\ \text{m}^2) \leq 4A_{PRAV}(= 837{,}28\ \text{m}^2)\ \times$$

Učinek trenja vetra na ploskve pri delovanju vetra v smeri Y se ne zanemari.

Sile trenja F_{fr} delujejo v smeri vetra, vzporedni zunanjim ploskvam. Upoštevajo se samo na delu referenčne površine A_{fr}, ki je prikazana na sliki 9.



Slika 9: Referenčna površina A_{fr} pri delovanju vetra v smeri Y

Trenjske sile zaradi trenja vetra F_{fr}

$$F_{fr} = c_{fr} \cdot q_p(z_e) \cdot A_{fr}$$

c_{fr} koeficient trenja (preglednica 7.10, SIST EN 1991-1-4:2005)
$c_{fr,stena}$ = 0,01 (gladka ploskev, npr. jeklo, gladek beton)
$c_{fr,streha}$ = 0,04 (zelo hrapava ploskev, npr. gube, rebra, pregibi)

$q_p(z_e)$ največji tlak pri sunkih vetra, referenčna višina z_e je enaka višini stavbe h = 9,25 m.

A_{fr} referenčna površina je dana na sliki 9. Sile trenja se upoštevajo na delu zunanjih ploskev, vzporednih s smerjo vetra, ki so za razdaljo, manjšo od $2b$ ali $4h$, oddaljene od privetrnega kapu ali vogala.

$$\min\{2b \text{ ali } 4h\} = \min\{23{,}40 \text{ m ali } 37{,}00 \text{ m}\} = 23{,}40 \text{ m}$$

$$A_{fr,stena} = 8{,}40 \text{ m} \cdot 8{,}64 \text{ m} = 72{,}58 \text{ m}^2$$

$$A_{fr,streha} = 8{,}40 \text{ m} \cdot 5{,}88 \text{ m} = 49{,}39 \text{ m}^2$$

Sila trenja na eno steno:

$$F_{fr,stena} = c_{fr,stena} \cdot q_p(h) \cdot A_{fr,stena} = 0{,}01 \cdot 0{,}58 \,\text{kN/m}^2 \cdot 72{,}58 \text{ m}^2 = 0{,}42 \text{ kN}$$

Sila trenja na eno strešino:

$$F_{fr,streha} = c_{fr,streha} \cdot q_p(h) \cdot A_{fr,streha} = 0{,}04 \cdot 0{,}58 \,\text{kN/m}^2 \cdot 49{,}39 \text{ m}^2 = 1{,}01 \text{ kN}$$

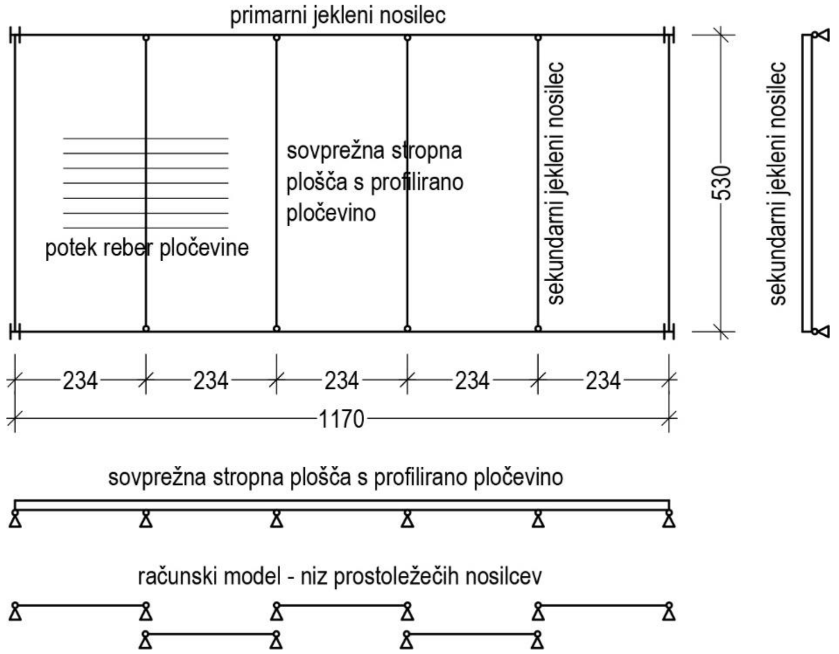
Sila trenja se upošteva pri pritrjevanju strešnih in fasadnih panelov na konstrukcijo.

4 MEDETAŽNA KONSTRUKCIJA

Medetažna ali stropna konstrukcija je sestavljena iz armiranobetonske plošče s profilirano pločevino in jeklenih nosilcev, ki skupaj tvorijo sovprežno konstrukcijo.

4.1 ZASNOVA

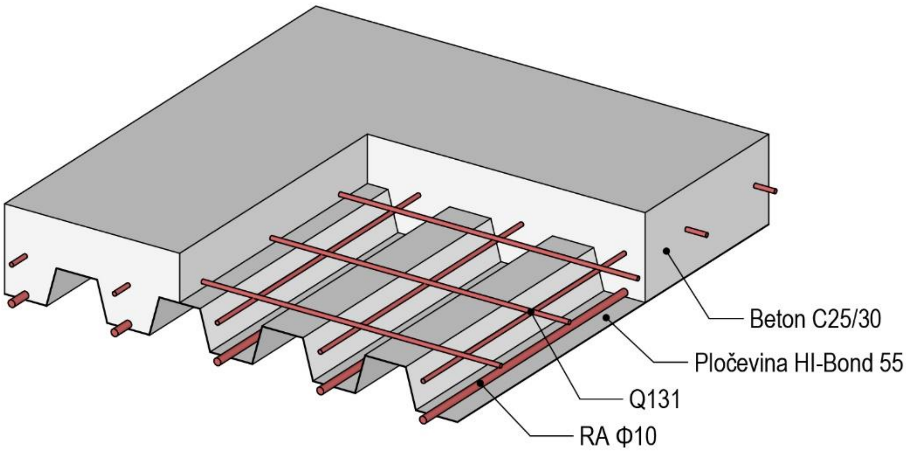
Grafična zasnova sovprežne konstrukcije je prikazana na sliki 10. Jekleni sekundarni nosilci so zasnovani kot prostoležeči, členkasto podprti na primarne jeklene nosilce. Razpon sekundarnih nosilcev je 5,30 m, medosna razdalja med njimi je 2,35 m. Valovi pločevine potekajo pravokotno na sekundarne nosilce.



Slika 10: Zasnova sovprežne medetažne konstrukcije

4.2 SOVPREŽNA PLOŠČA S PROFILIRANO PLOČEVINO

Sovprežna plošča je sestavljena iz strjenega armiranega betona in profilirane pločevine. Pločevina služi tudi kot delovna platforma, opaž za beton in kot natezna armatura. Lahka armaturna mreža se dodaja zaradi boljšega prečnega raznosa obtežbe, služi pa tudi za zagotavljanje zadostne protipožarne odpornosti in za zmanjševanje vpliva krčrnja betona na formiranje razpok. Za povečanje upogibne nosilnosti in vzdolžnega striga dodamo v vsak val ali vsak drugi val palice rebraste armature. Pri projektiranju sovprežnih stropov je potrebno upoštevati dve različni projektni stanji, montažno stanje (profilirana pločevina kot opaž) in končno stanje. Kontrola sovprežne plošče s pločevino je izvedena po Beg, Hladnik in Može, 2003. Uporabljene tabele so v prilogi E.



Slika 11: Sovprežna plošča s profilirano pločevino, betonom in armaturo

4.2.1 Podatki

Material

Beton	C25/30
Pločevina	HI-Bond 55, t = 0,8 mm, f_y = 250 Mpa
Armatura	S500
Strižni čepi	Nelson čepi Ø19, f_u = 45 kN/cm^2

Geometrija

h = 12 cm debelina plošče

d_p = 12 – 5,5/2 = 9,25 cm efektivna debelina plošče

L = 234 cm razmak med nosilci (podporami)

$b \geq$ 10 cm širina podpor

Obtežba

Lastna (pločevina + beton)

strjen beton g_l = 2,41 kN/m^2

svež beton g_l = 2,53 kN/m^2

Stalna (poglavje 3.1.3) g_s = 1,57 kN/m^2

Koristna (poglavje 3.2.1) q = 4,30 kN/m^2

4.2.2 Montažno stanje - faza betoniranja

Sovprežne plošče med gradnjo ne podpiramo, saj je pločevina sposobna prevesti obtežbo svežega betona na razponu do 250 cm.

Priloga E, Tabela 1: Dopustni razponi HI-Bond pločevine v vlogi opaža:

$L_{zp}^{max} = \min\{270 \text{ cm}, 250 \text{ cm}\} = 250 \text{ cm} > L = 234 \text{ cm}$ ✓

4.2.3 Končno stanje

4.2.3.1 Kontrola razmaka med podporami

MSN - Mejno stanje nosilnosti

Obtežba $p_{Sd,MSN} = 1{,}35 \cdot g_s + 1{,}5 \cdot q = 1{,}35 \cdot 1{,}57\,\mathrm{kN/m^2} + 1{,}5 \cdot 4{,}30\,\mathrm{kN/m^2}$

$p_{Sd,MSN} = 8{,}57\,\mathrm{kN/m^2}$

Po metodi delne strižne vezi upoštevam: metoda B6

- strižna odpornost pločevine - SN
- trenje na podpori - TP
- končno sidranje - KS
  - Nelson čep Ø19 v vsakem valu (na 15 cm),
  - čepi morajo biti privarjeni skozi pločevino,
  - *a* = 105 mm, upoštevam odmik čepa od roba pločevine, zagotavlja maksimalen vpliv končnega sidranja.
- armatura - A
  - RA Ø10 v vsakem valu (na sredini vala).

Priloga E, Tabela 8: Dopustni razponi HI-Bond sovprežnih stropov - MSN:

$p_{Sd,MSN} = 8\,\mathrm{kN/m^2} \rightarrow L^{max} = 468\ \mathrm{cm}$

$p_{Sd,MSN} = 10\,\mathrm{kN/m^2} \rightarrow L^{max} = 431\ \mathrm{cm}$

z linearno interpolacijo dobimo pri

$p_{Sd,MSN} = 8{,}57\,\mathrm{kN/m^2} \rightarrow L^{max} = 442\ \mathrm{cm}$

$L^{max} = 442\ \mathrm{cm} > L = 234\ \mathrm{cm}$ ✓

MSU - Mejno stanje uporabnosti

Obtežba $p_{Sd,MSU} = g_s + q = 1{,}57\,\mathrm{kN/m^2} + 4{,}30\,\mathrm{kN/m^2}$

$p_{Sd,MSU} = 5{,}87\,\mathrm{kN/m^2}$

Priloga E, Tabela 2: Dopustni razponi HI-Bond sovprežnih stropov - MSU:

$p_{Sd,MSU} = 5\,\mathrm{kN/m^2} \rightarrow L^{max} = 427\ \mathrm{cm}$

$p_{Sd,MSU} = 6\,\mathrm{kN/m^2} \rightarrow L^{max} = 409\ \mathrm{cm}$

z linearno interpolacijo dobimo pri

$p_{Sd,MSU} = 5{,}87\,\mathrm{kN/m^2} \rightarrow L^{max} = 425\ \mathrm{cm}$

$L^{max} = 425\ \mathrm{cm} > L = 234\ \mathrm{cm}$ ✓

4.2.3.2 Potrebna armatura

Negativna armatura nad podporami (zgornja armatura proti razpokanju betona)

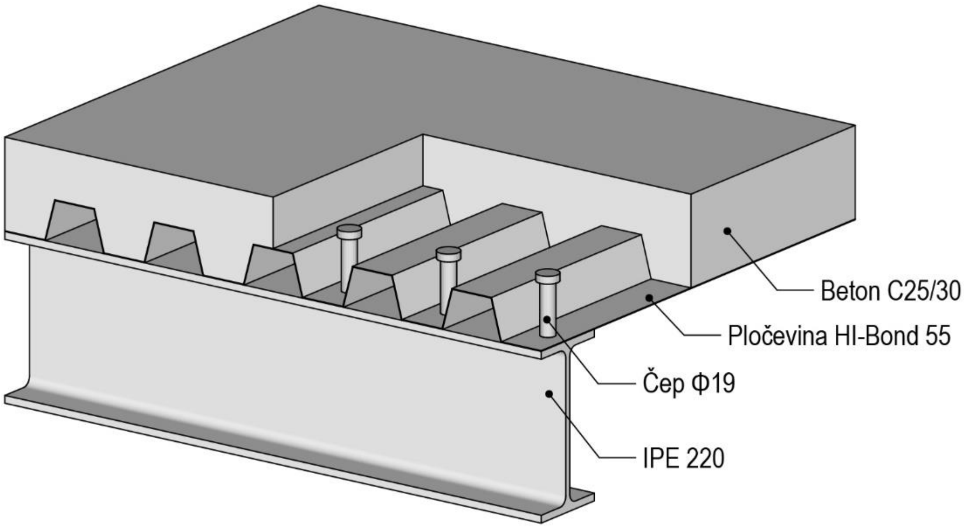
$$A_{neg.arm.}^{potr} = 0{,}004 \cdot 100\ \mathrm{cm} \cdot (12\ \mathrm{cm} - 5{,}5\ \mathrm{cm}) = 2{,}6\ \mathrm{cm^2/m}$$

Armatura za raznos obtežbe (priporočena, teoretično nepotrebna)

$$A_{raznos}^{potr} = 0{,}002 \cdot 100\ \mathrm{cm} \cdot (12\ \mathrm{cm} - 5{,}5\ \mathrm{cm}) = 1{,}3\ \mathrm{cm^2/m}$$

V obeh med seboj pravokotnih smereh je potrebno položiti enako armaturo. Izbrana mreža je Q131. V vsakem valu je palica RA Ø10 (na 15 cm).

4.3 SEKUNDARNI SOVPREŽNI JEKLENI NOSILEC



Slika 12: Sovprežni nosilec

Določitev višine jeklenega nosilca na osnovi enačbe: $h \approx b/25 = 530\,\text{cm}/25 = 21{,}2$ cm
Izberem prerez IPE 220, h_{IPE} = 22 cm.

4.3.1 Podatki

Material

Jeklo S235

Strižni čepi Nelson čepi Ø19, f_u = 45 kN/cm^2

Geometrija

h_c = 12 cm debelina plošče

$h_{c,eff}$ = 12 – 5,5/2 = 9,25 cm efektivna debelina plošče

L = 234 cm razmak med sekundarnimi nosilci

b = 530 cm dolžina sekundarnih nosilcev

Obtežba

Lastna

- IPE 220 g_{IPE} = 0,262 kN/m
- Sovprežna plošča s pločevino
  - strjen beton $g_{c,st}$ = 2,41 kN/m^2 · 2,34 m = 5,64 kN/m
  - svež beton $g_{c,sv}$ = 2,53 kN/m^2 · 2,34 m = 5,92 kN/m

Stalna (poglavje 3.1.3) g_s = 1,57 kN/m^2 · 2,34 m = 3,67 kN/m

Koristna (poglavje 3.2.1) q = 4,30 kN/m^2 · 2,34 m = 10,06 kN/m

Obtežba med betoniranjem q_m = 0,75 kN/m^2 · 2,34 m = 1,76 kN/m

- območje 3 m · 3 m (kopičenje betona) - 10% lastne teže betona
  q_{m1} = 0,10 · 2,53 kN/m^2 = 0,25 kN/m^2 oz. min. q_{m1} = 0,75 kN/m^2
- izven območja (delavci, oprema) q_{m2} = 0,75 kN/m^2

4.3.2 Montažno stanje - faza betoniranja

4.3.2.1 MSN - Mejno stanje nosilnosti

Obtežba

$$q_{Ed} = 1{,}35 \cdot g_{IPE} + 1{,}5 \cdot \left(g_{c,sv} + q_m\right) = 1{,}35 \cdot 0{,}262\,\mathrm{kN/m} + 1{,}5 \cdot (5{,}92 + 1{,}76)\,\mathrm{kN/m}$$

$$q_{Ed} = 11{,}87\,\mathrm{kN/m}$$

Maksimalni moment v polju in prečna sila

$$M_{Ed} = \frac{q_{Ed}\,b^2}{8} = \frac{11{,}87\ \mathrm{kN/m} \cdot 5{,}3^2\ \mathrm{m}^2}{8} = 41{,}68\,\mathrm{kNm}$$

$$V_{Ed} = \frac{q_{Ed}\,b}{2} = \frac{11{,}87\ \mathrm{kN/m} \cdot 5{,}3\ \mathrm{m}}{2} = 31{,}46\,\mathrm{kN}$$

Razred kompaktnosti

- stojina (notranji tlačeni del)

$$\frac{c}{t_w} = \frac{177\ \mathrm{mm}}{5{,}9\ \mathrm{mm}} = 30 \leq 72 \cdot \varepsilon = 72 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1$$

- pasnica (previsni deli pasnic)

$$\frac{c}{t_f} = \frac{40{,}05\ \mathrm{mm}}{9{,}2\ \mathrm{mm}} = 4{,}35 \leq 9 \cdot \varepsilon = 9 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

$$c = \frac{1}{2}(b - t_w - 2\,r) = \frac{1}{2} \cdot (110 - 5{,}9 - 2 \cdot 12)\ \mathrm{mm} = 40{,}05\ \mathrm{mm}$$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

- strig

$$\frac{h_w}{t_w} = \frac{201{,}6\ \mathrm{mm}}{5{,}9\ \mathrm{mm}} = 34{,}17 \leq 72 \cdot \frac{\varepsilon}{\eta} = 72 \cdot \frac{1}{1{,}2} = 60 \quad \rightarrow \text{stojina je kompaktna}$$

Upogibna nosilnost prereza (1. razred kompaktnosti)

$$M_{pl,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 285\ \mathrm{cm}^3 \cdot \frac{23{,}5\,\mathrm{kN/cm}^2}{1{,}0} = 6697{,}5\ \mathrm{kNcm} = 66{,}98\ \mathrm{kNm}$$

$$M_{Ed} = 41{,}68\ \mathrm{kNm} \leq M_{pl,Rd} = 66{,}98\ \mathrm{kNm} \quad \checkmark$$

Strižna nosilnost prereza

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 14{,}27\ \mathrm{cm}^2 \cdot \frac{23{,}5\,\mathrm{kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 193{,}61\ \mathrm{kN}$$

$$A_v = A - 2\,b\,t_f + (t_w + 2\,r)\,t_f$$

$$A_v = 33{,}4\ \mathrm{cm}^2 - 2 \cdot 11\ \mathrm{cm} \cdot 0{,}92\ \mathrm{cm} + (0{,}59\ \mathrm{cm} + 2 \cdot 1{,}2\ \mathrm{cm}) \cdot 0{,}92\ \mathrm{cm}$$

$$A_v = 10{,}41\ \mathrm{cm}^2 \geq \eta\,h_w\,t_w = 1{,}2 \cdot 20{,}16\ \mathrm{cm} \cdot 0{,}59\ \mathrm{cm} = 14{,}27\ \mathrm{cm}^2$$

$$A_v = 14{,}27\ \mathrm{cm}^2$$

$V_{Ed} = 31{,}46 \text{ kN} \leq V_{pl,Rd} = 193{,}61 \text{ kN}$ ✓

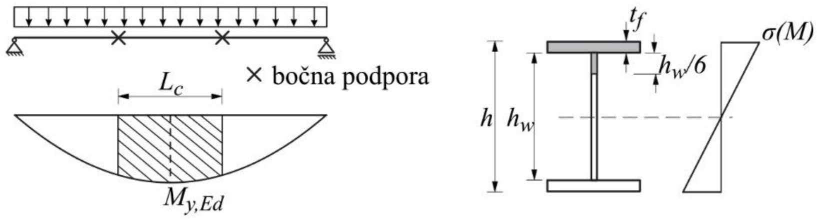
Interakcija M + V

$V_{Ed} = 31{,}46 \text{ kN} \leq 0{,}5 \cdot V_{pl,Rd} = 96{,}81 \text{ kN}$ → kontrola interakcije ni potrebna

Bočna zvrnitev (poenostavljeno preverjanje - metoda tlačene pasnice)

Pri elementih stavb z bočno podprto tlačeno pasnico ni nevarnosti bočne zvrnitve, če je za razmik med sosednjima bočnima podporama L_c oziroma za pripadajočo vitkost tlačene pasnice $\overline{\lambda_f}$ izpolnjen pogoj (glej tudi sliko 13):

$$\overline{\lambda_f} = \frac{k_c \, L_c}{i_{f,z} \, \lambda_1} \leq \overline{\lambda_{c,0}} \, \frac{M_{c,Rd}}{M_{y,Ed}}$$



Slika 13: Metoda tlačene pasnice

$$L_c \leq \overline{\lambda_{c,0}} \, \frac{M_{c,Rd}}{M_{Ed}} \, \frac{i_{f,z} \, \lambda_1}{k_c} = 0{,}5 \cdot \frac{6697{,}5 \text{ kNcm}}{4168 \text{ kNcm}} \cdot \frac{2{,}93 \text{ cm} \cdot 93{,}9}{0{,}94} = 235{,}16 \text{ cm}$$

$$M_{c,Rd} = W_y \cdot \frac{f_y}{\gamma_{M1}} = 285 \text{ cm}^3 \cdot \frac{23{,}5 \text{ kN/cm}^2}{1{,}0} = 6697{,}5 \text{ kNcm}$$

$$W_y = W_{pl,y} = 285 \text{ cm}^3 \quad \text{za 1. razred kompaktnosti}$$

$$i_{f,z} = \sqrt{\frac{I_{f,z}}{A_f}} = \sqrt{\frac{102{,}09 \text{ cm}^4}{11{,}86 \text{ cm}^2}} = 2{,}93 \text{ cm}$$

$$I_{f,z} = \frac{t_f \, b^3}{12} + \frac{(h_w/6) \, t_w^3}{12} = \frac{0{,}92 \cdot 11^3}{12} + \frac{(17{,}7/6) \cdot 0{,}59^3}{12} = 102{,}09 \text{ cm}^4$$

$$A_f = b \, t_f + \frac{h_w}{6} \, t_w = 11 \cdot 0{,}92 + \frac{17{,}7}{6} \cdot 0{,}59 = 11{,}86 \text{ cm}^2$$

$\lambda_1 = 93{,}9 \, \varepsilon = 93{,}9 \cdot 1 = 93{,}9$

$k_c = 0{,}94$ korekcijski faktor, odvisen od momentne linije

$\overline{\lambda_{c,0}} = \overline{\lambda_{LT,0}} + 0{,}1 = 0{,}4 + 0{,}1 = 0{,}5$

$$n = \frac{L}{L_c} = \frac{530 \text{ cm}}{235{,}16 \text{ cm}} = 2{,}25 \approx 3$$

$$L_{c,dej} = \frac{L}{n} = \frac{530 \text{ cm}}{3} = 176{,}67 \text{ cm}$$

Nosilec je potrebno v fazi betoniranja 2-krat bočno podpreti z začasnim povezjem na razdalji 177 cm in 353 cm.

4.3.2.2 MSU - Mejno stanje uporabnosti

Obtežba

$$q_{Ed} = g_{IPE} + g_{c,sv} + q_m = 0{,}262\,\mathrm{kN/m} + 5{,}92\,\mathrm{kN/m} + 1{,}76\,\mathrm{kN/m}$$

$$q_{Ed} = 7{,}94\,\mathrm{kN/m}$$

Kontrola pomikov

$$w = \frac{5\,q_{Ed}\,b^4}{384\,E_s\,I_y} = \frac{5 \cdot 0{,}0794\,\mathrm{kN/cm} \cdot 530^4\,\mathrm{cm}^4}{384 \cdot 21000\,\,\mathrm{kN/cm}^2 \cdot 2770\,\mathrm{cm}^4} = 1{,}40\,\mathrm{cm}$$

$$w_{dop} = \frac{b}{250} = \frac{530\,\mathrm{cm}}{250} = 2{,}12\,\mathrm{cm}$$

$$w = 1{,}40\,\mathrm{cm} \leq w_{dop} = 2{,}12\,\mathrm{cm}\quad \checkmark$$

4.3.3 Končno stanje

4.3.3.1 MSN - Mejno stanje nosilnosti

Obtežba

$$q_{Ed} = 1{,}35 \cdot (g_{IPE} + g_{c,st} + g_s) + 1{,}5 \cdot q$$

$$q_{Ed} = 1{,}35 \cdot (0{,}262 + 5{,}64 + 3{,}67)\,\mathrm{kN/m} + 1{,}5 \cdot 10{,}06\,\mathrm{kN/m}$$

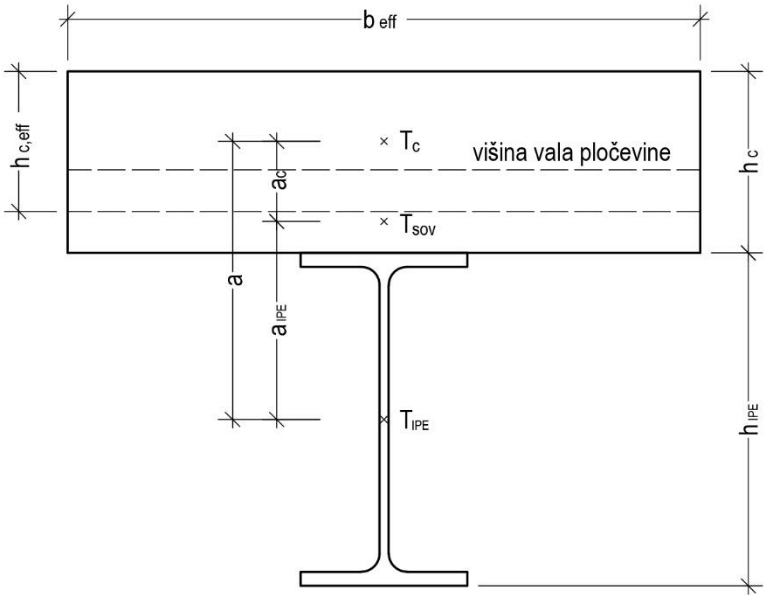
$$q_{Ed} = 28{,}01\,\mathrm{kN/m}$$

Maksimalni moment v polju in prečna sila

$$M_{Ed} = \frac{q_{Ed}\,b^2}{8} = \frac{28{,}01\,\,\mathrm{kN/m} \cdot 5{,}3^2\,\mathrm{m}^2}{8} = 98{,}35\,\mathrm{kNm}$$

$$V_{Ed} = \frac{q_{Ed}\,b}{2} = \frac{28{,}01\,\,\mathrm{kN/m} \cdot 5{,}3\,\mathrm{m}}{2} = 74{,}23\,\mathrm{kN}$$

Sodelujoča širina prereza in geometrijske karakteristike



Slika 14: Prerez nosilca in plošče s pločevino

$b_{eff} = 2\, b_{e1} = 2 \cdot 66{,}25 \text{ cm} = 132{,}5 \text{ cm}$

$$b_{e1} = \frac{b}{8} = \frac{530 \text{ cm}}{8} = 66{,}25 \text{ cm}$$

$h_{c,eff} = 12 \text{ cm} - \frac{5{,}5 \text{ cm}}{2} = 9{,}25 \text{ cm}$

$A_{IPE} = 33{,}4 \text{ cm}^2$

$I_{IPE} = 2770 \text{ cm}^4$

$A_c = b_{eff}\, h_{c,eff} = 132{,}5 \text{ cm} \cdot 132{,}5 \text{ cm} = 1225{,}63 \text{ cm}^2$

$I_c = \frac{b_{eff}\, h_{c,eff}^3}{12} = \frac{132{,}5 \text{ cm} \cdot 9{,}25^3 \text{ cm}^3}{12} = 8738{,}96 \text{ cm}^4$

Vplive lezenja se upošteva z razmerjem elastičnih modulov ($t = t_\infty$)

$n_\infty = 2\, n_0 = 2 \cdot 6{,}77 = 13{,}54$

$$n_0 = \frac{E_s}{E_{cm}} = \frac{21000 \text{ kN/cm}^2}{3100 \text{ kN/cm}^2} = 6{,}77$$

$A_{sov} = A_{IPE} + \frac{A_c}{n_\infty} = 33{,}4 \text{ cm}^2 + \frac{1225{,}63 \text{ cm}^2}{13{,}54} = 123{,}92 \text{ cm}^2$

$I_{sov} = I_{IPE} + A_{IPE}\, a_{IPE}^2 + \frac{I_c + A_c\, a_c^2}{n_\infty}$

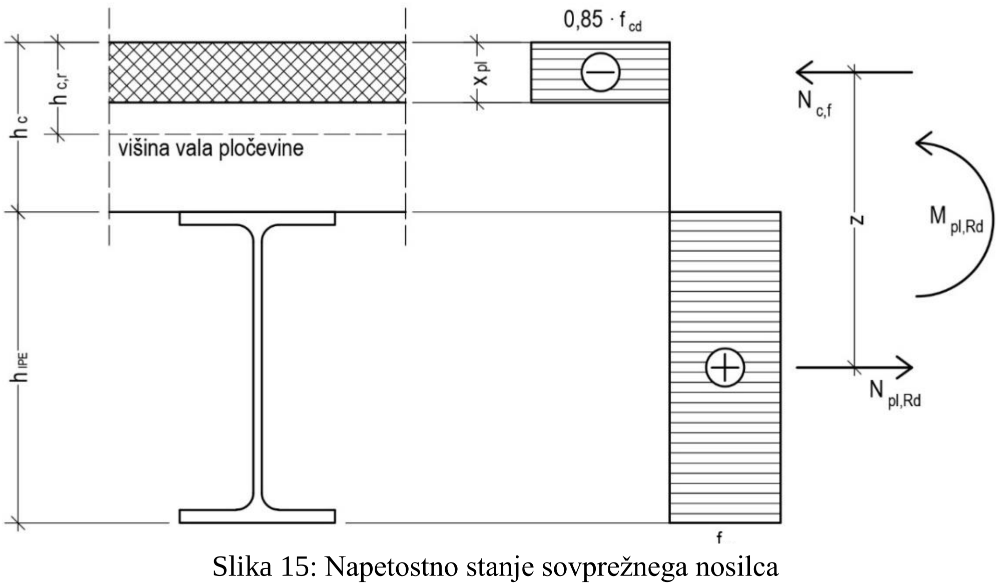
$I_{sov} = 2770 + 33{,}4 \cdot 13{,}43^2 + \frac{8738{,}96 + 1225{,}63 \cdot 4{,}95^2}{13{,}54} = 11657{,}55 \text{ cm}^4$

$$a = \frac{h_{IPE}}{2} + h_c - \frac{h_{c,eff}}{2} = \frac{22}{2} + 12 - \frac{9{,}25}{2} = 18{,}38 \text{ cm}$$

$$a_{IPE} = a\, \frac{A_c}{A_{sov}\, n_\infty} = 18{,}38 \cdot \frac{1225{,}63}{123{,}92 \cdot 13{,}54} = 13{,}43 \text{ cm}$$

$$a_c = a\, \frac{A_{IPE}}{A_{sov}} = 18{,}38 \cdot \frac{33{,}4}{123{,}92} = 4{,}95 \text{ cm}$$

Kontrola upogibne nosilnosti



Slika 15: Napetostno stanje sovprežnega nosilca

$N_{c,f} = N_{pl,Rd}$

$N_{c,f} = \alpha\, f_{cd}\, x_{pl}\, b_{eff}$

$$N_{pl,Rd} = A_{IPE}\,\frac{f_y}{\gamma_{M0}} = 33{,}4\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 784{,}9\ \text{kN}$$

$$x_{pl} = \frac{N_{pl,Rd}}{\alpha\,\frac{f_{ck}}{\gamma_c}\,b_{eff}} = \frac{784{,}9\ \text{kN}}{0{,}85 \cdot \frac{2{,}5\ \text{kN/cm}^2}{1{,}5} \cdot 132{,}5\ \text{cm}} = 4{,}18\ \text{cm}\ \leq h_{c,r} = 6{,}5\ \text{cm}\ \checkmark$$

$h_{c,r} = 6{,}5$ cm   višina betona nad valom

$M_{pl,Rd} = N_{pl,Rd}\, z = 784{,}9\ \text{kN} \cdot 20{,}91\ \text{cm} = 16412{,}26\ \text{kNcm}$

$$z = \frac{h_{IPE}}{2} + h_c - \frac{x_{pl}}{2} = \frac{22}{2} + 12 - \frac{4{,}18}{2} = 20{,}91\ \text{cm}$$

$M_{Ed} = 98{,}35\ \text{kNm} \leq M_{pl,Rd} = 164{,}12\ \text{kNm}$ ✓

Kontrola strižne nosilnosti - vertikalni strig

Predpostavka: celotno strižno silo prevzame jekleni nosilec

$V_{pl,Rd} = 193{,}61$ kN ...(poglavje 4.3.2.1)

$V_{Ed} = 74{,}23\ \text{kN} \leq V_{pl,Rd} = 193{,}61\ \text{kN}$ ✓

Interakcija M + V

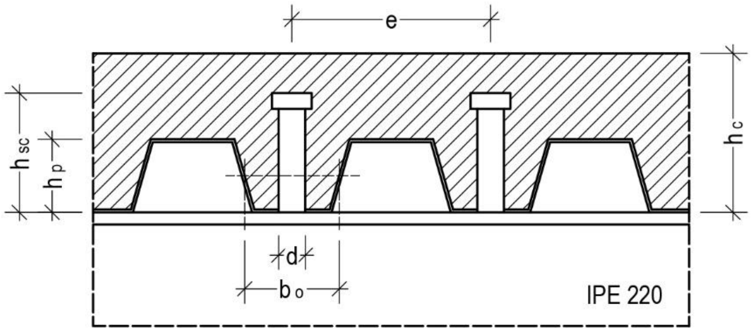
$V_{Ed} = 74{,}23\ \text{kN} \leq 0{,}5 \cdot V_{pl,Rd} = 96{,}81\ \text{kN}$ → kontrola interakcije ni potrebna

4.3.3.2 Kontrola čepov – vzdolžni strig

Upoštevana je polna sovprežnost.

Strižni čepi Nelson Ø19:

$d = 1{,}9$ cm
$h_{sc} = 9$ cm
$f_u = 45\ \text{kN/cm}^2$



Slika 16: Vzdolžni prerez sovprežnega nosilca s čepi

Vzdolžna strižna sila

$F_{cf} = N_{pl,Rd} = 784{,}9$ kN (poglavje 4.3.3.1, slika 15)

Strižna nosilnost čepov z glavo

Projektna strižna nosilnost enega avtomatsko varjenega čepa

$P_{Rd} = \min\{P_{Rd,1}\ \text{ali}\ P_{Rd,2}\} = 73{,}73$ kN

$$P_{Rd,1} = \frac{0{,}8\, f_u \, \frac{\pi\, d^2}{4}}{\gamma_v} = \frac{0{,}8 \cdot 45\ \text{kN/cm}^2 \cdot \frac{\pi \cdot 1{,}9^2\ \text{cm}^2}{4}}{1{,}25} = 81{,}66\ \text{kN}$$

$$P_{Rd,2} = \frac{0{,}29\, \alpha\, d^2 \sqrt{f_{ck}\, E_{cm}}}{\gamma_v} = \frac{0{,}29 \cdot 1 \cdot 1{,}9^2 \cdot \sqrt{2{,}5 \cdot 3100}}{1{,}25} = 73{,}73\ \text{kN}$$

$$\alpha = 1 \ \ \text{za} \ \ \frac{h_{sc}}{d} = \frac{9\ \text{cm}}{1{,}9\ \text{cm}} = 4{,}74 > 4$$

Projektna strižna nosilnost enega čepa, skupaj s profilirano jekleno pločevino (valovi so prečno na podporni nosilec)

$$P_{t,Rd} = k_t\, P_{Rd} = 0{,}61 \cdot 73{,}73\ \text{kN} = 44{,}98\ \text{kN}$$

$$k_t = \frac{0{,}7}{\sqrt{n_r}} \cdot \frac{b_o}{h_p} \cdot \left(\frac{h_{sc}}{h_p} - 1\right) = \frac{0{,}7}{\sqrt{1}} \cdot \frac{7{,}5}{5{,}5} \cdot \left(\frac{9}{5{,}5} - 1\right) = 0{,}61 \leq k_{t,max} = 0{,}85$$

$n_r = 1$ število čepov v valu (ne več kot 2)

$b_o = 7{,}5$ cm srednja širina vala pločevine

$h_p = 5{,}5$ cm višina vala pločevine

Število in razpored čepov

$$e = \frac{L}{2\, n} = \frac{530\ cm}{2 \cdot 17{,}45} = 15{,}19\ \text{cm}$$

$$n = \frac{F_{cf}}{P_{t,Rd}} = \frac{784{,}9\ kN}{44{,}98\ kN} = 17{,}45\ (\approx 18\ \text{čepov na}\ L/2)$$

$e = 15{,}19\ \text{cm} \geq 5\ \text{d} = 9{,}5\ \text{cm}$ ✓

$e = 15{,}19\ \text{cm} \leq \min\{6\ \text{d}\,;80\ \text{cm}\} = 72\ \text{cm}$ ✓

Izberem 36 čepov na 530 cm v vsakem valu, e_{dej} = 15 cm.

Pogoj za enakomerno razporeditev čepov

$$\frac{M_{pl,Rd,sov}}{M_{pl,Rd,IPE}} = \frac{164{,}12\ \text{kNm}}{66{,}98\ \text{kNm}} = 2{,}45 \leq 2{,}5$$ ✓

4.3.3.3 MSU - Mejno stanje uporabnosti

Obtežba

$$q_{Ed} = 1{,}0 \cdot (g_{IPE} + g_{c,st} + g_s) + 1{,}0 \cdot q$$

$$q_{Ed} = 1{,}0 \cdot (0{,}262 + 5{,}64 + 3{,}67)\ \text{kN/m} + 1{,}0 \cdot 10{,}06\ \text{kN/m}$$

$$q_{Ed} = 19{,}63\ \text{kN/m}$$

Kontrola pomikov

$$w = \frac{5\, q_{Ed}\, b^4}{384\, E_s\, I_{sov}} = \frac{5 \cdot 0{,}1963\ \text{kN/cm} \cdot 530^4\ \text{cm}^4}{384 \cdot 21000\ \text{kN/cm}^2 \cdot 11657{,}55\ \text{cm}^4} = 0{,}82\ \text{cm}$$

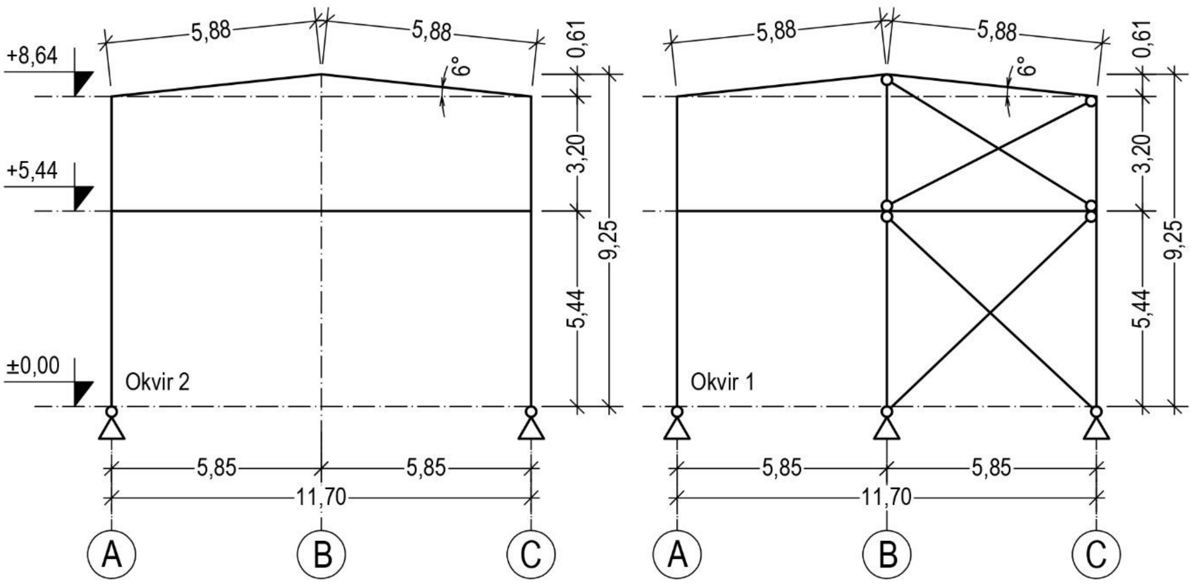
$$w_{dop} = \frac{b}{250} = \frac{530\ \text{cm}}{250} = 2{,}12\ \text{cm}$$

$w = 0{,}82\ \text{cm} \leq w_{dop} = 2{,}12\ \text{cm}$ ✓

5 OKVIRI

5.1 ZASNOVA

Konstrukcija je v vzdolžni smeri sestavljena iz 7 okvirov. Medosni razmaki med njimi so enaki, to je 5,30 m (slika 1). Pri dimenzioniranju obravnavamo najbolj obremenjeni notranji okvir v osi 2 in zunanji okvir v osi 1. Na sliki 17 je prikazana grafična zasnova notranjih momentnih okvirov 2, 3, 4, 5 in 6. Zunanja (čelna) okvira s centričnim povezjem 1 in 7 imata na sredini še dva stebra na katera se pritrdi čelna fasada in sta členkasto vpeta v okvir. Dopustna razdalja med podporami za fasadne panele je 6,50 m (poglavje 9 Streha in fasada).



Slika 17: Zasnova notranjega in zunanjega okvira

5.2 VPLIVI NA OKVIRE

5.2.1 Obtežba na notranji momentni okvir 2

Vplivno območje obtežbe na en notranji okvir b = 5,30 m.

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija g_m = 24,93 kN/m

  $g_m = g'_m\, b + n\, g_{IPE}\, b/L$

  g_m = 4,57 kN/m^2 · 5,30 m + 6 · 0,262 kN/m · 5,30 m / 11,70 m
- streha g_s = 3,18 kN/m

  $g_s = g'_s\, b$ = 0,60 kN/m^2 · 5,30 m
- fasada g_f = 1,59 kN/m

  $g_f = g'_f\, b$ = 0,30 kN/m^2 · 5,30 m

Spremenljiva obtežba

- koristna obtežba medetaže q_m = 22,79 kN/m

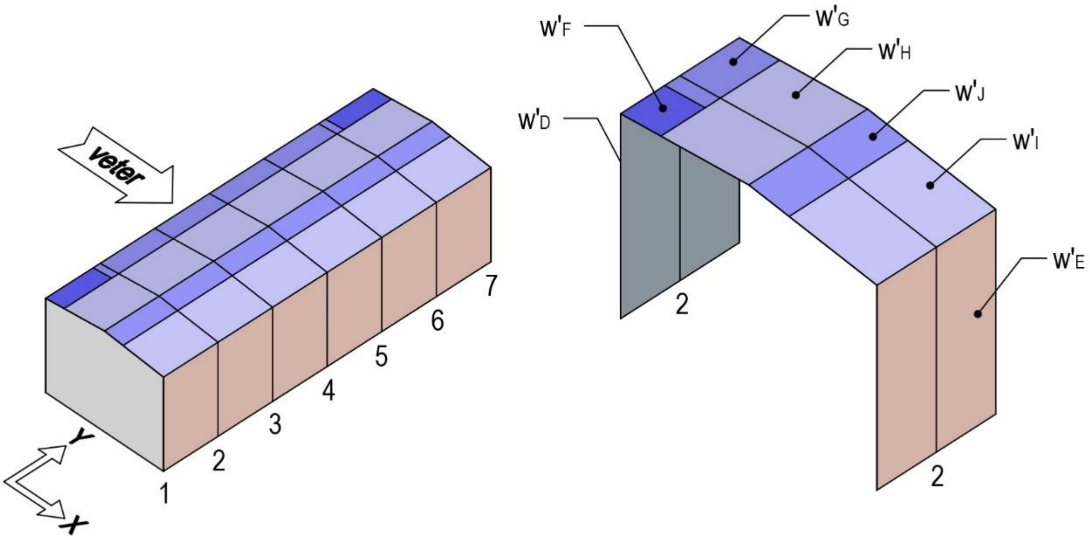
  $q_m = q'_m\, b$ = 4,30 kN/m^2 · 5,30 m

- sneg $s =$ 6,73 kN/m

  $s = s'\,b = 1{,}27\ \text{kN/m}^2 \cdot 5{,}30\ \text{m}$

- veter

  Za dvokapnice je treba obravnavati štiri primere, kjer so največje ali najmanjše vrednosti za področja F, G in H kombinirane z največjimi in najmanjšimi vrednostmi v področjih I in J. Mešanje pozitivnih in negativnih vrednosti na isti stršini ni dovoljeno. Za obravnavani okvir izberem dve kombinaciji vetra, ki delujeta najbolj neugodno. Na sliki 18 je prikazana obtežba vetra (delovanje vetra v smeri X) na vse okvire in posebaj na okvir 2, ki je najbolj obremenjen.



Slika 18: Obtežba vetra na okvire pri delovanju vetra v smeri X in okvir 2

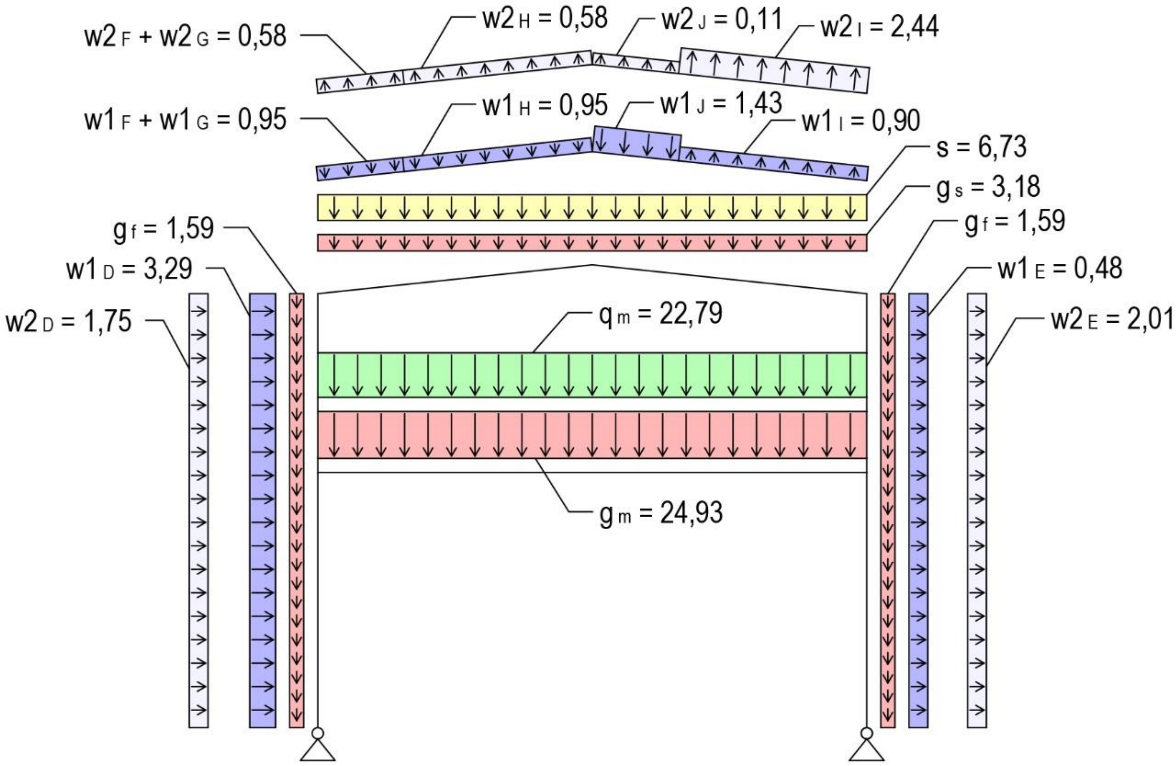
Preglednica 10: Kombinacija 1 - obtežba vetra (w1) na okvir 2 (pri delovanju vetra v smeri X in notranjem srku)

<table>
<tr><th>Področje</th><th>w' [kN/m^2]</th><th>b [m]</th><th colspan="2">$w1 = w'\,b$ [kN/m]</th><th>Opomba</th></tr>
<tr><td>D</td><td>+0,62</td><td>5,30</td><td colspan="2">+3,29</td><td></td></tr>
<tr><td>E</td><td>–0,09</td><td>5,30</td><td colspan="2">–0,48</td><td></td></tr>
<tr><td>F</td><td>+0,18</td><td>0,68</td><td>+0,12</td><td rowspan="2">+0,95</td><td rowspan="2">širina področja je 1,85 m</td></tr>
<tr><td>G</td><td>+0,18</td><td>4,63</td><td>+0,83</td></tr>
<tr><td>H</td><td>+0,18</td><td>5,30</td><td colspan="2">+0,95</td><td>širina področja je 4,03 m</td></tr>
<tr><td>I</td><td>–0,17</td><td>5,30</td><td colspan="2">–0,90</td><td>širina področja je 4,03 m</td></tr>
<tr><td>J</td><td>+0,27</td><td>5,30</td><td colspan="2">+1,43</td><td>širina področja je 1,85 m</td></tr>
</table>

Preglednica 11: Kombinacija 2 - obtežba vetra (w2) na okvir 2 (pri delovanju vetra v smeri X in notranjem tlaku)

<table>
<tr><th>Področje</th><th>w' [kN/m^2]</th><th>b [m]</th><th colspan="2">$w2 = w'\,b$ [kN/m]</th><th>Opomba</th></tr>
<tr><td>D</td><td>+0,33</td><td>5,30</td><td colspan="2">+1,75</td><td></td></tr>
<tr><td>E</td><td>–0,38</td><td>5,30</td><td colspan="2">–2,01</td><td></td></tr>
<tr><td>F</td><td>–0,11</td><td>0,68</td><td>–0,07</td><td rowspan="2">–0,58</td><td rowspan="2">širina področja je 1,85 m</td></tr>
<tr><td>G</td><td>–0,11</td><td>4,63</td><td>–0,51</td></tr>
<tr><td>H</td><td>–0,11</td><td>5,30</td><td colspan="2">–0,58</td><td>širina področja je 4,03 m</td></tr>
<tr><td>I</td><td>–0,46</td><td>5,30</td><td colspan="2">–2,44</td><td>širina področja je 4,03 m</td></tr>
<tr><td>J</td><td>–0,02</td><td>5,30</td><td colspan="2">–0,11</td><td>širina področja je 1,85 m</td></tr>
</table>

Na sliki 19 je prikazana razporeditev vse obtežbe na okvir 2 v programu SCIA Engineer.



Slika 19: Obtežba na momentnem okviru 2 v kN/m

5.2.2 Obtežba na zunanji okvir 1 s centričnim povezjem (sistem nateznih diagonal)

Vplivno območje obtežbe na en zunanji okvir b = 2,65 m.

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija $g_m =$ 12,47 kN/m

  $g_m = g'_m\, b + n\, g_{IPE}\, b/L$

  g_m = 4,57 kN/m^2 · 2,65 m + 6 · 0,262 kN/m · 2,65 m / 11,70 m
- streha $g_s =$ 1,59 kN/m

  $g_s = g'_s\, b$ = 0,60 kN/m^2 · 2,65 m
- fasada $g_{f1} =$ 1,69 kN/m

  $g_{f2} =$ 1,73 kN/m

  $g_{f1} = g'_f\, b + g'_f\, A_1/L$ = 0,30 kN/m^2 · 2,65 m + 0,30 kN/m^2 · 25,72 m^2 / 8,64 m

  $g_{f2} = g'_f\, A_2/L$ = 0,30 kN/m^2 · 53,22 m^2 / 9,25 m

Spremenljiva obtežba

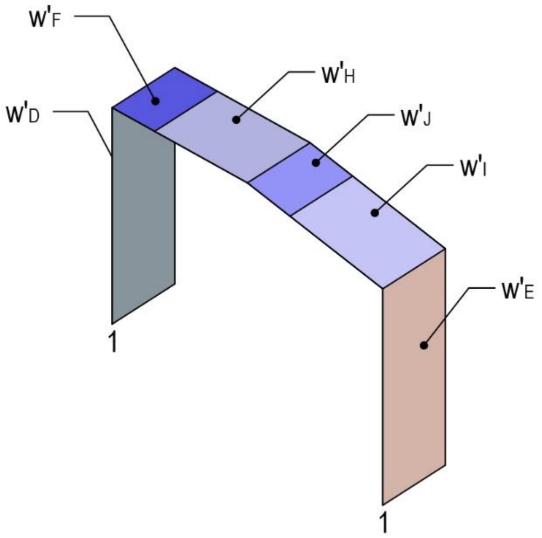
- koristna obtežba medetaže $q_m =$ 11,40 kN/m

  $q_m = q'_m\, b$ = 4,30 kN/m^2 · 2,65 m
- sneg $s =$ 3,37 kN/m

  $s = s'\, b$ = 1,27 kN/m^2 · 2,65 m

- veter

  Za dvokapnice je treba obravnavati štiri primere, kjer so največje ali najmanjše vrednosti za področja F in H kombinirane z največjimi in najmanjšimi vrednostmi v področjih I in J. Mešanje pozitivnih in negativnih vrednosti na isti stršini ni dovoljeno. Za obravnavani okvir izberem dve kombinaciji vetra, ki delujeta najbolj neugodno. Na sliki 20 je prikazana obtežba vetra (delovanje vetra v smeri X) na zunanji okvir 1.



Slika 20: Obtežba vetra na okvir 1 pri delovanju vetra v smeri X

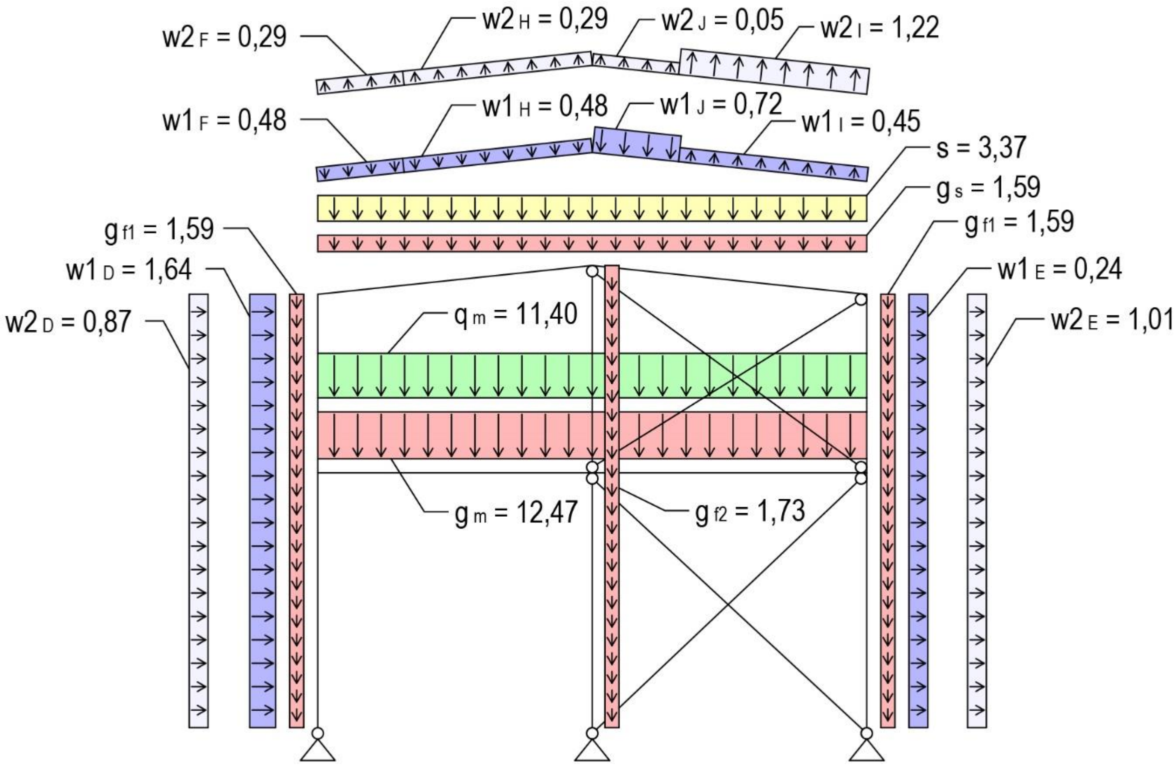
Preglednica 12: Kombinacija 1 - obtežba vetra (w1) na okvir 1 (pri delovanju vetra v smeri X in notranjem srku)

Področje	***w'* [kN/m²]**	***b* [m]**	***w1 = w' b* [kN/m]**	**Opomba**
D	+0,62	2,65	+1,64	
E	–0,09	2,65	–0,24	
F	+0,18	2,65	+0,48	širina področja je 1,85 m
H	+0,18	2,65	+0,48	širina področja je 4,03 m
I	–0,17	2,65	–0,45	širina področja je 4,03 m
J	+0,27	2,65	+0,72	širina področja je 1,85 m

Preglednica 13: Kombinacija 2 - obtežba vetra (w2) na okvir 1 (pri delovanju vetra v smeri X in notranjem tlaku)

Področje	***w'* [kN/m²]**	***b* [m]**	***w2 = w' b* [kN/m]**	**Opomba**
D	+0,33	2,65	+0,87	
E	–0,38	2,65	–1,01	
F	–0,11	2,65	–0,29	širina področja je 1,85 m
H	–0,11	2,65	–0,29	širina področja je 4,03 m
I	–0,46	2,65	–1,22	širina področja je 4,03 m
J	–0,02	2,65	–0,05	širina področja je 1,85 m

Na sliki 21 je prikazana razporeditev vse obtežbe na okvir 1 v programu SCIA Engineer.



Slika 21: Obtežba na okviru 1 v kN/m

5.2.3 Obtežne kombinacije

Za vsak kritični obtežni primer je treba določiti projektne vrednosti učinkov vplivov s kombiniranjem vrednosti vplivov, za katere se ocenjuje, da se bodo pojavili sočasno.

5.2.3.1 MSN - Mejno stanje nosilnosti

Splošna oblika kombinacije učinkov vplivov je

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i}$$

kjer so:

$G_{k,j}$ lastna in stalna obtežba

$Q_{k,1}$ prevladujoči spremenljivi vpliv

$Q_{k,i}$ ostali spremenljivi vplivi

γ varnostni faktor (1,35 za lastno in stalno obtežbo ter 1,5 za spremenljivo obtežbo)

ψ_0 kombinacijski faktor, ki izraža majhno verjetnost, da bi več spremenljivih vplivov nastopilo istočasno v maksimalni vrednosti (0,7 za koristno obtežbo, 0,6 za obtežbo vetra in 0,5 za obtežbo snega)

Preglednica 14: Obtežne kombinacije za MSN

Kombinacija	**Lastna in stalna obtežba *G***	**Koristna obtežba *Q***	**Obtežba snega *S***	**Obtežba vetra *W***
MSN1	1,35 · *G*	1,5 · *Q*	1,5 · 0,5 · *S*	1,5 · 0,6 · *W1*
MSN2	1,35 · *G*	1,5 · *Q*	1,5 · 0,5 · *S*	1,5 · 0,6 · *W2*
MSN3	1,35 · *G*	1,5 · 0,7 · *Q*	1,5 · *S*	1,5 · 0,6 · *W1*
MSN4	1,35 · *G*	1,5 · 0,7 · *Q*	1,5 · *S*	1,5 · 0,6 · *W2*
MSN5	1,35 · *G*	1,5 · 0,7 · *Q*	1,5 · 0,5 · *S*	1,5 · *W1*
MSN6	1,35 · *G*	1,5 · 0,7 · *Q*	1,5 · 0,5 · *S*	1,5 · *W2*
MSN7	1,35 · *G*	1,5 · *Q*	1,5 · 0,5 · *S*	/
MSN8	1,35 · *G*	1,5 · 0,7 · *Q*	1,5 · *S*	/
MSN9	1,35 · *G*	1,5 · *Q*	/	1,5 · 0,6 · *W1*
MSN10	1,35 · *G*	1,5 · *Q*	/	1,5 · 0,6 · *W2*
MSN11	1,35 · *G*	1,5 · 0,7 · *Q*	/	1,5 · *W1*
MSN12	1,35 · *G*	1,5 · 0,7 · *Q*	/	1,5 · *W2*
MSN13	1,00 · *G*	/	/	1,5 · *W1*
MSN14	1,00 · *G*	/	/	1,5 · *W2*

5.2.3.2 MSU - Mejno stanje uporabnosti

Splošna oblika kombinacije učinkov vplivov je

$$\sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i}\, Q_{k,i}$$

kjer so:

$G_{k,j}$ lastna in stalna obtežba

$Q_{k,1}$ prevladujoči spremenljivi vpliv

$Q_{k,i}$ ostali spremenljivi vplivi

ψ_0 kombinacijski faktor, ki izraža majhno verjetnost, da bi več spremenljivih vplivov nastopilo istočasno v maksimalni vrednosti (0,7 za koristno obtežbo, 0,6 za obtežbo vetra in 0,5 za obtežbo snega)

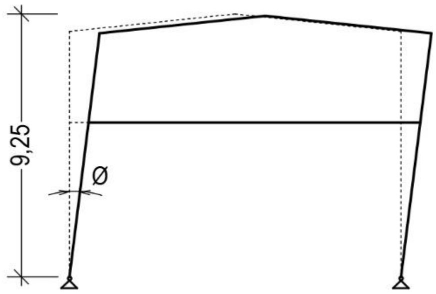
Preglednica 15: Obtežne kombinacije za MSU

Kombinacija	**Lastna in stalna obtežba *G***	**Koristna obtežba *Q***	**Obtežba snega *S***	**Obtežba vetra *W***
MSU1	*G*	*Q*	0,5 · *S*	0,6 · *W1*
MSU2	*G*	*Q*	0,5 · *S*	0,6 · *W2*
MSU3	*G*	0,7 · *Q*	*S*	0,6 · *W1*
MSU4	*G*	0,7 · *Q*	*S*	0,6 · *W2*
MSU5	*G*	0,7 · *Q*	0,5 · *S*	*W1*
MSU6	*G*	0,7 · *Q*	0,5 · *S*	*W2*
MSU7	*G*	*Q*	0,5 · *S*	/
MSU8	*G*	0,7 · *Q*	*S*	/

MSU9	*G*	*Q*	/	0,6 · *W1*
MSU10	*G*	*Q*	/	0,6 · *W2*
MSU11	*G*	0,7 · *Q*	/	*W1*
MSU12	*G*	0,7 · *Q*	/	*W2*

5.2.4 Globalna nepopolnost v prečni smeri

Pri pomičnih okvirih se vpliv nepopolnosti upošteva z nadomestnimi globalnimi nepopolnostmi v obliki nadomestnega vodoravnega zamika okvira (slika 22).



Slika 22: Nadomestni vodoravni zamik okvira

$$\phi = \phi_0 \, \alpha_h \, \alpha_m = \frac{1}{200} \cdot \frac{2}{3} \cdot \frac{\sqrt{3}}{2} = 0{,}00289$$

$$\phi_0 = \frac{1}{200}$$

$$\alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{9{,}25}} = 0{,}658 \quad vendar \; \frac{2}{3} \leq \alpha_h \leq 1{,}0$$

$$\alpha_m = \sqrt{0{,}5 \cdot \left(1 + \frac{1}{m}\right)} = \sqrt{0{,}5 \cdot \left(1 + \frac{1}{2}\right)} = \frac{\sqrt{3}}{2}$$

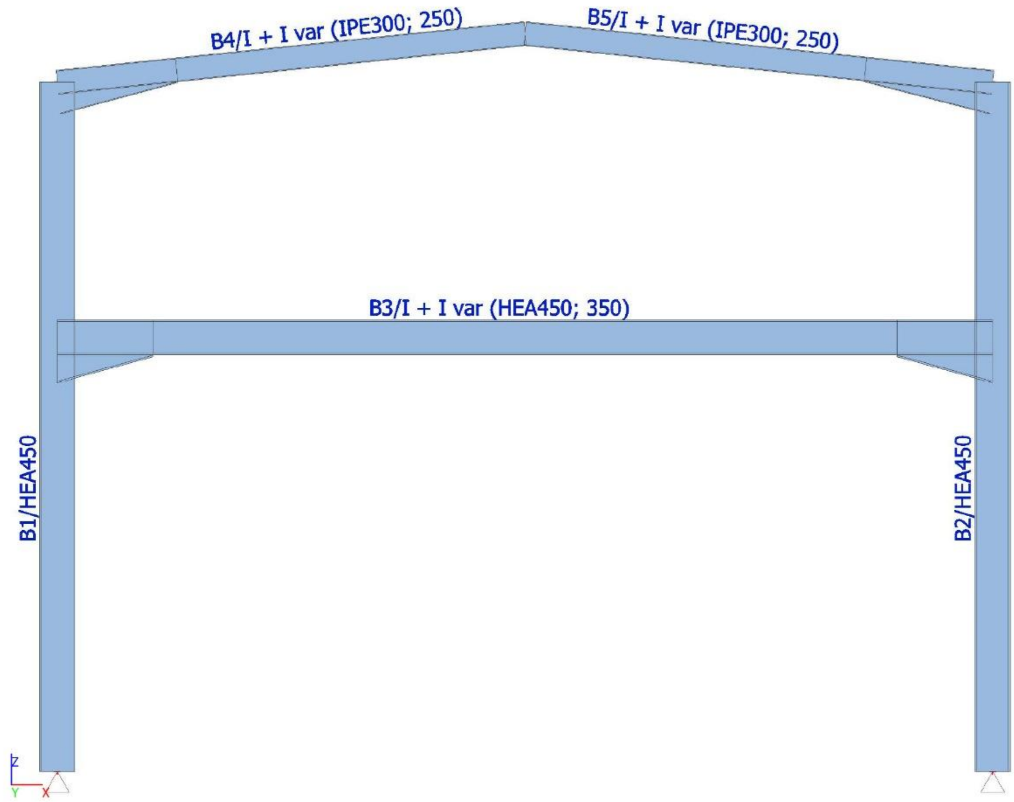
$m = 2$ število stebrov (nosijo vsaj 50% povprečne navpične obtežbe)

$$d_x = \phi \cdot 1000 = 2{,}89 \text{ mm/m}$$

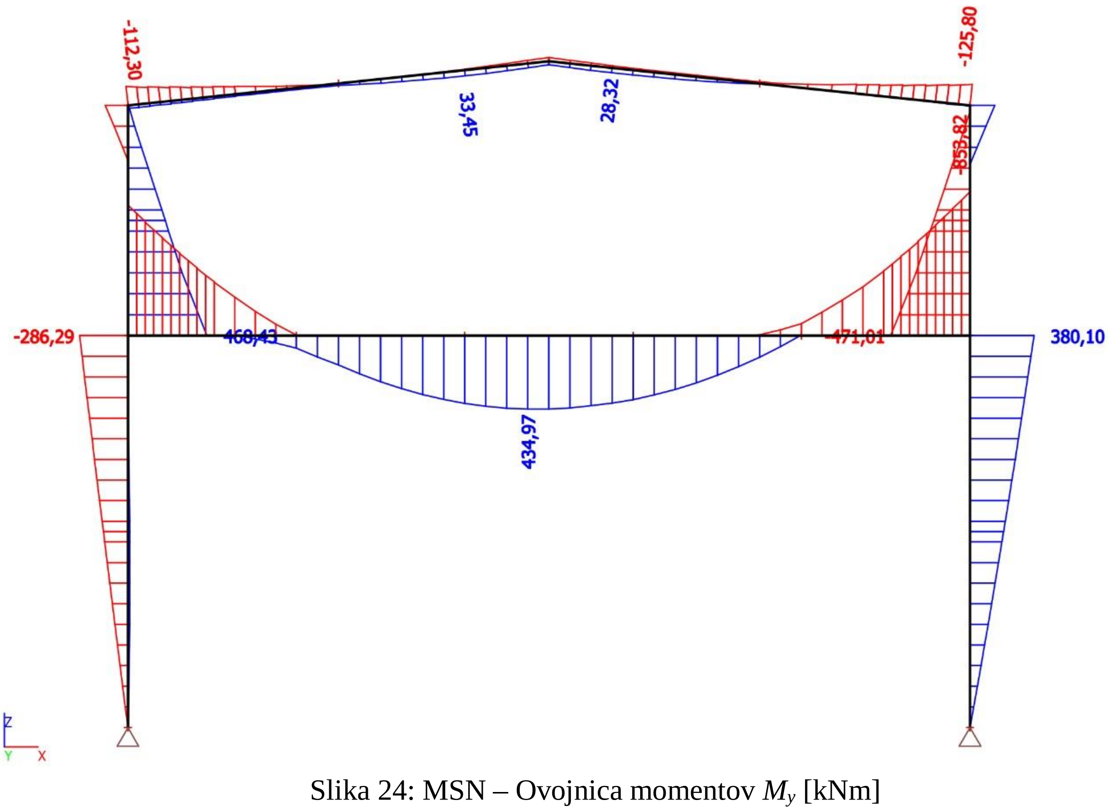
5.3 MSN - MEJNO STANJE NOSILNOSTI

5.3.1 Analiza momentnega okvira 2

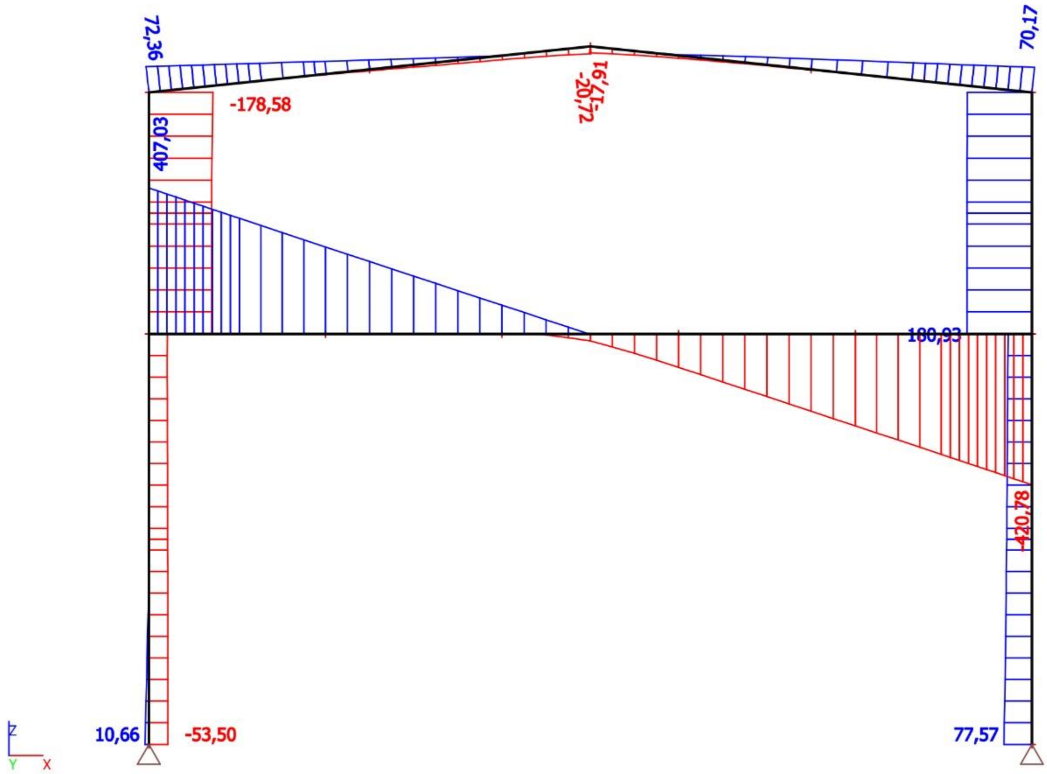
Kot merodajno obremenitev upoštevam ovojnico obtežnih kombinacij MSN z globalno nepopolnostjo v prečni smeri. Rezultati iz programa SCIA Engineer so prikazani na slikah 23, 24, 25 in 26.



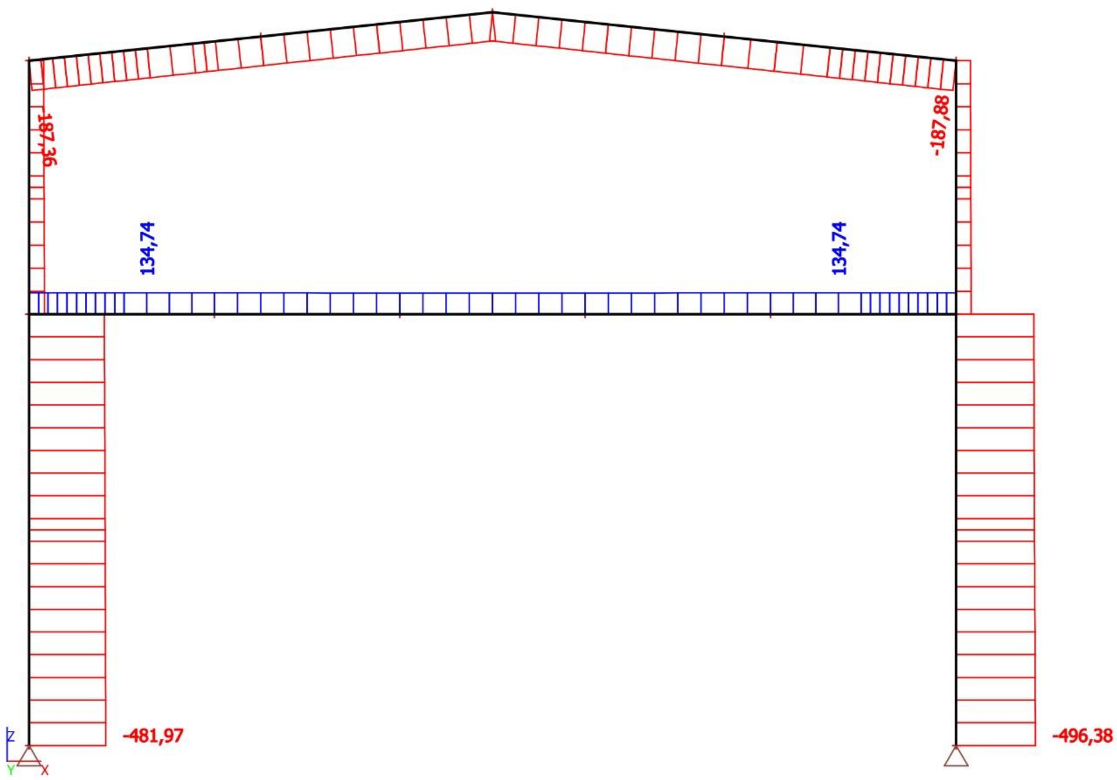
Slika 23: Statični model



Slika 24: MSN – Ovojnica momentov M_y [kNm]



Slika 25: MSN – Ovojnica prečnih sil *V* [kN]



Slika 26: MSN – Ovojnica osnih sil *N* [kN]

5.3.2 Dimenzioniranje momentnega okvira 2

Računske kontrole so izvedene s programom SCIA Engineer in so v prilogi (Priloga A – Kontrola nosilnosti in stabilnosti momentnega okvira 2). V nadaljevanju je prikazan povzetek izpisa.

5.3.2.1 Steber (B2) - HEA 450, S235

Notranje količine (MSN9)

$N_{Ed} = 466{,}47$ kN (tlak)

$V_{Ed} = 72{,}21$ kN

$M_{y,Ed} = 380{,}04$ kNm

Razred kompaktnosti prereza

- stojina (notranji tlačeni del)

$$\frac{c}{t_w} = 29{,}91 \leq 33{,}00 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

- pasnica (previsni deli pasnic)

$$\frac{c}{t_f} = 5{,}58 \leq 9 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

Nosilnost prereza

- tlak

$$N_{c,Rd} = \frac{A\,f_y}{\gamma_{M0}} = 4183{,}00\text{ kN} \geq N_{Ed} = 466{,}47\text{ kN} \checkmark$$

- strig

$$V_{pl,Rd} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = 892{,}15\text{ kN} \geq V_{Ed} = 72{,}21\text{ kN} \checkmark$$

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$$N_{b,Rd} = \frac{\chi\,A\,f_y}{\gamma_{M1}} = 3045{,}44\text{ kN} \geq N_{Ed} = 466{,}47\text{ kN} \checkmark$$

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\frac{\chi_y\,N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$0{,}11 + 0{,}53 + 0{,}00 = 0{,}65 \leq 1 \checkmark$$

$$\frac{N_{Ed}}{\frac{\chi_z\,N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$0{,}15 + 0{,}28 + 0{,}00 = 0{,}43 \leq 1 \checkmark$$

5.3.2.2 Prečka medetaže (B3) - HEA 450 in vuta 350, S235

Notranje količine (MSN1)

$N_{Ed} = 112{,}36$ kN (nateg)

$V_{Ed} = 369{,}76$ kN

$M_{y,Ed} = 557{,}15$ kNm

Razred kompaktnosti prereza

Prerez (HEA 450 + vuta) spada v 3. razred kompaktnosti (elastični prerez).

Nosilnost prereza

- nateg

$$N_{t,Rd} = \frac{A\,f_y}{\gamma_{M0}} = 6008{,}53 \text{ kN} \geq N_{Ed} = 112{,}36 \text{ kN} \ \checkmark$$

- upogib

$$M_{el,Rd} = \frac{W_{el,min}\,f_y}{\gamma_{M0}} = 885{,}70 \text{ kNm} \geq M_{y,Ed} = 557{,}15 \text{ kNm} \ \checkmark$$

- strig

$$\frac{\tau_{Ed}}{f_y/(\sqrt{3}\,\gamma_{M0})} = 0{,}50 \leq 1 \ \checkmark$$

- upogib, strig in osna sila

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right) + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2 = 0{,}69 \leq 1 \ \checkmark$$

Odpornost elementov proti nestabilnosti

- bočna zvrnitev upogibno obremenjenih elementov

$\overline{\lambda_{LT}} = 0{,}19 \leq \overline{\lambda_{LT,0}} = 0{,}40$

Vpliv bočne zvrnitve se lahko zanemari.

5.3.2.3 Prečka strehe (B5) - IPE 300 in vuta 250, S235

Notranje količine (MSN3)

$N_{Ed} = 187{,}88$ kN (tlak)

$V_{Ed} = 70{,}17$ kN

$M_{y,Ed} = 125{,}80$ kNm

Razred kompaktnosti prereza

Prerez (IPE 300 + vuta) spada v 3. razred kompaktnosti (elastični prerez).

Nosilnost prereza

- tlak

$$N_{c,Rd} = \frac{A\,f_y}{\gamma_{M0}} = 2025{,}06 \text{ kN} \geq N_{Ed} = 187{,}88 \text{ kN} \ \checkmark$$

- upogib

$$M_{el,Rd} = \frac{W_{el,min}\, f_y}{\gamma_{M0}} = 264{,}55 \text{ kNm} \geq M_{y,Ed} = 125{,}80 \text{ kNm} \ \checkmark$$

- strig

$$\frac{\tau_{Ed}}{f_y/(\sqrt{3}\,\gamma_{M0})} = 0{,}17 \leq 1 \ \checkmark$$

- upogib, strig in osna sila

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right) + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2 = 0{,}55 \leq 1 \ \checkmark$$

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$$N_{b,Rd} = \frac{\chi\, A\, f_y}{\gamma_{M1}} = 1034{,}10 \text{ kN} \geq N_{Ed} = 187{,}88 \text{ kN} \ \checkmark$$

- bočna zvrnitev upogibno obremenjenih elementov

$$\frac{M_{Ed}}{M_{cr}} = 0{,}13 \leq \overline{\lambda_{LT,0}}^2 = 0{,}16$$

Vpliv bočne zvrnitve se lahko zanemari.

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\frac{\chi_y\, N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$0{,}10 + 0{,}51 + 0{,}00 = 0{,}61 \leq 1 \ \checkmark$$

$$\frac{N_{Ed}}{\frac{\chi_z\, N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

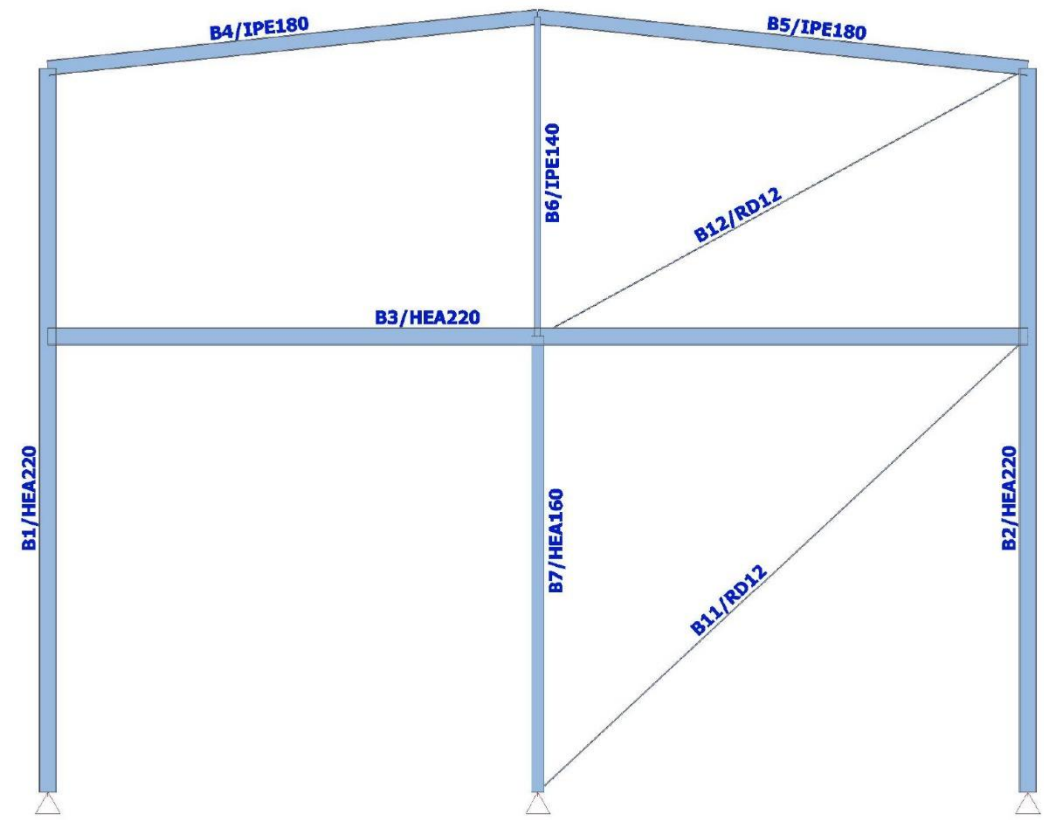
$$0{,}17 + 0{,}49 + 0{,}00 = 0{,}66 \leq 1 \ \checkmark$$

5.3.3 Analiza okvira 1 s centričnim povezjem

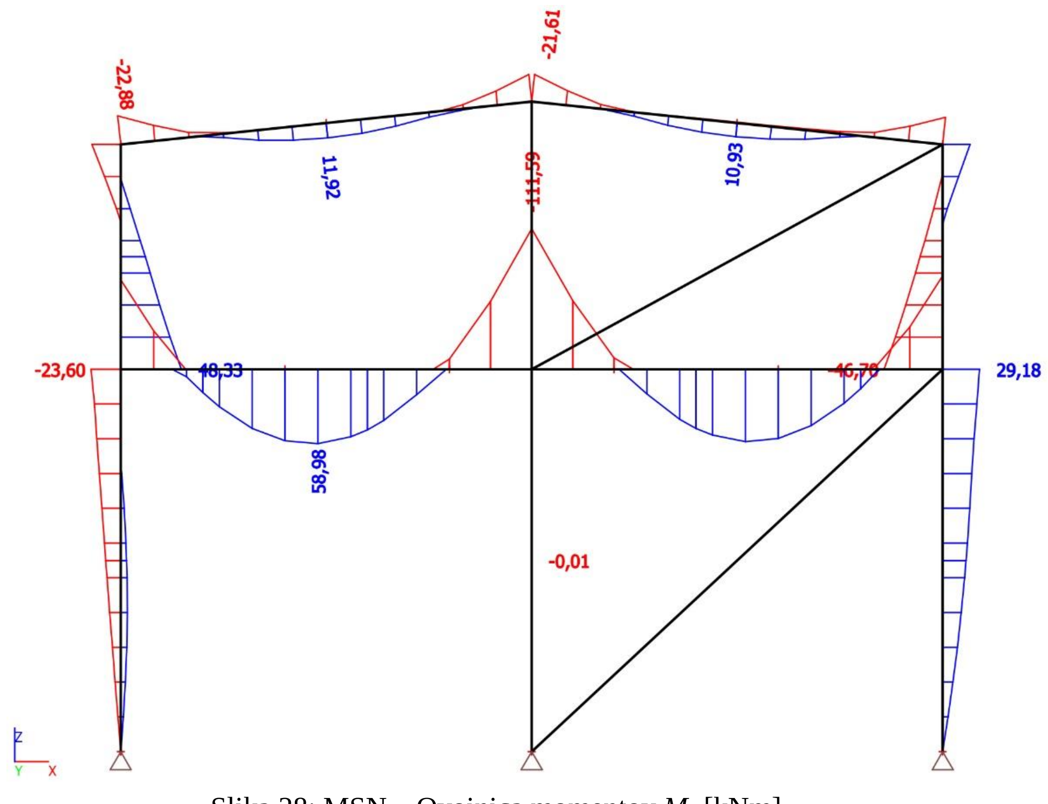
Kot merodajno obremenitev upoštevam ovojnico obtežnih kombinacij MSN z globalno nepopolnostjo v prečni smeri. Rezultati iz programa SCIA Engineer so prikazani na slikah 27, 28, 29 in 30.

5.3.4 Dimenzioniranje okvira 1 s centričnim povezjem

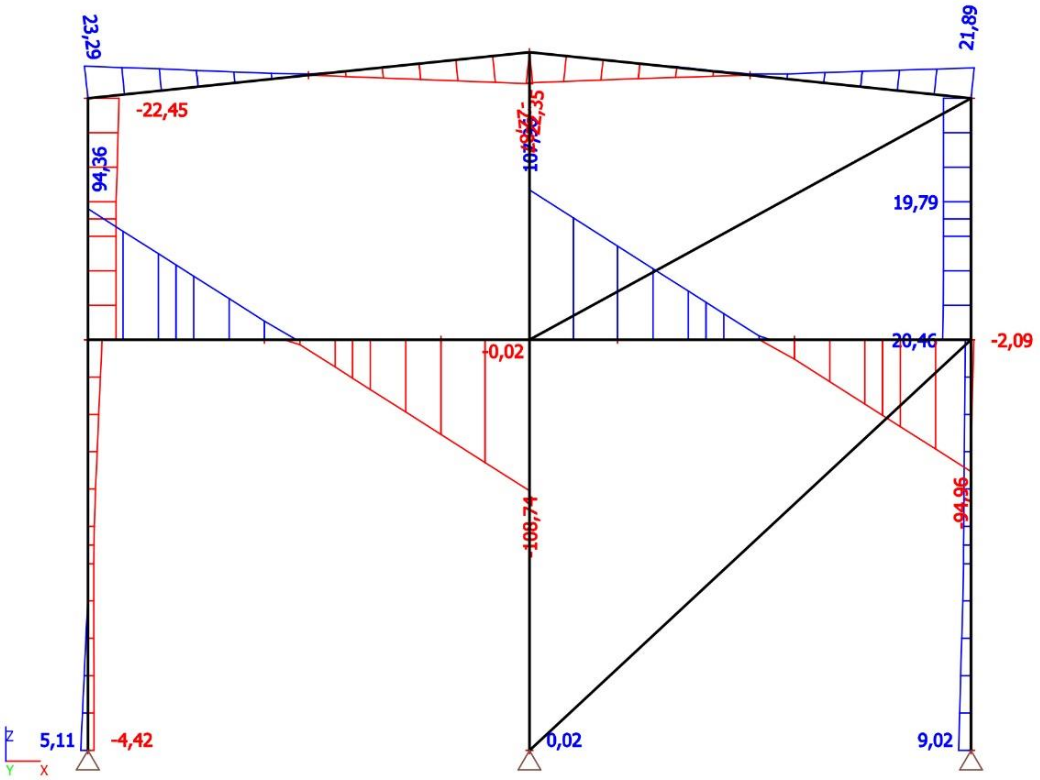
Računske kontrole so izvedene s programom SCIA Engineer in so v prilogi (Priloga A – Kontrola nosilnosti in stabilnosti okvira 1 s centričnim povezjem).



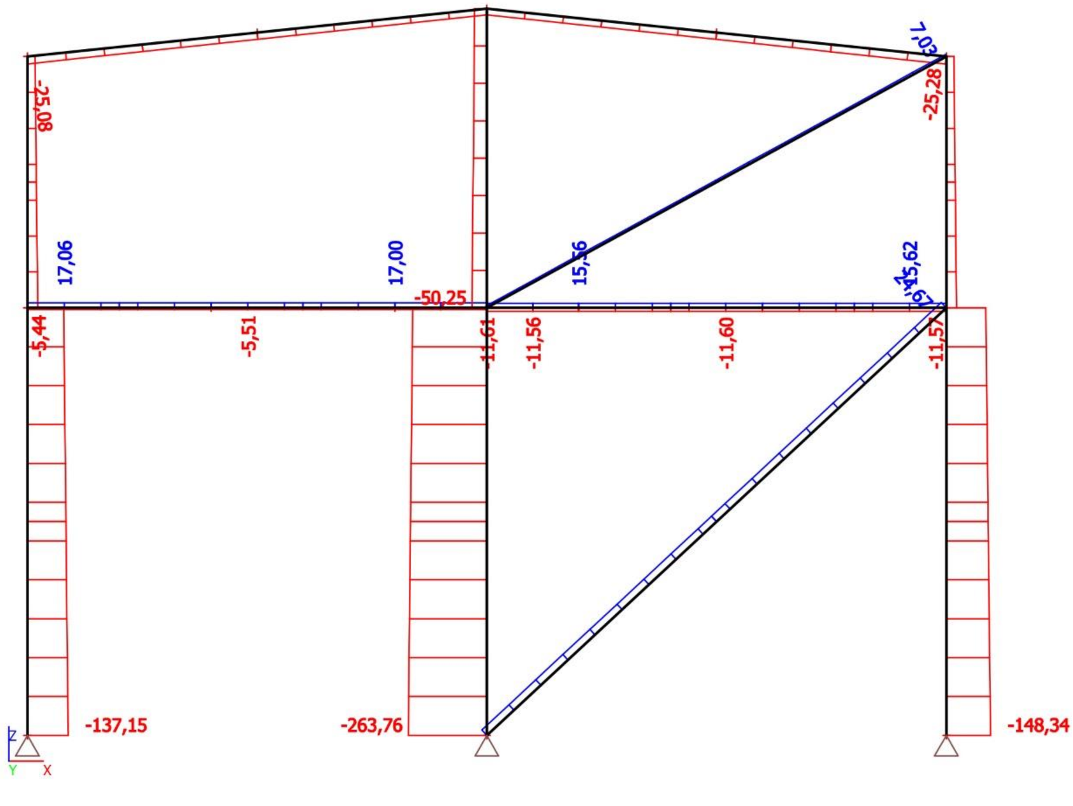
Slika 27: Statični model



Slika 28: MSN – Ovojnica momentov M_y [kNm]



Slika 29: MSN – Ovojnica prečnih sil *V* [kN]



Slika 30: MSN – Ovojnica osnih sil *N* [kN]

5.4 MSU - MEJNO STANJE UPORABNOSTI

Kot merodajno obremenitev upoštevam ovojnico obtežnih kombinacij MSU z globalno nepopolnostjo v prečni smeri.

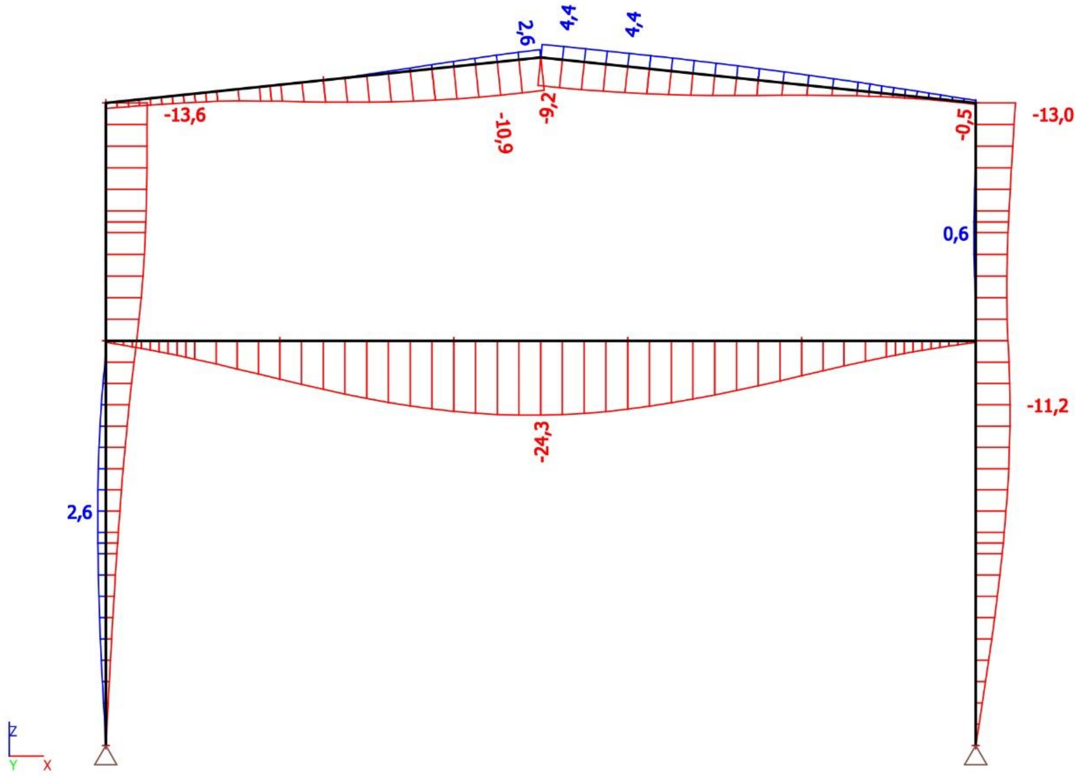
5.4.1 Kontrola vertikalnih pomikov momentnega okvira 2

Kontrola vertikalnih pomikov prečke medetaže

$$w_{dop} = \frac{L}{250} = \frac{11700 \text{ mm}}{250} = 46{,}8 \text{ mm} \geq w = 24{,}3 \text{ mm}$$

Kontrola vertikalnih pomikov prečke strehe

$$w_{dop} = \frac{L}{250} = \frac{5880 \text{ mm}}{250} = 23{,}5 \text{ mm} \geq w = 10{,}9 \text{ mm}$$



Slika 31: MSU - Ovojnica pomikov elementov v lokalni Z smeri [mm]

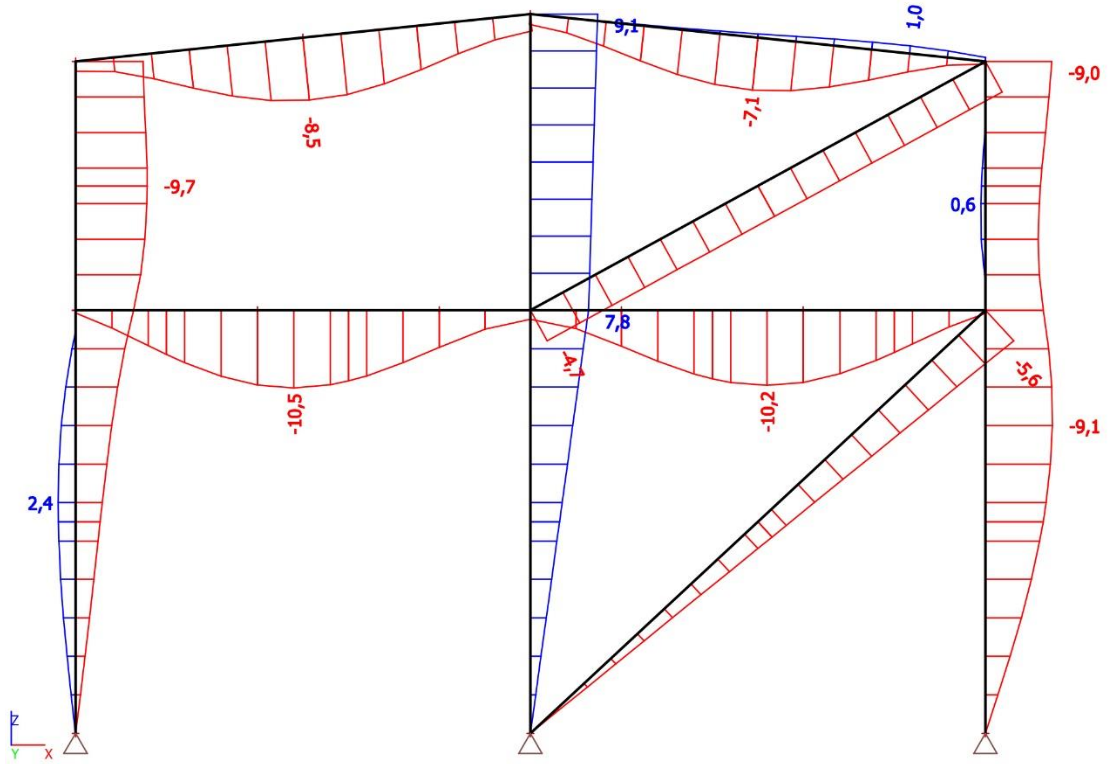
5.4.2 Kontrola vertikalnih pomikov okvira 1 s centričnim povezjem

Kontrola vertikalnih pomikov prečke medetaže

$$w_{dop} = \frac{L}{250} = \frac{5850 \text{ mm}}{250} = 23{,}4 \text{ mm} \geq w = 10{,}5 \text{ mm}$$

Kontrola vertikalnih pomikov prečke strehe

$$w_{dop} = \frac{L}{250} = \frac{5880 \text{ mm}}{250} = 23{,}5 \text{ mm} \geq w = 8{,}5 \text{ mm}$$

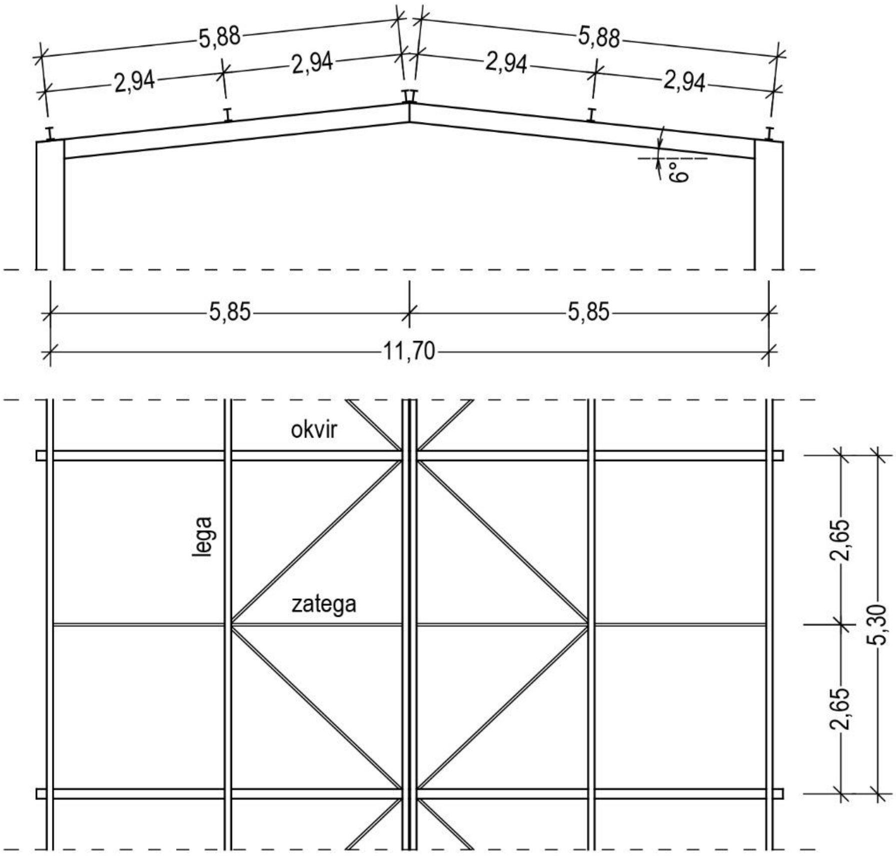


Slika 32: MSU - Ovojnica pomikov elementov v lokalni Z smeri [mm]

6 STREŠNE LEGE

6.1 ZASNOVA

Dopustna razdalja med legami za strešne panele je 3,40 m (poglavje 9 Streha in fasada).



Slika 33: Zasnova strešnih leg in zateg med njimi

6.2 VPLIVI NA STREŠNE LEGE

6.2.1 Obtežba

Vplivno območje obtežbe na notranjo (srednjo) lego v polju je b = 2,94 m.

Lastna teža

Lastna teža strešnih leg je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

$g_s = g'_s\, b \cos \alpha$ = 0,60 kN/m^2 · 2,94 m· cos 6° = 1,75 kN/m

Spremenljiva obtežba

sneg

$s = s'\, b \cos \alpha$ = 1,27 kN/m^2 · 2,94 m· cos 6° = 3,71 kN/m

veter

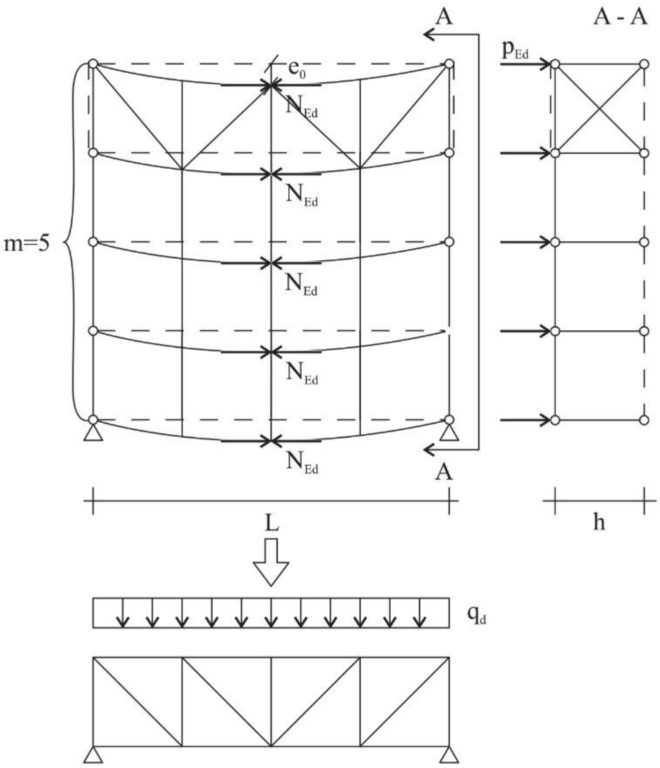
$w = w'_{max}\, b$ = 0,27 kN/m^2 · 2,94 m = 0,79 kN/m

6.2.2 Izbočne sile momentnega okvira - nepopolnosti pri globalni analizi povezij

Izbočne sile q_d na vodoravna povezja, ki podpirajo niz tlačnih pasov upogibnih nosilcev (slika 34), določimo z izrazom

$$q_d = \sum_{i=1}^{m} N_{Ed,i}\, 8\, \frac{(e_0 + \delta_q)}{L^2}$$

Če se sistem podprtih elementov in povezja računa po teoriji drugega reda, se δ_q v izrazu zanemari, nepopolnosti e_0 pa se upoštevajo z izbočno silo q_d. Da lahko odigrajo svojo vlogo, morajo biti povezja dovolj toga in δ_q naj ne bi presegal $L/500$ ali celo $L/1000$.



$$N_{Ed} = \frac{M_{Ed}}{h} = \frac{p_{Ed}L^2}{8h}$$

h – višina nosilcev

$$e_0 = \alpha_m \frac{L}{500}$$

$$\alpha_m = \sqrt{0,5\left(1 + \frac{1}{m}\right)}$$

m – število podprtih elementov

δ_q – upogibek povezja v lastni ravnini zaradi q_d in vseh drugih zunanjih obtežb (upogibov znanih po teoriji prvega reda)

Slika 34: Nadomestna izbočna sila pri vodoravnem povezju

$$q_d = \beta\, \frac{\sum_{i=1}^{m} N_{Ed,i}}{L} = \frac{1}{55,0} \cdot \frac{2,5 \cdot 111,50\ \text{kN} + 66,22\ \text{kN}}{11,70\ \text{m}} = 0,54\ \text{kN/m}$$

$$N_{Ed,2} = \frac{M_{Ed,2}}{h_2} = \frac{33,45\ \text{kNm}}{0,30\ \text{m}} = 111,50\ \text{kN}$$

$M_{Ed,2} = 35,92$ kNm moment v polju strešnega nosilca okvira IPE 300

$h_2 = 30$ cm višina strešnega nosilca okvira IPE 300

$m_2 = 2,5$ število podprtih elementov (notranji okviri, polovica obtežbe srednjega okvira gre na drugo povezje)

$$N_{Ed,1} = \frac{M_{Ed,1}}{h_1} = \frac{11,92\ \text{kNm}}{0,18\ \text{m}} = 66,22\ \text{kN}$$

$M_{Ed,1} = 11,92$ kNm moment v polju strešnega nosilca okvira IPE 180

$h_1 = 18$ cm višina strešnega nosilca okvira IPE 180

$$\beta = \frac{1}{55,0} \quad pri\ m = 3,5\ in\ \delta_q = \frac{1}{1500}$$

$$L = 11,70\ \text{m}$$

Izbočna sila v drugi (srednji) legi

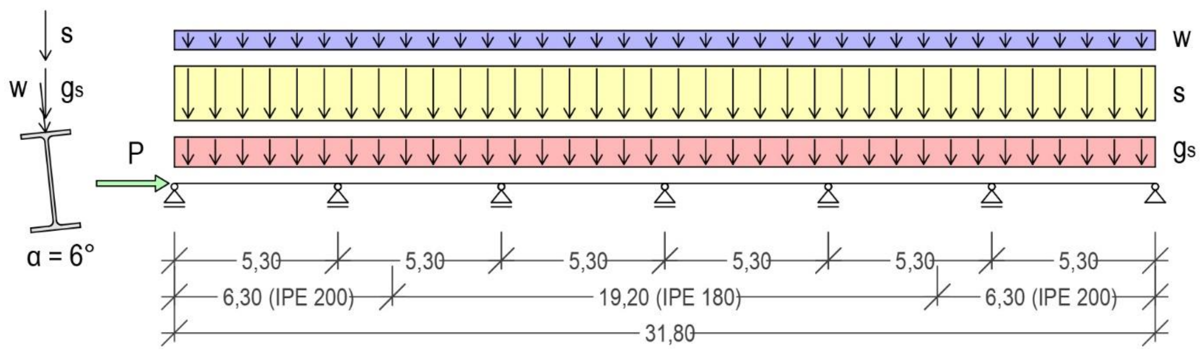
$$P = q_d\ b = 0,54\ \text{kN/m} \cdot 2,94\text{m} = 1,59\ \text{kN}$$

6.2.3 Obtežne kombinacije

Preglednica 16: Obtežne kombinacije za MSN in MSU

Kombinacija	**Lastna in stalna obtežba *G***	**Obtežba snega *S***	**Obtežba vetra *W***	**Izbočna sila *P***
MSN - Mejno stanje nosilnosti				
MSN1	1,35 · *G*	1,5 · *S*	1,5 · 0,6 · *W*	1,0 · *P*
MSN2	1,35 · *G*	1,5 · 0,5 · *S*	1,5 · *W*	1,0 · *P*
MSU - Mejno stanje uporabnosti				
MSU1	*G*	*S*	0,6 · *W*	1/1,4 · *P*
MSU2	*G*	0,5 · *S*	*W*	1/1,4 · *P*

Na sliki 35 je prikazana razporeditev vse obtežbe na strešni legi v programu SCIA Engineer.

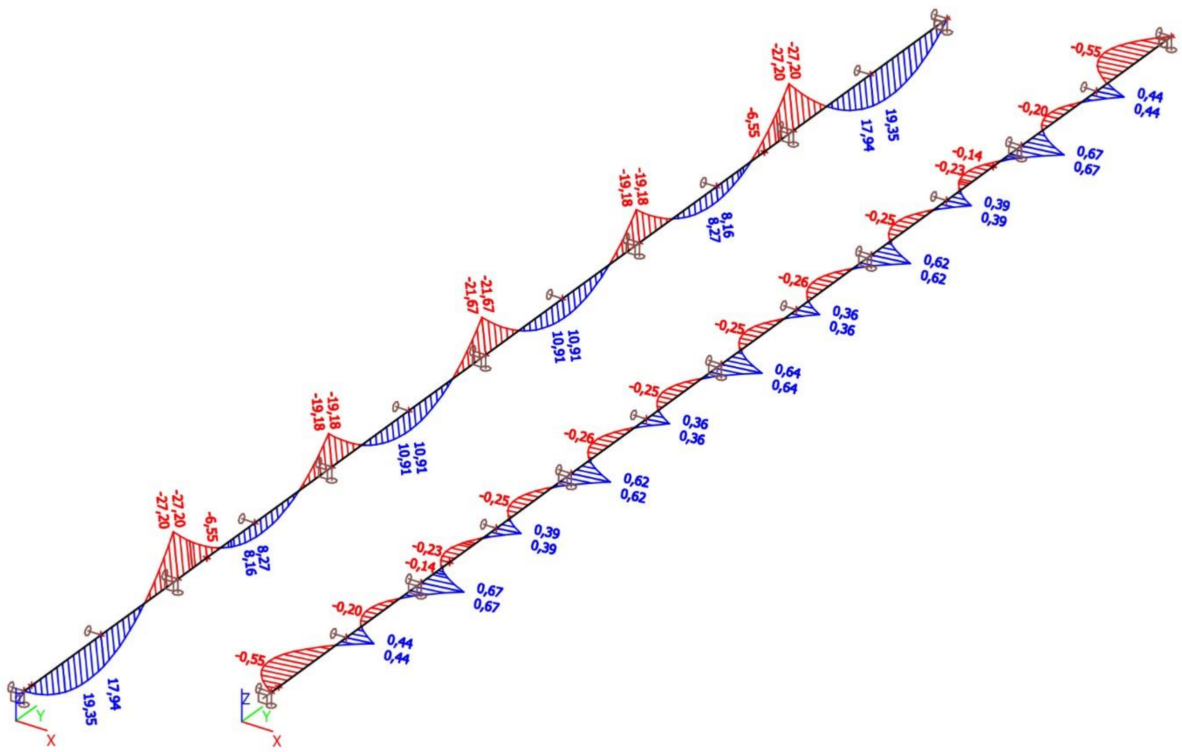


Slika 35: Obtežba na strešni legi

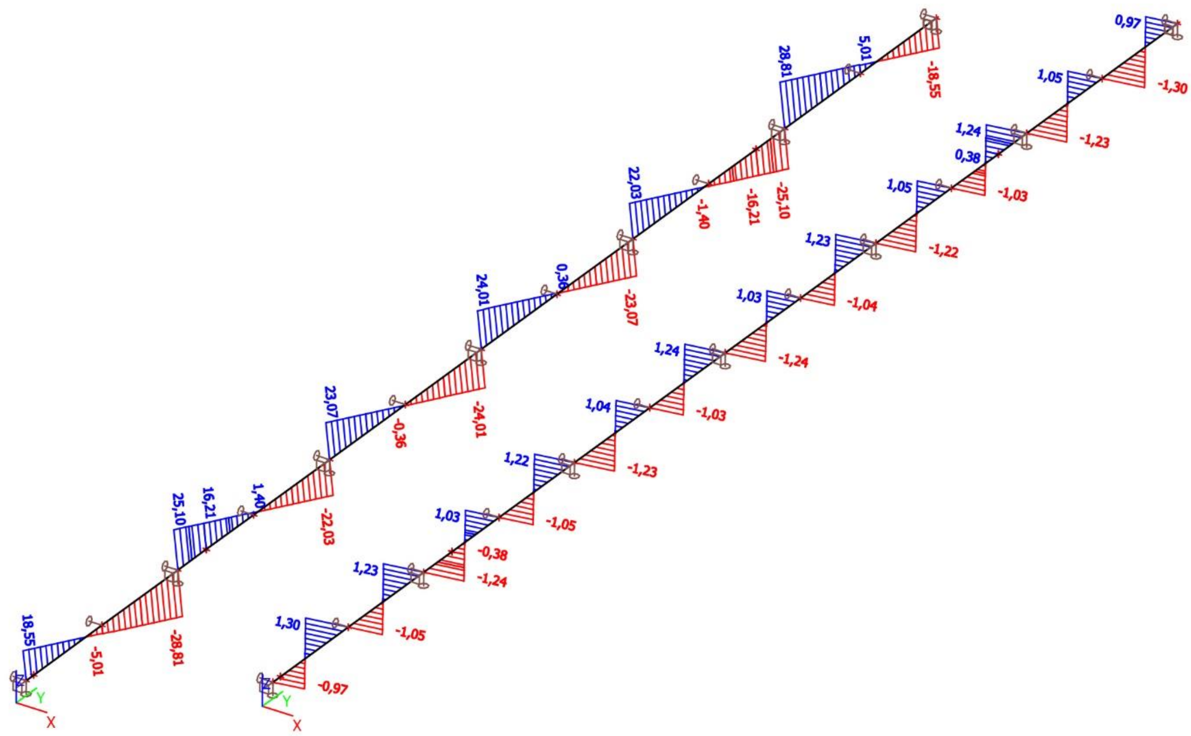
6.3 MSN - MEJNO STANJE NOSILNOSTI

6.3.1 Analiza strešne lege

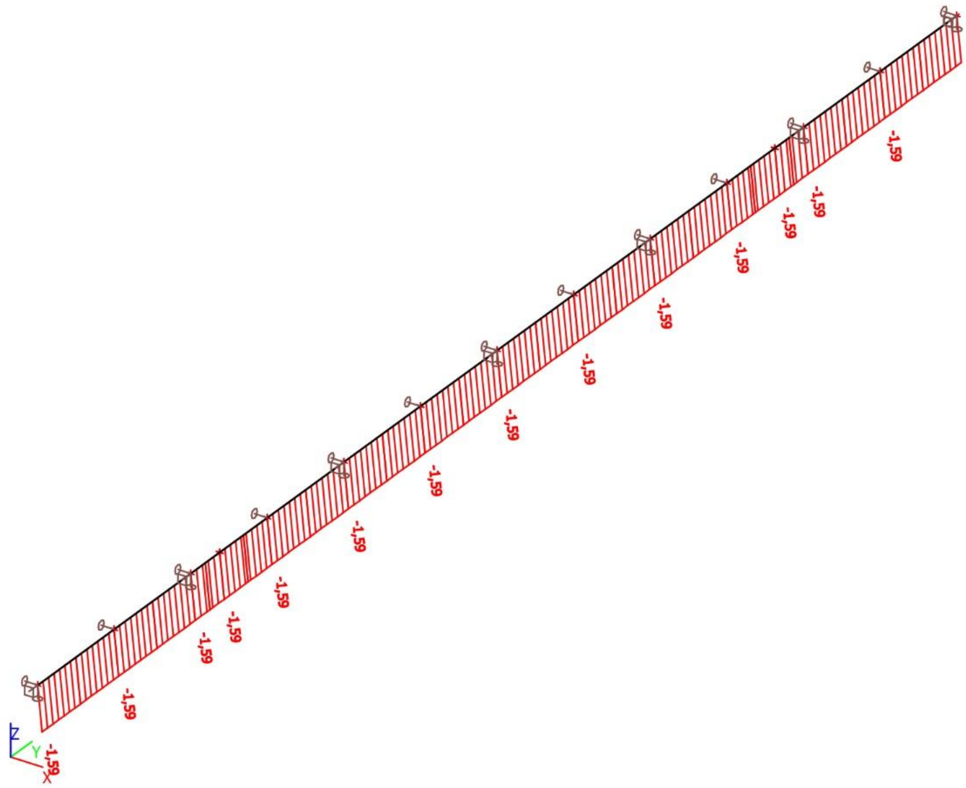
Kot merodajno obremenitev upoštevam ovojnico obtežnih kombinacij MSN. Rezultati iz programa SCIA Engineer so prikazani na slikah 36, 37 in 38.



Slika 36: MSN – Ovojnica momentov M_y in M_z [kNm]



Slika 37: MSN – Ovojnica prečnih sil V_z in V_y [kN]



Slika 38: MSN – Ovojnica osnih sil *N* [kN]

6.3.2 Dimenzioniranje strešne lege

Računske kontrole so izvedene s programom SCIA Engineer in so v prilogi (Priloga B – Kontrola nosilnosti in stabilnosti strešne lege). V nadaljevanju je prikazan povzetek izpisa.

6.3.2.1 Lega - IPE 200, S235, *L* = 6,30 m

Notranje količine (MSN1)

$$N_{Ed} = 1{,}59 \text{ kN (tlak)}$$

$$V_{y,Ed} = 0{,}82 \text{ kN}$$

$$V_{z,Ed} = 0{,}04 \text{ kN}$$

$$M_{y,Ed} = 19{,}35 \text{ kNm}$$

$$M_{z,Ed} = 0{,}16 \text{ kNm}$$

Razred kompaktnosti prereza

- stojina (notranji tlačeni del)

$$\frac{c}{t_w} = 28{,}39 \leq 71{,}36 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

- pasnica (previsni deli pasnic)

$$\frac{c}{t_f} = 4{,}14 \leq 9 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

Nosilnost prereza

- tlak

$$N_{c,Rd} = \frac{A\,f_y}{\gamma_{M0}} = 669{,}75\text{ kN} \geq N_{Ed} = 1{,}59\text{ kN} ✓$$

- upogib

$$M_{pl,y,Rd} = \frac{W_{pl,y}\,f_y}{\gamma_{M0}} = 51{,}94\text{ kNm} \geq M_{y,Ed} = 19{,}35\text{ kNm} ✓$$

$$M_{pl,z,Rd} = \frac{W_{pl,z}\,f_y}{\gamma_{M0}} = 10{,}48\text{ kNm} \geq M_{z,Ed} = 0{,}16\text{ kNm} ✓$$

- strig

$$V_{pl,y,Rd} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = 244{,}02\text{ kN} \geq V_{y,Ed} = 0{,}82\text{ kN} ✓$$

$$V_{pl,z,Rd} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = 190{,}17\text{ kN} \geq V_{z,Ed} = 0{,}04\text{ kN} ✓$$

- upogib, strig in osna sila

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1$$

$$0{,}14 + 0{,}02 = 0{,}16 \leq 1 ✓$$

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$$\frac{N_{Ed}}{N_{cr}} = 0{,}01 \leq 0{,}04$$

Vpliv uklona se lahko zanemari.

- bočna zvrnitev upogibno obremenjenih elementov

$$L_c = 2{,}65\text{ m}$$

$$M_{b,Rd} = \chi_{LT}\,W_{pl,y}\,\frac{f_y}{\gamma_{M1}} = 40{,}21\text{ kNm} \geq M_{y,Ed} = 19{,}35\text{ kNm} ✓$$

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\frac{\chi_y\,N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$0{,}00 + 0{,}68 + 0{,}06 = 0{,}74 \leq 1 ✓$$

$$\frac{N_{Ed}}{\frac{\chi_z\,N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$0{,}00 + 0{,}36 + 0{,}05 = 0{,}41 \leq 1 ✓$$

6.3.2.2 Lega - IPE 180, S235, *L* = 19,20 m

Notranje količine (MSN1)

$N_{Ed} = 1{,}59$ kN (tlak)

$V_{y,Ed} = 1{,}24$ kN

$V_{z,Ed} = 24{,}01$ kN

$M_{y,Ed} = 21{,}67$ kNm

$M_{z,Ed} = 0{,}64$ kNm

Razred kompaktnosti prereza

- stojina (notranji tlačeni del)

  $\frac{c}{t_w} = 27{,}55 \leq 71{,}26 \quad \rightarrow 1.\,\text{razred kompaktnosti}$

- pasnica (previsni deli pasnic)

  $\frac{c}{t_f} = 4{,}23 \leq 9 \quad \rightarrow 1.\,\text{razred kompaktnosti}$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

Nosilnost prereza

- tlak

  $$N_{c,Rd} = \frac{A\,f_y}{\gamma_{M0}} = 561{,}65\text{ kN} \geq N_{Ed} = 1{,}59\text{ kN} \checkmark$$

- upogib

  $$M_{pl,y,Rd} = \frac{W_{pl,y}\,f_y}{\gamma_{M0}} = 39{,}01\text{ kNm} \geq M_{y,Ed} = 21{,}67\text{ kNm} \checkmark$$

  $$M_{pl,z,Rd} = \frac{W_{pl,z}\,f_y}{\gamma_{M0}} = 8{,}13\text{ kNm} \geq M_{z,Ed} = 0{,}64\text{ kNm} \checkmark$$

- strig

  $$V_{pl,y,Rd} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = 207{,}83\text{ kN} \geq V_{y,Ed} = 24{,}01\text{ kN} \checkmark$$

  $$V_{pl,z,Rd} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = 152{,}01\text{ kN} \geq V_{z,Ed} = 0{,}64\text{ kN} \checkmark$$

- upogib, strig in osna sila

  $$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1$$

  $$0{,}31 + 0{,}08 = 0{,}39 \leq 1 \checkmark$$

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

  $$\frac{N_{Ed}}{N_{cr}} = 0{,}01 \leq 0{,}04$$

Vpliv uklona se lahko zanemari.

- bočna zvrnitev upogibno obremenjenih elementov

$$\frac{M_{Ed}}{M_{cr}} = 0{,}16 \leq \overline{\lambda_{LT,0}}^2 = 0{,}16$$

Vpliv bočne zvrnitve se lahko zanemari.

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

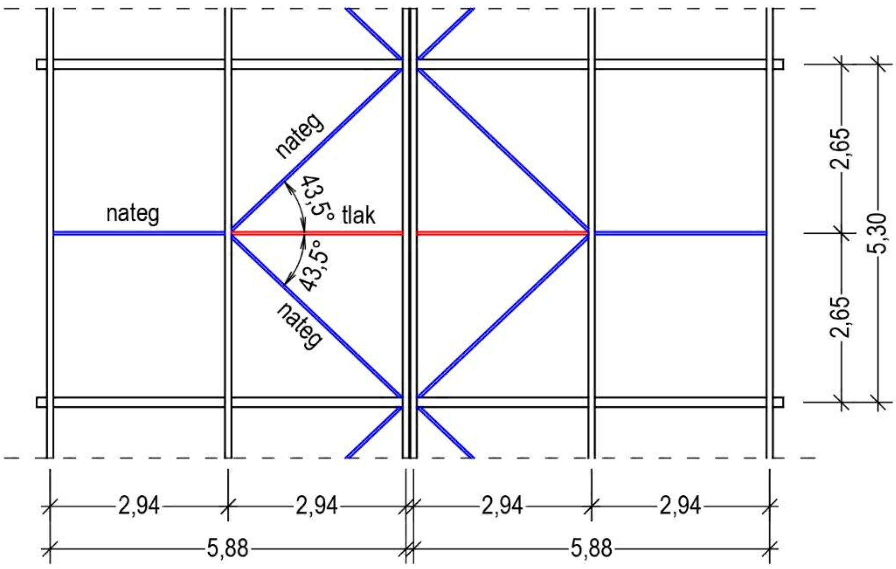
$0{,}00 + 0{,}56 + 0{,}08 = 0{,}64 \leq 1$ ✓

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

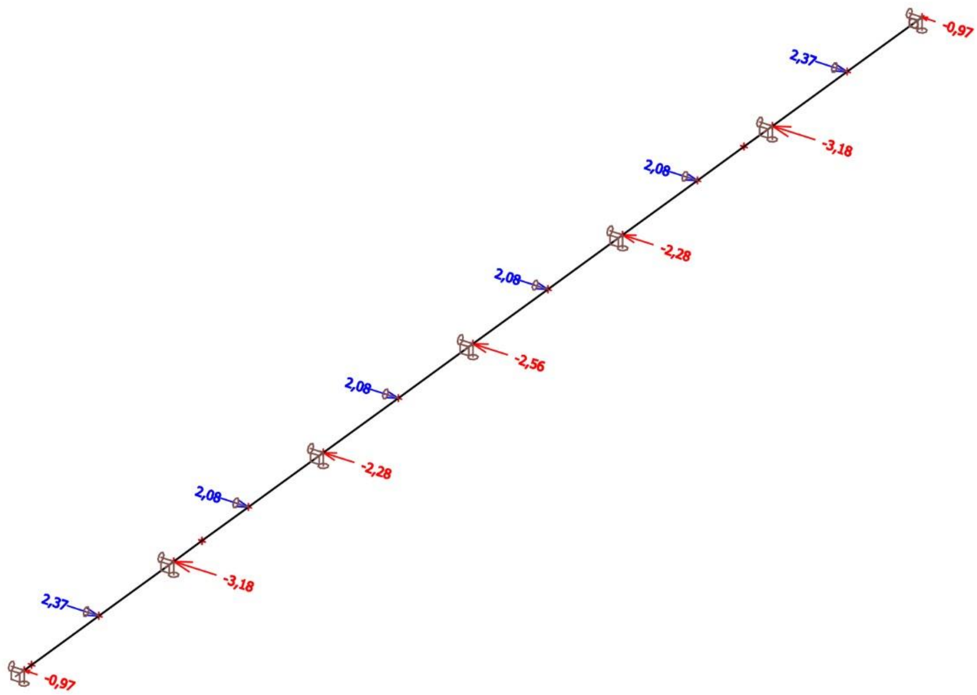
$0{,}00 + 0{,}29 + 0{,}08 = 0{,}37 \leq 1$ ✓

6.3.3 Dimenzioniranje zateg med strešmimi legami

Srednja lega je podprta proti bočni zvrnitvi preko dveh diagonal. Diagonali sta vpeti v momentna okvira. Spodnja in zgornja lega pa sta podprti proti bočni zvrnitvi preko nateznega in tlačenega elementa v srednjo lego.



Slika 39: Zasnova elementov zateg



Slika 40: Obremenitev zateg srednje strešne lege [kN]

6.3.3.1 Diagonalna natezna zatega

Sila, ki odpade na diagonalni zategi

$$R = 2{,}37 \text{ kN}$$

$$N_{Ed} = (n-1)R = (3-1) \cdot 2{,}37 \text{ kN} = 4{,}74 \text{ kN}$$

$n = 3$ število strešnih leg na polovici strehe (spodnja in zgornja lega prevzame polovico obtežbe)

Sila v eni diagonalni zategi (nateg)

$$N_{Ed,1} = \frac{N_{Ed}/2}{\cos\alpha} = \frac{2{,}37 \text{ kN}}{\cos 43{,}5°} = 3{,}27 \text{ kN}$$

Potrebni prerez natezne palice

$$N_{Ed} \leq N_{pl,Rd} = A\frac{f_y}{\gamma_{M0}}$$

$$A \geq N_{Ed,1}\frac{\gamma_{M0}}{f_y} = 3{,}27 \text{ kN} \cdot \frac{1{,}0}{23{,}5 \text{ kN/cm}^2} = 0{,}14 \text{ cm}^2$$

Izberem jekleno palico $\phi 12, A_{dej} = 1{,}13 \text{ cm}^2$

6.3.3.2 Natezna zatega

Sila, ki odpade na natezno zatego

$$R = 2{,}37 \text{ kN}$$

$$N_{Ed} = \frac{R}{2} = \frac{2{,}36 \text{ kN}}{2} = 1{,}19 \text{ kN}$$

Potrebni prerez natezne palice

$$A \geq N_{Ed} \frac{\gamma_{M0}}{f_y} = 1{,}19 \text{ kN} \cdot \frac{1{,}0}{23{,}5 \text{ kN/cm}^2} = 0{,}05 \text{ cm}^2$$

Izberem jekleno palico $\phi 12, A_{dej} = 1{,}13 \text{ cm}^2$

6.3.3.3 Tlačna zatega

Sila, ki odpade na tlačno zatego

$$R = 2{,}37 \text{ kN}$$

$$N_{Ed} = \frac{R}{2} = \frac{2{,}37 \text{ kN}}{2} = 1{,}19 \text{ kN}$$

Uklon tlačene palice

$$N_{Ed} \leq N_{b,Rd} = \chi A \frac{f_y}{\gamma_{M1}}$$

$$\bar{\lambda} \leq 3 \quad \rightarrow \quad \chi^c > 0{,}1$$

$$A \geq \frac{N_{Ed}\, \gamma_{M1}}{\chi\, f_y} = \frac{1{,}19 \text{ kN} \cdot 1{,}0}{0{,}1 \cdot 23{,}5 \text{ kN/cm}^2} = 0{,}50 \text{ cm}^2$$

Izberem jekleno cev $\phi 26{,}9$ mm; $t = 2{,}6$ mm; $A_{dej} = 1{,}98 \text{ cm}^2$

6.4 MSU - MEJNO STANJE UPORABNOSTI

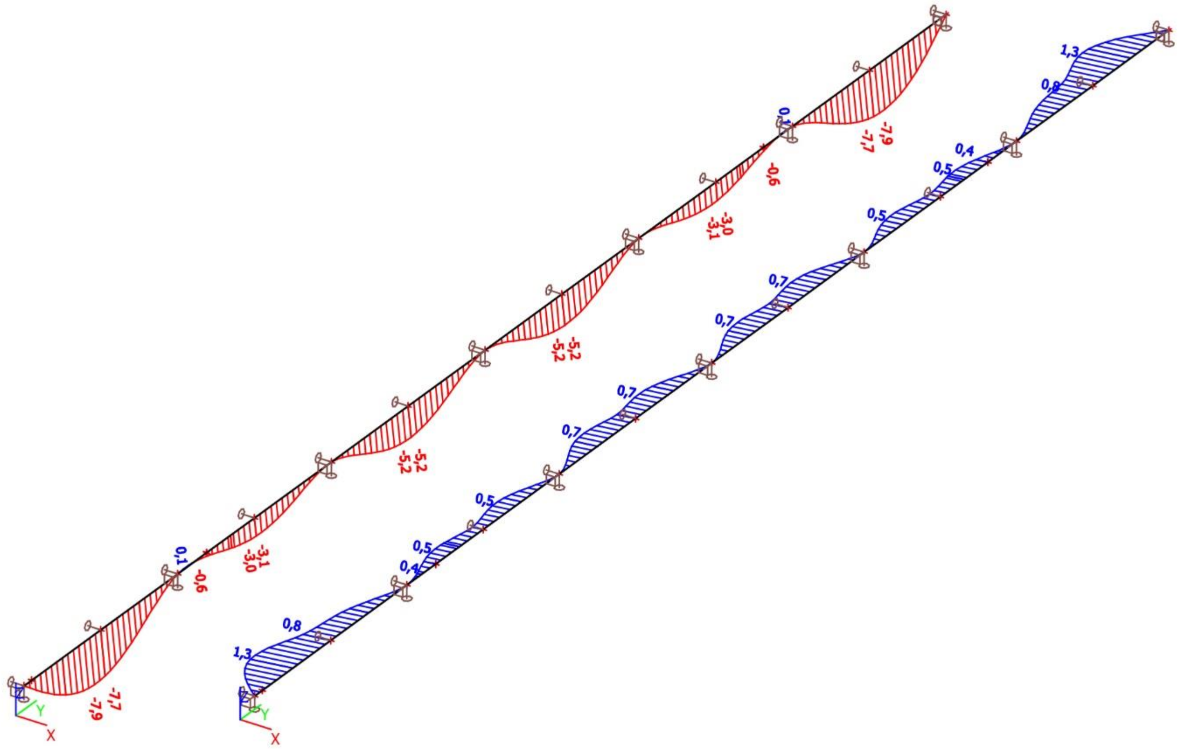
Kot merodajno obremenitev upoštevam ovojnico obtežnih kombinacij MSU. Pomiki strešne lege v lokalni Z in Y smeri so prikazani na sliki 41.

Kontrola vertikalnih pomikov strešne lege v lokalni Z smeri

$$w_{dop} = \frac{l_z}{250} = \frac{5300 \text{ mm}}{250} = 21{,}2 \text{ mm} \geq w = 7{,}9 \text{ mm}$$

Kontrola vertikalnih pomikov strešne lege v lokalni Y smeri

$$v_{dop} = \frac{l_y}{250} = \frac{2650 \text{ mm}}{250} = 10{,}6 \text{ mm} \geq v = 1{,}3 \text{ mm}$$

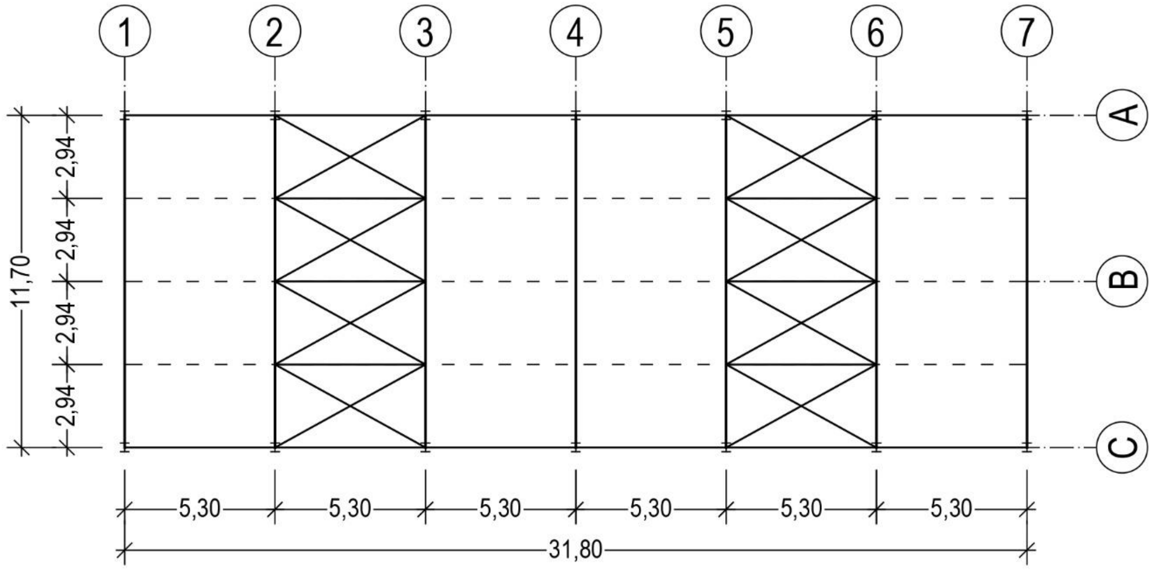


Slika 41: MSU - Ovojnica pomikov lege v lokalni Z in Y smeri [mm]

7 HORIZONTALNO POVEZJE

7.1 ZASNOVA

Horizontalno povezje v ravnini strehe je sestavljeno iz nateznih in tlačnih elementov. Prevzema vetrno obtežbo in izbočne sile momentnih okvirov.



Slika 42: Zasnova horizontalnega povezja - zavetrovanja

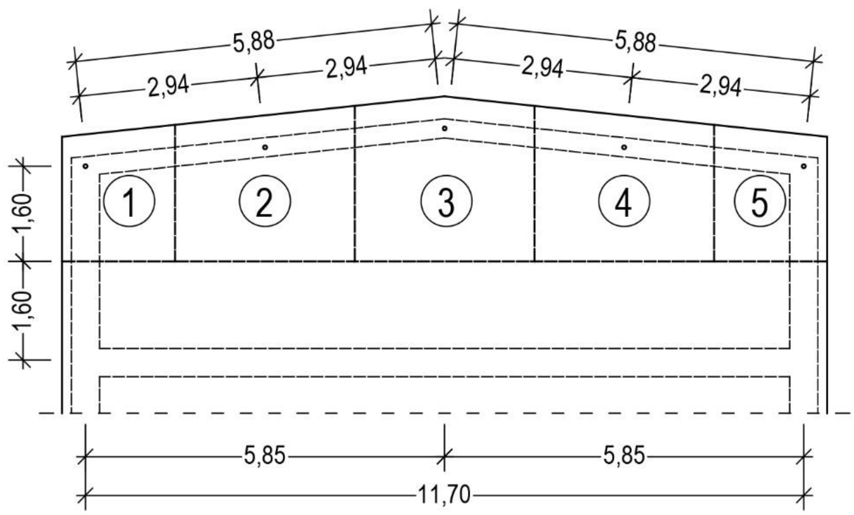
7.2 VPLIVI NA POVEZJE

7.2.1 Obtežba vetra

Obtežba vetra na steno pri delovanju vetra v smeri Y (poglavje 3.2.3.5, preglednica 9)

$$w = w_D = 0{,}59\,\mathrm{kN/m^2}$$

Na sliki 43 so prikazana vplivna območja obtežbe vetra, ki jih preko točkovnih sil prevzamejo natezni in tlačni elementi horizontalnega povezja.



Slika 43: Vplivna območja obtežbe vetra

Sile, ki se prenesejo na posamezna območja

$$F_{w,i} = w\,A_i$$

Preglednica 17: Točkovne sile vetra

Območje	***w*** [kN/m²]	***Aᵢ*** [m²]	***F_w,i*** [kN]
1	0,59	3,50	2,07
2	0,59	6,95	4,10
3	0,59	7,67	4,53
4	0,59	6,95	4,10
5	0,59	3,50	2,07

7.2.2 Izbočne sile momentnega okvira – lokalne nadomestne nepopolnosti

Izračun izbočne sile *P*, ki odpade na en tlačni element, je opisan v poglavju 6.2.2. *P* je izbočna sila 3,5 okvirov (polovica srednjega okvira 4 se prenese na drugo povezje).

$$P = 1{,}59 \text{ kN}$$

7.2.3 Obtežne kombinacije

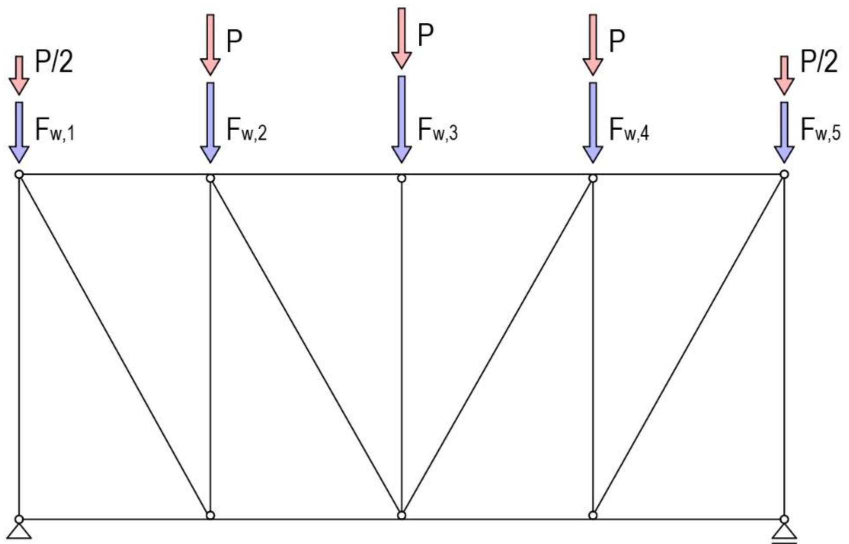
MSN

$$1{,}5\,F_w + 1{,}0\,P$$

MSU

$$1{,}0\,F_w + \frac{1}{1{,}4}\,P$$

Na sliki 44 je prikazan statični model in razporeditev vse obtežbe na povezju v programu SCIA Engineer.

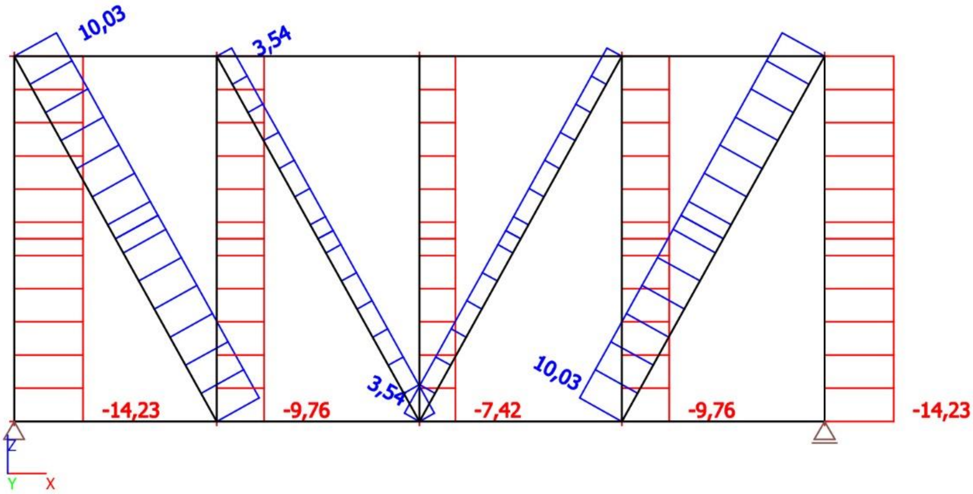


Slika 44: Razporeditev obtežbe na horizontalno povezje

7.3 MSN - MEJNO STANJE NOSILNOSTI

7.3.1 Analiza horizontalnega povezja

Rezultati iz programa SCIA Engineer so prikazani na sliki 45.



Slika 45: MSN – Osne sile v elementih povezja [kN]

7.3.2 Dimenzioniranje elementov povezja

7.3.2.1 Natezni element, S235

Maksimalno obremenjeni natezni element

$$N_{Ed} = 10{,}03 \text{ kN}$$

Potrebni prerez natezne palice

$$A \geq N_{Ed} \frac{\gamma_{M0}}{f_y} = 10{,}03 \text{ kN} \cdot \frac{1{,}0}{23{,}5 \text{ kN/cm}^2} = 0{,}43 \text{ cm}^2$$

Izberem jekleno palico $\phi 12, A_{dej} = 1{,}13 \text{ cm}^2$

7.3.2.2 Tlačni element, S235

Maksimalno obremenjeni tlačeni element

$$N_{Ed} = 14{,}23 \text{ kN}$$

Prerez tlačenega elementa

$$A \geq N_{Ed} \frac{\gamma_{M0}}{f_y} = 14{,}23 \text{ kN} \cdot \frac{1{,}0}{23{,}5 \text{ kN/cm}^2} = 0{,}61 \text{ cm}^2$$

Izberem jekleno cev $\phi 60{,}3$ mm; $t = 3{,}2$ mm; $A_{dej} = 5{,}74 \text{cm}^2$

Uklon tlačenega elementa

$$L_u = 530 \text{ cm}$$

$$N_{b,Rd} = \frac{\chi\, A\, f_y}{\gamma_{M1}} = \frac{0{,}12 \cdot 5{,}74\text{cm}^2 \cdot 23{,}5\,\text{kN/cm}^2}{1{,}0} = 16{,}19\ \text{kN} \geq N_{Ed} = 14{,}23\ \text{kN}\ \checkmark$$

$$uklonska\ krivulja\ a\ \longrightarrow\ \chi = 0{,}12$$

$$\bar{\lambda} = \frac{L_u}{i\,\lambda_1} = \frac{530\ \text{cm}}{2{,}023\ \text{cm} \cdot 93{,}9} = 2{,}79$$

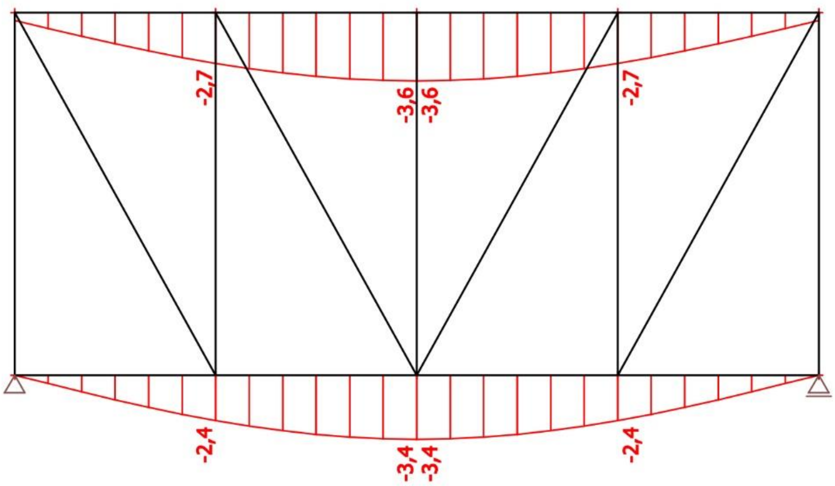
$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{23{,}5\ \text{cm}^4}{5{,}74\ \text{cm}^2}} = 2{,}023\ \text{cm}$$

$$I = 23{,}5\ \text{cm}^4$$

$$\lambda_1 = 93{,}9 \cdot \varepsilon = 93{,}9$$

7.4 MSU - MEJNO STANJE UPORABNOSTI

Kot merodajno obremenitev upoštevam kombinacijo MSU. Pomiki strešnega nosilca okvira so prikazani na sliki 46.



Slika 46: MSU – Pomik strešnega nosilca okvira [mm]

Kontrola horizontalnega predpostavljenega pomika strešnega nosilca okvira pri izračunu izbočne sile momentnega okvira (poglavje 6.2.2)

$$\delta_q = \frac{L}{1500} = \frac{1170\ \text{cm}}{1500} = 0{,}78\ \text{cm} \geq\ \delta = 0{,}36\ \text{cm}$$

8 POTRESNO PROJEKTNO STANJE

Kombinacija vplivov za potresno projektno stanje (SIST EN 1990, poglavje 6.4.3.4)

$$G_k + \gamma_I \cdot A_{Ed} + \psi_2 \cdot Q_k$$

kjer so

$\gamma_I \cdot A_{Ed}$ potresni vpliv (potresne sile)

$G_k + \psi_2 \cdot Q_k$ nepotresni vliv (gravitacijske sile)

γ_I faktor pomembnosti stavbe (SIST EN 1998-1, poglavje 4.2.5)

ψ_2 koeficient za spremenljive vplive (SIST EN 1998-1, poglavje 4.2.4)

8.1 GLOBALNA ANALIZA

Konstrukcija je pravilna v tlorisu in po višini. Dovoljena poenostavitev po SIST EN 1998-1, poglavje 4.2.3:

- Ravninski model (omogočena obravnava potresa za vsako smer posebej)
- Metoda vodoravnih sil (linearno elastična analiza »Base Shear«)
- Faktor obnašanja $q = q_{ref}$ (ni redukcije q faktorja zaradi neregularnosti konstrukcije).

Izpolnjen je tudi pogoj SIST EN 1998-1, poglavje 4.3.3.2.1, omejitev osnovnih nihajnih časov v obeh glavnih smereh X in Y (glej račun nihajnih časov):

$$T_1 \leq \begin{cases} 4T_c \\ 2{,}0 \text{ s} \end{cases}$$

Po Beg, Pogačnik, 2009, sta za projektiranje potresno odpornih stavb predvidena dva načina projektiranja (preglednica 18), vezana na:

a) konstrukcije z majhnim sipanjem energije,
b) konstrukcije s sposobnostjo sipanja energije.

Preglednica 18: Načini projektiranja jeklenih konstrukcij

Načini projektiranja	**Stopnja duktilnosti konstrukcij**	**Območja referenčnih vrednosti faktorjev obnašanja *q***
a) konstrukcije z majhnim sipanjem energije	DCL (nizka)	$q \leq 1{,}5$ vsi razredi kompaktnosti prečnih prerezov
b) konstrukcije s sposobnostjo sipanja energije	DCM (srednja)	$1{,}5 < q \leq 2$ 1., 2. ali 3. razred kompaktnosti prečnih prer.
		$2 < q \leq 4$ 1. ali 2. razred kompaktnosti prečnih prerezov
	DCH (visoka)	$4 < q \leq 8$ 1. razred kompaktnosti prečnih prerezov

Pri načinu a) se lahko notranje sile izračunajo z elastično globalno analizo, dimenzioniranje pa se v celoti opravi v skladu s standardi iz skupine Evrokod 3. Tak pristop se priporoča le za področja z nizko seizmičnostjo ($a_g \leq 0{,}08\ g$, $a_g\ S \leq 0{,}1\ g$). Takih področij v Sloveniji ni, vendar se način a) brez večjih pomislekov lahko uporabi za enoetažne stavbe in za geometrijsko pravilne večetažne stavbe. Pri zahtevnejših konstrukcijah (izrazita geometrijska nepravilnost, izjemno velike mase, velika višina) je uporaba vprašljiva in se priporoča upoštevanje načel metode načrtovanja nosilnosti vsaj za najbolj izpostavljene konstrukcijske elemente.

Pri načinu b) je treba določiti območja sipanja energije, kjer se bo konstrukcija obnašala neelastično in kjer je treba zagotoviti zadostno duktilnost in nosilnost.

V diplomski nalogi sta predstavljena oba načina, in sicer

- Poglavje 8.2 Konstrukcije z majhnim sipanjem energije, način a)
- Poglavje 8.3 Primer konstrukcije s sposobnostjo sipanja energije, način b)

Na koncu poglavja je primerjava dimenzioniranih konstrukcijskih elementov na oba načina.

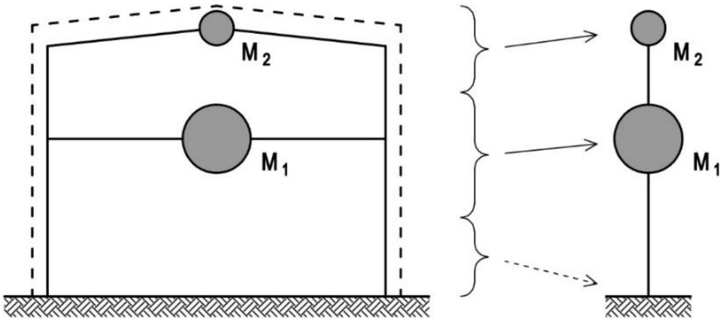
8.2 KONSTRUKCIJE Z MAJHNIM SIPANJEM ENERGIJE

Prereze vseh stebrov notranjih okvirov povečamo iz HEA 450 na HEA 500 in zunanje (krajne) sekundarne nosilce medetaže iz IPE 220 na IPE 240 (glej dimenzioniranje elementov).

8.2.1 Izračun mas

Kombinacija vplivov za potresno projektno stanje (SIST EN 1998-1, poglavje 3.2.4)

$$\sum G_{k,j} + \sum \psi_{E,i} \cdot Q_{k,i} \quad \text{kjer je} \quad \psi_{E,i} = \varphi \cdot \psi_{2,i}$$



Slika 47: Razporeditev mas po etažah in računski model

Preglednica 19: Izračun mase objekta po etažah

G $G_{k,1}$ **– lastna in stalna teža medetaže**				
	[kN/m]	[m]	kosov	[kN]
Steber HEA 500	1,521	4,32	10	65,71
Steber HEA 220	0,495	4,32	4	8,55
Steber HEA 160	0,298	2,72	2	1,62
Steber IPE 140	0,127	1,60	2	0,41
Prečka HEA 450	1,373	11,70	5	80,32
Prečka HEA 220	0,495	11,70	2	11,58
Sekundarni nosilci IPE 220	0,257	5,30	24	32,69
Sekundarni nosilci IPE 240	0,301	5,30	12	19,14
Vertikalno povezje, čelne pločevine, vijaki, ...	Ocena 10% teže			22,40
	[kN/m^2]	[m^2]		[kN]
Stalna teža (AB plošča, parket, estrih, ...)	4,480	372		1666,56
Fasada FTV 150 (bočna in čelna)	0,291	387		112,62
	[kN/100]		kosov	[kN]
Čepi Ø19	0,236		1296	3,06
			$G_{k,1}$ **[kN]**	**2.028,60**
			[kg]	**206.798**
$Q_{k,1}$ **– koristna teža medetaže**				
	[kN/m^2]	[m^2]		[kN]
Koristna teža (pisarne, predelne stene, ...)	4,300	372		1599,60
			$Q_{k,1}$ **[kN]**	**1.599,60**
			[kg]	**163.058**

$G_{k,2}$ – lastna in stalna teža strehe				
	[kN/m]	[m]	kosov	[kN]
Steber HEA 500	1,521	1,60	10	24,34
Steber HEA 220	0,495	1,60	4	3,17
Steber IPE 140	0,127	2,21	2	0,56
Strešni nosilec IPE 300	0,414	5,88	10	24,34
Strešni nosilec IPE 180	0,184	5,88	4	4,33
Strešna lega IPE 180	0,184	19,20	6	21,20
Strešna lega IPE 200	0,220	6,30	12	16,63
Vertikalno in horizontalno povezje, zatege, vijaki, ...	Ocena 10% teže			9,60
	[kN/m^2]	[m^2]		[kN]
Stalna teža (spuščen strop, inštalacije, ...)	0,300	372		111,60
Fasada FTV 150 (bočna in čelna)	0,291	191		55,58
Fasada SNV 150 (streha)	0,291	402		116,98
			$G_{k,1}$ [kN]	**389,80**
			[kg]	**39.735**

Teža 1. etaže

$$F_{M1} = G_{k,1} + \varphi \cdot \psi_{2,1} \cdot Q_{k,1} = 2028{,}60 \text{ kN} + 0{,}8 \cdot 0{,}3 \cdot 1599{,}60 \text{ kN} = 2412{,}50 \text{ kN}$$

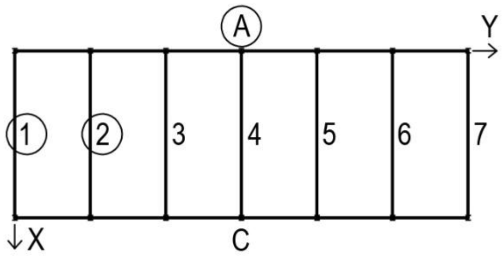
$\varphi = 0{,}8$ kategorija A do C: zasedba nekaterih etaž je povezana

$\psi_{2,1} = 0{,}3$ kategorija B: pisarne

Teža 2. etaže - strehe

$$F_{M2} = G_{k,2} = 389{,}80 \text{ kN}$$

Delež etažnih mas za okvir 1 in 2 ter okvir A



Slika 48: Tloris konstrukcije z označenimi deli za ravninsko analizo potresa

Prečna smer X (prvi in sedmi okvir sta obremenjena polovično)

$$F_{M1}^{(2)} = \frac{F_{M1}}{6} = \frac{2412{,}50 \text{ kN}}{6} = 402{,}08 \text{ kN} \quad (40.987 \text{ kg})$$

$$F_{M1}^{(1)} = \frac{F_{M1}^{(2)}}{2} = \frac{402{,}08 \text{ kN}}{2} = 201{,}04 \text{ kN} \quad (20.494 \text{ kg})$$

$$F_{M2}^{(2)} = \frac{F_{M2}}{6} = \frac{389{,}80 \text{ kN}}{6} = 64{,}97 \text{ kN} \quad (6.622 \text{ kg})$$

$$F_{M2}^{(1)} = \frac{F_{M2}^{(2)}}{2} = \frac{64{,}97 \text{ kN}}{2} = 32{,}49 \text{ kN} \quad (3.311 \text{ kg})$$

Vzdolžna smer Y

$$F_{M1}^{(A)} = \frac{F_{M1}}{2} = \frac{2412{,}50 \text{ kN}}{2} = 1206{,}25 \text{ kN} \quad (122.961 \text{ kg})$$

$$F_{M2}^{(A)} = \frac{F_{M2}}{2} = \frac{389{,}80 \text{ kN}}{2} = 194{,}90 \text{ kN} \quad (19.867 \text{ kg})$$

8.2.2 Faktor obnašanja *q*

Prečna smer X:

- momentni okvir (notranji), okvir s centričnim povezjem (zunanji)
- prečni prerezi so v 1., 2., 3. ali 4. razredu kompaktnosti
- razred nizke duktilnosti (DCL): $q \leq 1{,}5$

$q_X = 1{,}5$

Vzdolžna smer Y:

- okvir s centričnim povezjem
- prečni prerezi so v 1., 2., 3. ali 4. razredu kompaktnosti
- razred nizke duktilnosti (DCL): $q \leq 1{,}5$

$q_Y = 1{,}5$

8.2.3 Nihajni časi

Osnovni nihajni časi T_1 vseh ravninskih modelov so določeni v SCIA Engineer

$T_{1,X}^{(2)} = 0{,}81$ s notranji momentni okvir 2

$T_{1,X}^{(1)} = 0{,}20$ s zunanji okvir 1 s centričnim povezjem

$T_{1,Y} = 0{,}70$ s vzdolžni okvir A s centričnim povezjem

8.2.4 Spekter pospeškov

Objekt se nahaja v naselju Grosuplje. Podatke o pospeških temeljnih tal dobimo iz karte potresne nevarnosti Slovenije, veljavne v času uporabe standarda.

Projektni pospešek tal

$$a_g = 0{,}200\, g$$

Tip tal C

$$S = 1{,}15$$

$$T_B = 0{,}20 \text{ s}$$

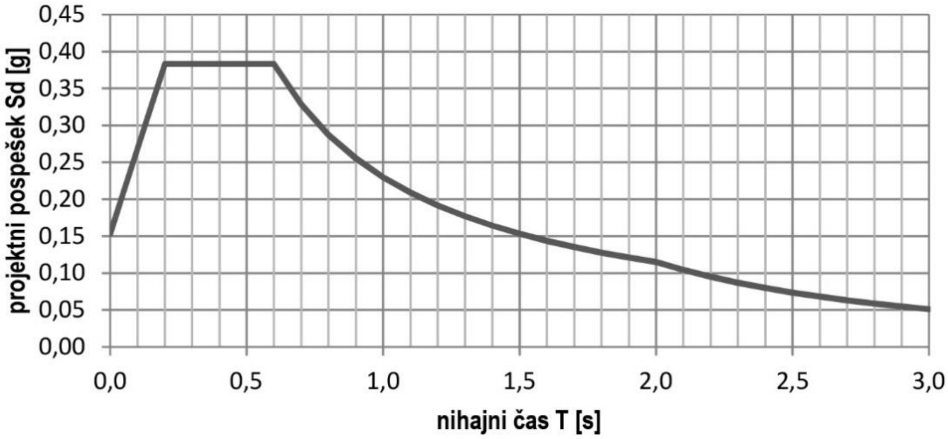
$$T_C = 0{,}60 \text{ s}$$

$$T_D = 2{,}00 \text{ s}$$

Na sliki 49 so prikazana območja nihajnih časov.

$$T_B \leq T_{1,X}^{(1)} \leq T_C$$

$$T_C \leq T_{1,X}^{(2)}, T_{1,Y} \leq T_D$$



Slika 49: Projektni spekter, faktor obnašanja q = 1,5

Projektni pospešek v smeri X, okvir 2

$$S_{d,X}^{(2)} = a_g\ S\ \frac{2{,}5}{q_X} \left[\frac{T_C}{T_{1,X}^{(2)}}\right] \geq \beta\ a_g = 0{,}2 \cdot 0{,}2\ g = 0{,}04\ g$$

$$S_{d,X}^{(2)} = 0{,}2\ g \cdot 1{,}15 \cdot \frac{2{,}5}{1{,}5} \cdot \left[\frac{0{,}60}{0{,}81}\right] = 0{,}284\ g = p_{X,2}\ g$$

Projektni pospešek v smeri X, okvir 1

$$S_{d,X}^{(1)} = a_g\ S\ \frac{2{,}5}{q_X} = 0{,}2\ g \cdot 1{,}15 \cdot \frac{2{,}5}{1{,}5} = 0{,}383\ g = p_{X,1}\ g$$

Projektni pospešek v smeri Y

$$S_{d,Y} = a_g\ S\ \frac{2{,}5}{q_Y} \left[\frac{T_C}{T_{1,Y}}\right] \geq \beta\ a_g = 0{,}2 \cdot 0{,}2\ g = 0{,}04\ g$$

$$S_{d,X} = 0{,}2\ g \cdot 1{,}15 \cdot \frac{2{,}5}{1{,}5} \cdot \left[\frac{0{,}60}{0{,}70}\right] = 0{,}329\ g = p_Y\ g$$

8.2.5 Celotna prečna sila (»Base Shear«) posameznega okvira

Celotna prečna sila F_b na mestu vpetja konstrukcije (SIST EN 1998-1, poglavje 4.3.3.2.2)

$$F_{b,i} = S_{d,i}(T_{1,i})\ m_i\ \lambda = p_i\ g\ m_i\ \lambda = p_i\ F_{Mi}\ \lambda$$

$$\lambda = 1{,}0$$

Prečni okvir 2

$$F_{b,X}^{(2)} = p_{X,2} \cdot \left(F_{M1}^{(2)} + F_{M2}^{(2)}\right) = 0{,}284 \cdot (402{,}08\ \text{kN} + 64{,}97\ \text{kN}) = 132{,}64\ \text{kN}$$

Prečni okvir 1

$$F_{b,X}^{(1)} = p_{X,1} \cdot \left(F_{M1}^{(1)} + F_{M2}^{(1)}\right) = 0{,}383 \cdot (201{,}04\ \text{kN} + 32{,}49\ \text{kN}) = 89{,}44\ \text{kN}$$

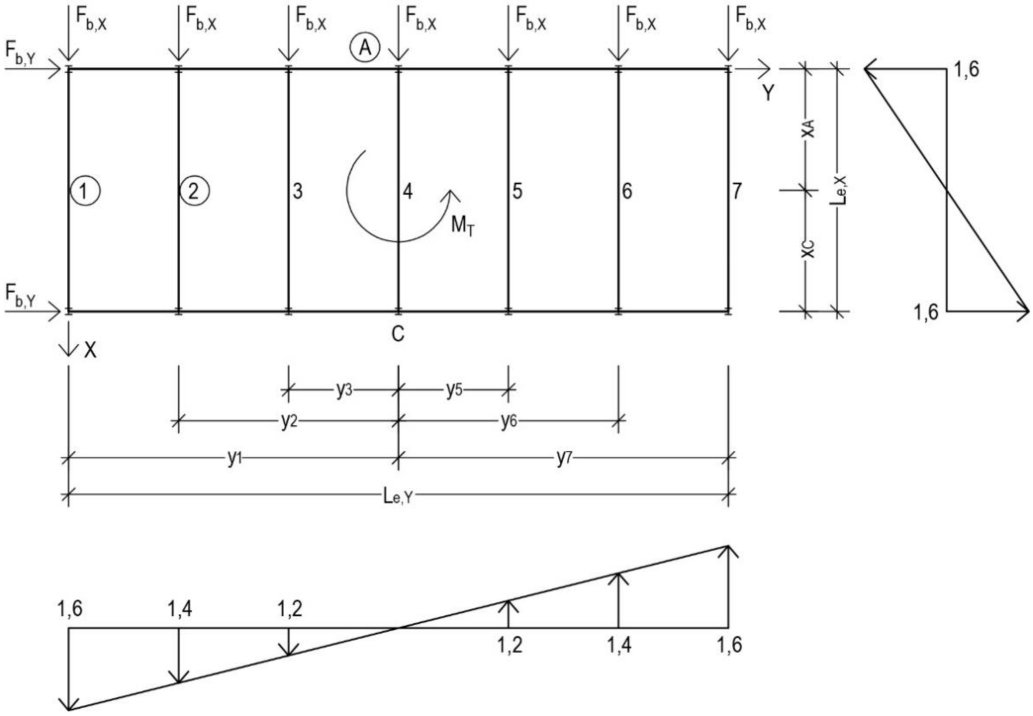
Vzdolžni okvir A

$$F_{b,Y} = p_Y \cdot \left(F_{M1}^{(A)} + F_{M2}^{(A)}\right) = 0{,}329 \cdot (1206{,}25\ \text{kN} + 194{,}90\ \text{kN}) = 460{,}98\ \text{kN}$$

8.2.6 Torzijski vpliv

Vpliv naključne torzije δ (SIST EN 1998-1, poglavje 4.3.3.2.4)

$$\delta = 1 + 1{,}2 \cdot \frac{x_i}{L_{e,i}} = \left(1 + 1{,}2 \cdot \frac{y_i}{L_{e,i}}\right)$$



Slika 50: Torzijski vpliv - analiza z dvema ravninskima modeloma

Smer X, okvir 2

$$\delta_{2,6} = 1 + 1{,}2 \cdot \frac{y_{2,6}}{L_{e,Y}} = 1 + 1{,}2 \cdot \frac{L_{e,Y}/3}{L_{e,Y}} = 1{,}4$$

Smer X, okvir 1

$$\delta_{1,7} = 1 + 1{,}2 \cdot \frac{y_{1,7}}{L_{e,Y}} = 1 + 1{,}2 \cdot \frac{L_{e,Y}/2}{L_{e,Y}} = 1{,}6$$

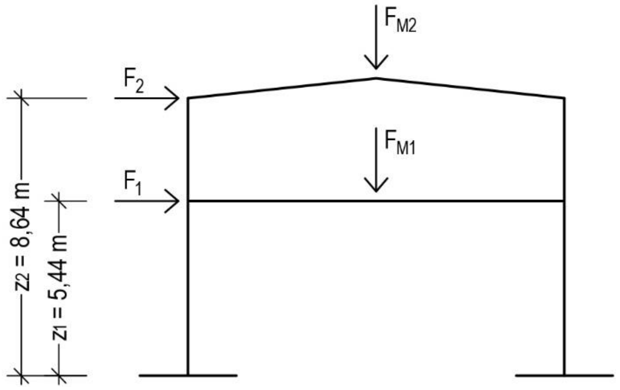
Smer Y, okvir A

$$\delta_{A,C} = 1 + 1{,}2 \cdot \frac{x_{A,C}}{L_{e,X}} = 1 + 1{,}2 \cdot \frac{L_{e,X}/2}{L_{e,X}} = 1{,}6$$

8.2.7 Razdelitev sil po višini

Vodoravne sile F_i v etažah (SIST EN 1998-1, poglavje 4.3.3.2.3).

$$F_i = F_b \cdot \frac{z_i \cdot F_{Mi}}{\sum z_j \cdot F_{Mj}}$$



Slika 51: Razdelitev sil po višini

Sile za okvir 2

$$F_1^{(2)} = F_{b,X}^{(2)} \cdot \frac{z_1 \cdot F_{M1}^{(2)}}{z_1 \cdot F_{M1}^{(2)} + z_2 \cdot F_{M2}^{(2)}} = 132{,}64 \cdot \frac{5{,}44 \cdot 402{,}08}{5{,}44 \cdot 402{,}08 + 8{,}64 \cdot 64{,}97} = 105{,}55 \text{ kN}$$

$$F_2^{(2)} = F_{b,X}^{(2)} \cdot \frac{z_2 \cdot F_{M2}^{(2)}}{z_1 \cdot F_{M1}^{(2)} + z_2 \cdot F_{M2}^{(2)}} = 132{,}64 \cdot \frac{8{,}64 \cdot 64{,}97}{5{,}44 \cdot 402{,}08 + 8{,}64 \cdot 64{,}97} = 27{,}09 \text{ kN}$$

Sile za okvir 1

$$F_1^{(1)} = F_{b,X}^{(1)} \cdot \frac{z_1 \cdot F_{M1}^{(1)}}{z_1 \cdot F_{M1}^{(1)} + z_2 \cdot F_{M2}^{(1)}} = 89{,}44 \cdot \frac{5{,}44 \cdot 201{,}04}{5{,}44 \cdot 201{,}04 + 8{,}64 \cdot 32{,}49} = 71{,}17 \text{ kN}$$

$$F_2^{(1)} = F_{b,X}^{(1)} \cdot \frac{z_2 \cdot F_{M2}^{(1)}}{z_1 \cdot F_{M1}^{(1)} + z_2 \cdot F_{M2}^{(1)}} = 89{,}44 \cdot \frac{8{,}64 \cdot 32{,}49}{5{,}44 \cdot 201{,}04 + 8{,}64 \cdot 32{,}49} = 18{,}27 \text{ kN}$$

Sile za okvir A

$$F_1^{(A)} = F_{b,Y} \cdot \frac{z_1 \cdot F_{M1}^{(A)}}{z_1 \cdot F_{M1}^{(A)} + z_2 \cdot F_{M2}^{(A)}} = 460{,}98 \cdot \frac{5{,}44 \cdot 1206{,}25}{5{,}44 \cdot 1206{,}25 + 8{,}64 \cdot 194{,}90} = 366{,}84 \text{ kN}$$

$$F_2^{(A)} = F_{b,Y} \cdot \frac{z_2 \cdot F_{M2}^{(A)}}{z_1 \cdot F_{M1}^{(A)} + z_2 \cdot F_{M2}^{(A)}} = 460{,}98 \cdot \frac{8{,}64 \cdot 194{,}90}{5{,}44 \cdot 1206{,}25 + 8{,}64 \cdot 194{,}90} = 94{,}14 \text{ kN}$$

8.2.8 Kontrola nosilnosti v prečni smeri – momentni okvir 2

8.2.8.1 Gravitacijski del obtežbe

$$G_k + \psi_2 \cdot Q_k$$

$$\psi_2 = \varphi \cdot \psi_{2,1} = 0{,}8 \cdot 0{,}3 = 0{,}24$$

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija $g_m =$ 24,93 kN/m
- streha $g_s =$ 3,18 kN/m
- fasada $g_f =$ 1,59 kN/m

Spremenljiva obtežba

- koristna obtežba medetaže $q_m =$ 22,79 kN/m

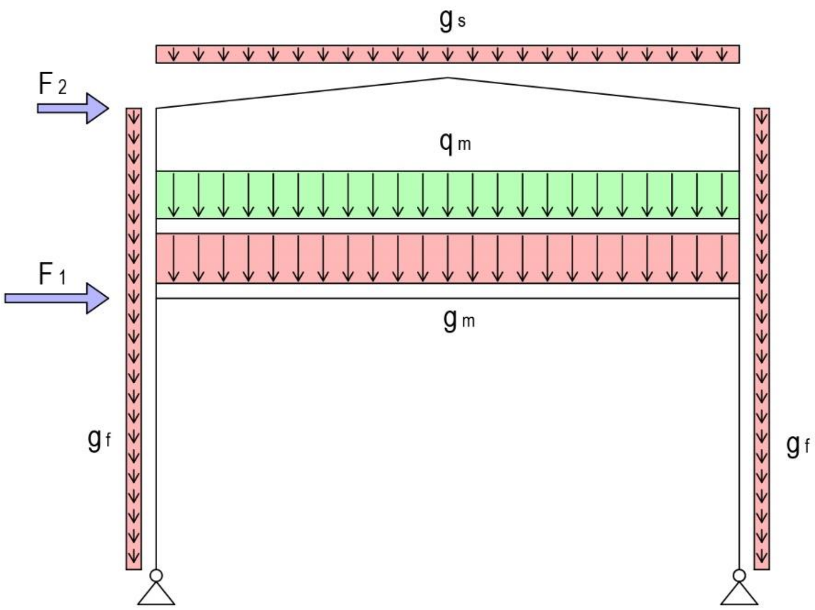
8.2.8.2 Potresni del obtežbe

$$F_1 = \gamma_I \cdot F_1^{(2)} \cdot \delta_{2,6} = 1{,}0 \cdot 105{,}55 \text{ kN} \cdot 1{,}4 = 147{,}77 \text{ kN}$$

$$F_2 = \gamma_I \cdot F_2^{(2)} \cdot \delta_{2,6} = 1{,}0 \cdot 27{,}09 \text{ kN} \cdot 1{,}4 = 37{,}93 \text{ kN}$$

$\gamma_I = 1{,}0$ faktor pomembnosti stavbe (kategorija II - običajne stavbe)

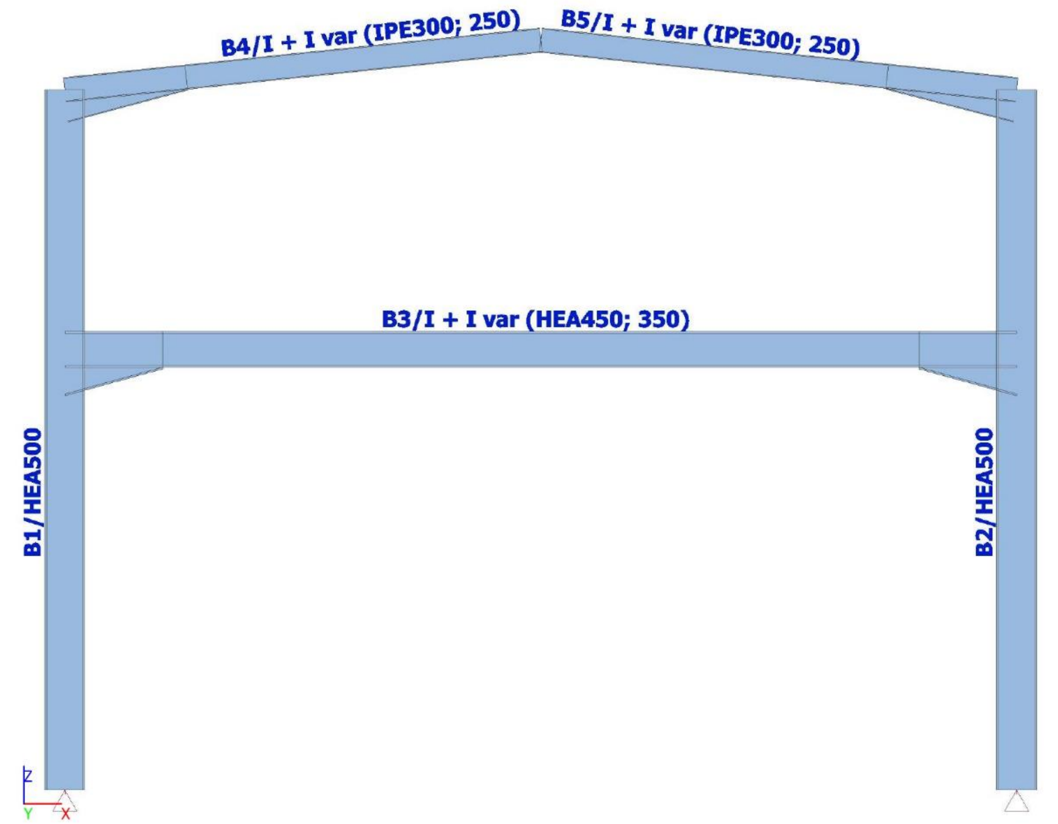
Na sliki 52 je prikazana razporeditev gravitacijske in potresne obtežbe na okvir 2 v programu SCIA Engineer.



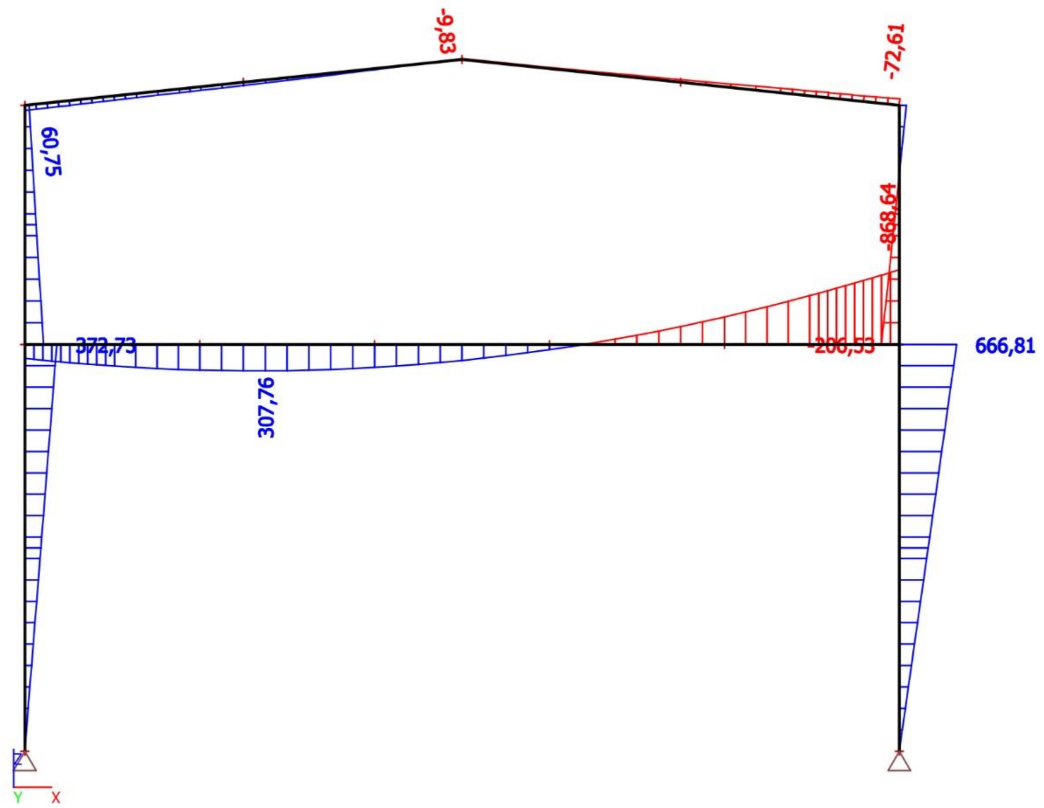
Slika 52: Obtežba na momentnem okviru 2

8.2.8.3 Analiza momentnega okvira 2

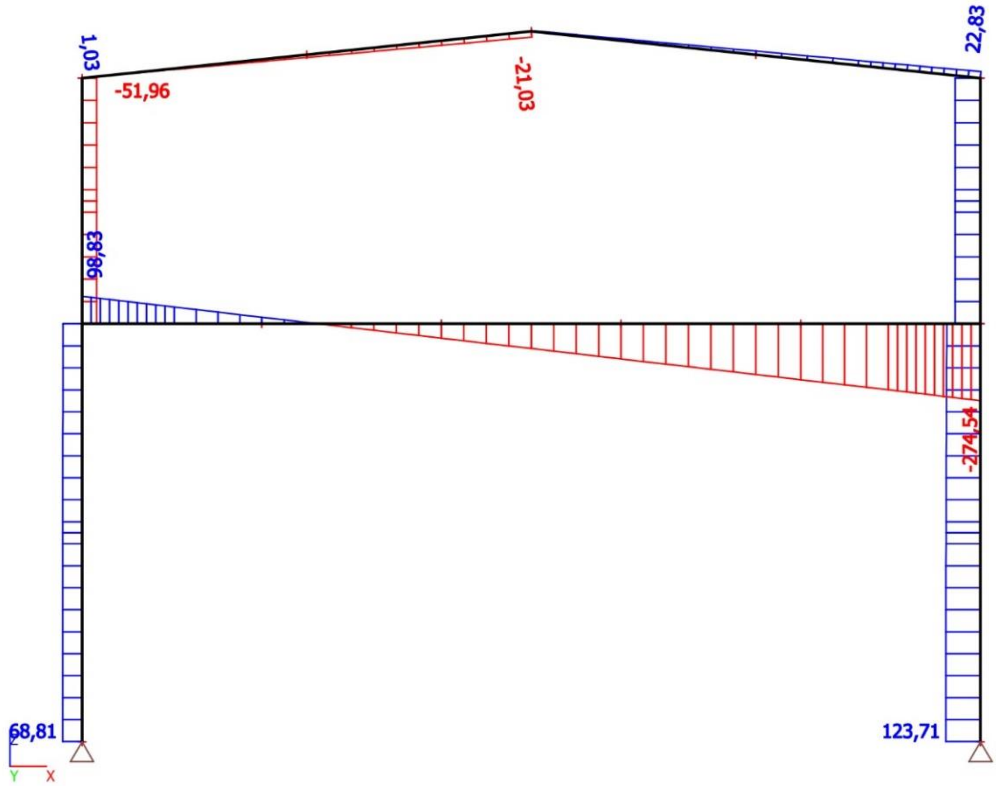
Kot merodajno obremenitev upoštevam ovojnico gravitacijske in potresne kombinacije. Rezultati iz programa SCIA Engineer so prikazani na slikah 53, 54, 55 in 56.



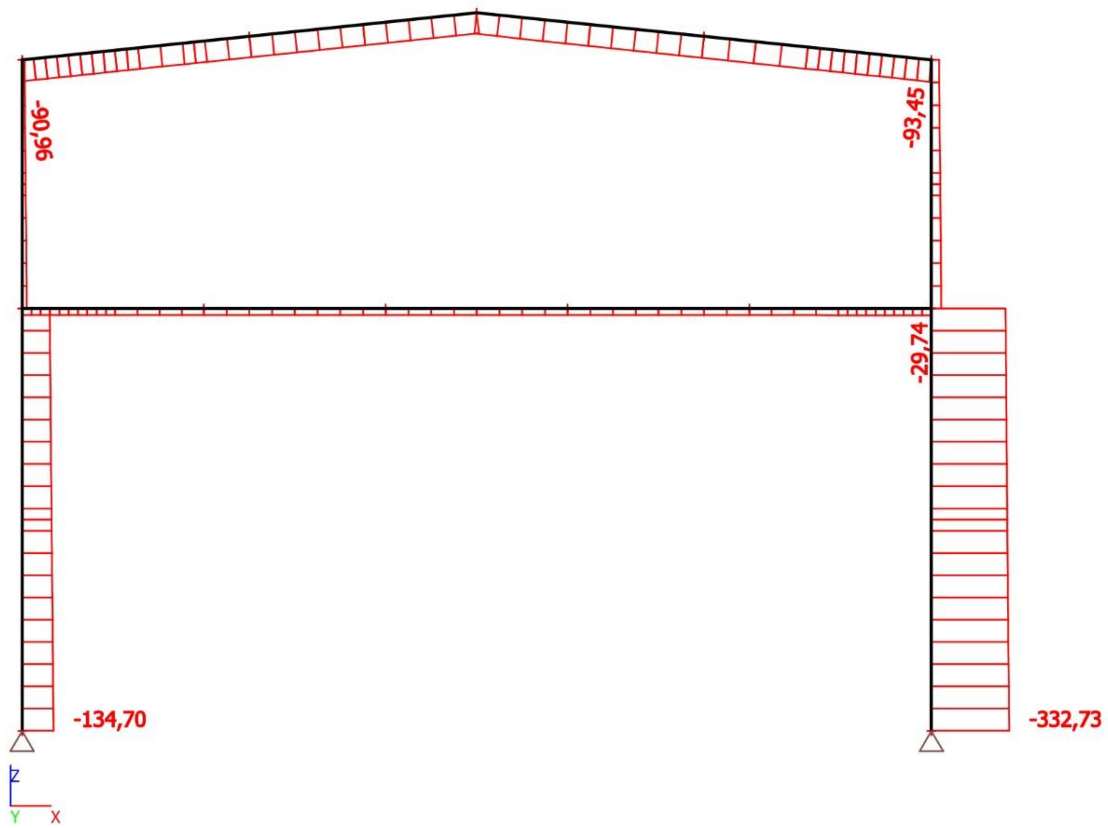
Slika 53: Statični model



Slika 54: Momenti M_y [kNm], gravitacijska in potresna obtežba



Slika 55: Prečne sile *V* [kN], gravitacijska in potresna obtežba



Slika 56: Osne sile *N* [kN], gravitacijska in potresna obtežba

8.2.8.4 Dimenzioniranje momentnega okvira 2

Računske kontrole so izvedene s programom SCIA Engineer in so v prilogi (Priloga C – Potresno projektno stanje, faktor obnašanja q = 1,5; Nosilnost prerezov in stabilnost momentnega okvira 2). Steber se poveča iz HEA 450 na HEA 500 zaradi pogoja »Odpornost elementov proti nestabilnosti, tlačno in upogibno obremenjeni elementi«. V nadaljevanju je prikazan povzetek izpisa za steber HEA 500.

8.2.8.4.1 Steber (B2) - HEA 500, S235

Notranje količine

$N_{Ed} = 317{,}77 \text{ kN (tlak)}$

$V_{Ed} = 120{,}65 \text{ kN}$

$M_{y,Ed} = 632{,}27 \text{ kNm}$

Razred kompaktnosti prereza

- stojina (notranji tlačeni del)

$\frac{c}{t_w} = 32{,}50 \leq 33{,}00 \quad \rightarrow 1.\text{ razred kompaktnosti}$

- pasnica (previsni deli pasnic)

$\frac{c}{t_f} = 5{,}09 \leq 9 \quad \rightarrow 1.\text{ razred kompaktnosti}$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

Nosilnost prereza

- tlak

$N_{c,Rd} = \frac{A\, f_y}{\gamma_{M0}} = 4653{,}00 \text{ kN} \geq N_{Ed} = 317{,}77 \text{ kN}$ ✓

- upogib

$M_{pl,Rd} = \frac{W_{pl,y}\, f_y}{\gamma_{M0}} = 928{,}26 \text{ kNm} \geq M_{y,Ed} = 632{,}27 \text{ kNm}$ ✓

- strig

$V_{pl,Rd} = \frac{A_v\, (f_y/\sqrt{3})}{\gamma_{M0}} = 1020{,}02 \text{ kN} \geq V_{Ed} = 120{,}65 \text{ kN}$ ✓

- upogib, strig in osna sila

$M_{N,y,Rd} = 928{,}25 \text{ kN} \geq M_{y,Ed} = 632{,}27 \text{ kNm}$ ✓

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$N_{b,Rd} = \frac{\chi\, A\, f_y}{\gamma_{M1}} = 3373{,}00 \text{ kN} \geq N_{Ed} = 317{,}77 \text{ kN}$ ✓

- bočna zvrnitev upogibno obremenjenih elementov

$L_c = 5{,}44 \text{ m}$

$$M_{b,Rd} = \chi_{LT}\, W_{pl,y}\, \frac{f_y}{\gamma_{M1}} = 859{,}98 \text{ kNm} \geq M_{y,Ed} = 632{,}27 \text{ kNm}\ \checkmark$$

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\frac{\chi_y\, N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$0{,}07 + 0{,}80 + 0{,}00 = 0{,}87 \leq 1\ \checkmark$$

$$\frac{N_{Ed}}{\frac{\chi_z\, N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

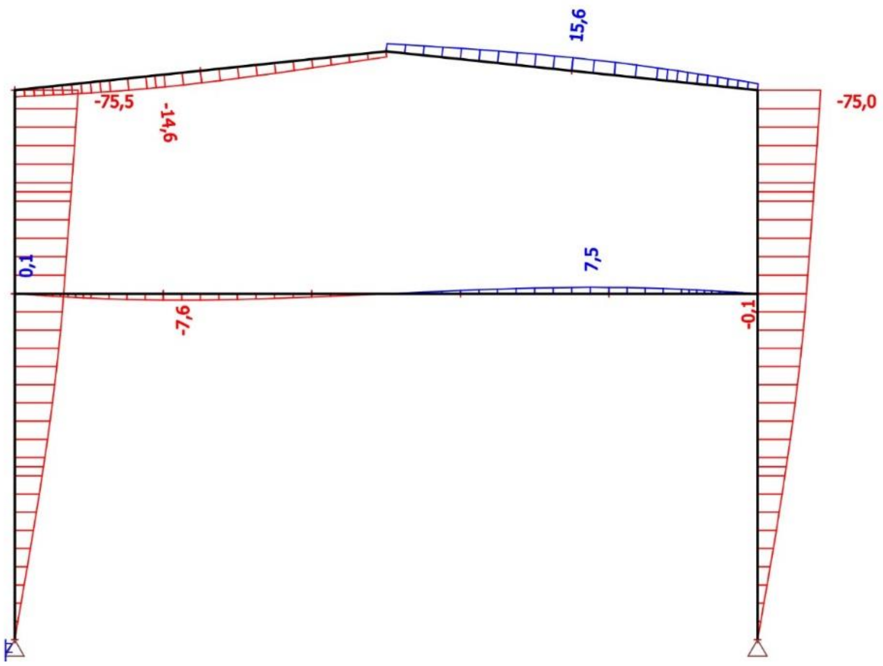
$$0{,}09 + 0{,}42 + 0{,}00 = 0{,}51 \leq 1\ \checkmark$$

8.2.9 Kontrola poškodb (kontrola relativnih pomikov etaž) – okvir 2

Pogoj za stavbe, pri katerih so nekonstrukcijski elementi pritrjeni na konstrukcijo tako, da deformacije konstrukcije nanje ne vplivajo (SIST EN 1998-1, poglavje 4.4.3.2)

$$d_{r,i} \cdot \nu \leq 0{,}010 \cdot h_i$$

$$\nu\ (\gamma_I = 1{,}0) = 0{,}5$$



Slika 57: Horizontalni pomiki etaž za potresni del obtežbe [mm]

$$d_{r,1} = d_{e,1} \cdot q_X = \left(\frac{5{,}83 \text{ cm} + 5{,}81 \text{ cm}}{2}\right) \cdot 1{,}5 = 5{,}82 \text{ cm} \cdot 1{,}5 = 8{,}73 \text{ cm}$$

$$d_{r,2} = d_{e,1} \cdot q_X = \left(\frac{7{,}55 \text{ cm} + 7{,}50 \text{ cm}}{2} - 5{,}82 \text{ cm}\right) \cdot 1{,}5 = 2{,}56 \text{ cm}$$

1. etaža

$$d_{r,1} \cdot \nu = 8{,}73 \text{ cm} \cdot 0{,}5 = 4{,}37 \text{ cm} \leq 0{,}010 \cdot h_1 = 0{,}010 \cdot 544 \text{ cm} = 5{,}44 \text{ cm}\ \checkmark$$

2. etaža - streha

$$d_{r,2} \cdot \nu = 2{,}56 \text{ cm} \cdot 0{,}5 = 1{,}28 \text{ cm} \leq 0{,}010 \cdot h_2 = 0{,}010 \cdot 320 \text{ cm} = 3{,}20 \text{ cm}\ \checkmark$$

8.2.10 Kontrola nosilnosti v prečni smeri - okvir 1 s centričnim povezjem (sistem nateznih diagonal)

8.2.10.1 Gravitacijski del obtežbe

$$G_k + \psi_2 \cdot Q_k$$

$$\psi_2 = \varphi \cdot \psi_{2,1} = 0{,}8 \cdot 0{,}3 = 0{,}24$$

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija g_m = 12,47 kN/m
- streha g_s = 1,59 kN/m
- fasada g_{f1} = 1,69 kN/m
  g_{f2} = 1,73 kN/m

Spremenljiva obtežba

- koristna obtežba medetaže q_m = 11,40 kN/m

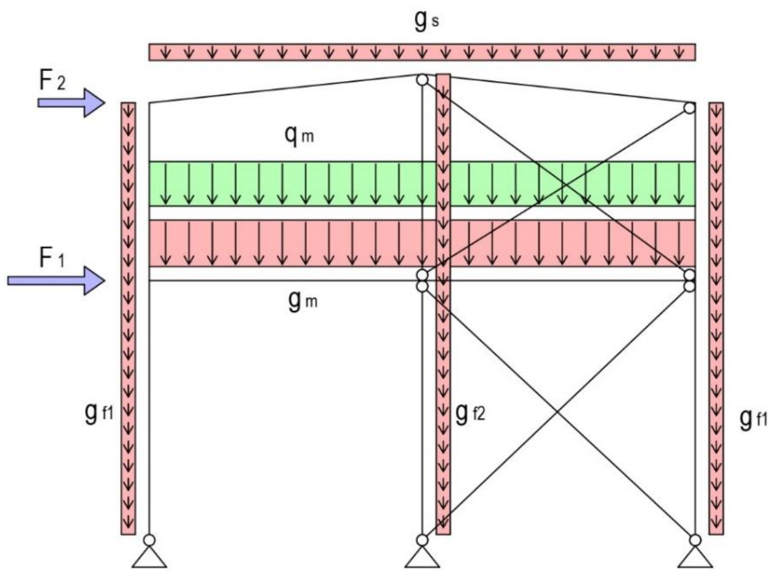
8.2.10.2 Potresni del obtežbe

$$F_1 = \gamma_I \cdot F_1^{(1)} \cdot \delta_{1,7} = 1{,}0 \cdot 71{,}17 \text{ kN} \cdot 1{,}6 = 113{,}87 \text{ kN}$$

$$F_2 = \gamma_I \cdot F_2^{(1)} \cdot \delta_{1,7} = 1{,}0 \cdot 18{,}27 \text{ kN} \cdot 1{,}6 = 29{,}23 \text{ kN}$$

$\gamma_I = 1{,}0$ faktor pomembnosti stavbe (kategorija II - običajne stavbe)

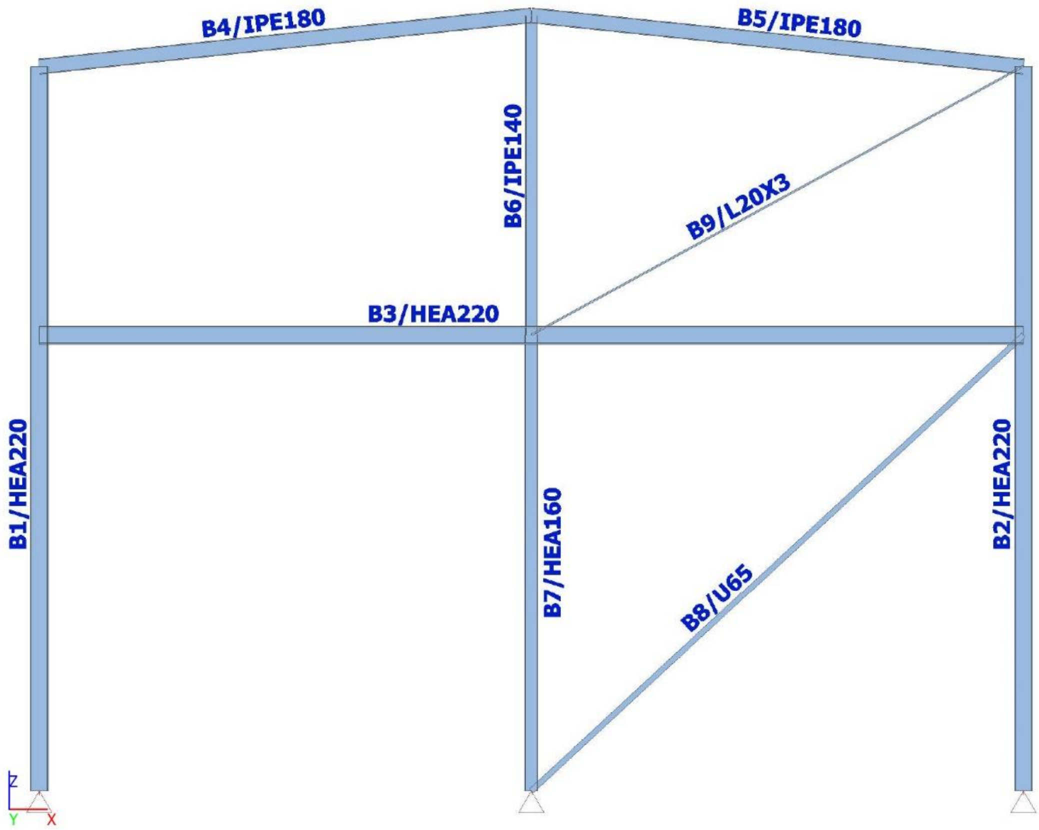
Na sliki 58 je prikazana razporeditev gravitacijske in potresne obtežbe na okvir 1 v programu SCIA Engineer.



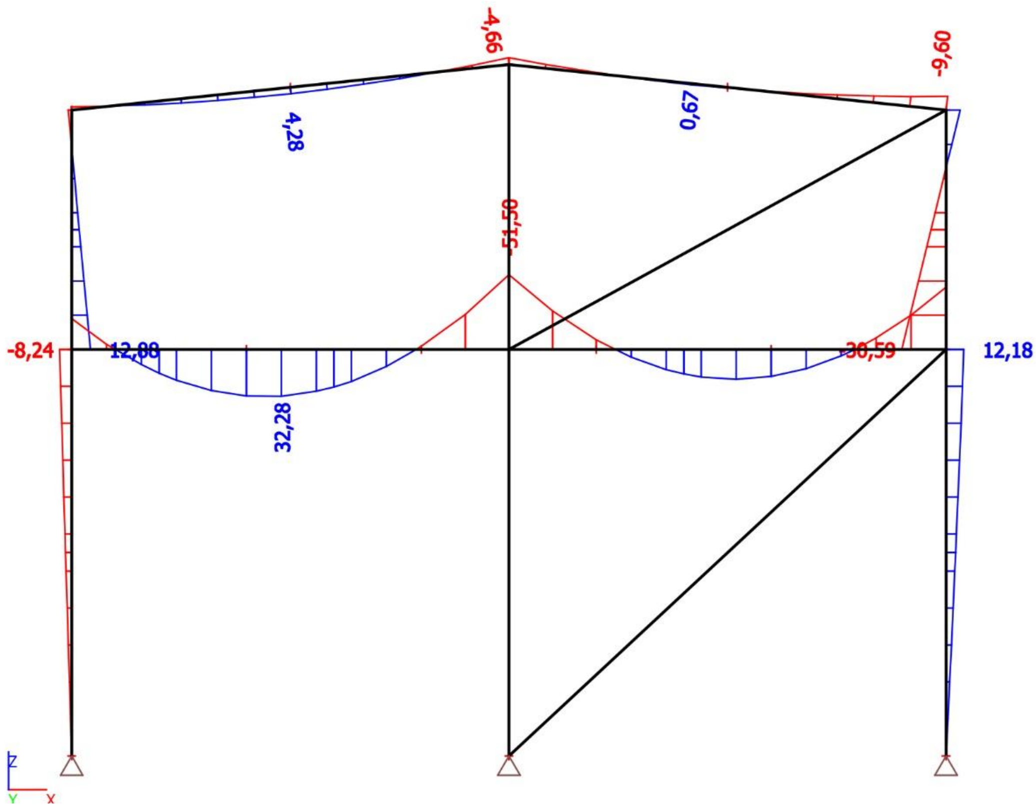
Slika 58: Obtežba na okviru 1 s centričnim povezjem

8.2.10.3 Analiza okvira 1 s centričnim povezjem

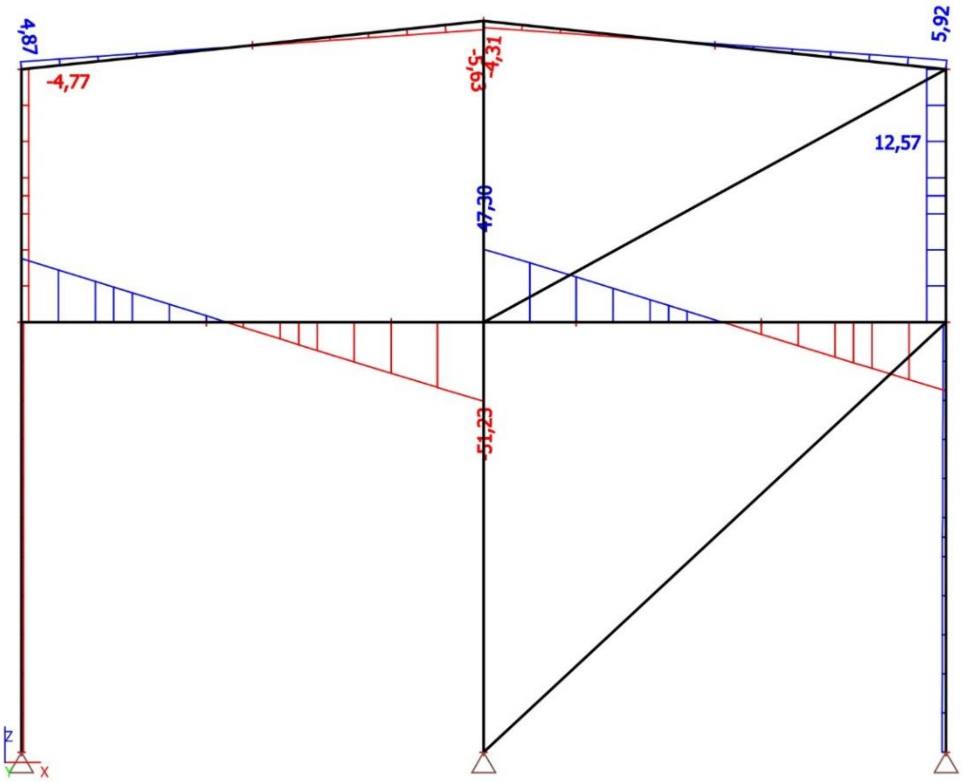
Kot merodajno obremenitev upoštevam ovojnico gravitacijske in potresne kombinacije. Rezultati iz programa SCIA Engineer so prikazani na slikah 59, 60, 61 in 62.



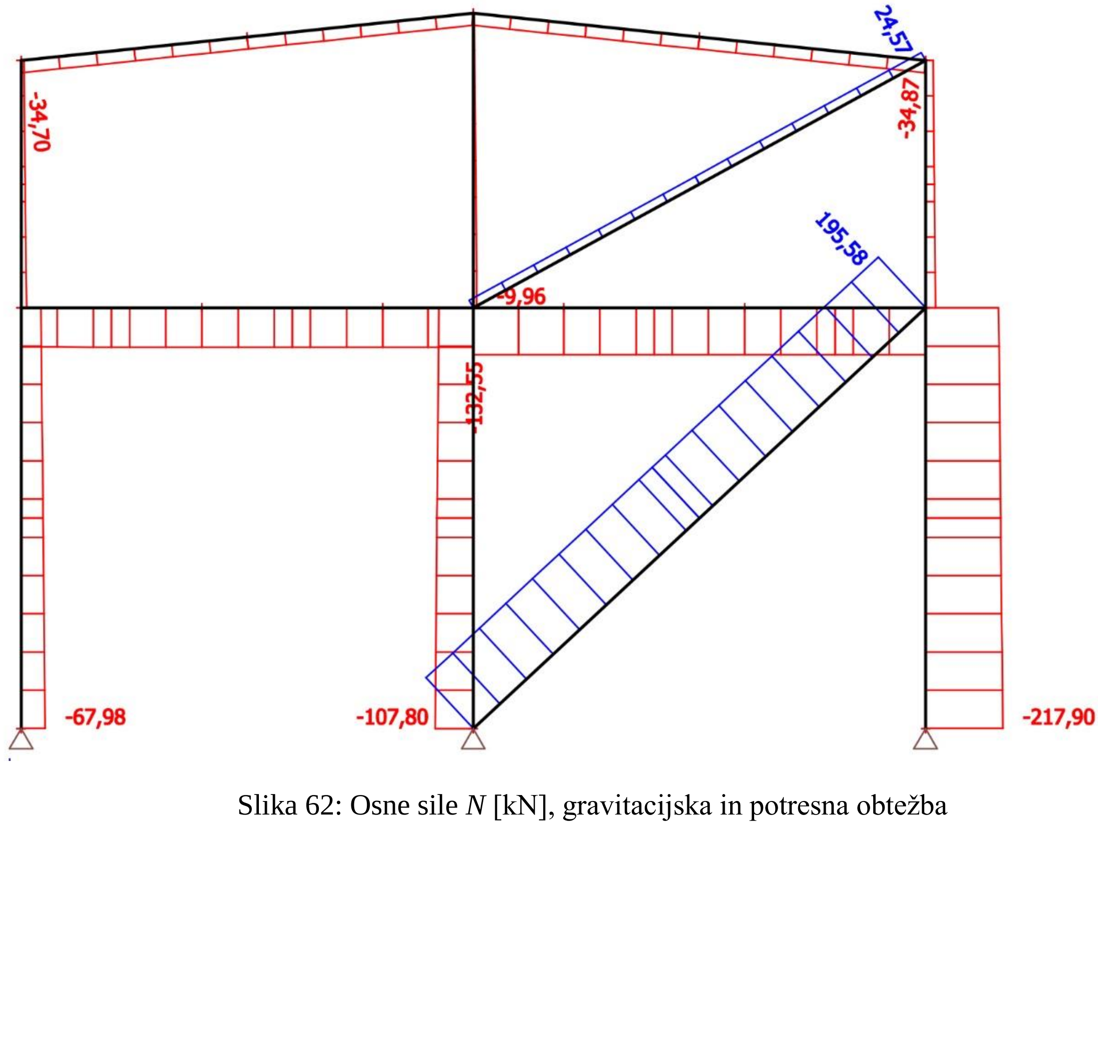
Slika 59: Statični model



Slika 60: Momenti M_y [kNm], gravitacijska in potresna obtežba



Slika 61: Prečne sile *V* [kN], gravitacijska in potresna obtežba



Slika 62: Osne sile *N* [kN], gravitacijska in potresna obtežba

8.2.10.4 Dimenzioniranje okvira 1 s centričnim povezjem

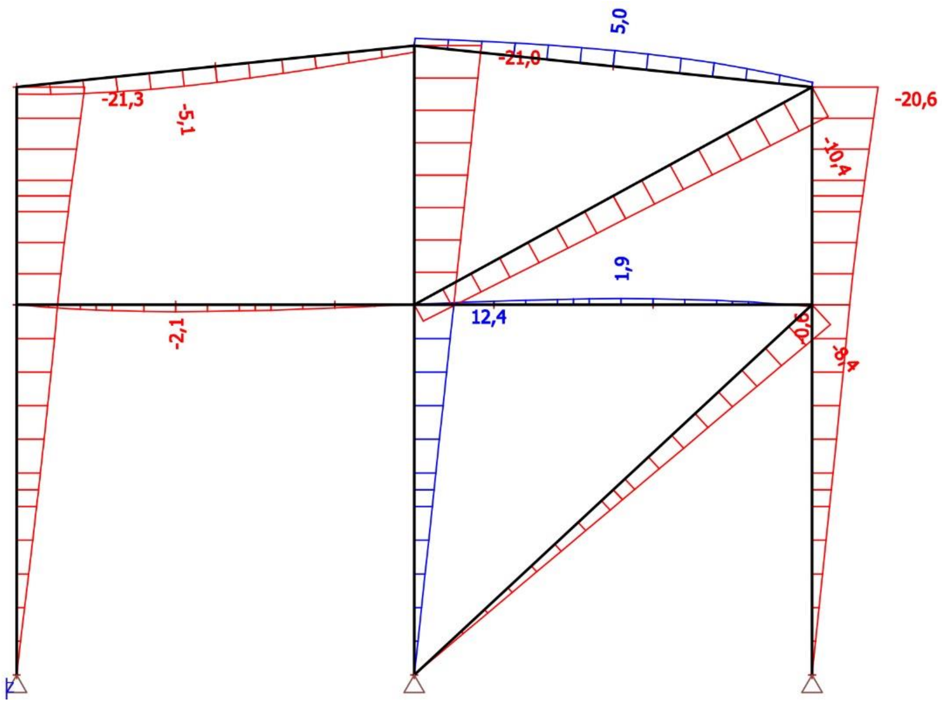
Računske kontrole so izvedene s programom SCIA Engineer in so v prilogi (Priloga C – Potresno projektno stanje, faktor obnašanja q = 1,5; Nosilnost prerezov in stabilnost okvira 1 s centričnim povezjem). Povečata se diagonali v obeh nadstropjih zaradi pogoja »Natezna nosilnost prereza«.

8.2.11 Kontrola poškodb (kontrola relativnih pomikov etaž) – okvir 1

Pogoj za stavbe, pri katerih so nekonstrukcijski elementi pritrjeni na konstrukcijo tako, da deformacije konstrukcije nanje ne vplivajo (SIST EN 1998-1, poglavje 4.4.3.2)

$$d_{r,i} \cdot \nu \leq 0{,}010 \cdot h_i$$

$$\nu\ (\gamma_I = 1{,}0) = 0{,}5$$



Slika 63: Horizontalni pomiki etaž za potresni del obtežbe [mm]

$$d_{r,1} = d_{e,1} \cdot q_X = \left(\frac{1{,}29\text{ cm} + 1{,}18\text{ cm}}{2}\right) \cdot 1{,}5 = 1{,}24\text{ cm} \cdot 1{,}5 = 1{,}86\text{ cm}$$

$$d_{r,2} = d_{e,1} \cdot q_X = \left(\frac{2{,}13\text{ cm} + 2{,}06\text{ cm}}{2} - 1{,}24\text{ cm}\right) \cdot 1{,}5 = 1{,}28\text{ cm}$$

1. etaža

$$d_{r,1} \cdot \nu = 1{,}86\text{ cm} \cdot 0{,}5 = 0{,}93\text{ cm} \leq 0{,}010 \cdot h_1 = 0{,}010 \cdot 544\text{ cm} = 5{,}44\text{ cm} \quad \checkmark$$

2. etaža - streha

$$d_{r,2} \cdot \nu = 1{,}28\text{ cm} \cdot 0{,}5 = 0{,}64\text{ cm} \leq 0{,}010 \cdot h_2 = 0{,}010 \cdot 320\text{ cm} = 3{,}20\text{ cm} \quad \checkmark$$

8.2.12 Kontrola nosilnosti v vzdolžni smeri – centrično povezje (sistem nateznih diagonal)

8.2.12.1 Gravitacijski del obtežbe

Gravitacijski del obtežbe na diagonale nima vpliva, vpliv na stebre je že zajet v analizi prečne smeri X. Vpliva samo na prečke.

$$G_k + \psi_2 \cdot Q_k$$

$$\psi_2 = \varphi \cdot \psi_{2,1} = 0{,}8 \cdot 0{,}3 = 0{,}24$$

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija $g_m = 5{,}35$ kN/m

  $g_m = g'_m\, b' = 4{,}57$ kN/m$^2 \cdot 1{,}17$ m

Spremenljiva obtežba

- koristna obtežba medetaže $q_m = 5{,}03$ kN/m

  $q_m = q'_m\, b' = 4{,}30$ kN/m$^2 \cdot 1{,}17$ m

8.2.12.2 Potresni del obtežbe

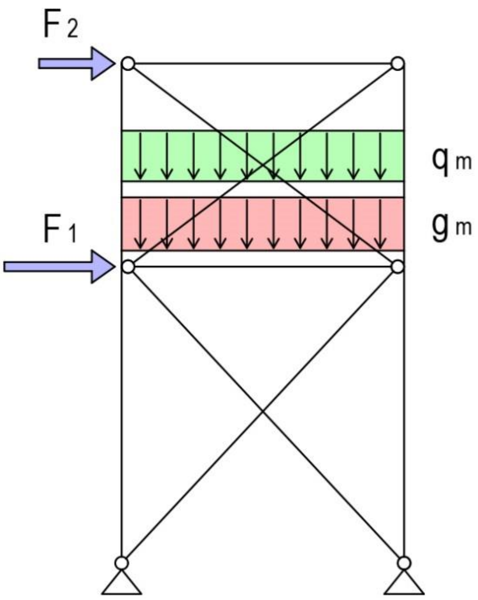
Potresni del obtežbe se razdeli na dve povezji.

$$F_1 = \frac{1}{2} \cdot \gamma_I \cdot F_1^{(A)} \cdot \delta_{A,C} = \frac{1}{2} \cdot 1{,}0 \cdot 366{,}84 \text{ kN} \cdot 1{,}6 = 293{,}47 \text{ kN}$$

$$F_2 = \frac{1}{2} \cdot \gamma_I \cdot F_2^{(A)} \cdot \delta_{A,C} = \frac{1}{2} \cdot 1{,}0 \cdot 94{,}14 \text{ kN} \cdot 1{,}6 = 75{,}31 \text{ kN}$$

$\gamma_I = 1{,}0$ faktor pomembnosti stavbe (kategorija II - običajne stavbe)

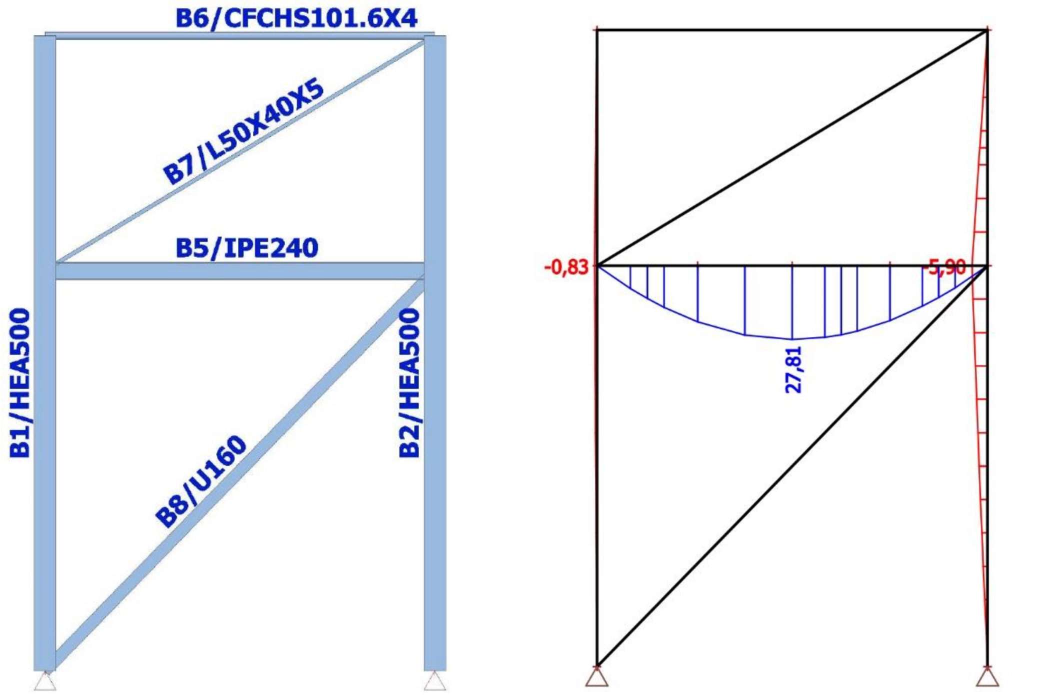
Na sliki 64 je prikazana razporeditev gravitacijske in potresne obtežbe na centrično povezje v programu SCIA Engineer.



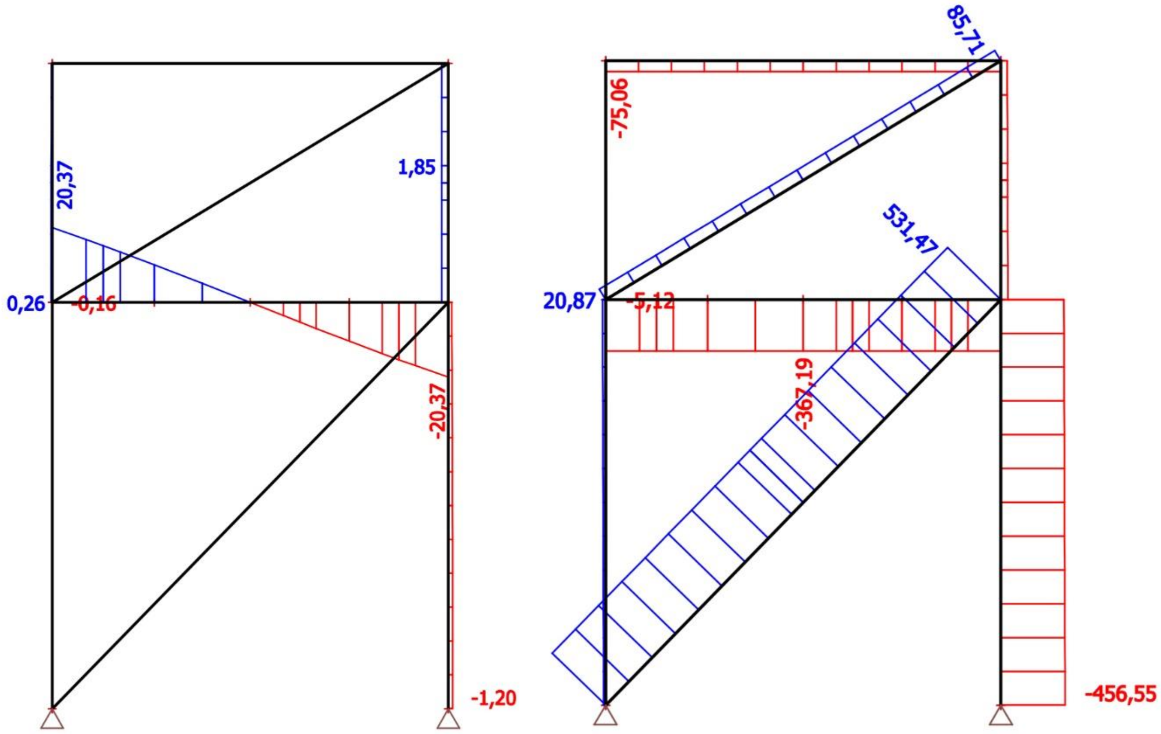
Slika 64: Obtežba na centričnem povezju

8.2.12.3 Analiza centričnega povezja

Kot merodajno obremenitev upoštevam ovojnico gravitacijske in potresne kombinacije. Rezultati iz programa SCIA Engineer so prikazani na slikah 65 in 66.



Slika 65: Statični model in momenti M_y [kNm], gravitacijska in potresna obtežba



Slika 66: Prečne sile V [kN] in osne sile N [kN], gravitacijska in potresna obtežba

8.2.12.4 Dimenzioniranje centričnega povezja in prečk

Računske kontrole so izvedene s programom SCIA Engineer in so v prilogi (Priloga D.3: Nosilnost prerezov in stabilnost centričnega povezja - potresno projektno stanje, faktor obnašanja q = 1,5). Poveča se prečko iz IPE 220 na IPE 240 zaradi pogoja »odpornost elementov proti nestabilnosti, tlačno in upogibno obremenjeni elementi«. V nadaljevanju je prikazan povzetek izpisa.

8.2.12.4.1 Prečka (B5) - IPE 240, S235

Notranje količine

$$N_{Ed} = 367{,}15 \text{ kN (tlak)}$$

$$V_{Ed} = 10{,}27 \text{ kN}$$

$$M_{y,Ed} = 21{,}16 \text{ kNm}$$

Razred kompaktnosti prereza

- stojina (notranji tlačeni del)

$$\frac{c}{t_w} = 30{,}71 \leq 33{,}00 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

- pasnica (previsni deli pasnic)

$$\frac{c}{t_f} = 4{,}28 \leq 9 \quad \rightarrow 1.\,\text{razred kompaktnosti}$$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

Nosilnost prereza

- tlak

$$N_{c,Rd} = \frac{A\, f_y}{\gamma_{M0}} = 918{,}85 \text{ kN} \geq N_{Ed} = 367{,}15 \text{ kN } \checkmark$$

- upogib

$$M_{pl,Rd} = \frac{W_{pl,y}\, f_y}{\gamma_{M0}} = 86{,}25 \text{ kNm} \geq M_{y,Ed} = 21{,}16 \text{ kNm } \checkmark$$

- strig

$$V_{pl,Rd} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = 259{,}52 \text{ kN} \geq V_{Ed} = 10{,}27 \text{ kN } \checkmark$$

- upogib, strig in osna sila

$$M_{y,Ed} = 21{,}16 \text{ kNm} \leq M_{N,y,Rd} = 64{,}67 \text{ kNm } \checkmark$$

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$$N_{b,Rd} = \frac{\chi\, A\, f_y}{\gamma_{M1}} = 795{,}67 \text{ kN} \geq N_{Ed} = 367{,}15 \text{ kN } \checkmark$$

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\frac{\chi_y\, N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$0{,}44 + 0{,}39 + 0{,}00 = 0{,}83 \leq 1$ ✓

$$\frac{N_{Ed}}{\frac{\chi_z\, N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$0{,}47 + 0{,}21 + 0{,}00 = 0{,}68 \leq 1$ ✓

8.2.12.4.2 Prečka (B6) - Cev 101,6 x 4, S235

Notranje količine

$N_{Ed} = 75{,}06$ kN (tlak)

$V_{Ed} = 0{,}00$ kN

$M_{y,Ed} = 0{,}00$ kNm

Razred kompaktnosti prereza

- cevni prerezi

$\frac{d}{t} = 25{,}40 \leq 50{,}00$ → 1. razred kompaktnosti

Prerez spada v 1. razred kompaktnosti (plastični prerez).

Nosilnost prereza

- tlak

$N_{c,Rd} = \frac{A\, f_y}{\gamma_{M0}} = 288{,}11$ kN $\geq N_{Ed} = 75{,}06$ kN ✓

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$N_{b,Rd} = \frac{\chi\, A\, f_y}{\gamma_{M1}} = 79{,}19$ kN $\geq N_{Ed} = 75{,}06$ kN ✓

8.2.12.4.3 Diagonala (B7) - L 50x40x5, S235

Notranje količine

$N_{Ed} = 85{,}71$ kN (nateg)

Nosilnost prereza

- nateg

$N_{t,Rd} = \frac{A\, f_y}{\gamma_{M0}} = 100{,}35$ kN $\geq N_{Ed} = 85{,}71$ kN ✓

8.2.12.4.4 Diagonala (B8) - U 160, S235

Notranje količine

$$N_{Ed} = 531{,}47 \text{ kN (nateg)}$$

Nosilnost prereza

- nateg

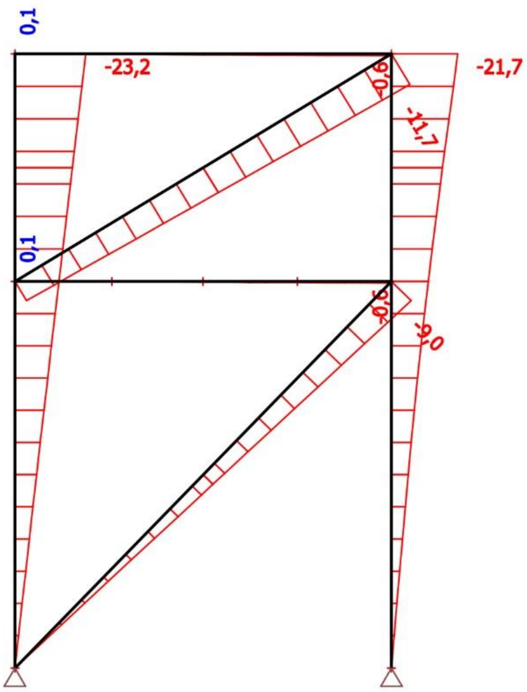
$$N_{t,Rd} = \frac{A\,f_y}{\gamma_{M0}} = 564{,}00 \text{ kN} \geq N_{Ed} = 531{,}47 \text{ kN} \checkmark$$

8.2.13 Kontrola poškodb (kontrola relativnih pomikov etaž) – okvir A

Pogoj za stavbe, pri katerih so nekonstrukcijski elementi pritrjeni na konstrukcijo tako, da deformacije konstrukcije nanje ne vplivajo (SIST EN 1998-1, poglavje 4.4.3.2)

$$d_{r,i} \cdot \nu \leq 0{,}010 \cdot h_i$$

$$\nu\ (\gamma_I = 1{,}0) = 0{,}5$$



Slika 67: Horizontalni pomiki etaž za potresni del obtežbe [mm]

$$d_{r,1} = d_{e,1} \cdot q_Y = \left(\frac{1{,}44 \text{ cm} + 1{,}20 \text{ cm}}{2}\right) \cdot 1{,}5 = 1{,}32 \text{ cm} \cdot 1{,}5 = 1{,}98 \text{ cm}$$

$$d_{r,2} = d_{e,1} \cdot q_Y = \left(\frac{2{,}32 \text{ cm} + 2{,}17 \text{ cm}}{2} - 1{,}32 \text{ cm}\right) \cdot 1{,}5 = 1{,}39 \text{ cm}$$

1. etaža

$$d_{r,1} \cdot \nu = 1{,}98 \text{ cm} \cdot 0{,}5 = 0{,}99 \text{ cm} \leq 0{,}010 \cdot h_1 = 0{,}010 \cdot 544 \text{ cm} = 5{,}44 \text{ cm} \checkmark$$

2. etaža - streha

$$d_{r,2} \cdot \nu = 1{,}39 \text{ cm} \cdot 0{,}5 = 0{,}70 \text{ cm} \leq 0{,}010 \cdot h_2 = 0{,}010 \cdot 320 \text{ cm} = 3{,}20 \text{ cm} \checkmark$$

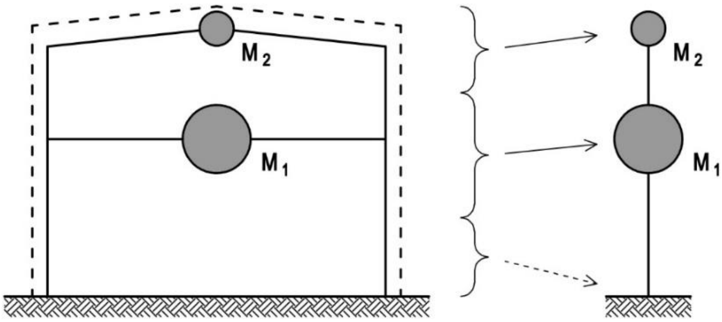
8.3 PRIMER KONSTRUKCIJE S SPOSOBNOSTJO SIPANJA ENERGIJE

Prereze vseh stebrov notranjih okvirov povečamo iz HEA 450 na HEA 500 (glej dimenzioniranje elementov).

8.3.1 Izračun mas

Kombinacija vplivov za potresno projektno stanje (SIST EN 1998-1, poglavje 3.2.4)

$$\sum G_{k,j} + \sum \psi_{E,i} \cdot Q_{k,i} \quad \text{kjer je} \quad \psi_{E,i} = \varphi \cdot \psi_{2,i}$$



Slika 68: Razporeditev mas po etažah in računski model

Preglednica 20: Izračun mase objekta po etažah

G$_{k,1}$ **– lastna in stalna teža medetaže**				
	[kN/m]	[m]	kosov	[kN]
Steber HEA 500	1,521	4,32	10	65,71
Steber HEA 220	0,495	4,32	4	8,55
Steber HEA 160	0,298	2,72	2	1,62
Steber IPE 140	0,127	1,60	2	0,41
Prečka HEA 450	1,373	11,70	5	80,32
Prečka HEA 220	0,495	11,70	2	11,58
Sekundarni nosilci IPE 220	0,257	5,30	36	49,04
Vertikalno povezje, čelne pločevine, vijaki, ...	Ocena 10% teže			22,40
	[kN/m^2]	[m^2]		[kN]
Stalna teža (AB plošča, parket, estrih, ...)	4,480	372		1666,56
Fasada FTV 150 (bočna in čelna)	0,291	387		112,62
	[kN/100]		kosov	[kN]
Čepi Ø19	0,236		1296	3,06
			G$_{k,1}$ **[kN]**	**2.028,60**
			[kg]	**206.798**
Q$_{k,1}$ **– koristna teža medetaže**				
	[kN/m^2]	[m^2]		[kN]
Koristna teža (pisarne, predelne stene, ...)	4,300	372		1599,60
			Q$_{k,1}$ **[kN]**	**1.599,60**
			[kg]	**163.058**

$G_{k,2}$ – lastna in stalna teža strehe				
	[kN/m]	[m]	kosov	[kN]
Steber HEA 500	1,521	1,60	10	24,34
Steber HEA 220	0,495	1,60	4	3,17
Steber IPE 140	0,127	2,21	2	0,56
Strešni nosilec IPE 300	0,414	5,88	10	24,34
Strešni nosilec IPE 180	0,184	5,88	4	4,33
Strešna lega IPE 180	0,184	19,20	6	21,20
Strešna lega IPE 200	0,220	6,30	12	16,63
Vertikalno in horizontalno povezje, zatege, vijaki, ...	Ocena 10% teže			9,60
	[kN/m²]	[m²]		[kN]
Stalna teža (spuščen strop, inštalacije, ...)	0,300	372		111,60
Fasada FTV 150 (bočna in čelna)	0,291	191		55,58
Fasada SNV 150 (streha)	0,291	402		116,98
			$G_{k,1}$ [kN]	**389,80**
			[kg]	**39.735**

Teža 1. etaže

$$F_{M1} = G_{k,1} + \varphi \cdot \psi_{2,1} \cdot Q_{k,1} = 2028{,}60 \text{ kN} + 0{,}8 \cdot 0{,}3 \cdot 1599{,}60 \text{ kN} = 2412{,}50 \text{ kN}$$

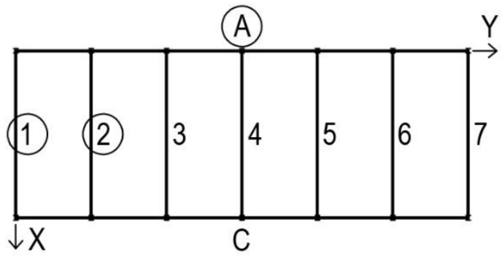
$\varphi = 0{,}8$ kategorija A do C: zasedba nekaterih etaž je povezana

$\psi_{2,1} = 0{,}3$ kategorija B: pisarne

Teža 2. etaže - strehe

$$F_{M2} = G_{k,2} = 389{,}80 \text{ kN}$$

Delež etažnih mas za okvir 1 in 2 ter okvir A



Slika 69: Tloris konstrukcije z označenimi deli za ravninsko analizo potresa

Prečna smer X (prvi in sedmi okvir sta obremenjena polovično)

$$F_{M1}^{(2)} = \frac{F_{M1}}{6} = \frac{2412{,}50 \text{ kN}}{6} = 402{,}08 \text{ kN} \quad (40.987 \text{ kg})$$

$$F_{M1}^{(1)} = \frac{F_{M1}^{(2)}}{2} = \frac{402{,}08 \text{ kN}}{2} = 201{,}04 \text{ kN} \quad (20.494 \text{ kg})$$

$$F_{M2}^{(2)} = \frac{F_{M2}}{6} = \frac{389{,}80 \text{ kN}}{6} = 64{,}97 \text{ kN} \quad (6.622 \text{ kg})$$

$$F_{M2}^{(1)} = \frac{F_{M2}^{(2)}}{2} = \frac{64{,}97 \text{ kN}}{2} = 32{,}49 \text{ kN} \quad (3.311 \text{ kg})$$

Vzdolžna smer Y

$$F_{M1}^{(A)} = \frac{F_{M1}}{2} = \frac{2412{,}50 \text{ kN}}{2} = 1206{,}25 \text{ kN} \quad (122.961 \text{ kg})$$

$$F_{M2}^{(A)} = \frac{F_{M2}}{2} = \frac{389{,}80 \text{ kN}}{2} = 194{,}90 \text{ kN} \quad (19.867 \text{ kg})$$

8.3.2 Faktor obnašanja q

Prečna smer X:

- momentni okvir (notranji), okvir s centričnim povezjem (zunanji)
- disipacija energije v prečkah momentnega okvira in nateznih diagonalah
- prečni prerezi so v 1. ali 2. razredu kompaktnosti
- razred srednje duktilnosti (DCM): $q \leq 4$

$q_X = 4$

Vzdolžna smer Y:

- okvir s centričnim povezjem
- disipacija energije v nateznih diagonalah, tlačne se izklonijo
- prečni prerezi so v 1. ali 2. razredu kompaktnosti
- razred srednje duktilnosti (DCM): $q \leq 4$

$q_Y = 4$

8.3.3 Nihajni časi

Osnovni nihajni časi T_1 vseh ravninskih modelov so določeni v SCIA Engineer

$T_{1,X}^{(2)} = 0{,}81$ s notranji momentni okvir 2

$T_{1,X}^{(1)} = 0{,}20$ s zunanji okvir 1 s centričnim povezjem

$T_{1,Y} = 0{,}70$ s vzdolžni okvir A s centričnim povezjem

8.3.4 Spekter pospeškov

Objekt se nahaja v naselju Grosuplje. Podatke o pospeških temeljnih tal dobimo iz karte potresne nevarnosti Slovenije, veljavne v času uporabe standarda.

Projektni pospešek tal

$$a_g = 0{,}200\, g$$

Tip tal C

$$S = 1{,}15$$

$$T_B = 0{,}20 \text{ s}$$

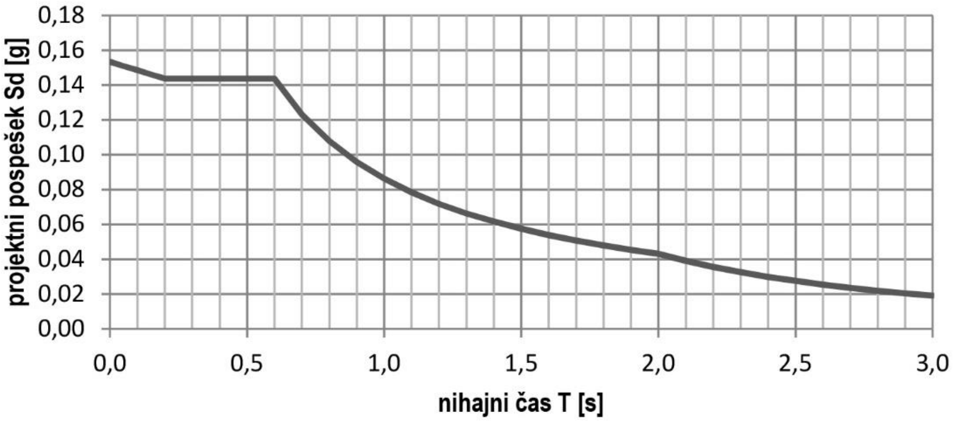
$$T_C = 0{,}60 \text{ s}$$

$$T_D = 2{,}00 \text{ s}$$

Na sliki 70 so prikazana območja nihajnih časov.

$$T_B \leq T_{1,X}^{(1)} \leq T_C$$

$$T_C \leq T_{1,X}^{(2)}, T_{1,Y} \leq T_D$$



Slika 70: Projektni spekter, faktor obnašanja q = 4

Projektni pospešek v smeri X, okvir 2

$$S_{d,X}^{(2)} = a_g\ S\ \frac{2{,}5}{q_X} \left[\frac{T_C}{T_{1,X}^{(2)}}\right] \geq \beta\ a_g = 0{,}2 \cdot 0{,}2\ g = 0{,}04\ g$$

$$S_{d,X}^{(2)} = 0{,}2\ g \cdot 1{,}15 \cdot \frac{2{,}5}{4} \cdot \left[\frac{0{,}60}{0{,}81}\right] = 0{,}106\ g = p_{X,2}\ g$$

Projektni pospešek v smeri X, okvir 1

$$S_{d,X}^{(1)} = a_g\ S\ \frac{2{,}5}{q_X} = 0{,}2\ g \cdot 1{,}15 \cdot \frac{2{,}5}{4} = 0{,}144\ g = p_{X,1}\ g$$

Projektni pospešek v smeri Y

$$S_{d,Y} = a_g\ S\ \frac{2{,}5}{q_Y} \left[\frac{T_C}{T_{1,Y}}\right] \geq \beta\ a_g = 0{,}2 \cdot 0{,}2\ g = 0{,}04\ g$$

$$S_{d,X} = 0{,}2\ g \cdot 1{,}15 \cdot \frac{2{,}5}{4} \cdot \left[\frac{0{,}60}{0{,}70}\right] = 0{,}123\ g = p_Y\ g$$

8.3.5 Celotna prečna sila (»Base Shear«) posameznega okvira

Celotna prečna sila F_b na mestu vpetja konstrukcije (SIST EN 1998-1, poglavje 4.3.3.2.2)

$$F_{b,i} = S_{d,i}(T_{1,i})\ m_i\ \lambda = p_i\ g\ m_i\ \lambda = p_i\ F_{Mi}\ \lambda$$

$$\lambda = 1{,}0$$

Prečni okvir 2

$$F_{b,X}^{(2)} = p_{X,2} \cdot \left(F_{M1}^{(2)} + F_{M2}^{(2)}\right) = 0{,}106 \cdot (402{,}08\ \text{kN} + 64{,}97\ \text{kN}) = 49{,}51\ \text{kN}$$

Prečni okvir 1

$$F_{b,X}^{(1)} = p_{X,1} \cdot \left(F_{M1}^{(1)} + F_{M2}^{(1)}\right) = 0{,}144 \cdot (201{,}04\ \text{kN} + 32{,}49\ \text{kN}) = 33{,}63\ \text{kN}$$

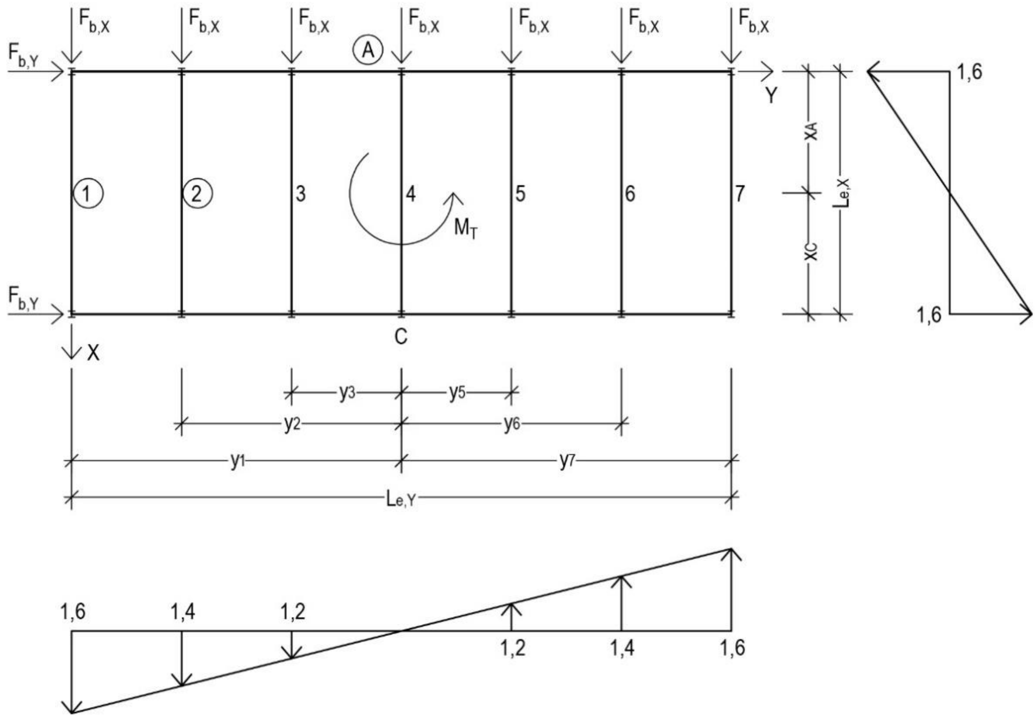
Vzdolžni okvir A

$$F_{b,Y} = p_Y \cdot \left(F_{M1}^{(A)} + F_{M2}^{(A)}\right) = 0{,}123 \cdot (1206{,}25\ \text{kN} + 194{,}90\ \text{kN}) = 172{,}34\ \text{kN}$$

8.3.6 Torzijski vpliv

Vpliv naključne torzije δ (SIST EN 1998-1, poglavje 4.3.3.2.4)

$$\delta = 1 + 1{,}2 \cdot \frac{x_i}{L_{e,i}} = \left(1 + 1{,}2 \cdot \frac{y_i}{L_{e,i}}\right)$$



Slika 71: Torzijski vpliv - analiza z dvema ravninskima modeloma

Smer X, okvir 2

$$\delta_{2,6} = 1 + 1{,}2 \cdot \frac{y_{2,6}}{L_{e,Y}} = 1 + 1{,}2 \cdot \frac{L_{e,Y}/3}{L_{e,Y}} = 1{,}4$$

Smer X, okvir 1

$$\delta_{1,7} = 1 + 1{,}2 \cdot \frac{y_{1,7}}{L_{e,Y}} = 1 + 1{,}2 \cdot \frac{L_{e,Y}/2}{L_{e,Y}} = 1{,}6$$

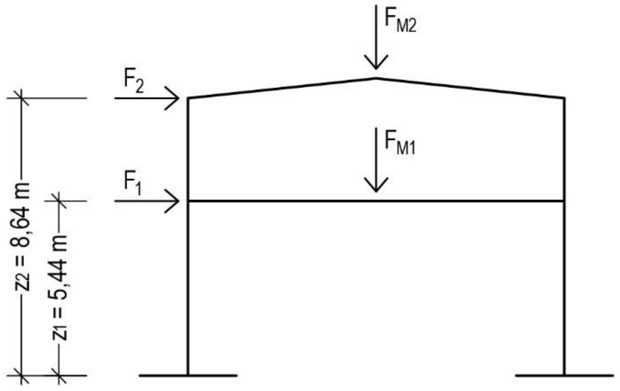
Smer Y, okvir A

$$\delta_{A,C} = 1 + 1{,}2 \cdot \frac{x_{A,C}}{L_{e,X}} = 1 + 1{,}2 \cdot \frac{L_{e,X}/2}{L_{e,X}} = 1{,}6$$

8.3.7 Razdelitev sil po višini

Vodoravne sile F_i v etažah (SIST EN 1998-1, poglavje 4.3.3.2.3).

$$F_i = F_b \cdot \frac{z_i \cdot F_{Mi}}{\sum z_j \cdot F_{Mj}}$$



Slika 72: Razdelitev sil po višini

Sile za okvir 2

$$F_1^{(2)} = F_{b,X}^{(2)} \cdot \frac{z_1 \cdot F_{M1}^{(2)}}{z_1 \cdot F_{M1}^{(2)} + z_2 \cdot F_{M2}^{(2)}} = 49{,}51 \cdot \frac{5{,}44 \cdot 402{,}08}{5{,}44 \cdot 402{,}08 + 8{,}64 \cdot 64{,}97} = 39{,}40 \text{ kN}$$

$$F_2^{(2)} = F_{b,X}^{(2)} \cdot \frac{z_2 \cdot F_{M2}^{(2)}}{z_1 \cdot F_{M1}^{(2)} + z_2 \cdot F_{M2}^{(2)}} = 49{,}51 \cdot \frac{8{,}64 \cdot 64{,}97}{5{,}44 \cdot 402{,}08 + 8{,}64 \cdot 64{,}97} = 10{,}11 \text{ kN}$$

Sile za okvir 1

$$F_1^{(1)} = F_{b,X}^{(1)} \cdot \frac{z_1 \cdot F_{M1}^{(1)}}{z_1 \cdot F_{M1}^{(1)} + z_2 \cdot F_{M2}^{(1)}} = 33{,}63 \cdot \frac{5{,}44 \cdot 201{,}04}{5{,}44 \cdot 201{,}04 + 8{,}64 \cdot 32{,}49} = 26{,}76 \text{ kN}$$

$$F_2^{(1)} = F_{b,X}^{(1)} \cdot \frac{z_2 \cdot F_{M2}^{(1)}}{z_1 \cdot F_{M1}^{(1)} + z_2 \cdot F_{M2}^{(1)}} = 33{,}63 \cdot \frac{8{,}64 \cdot 32{,}49}{5{,}44 \cdot 201{,}04 + 8{,}64 \cdot 32{,}49} = 6{,}87 \text{ kN}$$

Sile za okvir A

$$F_1^{(A)} = F_{b,Y} \cdot \frac{z_1 \cdot F_{M1}^{(A)}}{z_1 \cdot F_{M1}^{(A)} + z_2 \cdot F_{M2}^{(A)}} = 172{,}34 \cdot \frac{5{,}44 \cdot 1206{,}25}{5{,}44 \cdot 1206{,}25 + 8{,}64 \cdot 194{,}90} = 137{,}15 \text{ kN}$$

$$F_2^{(A)} = F_{b,Y} \cdot \frac{z_2 \cdot F_{M2}^{(A)}}{z_1 \cdot F_{M1}^{(A)} + z_2 \cdot F_{M2}^{(A)}} = 172{,}34 \cdot \frac{8{,}64 \cdot 194{,}90}{5{,}44 \cdot 1206{,}25 + 8{,}64 \cdot 194{,}90} = 35{,}19 \text{ kN}$$

8.3.8 Kontrola nosilnosti v prečni smeri - momentni okvir 2

8.3.8.1 Gravitacijski del obtežbe

$$G_k + \psi_2 \cdot Q_k$$

$$\psi_2 = \varphi \cdot \psi_{2,1} = 0{,}8 \cdot 0{,}3 = 0{,}24$$

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija g_m = 24,93 kN/m
- streha g_s = 3,18 kN/m
- fasada g_f = 1,59 kN/m

Spremenljiva obtežba

- koristna obtežba medetaže q_m = 22,79 kN/m

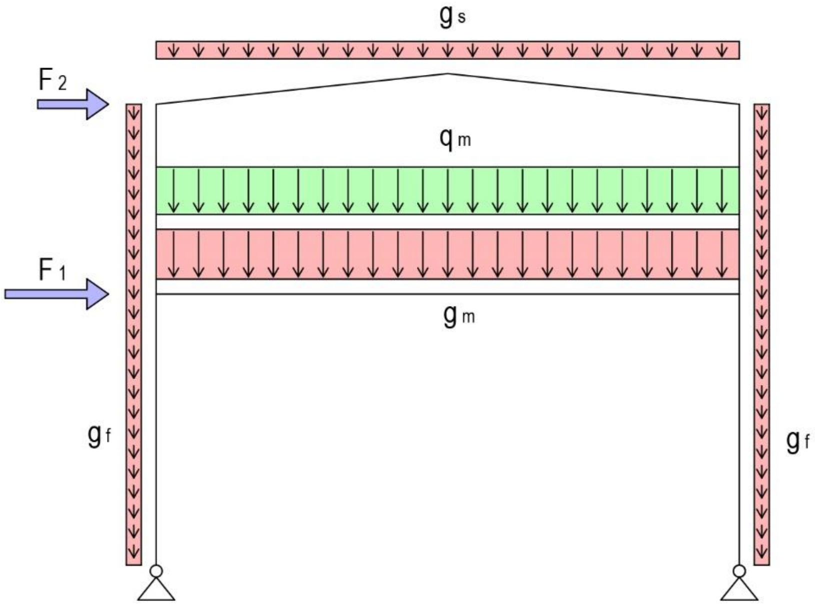
8.3.8.2 Potresni del obtežbe

$$F_1 = \gamma_I \cdot F_1^{(2)} \cdot \delta_{2,6} = 1{,}0 \cdot 39{,}40 \text{ kN} \cdot 1{,}4 = 55{,}16 \text{ kN}$$

$$F_2 = \gamma_I \cdot F_2^{(2)} \cdot \delta_{2,6} = 1{,}0 \cdot 10{,}11 \text{ kN} \cdot 1{,}4 = 14{,}15 \text{ kN}$$

$\gamma_I = 1{,}0$ faktor pomembnosti stavbe (kategorija II - običajne stavbe)

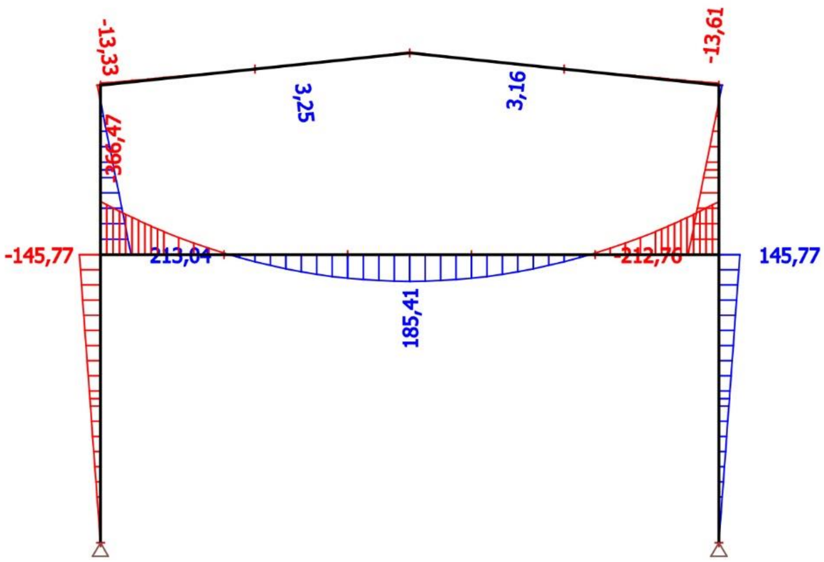
Na sliki 73 je prikazana razporeditev gravitacijske in potresne obtežbe na okvir 2 v programu SCIA Engineer.



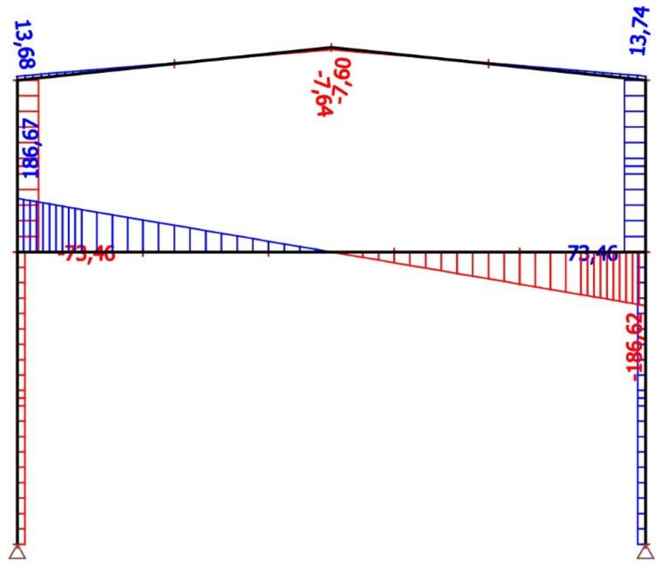
Slika 73: Obtežba na momentnem okviru 2

8.3.8.3 Analiza momentnega okvira 2

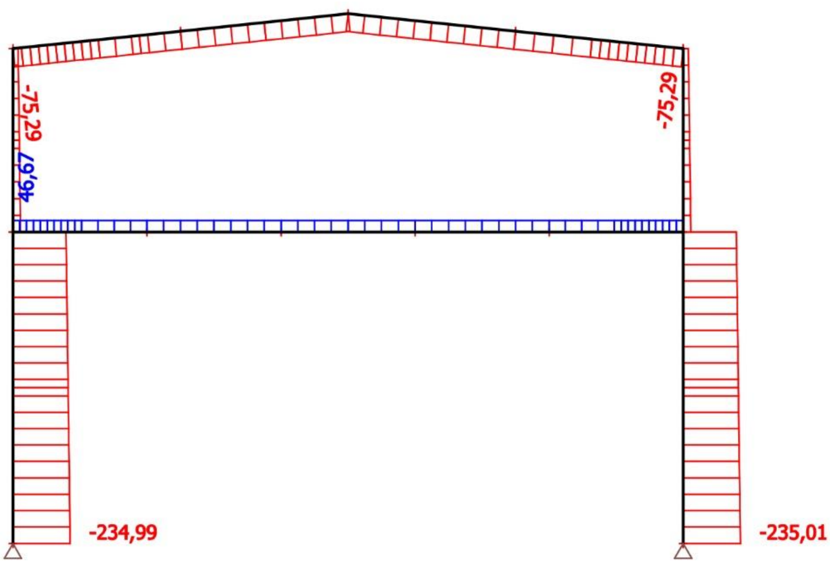
Notranje sile in momenti za gravitacijski in potresni del obtežbe so prikazani na slikah 74 do 79.



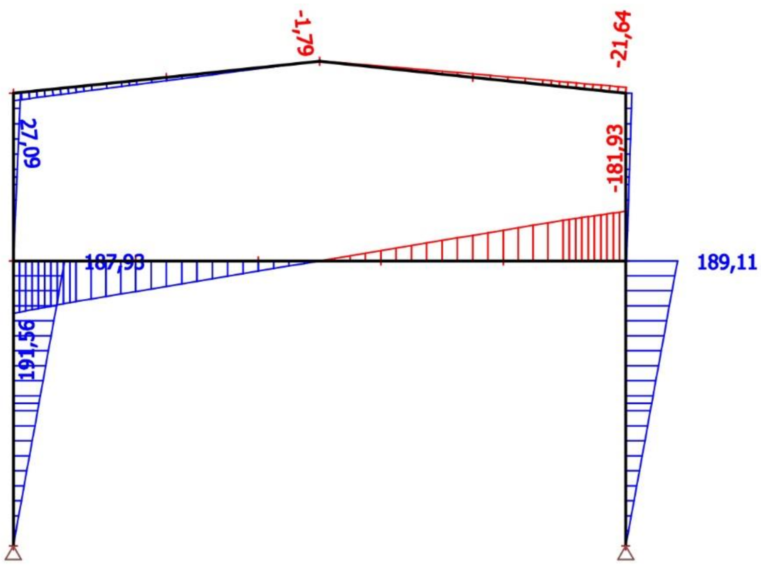
Slika 74: Momenti M_y [kNm], gravitacijska obtežba



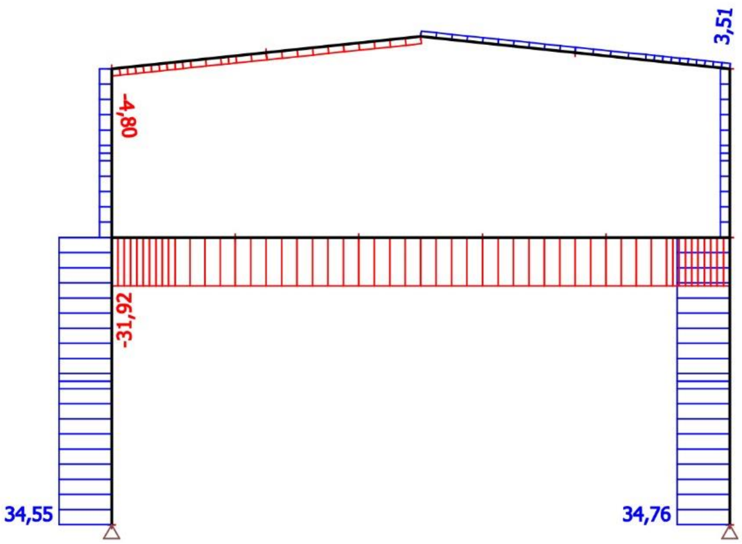
Slika 75: Prečne sile V [kN], gravitacijska obtežba



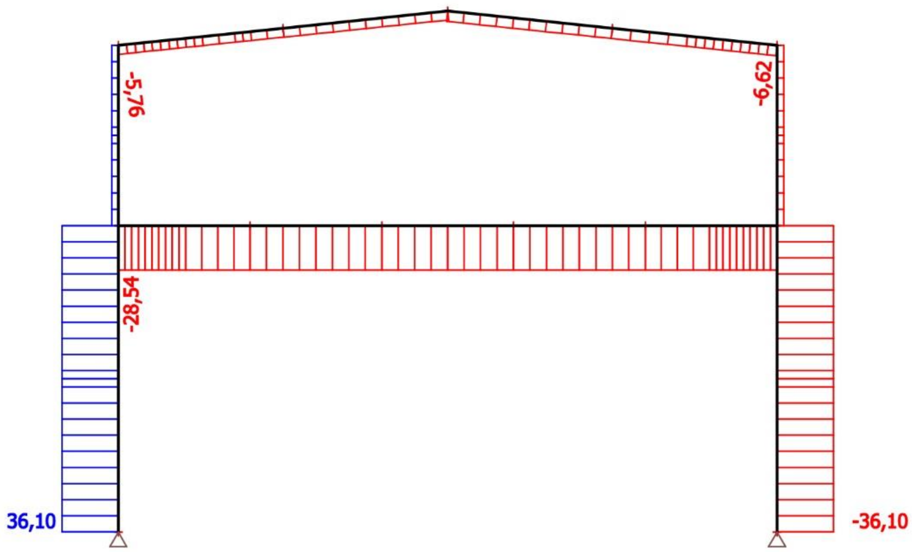
Slika 76: Osne sile N [kN], gravitacijska obtežba



Slika 77: Momenti M_y [kNm], potresna obtežba



Slika 78: Prečne sile V [kN], potresna obtežba

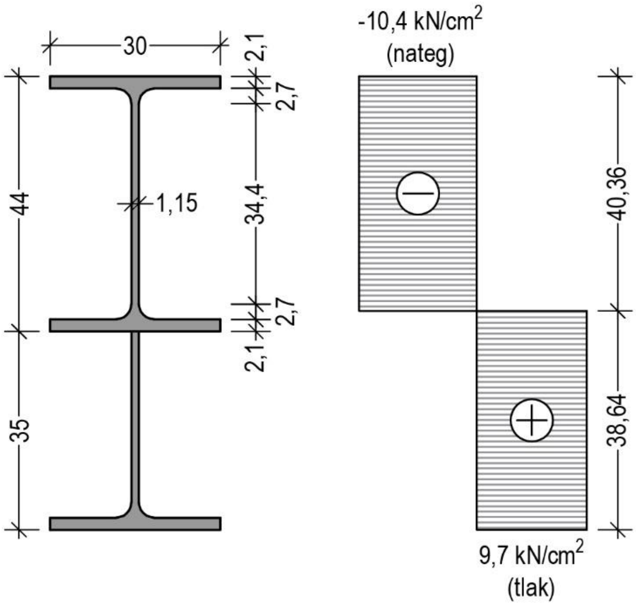


Slika 79: Osne sile N [kN], potresna obtežba

8.3.8.4 Kontrola kompaktnosti prerezov

Kontrolo kompaktnosti izvedemo za prečke z vutami, ostali prečni prerezi so v 1. razredu kompaktnosti (SCIA Engineer).

Prečka HEA 450 in vuta 350, S235



Slika 80: HEA 450 in vuta, geometrija ter normalne napetosti

- stojina vute (notranji tlačeni del)

$$\frac{c}{t_w} = \frac{(35 - 2,1 - 2,7)\ \text{cm}}{1,15\ \text{cm}} = 26,26 \leq 33 \cdot \varepsilon = 33,00$$

- pasnica (previsni deli pasnic)

$$\frac{c}{t_f} = \frac{\left(\frac{30}{2} - \frac{1,15}{2} - 2,7\right)\ \text{cm}}{2,1\ \text{cm}} = 5,58 \leq 9 \cdot \varepsilon = 9,00$$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

- prečni prerezi v strigu

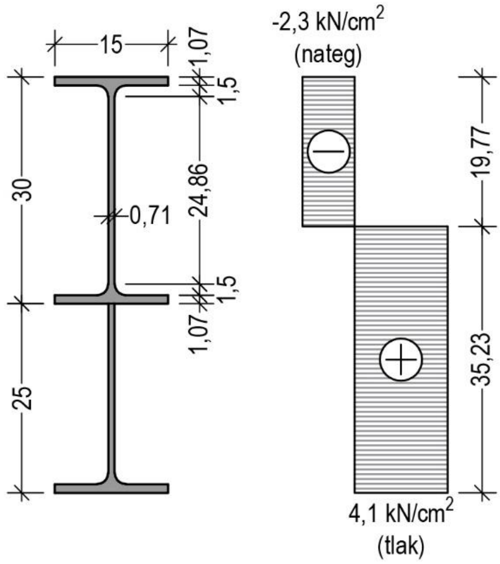
$$\frac{h_w}{t_w} = \frac{(44 - 2 \cdot 2,1)\ \text{cm}}{1,15\ \text{cm}} = 34,61 \leq 72 \cdot \frac{\varepsilon}{\eta} = 72 \cdot \frac{1,0}{1,2} = 60,00$$

Stojina HEA 450 je kompaktna (ni nevarnosti lokalnega izbočenja pločevine).

$$\frac{h_w}{t_w} = \frac{(35 - 2,1)\ \text{cm}}{1,15\ \text{cm}} = 28,61 \leq 72 \cdot \frac{\varepsilon}{\eta} = 72 \cdot \frac{1,0}{1,2} = 60,00$$

Stojina vute je kompaktna (ni nevarnosti lokalnega izbočenja pločevine).

Prečka IPE 300 in vuta 250, S235



Slika 81: IPE 300 in vuta, geometrija ter normalne napetosti

- stojina vute (notranji tlačeni del)

$$\frac{c}{t_w} = \frac{(25 - 1{,}07 - 1{,}5)\ \text{cm}}{0{,}71\ \text{cm}} = 31{,}59 \leq 33 \cdot \varepsilon = 33{,}00$$

- stojina IPE 300 (notranji tlačeni del)

$$\frac{c}{t_w} = \frac{(30 - 2 \cdot 1{,}07 - 2 \cdot 1{,}5)\ \text{cm}}{0{,}71\ \text{cm}} = 35{,}01 \leq \frac{36 \cdot \varepsilon}{\alpha} = \frac{36 \cdot 1{,}0}{0{,}341} = 105{,}57$$

$$\alpha = 1 - \frac{19{,}77\ \text{cm}}{30\ \text{cm}} = 0{,}341$$

- pasnica (previsni deli pasnic)

$$\frac{c}{t_f} = \frac{\left(\frac{15}{2} - \frac{0{,}71}{2} - 1{,}5\right)\ \text{cm}}{1{,}07\ \text{cm}} = 5{,}28 \leq 9 \cdot \varepsilon = 9{,}00$$

Prerez spada v 1. razred kompaktnosti (plastični prerez).

- prečni prerezi v strigu

$$\frac{h_w}{t_w} = \frac{(30 - 2 \cdot 1{,}07)\ \text{cm}}{1{,}07\ \text{cm}} = 26{,}04 \leq 72 \cdot \frac{\varepsilon}{\eta} = 72 \cdot \frac{1{,}0}{1{,}2} = 60{,}00$$

Stojina IPE 300 je kompaktna (ni nevarnosti lokalnega izbočenja pločevine).

$$\frac{h_w}{t_w} = \frac{(25 - 1{,}07)\ \text{cm}}{1{,}07\ \text{cm}} = 22{,}36 \leq 72 \cdot \frac{\varepsilon}{\eta} = 72 \cdot \frac{1{,}0}{1{,}2} = 60{,}00$$

Stojina vute je kompaktna (ni nevarnosti lokalnega izbočenja pločevine).

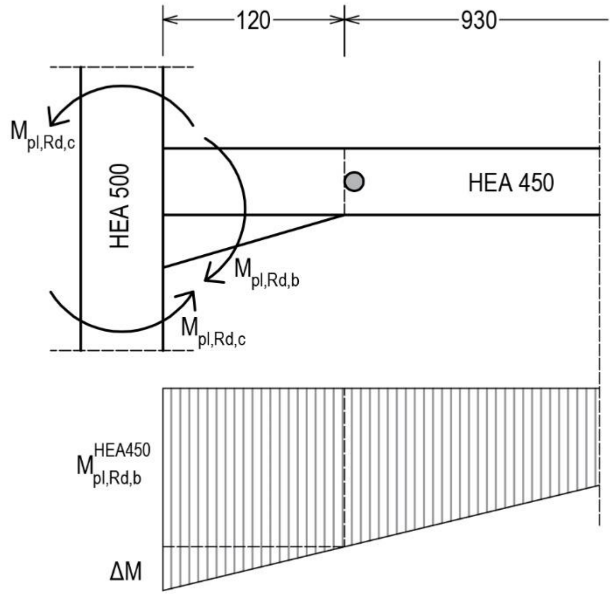
8.3.8.5 Globalna duktilnost

Pogoj duktilnosti za večetažne okvirne konstrukcije (SIST EN 1998-1, poglavje 4.4.2.3)

$$\sum M_{R,c} \geq 1{,}3 \sum M_{R,b}$$

$\sum M_{R,c}$ je vsota projektnih upogibnih nosilnosti stebrov, ki se stikajo v vozlišču.
$\sum M_{R,b}$ je vsota projektnih upogibnih nosilnosti nosilcev, ki se stikajo v vozlišču.

Pogoj duktilnosti preverimo za vozlišče medetaže momentnega okvira.



Slika 82: Vozlišče medetaže

$$M_{R,c} = M_{pl,Rd,c} = W_{pl,y,c} \cdot \frac{f_y}{\gamma_{M0}} = 3949\ \text{cm}^3 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 92802\ \text{kNcm}$$

$W_{pl,y,c} = 3949\ \text{cm}^3$ steber HEA 500

$$M_{R,b} = M_{pl,Rd,b} = M_{pl,Rd,b}^{HEA450} + \Delta M = 75576\ \text{kNcm} + 19503\ \text{kNcm} = 95079\ \text{kNcm}$$

$$M_{pl,Rd,b}^{HEA450} = W_{pl,y,b} \cdot \frac{f_y}{\gamma_{M0}} = 3216\ \text{cm}^3 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 75576\ \text{kNcm}$$

$W_{pl,y,b} = 3216\ \text{cm}^3$ prečka HEA 450

$$\Delta M = V_{pl,M} \cdot L_{vuta} = 162{,}53\ \text{kN} \cdot 120\ \text{cm} = 19503\ \text{kNcm}$$

$$V_{pl,M} = \frac{2 \cdot M_{pl,Rd,b}^{HEA450}}{L^{HEA450}} = \frac{2 \cdot 75576\ \text{kNcm}}{930\ \text{cm}} = 162{,}53\ \text{kN}$$

$$2 \cdot M_{R,c} = 185604\ \text{kNcm} \geq 1{,}3 \cdot M_{R,b} = 123603\ \text{kNcm}\ ✓$$

Plastični členek se razvije v prečki medetaže, tik pred začetkom vute.

8.3.8.6 Vpliv teorije drugega reda

Vpliv teorije drugega reda (SIST EN 1998, poglavje 4.4.2.2)

1. etaža

$$\theta_1 = \frac{P_{tot,1} \cdot d_{r,1}}{V_{tot,1} \cdot h_1} = \frac{470 \text{ kN} \cdot 86{,}60 \text{ mm}}{69{,}31 \text{ kN} \cdot 5440 \text{ mm}} = 0{,}108 \leq 0{,}100 \quad ✗$$

$$d_{r,1} = d_{e,1} \cdot q_X = 21{,}65 \text{ mm} \cdot 4 = 86{,}60 \text{ mm}$$

$$h_1 = 5440 \text{ mm}$$

$$P_{tot,1} = 2 \cdot 235 \text{ kN} = 470 \text{ kN}$$

$$V_{tot,1} = 34{,}55 \text{ kN} + 34{,}76 \text{ kN} = 69{,}31 \text{ kN}$$

2. etaža

$$\theta_2 = \frac{P_{tot,2} \cdot d_{r,2}}{V_{tot,2} \cdot h_2} = \frac{61{,}82 \text{ kN} \cdot 25{,}40 \text{ mm}}{14{,}15 \text{ kN} \cdot 3200 \text{ mm}} = 0{,}035 \leq 0{,}100 \quad ✓$$

$$d_{r,2} = (d_{e,2} - d_{e,1}) \cdot q_X = (28{,}00 \text{ mm} - 21{,}65 \text{ mm}) \cdot 4 = 25{,}40 \text{ mm}$$

$$h_2 = 3200 \text{ mm}$$

$$P_{tot,2} = 2 \cdot 31{,}41 \text{ kN} = 61{,}82 \text{ kN}$$

$$V_{tot,2} = 7{,}93 \text{ kN} + 6{,}22 \text{ kN} = 14{,}15 \text{ kN}$$

Vpliv teorije drugega reda je potrebno upoštevati s faktorjem k_δ, saj pogoja nista izpolnjena v obeh etažah.

$$k_\delta = \frac{1}{1-\theta} = \frac{1}{1-0{,}108} = 1{,}12$$

$$\theta = \max\{\theta_1; \theta_2\} = 0{,}108$$

S faktorjem k_δ pomnožimo horizontalni vpliv (seizmični del) potresnega projektnega stanja.

8.3.8.7 Dimenzioniranje prečke medetaže HEA 450, S235

Posebna pravila za pomične okvire (SIST EN 1998-1, poglavje 6.6.2).

Obremenitev

$$M_{Ed} = M_{Ed,G} + k_\delta \cdot M_{Ed,E} = 159{,}04 \text{ kNm} + 1{,}12 \cdot 149{,}09 \text{ kNm} = 326{,}02 \text{ kNm}$$

$$N_{Ed} = N_{Ed,G} + k_\delta \cdot N_{Ed,E} = 46{,}67 \text{ kN} - 1{,}12 \cdot 28{,}54 \text{ kN} = 14{,}71 \text{ kN (nateg)}$$

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} = 147{,}76 \text{ kN} + 162{,}53 \text{ kN} = 310{,}29 \text{ kN}$$

$$V_{Ed,M} = \frac{2\, M_{pl,y,Rd,b}}{L} = \frac{2 \cdot 75576 \text{ kNcm}}{930 \text{ cm}} = 162{,}53 \text{ kN}$$

Kontrola upogibne nosilnosti

$$\frac{M_{Ed}}{M_{pl,y,Rd}} = \frac{32602 \text{ kNcm}}{75576 \text{ kNcm}} = 0{,}43 \leq 1{,}0 \quad ✓$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 3216 \text{ cm}^3 \cdot \frac{23{,}5 \text{ kN/cm}^2}{1{,}0} = 75576 \text{ kNcm}$$

$$W_{pl,y} = 3216 \text{ cm}^3 \qquad \text{prečka HEA 450}$$

Kontrola osne sile

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{14{,}71\ \text{kN}}{4183{,}00\ \text{kN}} = 0{,}004 \leq 0{,}15 \ \checkmark$$

$$N_{pl,Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 178{,}00\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 4183{,}00\ \text{kN}$$

$$A = 178{,}00\ \text{cm}^2 \qquad \text{prečka HEA 450}$$

Kontrola prečne sile

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{310{,}29\ \text{kN}}{1163{,}57\ \text{kN}} = 0{,}27 \leq 0{,}5 \ \checkmark$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 85{,}76\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 1163{,}57\ \text{kN}$$

$$A_v = \max \begin{cases} A - 2\,b\,t_f + (t_w + 2\,r)\,t_f = \\ = 198 - 2 \cdot 30 \cdot 2{,}1 + (1{,}15 + 2 \cdot 2{,}7) \cdot 2{,}1 = 85{,}76\ cm^2 \\ \eta\,h_w\,t_w = \\ = 1{,}2 \cdot 39{,}8 \cdot 1{,}15 = 54{,}92\ cm^2 \end{cases}$$

8.3.8.8 Dimenzioniranje prečke strehe IPE 300, S235

Posebna pravila za pomične okvire (SIST EN 1998-1, poglavje 6.6.2).

Obremenitev

$$M_{Ed} = M_{Ed,G} + k_\delta \cdot M_{Ed,E} = 5{,}75\ \text{kNm} + 1{,}12 \cdot 19{,}31\ \text{kNm} = 27{,}38\ \text{kNm}$$

$$N_{Ed} = N_{Ed,G} + k_\delta \cdot N_{Ed,E} = 75{,}29\ \text{kN} + 1{,}12 \cdot 5{,}76\ \text{kN} = 81{,}74\ \text{kN (tlak)}$$

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} = 13{,}68\ \text{kN} + 33{,}69\ \text{kN} = 47{,}37\ \text{kN}$$

$$V_{Ed,M} = \frac{2\,M_{pl,y,Rd,b}}{L} = \frac{2 \cdot 14758\ \text{kNcm}}{876\ \text{cm}} = 33{,}69\ \text{kN}$$

Kontrola upogibne nosilnosti

$$\frac{M_{Ed}}{M_{pl,y,Rd}} = \frac{2738\ \text{kNcm}}{14758\ \text{kNcm}} = 0{,}19 \leq 1{,}0 \ \checkmark$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 628\ \text{cm}^3 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 14758\ \text{kNcm}$$

$$W_{pl,y} = 1420{,}32\ \text{cm}^3 \quad \text{prečka IPE 300}$$

Kontrola osne sile

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{81{,}74\ \text{kN}}{1264{,}30\ \text{kN}} = 0{,}06 \leq 0{,}15 \ \checkmark$$

$$N_{pl,Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 53{,}80\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 1264{,}30\ \text{kN}$$

$$A = 53{,}80\ \text{cm}^2 \qquad \text{prečka IPE 300}$$

Kontrola prečne sile

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{33{,}69\ \text{kN}}{344{,}76\ \text{kN}} = 0{,}1 \leq 0{,}5\ \checkmark$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 25{,}41\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 344{,}76\ \text{kN}$$

$$A_v = \max \begin{cases} A - 2\,b\,t_f + (t_w + 2\,r)\,t_f = \\ = 53{,}80 - 2 \cdot 15 \cdot 1{,}07 + (0{,}71 + 2 \cdot 1{,}5) \cdot 1{,}07 = 25{,}41\ cm^2 \\ \eta\,h_w\,t_w = \\ = 1{,}2 \cdot 27{,}86 \cdot 0{,}71 = 23{,}74\ cm^2 \end{cases}$$

8.3.8.9 Dimenzioniranje stebra HEA 500, S235

Steber se poveča iz HEA 450 na HEA 500 zaradi pogoja »Nosilnost prerezov, osno upogibna nosilnost«.

Posebna pravila za pomične okvire (SIST EN 1998-1, poglavje 6.6.3).

Obremenitev

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot k_\delta \cdot M_{Ed,E}$$

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot k_\delta \cdot N_{Ed,E}$$

$$V_{Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot k_\delta \cdot V_{Ed,E}$$

kjer so

γ_{ov} faktor dodatne nosilnosti (SIST EN 1998-1: 6.2), $\gamma_{ov} = 1{,}25$

Ω najmanjša vrednost Ω_i za vse nosilce (prečke) z območij sipanja

$$\Omega_i = \frac{M_{pl,y,Rd,i}\,\{f_y\}}{M_{Ed,i}\,\{M_G + M_E\}}$$

$$\Omega = \min \begin{cases} \Omega_1 = \dfrac{M_{pl,y,Rd}^{(1)}}{M_{Ed}^{(1)}} = \dfrac{75576\ \text{kNcm}}{32602\ \text{kNcm}} = 2{,}3 \\ \Omega_2 = \dfrac{M_{pl,y,Rd}^{(2)}}{M_{Ed}^{(2)}} = \dfrac{14758\ \text{kNcm}}{2738\ \text{kNcm}} = 5{,}4 \end{cases}$$

Prerez stebra ob vpetju 1-1 (slika 83)

$$M_{Ed}^{1-1} = 0$$

$$N_{Ed}^{1-1} = 235{,}01\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 36{,}10\ \text{kN} = 362{,}88\ \text{kN}$$

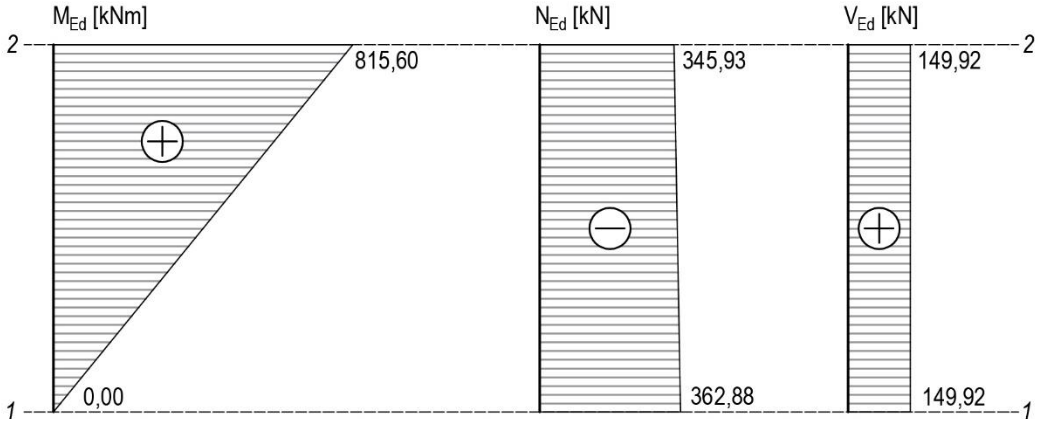
$$V_{Ed}^{1-1} = 26{,}80\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 34{,}76\ \text{kN} = 149{,}92\ \text{kN}$$

Prerez stebra pod prvo etažo 2-2 (slika 83)

$$M_{Ed}^{2-2} = 145{,}77\ \text{kNm} + 1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 189{,}11\ \text{kNm} = 815{,}60\ \text{kNm}$$

$$N_{Ed}^{2-2} = 218{,}06\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 36{,}10\ \text{kN} = 345{,}93\ \text{kN}$$

$$V_{Ed}^{2-2} = 26{,}80\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 34{,}76\ \text{kN} = 149{,}92\ \text{kN}$$



Slika 83: Diagrami notranjih količin za steber

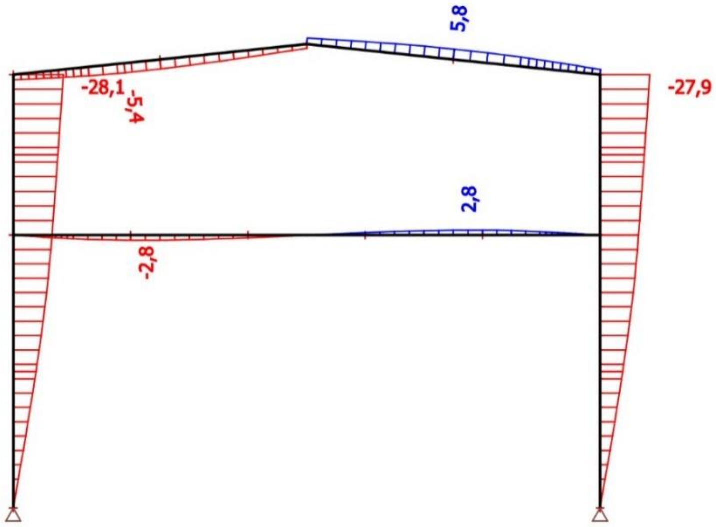
Kontrola zunanjih stebrov pomičnega okvira za potresnoodporne jeklene stavbe se izvede pri okviru s centričnim povezjem, kjer se upošteva kombinacija obremenitve iz obeh smeri X in Y.

8.3.9 Kontrola poškodb (kontrola relativnih pomikov etaž)

Pogoj za stavbe, pri katerih so nekonstrukcijski elementi pritrjeni na konstrukcijo tako, da deformacije konstrukcije nanje ne vplivajo (SIST EN 1998-1, poglavje 4.4.3.2)

$$d_{r,i} \cdot \nu \leq 0{,}010 \cdot h_i$$

$$\nu\ (\gamma_I = 1{,}0) = 0{,}5$$



Slika 84: Horizontalni pomiki etaž za potresni del obtežbe [mm]

$$d_{r,1} = d_{e,1} \cdot q_X = \left(\frac{2{,}17\text{ cm} + 2{,}16\text{ cm}}{2}\right) \cdot 4 = 2{,}17\ cm \cdot 4 = 8{,}68\text{ cm}$$

$$d_{r,2} = d_{e,1} \cdot q_X = \left(\frac{2{,}81\text{ cm} + 2{,}79\text{ cm}}{2} - 2{,}17\text{ cm}\right) \cdot 4 = 2{,}52\text{ cm}$$

1. etaža

$$d_{r,1} \cdot \nu = 8{,}68\text{ cm} \cdot 0{,}5 = 4{,}34\text{ cm} \leq 0{,}010 \cdot h_1 = 0{,}010 \cdot 544\text{ cm} = 5{,}44\text{ cm} \quad \checkmark$$

2. etaža - streha

$$d_{r,2} \cdot \nu = 2{,}52\text{ cm} \cdot 0{,}5 = 1{,}26\text{ cm} \leq 0{,}010 \cdot h_2 = 0{,}010 \cdot 320\text{ cm} = 3{,}20\text{ cm} \quad \checkmark$$

8.3.10 Kontrola nosilnosti v vzdolžni smeri – centrično povezje (sistem nateznih diagonal)

8.3.10.1 Gravitacijski del obtežbe

Gravitacijski del obtežbe na diagonale nima vpliva, vpliv na stebre je že zajet v analizi prečne smeri X. Vpliva samo na prečke.

$$G_k + \psi_2 \cdot Q_k$$

$$\psi_2 = \varphi \cdot \psi_{2,1} = 0{,}8 \cdot 0{,}3 = 0{,}24$$

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija $g_m = 5{,}35$ kN/m

  $g_m = g'_m\, b' = 4{,}57\ \text{kN/m}^2 \cdot 1{,}17$ m

Spremenljiva obtežba

- koristna obtežba medetaže $q_m = 5{,}03$ kN/m

  $q_m = q'_m\, b' = 4{,}30\ \text{kN/m}^2 \cdot 1{,}17$ m

8.3.10.2 Potresni del obtežbe

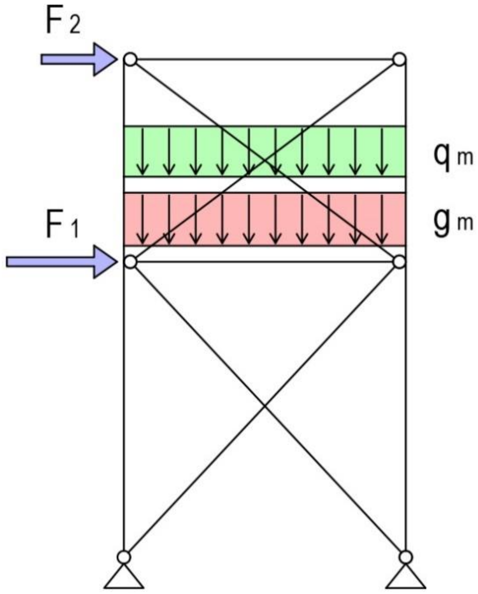
Potresni del obtežbe se razdeli na dve povezji.

$$F_1 = \frac{1}{2} \cdot \gamma_I \cdot F_1^{(A)} \cdot \delta_{A,C} = \frac{1}{2} \cdot 1{,}0 \cdot 137{,}15\ \text{kN} \cdot 1{,}6 = 109{,}72\ \text{kN}$$

$$F_2 = \frac{1}{2} \cdot \gamma_I \cdot F_2^{(A)} \cdot \delta_{A,C} = \frac{1}{2} \cdot 1{,}0 \cdot 35{,}19\ \text{kN} \cdot 1{,}6 = 28{,}15\ \text{kN}$$

$\gamma_I = 1{,}0$ faktor pomembnosti stavbe (kategorija II - običajne stavbe)

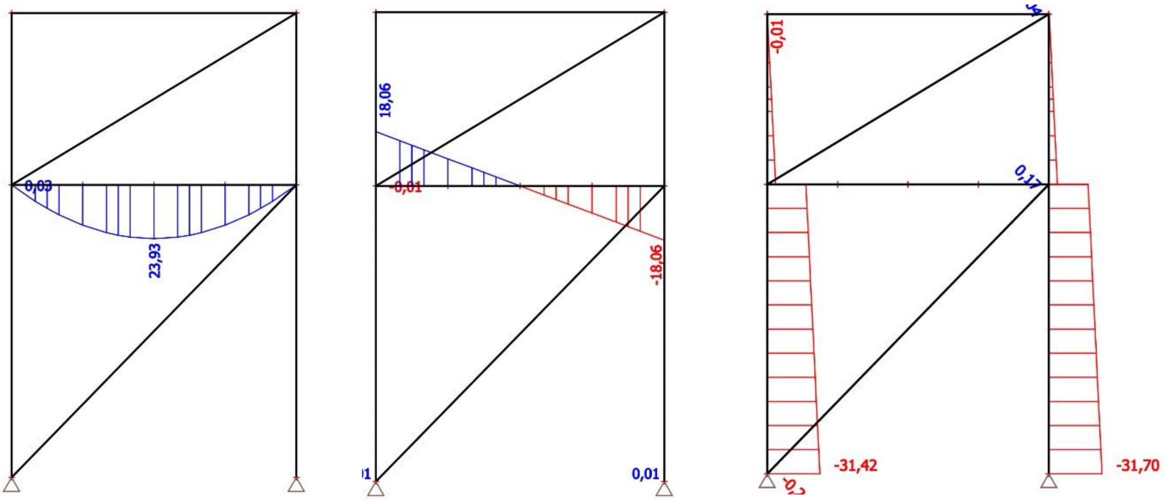
Na sliki 85 je prikazana razporeditev gravitacijske in potresne obtežbe na centrično povezje v programu SCIA Engineer.



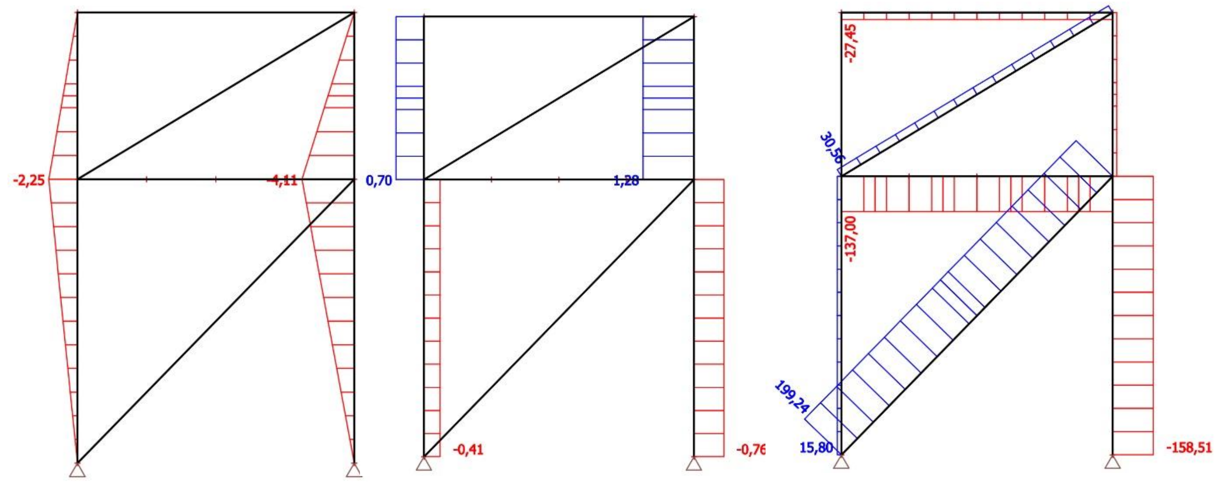
Slika 85: Obtežba na centričnem povezju

8.3.10.3 Analiza centričnega povezja

Notranje sile in momenti za gravitacijski in potresni del obtežbe so prikazani na slikah 86 in 87.



Slika 86: Momenti M [kNm], prečne sile V [kN] in osne sile N [kN], gravitacijska obtežba



Slika 87: Momenti M [kNm], prečne sile V [kN] in osne sile N [kN], potresna obtežba

8.3.10.4 Diagonalni elementi

Posebna pravila za okvire s centričnimi povezji (SIST EN 1998-1, poglavje 6.7.1)

$$\frac{A^+ - A^-}{A^+ + A^-} = 0 \leq 0{,}05 \quad \checkmark$$

kjer sta A^+ in A^- površini vodoravnih projekcij prečnih prerezov nateznih diagonal, pri čemer imajo vodoravni potresni vplivi pozitivno ali negativno smer. Velja za diagonali v obeh etažah.

Vsi prerezi so v 1. razredu kompaktnosti (SCIA Engineer).

8.3.10.4.1 Dimenzioniranje diagonale (B7) - L 25 x 3, S235

Notranje količine

$$N_{Ed} = 30{,}56 \text{ kN (nateg)}$$

Nosilnost prereza

- nateg

$$N_{t,Rd} = \frac{A\, f_y}{\gamma_{M0}} = \frac{1{,}42 \text{ cm}^2 \cdot 23{,}5 \text{ kN/cm}^2}{1{,}0} = 33{,}37 \text{ kN} \geq N_{Ed} = 30{,}56 \text{ kN} \checkmark$$

8.3.10.4.2 Dimenzioniranje diagonale (B8) - U 65, S235

Notranje količine

$$N_{Ed} = 199{,}24 \text{ kN (nateg)}$$

Nosilnost prereza

- nateg

$$N_{t,Rd} = \frac{A\, f_y}{\gamma_{M0}} = \frac{9{,}03 \text{ cm}^2 \cdot 23{,}5 \text{ kN/cm}^2}{1{,}0} = 212{,}21 \text{ kN} \geq N_{Ed} = 199{,}25 \text{ kN} \checkmark$$

8.3.10.5 Nosilci in stebri

Posebna pravila za okvire s centričnimi povezji (SIST EN 1998-1, poglavje 6.7.4). Faktor dodatne nosilnosti za nosilce in stebre, ki so obremenjeni z osno silo

$$\Omega = \min \frac{N_{pl,Rd,i}}{N_{Ed,i}}$$

$N_{pl,Rd,i}$ projektna nosilnost diagonale *i* pri nominalni trdnosti $f_{y,nom}$
$N_{Ed,i}$ projektna vrednost osne sile v isti diagonali *i* pri potresnem projektnem stanju

Enakomerno sipanje v diagonalah se zagotovi z omejitvijo razlike velikosti največjega in najmanjšega faktorja dodatne nosilnosti

$$\frac{\Omega_{max}}{\Omega_{min}} \leq 1{,}25$$

Vsi prerezi so v 1. razredu kompaktnosti (SCIA Engineer).

Izračun faktorjev dodatne nosilnosti

$$\Omega_1 = \frac{N_{pl,Rd,1}}{N_{Ed,1}} = \frac{212{,}21 \text{ kN}}{199{,}25 \text{ kN}} = 1{,}07$$

$$N_{pl,Rd,1} = A_1 \cdot \frac{f_y}{\gamma_{M0}} = 9{,}03 \text{ cm}^2 \cdot \frac{23{,}5 \text{ kN/cm}^2}{1{,}0} = 212{,}21 \text{ kN}$$

$$\Omega_2 = \frac{N_{pl,Rd,2}}{N_{Ed,2}} = \frac{33{,}37 \text{ kN}}{30{,}56 \text{ kN}} = 1{,}09$$

$$N_{pl,Rd,2} = A_2 \cdot \frac{f_y}{\gamma_{M0}} = 1{,}42\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 33{,}37\ \text{kN}$$

$$\frac{\Omega_{max}}{\Omega_{min}} = \frac{1{,}09}{1{,}07} = 1{,}02 \le 1{,}25\ \checkmark$$

$$\Omega = \min\,\{\Omega_1; \Omega_2\} = 1{,}07$$

8.3.10.5.1 Vpliv teorije drugega reda

Vpliv teorije drugega reda (SIST EN 1998, poglavje 4.4.2.2)

1. etaža

$$\theta_1 = \frac{P_{tot,1} \cdot d_{r,1}}{V_{tot,1} \cdot h_1} = \frac{63{,}40\ \text{kN} \cdot 48{,}60\ \text{mm}}{137{,}87\ \text{kN} \cdot 5440\ \text{mm}} = 0{,}004 \le 0{,}100\ \checkmark$$

$$d_{r,1} = d_{e,1} \cdot q_X = 12{,}15\ \text{mm} \cdot 4 = 48{,}60\ \text{mm}$$

$$h_1 = 5440\ \text{mm}$$

$$P_{tot,1} = 2 \cdot 31{,}70\ \text{kN} = 63{,}40\ \text{kN}$$

$$V_{tot,1} = F_1 + F_2 = 109{,}72\ \text{kN} + 28{,}15\ \text{kN} = 137{,}87\ \text{kN}$$

2. etaža

$$\theta_2 = \frac{P_{tot,2} \cdot d_{r,2}}{V_{tot,2} \cdot h_2} = \frac{10{,}14\ \text{kN} \cdot 34{,}00\ \text{mm}}{28{,}15\ \text{kN} \cdot 3200\ \text{mm}} = 0{,}004 \le 0{,}100\ \checkmark$$

$$d_{r,2} = (d_{e,2} - d_{e,1}) \cdot q_X = (20{,}65\ \text{mm} - 12{,}15\ \text{mm}) \cdot 4 = 34{,}00\ \text{mm}$$

$$h_2 = 3200\ \text{mm}$$

$$P_{tot,2} = 2 \cdot 5{,}07\ \text{kN} = 10{,}14\ \text{kN}$$

$$V_{tot,2} = F_2 = 28{,}15\ \text{kN}$$

Vpliv teorije drugega reda ni potrebno upoštevati, saj sta pogoja izpolnjena za obe etaži.

8.3.10.5.2 Dimenzioniranje prečke (B6) - Cev 88,9 x 3, S235

Notranje količine

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} = 0\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 1{,}07 \cdot 27{,}45\ \text{kN} = 40{,}39\ \text{kN (tlak)}$$

$\gamma_{ov} = 1{,}25$ faktor dodatne nosilnosti (SIST EN 1998-1, poglavje 6.2)

Nosilnost prereza

- tlak

$$N_{c,Rd} = \frac{A\, f_y}{\gamma_{M0}} = \frac{8{,}10\ \text{cm}^2 \cdot 23{,}5\ \text{kN/cm}^2}{1{,}0} = 190{,}35\ \text{kN} \ge N_{Ed} = 40{,}39\ \text{kN}\ \checkmark$$

Uklon tlačenega elementa

$$L_u = 530\ \text{cm}$$

$$N_{b,Rd} = \frac{\chi\, A\, f_y}{\gamma_{M1}} = \frac{0{,}26 \cdot 8{,}10\ cm^2 \cdot 23{,}5\ \text{kN/cm}^2}{1{,}0} = 49{,}49\ \text{kN} \ge N_{Ed,E} = 40{,}39\ \text{kN}\ \checkmark$$

$$uklonska\ krivulja\ a \longrightarrow \chi = 0{,}26$$

$$\bar{\lambda} = \frac{L_u}{i\ \lambda_1} = \frac{530\ \text{cm}}{3{,}04\ \text{cm} \cdot 93{,}9} = 1{,}86$$

$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{74{,}76\ \text{cm}^4}{8{,}10\ \text{cm}^2}} = 3{,}04\ \text{cm}$$

$$I = 74{,}76\ \text{cm}^4$$

$$\lambda_1 = 93{,}9\ \varepsilon = 93{,}9$$

8.3.10.5.3 Dimenzioniranje prečke (B5) - IPE 220, S235

Obremenitev

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} = 23{,}93\ \text{kNm} + 0\ \text{kNm} = 23{,}93\ \text{kNm}$$

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} = 0 + 1{,}1 \cdot 1{,}25 \cdot 1{,}07 \cdot 137{,}00 = 201{,}56\ \text{kN (tlak)}$$

$$V_{Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} = 18{,}06\ \text{kN} + 0\ \text{kN} = 18{,}06\ \text{kN}$$

Kontrola osno upogibne nosilnosti

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{201{,}56\ \text{kN}}{784{,}90\ \text{kN}} = 0{,}26 \leq 0{,}25\ \times$$

$$N_{pl,Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 33{,}40\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 784{,}90\ \text{kN}$$

$$A = 33{,}40\ \text{cm}^2 \qquad \text{prečka IPE 220}$$

Osna sila vpliva na zmanjšanje projektne plastične upogibne nosilnosti prereza.

$$\frac{M_{Ed}}{M_{N,y,Rd}} = \frac{2393{,}00\ \text{kNcm}}{6156{,}71\ \text{kNcm}} = 0{,}39 \leq 1{,}0\ \checkmark$$

$$M_{N,y,Rd} = \min \begin{cases} M_{pl,y,Rd} \\ M_{pl,y,Rd} \cdot \dfrac{1-n}{1-0{,}5\ a} \end{cases}$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 285{,}00\ \text{cm}^3 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 6697{,}50\ \text{kNcm}$$

$$W_{pl,y} = 285{,}00\ \text{cm}^3 \qquad \text{prečka IPE 220}$$

$$M_{pl,y,Rd} \cdot \frac{1-n}{1-0{,}5\ a} = 6697{,}50\ \text{kNcm} \cdot \frac{1-0{,}26}{1-0{,}5 \cdot 0{,}39} = 6156{,}71\ \text{kNcm}$$

$$n = \frac{N_{Ed}}{N_{pl,Rd}} = \frac{201{,}56\ \text{kN}}{784{,}90\ \text{kN}} = 0{,}26$$

$$a = \min \begin{cases} \dfrac{A - 2\ b\ t_f}{A} = \dfrac{33{,}40 - 2 \cdot 11 \cdot 0{,}92}{33{,}40} = 0{,}39 \\ 0{,}5 \end{cases}$$

$$M_{N,y,Rd} = 6156{,}71\ \text{kNcm}$$

Kontrola strižne nosilnosti

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{18{,}06 \text{ kN}}{215{,}86 \text{ kN}} = 0{,}08 \leq 0{,}5 \quad \checkmark$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 15{,}91 \text{ cm}^2 \cdot \frac{23{,}5 \text{ kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 215{,}86 \text{ kN}$$

$$A_v = \max \begin{cases} A - 2\, b\, t_f + (t_w + 2\, r)\, t_f = \\ = 33{,}40 - 2 \cdot 11 \cdot 0{,}92 + (0{,}59 + 2 \cdot 1{,}2) \cdot 0{,}92 = 15{,}91\ cm^2 \\ \eta\, h_w\, t_w = \\ = 1{,}2 \cdot 20{,}16 \cdot 0{,}59 = 14{,}27\ cm^2 \end{cases}$$

8.3.10.5.4 Dimenzioniranje stebra (B2) - HEA 500, S235

Kombinacija potresnega vpliva za smer X in Y (SIST EN 1998-1, poglavje 4.3.3.5.1)

$a)\ E_{Edx}$ "+" $0{,}30\ E_{Edy}$

$b)\ 0{,}30\ E_{Edx}$ "+" E_{Edy}

Kombinacije vplivov

$a)\ E_{Edx}$ "+" $0{,}30\ E_{Edy}$

Prerez 1-1 stebra ob vpetju (slika 88)

$$M_{Ed}^{1-1} = M_{Ed,G} + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot M_{Edx,E}\right) + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot M_{Edy,E}\right) =$$
$$= 0 + (0) + 0{,}30 \cdot (0) =$$
$$= 0 \text{ kNm}$$

$$N_{Ed}^{1-1} = N_{Ed,G} + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot N_{Edx,E}\right) + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot N_{Edy,E}\right) =$$
$$= 235{,}01 + (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 36{,}10) + 0{,}30 \cdot (1{,}1 \cdot 1{,}25 \cdot 1{,}07 \cdot 158{,}51) =$$
$$= 235{,}01 \text{ kN} + 127{,}87 \text{ kN} + 69{,}96 \text{ kN} = 432{,}84 \text{ kN}$$

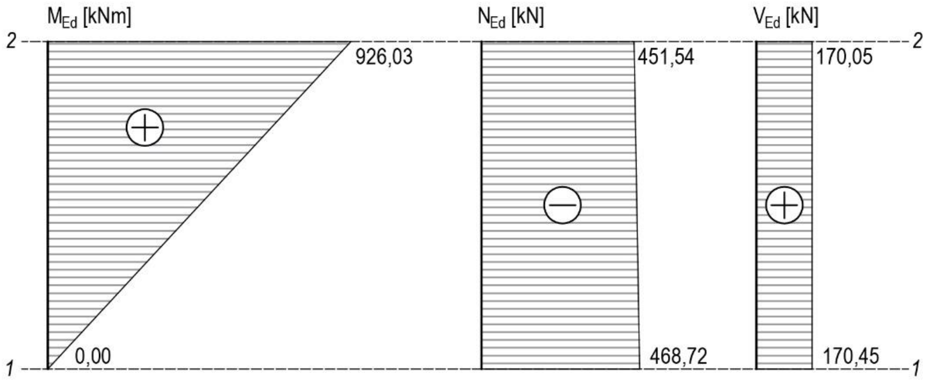
$$V_{Ed}^{1-1} = V_{Ed,G} + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot V_{Edx,E}\right) + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot V_{Edy,E}\right) =$$
$$= 26{,}80 + (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 34{,}76) + 0{,}30 \cdot (0) =$$
$$= 26{,}80 \text{ kN} + 123{,}12 \text{ kN} + 0 \text{ kN} = 149{,}92 \text{ kN}$$

Prerez 2-2 stebra pod prvo etažo (slika 88)

$$M_{Ed}^{2-2} = M_{Ed,G} + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot M_{Edx,E}\right) + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot M_{Edy,E}\right) =$$
$$= 145{,}77 + (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 189{,}11) + 0{,}30 \cdot (0) =$$
$$= 815{,}60 \text{ kNm}$$

$$N_{Ed}^{2-2} = N_{Ed,G} + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot N_{Edx,E}\right) + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot N_{Edy,E}\right) =$$
$$= 218{,}06 + (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 36{,}10) + 0{,}30 \cdot (1{,}1 \cdot 1{,}25 \cdot 1{,}07 \cdot 158{,}51) =$$
$$= 218{,}06 \text{ kN} + 127{,}87 \text{ kN} + 69{,}96 \text{ kN} = 415{,}89 \text{ kN}$$

$$V_{Ed}^{2-2} = V_{Ed,G} + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot V_{Edx,E}\right) + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot V_{Edy,E}\right) =$$
$$= 26{,}80 + (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 34{,}76) + 0{,}30 \cdot (0) =$$
$$= 149{,}92 \text{ kN}$$



Slika 88: Kombinacija a), diagrami notranjih količin za steber

$b)$ 0,30 E_{Edx} "+" E_{Edy}

Prerez 1-1 stebra ob vpetju (slika 89)

$$M_{Ed}^{1-1} = M_{Ed,G} + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot M_{Edx,E}\right) + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot M_{Edy,E}\right) =$$

$$= 0 + 0{,}30 \cdot (0) + (0) =$$

$$= 0 \text{ kNm}$$

$$N_{Ed}^{1-1} = N_{Ed,G} + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot N_{Edx,E}\right) + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot N_{Edy,E}\right) =$$

$$= 235{,}01 + 0{,}30 \cdot (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 36{,}10) + (1{,}1 \cdot 1{,}25 \cdot 1{,}07 \cdot 158{,}51) =$$

$$= 235{,}01 \text{ kN} + 38{,}36 \text{ kN} + 233{,}20 \text{ kN} = 506{,}57 \text{ kN}$$

$$V_{Ed}^{1-1} = V_{Ed,G} + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot V_{Edx,E}\right) + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot V_{Edy,E}\right) =$$

$$= 26{,}80 + 0{,}30 \cdot (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 34{,}76) + (0) =$$

$$= 26{,}80 \text{ kN} + 36{,}94 \text{ kN} + 0 \text{ kN} = 63{,}74 \text{ kN}$$

Prerez 2-2 stebra pod prvo etažo

$$M_{Ed}^{2-2} = M_{Ed,G} + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot M_{Edx,E}\right) + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot M_{Edy,E}\right) =$$

$$= 145{,}77 + 0{,}30 \cdot (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 189{,}11) + (0) =$$

$$= 346{,}72 \text{ kNm}$$

$$N_{Ed}^{2-2} = N_{Ed,G} + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot N_{Edx,E}\right) + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot N_{Edy,E}\right) =$$

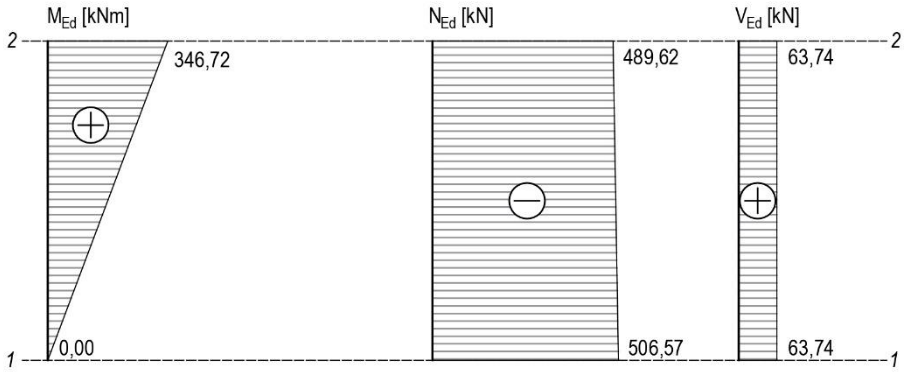
$$= 218{,}06 + 0{,}30 \cdot (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 36{,}10) + (1{,}1 \cdot 1{,}25 \cdot 1{,}07 \cdot 158{,}51) =$$

$$= 218{,}06 \text{ kN} + 38{,}36 \text{ kN} + 233{,}20 \text{ kN} = 489{,}62 \text{ kN}$$

$$V_{Ed}^{2-2} = V_{Ed,G} + 0{,}30 \cdot \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{precka} \cdot k_\delta \cdot V_{Edx,E}\right) + \left(1{,}1 \cdot \gamma_{ov} \cdot \Omega^{diag} \cdot V_{Edy,E}\right) =$$

$$= 26{,}80 + 0{,}30 \cdot (1{,}1 \cdot 1{,}25 \cdot 2{,}3 \cdot 1{,}12 \cdot 34{,}76) + (0) =$$

$$= 63{,}74 \text{ kN}$$



Slika 89: Kombinacija b), diagrami notranjih količin za steber

Merodajna kombinacija obtežb iz obeh smeri je $a)\ E_{Edx}$ "+" $0{,}30\ E_{Edy}$.

Kontrola osno upogibne nosilnosti

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{432{,}84\text{ kN}}{4653{,}00\text{ kN}} = 0{,}09 \leq 0{,}25\ \checkmark$$

$$N_{pl,Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 198{,}00\text{ cm}^2 \cdot \frac{23{,}5\text{ kN/cm}^2}{1{,}0} = 4653{,}00\text{ kN}$$

$$A = 198{,}00\text{ cm}^2 \qquad \text{steber HEA 500}$$

$$N_{Ed} = 432{,}84\text{ kN} \leq \frac{0{,}5\ h_w\ t_w\ f_y}{\gamma_{M0}} = \frac{0{,}5 \cdot 44{,}4 \cdot 1{,}20 \cdot 23{,}5}{1{,}0} = 626{,}04\text{ kN}\ \checkmark$$

Osna sila ne vpliva na zmanjšanje projektne plastične upogibne nosilnosti prereza.

$$\frac{M_{Ed}}{M_{pl,y,Rd}} = \frac{81560\text{ kNcm}}{92801{,}50\text{ kNcm}} = 0{,}88 \leq 1{,}0\ \checkmark$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 3949\text{ cm}^3 \cdot \frac{23{,}5\text{ kN/cm}^2}{1{,}0} = 92801{,}50\text{ kNcm}$$

$$W_{pl,y} = 3949\text{ cm}^3 \qquad \text{steber HEA 500}$$

Kontrola strižne nosilnosti

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{149{,}92\text{ kN}}{1020{,}02\text{ kN}} = 0{,}15 \leq 0{,}5\ \checkmark$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 75{,}18\text{ cm}^2 \cdot \frac{23{,}5\text{ kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 1020{,}02\text{ kN}$$

$$A_v = \max\begin{cases} A - 2\ b\ t_f + (t_w + 2\ r)\ t_f = \\ = 198 - 2 \cdot 30 \cdot 2{,}3 + (1{,}2 + 2 \cdot 2{,}7) \cdot 2{,}3 = 75{,}18\ cm^2 \\ \eta\ h_w\ t_w = \\ = 1{,}2 \cdot 44{,}4 \cdot 1{,}2 = 63{,}94\ cm^2 \end{cases}$$

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$$N_{b,Rd} = \chi\, A\, \frac{f_y}{\gamma_{M1}} \geq N_{Ed}$$

$$\chi = \min \begin{cases} \dfrac{1}{\phi + \sqrt{\phi^2 - \overline{\lambda}^2}} = \dfrac{1}{0{,}544 + \sqrt{0{,}544^2 - 0{,}27^2}} = 0{,}984 \\ 1{,}0 \end{cases}$$

$$\phi = 0{,}5\left[1 + \alpha\left(\overline{\lambda} - 0{,}2\right) + \overline{\lambda}^2\right] =$$

$$= 0{,}5 \cdot [1 + 0{,}21 \cdot (0{,}27 - 0{,}2) + 0{,}27^2] = 0{,}544$$

$$\alpha = 0{,}21 \qquad \text{(SIST EN 1993-1-1: 6.3.1.2)}$$

$$\overline{\lambda} = \sqrt{\frac{A\, f_y}{N_{cr}}} = \sqrt{\frac{198\ \text{cm}^2 \cdot 23{,}5\ \text{kN/cm}^2}{64171\ \text{kN}}} = 0{,}27$$

$$N_{cr} = \frac{\pi^2\, E\, I_y}{l_u^2} = \frac{\pi^2 \cdot 21000\,\text{kN/cm}^2 \cdot 86970\ \text{cm}^4}{(544\ \text{cm})^2} = 64171\ \text{kN}$$

$$\frac{N_{Ed}}{N_{cr}} = \frac{432{,}84\ kN}{64171\ kN} = 0{,}007 \leq 0{,}04$$

Vpliv uklona se lahko zanemari.

- bočna zvrnitev upogibno obremenjenih elementov

$$M_{b,Rd} = \chi_{LT}\, W_{pl,y}\, \frac{f_y}{\gamma_{M1}} = 0{,}979 \cdot 3949 \cdot \frac{23{,}5}{1{,}0} = 90853\ \text{kNcm} \geq M_{y,Ed} = 81560\ \text{kNm}\ \checkmark$$

$$\chi_{LT} = \min \begin{cases} \dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\, \overline{\lambda}_{LT}^2}} = \dfrac{1}{0{,}599 + \sqrt{0{,}599^2 - 0{,}75 \cdot 0{,}49^2}} = 0{,}979 \\ \dfrac{1}{\overline{\lambda}_{LT}^2} = \dfrac{1}{0{,}49^2} = 4{,}165 \\ 1{,}0 \end{cases}$$

$$\phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}\right) + \beta\, \overline{\lambda}_{LT}^2\right] =$$

$$= 0{,}5 \cdot [1 + 0{,}21 \cdot (0{,}49 - 0{,}40) + 0{,}75 \cdot 0{,}49^2] = 0{,}599$$

$$\alpha_{LT} = 0{,}21 \qquad \text{(SIST EN 1993-1-1: 6.3.2.2)}$$

$$\overline{\lambda}_{LT} = \sqrt{\frac{W_{pl,y}\, f_y}{M_{cr}}} = \sqrt{\frac{3949\ \text{cm}^2 \cdot 23{,}5\ \text{kN/cm}^2}{383030\ \text{kNcm}}} = 0{,}49 \geq \overline{\lambda}_{LT,0} = 0{,}40$$

$$M_{cr} = C_1 \frac{\pi}{k\, L} \sqrt{E\, I_z\, G\, I_t + \frac{\pi^2\, E\, I_\omega\, E\, I_z}{(k_\omega\, L)^2}} = 1{,}77 \cdot \frac{\pi}{1{,}0 \cdot 544} \cdot$$

$$\sqrt{21000 \cdot 10370\ \cdot 8077 \cdot 309 + \frac{\pi^2 \cdot 21000 \cdot 5643000 \cdot 21000 \cdot 10370}{(1{,}0 \cdot 544)^2}} = 383030\ \text{kNcm}$$

$$C_1 = 1{,}77\ (k_z = 1{,}0)$$

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\chi_y \frac{N_{pl,Rd}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,pl,Rd}}{\gamma_{M1}}} \leq 1$$

$$\frac{432{,}84 \text{ kN}}{0{,}984 \cdot 4653 \text{ kN}/1{,}0} + 0{,}628 \cdot \frac{81560 \text{ kNcm}}{0{,}979 \cdot 92801{,}5 \text{ kNcm}/1{,}0} = 0{,}09 + 0{,}56 = 0{,}65 \leq 1 \checkmark$$

$$k_{yy} = \min \begin{cases} C_{my}\left(1 + (\bar{\lambda} - 0{,}2)\frac{N_{Ed}}{\chi_y\ N_{pl,Rd}/\gamma_{M1}}\right) \\ C_{my}\left(1 + 0{,}8\frac{N_{Ed}}{\chi_y\ N_{pl,Rd}/\gamma_{M1}}\right) \end{cases}$$

$$k_{yy} = \min \begin{cases} 0{,}6 \cdot \left(1 + (0{,}27 - 0{,}2) \cdot \frac{432{,}84 \text{ kN}}{0{,}984 \cdot 4653 \text{ kN}/1{,}0}\right) = 0{,}628 \\ 0{,}6 \cdot \left(1 + 0{,}8 \cdot \frac{432{,}84 \text{ kN}}{0{,}984 \cdot 4653 \text{ kN}/1{,}0}\right) = 0{,}645 \end{cases}$$

Faktor nadomestnega upogibnega momenta

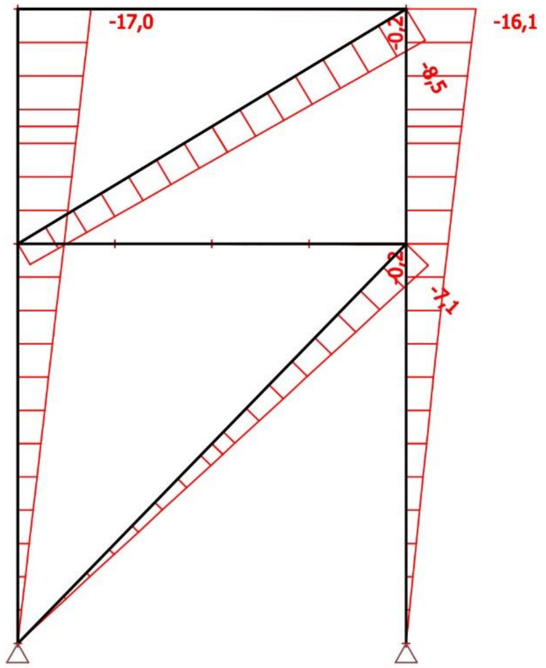
$$C_{my} = 0{,}6 + 0{,}4\,\psi = 0{,}6 + 0{,}4 \cdot 0 = 0{,}6 \leq 0{,}4$$

8.3.11 Kontrola poškodb (kontrola relativnih pomikov etaž) – okvir A

Pogoj za stavbe, pri katerih so nekonstrukcijski elementi pritrjeni na konstrukcijo tako, da deformacije konstrukcije nanje ne vplivajo (SIST EN 1998-1, poglavje 4.4.3.2)

$$d_{r,i} \cdot \nu \leq 0{,}010 \cdot h_i$$

$$\nu\ (\gamma_I = 1{,}0) = 0{,}5$$



Slika 90: Horizontalni pomiki etaž za potresni del obtežbe [mm]

$$d_{r,1} = d_{e,1} \cdot q_Y = \left(\frac{1{,}27\ \text{cm} + 1{,}16\ \text{cm}}{2}\right) \cdot 4 = 1{,}22\ \text{cm} \cdot 4 = 4{,}88\ \text{cm}$$

$$d_{r,2} = d_{e,1} \cdot q_Y = \left(\frac{2{,}11\ \text{cm} + 2{,}02\ \text{cm}}{2} - 1{,}22\ \text{cm}\right) \cdot 4 = 3{,}40\ \text{cm}$$

1. etaža

$$d_{r,1} \cdot \nu = 4{,}88\ \text{cm} \cdot 0{,}5 = 2{,}44\ \text{cm} \leq 0{,}010 \cdot h_1 = 0{,}010 \cdot 544\ \text{cm} = 5{,}44\ \text{cm} \quad ✓$$

2. etaža - streha

$$d_{r,2} \cdot \nu = 3{,}40\ \text{cm} \cdot 0{,}5 = 1{,}70\ \text{cm} \leq 0{,}010 \cdot h_2 = 0{,}010 \cdot 320\ \text{cm} = 3{,}20\ \text{cm} \quad ✓$$

8.3.12 Kontrola nosilnosti v prečni smeri - okvir 1 s centričnim povezjem (sistem nateznih diagonal)

8.3.12.1 Gravitacijski del obtežbe

$$G_k + \psi_2 \cdot Q_k$$

$$\psi_2 = \varphi \cdot \psi_{2,1} = 0{,}8 \cdot 0{,}3 = 0{,}24$$

Lastna teža

Lastna teža jeklenih nosilcev okvira je zajeta v izračunu s programom SCIA Engineer.

Stalna obtežba

- medetažna konstrukcija $g_m =$ 12,47 kN/m
- streha $g_s =$ 1,59 kN/m
- fasada $g_{f1} =$ 1,69 kN/m

$g_{f2} =$ 1,73 kN/m

Spremenljiva obtežba

- koristna obtežba medetaže $q_m =$ 11,40 kN/m

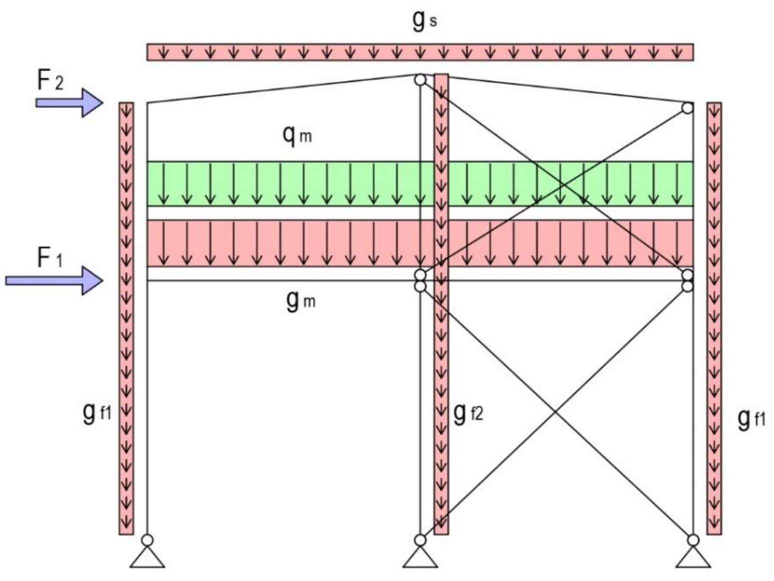
8.3.12.2 Potresni del obtežbe

$$F_1 = \gamma_I \cdot F_1^{(1)} \cdot \delta_{1,7} = 1{,}0 \cdot 26{,}76\ \text{kN} \cdot 1{,}6 = 42{,}82\ \text{kN}$$

$$F_2 = \gamma_I \cdot F_2^{(1)} \cdot \delta_{1,7} = 1{,}0 \cdot 6{,}87\ \text{kN} \cdot 1{,}6 = 10{,}99\ \text{kN}$$

$\gamma_I = 1{,}0$ faktor pomembnosti stavbe (kategorija II - običajne stavbe)

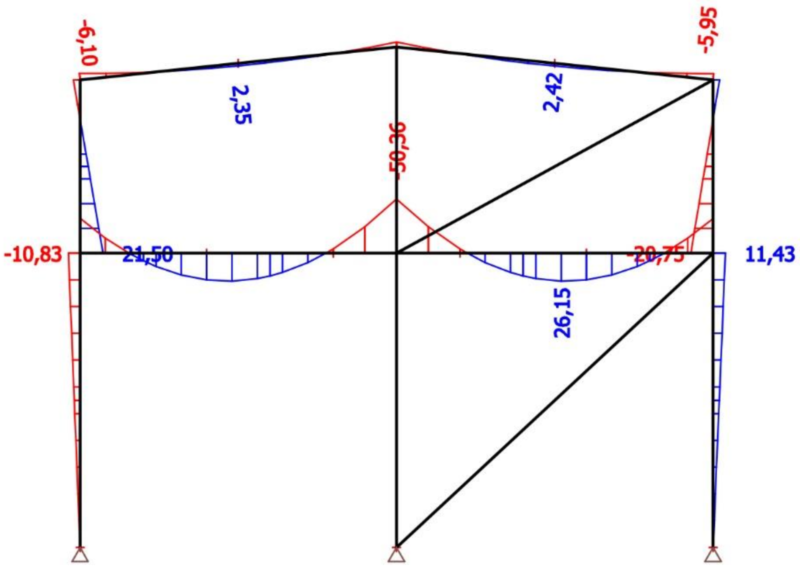
Na sliki 91 je prikazana razporeditev gravitacijske in potresne obtežbe na okvir 1 v programu SCIA Engineer.



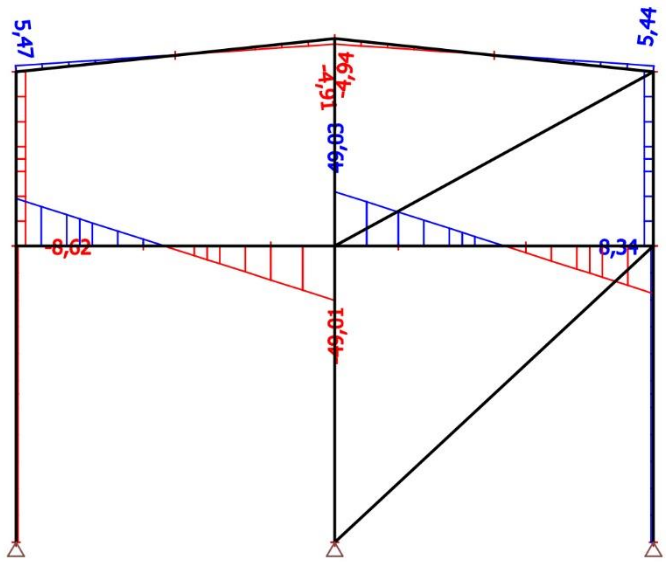
Slika 91: Obtežba na okviru 1 s centričnim povezjem

8.3.12.3 Analiza okvira 1 s centričnim povezjem

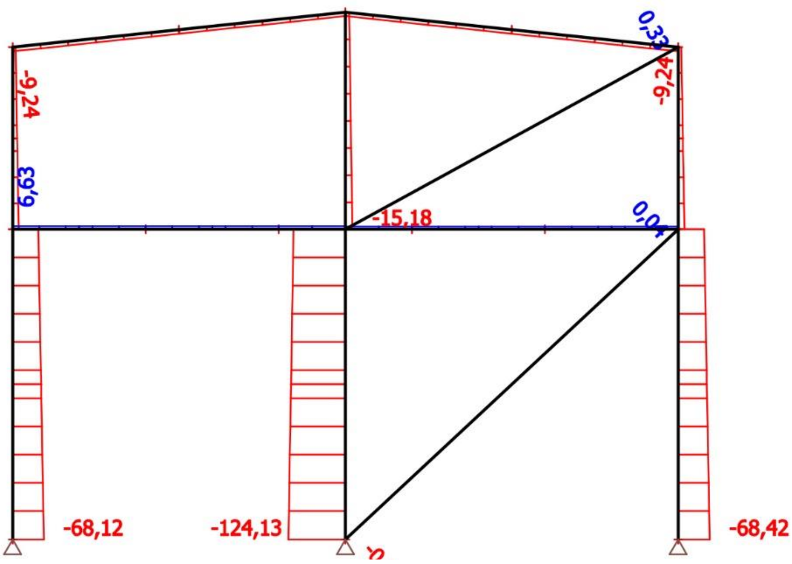
Notranje sile in momenti za gravitacijski in potresni del obtežbe so prikazani na slikah 92 do 97.



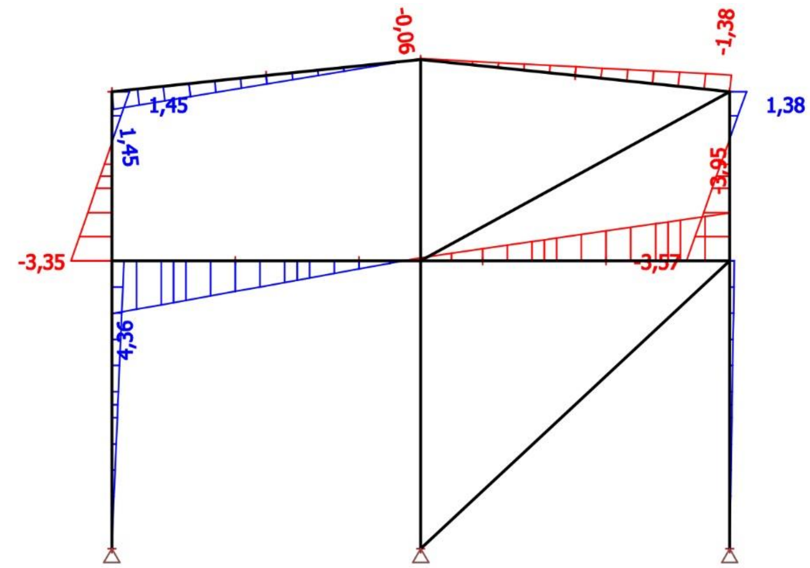
Slika 92: Momenti M_y [kNm], gravitacijska obtežba



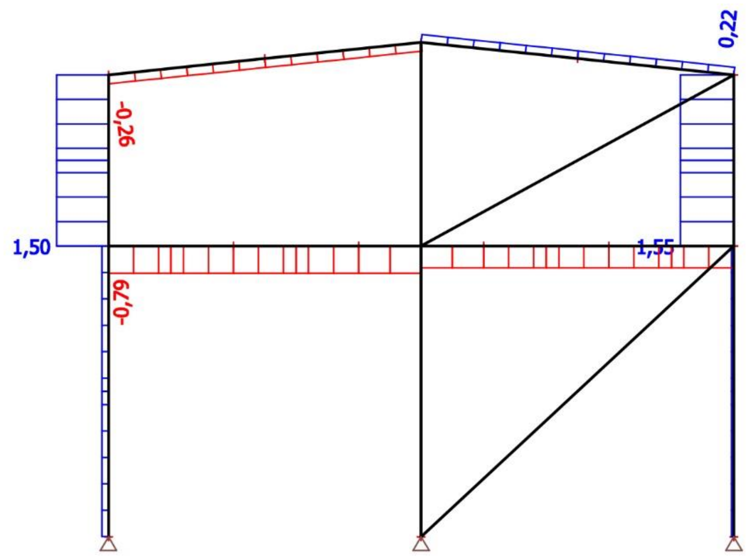
Slika 93: Prečne sile V [kN], gravitacijska obtežba



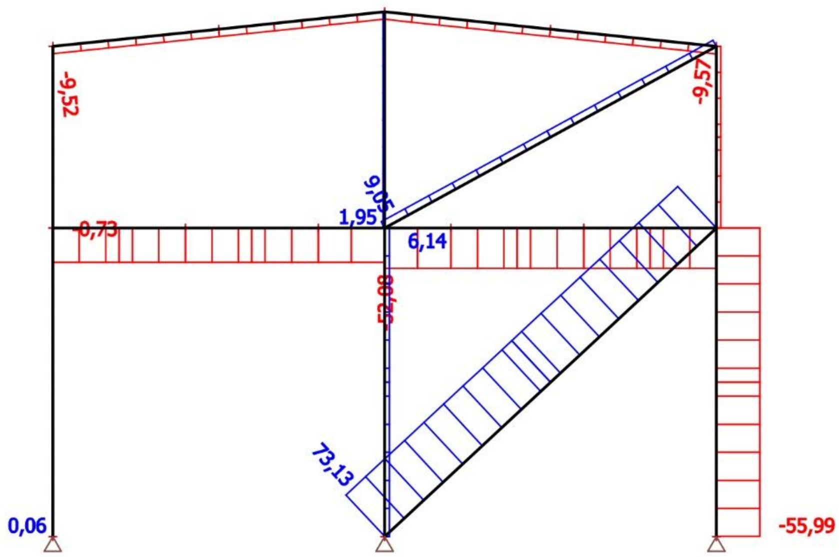
Slika 94: Osne sile N [kN], gravitacijska obtežba



Slika 95: Momenti M_y [kNm], potresna obtežba



Slika 96: Prečne sile V [kN], potresna obtežba



Slika 97: Osne sile N [kN], potresna obtežba

8.3.12.4 Diagonalni elementi

Posebna pravila za okvire s centričnimi povezji (SIST EN 1998-1, poglavje 6.7.1)

$$\frac{A^+ - A^-}{A^+ + A^-} = 0 \leq 0{,}05 \quad \checkmark$$

kjer sta A^+ in A^- površini vodoravnih projekcij prečnih prerezov nateznih diagonal, pri čemer imajo vodoravni potresni vplivi pozitivno ali negativno smer. Velja za diagonali v obeh etažah.

Vsi prerezi so v 1. razredu kompaktnosti (SCIA Engineer).

8.3.12.4.1 Dimenzioniranje diagonale (B7) - Palica Ø12, S235

Notranje količine

$$N_{Ed} = 9{,}05 \text{ kN (nateg)}$$

Nosilnost prereza

- nateg

$$N_{t,Rd} = \frac{A\, f_y}{\gamma_{M0}} = \frac{1{,}13 \text{ cm}^2 \cdot 23{,}5 \text{ kN/cm}^2}{1{,}0} = 26{,}56 \text{ kN} \geq N_{Ed} = 9{,}05 \text{ kN} \quad \checkmark$$

8.3.12.4.2 Dimenzioniranje diagonale (B8) - U 65, S235

Notranje količine

$$N_{Ed} = 73{,}13 \text{ kN (nateg)}$$

Nosilnost prereza

- nateg

$$N_{t,Rd} = \frac{A\, f_y}{\gamma_{M0}} = \frac{9{,}03 \text{ cm}^2 \cdot 23{,}5 \text{ kN/cm}^2}{1{,}0} = 212{,}21 \text{ kN} \geq N_{Ed} = 73{,}13 \text{ kN} \quad \checkmark$$

8.3.12.5 Nosilci in stebri

Posebna pravila za okvire s centričnimi povezji (SIST EN 1998-1, poglavje 6.7.4). Faktor dodatne nosilnosti za nosilce in stebre, ki so obremenjeni z osno silo

$$\Omega = \min \frac{N_{pl,Rd,i}}{N_{Ed,i}}$$

$N_{pl,Rd,i}$ projektna nosilnost diagonale *i* pri nominalni trdnosti $f_{y,nom}$
$N_{Ed,i}$ projektna vrednost osne sile v isti diagonali *i* pri potresnem projektnem stanju

Enakomerno sipanje v diagonalah se zagotovi z omejitvijo razlike velikosti največjega in najmanjšega faktorja dodatne nosilnosti

$$\frac{\Omega_{max}}{\Omega_{min}} \leq 1{,}25$$

Vsi prerezi so v 1. razredu kompaktnosti (SCIA Engineer).

Izračun faktorjev dodatne nosilnosti

$$\Omega_1 = \frac{N_{pl,Rd,1}}{N_{Ed,1}} = \frac{212{,}21\ \mathrm{kN}}{73{,}13\ \mathrm{kN}} = 2{,}90$$

$$N_{pl,Rd,1} = A_1 \cdot \frac{f_y}{\gamma_{M0}} = 9{,}03\ \mathrm{cm}^2 \cdot \frac{23{,}5\ \mathrm{kN/cm}^2}{1{,}0} = 212{,}21\ \mathrm{kN}$$

$$\Omega_2 = \frac{N_{pl,Rd,2}}{N_{Ed,2}} = \frac{26{,}56\ \mathrm{kN}}{9{,}05\ \mathrm{kN}} = 2{,}93$$

$$N_{pl,Rd,2} = A_2 \cdot \frac{f_y}{\gamma_{M0}} = 1{,}13\ \mathrm{cm}^2 \cdot \frac{23{,}5\ \mathrm{kN/cm}^2}{1{,}0} = 26{,}56\ \mathrm{kN}$$

$$\frac{\Omega_{max}}{\Omega_{min}} = \frac{2{,}93}{2{,}90} = 1{,}01 \leq 1{,}25\ \checkmark$$

$$\Omega = \min\ \{\Omega_1; \Omega_2\} = 2{,}90$$

8.3.12.5.1 Vpliv teorije drugega reda

Vpliv teorije drugega reda (SIST EN 1998, poglavje 4.4.2.2)

1. etaža

$$\theta_1 = \frac{P_{tot,1} \cdot d_{r,1}}{V_{tot,1} \cdot h_1} = \frac{260{,}67\ \mathrm{kN} \cdot 18{,}40\ \mathrm{mm}}{53{,}81\ \mathrm{kN} \cdot 5440\ \mathrm{mm}} = 0{,}016 \leq 0{,}100\ \checkmark$$

$$d_{r,1} = d_{e,1} \cdot q_X = 4{,}60\ \mathrm{mm} \cdot 4 = 18{,}40\ \mathrm{mm}$$

$$h_1 = 5440\ \mathrm{mm}$$

$$P_{tot,1} = 68{,}12\ \mathrm{kN} + 124{,}13\ \mathrm{kN} + 68{,}42\ \mathrm{kN} = 260{,}67\ \mathrm{kN}$$

$$V_{tot,1} = F_1 + F_2 = 42{,}82\ \mathrm{kN} + 10{,}99\ \mathrm{kN} = 53{,}81\ \mathrm{kN}$$

2. etaža

$$\theta_2 = \frac{P_{tot,2} \cdot d_{r,2}}{V_{tot,2} \cdot h_2} = \frac{42{,}11\ \mathrm{kN} \cdot 13{,}00\ \mathrm{mm}}{10{,}99\ \mathrm{kN} \cdot 3200\ \mathrm{mm}} = 0{,}016 \leq 0{,}100\ \checkmark$$

$$d_{r,2} = (d_{e,2} - d_{e,1}) \cdot q_X = (7{,}85\ \mathrm{mm} - 4{,}60\ \mathrm{mm}) \cdot 4 = 13{,}00\ \mathrm{mm}$$

$$h_2 = 3200\ \mathrm{mm}$$

$$P_{tot,1} = 13{,}39\ \mathrm{kN} + 15{,}18\ \mathrm{kN} + 13{,}54\ \mathrm{kN} = 42{,}11\ \mathrm{kN}$$

$$V_{tot,2} = F_2 = 10{,}99\ \mathrm{kN}$$

Vpliv teorije drugega reda ni potrebno upoštevati, saj sta pogoja izpolnjena za obe etaži.

8.3.12.5.2 Dimenzioniranje prečke medetaže HEA 220, S235

Obremenitev

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} = 32{,}18\ \text{kNm} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 3{,}95\ \text{kNm} = 47{,}93\ \text{kNm}$$

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} = -6{,}65\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 52{,}08\ \text{kN} = 201{,}02\ \text{kN (tlak)}$$

$$V_{Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} = 42{,}82\ \text{kN} + 0\ \text{kN} = 42{,}82\ \text{kN}$$

Kontrola upogibne nosilnosti

$$\frac{M_{Ed}}{M_{pl,y,Rd}} = \frac{4793\ \text{kNcm}}{13348\ \text{kNcm}} = 0{,}36 \leq 1{,}0\ \checkmark$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 568\ \text{cm}^3 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 13348\ \text{kNcm}$$

$W_{pl,y} = 568\ \text{cm}^3$ prečka HEA 220

Kontrola osne sile

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{201{,}02\ \text{kN}}{1511{,}05\ \text{kN}} = 0{,}13 \leq 0{,}15\ \checkmark$$

$$N_{pl,Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 64{,}30\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 1511{,}05\ \text{kN}$$

$A = 64{,}30\ \text{cm}^2$ prečka HEA 220

Kontrola prečne sile

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{42{,}82\ \text{kN}}{279{,}90\ \text{kN}} = 0{,}15 \leq 0{,}5\ \checkmark$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 20{,}63\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 279{,}90\ \text{kN}$$

$$A_v = \max \begin{cases} A - 2\,b\,t_f + (t_w + 2\,r)\,t_f = \\ = 64{,}3 - 2 \cdot 22 \cdot 1{,}1 + (0{,}7 + 2 \cdot 1{,}8) \cdot 1{,}1 = 20{,}63\ cm^2 \\ \eta\, h_w\, t_w = \\ = 1{,}2 \cdot 18{,}8 \cdot 0{,}7 = 15{,}79\ cm^2 \end{cases}$$

8.3.12.5.3 Dimenzioniranje prečke strehe IPE 180, S235

Obremenitev

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} = 5{,}95\ \text{kNm} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 1{,}38\ \text{kNm} = 11{,}45\ \text{kNm}$$

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} = 9{,}24\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 9{,}57\ \text{kN} = 47{,}40\ \text{kN (tlak)}$$

$$V_{Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} = 5{,}44\ \text{kN} + 0\ \text{kN} = 5{,}44\ \text{kN}$$

Kontrola upogibne nosilnosti

$$\frac{M_{Ed}}{M_{pl,y,Rd}} = \frac{1145\ \text{kNcm}}{3910{,}40\ \text{kNcm}} = 0{,}29 \leq 1{,}0\ \checkmark$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 166{,}40\ \text{cm}^3 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 3910{,}40\ \text{kNcm}$$

$$W_{pl,y} = 166{,}40\ \text{cm}^3 \quad \text{prečka IPE 180}$$

Kontrola osne sile

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{47{,}40\ \text{kN}}{561{,}65\ \text{kN}} = 0{,}08 \leq 0{,}15\ \checkmark$$

$$N_{pl,Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 23{,}90\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 561{,}65\ \text{kN}$$

$$A = 23{,}90\ \text{cm}^2 \quad \text{prečka IPE 180}$$

Kontrola prečne sile

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{5{,}44\ \text{kN}}{151{,}96\ \text{kN}} = 0{,}04 \leq 0{,}5\ \checkmark$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 11{,}20\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 151{,}96\ \text{kN}$$

$$A_v = \max \begin{cases} A - 2\,b\,t_f + (t_w + 2\,r)\,t_f = \\ = 23{,}90 - 2 \cdot 9{,}1 \cdot 0{,}8 + (0{,}53 + 2 \cdot 0{,}9) \cdot 0{,}8 = 11{,}20\ cm^2 \\ \eta\, h_w\, t_w = \\ = 1{,}2 \cdot 16{,}4 \cdot 0{,}53 = 10{,}43\ cm^2 \end{cases}$$

8.3.12.5.4 Dimenzioniranje stebra HEA 220, S235

Obremenitev

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E}$$

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E}$$

$$V_{Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E}$$

Prerez stebra ob vpetju 1-1

$$M_{Ed}^{1-1} = 0$$

$$N_{Ed}^{1-1} = 68{,}43\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 56{,}00\ \text{kN} = 314{,}83\ \text{kN}$$

$$V_{Ed}^{1-1} = 2{,}10\ \text{kN} + 0\ \text{kN} = 2{,}10\ \text{kN}$$

Prerez stebra pod prvo etažo 2-2

$$M_{Ed}^{2-2} = 11{,}43\ \text{kNm} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 0{,}40\ \text{kNm} = 13{,}19\ \text{kNm}$$

$$N_{Ed}^{2-2} = 56{,}54\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 56{,}00\ \text{kN} = 302{,}94\ \text{kN}$$

$$V_{Ed}^{2-2} = 2{,}10\ \text{kN} + 1{,}1 \cdot 1{,}25 \cdot 2{,}9 \cdot 0{,}07\ \text{kN} = 2{,}41\ \text{kN}$$

Kontrola osno upogibne nosilnosti

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{302{,}94\ \text{kN}}{1511{,}05\ \text{kN}} = 0{,}21 \leq 0{,}25\ \checkmark$$

$$N_{pl,Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 64{,}30\ \text{cm}^2 \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 1511{,}05\ \text{kN}$$

$$A = 64{,}30 \text{ cm}^2 \text{ steber HEA 220}$$

$$N_{Ed} = 314{,}83 \text{ kN} \leq \frac{0{,}5\, h_w\, t_w\, f_y}{\gamma_{M0}} = \frac{0{,}5 \cdot 18{,}8 \cdot 0{,}7 \cdot 23{,}5}{1{,}0} = 154{,}63 \text{ kN} \quad ✗$$

Osna sila vpliva na zmanjšanje projektne plastične upogibne nosilnosti prereza.

$$\frac{M_{Ed}}{M_{N,y,Rd}} = \frac{1319 \text{ kNcm}}{12051 \text{ kNcm}} = 0{,}11 \leq 1{,}0 \quad ✓$$

$$M_{N,y,Rd} = \min \begin{cases} M_{pl,y,Rd} \\ M_{pl,y,Rd} \cdot \dfrac{1-n}{1-0{,}5\, a} \end{cases}$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 568 \text{ cm}^3 \cdot \frac{23{,}5 \text{ kN/cm}^2}{1{,}0} = 13348 \text{ kNcm}$$

$$W_{pl,y} = 568 \text{ cm}^3 \qquad \text{steber HEA 220}$$

$$M_{pl,y,Rd} \cdot \frac{1-n}{1-0{,}5\, a} = 13348 \text{ kNcm} \cdot \frac{1-0{,}21}{1-0{,}5 \cdot 0{,}25} = 12051 \text{ kNcm}$$

$$n = \frac{N_{Ed}}{N_{pl,Rd}} = \frac{302{,}94 \text{ kN}}{1511{,}05 \text{ kN}} = 0{,}21$$

$$a = \min \begin{cases} \dfrac{A - 2\, b\, t_f}{A} = \dfrac{64{,}30 - 2 \cdot 22 \cdot 1{,}1}{64{,}30} = 0{,}25 \\ 0{,}5 \end{cases}$$

$$M_{N,y,Rd} = 12051 \text{ kNcm}$$

Kontrola strižne nosilnosti

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{2{,}41 \text{ kN}}{279{,}90 \text{ kN}} = 0{,}01 \leq 0{,}5 \quad ✓$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = 20{,}63 \text{ cm}^2 \cdot \frac{23{,}5 \text{ kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 279{,}90 \text{ kN}$$

$$A_v = \max \begin{cases} A - 2\, b\, t_f + (t_w + 2\, r)\, t_f = \\ = 64{,}3 - 2 \cdot 22 \cdot 1{,}1 + (0{,}7 + 2 \cdot 1{,}8) \cdot 1{,}1 = 20{,}63\, cm^2 \\ \eta\, h_w\, t_w = \\ = 1{,}2 \cdot 18{,}8 \cdot 0{,}7 = 15{,}79\, cm^2 \end{cases}$$

Odpornost elementov proti nestabilnosti

- uklonska nosilnost

$$N_{b,Rd} = \chi\, A\, \frac{f_y}{\gamma_{M1}} = 0{,}877 \cdot 64{,}3 \text{ cm}^2 \cdot \frac{23{,}5 \text{ kN/cm}^2}{1{,}0} = 1325 \text{ kN} \geq N_{Ed} = 302{,}94 \text{ kN} \quad ✓$$

$$\chi = \min \begin{cases} \dfrac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \dfrac{1}{0{,}744 + \sqrt{0{,}744^2 - 0{,}63^2}} = 0{,}877 \\ 1{,}0 \end{cases}$$

$$\phi = 0{,}5 \left[1 + \alpha \left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] =$$

$$= 0{,}5 \cdot [1 + 0{,}21 \cdot (0{,}63 - 0{,}2) + 0{,}63^2] = 0{,}744$$

$$\alpha = 0{,}21 \qquad \text{(SIST EN 1993-1-1: 6.3.1.2)}$$

$$\bar{\lambda} = \sqrt{\frac{A\, f_y}{N_{cr}}} = \sqrt{\frac{64{,}3\ \text{cm}^2 \cdot 23{,}5\ \ \text{kN/cm}^2}{3789\ \text{kN}}} = 0{,}63$$

$$N_{cr} = \frac{\pi^2\, E\, I_y}{l_u^2} = \frac{\pi^2 \cdot 21000\,\text{kN/cm}^2 \cdot 5410\ \text{cm}^4}{(544\ \text{cm})^2} = 3789\ \text{kN}$$

- bočna zvrnitev upogibno obremenjenih elementov

$$M_{b,Rd} = \chi_{LT}\, W_{pl,y}\, \frac{f_y}{\gamma_{M1}} = 0{,}953 \cdot 568 \cdot \frac{23{,}5}{1{,}0} = 12721\ \text{kNcm} \geq M_{y,Ed} = 1319\ \text{kNm}\ ✓$$

$$\chi_{LT} = \min \begin{cases} \dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\, \bar{\lambda}_{LT}^2}} = \dfrac{1}{0{,}645 + \sqrt{0{,}645^2 - 0{,}75 \cdot 0{,}58^2}} = 0{,}953 \\ \dfrac{1}{\bar{\lambda}_{LT}^2} = \dfrac{1}{0{,}58^2} = 2{,}973 \\ 1{,}0 \end{cases}$$

$$\phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}\right) + \beta\, \bar{\lambda}_{LT}^2\right] =$$

$$= 0{,}5 \cdot [1 + 0{,}21 \cdot (0{,}58 - 0{,}40) + 0{,}75 \cdot 0{,}58^2] = 0{,}645$$

$\alpha_{LT} = 0{,}21$ (SIST EN 1993-1-1: 6.3.2.2)

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y}\, f_y}{M_{cr}}} = \sqrt{\frac{568\ \text{cm}^2 \cdot 23{,}5\ \ \text{kN/cm}^2}{39549\ \text{kNcm}}} = 0{,}58 \geq \bar{\lambda}_{LT,0} = 0{,}40$$

$$M_{cr} = C_1 \frac{\pi}{k\, L} \sqrt{E\, I_z\, G\, I_t + \frac{\pi^2\, E\, I_\omega\, E\, I_z}{(k_\omega\, L)^2}} = 1{,}77 \cdot \frac{\pi}{1{,}0 \cdot 544} \cdot$$

$$\sqrt{21000 \cdot 1950\ \ \cdot 8077 \cdot 28{,}50\ + \frac{\pi^2 \cdot 21000 \cdot 193300 \cdot 21000 \cdot 1950}{(1{,}0 \cdot 544)^2}} = 39549\ \text{kNcm}$$

$C_1 = 1{,}77\ (k_z = 1{,}0)$

- tlačno in upogibno obremenjeni elementi

$$\frac{N_{Ed}}{\chi_y \dfrac{N_{pl,Rd}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} \dfrac{M_{y,pl,Rd}}{\gamma_{M1}}} \leq 1$$

$$\frac{302{,}94\ \text{kN}}{0{,}877 \cdot 1511\ \text{kN}/1{,}0} + 0{,}659 \cdot \frac{1319\ \text{kNcm}}{0{,}953 \cdot 13348\ \text{kNcm}/1{,}0} = 0{,}23 + 0{,}07 = 0{,}30 \leq 1\ ✓$$

$$k_{yy} = \min \begin{cases} C_{my}\left(1 + (\bar{\lambda} - 0{,}2)\dfrac{N_{Ed}}{\chi_y\ N_{pl,Rd}/\gamma_{M1}}\right) \\ C_{my}\left(1 + 0{,}8 \dfrac{N_{Ed}}{\chi_y\ N_{pl,Rd}/\gamma_{M1}}\right) \end{cases}$$

$$k_{yy} = \min \begin{cases} 0{,}6 \cdot \left(1 + (0{,}63 - 0{,}2) \cdot \dfrac{302{,}94\ \text{kN}}{0{,}877 \cdot 1511{,}05\ \text{kN}/1{,}0}\right) = 0{,}659 \\ 0{,}6 \cdot \left(1 + 0{,}8 \cdot \dfrac{302{,}94\ \text{kN}}{0{,}877 \cdot 1511{,}05\ \text{kN}/1{,}0}\right) = 0{,}710 \end{cases}$$

Faktor nadomestnega upogibnega momenta

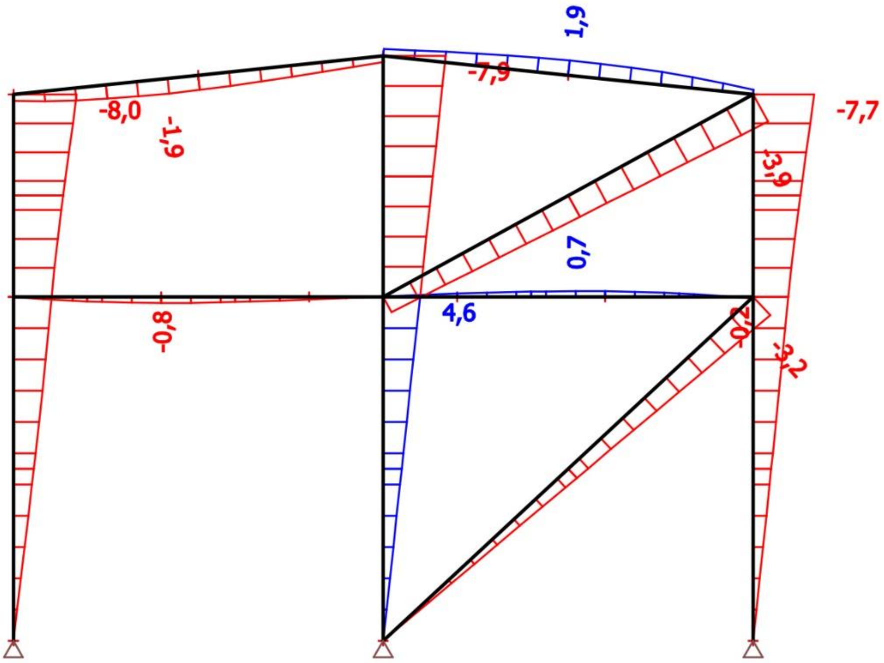
$$C_{my} = 0{,}6 + 0{,}4\,\psi = 0{,}6 + 0{,}4 \cdot 0 = 0{,}6 \leq 0{,}4$$

8.3.13 Kontrola poškodb (kontrola relativnih pomikov etaž) – okvir 1

Pogoj za stavbe, pri katerih so nekonstrukcijski elementi pritrjeni na konstrukcijo tako, da deformacije konstrukcije nanje ne vplivajo (SIST EN 1998-1, poglavje 4.4.3.2)

$$d_{r,i} \cdot \nu \leq 0{,}010 \cdot h_i$$

$$\nu\ (\gamma_I = 1{,}0) = 0{,}5$$



Slika 98: Horizontalni pomiki etaž za potresni del obtežbe [mm]

$$d_{r,1} = d_{e,1} \cdot q_Y = \left(\frac{0{,}48\text{ cm} + 0{,}44\text{ cm}}{2}\right) \cdot 4 = 0{,}46\text{ cm} \cdot 4 = 1{,}84\text{ cm}$$

$$d_{r,2} = d_{e,1} \cdot q_Y = \left(\frac{0{,}80\text{ cm} + 0{,}77\text{ cm}}{2} - 0{,}46\text{ cm}\right) \cdot 4 = 1{,}30\text{ cm}$$

1. etaža

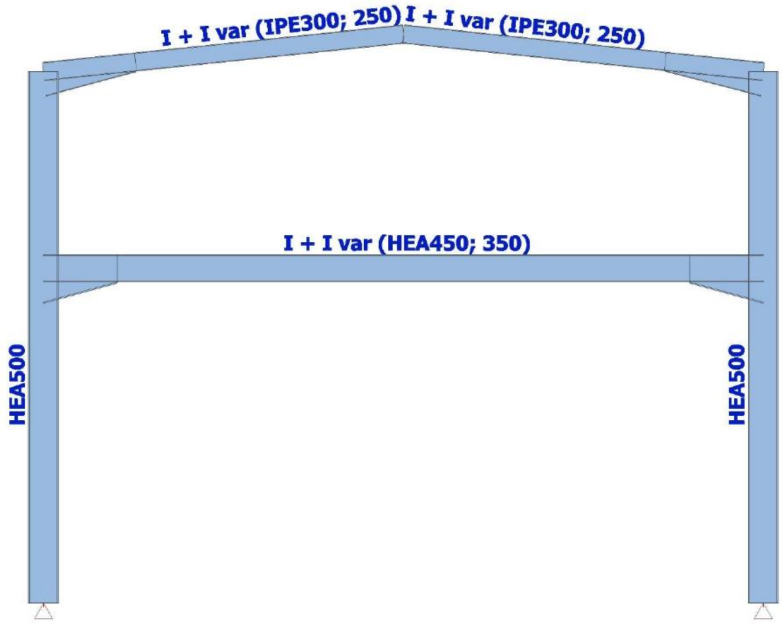
$$d_{r,1} \cdot \nu = 1{,}84\text{ cm} \cdot 0{,}5 = 0{,}92\text{ cm} \leq 0{,}010 \cdot h_1 = 0{,}010 \cdot 544\text{ cm} = 5{,}44\text{ cm} \quad ✓$$

2. etaža - streha

$$d_{r,2} \cdot \nu = 1{,}30\text{ cm} \cdot 0{,}5 = 0{,}65\text{ cm} \leq 0{,}010 \cdot h_2 = 0{,}010 \cdot 320\text{ cm} = 3{,}20\text{ cm} \quad ✓$$

8.4 POVZETEK POTRESNEGA PROJEKTNEGA STANJA NA OBA NAČINA

8.4.1 Momentni okvir 2



Slika 99: Okvir 2, MSN in MSU ter PPS, *q* = 1,5

Preglednica 21: Okvir 2, razred nizke duktilnosti (DCL), faktor obnašanja *q* = 1,5

Element	**MSN in MSU**	**PPS *q* = 1,5**	**Kritičen pogoj povečanja**
Steber	HEA 450	HEA 500	Odpornost elementov proti nestabilnosti, tlačno in upogibno obremenjeni elementi (poglavje 8.2.8.4.1)
Prečka	HEA 450	HEA 450	
Prečka	IPE 300	IPE 300	

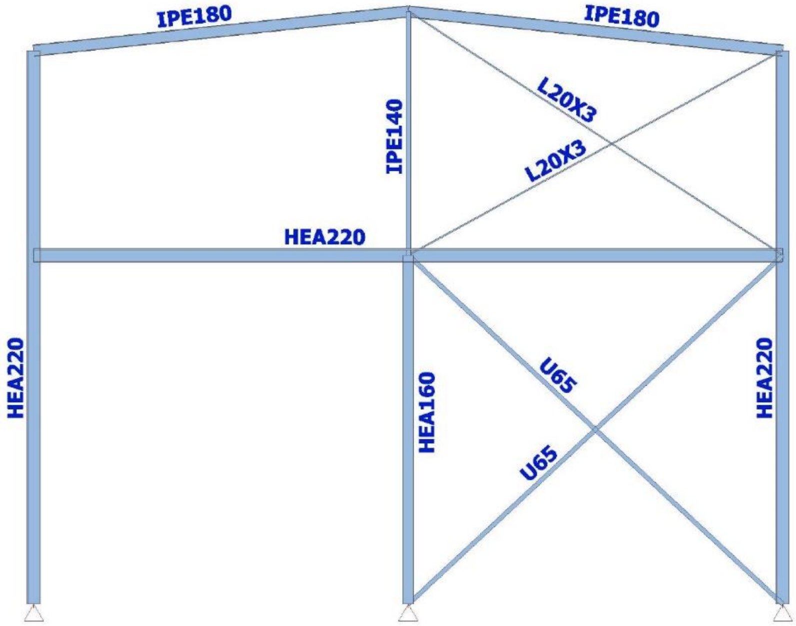
Preglednica 22: Okvir 2, razred srednje duktilnosti (DCM), faktor obnašanja *q* = 4

Element	**MSN in MSU**	**PPS *q* = 4**	**Kritičen pogoj povečanja**
Steber	HEA 450	HEA 500	Nosilnost prečnih prerezov, upogib in osna sila (poglavje 8.3.10.6.3)
Prečka	HEA 450	HEA 450	
Prečka	IPE 300	IPE 300	

Preglednica 23: Horizontalni pomiki etaž za potresni del obtežbe

Etaža	**Pomik, PPS q = 1,5**	**Pomik, PPS q = 4**	**Dopustni pomik**
1. etaža	4,37 cm	4,34 cm	5,44 cm
2. etaža	1,28 cm	1,26 cm	3,20 cm

8.4.2 Okvir 1 s centričnim povezjem (sistem nateznih diagonal)



Slika 100: Okvir 1, MSN in MSU ter PPS, q = 1,5

Preglednica 24: Okvir 1, razred nizke duktilnosti (DCL), faktor obnašanja q = 1,5

Element	**MSN in MSU**	**PPS q = 1,5**	**Kritičen pogoj povečanja**
Steber	HEA 220	HEA 220	
Prečka	HEA 220	HEA 220	
Prečka	IPE 180	IPE 180	
Diagonala	Ø12	U 65	Nosilnost prečnih prerezov, nateg (poglavje 8.2.10.4)
Diagonala	Ø12	L 20×3	Nosilnost prečnih prerezov, nateg (poglavje 8.2.10.4)

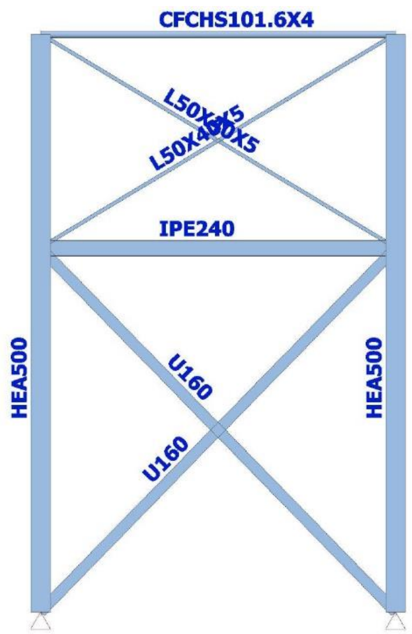
Preglednica 25: Okvir 1, razred srednje duktilnosti (DCM), faktor obnašanja q = 4

Element	**MSN in MSU**	**PPS q = 4**	**Kritičen pogoj povečanja**
Steber	HEA 220	HEA 220	
Prečka	HEA 220	HEA 220	
Prečka	IPE 180	IPE 180	
Diagonala	Ø12	U 65	Nosilnost prečnih prerezov, nateg in faktor dodatne nosilnosti Ω za zagotovitev enakomernega sipanja energije v diagonalah (poglavje 8.3.12.9.2)
Diagonala	Ø12	Ø12	

Preglednica 26: Horizontalni pomiki etaž za potresni del obtežbe

Etaža	**Pomik, PPS q = 1,5**	**Pomik, PPS q = 4**	**Dopustni pomik**
1. etaža	0,93 cm	0,92 cm	5,44 cm
2. etaža	0,64 cm	0,65 cm	3,20 cm

8.4.3 Okvir A s centričnim povezjem (sistem nateznih diagonal)



Slika 101: Okvir A, MSN in MSU ter PPS, q = 1,5

Preglednica 27: Okvir A, razred nizke duktilnosti (DCL), faktor obnašanja q = 1,5

Element	MSN in MSU	PPS q = 1,5	Kritičen pogoj povečanja
Steber	HEA 450	HEA 500	Momentni okvir 2, PPS, q = 1,5
Prečka	IPE 220	IPE 240	Odpornost elementov proti nestabilnosti, tlačno in upogibno obremenjeni elementi (poglavje 8.2.12.4.1)
Prečka	Cev Ø60,3×3,2	Cev Ø101,6×4	Odpornost elementov proti nestabilnosti, uklonska nosilnost (poglavje 8.2.12.4.2)
Diagonala	/	U 160	
Diagonala	/	L 50×40×5	

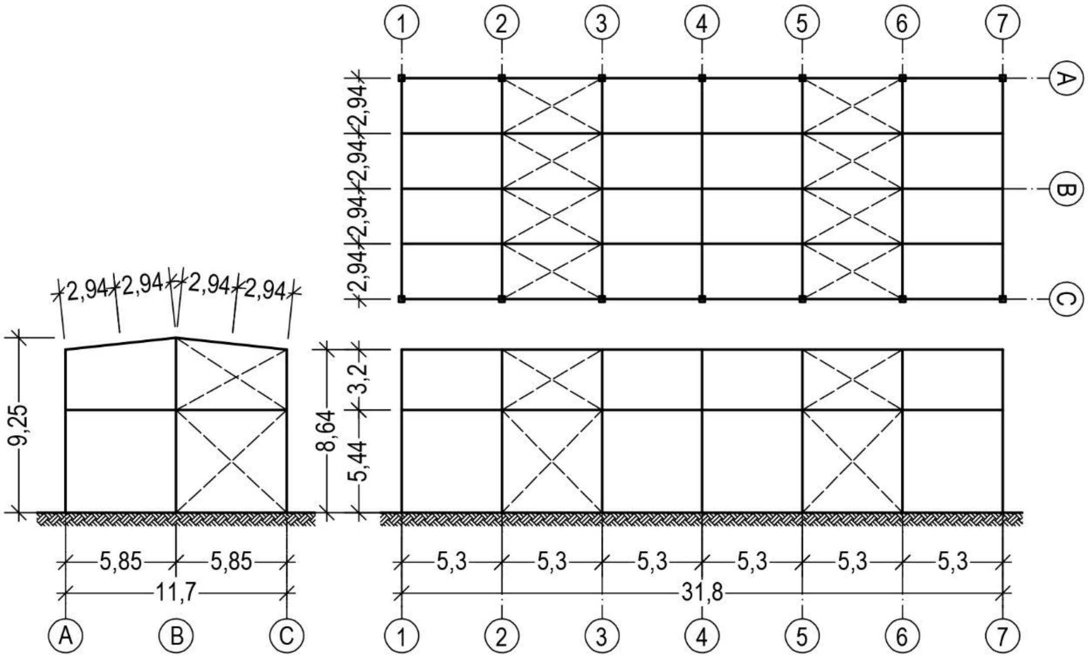
Preglednica 28: Okvir A, razred srednje duktilnosti (DCM), faktor obnašanja q = 4

Element	MSN in MSU	PPS q = 4	Kritičen pogoj povečanja
Steber	HEA 450	HEA 500	Momentni okvir 2, PPS, q = 4
Prečka	IPE 220	IPE 220	
Prečka	Cev Ø60,3×3,2	Cev Ø88,9×3	Odpornost elementov proti nestabilnosti, uklonska nosilnost (poglavje 8.3.10.6.1)
Diagonala	/	U 65	
Diagonala	/	L 25×3	

Preglednica 29: Horizontalni pomiki etaž za potresni del obtežbe

Etaža	Pomik, PPS q = 1,5	Pomik, PPS q = 4	Dopustni pomik
1. etaža	0,99 cm	2,44 cm	5,44 cm
2. etaža	0,70 cm	1,70 cm	3,20 cm

9 STREHA IN FASADA



Slika 102: Zasnova strehe in fasade

9.1 STREŠNI PANELI

Na strešne lege so pritrjeni ognjevarni strešni paneli Trimo SNV 150. Debelina panela je 15 cm. Dopustna razdalja med legami za strešne panele Trimo SNV 150 pri obremenitvi snega 1,27 kN/m^2 je 3,40 m (Priloga D: Strešni paneli Trimo - dopustne razdalje med podporami). Paneli se pritrjujejo na lege po navodilih proizvajalca.

9.2 FASADNI PANELI

Direktno na stebre so horizontalno pritrjeni fasadni paneli TrimoRaster FTV R 150. Debelina panela je 15 cm. Dopustna razdalja med stebri za fasadne panele TrimoRaster FTV R 150 pri pritisku vetra 0,59 kN/m^2 in srku vetra 0,30 kN/m^2 je 6,50 m (Priloga D: Fasadni paneli Trimo - dopustne razdalje med podporami). Paneli se pritrjujejo na stebre po navodilih proizvajalca.

10 ZNAČILNI SPOJI

Spoje dimenzioniramo kot polno nosilne, da zagotovimo nastanek plastičnega členka v prečkah in diagonalah.

$$R_d \geq 1{,}1\, \gamma_{ov}\, R_{fy}$$

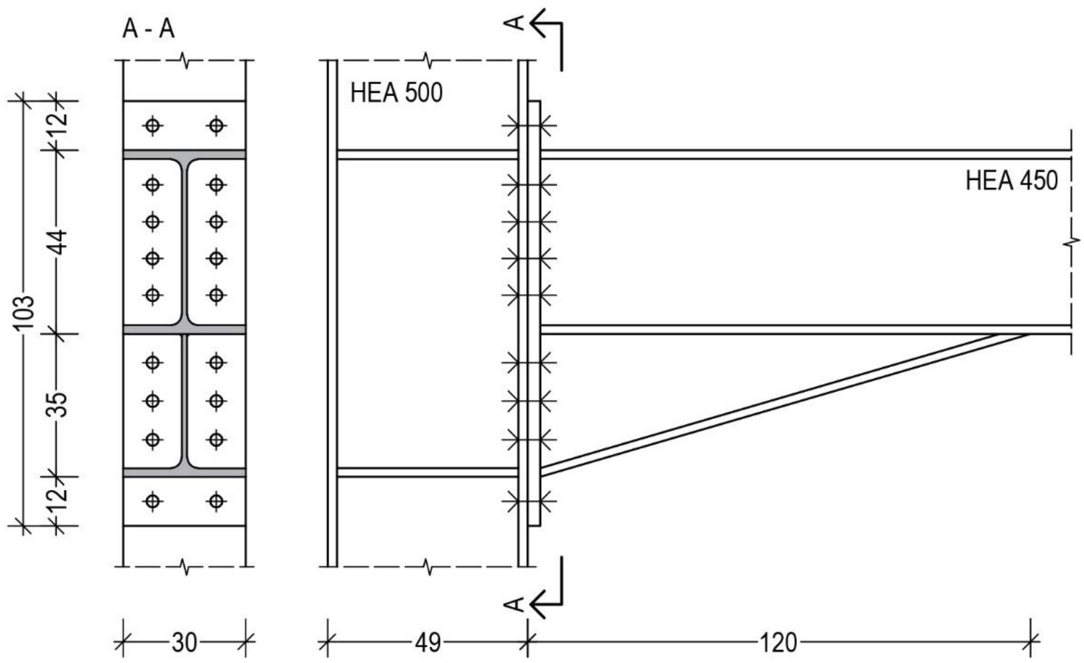
R_d odpornost spoja v skladu s SIST EN 1993-1-8

R_{fy} plastična odpornost priključenega elementa, sposobnega sipati energijo, izračunana na podlagi projektne vrednosti napetosti tečenja f_y

$\gamma_{ov} = 1{,}25$ faktor dodatne nosilnosti (SIST EN 1998-1, poglavje 6.2)

10.1 VIJAČENI MOMENTNI SPOJ PREČKE NA STEBER

10.1.1 Zasnova



Slika 103: Vijačeni momentni spoj

10.1.2 Obremenitev

$$M_{Ed} = 1{,}1 \cdot \gamma_{ov} \cdot M_{Rd,b} = 1{,}1 \cdot 1{,}25 \cdot 95079 \text{ kNcm} = 130734 \text{ kNcm}$$

$$M_{Rd,b} = M_{pl,Rd,b} + \Delta M = 75576 \text{ kNcm} + 19503 \text{ kNcm} = 95079 \text{ kNcm}$$

$$M_{pl,Rd,b} = W_{pl,y,b} \cdot \frac{f_y}{\gamma_{M0}} = 3216 \text{ cm}^3 \cdot \frac{23{,}5 \text{ kN/cm}^2}{1{,}0} = 75576 \text{ kNcm}$$

$W_{pl,y,b} = 3216 \text{ cm}^3$ prečka HEA 450

$$\Delta M = V_{pl,M} \cdot L_{vuta} = 162{,}53 \text{ kN} \cdot 120 \text{ cm} = 19503 \text{ kNcm}$$

$$V_{pl,M} = \frac{2 \cdot M_{pl,Rd,b}}{L^{HEA450}} = \frac{2 \cdot 75576 \text{ kNcm}}{930 \text{ cm}} = 162{,}53 \text{ kN}$$

$$V_{Ed} = 1{,}1 \cdot \gamma_{ov} \cdot V^*_{Ed} = 1{,}1 \cdot 1{,}25 \cdot 310{,}29 \text{ kN} = 427 \text{ kN}$$

$$V_{Ed}^{*} = V_{Ed,G} + V_{pl,M} = 147{,}76\ \text{kN} + 162{,}53\ \text{kN} = 310{,}29\ \text{kN}$$

10.1.3 Dimenzioniranje spoja

Dimenzije čelne pločevine in razpored vijakov

Vijaki M27; 8,8

$d = 27$ mm	premer vijaka
$d_0 = 30$ mm	premer luknje za vijak
$A = 5{,}73\ \text{cm}^2$	prerez vijaka
$A_s = 4{,}59\ \text{cm}^2$	strižni prerez vijaka
$f_{ub} = 80\ \text{kN/cm}^2$	natezna trdnost vijaka

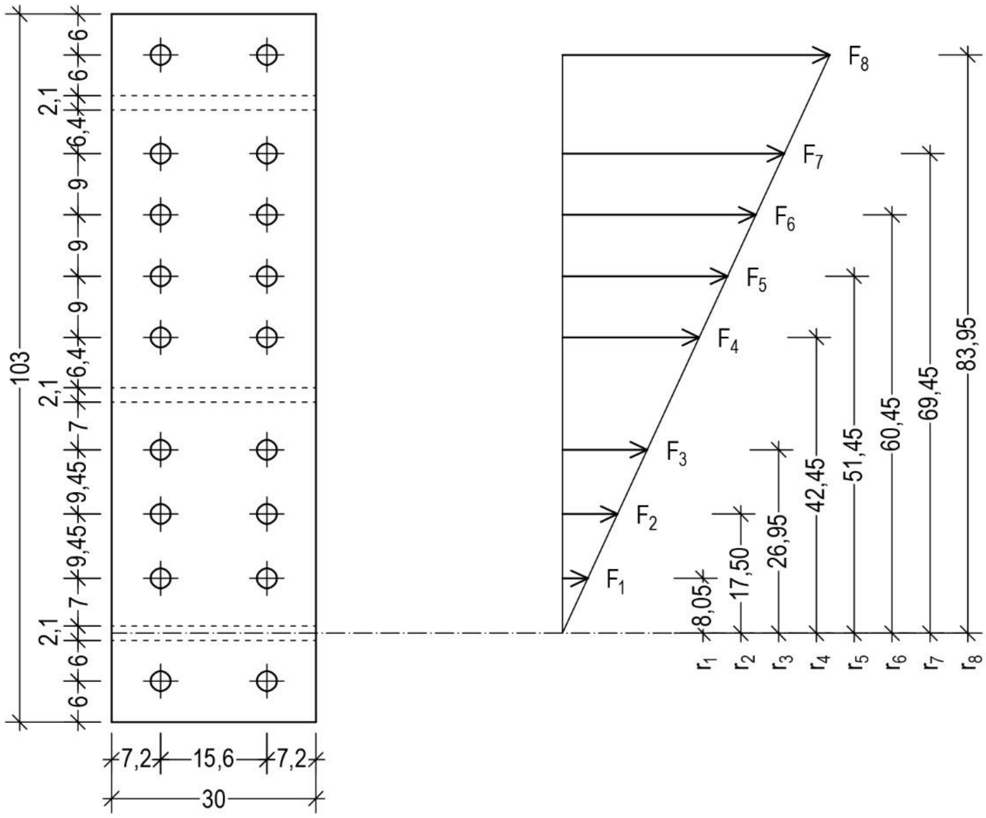
Razporeditev vijakov (slika 104)

$e_1 \approx 2\ d_0 = 54$ mm

$p_1 \approx 3\ d_0 = 81$ mm

$e_2 \approx 1{,}5\ d_0 = 40{,}5$ mm

$p_2 \approx 3\ d_0 = 81$ mm



Slika 104: Izbrane dimenzije čelne pločevine, razpored vijakov in sile v vijakih

Sile v vijakih v eni vrsti

$$F_1 = \frac{r_1 \cdot M_{Ed}}{\sum r_i^2} = \frac{8{,}05\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 49{,}94\ \text{kN}$$

$$F_2 = \frac{r_2 \cdot M_{Ed}}{\sum r_i^2} = \frac{17{,}50\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 108{,}57\ \text{kN}$$

$$F_3 = \frac{r_3 \cdot M_{Ed}}{\sum r_i^2} = \frac{26{,}95\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 167{,}20\ \text{kN}$$

$$F_4 = \frac{r_4 \cdot M_{Ed}}{\sum r_i^2} = \frac{42{,}45\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 263{,}37\ \text{kN}$$

$$F_5 = \frac{r_5 \cdot M_{Ed}}{\sum r_i^2} = \frac{51{,}45\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 319{,}20\ \text{kN}$$

$$F_6 = \frac{r_6 \cdot M_{Ed}}{\sum r_i^2} = \frac{60{,}45\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 375{,}04\ \text{kN}$$

$$F_7 = \frac{r_7 \cdot M_{Ed}}{\sum r_i^2} = \frac{69{,}45\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 430{,}88\ \text{kN}$$

$$F_8 = \frac{r_8 \cdot M_{Ed}}{\sum r_i^2} = \frac{83{,}95\ \text{cm} \cdot 130734\ \text{kNcm}}{21072\ \text{cm}^2} = 520{,}84\ \text{kN} = F_{max}$$

Kontrola natezne nosilnosti vijaka

$$F_{t,Ed} = \frac{F_{max}}{n} = \frac{520{,}84\ \text{kN}}{2} = 260{,}42\ \text{kN} \leq F_{t,Rd} \quad \checkmark$$

$n = 2$ število vojakov v eni vrsti

$$F_{t,Rd} = \frac{0{,}9 \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0{,}9 \cdot 80\ \text{kN/cm}^2 \cdot 5{,}73\ \text{cm}^2}{1{,}25} = 330{,}05\ \text{kN}$$

Kontrola strižne nosilnosti vijaka

$$F_{v,Ed} = \frac{V_{Ed}}{m} = \frac{427\ \text{kN}}{18} = 23{,}72\ \text{kN} \leq F_{v,Rd} \quad \checkmark$$

$m = 18$ strižno silo prevzamejo vsi vijaki

$$F_{v,Rd} = n \cdot \frac{0{,}6 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = 1 \cdot \frac{0{,}6 \cdot 80\ \text{kN/cm}^2 \cdot 4{,}59\ \text{cm}^2}{1{,}25} = 176{,}26\ \text{kN}$$

$n = 1$ strižna ravnina

Interakcija nateg in strig

$$\frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} \leq 1{,}0$$

$$0{,}56 + 0{,}13 = 0{,}69 \leq 1{,}0 \quad \checkmark$$

Kontrola nosilnosti na bočni pritisk

Čelna pločevina $t_{čp} = 30\ \text{mm}$, $(t_{čp} \geq d)$

$$F_{b,Rd} = \frac{2{,}5 \cdot \alpha \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{2{,}5 \cdot 0{,}667 \cdot 36 \cdot 2{,}7 \cdot 2{,}3}{1{,}25} = 298{,}23\ \text{kN} \geq F_{v,Rd} \quad \checkmark$$

$f_u = 36\ \text{kN/cm}^2$ natezna trdnost S235

$$t = \min\{t_f, t_{čp}\} = 2{,}3 \text{ cm}$$

$$t_f = 2{,}3 \text{ cm} \qquad \text{HEA 500}$$

$$\alpha = \min \begin{cases} \dfrac{e_1}{3\,d_0} = \dfrac{6}{3 \cdot 3} = 0{,}667 \\ \dfrac{p_1}{3\,d_0} - \dfrac{1}{4} = \dfrac{9}{3 \cdot 3} - \dfrac{1}{4} = 0{,}75 \\ \dfrac{f_{ub}}{f_u} = \dfrac{80}{36} = 2{,}222 \\ 1{,}0 \end{cases}$$

Kontrola preboja pločevine

$$B_{b,Rd} = \frac{0{,}6 \cdot f_u \cdot \pi \cdot d_m \cdot t}{\gamma_{M2}} = \frac{0{,}6 \cdot 36 \cdot \pi \cdot 4{,}26 \cdot 2{,}3}{1{,}25} = 531{,}90 \text{ kN} \geq F_{t,Rd} \;\checkmark$$

$d_m = 42{,}60$ mm premer glave vijaka

10.1.4 Steber v območju spoja

Pasnica stebra v območju netezne obremenitve

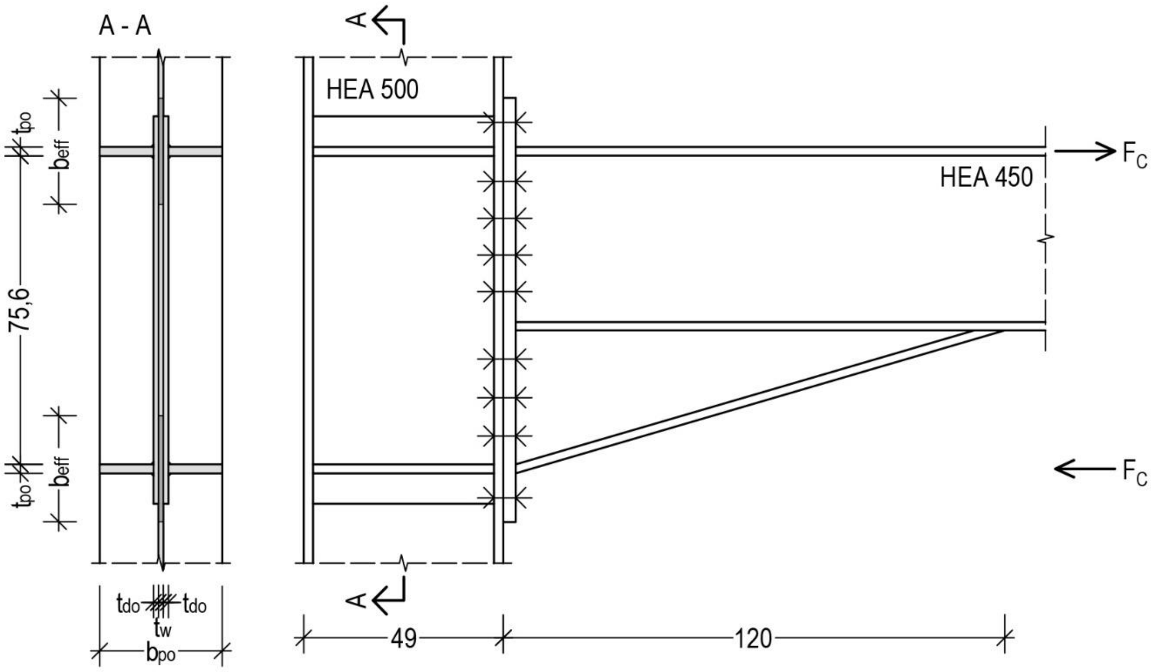
$$t_f = 23 \text{ mm} \geq 0{,}5 \cdot t_{čp} = 15 \text{ mm} \;\checkmark$$

$$t_f = 23 \text{ mm} \geq 0{,}8 \cdot d = 21{,}6 \text{ mm} \;\checkmark$$

(če pogoja nista izpolnjena, dodamo podložne ploščice)

Kontrola vnosa koncentrirane sile F_C v steber

Silo F_C prevzamejo del stojine stebra in prečne ojačitve stebra.



Slika 105: Vnos koncentrirane sile ter prečna in dodatna ojačitev

$$F_C = \sum_{i=1}^{n} F_i = 2235 \text{ kN}$$

Sila, ki jo prevzame stojina

$$N_{Rd,w} = b_{eff}\, t_w\, \frac{f_y}{\gamma_{M0}} = 25{,}75\ \text{cm} \cdot 1{,}2\ \text{cm} \cdot \frac{23{,}5\ \text{kN/cm}^2}{1{,}0} = 726{,}15\ \text{kN}$$

$$b_{eff} = t_f + 2\, t_{čp} + 2\, a\sqrt{2} + 5\, K$$

$$b_{eff} = 2{,}3 + 2 \cdot 3 + 2 \cdot 0{,}6 \cdot \sqrt{2} + 5 \cdot 3{,}15 = 25{,}75\ \text{cm}$$

$a = 0{,}5\, t_w = 0{,}5 \cdot 1{,}2\ \text{cm} = 0{,}6\ \text{cm}$ debelina zvara med pasnico stebra in prečno ojačitvijo

$$K = t_f + a\sqrt{2} = 2{,}3\ \text{cm} + 0{,}6 \cdot \sqrt{2}\ \text{cm} = 3{,}15\ \text{cm}$$

Sila, ki odpade na prečne ojačitve

$$N_{Rd,po} = F_C - N_{Rd,w} = 2235\ \text{kN} - 726{,}15\ \text{kN} = 1508{,}85\ \text{kN}$$

$$N_{Rd,po} \le b_{po}\, t_{po}\, \frac{f_y}{\gamma_{M0}}$$

$$t_{po} \ge \frac{N_{Rd,po}\, \gamma_{M0}}{b_{po}\, f_y} = \frac{1508{,}85\ kN \cdot 1{,}0}{30\ \text{cm} \cdot 23{,}5\ \text{kN/cm}^2} = 2{,}14\ \text{cm}$$

Izbrana debelina prečne ojačitve $t_{po} = 2{,}2$ cm

Stojina stebra v strigu

$$V_{Ed} = F_C = 2235\ \text{kN} \le V_{pl,Rd}$$

$$V_{pl,Rd} = \frac{A_v\, f_y}{\sqrt{3}\, \gamma_{M0}} = \frac{75{,}18\ \text{cm}^2 \cdot 23{,}5\ \text{kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 1020\ \text{kN}$$

$$A_v = \max \begin{cases} A - 2\, b\, t_f + (t_w + 2\, r)\, t_f = \\ = 198 - 2 \cdot 30 \cdot 2{,}3 + (1{,}2 + 2 \cdot 2{,}7) \cdot 2{,}3 = 75{,}18\ cm^2 \\ \eta\, h_w\, t_w = \\ = 1{,}2 \cdot 44{,}4 \cdot 1{,}2 = 63{,}94\ cm^2 \end{cases}$$

Stojino stebra je potrebno dodatno ojačati.

$$\Delta V_{Ed} = V_{Ed} - V_{pl,Rd} = 2235\ \text{kN} - 1020\ \text{kN} = 1215\ \text{kN}$$

$$\Delta V_{Ed} \le h_w\, t_{do}\, \frac{f_y}{\sqrt{3}\, \gamma_{M0}}$$

$$t_{do} \ge \frac{\Delta V_{Ed}\, \sqrt{3}\, \gamma_{M0}}{h_w\, f_y} = \frac{1215\ kN \cdot \sqrt{3} \cdot 1{,}0}{39\ \text{cm} \cdot 23{,}5\ \text{kN/cm}^2} = 2{,}29\ \text{cm}$$

Izbrana debelina dodatne ojačitve stebra $t_{do} = 1{,}2$ cm obojestransko.

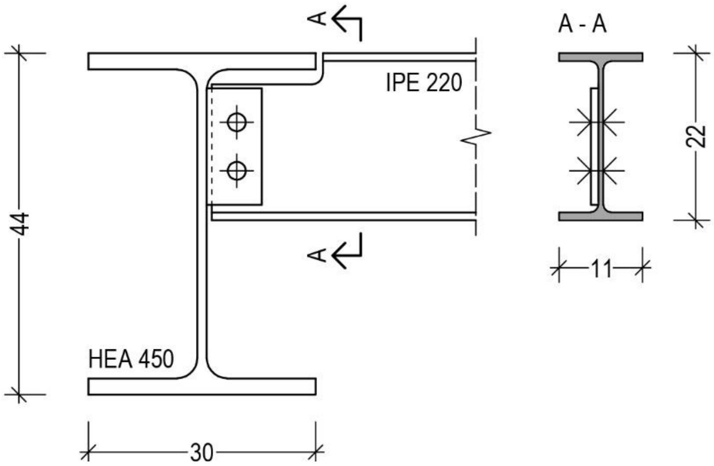
10.1.5 Zvari

Vsi zvari so kotni polno nosilni.

$$a_{max} = 0{,}46\ \text{t}$$

10.2 VIJAČENI ČLENKASTI SPOJ PREČKE NA PREČKO

10.2.1 Zasnova



Slika 106: Vijačeni členkasti spoj

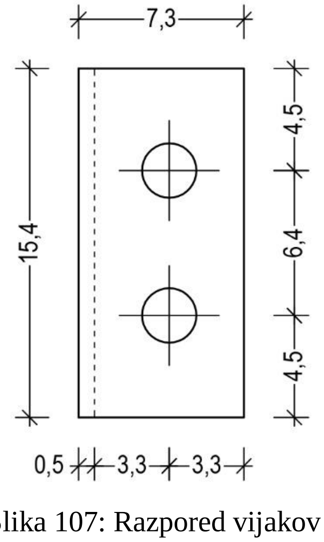
10.2.2 Obremenitev

$V_{Ed} = 74{,}23$ kN (poglavje 4.3.3.1)

10.2.3 Dimenzioniranje spoja

Dimenzije vezne pločevine in razpored vijakov
Vijaki M24; 8,8

$d = 22$ mm	premer vijaka
$d_0 = 24$ mm	premer luknje za vijak
$A = 3{,}8$ cm^2	prerez vijaka
$A_s = 3{,}03$ cm^2	strižni prerez vijaka
$f_{ub} = 80$ kN/cm^2	natezna trdnost vijaka



Slika 107: Razpored vijakov

Razporeditev vijakov (slika 109)

$$e_1 \approx 2\, d_0 = 48 \text{ mm}$$

$$p_1 \approx 3\, d_0 = 72 \text{ mm}$$

$$e_2 \approx 1{,}5\, d_0 = 36 \text{ mm}$$

$$\Delta = 5 \text{ mm}$$

Debelina vezne pločevine

$$M_{Ed} \leq W_{vp}\, \frac{f_y}{\gamma_{M0}} = \frac{h_{vp}^2\, t_{vp}}{6}\, \frac{f_y}{\gamma_{M0}}$$

$$t_{vp} \geq \frac{6\, M_{Ed}\, \gamma_{M0}}{h_{vp}^2\, f_y} = \frac{6 \cdot 282{,}07 \text{ kNcm} \cdot 1{,}0}{(15{,}4 \text{ cm})^2 \cdot 23{,}5 \text{ kN/cm}^2} = 0{,}30 \text{ cm}$$

$$M_{Ed} = V_{Ed}\, (\Delta + e_2) = 74{,}23 \text{ kN} \cdot 3{,}8 \text{ cm} = 282{,}07 \text{ kNcm}$$

Izbrana debelina vezne pločevine $t_{vp} = 1$ cm

Kontrola strižne nosilnosti vezne pločevine

$$V_{pl,Rd} = \frac{A_v\, f_y}{\sqrt{3}\, \gamma_{M0}} = \frac{15{,}4 \text{ cm}^2 \cdot 23{,}5 \text{ kN/cm}^2}{\sqrt{3} \cdot 1{,}0} = 208{,}94 \text{ kN}$$

$$A_v = h_{vp}\, t_{vp} = 15{,}4 \text{ cm} \cdot 1 \text{ cm} = 15{,}4 \text{ cm}^2$$

$$0{,}5\, V_{pl,Rd} = 104{,}47\, kN \geq V_{Ed} = 74{,}23 \text{ kN} \quad \checkmark$$

Zvar med vezno pločevino in nosilcem

$$\text{a} = 4 \text{ mm} \quad (3\, mm \leq a \leq 0{,}46\, t = 4{,}6 \text{ mm})$$

$$l_{zv} = h_{vp} - 2\, a = 15{,}4 \text{ cm} - 2 \cdot 0{,}4 \text{ cm} = 14{,}6 \text{ cm}$$

Kontrola nosilnosti zvarov

$$v_{\parallel} = \frac{V_{Ed}}{2\, a\, l_{zv}} = \frac{74{,}23 \text{ kN}}{2 \cdot 0{,}4 \text{ cm} \cdot 14{,}6 \text{ cm}} = 6{,}36 \text{ kN/cm}^2$$

$$n = \frac{M_{Ed}}{W_{zv}} = \frac{6\, M_{Ed}}{2\, a\, l_{zv}^2} = \frac{6 \cdot 282{,}07 \text{ kNcm}}{2 \cdot 0{,}4 \text{ cm} \cdot (14{,}6 \text{ cm})^2} = 9{,}92 \text{ kN/cm}^2$$

$$\sqrt{v_{\parallel}^2 + n^2} = \sqrt{(6{,}36 \text{ cm})^2 + (9{,}92 \text{ cm})^2} = 11{,}78 \text{ kN/cm}^2 \leq f_{vw,Rd} \quad \checkmark$$

$$f_{vw,Rd} = \frac{f_u}{\sqrt{3}\, \beta_w\, \gamma_{M2}} = \frac{36 \text{ kN/cm}^2}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 20{,}78 \text{ kN/cm}^2$$

Kontrola strižne nosilnosti vijaka

$$F_v = \frac{V_{Ed}}{m} = \frac{74{,}23 \text{ kN}}{2} = 37{,}12 \text{ kN}$$

$m = 2$ strižno silo prevzameta oba vijaka

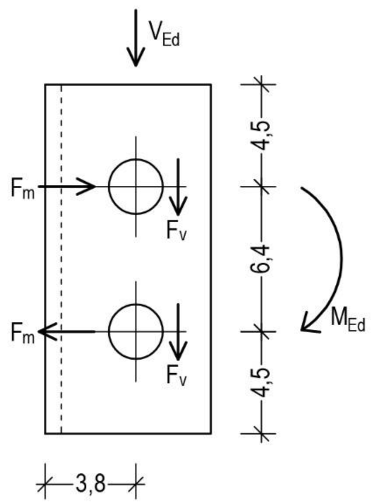
$$F_m = \frac{M_{Ed}}{p_1} = \frac{282{,}07 \text{ kNcm}}{6{,}4 \text{ cm}} = 44{,}07 \text{ kN}$$

$p_1 = 6{,}4$ cm razmak med vijakoma

$$F_{v,Ed} = \sqrt{F_v^2 + F_m^2} = \sqrt{(37{,}12 \text{ kN})^2 + (44{,}07 \text{ kN})^2} = 57{,}62 \text{ kN} \leq F_{v,Rd} \quad \checkmark$$

$$F_{v,Rd} = n \cdot \frac{0{,}6 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = 1 \cdot \frac{0{,}6 \cdot 80\,\mathrm{kN/cm^2} \cdot 3{,}03\,\mathrm{cm^2}}{1{,}25} = 116{,}35\ kN$$

$n = 1$ strižna ravnina



Slika 108: Strižna nosilnost vijaka

Kontrola nosilnosti na bočni pritisk

$$F_{b,Rd} = \frac{2{,}5 \cdot \alpha \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{2{,}5 \cdot 0{,}63 \cdot 36 \cdot 2{,}2 \cdot 0{,}59}{1{,}25} = 58{,}88\ \mathrm{kN} \geq F_{v,Ed} = 57{,}62\ kN \quad \checkmark$$

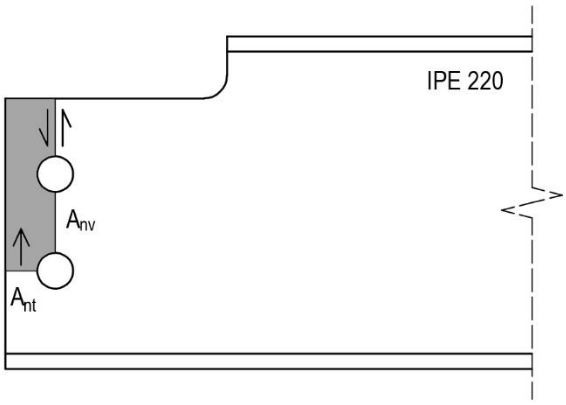
$f_u = 36\ \mathrm{kN/cm^2}$ natezna trdnost S235

$t = \min\{t_w, t_{vp}\} = 0{,}59\ \mathrm{cm}$

$t_w = 0{,}59\ \mathrm{cm}$ IPE 220

$$\alpha = \min \begin{cases} \dfrac{e_1}{3\,d_0} = \dfrac{4{,}5}{3 \cdot 2{,}4} = 0{,}63 \\ \dfrac{p_1}{3\,d_0} - \dfrac{1}{4} = \dfrac{6{,}4}{3 \cdot 2{,}4} - \dfrac{1}{4} = 0{,}64 \\ \dfrac{f_{ub}}{f_u} = \dfrac{80}{36} = 2{,}22 \\ 1{,}0 \end{cases}$$

Strižni iztrg (»Block Shear«)



Slika 109: Strižni iztrg

$$V_{eff,Rd} = 0{,}5\ A_{nt}\ \frac{f_u}{\gamma_{M2}} + A_{nv}\ \frac{f_y}{\sqrt{3}\ \gamma_{M0}} = 0{,}5 \cdot 1{,}24 \cdot \frac{36}{1{,}25} + 4{,}31 \cdot \frac{23{,}5}{\sqrt{3} \cdot 1{,}0} = 76{,}33\ \text{kN}$$

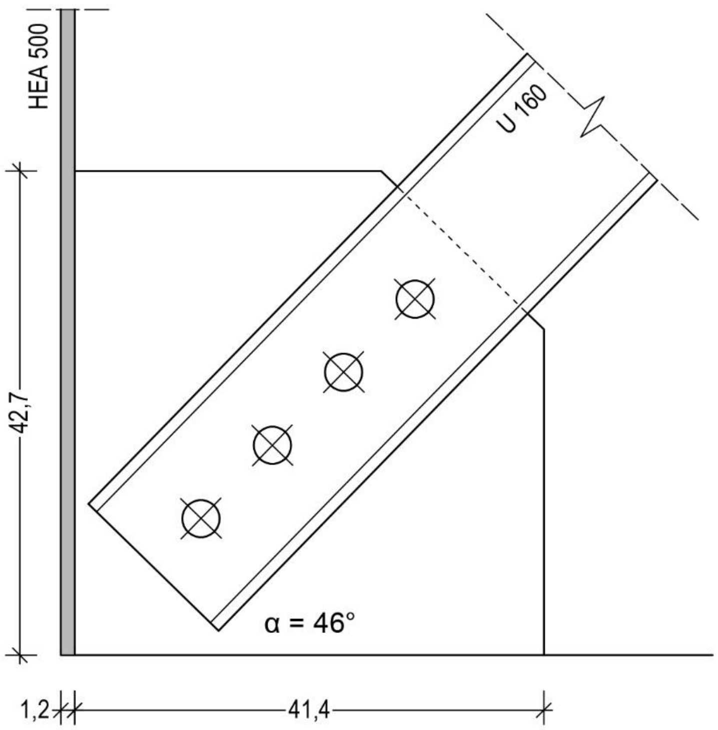
$$A_{nt} = \left(e_2 - \frac{d_0}{2}\right)\ t_f = \left(3{,}3\ \text{cm} - \frac{2{,}4\ \text{cm}}{2}\right) \cdot 0{,}59\ \text{cm} = 1{,}24\ \text{cm}^2$$

$$A_{nv} = (e_1 + p_1 - 1{,}5\ d_o)\ t_f = (4{,}5 + 6{,}4 - 1{,}5 \cdot 2{,}4) \cdot 0{,}59 = 4{,}31\ \text{cm}^2$$

$$V_{eff,Rd} = 76{,}33\ \text{kN} \geq V_{Ed} = 74{,}23\ kN \quad ✓$$

10.3 SPOJ NATEZNE DIAGONALE NA STEBER

10.3.1 Zasnova



Slika 110: Spoj diagonale na steber

10.3.2 Obremenitev

$$N_{Ed} = 1{,}1 \cdot \gamma_{ov} \cdot N_{t,Rd} = 1{,}1 \cdot 1{,}25 \cdot 564\ \text{kN} = 775\ \text{kN}$$

$$N_{t,Rd} = \frac{A\ f_y}{\gamma_{M0}} = \frac{24\ \text{cm}^2 \cdot 23{,}5\ \text{kN/cm}^2}{1{,}0} = 564\ \text{kN}$$

10.3.3 Dimenzioniranje spoja

Vijaki M30; 8,8

$d = 30$ mm	premer vijaka
$d_0 = 33$ mm	premer luknje za vijak
$A = 7{,}07\ \text{cm}^2$	prerez vijaka

$A_s = 5{,}61\ \text{cm}^2$ strižni prerez vijaka

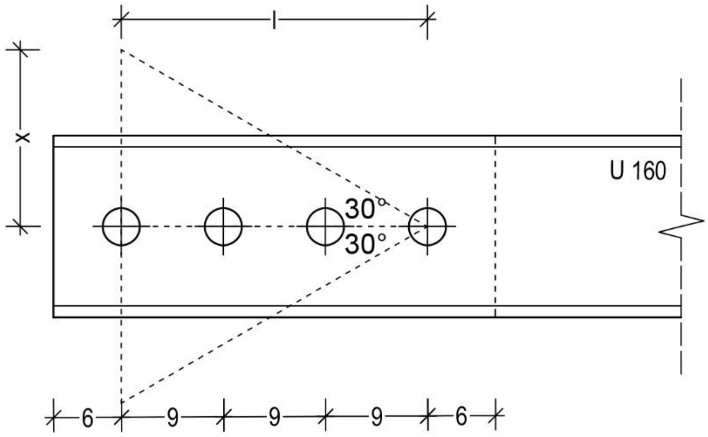
$f_{ub} = 80\ \text{kN/cm}^2$ natezna trdnost vijaka

Razporeditev vijakov (slika 113)

$e_1 \approx 2\ d_0 = 66\ \text{mm}$

$p_1 \approx 3\ d_0 = 99\ \text{mm}$

$e_2 \approx 1{,}5\ d_0 = 49{,}5\ \text{mm}$



Slika 111: Razpored vijakov

Kontrola strižne nosilnosti vijaka

$$F_{v,Ed} = \frac{N_{Ed}}{m} = \frac{775\ \text{kN}}{4} = 194\ kN \leq F_{v,Rd}\ \checkmark$$

$m = 4$ strižno silo prevzamejo vsi vijaki

$$F_{v,Rd} = n \cdot \frac{0{,}6 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = 1 \cdot \frac{0{,}6 \cdot 80\ \text{kN/cm}^2 \cdot 5{,}61\ \text{cm}^2}{1{,}25} = 215{,}42\ kN$$

$n = 1$ strižna ravnina

Vezna pločevina

$$t_{vp} \geq \frac{N_{Ed}\ \gamma_{M0}}{f_y\ (b - d_0)} = \frac{775\ \text{kN} \cdot 1{,}0}{23{,}5\ \text{kN/cm}^2 \cdot (31{,}18\ \text{cm} - 3{,}3\ \text{cm})} = 1{,}18\ \text{cm}$$

$$b = 2\ x = 2 \cdot 15{,}59\ \text{cm} = 31{,}18\ \text{cm}$$

$$x = tg\gamma \cdot l = tg30^{\circ} \cdot 27\ \text{cm} = 15{,}59\ \text{cm}$$

$$l = 3\ p_1 = 3 \cdot 9\ \text{cm} = 27\ \text{cm}$$

Izbrana debelina vezne pločevine $t_{vp} = 1{,}4\ \text{cm}$

Kontrola neto prereza

$$N_{Ed} = 775\ \text{kN} \leq \frac{A_{net}\ f_y}{\gamma_{M0}} = \frac{21{,}53\ \text{cm}^2 \cdot 23{,}5\ \text{kN/cm}^2}{1{,}0} = 505{,}8\ \text{kN}\ \times$$

$$A_{net} = A - d \cdot d_0 = 24\ \text{cm}^2 - 0{,}75\ \text{cm} \cdot 3{,}3\ \text{cm} = 21{,}53\ \text{cm}^2$$

Na osnovni U profil privarimo podložno pločevino.

$$\Delta N_{Ed} = N_{Ed} - 505{,}8\ \text{kN} = 775\ \text{kN} - 505{,}8\ \text{kN} = 269{,}2\ \text{kN}$$

$$t_{pp} \geq \frac{\Delta N_{Ed}\,\gamma_{M0}}{f_y\left(h - d_0 - 2\,(t + r)\right)} = \frac{269{,}2 \cdot 1{,}0}{23{,}5 \cdot \left(16 - 3{,}3 - 2 \cdot (1{,}05 + 1{,}05)\right)} = 1{,}35\ \text{cm}$$

Izbrana debelina podložne pločevine $t_{pp} = 1{,}4$ cm

Kontrola nosilnosti na bočni pritisk

$$F_{b,Rd} = m \cdot \frac{2{,}5 \cdot \alpha \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = 4 \cdot \frac{2{,}5 \cdot 0{,}67 \cdot 36 \cdot 3 \cdot 1{,}4}{1{,}25} = 810{,}4\ \text{kN} \geq N_{Ed} = 775\ \text{kN} \quad \checkmark$$

$f_u = 36\ \text{kN/cm}^2$ natezna trdnost S235

$$t = \min\{t_U, t_{vp}\} = 1{,}4\ \text{cm}$$

$$t_U = d + t_{pp} = 0{,}75\ \text{cm} + 1{,}4\ \text{cm} = 2{,}15\ \text{cm}$$

$$\alpha = \min\begin{cases} \dfrac{e_1}{3\,d_0} = \dfrac{6}{3 \cdot 3} = 0{,}67 \\ \dfrac{p_1}{3\,d_0} - \dfrac{1}{4} = \dfrac{9}{3 \cdot 3} - \dfrac{1}{4} = 0{,}75 \\ \dfrac{f_{ub}}{f_u} = \dfrac{80}{36} = 2{,}22 \\ 1{,}0 \end{cases}$$

10.3.4 Zvari

Debelina zvara

$$3\ \text{mm} \leq a \leq 0{,}46\ t = 5{,}52\ \text{mm}$$

$$t = \min\{t_w, t_{vp}\} = 1{,}2\ \text{cm}$$

Izbrana debelina zvara $a = 5$ mm

Dolžina zvarov in prerez

$$l_{zv,1} = l_1 - 2\,a = 41{,}4\ \text{cm} - 2 \cdot 0{,}5\ \text{cm} = 40{,}4\ \text{cm}$$

$$l_{zv,2} = l_2 - 2\,a = 42{,}7\ \text{cm} - 2 \cdot 0{,}5\ \text{cm} = 41{,}7\ \text{cm}$$

$$A = 2 \cdot a \cdot \left(l_{zv,1} + l_{zv,2}\right) = 2 \cdot 0{,}5\ \text{cm} \cdot (40{,}4\ \text{cm} + 41{,}7\ \text{cm}) = 82{,}1\ \text{cm}^2$$

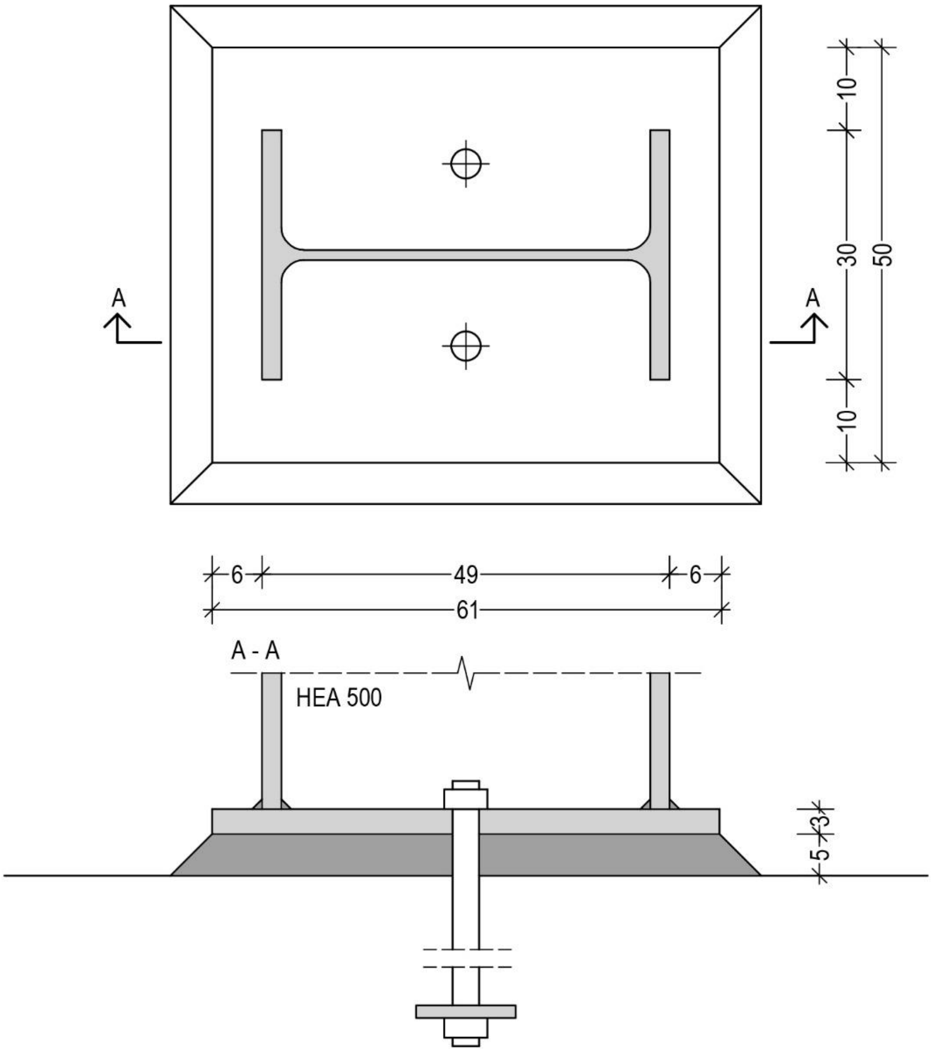
Kontrola nosilnosti kotnega zvara

$$f_{vw,Rd} = \frac{f_u}{\sqrt{3}\,\beta_w\,\gamma_{M2}} = \frac{36\ \text{kN/cm}^2}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 20{,}78\,\text{kN/cm}^2$$

$$\sigma_{zv} = \frac{N_{Ed}}{A} = \frac{775\ \text{kN}}{82{,}1\ \text{cm}^2} = 9{,}44\,\text{kN/cm}^2 \leq f_{vw,Rd} \quad \checkmark$$

10.4 ČLENKASTI SPOJ STEBRA NA TEMELJ

10.4.1 Zasnova



Slika 112: Spoj stebra na temelj

10.4.2 Obremenitev

Reakcije okvira A s centričnim povezjem

$N_{Ed} = R_z = 391$ kN (nateg, izvlek)

$V_{Ed} = R_x = 370$ kN

Reakcije momentnega okvira 2

$N_{Ed} = R_z = 496$ kN (tlak)

10.4.3 Dimenzioniranje

Izbrana debelina čelne pločevine $t_{čp} = 3$ cm

Zvar med čelno pločevino in stebrom HEA 500 (polno nosilen)

$3\ \text{mm} \leq a_f \leq 0{,}46\ t = 10{,}58\ \text{mm}$

$t = \min\{t_f, t_{čp}\} = 2{,}3\ \text{cm}$

Izbrana debelina zvara $a_f = 10\ \text{mm}$

$3\ \text{mm} \leq a_w \leq 0{,}46\ t = 5{,}52\ \text{mm}$

$t = \min\{t_w, t_{čp}\} = 1{,}2\ \text{cm}$

Izbrana debelina zvara $a_w = 5\ \text{mm}$

Sidrni vijaki M33; 8,8

$d = 33\ \text{mm}$ premer vijaka

$d_0 = 36\ \text{mm}$ premer luknje za vijak

$A = 8{,}55\ \text{cm}^2$ prerez vijaka

$A_s = 6{,}94\ \text{cm}^2$ strižni prerez vijaka

$f_{ub} = 80\ \text{kN/cm}^2$ natezna trdnost vijaka

Kontrola natezne nosilnosti vijaka

$$F_{t,Ed} = \frac{N_{Ed}}{n} = \frac{391\ \text{kN}}{2} = 196\ \text{kN} \leq F_{t,Rd}\ ✓$$

$n = 2$ število vojakov v spoju

$$F_{t,Rd} = \frac{0{,}9 \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0{,}9 \cdot 80\ \text{kN/cm}^2 \cdot 8{,}55\ \text{cm}^2}{1{,}25} = 492{,}48\ \text{kN}$$

Kontrola strižne nosilnosti vijaka

$$F_{v,Ed} = \frac{V_{Ed}}{m} = \frac{370\ \text{kN}}{18} = 185\ \text{kN} \leq F_{v,Rd}\ ✓$$

$m = 2$ strižno silo prevzameta oba vijaka

$$F_{v,Rd} = n \cdot \frac{0{,}6 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = 1 \cdot \frac{0{,}6 \cdot 80\ \text{kN/cm}^2 \cdot 6{,}94\ \text{cm}^2}{1{,}25} = 266{,}50\ \text{kN}$$

$n = 1$ strižna ravnina

Interakcija nateg in strig

$$\frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} \leq 1{,}0$$

$0{,}28 + 0{,}69 = 0{,}97 \leq 1{,}0$ ✓

Sidrna pločevina

Izbrana sidrna pločevina dimenzij $a/t = 120/15\ \text{mm}$

Kontaktne napetosti

$$A_{kn} = a^2 - \pi\,(d_0/2)^2 = (12\ \text{cm})^2 - \pi \cdot (3{,}6\ \text{cm}/2)^2 = 133{,}82\ \text{cm}^2$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{2{,}5\ \text{kN/cm}^2}{1{,}5} = 1{,}67\ \text{kN/cm}^2$$

$$N_{s,Rd} = A_{kn}\, f_{cd} = 133{,}82\ \text{cm}^2 \cdot 1{,}67\ \text{kN/cm}^2 = 223{,}48\ \text{kN} \geq F_{t,Ed} = 196\ \text{kN}$$

10.4.4 Betonsko podlitje

Izbrana debelina betonskega podlitja $t_{bp} = 5$ cm
Odpornost ploskve pri enakomerno razdeljeni obtežbi v obliki koncentrirane sile

$$F_{Rdu} = A_{c0}\, f_{cd} \sqrt{\frac{A_{c1}}{A_{c0}}} = 2990\ \text{cm}^2 \cdot 3{,}33\ \text{kN/cm}^2 \cdot \sqrt{\frac{14400\ \text{cm}^2}{2990\ \text{cm}^2}} = 21850\ \text{kN}$$

$$F_{Rdu} \leq 3\, A_{c0}\, f_{cd} = 3 \cdot 2990\ \text{cm}^2 \cdot 3{,}33\ \text{kN/cm}^2 = 29870\ \text{kN} \quad \checkmark$$

Za betonsko podlitje uporabimo beton C50/60

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{5\ \text{kN/cm}^2}{1{,}5} = 3{,}33\ \text{kN/cm}^2$$

Obremenjena površina

$$A_{c0} = b_1\, d_1 = 46\ \text{cm} \cdot 65\ \text{cm} = 2990\ \text{cm}^2$$

$$b_1 = b + 2\left(t_{čp} + t_{bp}\right) = 30\ \text{cm} + 2 \cdot (3 + 5)\ \text{cm} = 46\ \text{cm}$$

$$d_1 = h + 2\left(t_{čp} + t_{bp}\right) = 49\ \text{cm} + 2 \cdot (3 + 5)\ \text{cm} = 65\ \text{cm}$$

Projektna ploskev raznosa obremenitve

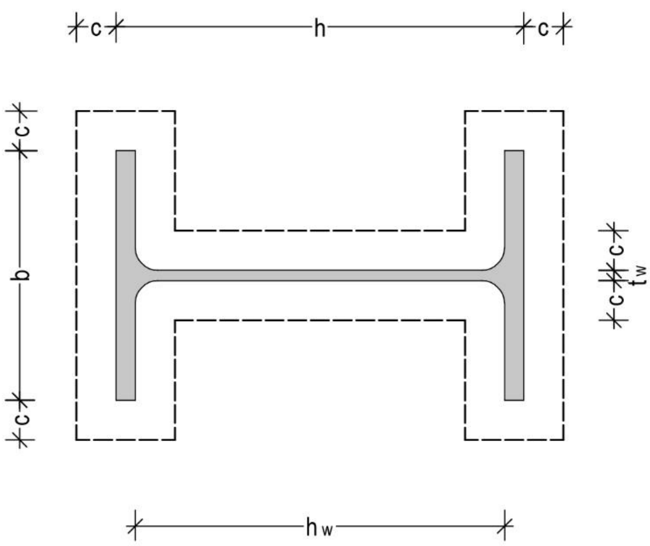
$$A_{c1} = b_2\, d_2 = 120\ \text{cm} \cdot 120\ \text{cm} = 14400\ \text{cm}^2$$

$$b_2 = 120\ \text{cm} \leq 3\, b_1 = 138\ \text{cm}$$

$$d_2 = 120\ \text{cm} \leq 3\, d_1 = 195\ \text{cm}$$

$$h_{tem} = 100\ \text{cm} \geq \begin{cases} b_2 - b_1 = 92\ \text{cm} \\ d_2 - d_1 = 55\ \text{cm} \end{cases}$$

Sodelujoča površina podlitja



Slika 113: Sodelujoča površina podlitja

$$c = t_{čp} \sqrt{\frac{f_y}{3\, f_{jd}\, \gamma_{M0}}} = 3\ cm \cdot \sqrt{\frac{23{,}5\ \text{kN/cm}^2}{3 \cdot 4{,}87\ \text{kN/cm}^2 \cdot 1{,}0}} = 4{,}83\ \text{cm}$$

Nosilnost podlitja

$$f_{jd} = \beta_j \sqrt{\frac{A_{c1}}{A_{c0}}}\, f_{cd} = \frac{2}{3} \cdot \sqrt{\frac{14400\ \text{cm}^2}{2990\ \text{cm}^2}} \cdot 3{,}33\ \text{kN/cm}^2 = 4{,}87\ \text{kN/cm}^2$$

$$\beta_j = \frac{2}{3}$$

$$A_{eff} = (b + 2\,c)(h + 2\,c) - 2\,(h_w - 2\,c)\left(\frac{b - t_w}{2}\right)$$

$$A_{eff} = (30 + 2 \cdot 4{,}83) \cdot (49 + 2 \cdot 4{,}83) - 2 \cdot (44{,}4 - 2 \cdot 4{,}83) \cdot \left(\frac{30 - 1{,}2}{2}\right) = 1326\ \text{cm}^2$$

Kontrola nosilnosti podlitja

$$F_{c,Rd} = A_{eff}\, f_{jd} = 1326\ \text{cm}^2 \cdot 4{,}87\ \text{kN/cm}^2 = 6458\ \text{kN}$$

$$N_{Ed} = 496\ \text{kN (tlak)} \leq F_{c,Rd}$$ ✓

11 ZAKLJUČEK

V diplomski nalogi sem izdelal statično analizo objekta iz jeklenih nosilnih elementov. Izdelal sem tudi vse potrebne kontrole nosilnosti prečnih prerezov ter stabilnosti nosilnih elementov konstrukcije v skladu z veljavnimi predpisi in standardi Evrokod in pripadajočimi slovenskimi nacionalnimi dodatki. Dimenzioniranje na potresno projektno stanje je prikazano na dva načina in sicer, konstrukcije z majhnim sipanjem energije ter konstrukcije s sposobnostjo sipanja energije. Dimenzionirani so tudi značilni spoji obravnavane jeklene konstrukcije.

Za notranje in zunanje (čelne) prečne okvire je bila merodajna obtežba za prečke (horizontalne nosilne elemente) iz kombinacij mejnega stanja nosilnosti (MSN). Pri potresnem projektnem stanju za oba načina je bilo potrebno povečati prereze stebrov in nateznih diagonal. Horizontalni pomiki etaž zaradi potresne obtežbe so enaki ne glede na način potresnega projektiranja, katera sta bila konstrukcije z majnim sipanjem energije ter konstrukcije s sposobnostjo sipanja energije.

Za vzdolžne okvire s centričnimi povezji pa je bila merodajna obtežba potresnega projektnega stanja. Razlika med obema načinoma potresnega projektiranja je očitnejša. Pri načinu konstrukcije z majhnim sipanjem energije so potrebni prečni prerezi elementov, ki prenašajo potresno obtežbo (tlačene prečke in natezne diagonale), večji. Zaradi večje togosti okvira pa so pomiki manjši kot pri okviru dimenzioniranem na način konstrukcije s sposobnostjo sipanja energije.

Pri izdelavi diplomske naloge sem nadgradil znanje pridobljeno tekom študija. Bolje sem se seznanil s predpisi in standardi Evrokod za projektiranje jeklenih in sovprežnih konstrukcij. Naučil pa sem se tudi uporabljati računalniški program SCIA Engineer za statično analizo in dimenzioniranje gradbenih objektov.

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PRILOGA A: IZPIS IZ PROGRAMA SCIA ENGINEER 14

Priloga A.1: Kontrola nosilnosti in stabilnosti momentnega okvira 2 - MSN

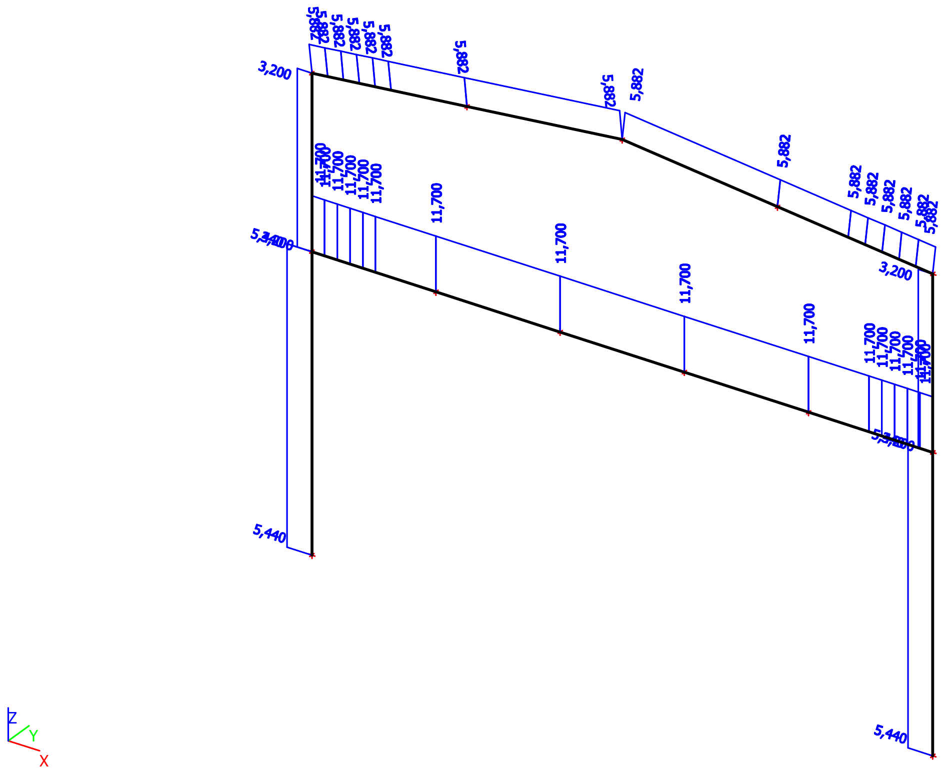
Priloga A.2: Kontrola nosilnosti in stabilnosti okvira 1 s centričnim povezjem (sistem nateznih diagonal) - MSN

1. Priloga A.1

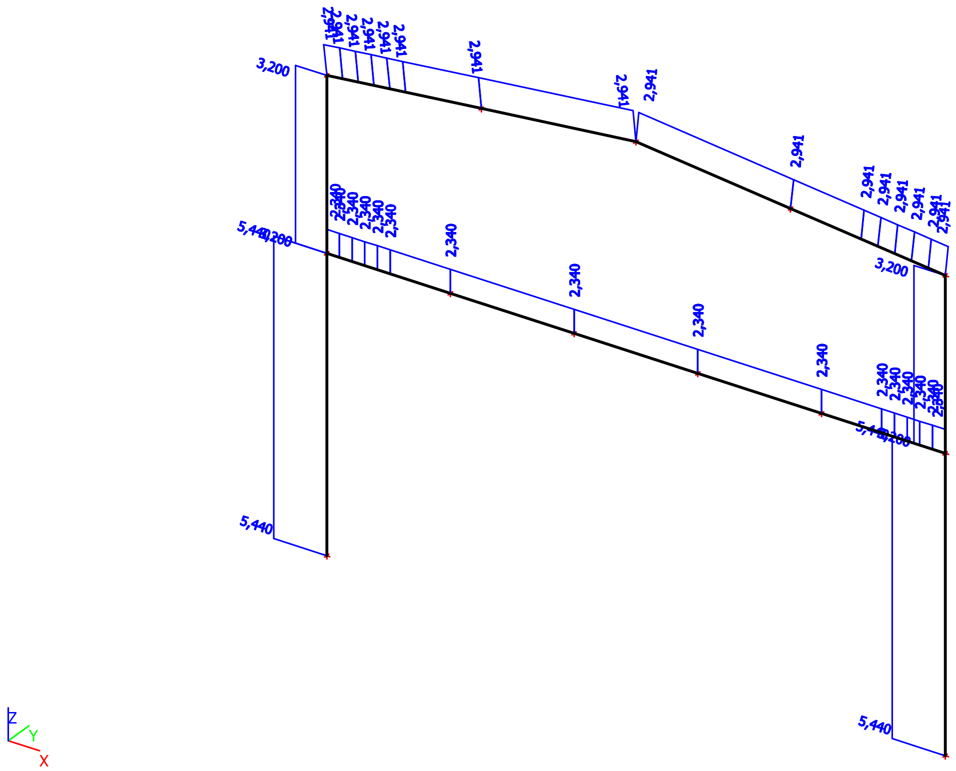
Linear calculation

Member	CS Name	Part	Sway y	Ly [m]	ky [-]	ly [m]	Lam y [-]	lyz [m]	l LTB [m]
			Sway z	Lz [m]	kz [-]	lz [m]	Lam z [-]		
B1	CS2	1	Yes	5,440	1,00	5,440	28,76	5,440	5,440
			No	5,440	1,00	5,440	74,58		
B1	CS2	2	Yes	3,200	1,00	3,200	16,92	3,200	3,200
			No	3,200	1,00	3,200	43,87		
B2	CS2	1	Yes	5,440	1,00	5,440	28,76	5,440	5,440
			No	5,440	1,00	5,440	74,58		
B2	CS2	2	Yes	3,200	1,00	3,200	16,92	3,200	3,200
			No	3,200	1,00	3,200	43,87		
B3	CS4	1	Yes	11,700	1,00	11,700	42,11	2,340	2,340
			No	2,340	1,00	2,340	32,76		
B3	CS4	2	Yes	11,700	1,00	11,700	45,68	2,340	2,340
			No	2,340	1,00	2,340	32,32		
B3	CS4	3	Yes	11,700	1,00	11,700	49,69	2,340	2,340
			No	2,340	1,00	2,340	31,86		
B3	CS4	4	Yes	11,700	1,00	11,700	54,13	2,340	2,340
			No	2,340	1,00	2,340	31,40		
B3	CS4	5	Yes	11,700	1,00	11,700	58,84	2,340	2,340
			No	2,340	1,00	2,340	30,94		
B3	CS4	6	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	7	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	8	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	9	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	10	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	11	Yes	11,700	1,00	11,700	58,84	2,340	2,340
			No	2,340	1,00	2,340	30,94		
B3	CS4	12	Yes	11,700	1,00	11,700	54,13	2,340	2,340
			No	2,340	1,00	2,340	31,40		
B3	CS4	13	Yes	11,700	1,00	11,700	49,69	2,340	2,340
			No	2,340	1,00	2,340	31,86		
B3	CS4	14	Yes	11,700	1,00	11,700	45,68	2,340	2,340
			No	2,340	1,00	2,340	32,32		
B3	CS4	15	Yes	11,700	1,00	11,700	42,11	2,340	2,340
			No	2,340	1,00	2,340	32,76		
B4	CS7	1	Yes	5,882	1,00	5,882	31,38	2,941	2,941
			No	2,941	1,00	2,941	90,70		
B4	CS7	2	Yes	5,882	1,00	5,882	34,13	2,941	2,941
			No	2,941	1,00	2,941	88,90		
B4	CS7	3	Yes	5,882	1,00	5,882	37,25	2,941	2,941
			No	2,941	1,00	2,941	87,06		
B4	CS7	4	Yes	5,882	1,00	5,882	40,72	2,941	2,941
			No	2,941	1,00	2,941	85,19		
B4	CS7	5	Yes	5,882	1,00	5,882	44,40	2,941	2,941
			No	2,941	1,00	2,941	83,27		
B4	CS7	6	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		
B4	CS7	7	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		
B5	CS7	1	Yes	5,882	1,00	5,882	31,38	2,941	2,941
			No	2,941	1,00	2,941	90,70		
B5	CS7	2	Yes	5,882	1,00	5,882	34,13	2,941	2,941
			No	2,941	1,00	2,941	88,90		
B5	CS7	3	Yes	5,882	1,00	5,882	37,25	2,941	2,941
			No	2,941	1,00	2,941	87,06		
B5	CS7	4	Yes	5,882	1,00	5,882	40,72	2,941	2,941
			No	2,941	1,00	2,941	85,19		
B5	CS7	5	Yes	5,882	1,00	5,882	44,40	2,941	2,941
			No	2,941	1,00	2,941	83,27		
B5	CS7	6	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		
B5	CS7	7	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		

2. Iy



3. Iz



4. Result classes

4.1. Result classes - All MSN Nonlinear

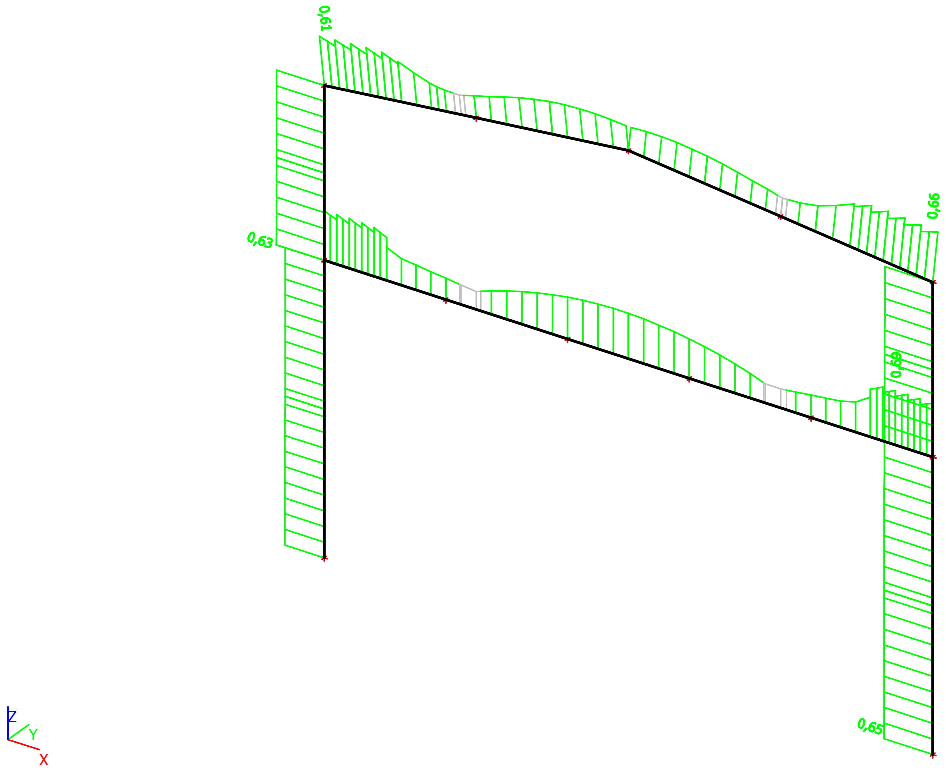
Name	List
All MSN Nonlinear	MSN1nelin
	MSN2nelin
	MSN3nelin
	MSN4nelin

Name	List
	MSN5nelin
	MSN6nelin
	MSN7nelin
	MSN8nelin
	MSN9nelin
	MSN10nelin
	MSN11nelin
	MSN12nelin
	MSN13nelin
	MSN14nelin

4.1.1. Check of steel

Nonlinear calculation, Extreme : Member
Selection : All
Class : All MSN Nonlinear

Member	css	mat	Case	dx [m]	un.check [-]	sec.check [-]	stab.check [-]
B1	CS2 - HEA450	S 235	MSN7nelin	5,440	0,63	0,62	0,63
B2	CS2 - HEA450	S 235	MSN9nelin	0,000	0,65	**0,11**	0,65
B3	CS4 - I + I var	S 235	MSN1nelin	10,980	**0,69**	**0,69**	**0,00**
B4	CS7 - I + I var	S 235	MSN3nelin	0,000	**0,61**	0,50	0,61
B5	CS7 - I + I var	S 235	MSN3nelin	0,000	0,66	0,55	**0,66**



Nonlinear calculation, Extreme : Cross-section
Selection : All
Class : All MSN Nonlinear

Member B2	**8,640 m**	**HEA450**	**S 235**	**MSN9nelin**	**0,65 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	29,91
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5,58
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-466,47	kN
Vy,Ed	0,00	kN
Vz,Ed	72,21	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	178,0000	cm^2
Nc,Rd	4183,00	kN
Unity check	0,11	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	65,7550	cm^2
Vpl,z,Rd	892,15	kN
Unity check	0,08	-

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	29,91
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5,58
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,440	5,440	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,440	5,440	m
Critical Euler load Ncr	44612,92	6632,41	kN
Slenderness Lambda	28,76	74,58	
Relative slenderness Lambda,rel	0,31	0,79	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	

Buckling parameters	yy	zz	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,98	0,73	
Buckling resistance Nb,Rd	4082,77	3045,44	kN

Flexural Buckling verification		
Cross-section area A	178,0000	cm^2
Buckling resistance Nb,Rd	3045,44	kN
Unity check	0,15	-

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.062	
kyz	0.899	
kzy	0.559	
kzz	1.114	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	178.0000	cm^2
Wy	3216.6700	cm^3
Wz	966.6670	cm^3
NRk	4183.00	kN
My,Rk	755.92	kNm
Mz,Rk	227.17	kNm
My,Ed	380.04	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	1798.25	kNm
reduced slenderness 0	0.65	
Psi y	0.000	
Psi z	1.000	
Cmy,0	0.996	
Cmz,0	1.017	
Cmy	0.999	
Cmz	1.017	
CmLT	1.052	
muy	1.000	
muz	0.980	
wy	1.109	
wz	1.500	
npl	0.112	
aLT	0.996	
bLT	0.000	
cLT	0.390	
dLT	0.000	
eLT	1.110	
Cyy	0.999	
Cyz	0.849	
Czy	0.960	
Czz	0.962	

Unity check (6.61) = 0.11 + 0.53 + 0.00 = 0.65
Unity check (6.62) = 0.15 + 0.28 + 0.00 = 0.43

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	34.609

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B3	**11,700 m**	**I + I var**	**S 235**	**MSN1nelin**	**0,69 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Welded	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.

The critical check is on position 10.980 m

Internal forces	Calculated	Unit
N,Ed	112,36	kN
Vy,Ed	0,00	kN
Vz,Ed	-369,76	kN
T,Ed	0,00	kNm
My,Ed	-557,15	kNm
Mz,Ed	0,00	kNm

Section properties

A	2.556822e+004 mm^2		
Ay/A	0.711	Az/A	0.271
Iy	1.194644e+009 mm^4	Iz	1.419727e+008 mm^4
Iyz	1.084202e-007 mm^4	It	3.051402e+006 mm^4
Iw	7.561526e+012 mm^6		
Wely	3.768931e+006 mm^3	Welz	9.464847e+005 mm^3
Wply	4.608365e+006 mm^3	Wplz	1.445252e+006 mm^3
cy	242.74 mm	cz	150.00 mm
dy	-0.00 mm	dz	-12.31 mm

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	255,6822	cm^2
Npl,Rd	6008,53	kN
Nu,Rd	6627,28	kN
Nt,Rd	6008,53	kN
Unity check	0,02	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Wel,y,min	3768,9311	cm^3
Mel,y,Rd	885,70	kNm
Unity check	0,63	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Tau,Vz,Ed	6,8	kN/cm^2
Tau,Rd	13,6	kN/cm^2
Unity check	0,50	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification		
Fibre	19	
Sigma,N,Ed	-0,4	kN/cm^2
Sigma,My,Ed	-12,5	kN/cm^2
Sigma,Mz,Ed	0,0	kN/cm^2
Sigma,tot,Ed	-13,0	kN/cm^2
Tau,Vy,Ed	0,0	kN/cm^2
Tau,Vz,Ed	5,7	kN/cm^2
Tau,t,Ed	0,0	kN/cm^2
Tau,tot,Ed	5,7	kN/cm^2
Sigma,von Mises,Ed	16,3	kN/cm^2
Unity check	0,69	-

The member satisfies the section check.

...::STABILITY CHECK::...

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	3768.9311	cm^3
Elastic critical moment Mcr	24288.26	kNm
Relative slenderness Lambda,LT	0.19	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.340	m
k	1.00	
kw	1.00	
C1	1.88	
C2	0.05	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

The member satisfies the stability check.

Member B5	5,882 m	I + I var	S 235	MSN3nelin	0,66 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Welded	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-187,88	kN
Vy,Ed	0,00	kN
Vz,Ed	70,17	kN
T,Ed	0,00	kNm
My,Ed	-125,80	kNm
Mz,Ed	0,00	kNm

Section properties

A	8.617282e+003 mm^2		
Ay/A	0.555	Az/A	0.453
Iy	3.027227e+008 mm^4	Iz	9.059495e+006 mm^4
Iyz	2.168404e-007 mm^4	It	2.452421e+005 mm^4
Iw	4.016613e+011 mm^6		
Wely	1.125756e+006 mm^3	Welz	1.207933e+005 mm^3
Wply	1.339771e+006 mm^3	Wplz	1.888354e+005 mm^3
cy	257.16 mm	cz	75.00 mm
dy	-0.00 mm	dz	-4.55 mm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	86,1728	cm^2
Nc,Rd	2025,06	kN
Unity check	0,09	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Wel,y,min	1125,7556	cm^3
Mel,y,Rd	264,55	kNm
Unity check	0,48	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Tau,Vz,Ed	2,2	kN/cm^2
Tau,Rd	13,6	kN/cm^2
Unity check	0,17	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification		
Fibre	26	
Sigma,N,Ed	2,2	kN/cm^2

Elastic verification		
Sigma,My,Ed	10,7	kN/cm^2
Sigma,Mz,Ed	0,0	kN/cm^2
Sigma,tot,Ed	12,9	kN/cm^2
Tau,Vy,Ed	0,0	kN/cm^2
Tau,Vz,Ed	0,4	kN/cm^2
Tau,t,Ed	0,0	kN/cm^2
Tau,tot,Ed	0,4	kN/cm^2
Sigma,von Mises,Ed	12,9	kN/cm^2
Unity check	0,55	-

The member satisfies the section check.

...::STABILITY CHECK::...

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,882	2,941	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,882	2,941	m
Critical Euler load Ncr	18136,59	2171,07	kN
Slenderness Lambda	31,38	90,70	
Relative slenderness Lambda,rel	0,33	0,97	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	b	c	
Imperfection Alpha	0,34	0,49	
Reduction factor Chi	0,95	0,56	
Buckling resistance Nb,Rd	1926,74	1134,10	kN

Flexural Buckling verification		
Cross-section area A	86,1728	cm^2
Buckling resistance Nb,Rd	1134,10	kN
Unity check	0,17	-

Torsional (-Flexural) Buckling check
According to article EN 1993-1-1 : 6.3.1.1. and formula (6.46)

Table of values		
Torsional Buckling length	2.941	m
Ncr,T	3205.89	kN
Ncr,TF	2168.48	kN
Relative slenderness Lambda,T	0.97	
Limit slenderness Lambda,0	0.20	
Buckling curve	c	
Imperfection Alpha	0.49	
A	86.1728	cm^2
Reduction factor Chi	0.56	
Buckling resistance Nb,Rd	1133.40	kN
Unity check	0.17	-

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	1125.7556	cm^3
Elastic critical moment Mcr	981.96	kNm
Relative slenderness Lambda,LT	0.52	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.941	m
k	1.00	
kw	1.00	
C1	1.96	
C2	0.05	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.082	
kyz	1.117	
kzy	1.039	
kzz	1.073	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	86.1728	cm^2
Wy	1125.7556	cm^3
Wz	120.7933	cm^3
NRk	2025.06	kN
My,Rk	264.55	kNm
Mz,Rk	28.39	kNm
My,Ed	-125.80	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	501.98	kNm
reduced slenderness 0	0.73	
Psi y	-0.160	
Psi z	1.000	
Cmy,0	0.993	
Cmz,0	1.021	
Cmy	0.998	
Cmz	1.021	
CmLT	1.073	
muy	0.999	
muz	0.960	
wy	1.190	
wz	1.500	
npl	0.093	
aLT	0.999	
bLT	0.000	
cLT	0.359	
dLT	0.000	
eLT	0.509	
Cyy	0.990	
Cyz	0.830	
Czy	0.939	
Czz	0.971	

Unity check (6.61) = 0.10 + 0.51 + 0.00 = 0.61
Unity check (6.62) = 0.17 + 0.49 + 0.00 = 0.66

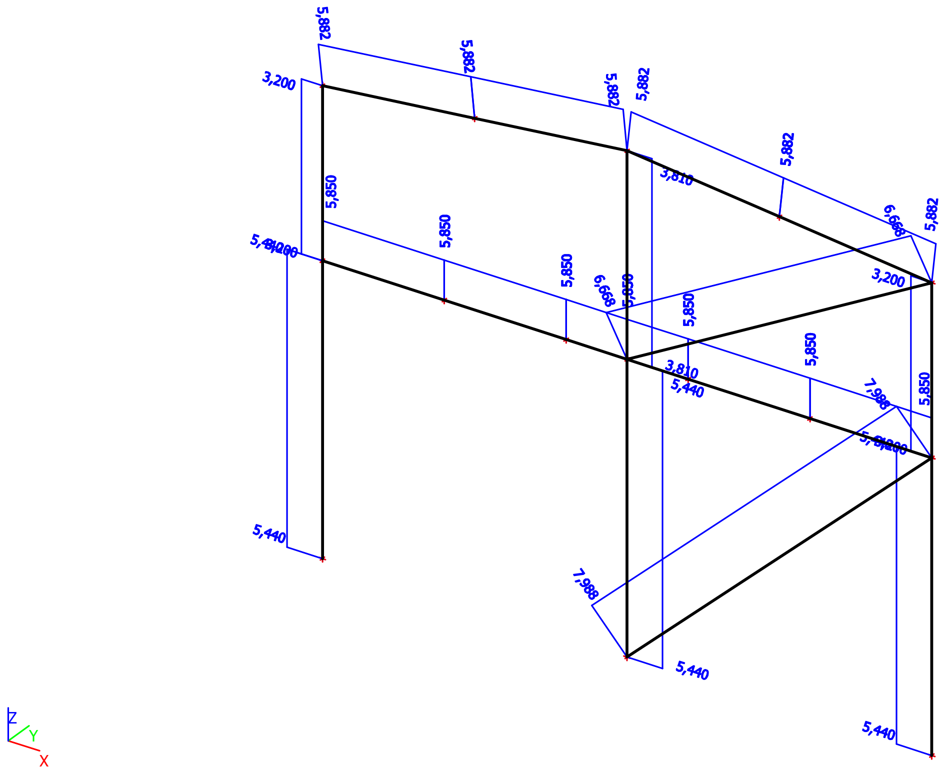
The member satisfies the stability check.

1. Priloga A.2

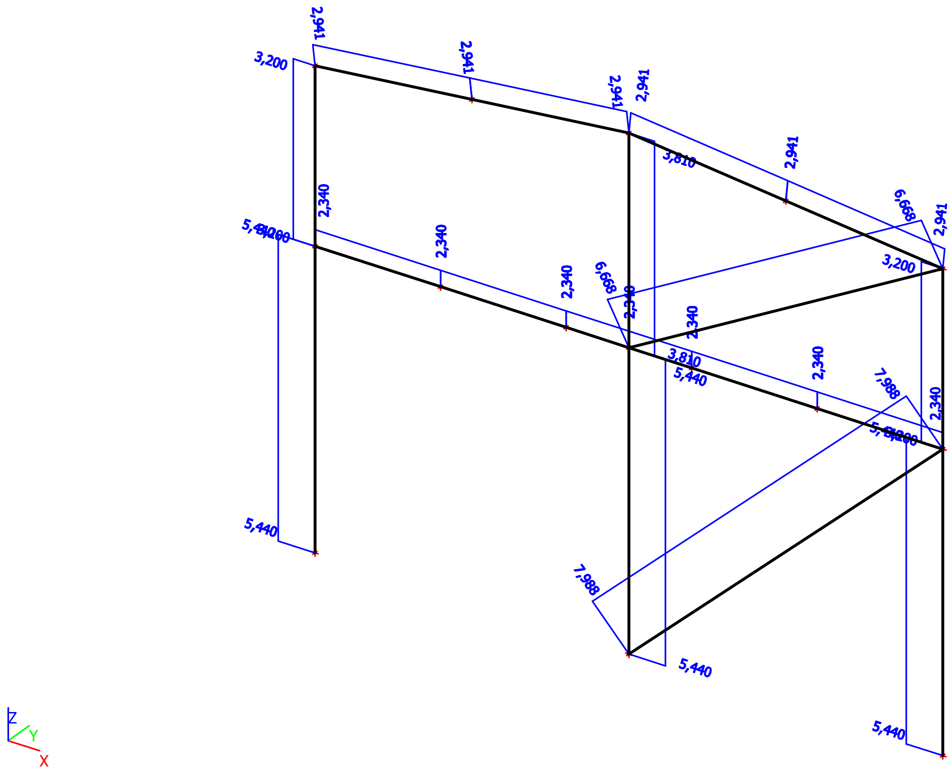
Linear calculation

Member	CS Name	Part	Sway y	Ly [m]	ky [-]	ly [m]	Lam y [-]	lyz [m]	l LTB [m]
			Sway z	Lz [m]	kz [-]	lz [m]	Lam z [-]		
B1	CS2	1	Yes	5,440	1,00	5,440	59,31	5,440	5,440
			No	5,440	1,00	5,440	98,53		
B1	CS2	2	Yes	3,200	1,00	3,200	34,89	3,200	3,200
			No	3,200	1,00	3,200	57,96		
B2	CS2	1	Yes	5,440	1,00	5,440	59,31	5,440	5,440
			No	5,440	1,00	5,440	98,53		
B2	CS2	2	Yes	3,200	1,00	3,200	34,89	3,200	3,200
			No	3,200	1,00	3,200	57,96		
B3	CS2	1	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS2	2	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS2	3	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS2	4	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS2	5	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS2	6	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B4	CS3	1	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B4	CS3	2	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B5	CS3	1	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B5	CS3	2	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B6	CS9	1	Yes	3,810	1,00	3,810	66,34	3,810	3,810
			No	3,810	1,00	3,810	230,26		
B7	CS10	1	Yes	5,440	1,00	5,440	82,92	5,440	5,440
			No	5,440	1,00	5,440	136,53		
B11	CS12	1	Yes	7,988	1,00	7,988	2690,50	7,988	7,988
			No	7,988	1,00	7,988	2690,50		
B12	CS12	1	Yes	6,668	1,00	6,668	2245,77	6,668	6,668
			No	6,668	1,00	6,668	2245,77		

2. Iy



3. Iz



4. Result classes

4.1. Result classes - All MSN Nonlinear

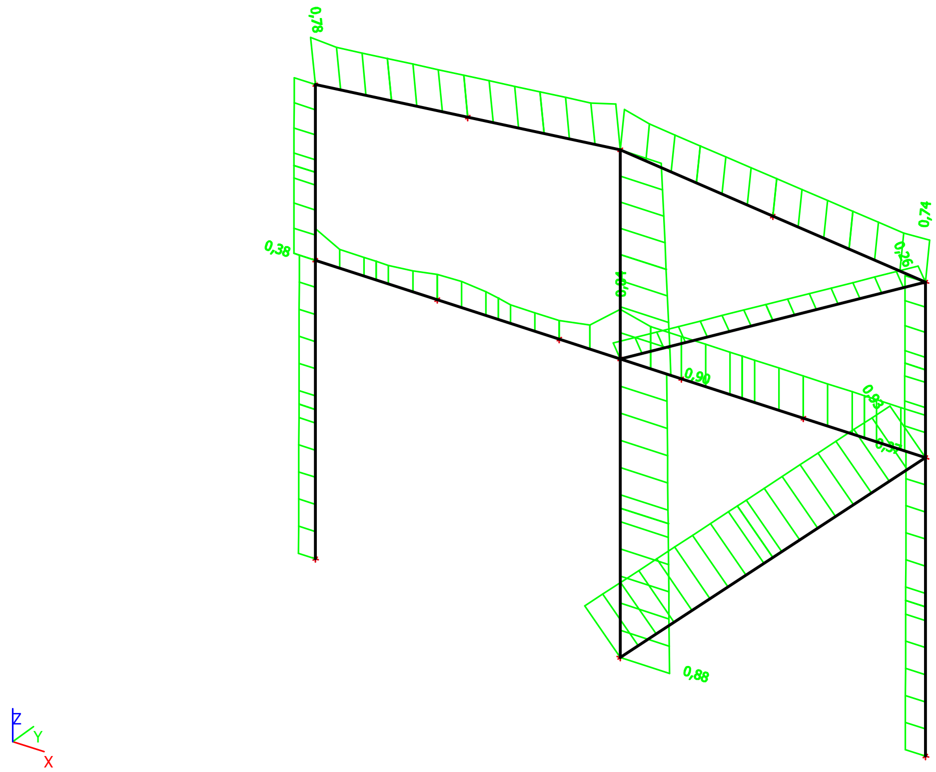
Name	List
All MSN Nonlinear	MSN1nelin
	MSN2nelin
	MSN3nelin
	MSN4nelin

Name	List
	MSN5nelin
	MSN6nelin
	MSN7nelin
	MSN8nelin
	MSN9nelin
	MSN10nelin
	MSN11nelin
	MSN12nelin
	MSN13nelin
	MSN14nelin

4.1.1. Check of steel

Nonlinear calculation, Extreme : Member
Selection : All
Class : All MSN Nonlinear

Member	css	mat	Case	dx [m]	un.check [-]	sec.check [-]	stab.check [-]
B1	CS2 - HEA220	S 235	MSN7nelin	5,440	0,38	0,36	0,38
B2	CS2 - HEA220	S 235	MSN2nelin	5,440	0,37	0,35	0,37
B3	CS2 - HEA220	S 235	MSN10nelin	5,850	0,84	0,84	**0,00**
B4	CS3 - IPE180	S 235	MSN3nelin	0,000	0,78	0,59	0,78
B5	CS3 - IPE180	S 235	MSN3nelin	0,000	0,74	0,54	0,74
B6	CS9 - IPE140	S 235	MSN3nelin	3,810	0,90	**0,13**	**0,90**
B7	CS10 - HEA160	S 235	MSN7nelin	5,440	0,88	0,29	0,88
B11	CS12 - RD12	S 235	MSN5nelin	7,988	**0,93**	**0,93**	0,00
B12	CS12 - RD12	S 235	MSN5nelin	6,668	**0,26**	0,26	0,00



Nonlinear calculation, Extreme : Cross-section
Selection : All
Class : All MSN Nonlinear

Member B3	**11,700 m**	**HEA220**	**S 235**	**MSN10nelin**	**0,84 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	21,71
Class 1 Limit	72,67
Class 2 Limit	83,77
Class 3 Limit	124,01

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	8,05
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 5.850 m

Internal forces	Calculated	Unit
N,Ed	9,31	kN
Vy,Ed	0,00	kN
Vz,Ed	-108,74	kN
T,Ed	0,00	kNm
My,Ed	-111,59	kNm
Mz,Ed	0,00	kNm

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	64,3000	cm^2
Npl,Rd	1511,05	kN
Nu,Rd	1666,66	kN
Nt,Rd	1511,05	kN
Unity check	0,01	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	566,6670	cm^3
Mpl,y,Rd	133,17	kNm
Unity check	0,84	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	20,6300	cm^2
Vpl,z,Rd	279,90	kN
Unity check	0,39	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	133,17	kNm
Alpha	2,00	
Mpl,z,Rd	63,55	kNm
Beta	1,00	

Unity check (6.41) = 0,70 + 0,00 = 0,70 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	21,71
Class 1 Limit	73,16

Class 2 Limit	84,34
Class 3 Limit	124,02

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	8,05
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	566.6670	cm^3
Elastic critical moment Mcr	1138.65	kNm
Relative slenderness Lambda,LT	0.34	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.340	m
k	1.00	
kw	1.00	
C1	1.35	
C2	0.63	
C3	0.41	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	26.857

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B4	**5,882 m**	**IPE180**	**S 235**	**MSN3nelin**	**0,78 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	27,55
Class 1 Limit	61,91
Class 2 Limit	71,29
Class 3 Limit	107,57

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,23
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-25,08	kN
Vy,Ed	0,00	kN
Vz,Ed	23,29	kN
T,Ed	0,00	kNm
My,Ed	-22,88	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	23,9000	cm^2
Nc,Rd	561,65	kN
Unity check	0,04	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	166,0000	cm^3
Mpl,y,Rd	39,01	kNm
Unity check	0,59	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	11,2040	cm^2
Vpl,z,Rd	152,01	kN
Unity check	0,15	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	39,01	kNm
Alpha	2,00	
Mpl,z,Rd	8,13	kNm
Beta	1,00	

Unity check (6.41) = 0,34 + 0,00 = 0,34 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	27,55
Class 1 Limit	61,91
Class 2 Limit	71,29
Class 3 Limit	107,57

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,23
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,882	2,941	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,882	2,941	m
Critical Euler load Ncr	789,04	242,04	kN
Slenderness Lambda	79,23	143,06	
Relative slenderness Lambda,rel	0,84	1,52	

Buckling parameters	yy	zz	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,77	0,33	
Buckling resistance Nb,Rd	432,36	187,48	kN

Flexural Buckling verification		
Cross-section area A	23,9000	cm^2
Buckling resistance Nb,Rd	187,48	kN
Unity check	0,13	-

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	166.0000	cm^3
Elastic critical moment Mcr	116.26	kNm
Relative slenderness Lambda,LT	0.58	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	a	
Imperfection Alpha,LT	0.21	
Reduction factor Chi,LT	0.90	
Buckling resistance Mb.Rd	35.02	kNm
Unity check	0.65	-

Mcr Parameters		
LTB length	2.941	m
k	1.00	
kw	1.00	
C1	3.14	
C2	0.43	
C3	1.00	

Note: C Parameters according to ECCS 119 2006 / Galea 2002
load in center of gravity

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.102	
kyz	1.277	
kzy	0.581	
kzz	1.125	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	23.9000	cm^2
Wy	166.0000	cm^3
Wz	34.6000	cm^3
NRk	561.65	kN
My,Rk	39.01	kNm
Mz,Rk	8.13	kNm
My,Ed	-22.88	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	36.98	kNm
reduced slenderness 0	1.03	
Psi y	0.945	
Psi z	1.000	
Cmy,0	0.979	
Cmz,0	1.025	
Cmy	0.996	
Cmz	1.025	
CmLT	1.057	
muy	0.993	
muz	0.929	
wy	1.137	
wz	1.500	
npl	0.045	
aLT	0.996	
bLT	0.000	
cLT	0.664	
dLT	0.000	
eLT	0.208	
Cyy	0.979	
Cyz	0.612	

Table of values		
Czy	0.909	
Czz	0.944	

Unity check (6.61) = 0.06 + 0.72 + 0.00 = 0.78
Unity check (6.62) = 0.13 + 0.38 + 0.00 = 0.51

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	30.943

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B6	3,810 m	IPE140	S 235	MSN3nelin	0,90 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	23,87
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	3,93
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 3.810   m

Internal forces	Calculated	Unit
N,Ed	-50,25	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	16,4000	cm^2
Nc,Rd	385,40	kN
Unity check	0,13	-

Shear check for Vy
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	10,6239	cm^2
Vpl,y,Rd	144,14	kN
Unity check	0,00	-

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	23,87
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	3,93
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	3,810	3,810	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	3,810	3,810	m
Critical Euler load Ncr	772,44	64,11	kN
Slenderness Lambda	66,34	230,26	
Relative slenderness Lambda,rel	0,71	2,45	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,84	0,14	
Buckling resistance Nb,Rd	325,56	55,80	kN

Warning: Slenderness 230,26 is larger than the limit value of 200,00.

Flexural Buckling verification		
Cross-section area A	16,4000	cm^2
Buckling resistance Nb,Rd	55,80	kN
Unity check	0,90	-

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
kyy	2.826	
kyz	7.348	
kzy	0.697	
kzz	1.812	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	16.4000	cm^2
Wy	88.3000	cm^3
Wz	19.3000	cm^3
NRk	385.40	kN
My,Rk	20.75	kNm
Mz,Rk	4.54	kNm
My,Ed	0.00	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	12.04	kNm
reduced slenderness 0	1.31	
Psi y	1.000	
Psi z	1.000	
Cmy,0	1.016	
Cmz,0	1.024	
Cmy	1.016	
Cmz	1.024	
CmLT	2.302	
muy	0.989	
muz	0.244	
wy	1.142	
wz	1.500	
npl	0.130	
aLT	0.995	
bLT	0.000	
cLT	0.000	
dLT	0.000	

Table of values		
eLT	0.000	
Cyy	0.875	
Cyz	0.438	
Czy	0.458	
Czz	0.637	

Unity check (6.61) = 0.15 + 0.00 + 0.01 = 0.16
Unity check (6.62) = 0.90 + 0.00 + 0.00 = 0.90

The member satisfies the stability check.

Member B7	5,440 m	HEA160	S 235	MSN7nelin	0,88 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	17,33
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6,89
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 5.440 m

Internal forces	Calculated	Unit
N,Ed	-263,76	kN
Vy,Ed	0,01	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	38,8000	cm^2
Nc,Rd	911,80	kN
Unity check	0,29	-

Shear check for Vy
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	30,0600	cm^2
Vpl,y,Rd	407,85	kN
Unity check	0,00	-

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	17,33
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6,89
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,440	5,440	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,440	5,440	m
Critical Euler load Ncr	1169,60	431,42	kN
Slenderness Lambda	82,92	136,53	
Relative slenderness Lambda,rel	0,88	1,45	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	b	c	
Imperfection Alpha	0,34	0,49	
Reduction factor Chi	0,67	0,33	
Buckling resistance Nb,Rd	612,85	300,88	kN

Flexural Buckling verification		
Cross-section area A	38,8000	cm^2
Buckling resistance Nb,Rd	300,88	kN
Unity check	0,88	-

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
kyy	2.625	
kyz	2.363	
kzy	1.400	
kzz	1.775	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	38.8000	cm^2
Wy	245.0000	cm^3
Wz	117.5000	cm^3
NRk	911.80	kN
My,Rk	57.58	kNm
Mz,Rk	27.61	kNm
My,Ed	0.00	kNm
Mz,Ed	-0.01	kNm
Interaction Method 1		
Mcr0	72.11	kNm
reduced slenderness 0	0.89	
Psi y	1.000	
Psi z	1.000	
Cmy,0	1.054	
Cmz,0	1.018	
Cmy	1.054	
Cmz	1.018	
CmLT	1.897	
muy	0.913	
muz	0.487	
wy	1.114	
wz	1.500	
npl	0.289	
aLT	0.993	
bLT	0.000	
cLT	0.000	
dLT	0.000	
eLT	0.000	
Cyy	0.898	
Cyz	0.705	
Czy	0.464	
Czz	0.719	

Unity check (6.61) = 0.43 + 0.00 + 0.00 = 0.43
Unity check (6.62) = 0.88 + 0.00 + 0.00 = 0.88

The member satisfies the stability check.

Member B11	**7,988 m**	**RD12**	**S 235**	**MSN5nelin**	**0,93 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.

The critical check is on position 7.988 m

Internal forces	Calculated	Unit
N,Ed	24,67	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	1,1304	cm^2
Npl,Rd	26,56	kN
Nu,Rd	29,30	kN
Nt,Rd	26,56	kN
Unity check	0,93	-

The member satisfies the section check.

...::STABILITY CHECK::...

The member satisfies the stability check.

PRILOGA B: IZPIS IZ PROGRAMA SCIA ENGINEER 14

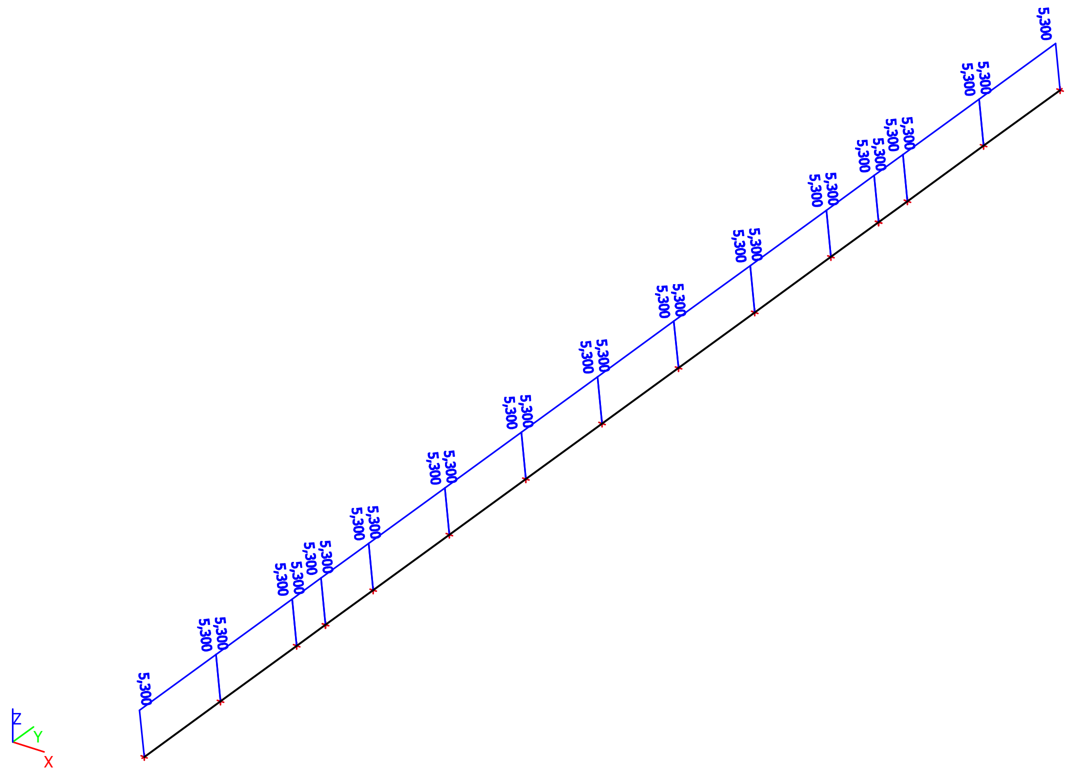
Kontrola nosilnosti in stabilnosti strešne lege - MSN

1. Priloga B

Linear calculation

Member	CS Name	Part	Sway y	Ly [m]	ky [-]	ly [m]	Lam y [-]	lyz [m]	l LTB [m]
			Sway z	Lz [m]	kz [-]	lz [m]	Lam z [-]		
B1	CS2	1	Yes	5,300	1,00	5,300	64,19	2,650	2,650
			No	2,650	1,00	2,650	118,72		
B2	CS2	1	Yes	5,300	1,00	5,300	64,19	2,650	2,650
			No	2,650	1,00	2,650	118,72		
B3	CS2	1	Yes	5,300	1,00	5,300	64,19	2,650	2,650
			No	2,650	1,00	2,650	118,72		
B4	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B5	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B6	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B7	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B8	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B9	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B10	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B11	CS2	1	Yes	5,300	1,00	5,300	64,19	2,650	2,650
			No	2,650	1,00	2,650	118,72		
B12	CS2	1	Yes	5,300	1,00	5,300	64,19	2,650	2,650
			No	2,650	1,00	2,650	118,72		
B13	CS1	1	Yes	5,300	1,00	5,300	71,40	2,650	2,650
			No	2,650	1,00	2,650	128,91		
B14	CS2	1	Yes	5,300	1,00	5,300	64,19	2,650	2,650
			No	2,650	1,00	2,650	118,72		

2. ly



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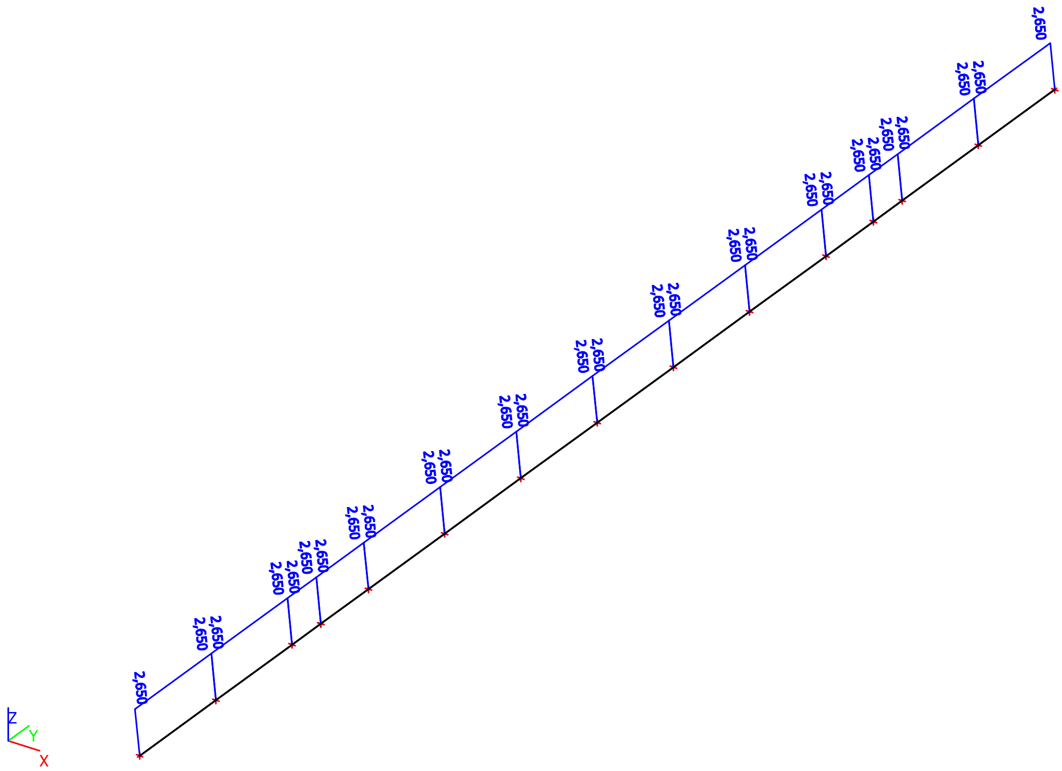


4. Result classes

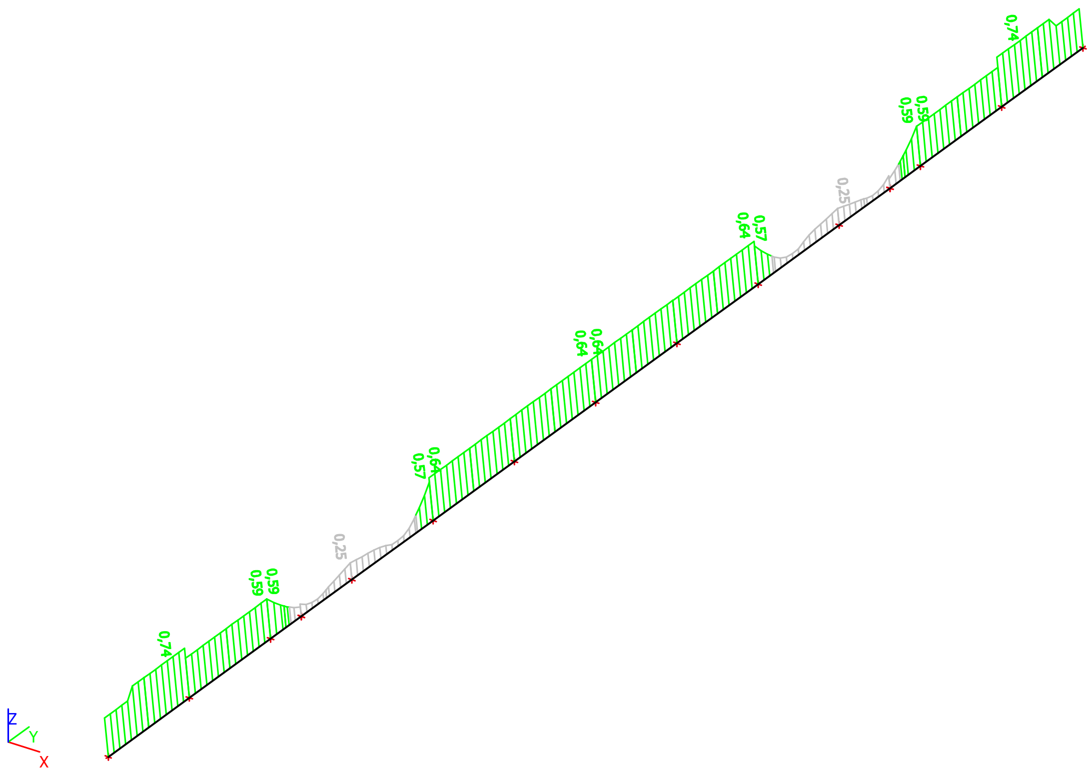
4.1. Result classes - All MSNnelin

Name	List
All MSNnelin	MSN1nelin
	MSN2nelin

4.1.1. Check of steel

Nonlinear calculation, Extreme : Member
Selection : All
Class : All MSNnelin

Member	css	mat	Case	dx [m]	un.check [-]	sec.check [-]	stab.check [-]
B1	CS2 - IPE200	S 235	MSN1nelin	2,082	0,74	0,37	0,74
B2	CS2 - IPE200	S 235	MSN1nelin	2,650	0,59	0,52	0,59
B3	CS2 - IPE200	S 235	MSN1nelin	0,000	0,59	0,52	0,59
B4	CS1 - IPE180	S 235	MSN1nelin	2,650	0,57	0,49	0,57
B5	CS1 - IPE180	S 235	MSN1nelin	0,000	0,64	0,49	0,64
B6	CS1 - IPE180	S 235	MSN1nelin	2,650	0,64	**0,56**	0,64
B7	CS1 - IPE180	S 235	MSN1nelin	0,000	0,64	0,56	0,64
B8	CS1 - IPE180	S 235	MSN1nelin	2,650	0,64	0,49	0,64
B9	CS1 - IPE180	S 235	MSN1nelin	0,000	0,57	0,49	0,57
B10	CS1 - IPE180	S 235	MSN1nelin	0,000	**0,25**	**0,21**	**0,25**
B11	CS2 - IPE200	S 235	MSN1nelin	0,000	0,59	0,52	0,59
B12	CS2 - IPE200	S 235	MSN1nelin	0,568	**0,74**	0,37	**0,74**
B13	CS1 - IPE180	S 235	MSN1nelin	1,650	0,25	0,21	0,25
B14	CS2 - IPE200	S 235	MSN1nelin	1,000	0,59	0,52	0,59



Nonlinear calculation, Extreme : Cross-section
Selection : All
Class : All MSNnelin

Member B12	**2,650 m**	**IPE200**	**S 235**	**MSN1nelin**	**0,74 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	28,39
Class 1 Limit	71,36
Class 2 Limit	82,17
Class 3 Limit	121,87

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,14
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,83

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 0.568 m

Internal forces	Calculated	Unit
N,Ed	-1,59	kN
Vy,Ed	-0,82	kN
Vz,Ed	-0,04	kN
T,Ed	0,00	kNm
My,Ed	19,35	kNm
Mz,Ed	-0,16	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	28,5000	cm^2
Nc,Rd	669,75	kN
Unity check	0,00	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	221,0000	cm^3
Mpl,y,Rd	51,94	kNm
Unity check	0,37	-

Bending moment check for Mz
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,z	44,6000	cm^3
Mpl,z,Rd	10,48	kNm
Unity check	0,02	-

Shear check for Vy
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	17,9856	cm^2
Vpl,y,Rd	244,02	kN
Unity check	0,00	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	14,0160	cm^2
Vpl,z,Rd	190,17	kN
Unity check	0,00	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	51,94	kNm
Alpha	2,00	
Mpl,z,Rd	10,48	kNm
Beta	1,00	

Unity check (6.41) = 0,14 + 0,02 = 0,15 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	28,39
Class 1 Limit	71,36
Class 2 Limit	82,17
Class 3 Limit	121,75

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,14
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,93

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,300	2,650	m
Buckling factor k	1,00	1,00	

Buckling parameters	yy	zz	
Buckling length Lcr	5,300	2,650	m
Critical Euler load Ncr	1433,64	419,10	kN
Slenderness Lambda	64,19	118,72	
Relative slenderness Lambda,rel	0,68	1,26	
Limit slenderness Lambda,rel,0	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	221.0000	cm^3
Elastic critical moment Mcr	74.26	kNm
Relative slenderness Lambda,LT	0.84	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	a	
Imperfection Alpha,LT	0.21	
Reduction factor Chi,LT	0.77	
Buckling resistance Mb.Rd	40.21	kNm
Unity check	0.48	-

Mcr Parameters		
LTB length	2.650	m
k	1.00	
kw	1.00	
C1	1.18	
C2	0.18	
C3	1.00	

Note: C Parameters according to ECCS 119 2006 / Galea 2002
load in center of gravity

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.004	
kyz	1.095	
kzy	0.527	
kzz	1.005	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	28.5000	cm^2
Wy	221.0000	cm^3
Wz	44.6000	cm^3
NRk	669.75	kN
My,Rk	51.94	kNm
Mz,Rk	10.48	kNm
My,Ed	-27.20	kNm
Mz,Ed	-0.55	kNm
Interaction Method 1		
Mcr0	63.01	kNm
reduced slenderness 0	0.91	
Psi y	0.000	
Psi z	0.000	
Cmy,0	1.000	
Cmz,0	1.000	
Cmy	1.000	
Cmz	1.000	
CmLT	1.000	
muy	1.000	
muz	1.000	
wy	1.139	
wz	1.500	
npl	0.002	
aLT	0.996	
bLT	0.015	
cLT	0.735	
dLT	0.024	
eLT	0.392	
Cyy	0.997	
Cyz	0.631	
Czy	0.993	

Table of values		
Czz	0.998	

Unity check (6.61) = 0.00 + 0.68 + 0.06 = 0.74
Unity check (6.62) = 0.00 + 0.36 + 0.05 = 0.41

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	32.679

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B6	2,650 m	IPE180	S 235	MSN1nelin	0,64 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	27,55
Class 1 Limit	71,26
Class 2 Limit	82,06
Class 3 Limit	122,22

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,23
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,96

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 2.650 m

Internal forces	Calculated	Unit
N,Ed	-1,59	kN
Vy,Ed	1,24	kN
Vz,Ed	-24,01	kN
T,Ed	0,00	kNm
My,Ed	-21,67	kNm
Mz,Ed	0,64	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	23,9000	cm^2
Nc,Rd	561,65	kN
Unity check	0,00	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	166,0000	cm^3
Mpl,y,Rd	39,01	kNm
Unity check	0,56	-

Bending moment check for Mz
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,z	34,6000	cm^3
Mpl,z,Rd	8,13	kNm
Unity check	0,08	-

Shear check for Vy
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	15,3179	cm^2
Vpl,y,Rd	207,83	kN
Unity check	0,01	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	11,2040	cm^2
Vpl,z,Rd	152,01	kN
Unity check	0,16	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	39,01	kNm
Alpha	2,00	
Mpl,z,Rd	8,13	kNm
Beta	1,00	

Unity check (6.41) = 0,31 + 0,08 = 0,39 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	27,55
Class 1 Limit	71,26
Class 2 Limit	82,06
Class 3 Limit	120,98

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,23
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,99

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,300	2,650	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,300	2,650	m
Critical Euler load Ncr	971,75	298,09	kN
Slenderness Lambda	71,40	128,91	
Relative slenderness Lambda,rel	0,76	1,37	
Limit slenderness Lambda,rel,0	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	166.0000	cm^3
Elastic critical moment Mcr	136.22	kNm
Relative slenderness Lambda,LT	0.54	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.650	m
k	1.00	
kw	1.00	
C1	3.20	
C2	0.41	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.005	
kyz	0.984	
kzy	0.527	
kzz	1.004	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	23.9000	cm^2
Wy	166.0000	cm^3
Wz	34.6000	cm^3
NRk	561.65	kN
My,Rk	39.01	kNm
Mz,Rk	8.13	kNm
My,Ed	-21.67	kNm
Mz,Ed	0.64	kNm
Interaction Method 1		
Mcr0	42.51	kNm
reduced slenderness 0	0.96	
Psi y	0.885	
Psi z	0.568	
Cmy,0	0.999	
Cmz,0	0.996	
Cmy	1.000	
Cmz	0.996	
CmLT	1.000	
muy	1.000	
muz	1.000	
wy	1.137	
wz	1.500	
npl	0.003	
aLT	0.996	
bLT	0.020	
cLT	0.594	
dLT	0.023	
eLT	0.247	
Cyy	0.996	
Cyz	0.701	
Czy	0.992	
Czz	0.998	

Unity check (6.61) = 0.00 + 0.56 + 0.08 = 0.64
Unity check (6.62) = 0.00 + 0.29 + 0.08 = 0.37

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	30.943

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

PRILOGA C: IZPIS IZ PROGRAMA SCIA ENGINEER 14

Priloga C.1: Kontrola nosilnosti in stabilnosti momentnega okvira 2 - potresno projektno stanje, faktor obnašanja q = 1,5

Priloga C.2: Kontrola nosilnosti in stabilnosti okvira 1 s centričnim povezjem (sistem nateznih diagonal) - potresno projektno stanje, faktor obnašanja q = 1,5

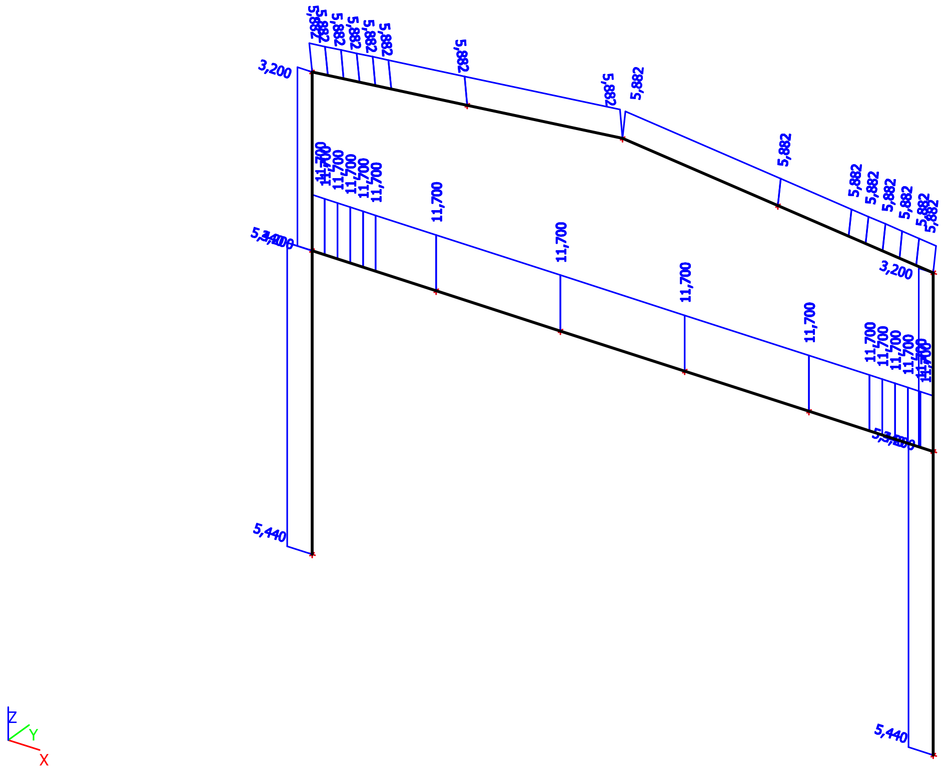
Priloga C.3: Kontrola nosilnosti in stabilnosti okvira A s centričnim povezjem (sistem nateznih diagonal) - potresno projektno stanje, faktor obnašanja q = 1,5

1. Priloga C.1

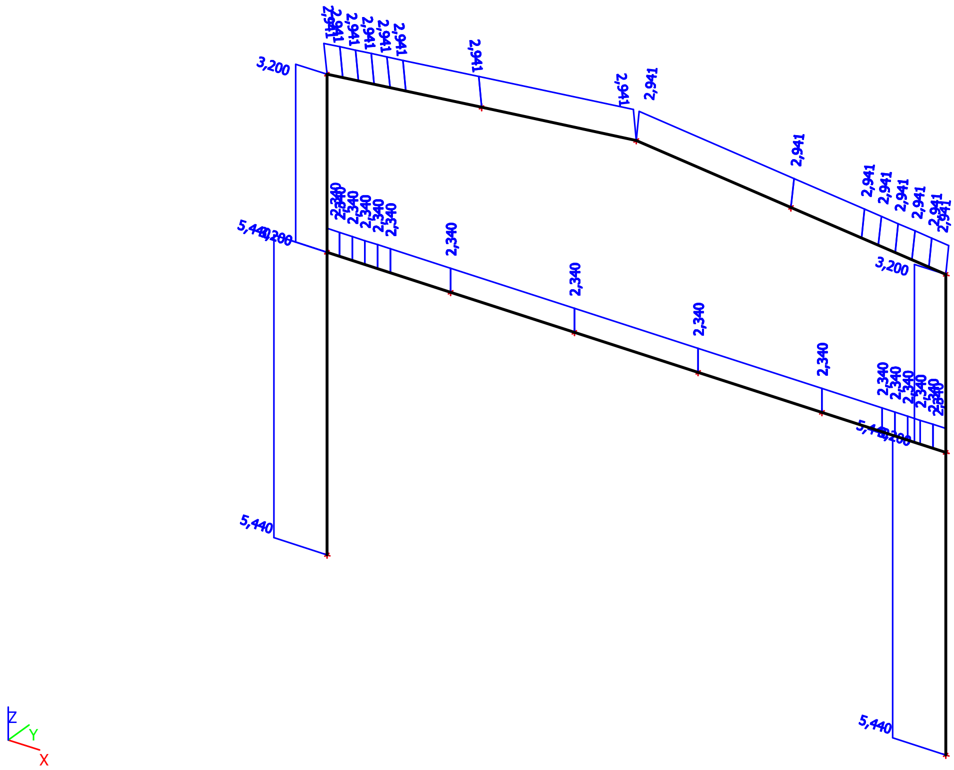
Linear calculation

Member	CS Name	Part	Sway y	Ly [m]	ky [-]	ly [m]	Lam y [-]	lyz [m]	l LTB [m]
			Sway z	Lz [m]	kz [-]	lz [m]	Lam z [-]		
B1	CS1	1	Yes	5,440	1,00	5,440	25,95	5,440	5,440
			No	5,440	1,00	5,440	75,06		
B1	CS1	2	Yes	3,200	1,00	3,200	15,27	3,200	3,200
			No	3,200	1,00	3,200	44,15		
B2	CS1	1	Yes	5,440	1,00	5,440	25,95	5,440	5,440
			No	5,440	1,00	5,440	75,06		
B2	CS1	2	Yes	3,200	1,00	3,200	15,27	3,200	3,200
			No	3,200	1,00	3,200	44,15		
B3	CS4	1	Yes	11,700	1,00	11,700	42,11	2,340	2,340
			No	2,340	1,00	2,340	32,76		
B3	CS4	2	Yes	11,700	1,00	11,700	45,68	2,340	2,340
			No	2,340	1,00	2,340	32,32		
B3	CS4	3	Yes	11,700	1,00	11,700	49,69	2,340	2,340
			No	2,340	1,00	2,340	31,86		
B3	CS4	4	Yes	11,700	1,00	11,700	54,13	2,340	2,340
			No	2,340	1,00	2,340	31,40		
B3	CS4	5	Yes	11,700	1,00	11,700	58,84	2,340	2,340
			No	2,340	1,00	2,340	30,94		
B3	CS4	6	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	7	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	8	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	9	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	10	Yes	11,700	1,00	11,700	61,84	2,340	2,340
			No	2,340	1,00	2,340	32,10		
B3	CS4	11	Yes	11,700	1,00	11,700	58,84	2,340	2,340
			No	2,340	1,00	2,340	30,94		
B3	CS4	12	Yes	11,700	1,00	11,700	54,13	2,340	2,340
			No	2,340	1,00	2,340	31,40		
B3	CS4	13	Yes	11,700	1,00	11,700	49,69	2,340	2,340
			No	2,340	1,00	2,340	31,86		
B3	CS4	14	Yes	11,700	1,00	11,700	45,68	2,340	2,340
			No	2,340	1,00	2,340	32,32		
B3	CS4	15	Yes	11,700	1,00	11,700	42,11	2,340	2,340
			No	2,340	1,00	2,340	32,76		
B4	CS7	1	Yes	5,882	1,00	5,882	31,38	2,941	2,941
			No	2,941	1,00	2,941	90,70		
B4	CS7	2	Yes	5,882	1,00	5,882	34,13	2,941	2,941
			No	2,941	1,00	2,941	88,90		
B4	CS7	3	Yes	5,882	1,00	5,882	37,25	2,941	2,941
			No	2,941	1,00	2,941	87,06		
B4	CS7	4	Yes	5,882	1,00	5,882	40,72	2,941	2,941
			No	2,941	1,00	2,941	85,19		
B4	CS7	5	Yes	5,882	1,00	5,882	44,40	2,941	2,941
			No	2,941	1,00	2,941	83,27		
B4	CS7	6	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		
B4	CS7	7	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		
B5	CS7	1	Yes	5,882	1,00	5,882	31,38	2,941	2,941
			No	2,941	1,00	2,941	90,70		
B5	CS7	2	Yes	5,882	1,00	5,882	34,13	2,941	2,941
			No	2,941	1,00	2,941	88,90		
B5	CS7	3	Yes	5,882	1,00	5,882	37,25	2,941	2,941
			No	2,941	1,00	2,941	87,06		
B5	CS7	4	Yes	5,882	1,00	5,882	40,72	2,941	2,941
			No	2,941	1,00	2,941	85,19		
B5	CS7	5	Yes	5,882	1,00	5,882	44,40	2,941	2,941
			No	2,941	1,00	2,941	83,27		
B5	CS7	6	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		
B5	CS7	7	Yes	5,882	1,00	5,882	47,20	2,941	2,941
			No	2,941	1,00	2,941	87,82		

2. Iy



3. Iz



4. Result classes

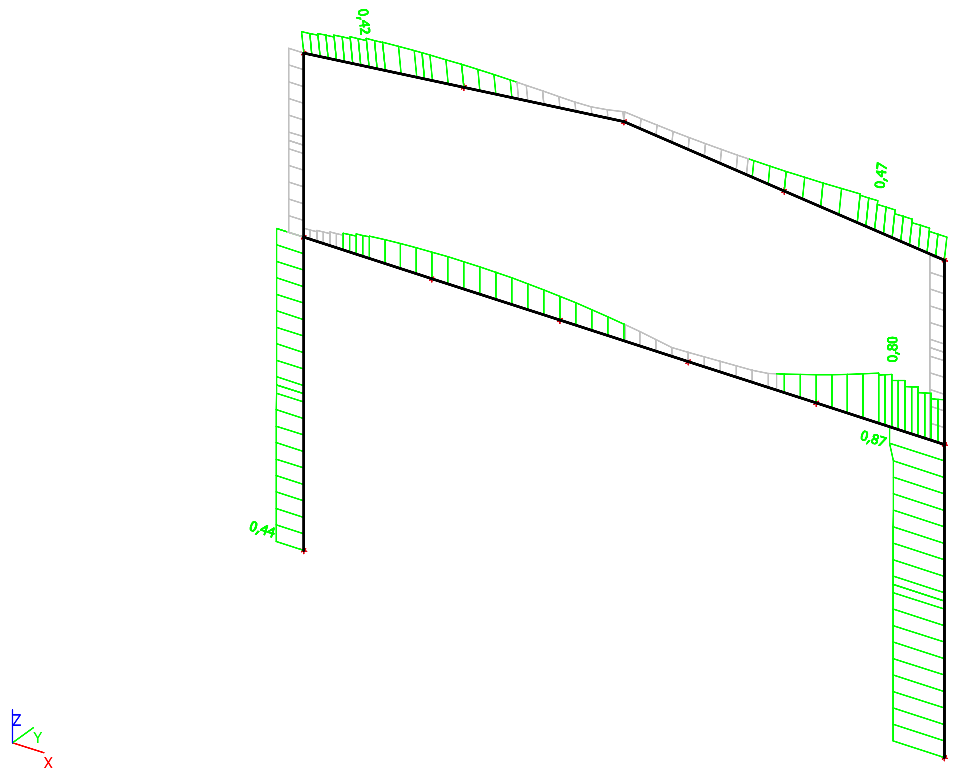
4.1. Result classes - RC1

Name	List
RC1	Skupaj

4.1.1. Check of steel

Nonlinear calculation, Extreme : Member
Selection : All
Class : RC1

Member	css	mat	Case	dx [m]	un.check [-]	sec.check [-]	stab.check [-]
B1	CS1 - HEA500	S 235	Skupaj	0,000	0,44	**0,07**	0,44
B2	CS1 - HEA500	S 235	Skupaj	5,154	**0,87**	0,68	**0,87**
B3	CS4 - I + I var	S 235	Skupaj	10,740	0,80	**0,80**	0,80
B4	CS7 - I + I var	S 235	Skupaj	1,200	**0,42**	0,38	**0,42**
B5	CS7 - I + I var	S 235	Skupaj	1,280	0,47	0,37	0,47



Nonlinear calculation, Extreme : Cross-section
Selection : All
Class : RC1

Member B2	**8,640 m**	**HEA500**	**S 235**	**Skupaj**	**0,87 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	32,50
Class 1 Limit	53,67
Class 2 Limit	61,80
Class 3 Limit	103,16

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5,09

Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 5.154 m

Internal forces	Calculated	Unit
N,Ed	-317,77	kN
Vy,Ed	0,00	kN
Vz,Ed	120,65	kN
T,Ed	0,00	kNm
My,Ed	632,27	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	198,0000	cm^2
Nc,Rd	4653,00	kN
Unity check	0,07	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	3950,0000	cm^3
Mpl,y,Rd	928,25	kNm
Unity check	0,68	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	75,1800	cm^2
Vpl,z,Rd	1020,02	kN
Unity check	0,12	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Mpl,y,Rd	928,25	kNm
Unity check	0,68	-

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	32,50
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5,09
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,440	5,440	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,440	5,440	m
Critical Euler load Ncr	60931,31	7283,74	kN

Buckling parameters	yy	zz	
Slenderness Lambda	25,95	75,06	
Relative slenderness Lambda,rel	0,28	0,80	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,98	0,72	
Buckling resistance Nb,Rd	4573,72	3373,00	kN

Flexural Buckling verification		
Cross-section area A	198,0000	cm^2
Buckling resistance Nb,Rd	3373,00	kN
Unity check	0,09	-

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	3950.0000	cm^3
Elastic critical moment Mcr	3820.43	kNm
Relative slenderness Lambda,LT	0.49	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	a	
Imperfection Alpha,LT	0.21	
Reduction factor Chi,LT	0.93	
Buckling resistance Mb.Rd	859.98	kNm
Unity check	0.74	-

Mcr Parameters		
LTB length	5.440	m
k	1.00	
kw	1.00	
C1	1.76	
C2	0.00	
C3	1.00	

Note: C Parameters according to ECCS 119 2006 / Galea 2002
load in center of gravity

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.036	
kyz	1.021	
kzy	0.542	
kzz	1.090	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	198.0000	cm^2
Wy	3950.0000	cm^3
Wz	1058.3300	cm^3
NRk	4653.00	kN
My,Rk	928.25	kNm
Mz,Rk	248.71	kNm
My,Ed	666.81	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	2167.14	kNm
reduced slenderness 0	0.65	
Psi y	0.000	
Psi z	1.000	
Cmy,0	0.998	
Cmz,0	1.011	
Cmy	1.000	
Cmz	1.011	
CmLT	1.031	
muy	1.000	
muz	0.988	
wy	1.113	
wz	1.500	
npl	0.068	
aLT	0.996	
bLT	0.000	
cLT	0.612	

Table of values		
dLT	0.000	
eLT	1.693	
Cyy	0.999	
Cyz	0.721	
Czy	0.975	
Czz	0.957	

Unity check (6.61) = 0.07 + 0.80 + 0.00 = 0.87
Unity check (6.62) = 0.09 + 0.42 + 0.00 = 0.51

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	37.000

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B3	11,700 m	I + I var	S 235	Skupaj	0,80 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Welded	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.

The critical check is on position 10.740 m

Internal forces	Calculated	Unit
N,Ed	-29,09	kN
Vy,Ed	0,00	kN
Vz,Ed	-243,45	kN
T,Ed	0,00	kNm
My,Ed	-624,19	kNm
Mz,Ed	0,00	kNm

Section properties

A	2.481155e+004 mm^2		
Ay/A	0.731	Az/A	0.243
Iy	9.809805e+008 mm^4	Iz	1.419644e+008 mm^4
Iyz	-4.035583e-007 mm^4	It	3.018046e+006 mm^4
Iw	6.333249e+012 mm^6		
Wely	3.340527e+006 mm^3	Welz	9.464291e+005 mm^3
Wply	4.109909e+006 mm^3	Wplz	1.443077e+006 mm^3
cy	200.25 mm	cz	150.00 mm
dy	-0.00 mm	dz	-13.60 mm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	248,1155	cm^2
Nc,Rd	5830,71	kN
Unity check	0,00	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Wel,y,min	3340,5268	cm^3
Mel,y,Rd	785,02	kNm
Unity check	0,80	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Tau,Vz,Ed	5,0	kN/cm^2
Tau,Rd	13,6	kN/cm^2
Unity check	0,37	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification		
Fibre	14	
Sigma,N,Ed	0,1	kN/cm^2
Sigma,My,Ed	-18,7	kN/cm^2
Sigma,Mz,Ed	0,0	kN/cm^2
Sigma,tot,Ed	-18,6	kN/cm^2
Tau,Vy,Ed	0,0	kN/cm^2
Tau,Vz,Ed	1,1	kN/cm^2
Tau,t,Ed	0,0	kN/cm^2
Tau,tot,Ed	1,1	kN/cm^2
Sigma,von Mises,Ed	18,7	kN/cm^2
Unity check	0,79	-

The member satisfies the section check.

...::STABILITY CHECK::...

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	11,700	2,340	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	11,700	2,340	m
Critical Euler load Ncr	14852,78	53736,17	kN
Slenderness Lambda	58,84	30,94	
Relative slenderness Lambda,rel	0,63	0,33	
Limit slenderness Lambda,rel,0	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Torsional (-Flexural) Buckling check
According to article EN 1993-1-1 : 6.3.1.1. and formula (6.46)

Table of values		
Torsional Buckling length	2.340	m
Ncr,T	58112.87	kN
Ncr,TF	51953.46	kN
Relative slenderness Lambda,T	0.34	
Limit slenderness Lambda,0	0.20	

The slenderness or compression force is such that Torsional (-Flexural) Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4)

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	3340.5268	cm^3
Elastic critical moment Mcr	17422.98	kNm
Relative slenderness Lambda,LT	0.21	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.340	m
k	1.00	
kw	1.00	
C1	1.46	
C2	0.01	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.002	
kyz	1.001	

Table of values		
kzy	1.002	
kzz	1.001	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	248.1155	cm^2
Wy	3340.5268	cm^3
Wz	946.4291	cm^3
NRk	5830.71	kN
My,Rk	785.02	kNm
Mz,Rk	222.41	kNm
My,Ed	-624.19	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	11912.94	kNm
reduced slenderness 0	0.26	
Psi y	-0.186	
Psi z	1.000	
Cmy,0	0.999	
Cmz,0	1.000	
Cmy	1.000	
Cmz	1.000	
CmLT	1.000	
muy	1.000	
muz	1.000	
wy	1.230	
wz	1.500	
npl	0.005	
aLT	0.997	
bLT	0.000	
cLT	0.085	
dLT	0.000	
eLT	2.516	
Cyy	1.001	
Cyz	0.961	
Czy	1.000	
Czz	0.996	

Unity check (6.61) = 0.00 + 0.80 + 0.00 = 0.80
Unity check (6.62) = 0.00 + 0.80 + 0.00 = 0.80

The member satisfies the stability check.

Member B5	**5,882 m**	**I + I var**	**S 235**	**Skupaj**	**0,47 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Welded	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.

The critical check is on position 1.280 m

Internal forces	Calculated	Unit
N,Ed	-92,89	kN
Vy,Ed	0,00	kN
Vz,Ed	18,15	kN
T,Ed	0,00	kNm
My,Ed	-55,96	kNm
Mz,Ed	0,00	kNm

Section properties

A	7.258121e+003 mm^2		
Ay/A	0.656	Az/A	0.341
Iy	1.273700e+008 mm^4	Iz	9.053786e+006 mm^4
Iyz	-1.036027e-007 mm^4	It	2.224037e+005 mm^4
Iw	1.904247e+011 mm^6		

Wely	6.506732e+005 mm^3	Welz	1.207171e+005 mm^3
Wply	8.373645e+005 mm^3	Wplz	1.864229e+005 mm^3
cy	138.89 mm	cz	75.00 mm
dy	-0.00 mm	dz	-13.72 mm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	72,5812	cm^2
Nc,Rd	1705,66	kN
Unity check	0,05	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Wel,y,min	650,6732	cm^3
Mel,y,Rd	152,91	kNm
Unity check	0,37	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Tau,Vz,Ed	0,9	kN/cm^2
Tau,Rd	13,6	kN/cm^2
Unity check	0,07	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification		
Fibre	26	
Sigma,N,Ed	1,3	kN/cm^2
Sigma,My,Ed	6,1	kN/cm^2
Sigma,Mz,Ed	0,0	kN/cm^2
Sigma,tot,Ed	7,4	kN/cm^2
Tau,Vy,Ed	0,0	kN/cm^2
Tau,Vz,Ed	0,1	kN/cm^2
Tau,t,Ed	0,0	kN/cm^2
Tau,tot,Ed	0,1	kN/cm^2
Sigma,von Mises,Ed	7,4	kN/cm^2
Unity check	0,31	-

The member satisfies the section check.

...::STABILITY CHECK::...

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,882	2,941	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,882	2,941	m
Critical Euler load Ncr	7630,93	2169,71	kN
Slenderness Lambda	44,40	83,27	
Relative slenderness Lambda,rel	0,47	0,89	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	b	c	
Imperfection Alpha	0,34	0,49	
Reduction factor Chi	0,90	0,61	
Buckling resistance Nb,Rd	1528,41	1037,15	kN

Flexural Buckling verification		
Cross-section area A	72,5812	cm^2
Buckling resistance Nb,Rd	1037,15	kN
Unity check	0,09	-

Torsional (-Flexural) Buckling check
According to article EN 1993-1-1 : 6.3.1.1. and formula (6.46)

Table of values		
Torsional Buckling length	2.941	m
Ncr,T	3349.89	kN
Ncr,TF	2132.66	kN
Relative slenderness Lambda,T	0.89	
Limit slenderness Lambda,0	0.20	

Table of values		
Buckling curve	c	
Imperfection Alpha	0.49	
A	72.5812	cm^2
Reduction factor Chi	0.60	
Buckling resistance Nb,Rd	1029.08	kN
Unity check	0.09	-

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	650.6732	cm^3
Elastic critical moment Mcr	523.16	kNm
Relative slenderness Lambda,LT	0.54	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.941	m
k	1.00	
kw	1.00	
C1	1.41	
C2	0.02	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.044	
kyz	1.054	
kzy	1.027	
kzz	1.037	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	72.5812	cm^2
Wy	650.6732	cm^3
Wz	120.7171	cm^3
NRk	1705.66	kN
My,Rk	152.91	kNm
Mz,Rk	28.37	kNm
My,Ed	-55.96	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	371.47	kNm
reduced slenderness 0	0.64	
Psi y	0.135	
Psi z	1.000	
Cmy,0	0.997	
Cmz,0	1.010	
Cmy	0.999	
Cmz	1.010	
CmLT	1.033	
muy	0.999	
muz	0.983	
wy	1.287	
wz	1.500	
npl	0.054	
aLT	0.998	
bLT	0.000	
cLT	0.208	
dLT	0.000	
eLT	0.432	
Cyy	0.999	
Cyz	0.910	
Czy	0.983	
Czz	0.993	

Unity check (6.61) = 0.06 + 0.38 + 0.00 = 0.44
Unity check (6.62) = 0.09 + 0.38 + 0.00 = 0.47

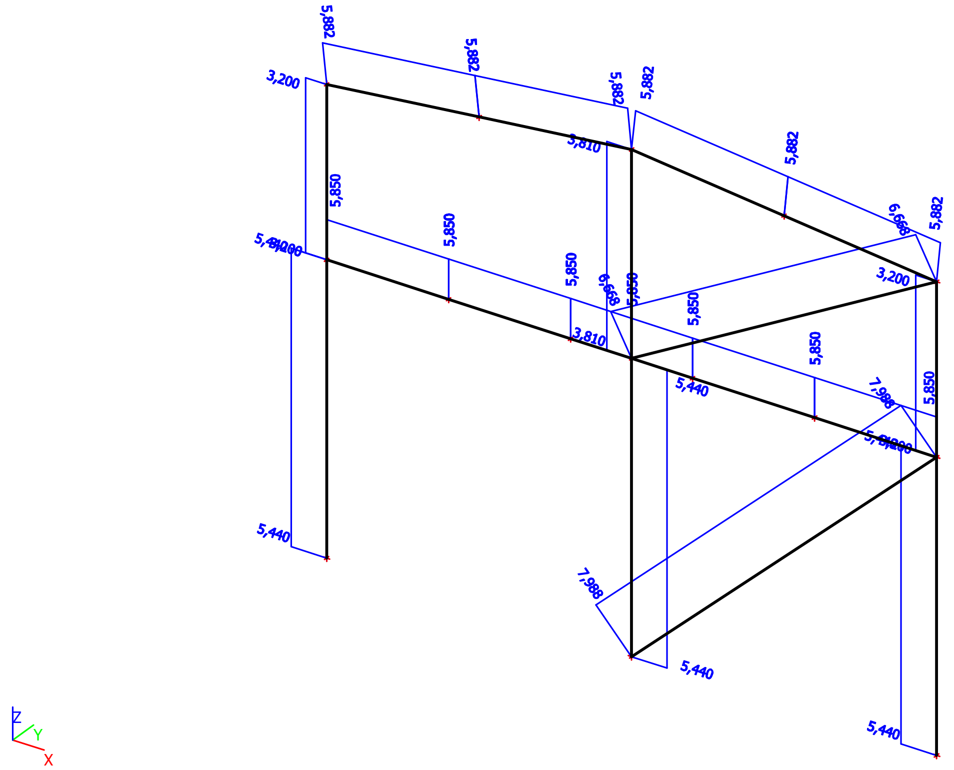
The member satisfies the stability check.

1. Priloga C.2

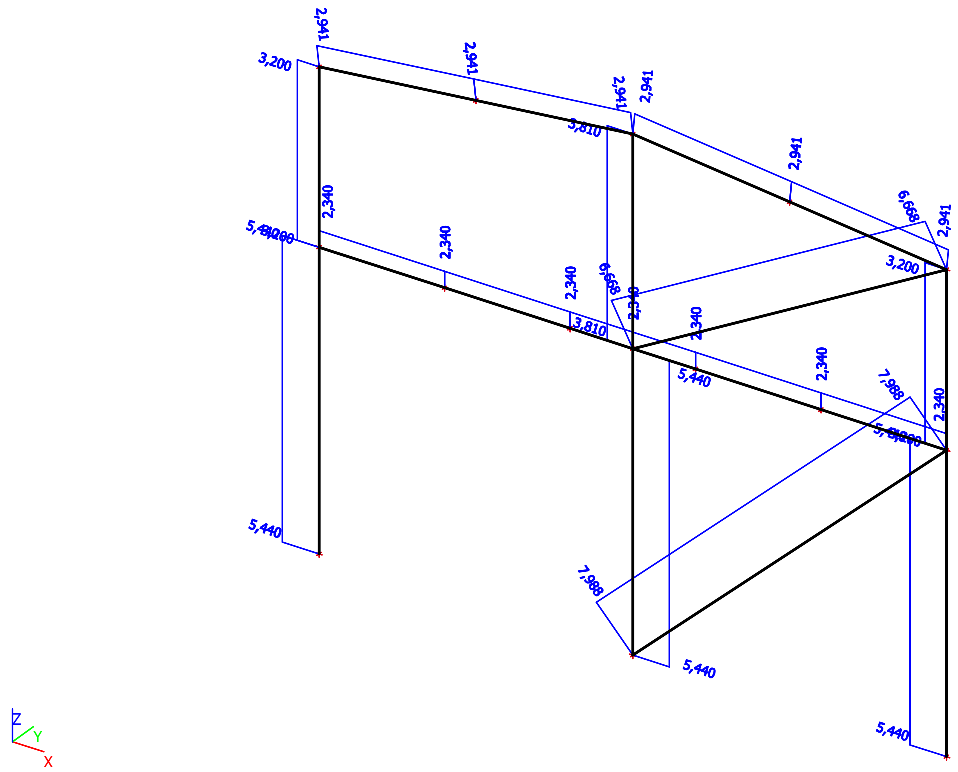
Linear calculation

Member	CS Name	Part	Sway y	Ly [m]	ky [-]	ly [m]	Lam y [-]	lyz [m]	l LTB [m]
			Sway z	Lz [m]	kz [-]	lz [m]	Lam z [-]		
B1	CS8	1	Yes	5,440	1,00	5,440	59,31	5,440	5,440
			No	5,440	1,00	5,440	98,53		
B1	CS8	2	Yes	3,200	1,00	3,200	34,89	3,200	3,200
			No	3,200	1,00	3,200	57,96		
B2	CS8	1	Yes	5,440	1,00	5,440	59,31	5,440	5,440
			No	5,440	1,00	5,440	98,53		
B2	CS8	2	Yes	3,200	1,00	3,200	34,89	3,200	3,200
			No	3,200	1,00	3,200	57,96		
B3	CS8	1	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS8	2	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS8	3	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS8	4	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS8	5	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B3	CS8	6	Yes	5,850	1,00	5,850	63,78	2,340	2,340
			No	2,340	1,00	2,340	42,38		
B4	CS11	1	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B4	CS11	2	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B5	CS11	1	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B5	CS11	2	Yes	5,882	1,00	5,882	79,23	2,941	2,941
			No	2,941	1,00	2,941	143,06		
B6	CS10	1	Yes	3,810	1,00	3,810	66,34	3,810	3,810
			No	3,810	1,00	3,810	230,26		
B7	CS9	1	Yes	5,440	1,00	5,440	82,92	5,440	5,440
			No	5,440	1,00	5,440	136,53		
B8	CS12	1	Yes	7,988	1,00	7,988	316,57	7,988	7,988
			No	7,988	1,00	7,988	639,29		
B9	CS13	1	Yes	6,668	1,00	6,668	1133,78	6,668	6,668
			No	6,668	1,00	6,668	1133,78		

2. Iy



3. Iz



4. Result classes

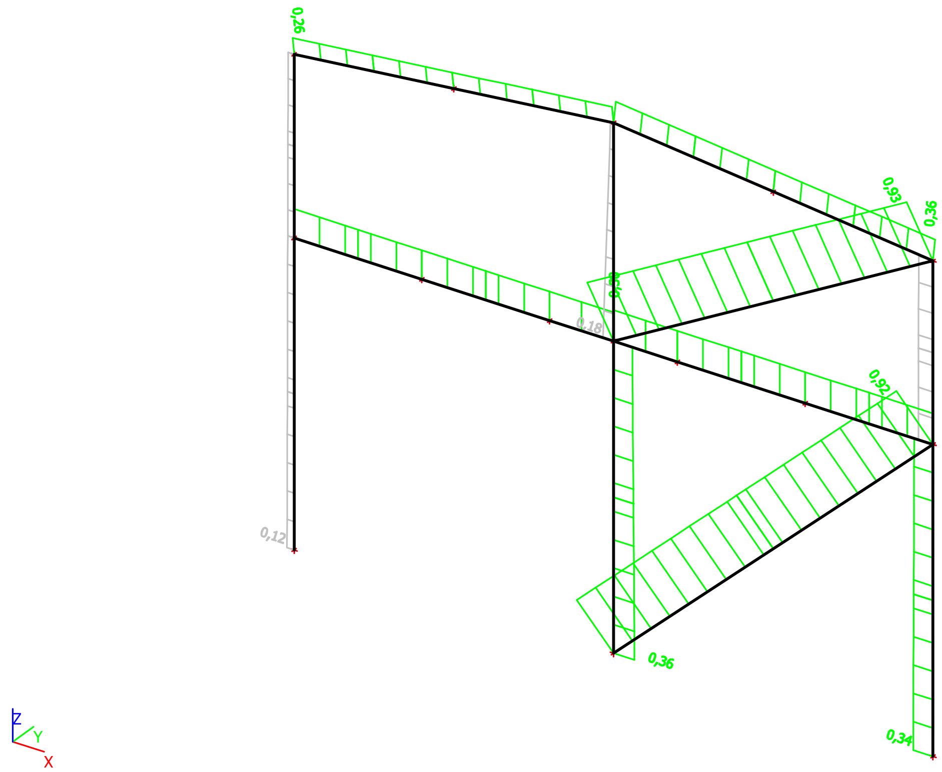
4.1. Result classes - RC1

Name	List
RC1	Skupaj

4.1.1. Check of steel

Nonlinear calculation, Extreme : Member
Selection : All
Class : RC1

Member	css	mat	Case	dx [m]	un.check [-]	sec.check [-]	stab.check [-]
B1	CS8 - HEA220	S 235	Skupaj	0,000	**0,12**	0,04	0,12
B2	CS8 - HEA220	S 235	Skupaj	0,000	0,34	0,14	0,34
B3	CS8 - HEA220	S 235	Skupaj	5,850	0,50	0,39	**0,50**
B4	CS11 - IPE180	S 235	Skupaj	0,000	0,26	0,06	0,26
B5	CS11 - IPE180	S 235	Skupaj	0,000	0,36	0,25	0,36
B6	CS10 - IPE140	S 235	Skupaj	0,000	0,18	**0,03**	0,18
B7	CS9 - HEA160	S 235	Skupaj	5,440	0,36	0,12	0,36
B8	CS12 - U65	S 235	Skupaj	7,988	0,92	0,92	**0,00**
B9	CS13 - L20X3	S 235	Skupaj	6,668	**0,93**	**0,93**	0,00



Nonlinear calculation, Extreme : Cross-section
Selection : All
Class : RC1

Member B3	**11,700 m**	**HEA220**	**S 235**	**Skupaj**	**0,50 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	21,71
Class 1 Limit	44,27
Class 2 Limit	50,97
Class 3 Limit	86,36

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	8,05
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

The critical check is on position 5.850 m

Internal forces	Calculated	Unit
N,Ed	-132,55	kN
Vy,Ed	0,00	kN
Vz,Ed	47,30	kN
T,Ed	0,00	kNm
My,Ed	-51,50	kNm
Mz,Ed	0,00	kNm

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	64,3000	cm^2
Nc,Rd	1511,05	kN
Unity check	0,09	-

Bending moment check for My

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	566,6670	cm^3
Mpl,y,Rd	133,17	kNm
Unity check	0,39	-

Shear check for Vz

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	20,6300	cm^2
Vpl,z,Rd	279,90	kN
Unity check	0,17	-

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Mpl,y,Rd	133,17	kNm
Unity check	0,39	-

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design

Decisive position for stability classification: 4,680 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	21,71
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	46,38

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	8,05
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,850	2,340	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,850	2,340	m
Critical Euler load Ncr	3276,46	7418,97	kN
Slenderness Lambda	63,78	42,38	
Relative slenderness Lambda,rel	0,68	0,45	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	b	c	
Imperfection Alpha	0,34	0,49	
Reduction factor Chi	0,80	0,87	
Buckling resistance Nb,Rd	1201,86	1314,36	kN

Flexural Buckling verification		
Cross-section area A	64,3000	cm^2
Buckling resistance Nb,Rd	1201,86	kN
Unity check	0,11	-

Torsional (-Flexural) Buckling check
According to article EN 1993-1-1 : 6.3.1.1. and formula (6.46)

Table of values		
Torsional Buckling length	2.340	m
Ncr,T	8390.37	kN
Ncr,TF	3276.46	kN
Relative slenderness Lambda,T	0.68	
Limit slenderness Lambda,0	0.20	
Buckling curve	c	
Imperfection Alpha	0.49	
A	64.3000	cm^2
Reduction factor Chi	0.74	
Buckling resistance Nb,Rd	1114.52	kN
Unity check	0.12	-

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	566.6670	cm^3
Elastic critical moment Mcr	1111.02	kNm
Relative slenderness Lambda,LT	0.35	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.340	m
k	1.00	
kw	1.00	
C1	1.32	
C2	0.50	
C3	0.41	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.020	
kyz	0.718	
kzy	0.538	
kzz	1.071	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	64.3000	cm^2
Wy	566.6670	cm^3
Wz	270.4170	cm^3
NRk	1511.05	kN
My,Rk	133.17	kNm
Mz,Rk	63.55	kNm
My,Ed	-51.50	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	844.70	kNm

Table of values		
reduced slenderness 0	0.40	
Psi y	0.830	
Psi z	1.000	
Cmy,0	0.970	
Cmz,0	1.004	
Cmy	0.991	
Cmz	1.004	
CmLT	1.000	
muy	0.991	
muz	0.998	
wy	1.100	
wz	1.500	
npl	0.088	
aLT	0.995	
bLT	0.000	
cLT	0.121	
dLT	0.000	
eLT	1.853	
Cyy	1.003	
Cyz	0.989	
Czy	0.983	
Czz	0.953	

Unity check (6.61) = 0.11 + 0.39 + 0.00 = 0.50
Unity check (6.62) = 0.12 + 0.21 + 0.00 = 0.33

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	26.857

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B5	5,882 m	IPE180	S 235	Skupaj	0,36 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	27,55
Class 1 Limit	58,70
Class 2 Limit	67,59
Class 3 Limit	87,12

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,23
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-34,87	kN
Vy,Ed	0,00	kN
Vz,Ed	5,92	kN
T,Ed	0,00	kNm
My,Ed	-9,60	kNm

Internal forces	Calculated	Unit
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	23,9000	cm^2
Nc,Rd	561,65	kN
Unity check	0,06	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	166,0000	cm^3
Mpl,y,Rd	39,01	kNm
Unity check	0,25	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	11,2040	cm^2
Vpl,z,Rd	152,01	kN
Unity check	0,04	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	39,01	kNm
Alpha	2,00	
Mpl,z,Rd	8,13	kNm
Beta	1,00	

Unity check (6.41) = 0,06 + 0,00 = 0,06 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	27,55
Class 1 Limit	58,70
Class 2 Limit	67,59
Class 3 Limit	87,12

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,23
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,882	2,941	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,882	2,941	m
Critical Euler load Ncr	789,04	242,04	kN
Slenderness Lambda	79,23	143,06	
Relative slenderness Lambda,rel	0,84	1,52	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	

Buckling parameters	yy	zz	
Reduction factor Chi	0,77	0,33	
Buckling resistance Nb,Rd	432,36	187,48	kN

Flexural Buckling verification		
Cross-section area A	23,9000	cm^2
Buckling resistance Nb,Rd	187,48	kN
Unity check	0,19	-

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	166.0000	cm^3
Elastic critical moment Mcr	88.93	kNm
Relative slenderness Lambda,LT	0.66	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	2.941	m
k	1.00	
kw	1.00	
C1	2.40	
C2	0.15	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.126	
kyz	1.038	
kzy	0.594	
kzz	1.177	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	23.9000	cm^2
Wy	166.0000	cm^3
Wz	34.6000	cm^3
NRk	561.65	kN
My,Rk	39.01	kNm
Mz,Rk	8.13	kNm
My,Ed	-9.60	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	36.98	kNm
reduced slenderness 0	1.03	
Psi y	0.486	
Psi z	1.000	
Cmy,0	0.962	
Cmz,0	1.035	
Cmy	0.988	
Cmz	1.035	
CmLT	1.071	
muy	0.989	
muz	0.899	
wy	1.137	
wz	1.500	
npl	0.062	
aLT	0.996	
bLT	0.000	
cLT	0.252	
dLT	0.000	
eLT	0.079	
Cyy	0.972	
Cyz	0.794	
Czy	0.875	
Czz	0.923	

Unity check (6.61) = 0.08 + 0.28 + 0.00 = 0.36
Unity check (6.62) = 0.19 + 0.15 + 0.00 = 0.33

Shear buckling check
in buckling field 1

According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	30.943

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B6	3,810 m	IPE140	S 235	Skupaj	0,18 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	23,87
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	3,93
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-9,96	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	16,4000	cm^2
Nc,Rd	385,40	kN
Unity check	0,03	-

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	23,87
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	3,93
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	3,810	3,810	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	3,810	3,810	m
Critical Euler load Ncr	772,44	64,11	kN
Slenderness Lambda	66,34	230,26	
Relative slenderness Lambda,rel	0,71	2,45	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,84	0,14	
Buckling resistance Nb,Rd	325,56	55,80	kN

Warning: Slenderness 230,26 is larger than the limit value of 200,00.

Flexural Buckling verification		
Cross-section area A	16,4000	cm^2
Buckling resistance Nb,Rd	55,80	kN
Unity check	0,18	-

The member satisfies the stability check.

Member B7	**5,440 m**	**HEA160**	**S 235**	**Skupaj**	**0,36 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	17,33
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6,89
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 5.440 m

Internal forces	Calculated	Unit
N,Ed	-107,80	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	38,8000	cm^2
Nc,Rd	911,80	kN
Unity check	0,12	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design

Decisive position for stability classification: 0,000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	17,33
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6,89
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,440	5,440	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,440	5,440	m
Critical Euler load Ncr	1169,60	431,42	kN
Slenderness Lambda	82,92	136,53	
Relative slenderness Lambda,rel	0,88	1,45	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	b	c	
Imperfection Alpha	0,34	0,49	
Reduction factor Chi	0,67	0,33	
Buckling resistance Nb,Rd	612,85	300,88	kN

Flexural Buckling verification		
Cross-section area A	38,8000	cm^2
Buckling resistance Nb,Rd	300,88	kN
Unity check	0,36	-

The member satisfies the stability check.

Member B8	7,988 m	U65	S 235	Skupaj	0,92 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

The critical check is on position 7.988 m

Internal forces	Calculated	Unit
N,Ed	195,58	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Tension check

According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	9,0300	cm^2
Npl,Rd	212,21	kN
Nu,Rd	234,06	kN
Nt,Rd	212,21	kN
Unity check	0,92	-

The member satisfies the section check.

...::STABILITY CHECK::...

The member satisfies the stability check.

Member B9	6,668 m	L20X3	S 235	Skupaj	0,93 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

The critical check is on position 6.668 m

Internal forces	Calculated	Unit
N,Ed	24,57	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	1,1200	cm^2
Npl,Rd	26,32	kN
Nu,Rd	29,03	kN
Nt,Rd	26,32	kN
Unity check	0,93	-

The member satisfies the section check.

...::STABILITY CHECK::...

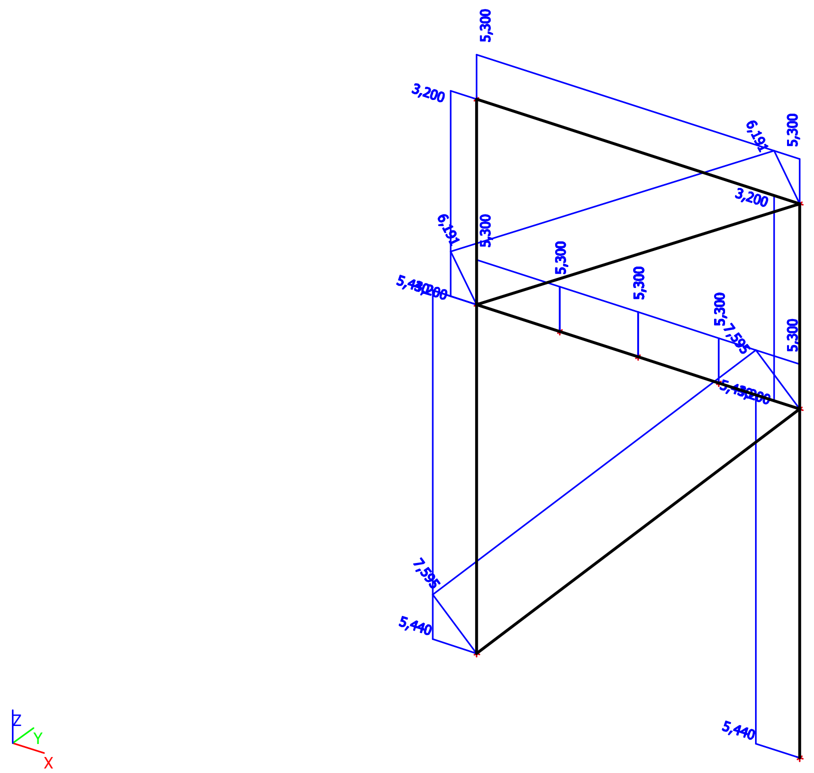
The member satisfies the stability check.

1. Priloga C.3

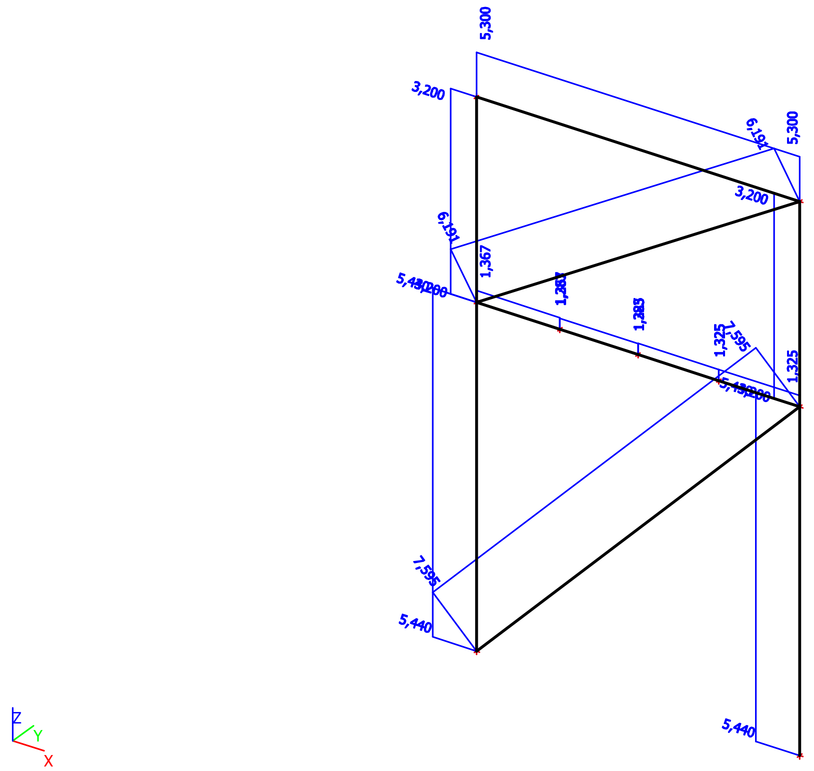
Second Order calculation

Member	CS Name	Part	Sway y	Ly [m]	ky [-]	ly [m]	Lam y [-]	lyz [m]	l LTB [m]
			Sway z	Lz [m]	kz [-]	lz [m]	Lam z [-]		
B1	CS11	1	Yes	5,440	1,00	5,440	25,95	5,440	5,440
			No	5,440	1,00	5,440	75,06		
B1	CS11	2	Yes	3,200	1,00	3,200	15,27	3,200	3,200
			No	3,200	1,00	3,200	44,15		
B2	CS11	1	Yes	5,440	1,00	5,440	25,95	5,440	5,440
			No	5,440	1,00	5,440	75,06		
B2	CS11	2	Yes	3,200	1,00	3,200	15,27	3,200	3,200
			No	3,200	1,00	3,200	44,15		
B5	CS8	1	Yes	5,300	1,00	5,300	53,12	1,367	1,367
			No	1,367	1,00	1,367	50,74		
B5	CS8	2	Yes	5,300	1,00	5,300	53,12	1,283	1,283
			No	1,283	1,00	1,283	47,59		
B5	CS8	3	Yes	5,300	1,00	5,300	53,12	1,325	1,325
			No	1,325	1,00	1,325	49,16		
B5	CS8	4	Yes	5,300	1,00	5,300	53,12	1,325	1,325
			No	1,325	1,00	1,325	49,16		
B6	CS12	1	Yes	5,300	1,00	5,300	153,44	5,300	5,300
			No	5,300	1,00	5,300	153,44		
B7	CS13	1	Yes	6,191	1,00	6,191	397,11	6,191	6,191
			No	6,191	1,00	6,191	527,22		
B8	CS9	1	Yes	7,595	1,00	7,595	122,34	7,595	7,595
			No	7,595	1,00	7,595	402,86		

2. ly



3. Iz



4. Result classes

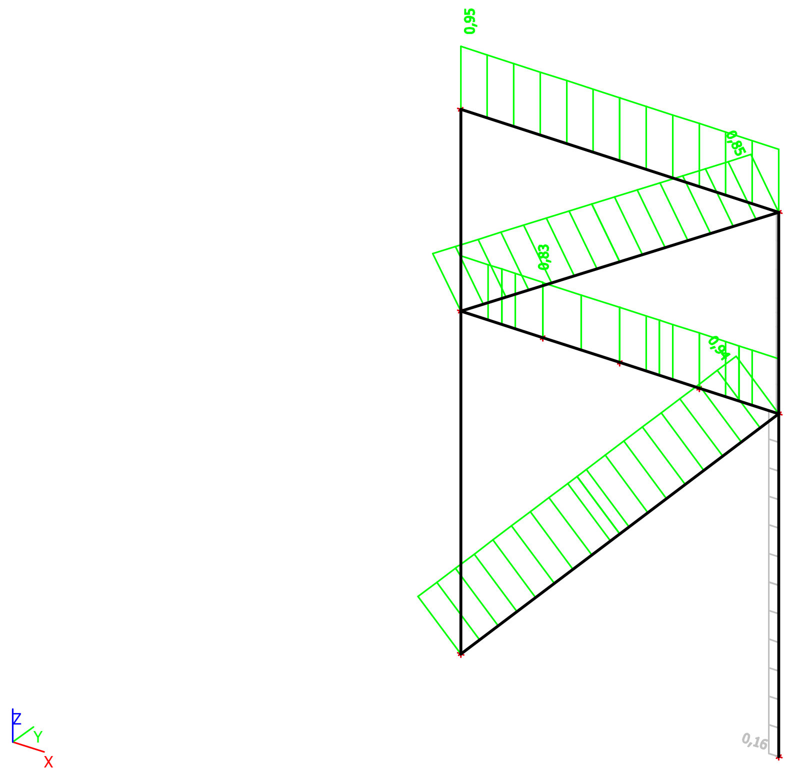
4.1. Result classes - RC1

Name	List
RC1	Ovojnica

4.1.1. Check of steel

Nonlinear calculation, Extreme : Member
Selection : All
Class : RC1

Member	css	mat	Case	dx [m]	un.check [-]	sec.check [-]	stab.check [-]
B1	CS11 - HEA500	S 235	Ovojnica	5,440	**0,00**	**0,00**	**0,00**
B2	CS11 - HEA500	S 235	Ovojnica	0,000	0,16	0,10	0,16
B5	CS8 - IPE240	S 235	Ovojnica	1,367	0,83	0,40	0,83
B6	CS12 - CFCHS101.6X4	S 235	Ovojnica	0,000	**0,95**	0,26	**0,95**
B7	CS13 - L50X40X5	S 235	Ovojnica	6,191	0,85	0,85	0,00
B8	CS9 - U160	S 235	Ovojnica	7,595	0,94	**0,94**	0,00



Nonlinear calculation, Extreme : Cross-section
Selection : All
Class : RC1

Member B2	**8,640 m**	**HEA500**	**S 235**	**Ovojnica**	**0,16 -**

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	32,50
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5,09
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-456,55	kN
Vy,Ed	-1,20	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	198,0000	cm^2
Nc,Rd	4653,00	kN
Unity check	0,10	-

Shear check for Vy
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	142,6800	cm^2
Vpl,y,Rd	1935,84	kN
Unity check	0,00	-

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	32,50
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5,09
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,440	5,440	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,440	5,440	m
Critical Euler load Ncr	60931,31	7283,74	kN
Slenderness Lambda	25,95	75,06	
Relative slenderness Lambda,rel	0,28	0,80	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,98	0,72	
Buckling resistance Nb,Rd	4573,72	3373,00	kN

Flexural Buckling verification		
Cross-section area A	198,0000	cm^2
Buckling resistance Nb,Rd	3373,00	kN
Unity check	0,14	-

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.062	
kyz	0.698	
kzy	0.559	
kzz	1.000	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	198.0000	cm^2
Wy	3950.0000	cm^3
Wz	1058.3300	cm^3
NRk	4653.00	kN
My,Rk	928.25	kNm
Mz,Rk	248.71	kNm
My,Ed	0.00	kNm
Mz,Ed	-5.90	kNm

Table of values		
Interaction Method 1		
Mcr0	2167.14	kNm
reduced slenderness 0	0.65	
Psi y	1.000	
Psi z	0.000	
Cmy,0	1.002	
Cmz,0	0.979	
Cmy	1.002	
Cmz	0.979	
CmLT	1.051	
muy	1.000	
muz	0.982	
wy	1.113	
wz	1.500	
npl	0.098	
aLT	0.996	
bLT	0.000	
cLT	0.000	
dLT	0.000	
eLT	0.000	
Cyy	0.999	
Cyz	1.043	
Czy	0.964	
Czz	1.026	

Unity check (6.61) = 0.10 + 0.00 + 0.02 = 0.12
Unity check (6.62) = 0.14 + 0.00 + 0.02 = 0.16

The member satisfies the stability check.

Member B5	5,300 m	IPE240	S 235	Ovojnica	0,83 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	30,71
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	54,87

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,28
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	13,77

=> Outstand Flanges Class 1
=> Section classified as Class 1 for cross-section design

The critical check is on position 1.367 m

Internal forces	Calculated	Unit
N,Ed	-367,15	kN
Vy,Ed	0,00	kN
Vz,Ed	10,27	kN
T,Ed	0,00	kNm
My,Ed	21,16	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	39,1000	cm^2
Nc,Rd	918,85	kN
Unity check	0,40	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	367,0000	cm^3
Mpl,y,Rd	86,25	kNm
Unity check	0,25	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	19,1276	cm^2
Vpl,z,Rd	259,52	kN
Unity check	0,04	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

MN,y,Rd	64,67	kNm
Unity check	0,33	-

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	30,71
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 1
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4,28
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

=> Outstand Flanges Class 1
=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,300	1,367	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,300	1,367	m
Critical Euler load Ncr	2871,71	3148,08	kN
Slenderness Lambda	53,12	50,74	
Relative slenderness Lambda,rel	0,57	0,54	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,90	0,87	
Buckling resistance Nb,Rd	829,31	795,67	kN

Flexural Buckling verification		
Cross-section area A	39,1000	cm^2
Buckling resistance Nb,Rd	795,67	kN
Unity check	0,46	-

Torsional (-Flexural) Buckling check
According to article EN 1993-1-1 : 6.3.1.1. and formula (6.46)

Table of values		
Torsional Buckling length	1.367	m
Ncr,T	4856.96	kN
Ncr,TF	2871.71	kN
Relative slenderness Lambda,T	0.57	
Limit slenderness Lambda,0	0.20	
Buckling curve	b	
Imperfection Alpha	0.34	

Table of values		
A	39.1000	cm^2
Reduction factor Chi	0.85	
Buckling resistance Nb,Rd	784.63	kN
Unity check	0.47	-

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	367.0000	cm^3
Elastic critical moment Mcr	645.07	kNm
Relative slenderness Lambda,LT	0.37	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	1.367	m
k	1.00	
kw	1.00	
C1	1.60	
C2	0.05	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check
According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)
Interaction Method 1

Table of values		
kyy	1.207	
kyz	0.656	
kzy	0.666	
kzz	1.232	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	39.1000	cm^2
Wy	367.0000	cm^3
Wz	73.9000	cm^3
NRk	918.85	kN
My,Rk	86.25	kNm
Mz,Rk	17.37	kNm
My,Ed	27.81	kNm
Mz,Ed	0.00	kNm
Interaction Method 1		
Mcr0	404.12	kNm
reduced slenderness 0	0.46	
Psi y	1.000	
Psi z	1.000	
Cmy,0	1.004	
Cmz,0	1.028	
Cmy	1.002	
Cmz	1.028	
CmLT	1.107	
muy	0.986	
muz	0.983	
wy	1.133	
wz	1.500	
npl	0.400	
aLT	0.997	
bLT	0.000	
cLT	0.135	
dLT	0.000	
eLT	1.360	
Cyy	1.039	
Cyz	1.208	
Czy	0.978	
Czz	0.928	

Unity check (6.61) = 0.44 + 0.39 + 0.00 = 0.83
Unity check (6.62) = 0.47 + 0.21 + 0.00 = 0.68

Shear buckling check
in buckling field 1
According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	35.548

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Member B6	**5,300 m**	**CFCHS101.6X4**	**S 235**	**Ovojnica**	**0,95 -**

Note: EN 1993-1-3 article 1.1(3) specifies that this part does not apply to cold formed CHS and RHS sections. The default EN 1993-1-1 code check is executed instead of the EN 1993-1-3 code check.

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Cold formed	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification for Tubular Sections
According to EN 1993-1-1 Table 5.2 Sheet 3

Maximum width-to-thickness ratio	25,40
Class 1 Limit	50,00
Class 2 Limit	70,00
Class 3 Limit	90,00

=> Section classified as Class 1 for cross-section design

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-75,06	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	12,2600	cm^2
Nc,Rd	288,11	kN
Unity check	0,26	-

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification for Tubular Sections
According to EN 1993-1-1 Table 5.2 Sheet 3

Maximum width-to-thickness ratio	25,40
Class 1 Limit	50,00
Class 2 Limit	70,00
Class 3 Limit	90,00

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check
According to article EN 1993-1-1 : 6.3.1.1. and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System Length L	5.300	5.300	m
Buckling factor k	1.00	1.00	
Buckling length Lcr	5.300	5.300	m
Critical Euler load Ncr	107.93	107.93	kN
Slenderness	153.44	153.44	
Relative slenderness Lambda	1.63	1.63	
Limit slenderness Lambda,0	0.20	0.20	
Buckling curve	c	c	
Imperfection Alpha	0.49	0.49	
Reduction factor Chi	0.27	0.27	
Buckling resistance Nb,Rd	79.19	79.19	kN

Table of values		
A	12.2600	cm^2
Buckling resistance Nb,Rd	79.19	kN
Unity check	0.95	-

The member satisfies the stability check.

Member B7	6,191 m	L50X40X5	S 235	Ovojnica	0,85 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

The critical check is on position 6.191 m

Internal forces	Calculated	Unit
N,Ed	85,71	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	4,2700	cm^2
Npl,Rd	100,35	kN
Nu,Rd	110,68	kN
Nt,Rd	100,35	kN
Unity check	0,85	-

The member satisfies the section check.

...::STABILITY CHECK::...

The member satisfies the stability check.

Member B8	7,595 m	U160	S 235	Ovojnica	0,94 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25

Material		
Yield strength fy	23,5	kN/cm^2
Ultimate strength fu	36,0	kN/cm^2
Fabrication	Rolled	

...::SECTION CHECK::...

The critical check is on position 7.595 m

Internal forces	Calculated	Unit
N,Ed	531,47	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	24,0000	cm^2
Npl,Rd	564,00	kN
Nu,Rd	622,08	kN
Nt,Rd	564,00	kN
Unity check	0,94	-

The member satisfies the section check.

...::STABILITY CHECK::...

The member satisfies the stability check.

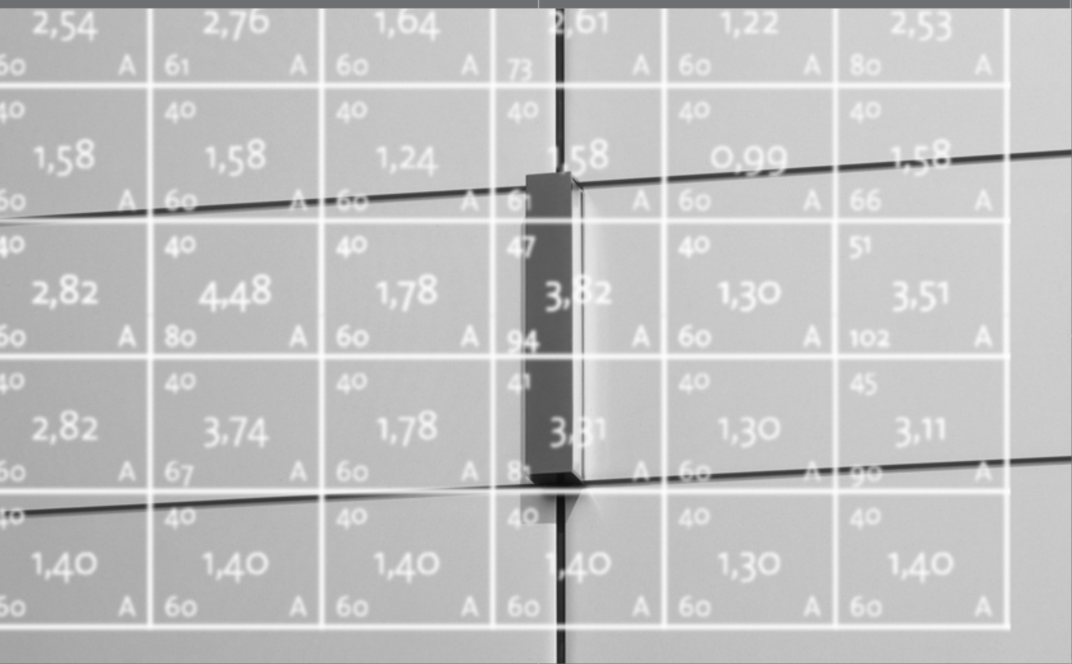
PRILOGA D: FASADNI IN STREŠNI PANELI TRIMO

Priloga D.1: Fasadni paneli Trimo - dopustne razdalje med podporami

Priloga D.2: Strešni paneli Trimo - dopustne razdalje med podporami

Trimo

Celovite rešitve



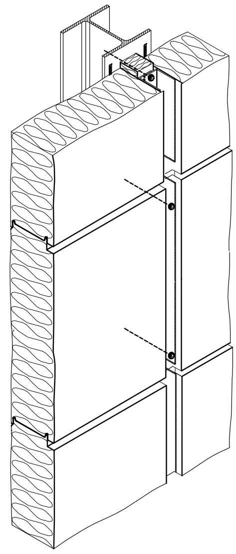
Dopustne razdalje med podporami

Modularni fasadni paneli **TrimoRaster**

TrimoRaster

TrimoRaster FTV R 80, TrimoRaster FTV R 100, TrimoRaster FTV R 120,
TrimoRaster FTV R 150, TrimoRaster FTV R 200 dopustne tabele

Dopustne tabele so skladne z nemškim gradbeno nadzornim dovoljenjem "ALGEMEINE BAUAUFSICHTLICHE ZULASSUNG Z - 10.4-240 + Gutachten Nr.Z - 510".



Maksimalne dopustne sile za dvojno rastersko pritrjevanje v odvisnosti od debeline panela

Debelina panela (mm)	80	100	120	150	200
Fmax[kN]	4,11	4,55	4,99	5,65	6,76

PREDPOSTAVKE IN NAPOTKI ZA UPORABO DOPUSTNIH TABEL

- Pozitivni pritisk, negativni pritisk in sile na vijake je potrebno določiti za posamezne države skladno z nacionalnimi standardi za vplive vsled vetra.
- V tabelah dopustnih razdalj so že vgrajeni varnostni faktorji za obremenitve skladno z nemško zulasungo Z-10.4-240, to pomeni da so obremenitve v tabelah karakterističnega značaja.
- Pri vseh obremenitvenih primerih ki nastopajo je potrebno upoštevati minimalno razdaljo med podporami.
- Dodatno so v tabeli navedene potrebne širine naleganja za konkretne razdalje med podporami v milimetrih [mm]. Pri tem označuje število desno nad razdaljo med podporami pripadajočo potrebno širino naleganja na končnih podporah.
- Poves je omejen na maksimalno l/100 za posamezne obtežne primere: pritiska vetra, srka vetra in temperaturne razlike ter neugodno kombinacijo obtežnih primerov vetra s temperaturo poleti - obtežba vetra je v tem primeru omejena na 60 %.
- Dopustne razdalje med podporami vezano na nosilnost pritrdišč so veljavne samo za primer ko sta sosednja panela enako dolga.
- Te dopustne tabele so uporabne za najbolj enostavne vgradne primere TrimoRastra, za vse ostale pa je potrebno izdelati individualne izračune dopustnih razdalj.
- Te dopustne tabele veljajo za vse tri barve skupine zunanje pločevine.
- Te dopustne tabele veljajo za vse tipe notranejga mikroporfila pločevine (s, v, g)
- Za vsak individualen primer vgradnje panela TrimoRaster je potrebno izdelati izračun potrebnega števila vijakov.

Fasadni panel FTV R 80 GG 0,7/0,6								
Pozitivni, negativni pritisk [kN/m²] (+/-)	0,25	0,50	0,75	1,00	1,25	1,50	1,75	2,00
Dopustna razdalja [m] -pozitivni pritisk (+)	6,50 (40)	6,50 (40)	5,36 (45)	4,64 (52)	4,15 (58)	3,79 (63)	3,51 (68)	3,28 (73)
Dopustna razdalja [m] -negativni pritisk (srk) (-)	6,50	6,05	4,94	4,28	3,83	3,49	3,23	3,02
Dopustna razdalja [m] Nosilnost rasterskega pritrjevanja	6,50	6,05	4,94	4,11	3,29	2,74	2,35	2,06

Fasadni panel FTV R 100 GG 0,7/0,6

Pozitivni, negativni pritisk [kN/m²] (+/-)	0,25	0,50	0,75	1,00	1,25	1,50	1,75	2,00
Dopustna razdalja [m] -pozitivni pritisk (+)	6,50 [40]	6,50 [40]	6,00 [50]	5,20 [58]	4,65 [65]	4,24 [71]	3,93 [76]	3,68 [82]
Dopustna razdalja [m] -negativni pritisk (srk) (-)	6,50	6,50	5,53	4,79	4,28	3,91	3,62	3,38
Dopustna razdalja [m] Nosilnost rasterskega pritrjevanja	6,50	6,50	5,53	4,55	3,64	3,03	2,60	2,28

Fasadni panel FTV R 120 GG 0,7/0,6

Pozitivni, negativni pritisk [kN/m²] (+/-)	0,25	0,50	0,75	1,00	1,25	1,50	1,75	2,00
Dopustna razdalja [m] -pozitivni pritisk (+)	6,50 [40]	6,50 [40]	6,50 [54]	5,70 [63]	5,09 [71]	4,65 [77]	4,30 [84]	4,03 [89]
Dopustna razdalja [m] -negativni pritisk (srk) (-)	6,50	6,50	6,06	5,25	4,69	4,28	3,97	3,71
Dopustna razdalja [m] Nosilnost rasterskega pritrjevanja	6,50	6,50	6,06	4,99	3,99	3,33	2,85	2,50

Fasadni panel FTV R 150 GG 0,7/0,6

Pozitivni, negativni pritisk [kN/m²] (+/-)	0,25	0,50	0,75	1,00	1,25	1,50	1,75	2,00
Dopustna razdalja [m] -pozitivni pritisk (+)	6,50 [40]	6,50 [40]	6,50 [54]	6,37 [71]	5,70 [79]	5,20 [87]	4,82 [94]	4,50 [100]
Dopustna razdalja [m] -negativni pritisk (srk) (-)	6,50	6,50	6,50	5,87	5,25	4,79	4,44	4,15
Dopustna razdalja [m] Nosilnost rasterskega pritrjevanja	6,50	6,50	6,50	5,65	4,52	3,77	3,23	2,83

Fasadni panel FTV R 200 GG 0,7/0,6

Pozitivni, negativni pritisk [kN/m²] (+/-)	0,25	0,50	0,75	1,00	1,25	1,50	1,75	2,00
Dopustna razdalja [m] -pozitivni pritisk (+)	6,50 [40]	6,50 [40]	6,50 [54]	6,50 [72]	6,50 [90]	6,01 [100]	5,56 [108]	5,20 [115]
Dopustna razdalja [m] -negativni pritisk (srk) (-)	6,50	6,50	6,50	6,50	6,06	5,54	5,12	4,79
Dopustna razdalja [m] Nosilnost rasterskega pritrjevanja	6,50	6,50	6,50	6,50	5,41	4,51	3,86	3,38

Založil: TRIMO d.d., Oblikovanje: Nataša Erjavec, Fotografija: Janez Erjavec, Tisk: Tiskarna ČukGraf, Naklada: 2000/SLO, 03/2008

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Prijateljeva cesta 12, 8210 Trebnje, Slovenija
t: +386 7 34 60 200, f: +386 7 34 60 127
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Celovite rešitve

Ognjevarni STREŠNI paneli Trimoterm SNV

Tehnični list

Ognjevarni STREŠNI paneli Trimoterm SNV

Vrhunske tehnične lastnosti ognjevarnih strešnih panelov Trimoterm SNV so rezultat skrbne izbire materialov ter sodobnega proizvodnega procesa.

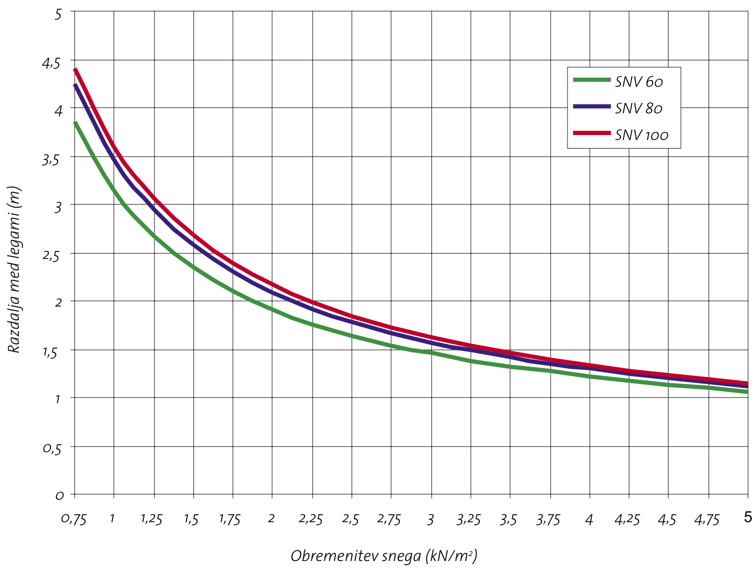
Tehnične karakteristike

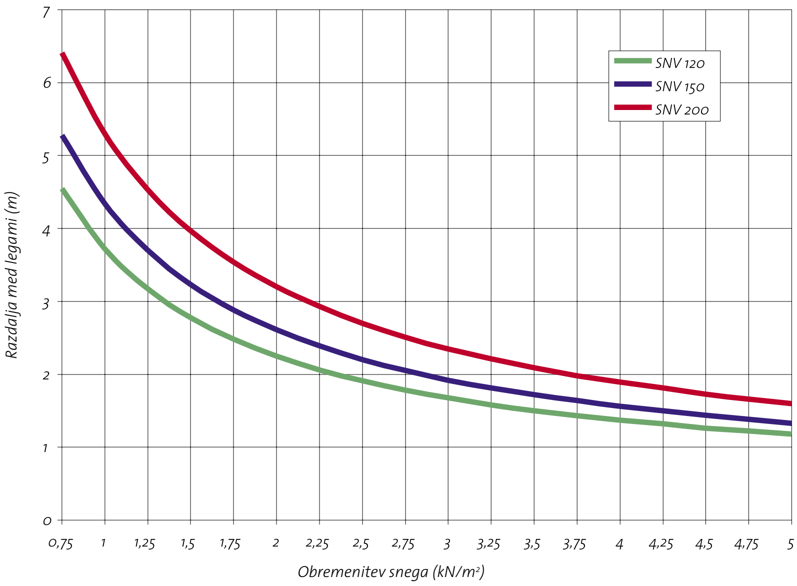
Tehnični podatki SNV		SNV 60	SNV 80	SNV 100	SNV 120	SNV 150	SNV 200
Debelina panela [mm]		60	80	100	120	150	200
Masa SNV [kg/m²]	Fe 0,6 / Fe 0,6	18,9	21,3	23,7	26,1	29,7	35,7
U toplotna prehodnost [W/m²K] (EN ISO 10211-2)		0,60	0,47	0,38	0,32	0,26	0,20
Razred ognjeodpornosti (EN 1365-2, EN 13501-2)*			REI 60	REI 90	REI 120	REI 150 →	
Gorljivost (EN 13501-1)		negorljivo polnilo iz mineralne volne, razred A1					
Rw zvočna izolativnost [dB] (EN ISO 140-3)			30 →				
Minimalni naklon		5° ali 3° z dodatnim tesnenjem					
Širina panelov [mm]		1000					
Dolžina panelov [m]		do 14					

* *Potrebna kontrola razdalj med podporami glede na statični sistem in obremenitve.*

Dopustne obremenitve

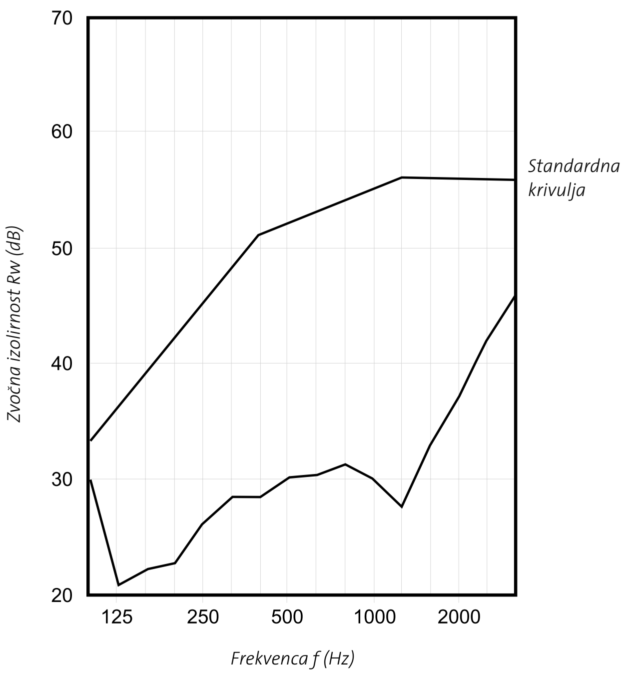
Diagram dopustnih razdalj Trimoterm STANDARD upošteva najneugodnejše obtežne primere zaradi obremenitve snega v skladu s splošnim prodajnim dovoljenjem Z-10.4-240. Krivulje nosilnosti so izračunane s programskim paketom Sand Stat 4® za standardne tipe profilov za večpoljni statični sistem vgradnje in zaprte objekte z normalnimi notranjimi temperaturami. Za vsak posamezni primer vgradnje je potrebno dokazati še pritrjevanje in potrebne širine podpor.





Zvočna izolativnost

Zvočna izolativnost je bila izmerjena na strešnem panelu Trimoterm SNV 100 št. poročila P 1489/97-510-1, ZAG Ljubljana.



Certifikati

APPROVED

Cert. No. 650a to LPS 1208

Sistemi barvnih zaščit

Jeklena pločevina je predhodno vroče cinkana z nanosom 275 g (Zn)/m² (EN 10142, EN 10147). Nanos barve je po “coil coating” postopku - barvanje med valji. Barva je sušena v peči pri temperaturi min. 200°C.

Osnovne lastnosti		SP	PVDF
Razred protikorozijske zaščite glede na EN ISO 12944-2		C3	C3
Oznaka protikorozijskega sistema glede na DIN 55928/8		3-160.2	3-600.1
Klasifikacija materiala glede na DIN 4102		A2	A2
Temperaturna obstojnost (°C)		do +80	do +110
Debelina nanosa (my)		25	25
Zunanja atmosfera	normalna	•	• • •
	mestna in industrijska	•	• • •
	ostra industrijska	-	• •
Morska atmosfera	1 do 10 km od obale	-	• •
	< 1 km od obale	-	•
Pogoji znotraj objektov	t ≤ 25°C , φ ≤ 80 %	• • •	• • •
	t ≤ 25°C , φ > 80 %	• •	• •
	t ≤ 50°C , φ > 80 %	-	• •
	brez ogrevanja	•	• •

••• *Primeren brez zadržkov*
•• *Zelo primeren*
• *Primeren*
- *Neprimeren*

Uporabljata se dva tipa protikorozijske zaščite pločevine:
- zaščita na osnovi polyestra z oznako SP standardna zaščita,
- zaščita na osnovi polyvinylidenfluorida z oznako PVDF na željo kupca.

Pridržujemo si pravico do tehničnih sprememb. Zadnja verzija dokumenta se nahaja na www.trimo.si.

Založil: TRIMO d.d., Oblikovanje: Meta Gabrijel, Fotografija: Janez Erjavec, Miran Kambič, Bojan Radovič, Tisk: Tiskarna Petrič, Naklada: 1500/SLO, 01/2006

Trimo

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Prijateljeva cesta 12, 8210 Trebnje, Slovenija
t: +386 7 34 60 200, f: +386 7 30 44 569
trimo@trimo.si, www.trimo.si

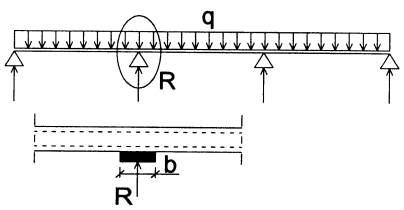
PRILOGA E: ANALIZA TRIMO HI-BOND SOVPREŽNIH STROPOV

Tabela 1: Dopustni razponi HI-Bond pločevine v vlogi opaža (v centimetrih) - mejna stanja nosilnosti in uporabnosti

pločevina: HI-Bond 55, t = 0,8 mm, f_{yk} = 250 MPa

Mejna stanja uporabnosti (upogibki)

h (cm)	L_{DOV} (cm)
12	278
14	261
16	248
18	237



Mejna stanja nosilnosti

h (cm)	upogibna nosilnost	Vnos koncentriranih sil nad vmesnimi podporami s širino b				
		b = 10 mm	b = 50 mm	b = 100 mm	b = 200 mm	b = 300 mm
12	261	212	239	250	259	261
14	243	193	219	230	239	242
16	228	177	203	213	223	227
18	216	164	189	200	209	213

Tabela 2: Dopustni razponi HI-Bond sovprežnih stropov (v centimetrih) - mejna stanja uporabnosti

pločevina: HI-Bond 55, t = 0.8 mm, f_{yk} = 250 MPa
beton: C 20/25, C 25/30, C 30/37, C 35/45

h (cm)	g_l (kN/m²)	vrsta betona	$p_{Sd\,MSU}$ (kN/m²) 2	3	4	5	6	7	8	9	10	12	14	16
12	2,41	C 20/25	504	470	445	424	406	391	378	367	357	339	322	308
		C 25/30	507	474	448	427	409	394	381	370	359	342	325	311
		C 30/37	511	477	451	430	412	397	384	372	362	344	327	313
		C 35/45	514	481	454	433	415	400	386	375	364	347	329	315
14	2,91	C 20/25	562	528	502	480	461	445	431	418	407	388	372	357
		C 25/30	566	532	505	483	464	448	434	421	410	391	375	359
		C 30/37	570	536	509	486	467	451	437	424	413	394	378	362
		C 35/45	574	540	512	490	471	454	440	427	416	396	380	364
16	3,41	C 20/25	618	584	556	533	514	497	482	469	457	436	419	404
		C 25/30	622	588	560	537	517	500	485	472	460	439	422	407
		C 30/37	627	592	564	541	521	504	489	475	463	442	424	409
		C 35/45	631	596	568	545	525	507	492	478	466	445	427	412
18	3,91	C 20/25	671	637	609	585	565	547	531	517	505	483	464	448
		C 25/30	676	642	613	590	569	551	535	521	508	486	467	451
		C 30/37	681	646	618	594	573	555	539	525	512	489	470	454
		C 35/45	686	651	622	598	577	559	543	528	515	493	474	457

Tabela 8: Dopustni razponi HI-Bond sovprežnih stropov (v centimetrih) - mejna stanja nosilnosti

pločevina: HI-Bond 55, t = 0,8 mm, f_{yk} = 250 MPa
beton: C 25/30

			$p_{Sd\ MSN}$ (kN/m^2)												
h(cm)	g_l (kN/m^2)	metoda	3	4	5	6	8	10	12	14	16	18	20	22	24
12	2,41	B1	395	368	345	326	296	274	256	241	228	217	208	200	193
		B2	464	431	405	383	347	321	299	282	267	254	243	234	225
		B3	471	438	411	388	353	325	304	286	271	258	247	237	229
		B4	522	485	455	430	391	360	336	317	300	286	273	262	253
		B5	588	547	513	485	440	406	379	356	337	321	308	295	284
		B6	626	582	545	515	468	431	402	379	359	342	327	314	302
14	2,91	B1	401	376	354	336	308	285	267	253	240	229	220	211	204
		B2	482	451	425	404	369	342	320	302	287	274	263	253	244
		B3	497	465	438	416	380	353	330	312	296	283	271	260	251
		B4	557	521	492	467	426	395	370	349	331	316	303	291	281
		B5	613	573	541	513	469	434	407	384	364	348	333	320	309
		B6	659	617	581	552	504	467	437	412	391	373	358	344	332
16	3,41	B1	406	382	362	345	318	296	278	263	251	240	230	222	215
		B2	496	467	443	422	388	361	339	321	305	292	280	270	261
		B3	517	487	461	439	404	376	353	334	318	304	292	281	271
		B4	585	550	521	496	456	424	399	377	359	343	329	317	306
		B5	633	595	564	537	493	459	431	408	388	371	356	342	331
		B6	686	645	611	582	534	497	467	441	420	401	385	371	358
18	3,91	B1	411	388	370	354	327	306	288	273	261	250	240	232	224
		B2	509	481	458	437	404	378	356	337	322	308	296	286	276
		B3	534	505	480	459	424	396	373	354	337	323	311	299	289
		B4	607	574	546	522	482	450	424	402	383	367	352	339	328
		B5	649	614	584	558	515	481	453	429	409	392	376	363	350
		B6	708	669	636	608	561	524	493	467	445	426	409	395	381

PRILOGA F: POZICIJSKI NAČRTI

List 1: Kosovnica

Risba 1: 3D pogled (A3)

Risba 2: Tloris nadstropja (A3)

Risba 3: Tloris strehe (A3)

Risba 4: Prerez 1-1, 2-2 (A3)

Risba 5: Prerez B-B (A3)

KOSOVNICA

Poz.	Opis	Profil	Material	Število elem. [kos]	Dolžina [mm]	Masa [kg/m]	Masa skupaj [kg]
1	steber	HEA 500	S235 JR	10	8635	155,00	13384,25
2	steber	HEA 240	S235 JR	4	8610	60,30	2076,73
3	steber	HEA 160	S235 JR	2	5180	30,40	314,94
4	steber	IPE 140	S235 JR	2	3610	12,90	93,14
5	nosilec	HEA 450	S235 J0	5	11160	140,00	7812,00
6	nosilec	HEA 240	S235 J0	2	11470	60,30	1383,28
7	nosilec	IPE 300	S235 JR	10	5610	42,20	2367,42
8	nosilec	IPE 200	S235 JR	4	5720	22,40	512,51
9	sek. nosilec	IPE 220	S235 JR	24	5220	26,20	3282,34
10	sek. nosilec	IPE 240	S235 JR	12	5220	30,70	1923,05
11	lega	IPE 200	S235 JR	12	6300	22,40	1693,44
12	lega	IPE 180	S235 JR	6	19300	18,80	2177,04
13	diagonala	U 160	S235 J0	8	7250	18,80	1090,40
14	diagonala	L 50x40x5	S235 J0	8	6075	3,35	162,81
15	diagonala	U 65	S235 J0	4	7750	7,09	219,79
16	diagonala	L 20x3	S235 J0	4	6790	0,88	23,90
17	diagonala	Ø12	S235 J0	16	5985	0,88	84,27
18	cev	101,6x4	S235 JR	12	5220	9,63	603,22
19	cev	60,3x3,2	S235 JR	8	5220	4,51	188,34
						Masa profilov skupaj [kg]	39392,87
						Vezna pločevina (15%)	5908,93
						Skupaj masa [kg]	45301,80

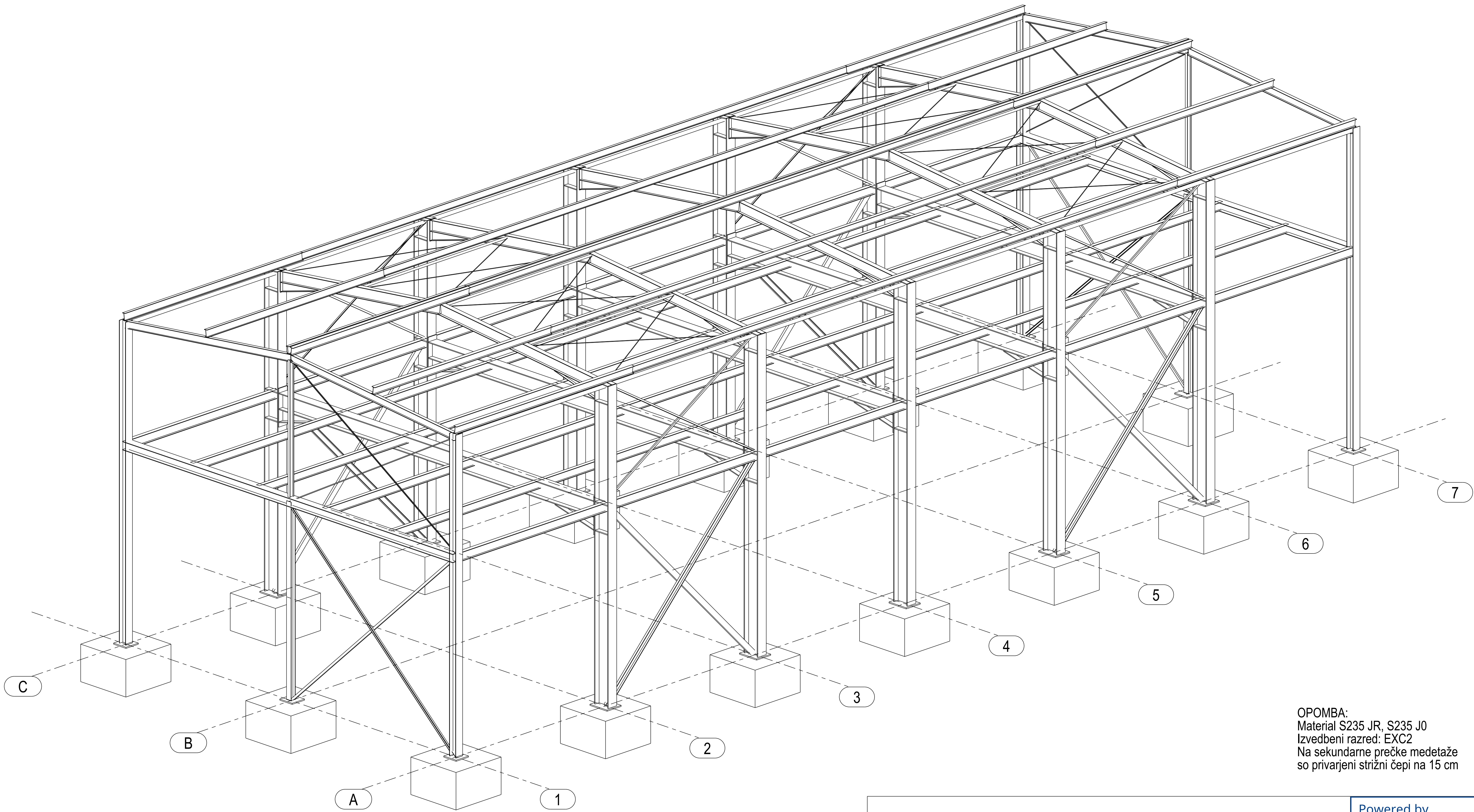
Površina tlorisa objekta

Pritličje: 372 m^2
Nadstropje: 372 m^2
Skupna površina: 744 m^2

Masa jeklene konstrukcije na površino

$$m = \frac{45301{,}80\ \mathrm{kg}}{744\ \mathrm{m}^2} = 60{,}89\ \mathrm{kg/m}^2$$

3d



OPOMBA:
Material S235 JR, S235 J0
Izvedbeni razred: EXC2
Na sekundarne prečke medetaže
so privarjeni strižni čepi na 15 cm

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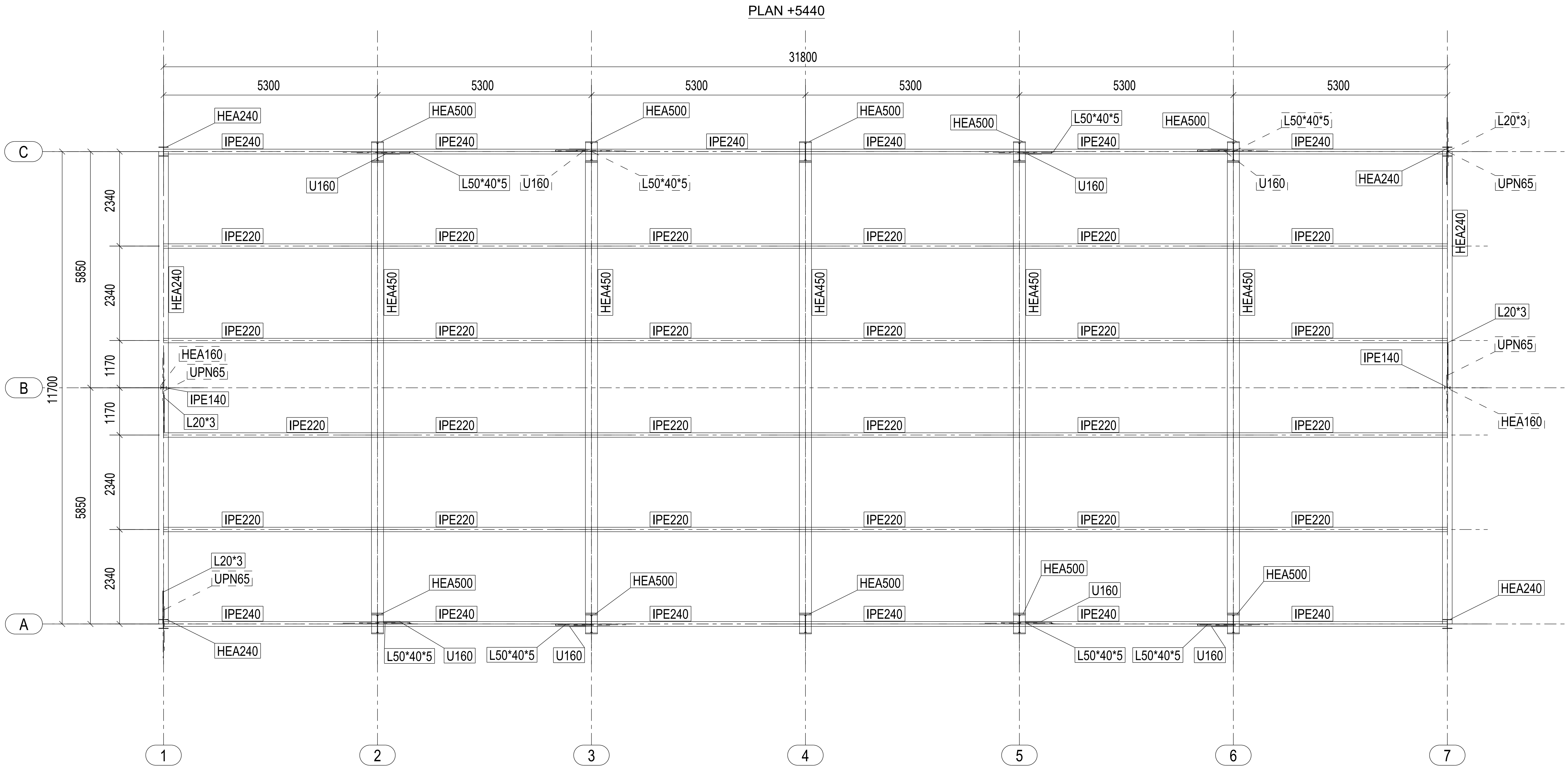
RISBA	3D pogled		
PROJEKT	Projekt dvo etažnega poslovno proizvodnega objekta		
VRSTA PROJ.	PGD	DATUM IZDELAVE	Avgust 2016
IZDELAL	Gregor Nučič	MERILO	1:100
ŠT. RISBE	1	REVIZIJA ŠT.	

OPOMBA:
Material S235 JR, S235 J0
Izvedbeni razred: EXC2
Na sekundarne prečke medetaže
so privarjeni strižni čepi na 15 cm



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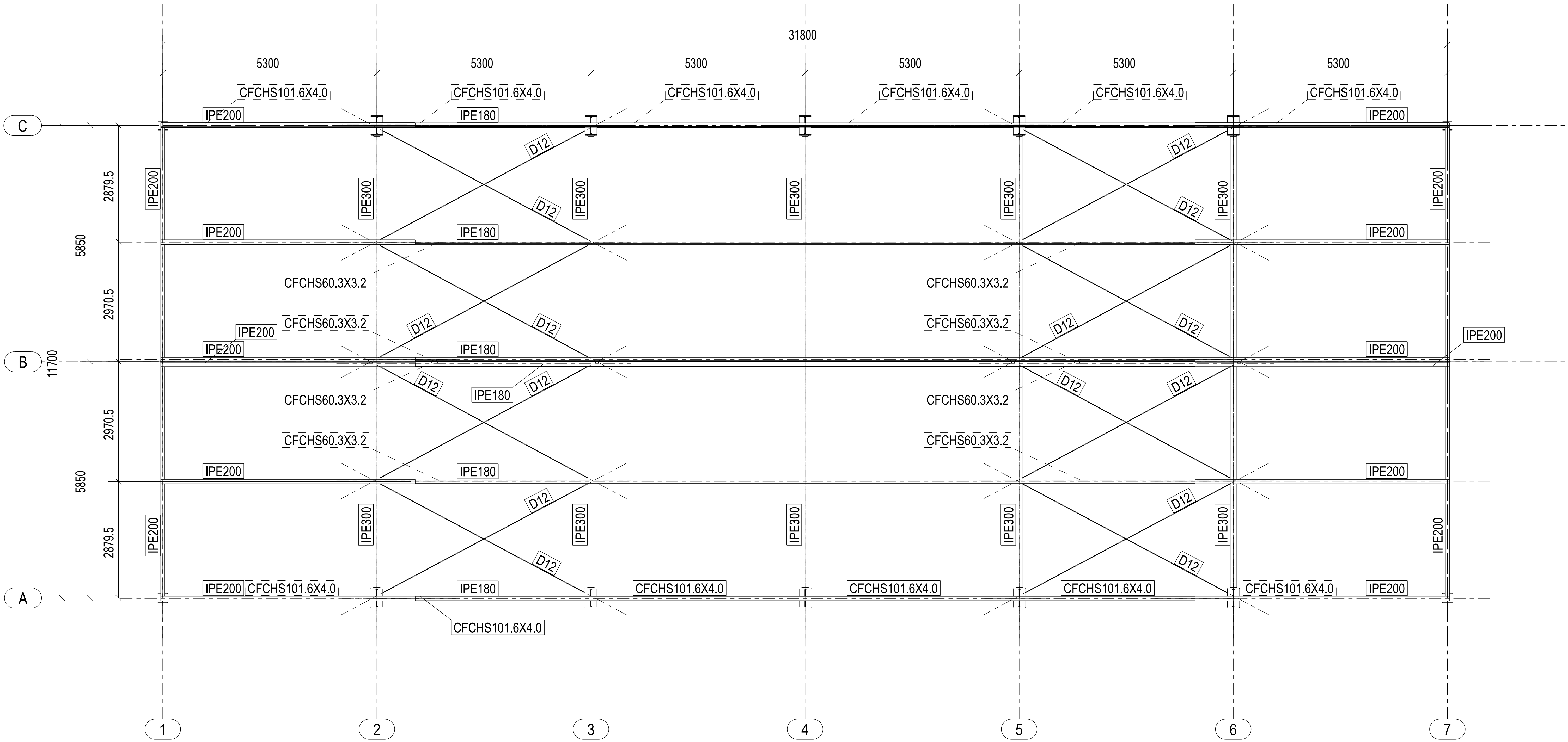
RISBA	Tloris nadstropja		
PROJEKT	Projekt dvo etažnega poslovno proizvodnega objekta		
VRSTA PROJ.	PGD	DATUM IZDELAVE	Avgust 2016
IZDELAL	Gregor Nučič	MERILO	1:100
ŠT. RISBE	2	REVIZIJA ŠT.	

OPOMBA:
Material S235 JR, S235 J0
Izvedbeni razred: EXC2
Na sekundarne prečke medetaže
so privarjeni strižni čepi na 15 cm

PLAN +8640



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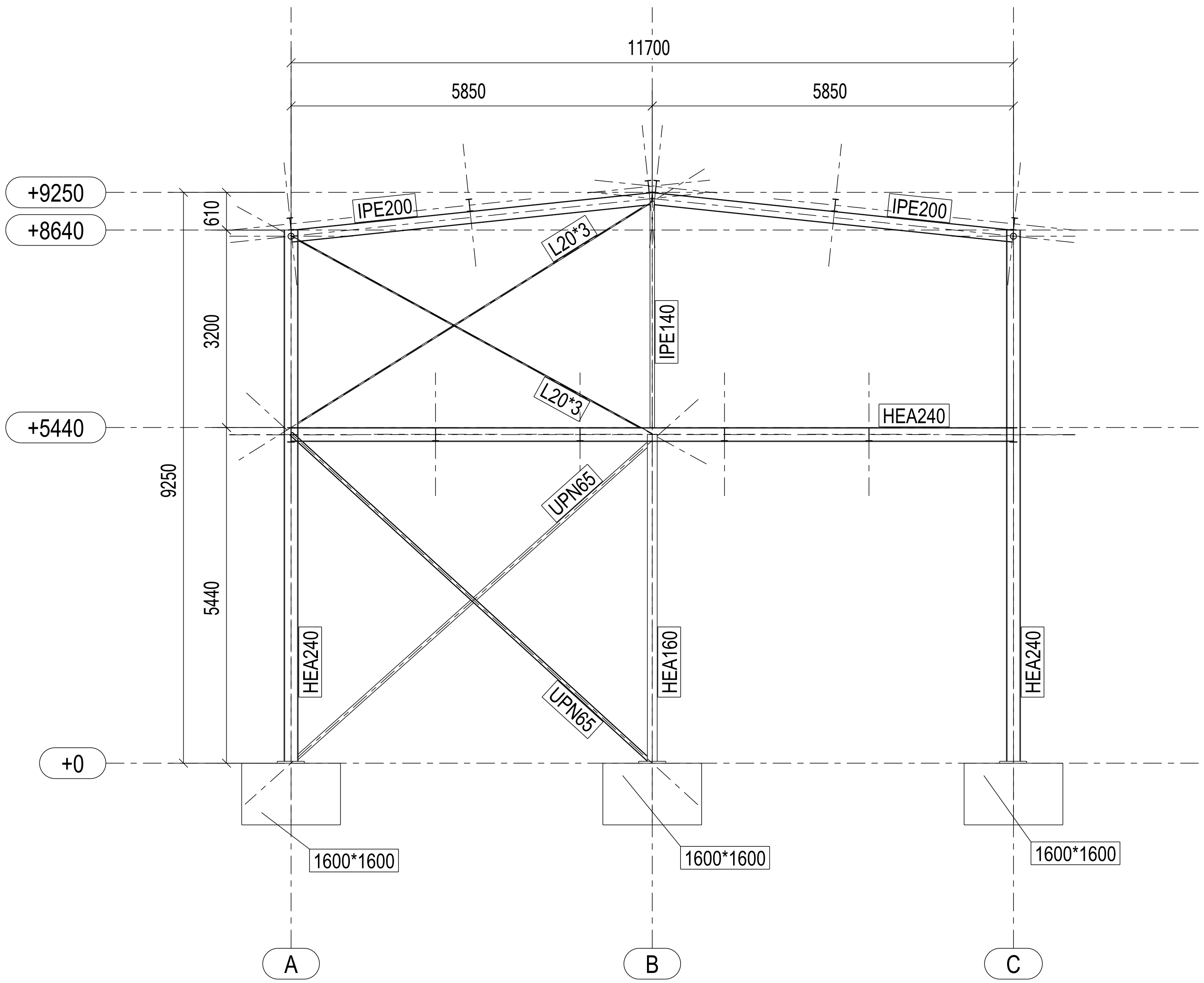
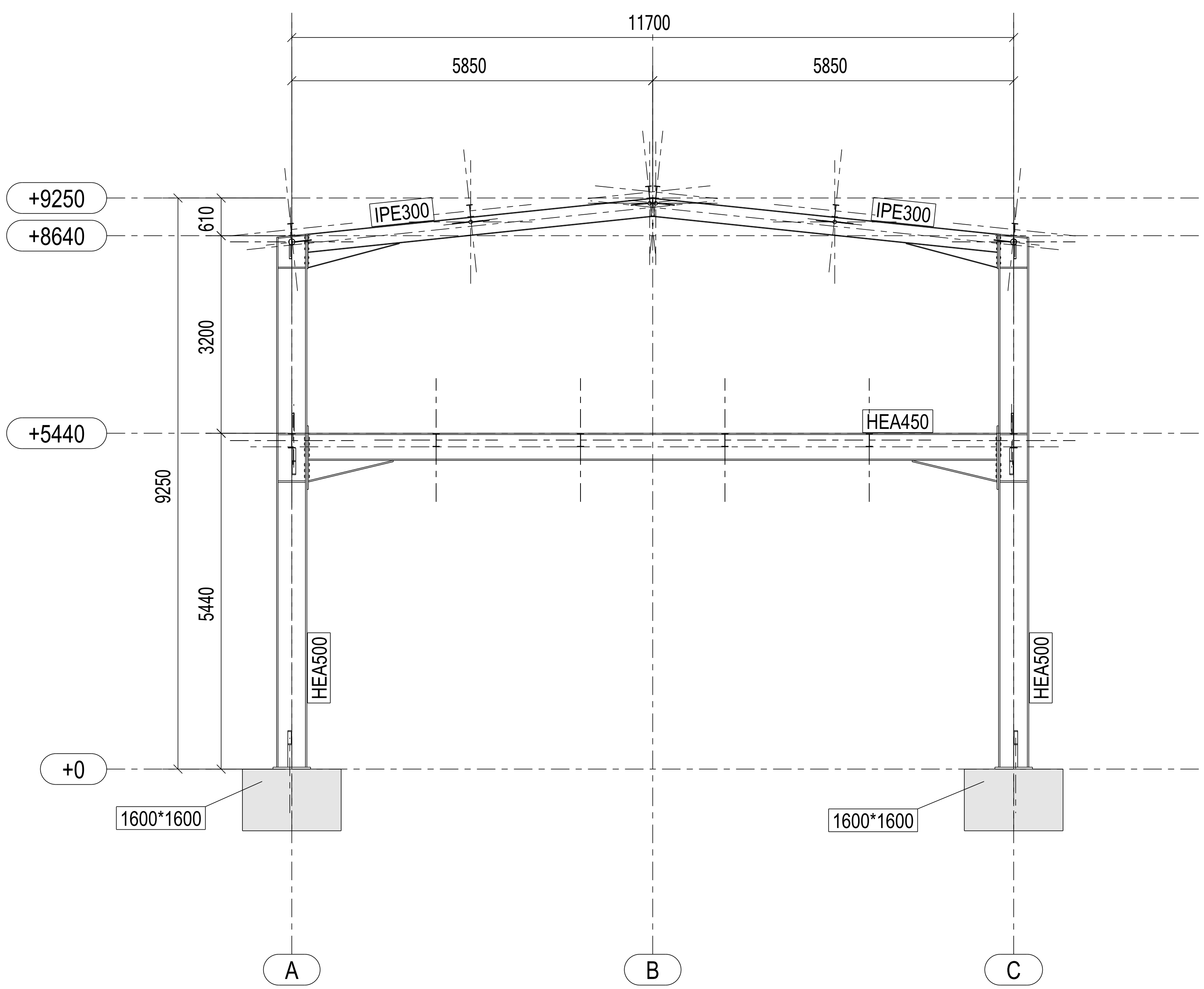
RISBA	Tloris strehe		
PROJEKT	Projekt dvo etažnega poslovno proizvodnega objekta		
VRSTA PROJ.	PGD	DATUM IZDELAVE	Avgust 2016
IZDELAL	Gregor Nučič	MERILO	1:100
ŠT. RISBE	3	REVIZIJA ŠT.	

GRID 1



GRID 2



OPOMBA:
Material S235 JR, S235 J0
Izvedbeni razred: EXC2
Na sekundarne prečke medetaže
so privarjeni strižni čepi na 15 cm

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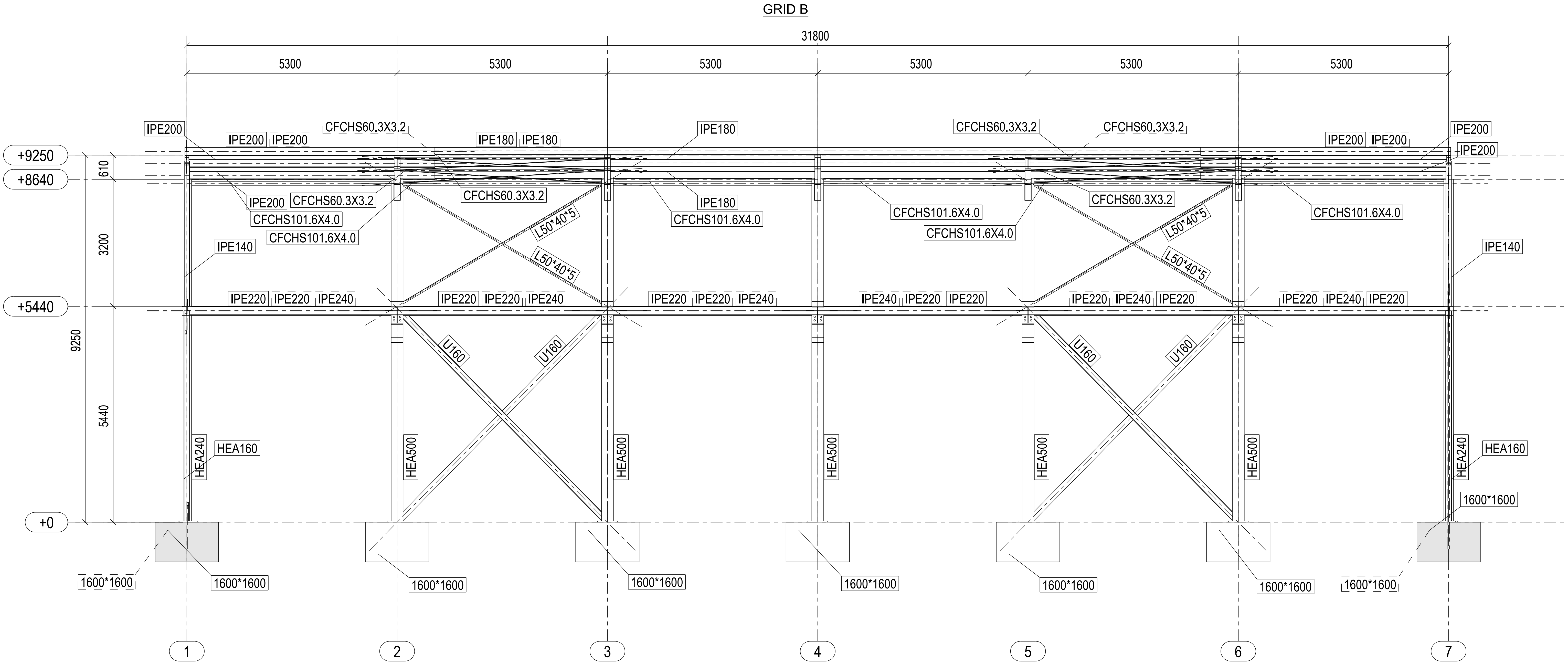
RISBA	Prerez 1-1, 2-2		
PROJEKT	Projekt dvo etažnega poslovno proizvodnega objekta		
VRSTA PROJ.	PGD	DATUM IZDELAVE	Avgust 2016
IZDELAL	Gregor Nučič	MERILO	1:100
ŠT. RISBE	4	REVIZIJA ŠT.	



OPOMBA:
Material S235 JR, S235 J0
Izvedbeni razred: EXC2
Na sekundarne prečke medetaže
so privarjeni strižni čepi na 15 cm

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RISBA	Prerez B-B		
PROJEKT	Projekt dvo etažnega poslovno proizvodnega objekta		
VRSTA PROJ.	PGD	DATUM IZDELAVE	Avgust 2016
IZDELAL	Gregor Nučič	MERILO	1:100
ŠT. RISBE	5	REVIZIJA ŠT.	